THICKNESS OF SURFACE AND BASE COURSES FOR FLEXIBLE PAVEMENTS

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SYNOPSIS

A method is described for determining the thickness of flexible pavements comprising an asphalt surface and one or more soil or aggregate bases on a given subgrade. The data required for evaluating a design are.

1. The subgrade bearing strength under the conditions of saturation and density expected to prevail
2. The bearing strength of soils or aggregate for use in the base or sub-base
3. The shear strength of the asphalt surface
4. The wheel loads and tire pressure to be carried

The method is based on fundamentals so that each problem can be evaluated by engineering methods of analysis A practical aspect is stressed throughout the development. Scientific precision is sacrificed in favor of conservative simplification wherever it is believed the foot rule of the construction job is more practical than the micrometer gauge of the scientist Compensating factors are pointed out where systematic error may result from idealized assumption An example is worked out for 60000-lb wheel loading on a 15-psi subgrade, indicating 4½ in. asphaltic concrete plus 6 in local aggregate base (bearing strength 27 psi.) on 8 in. compacted subgrade can be used A discussion of field and laboratory bearing tests, shear tests, etc stresses their practical application.

A method is proposed for determining the thickness of flexible pavements comprising an asphalt surface and one or more soil or aggregate base courses. It is a scientific method, having been developed on the basis of acceptable theories and selected fundamentals. In a larger sense, however, it is a practical method because each design problem can be evaluated by engineering analysis instead of recourse to purely empirical formulae. I have drawn freely on the published information of engineer-scientists and technologists who have appeared before The Highway Research Board and other societies. To them I extend my sincere thanks and if their names do not appear in this paper, it is because of feeling that their work is too well known to escape notice. It should be made clear that my part in developing the proposed method has been merely to assemble from any and all available sources the fundamentals necessary for a practical analysis of the engineer's problems in the design of asphalt pavements.

There are three main factors in the design problem:

1. The Applied Loads
2. The Distribution of Stress
3 The Subgrade Bearing Capacity.

Analysis of these three factors has been influenced more by the effect on a practical answer to the design problem than by its virtue as a scientifically correct refinement. Observations of how pavements are really built and the variety of conditions surrounding every job show that a flexible pavement design is better evaluated on the basis of a foot rule rather than a micrometer scale. Where conditions affecting the design have been idealized to simplify mathematical study, it is believed the discrepancies with respect to a more rigorous analysis will err on the conservative side; and, that by and large, the influences of such rationalization in various parts of the design method do tend to compensate for each other.

THE APPLIED LOADS

The stress distribution of the load on a tire imprint at the surface of a pavement has been frequently discussed It is known that due to sidewall stiffness and other factors, the stress is generally not uniform on the contact surface Over and under-inflation for given loads also have their influence. In terms of the finished design, however, such influences are likely to be of minor importance. A loading condition represented by the load–inflation quotient method of determining the area for an equivalent circle of contact is believed to be suitable for practical purposes because there are several compensating factors for whatever discrepancies may enter the design from its use. One important factor is the shape of the tire imprint The tire imprint will always have a larger perimeter than the equivalent circle for any given area of contact. Therefore, any assumption of stress concentration at the edge of the tire imprint will be compensated for by the higher shear stress necessary on the perimeter to achieve a balance of stresses when the circle is used; that is, we will have a higher unit stress on the boundary of the equivalent circle than would be calculated for the longer perimeter of the tire imprint. The same situation is relatively true for handling the problem of dual tire loadings. The equivalent circle has a shorter perimeter than the two separate tire imprints, or the boundary of the overlapping ellipses or other proposed shapes of the loaded area for this tire system Such compensating influences become more practical when thought of in terms of the asphalt surface over a flexible base and subgrade Shear is indicated as a dominant factor in the additional support contributed by the asphalt surface to a flexible pavement. If the use of an idealized, uniform average stress over an equivalent circle imposes the assumption of greater unit stress at the perimeter, the design will be influenced to require a thicker or stronger asphalt surface than would be required by the true shape, but this extra thickness due to the idealized shape is most probably necessary to compensate for the non-uniform stress distribution observed on the actual tire contact surface.

For the purpose of our design method, therefore, we have adopted the load-inflation quotient as a criterion and assume an average uniform stress distribution over an equivalent circle. Thus, for a 60,000-lb. airport wheel-loading and 75 psi inflation pressure, the design would be based upon the following

$$A = \frac{W}{P}, \text{ the load-inflation quotient.}$$

$$A = \frac{60000}{75} = 800 \text{ sq. in.}$$

where

A = Area of equivalent circle, sq in.

W = Wheel load, = 60000 lb.

p = Tire inflation pressure, = 75 psi.

An equivalent circle of this area has a diameter b = 31.90 in. and the perimeter area ratio is $\frac{P}{A} = \frac{4}{b} = 0.1253$.

DISTRIBUTION OF STRESS

The fundamental conception of the proposed design is based upon the demonstrated behavior of granular and cohesive soils when loaded through a finite bearing area Figures 1 and 2 are almost classic illustrations of what happens in granular material[1] and plastic clay.[2] These illustrations are similar in two respects

1 The action lines along which flow is indicated are alike.
2 Most significant, there is a finite limit to the depth of failure or zone where the soil particles are moved by the effect of the applied load

The demonstrations illustrated in Figures 1 and 2 show that the direction of flow seems to be caused by a similar system of forces.

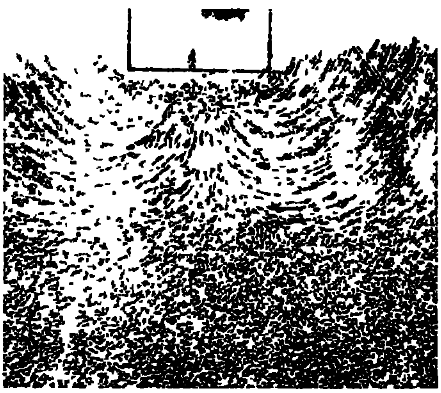

Figure 1. Movement of Granular Material under a Bearing Area

This system is generally explained by the conception of pressure bulbs that develop in the soil mass stressed under a bearing plate. For all practical purposes the pressure bulbs are assumed to be spherical as in the case of an elastic body of indefinite extent. Figure 3 is a photoelastic study approaching the ideal. The pressure bulbs are indicated by the alternate light and dark bands It is assumed in a soil mass, that an infinite number of pressure bulbs can be developed, each with its center on the vertical axis beneath the bearing plate and its surface intersecting the edge of the bearing area.

[1] "Internal Stability of Granular Material," W S Housel, *Proceedings* ASTM 1936.

[2] A Penetration Method for Measuring Soil Resistance," W. S Housel, *Proceedings* ASTM 1935.

The forces in a soil are interpreted as acting normal to the surface of the pressure bulbs and are larger on the small pressure bulbs than on the large pressure bulbs. This follows from the geometry of the system because the sum of applied stress as originating at the surface is spread over a larger surface on each larger pressure bulb. Since movement of a particle will be in the direction of least pressure, it is possible to trace a locus of potential action lines in Figure 3 by drawing lines from beneath the bearing area in a direction normal to each pressure band until the line emerges at the surface somewhere adjacent to the bearing area. Notice the similarity to the direction of movement observed in Figures 1 and 2.

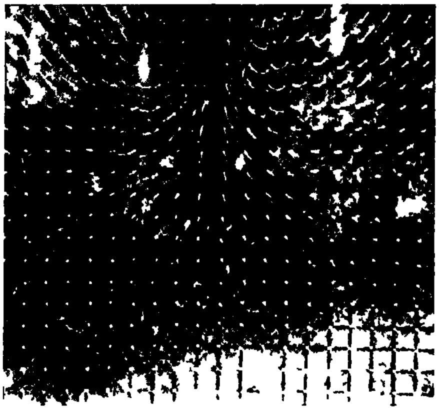

Figure 2. Movement of Plastic Material under a Bearing Area

Attention is directed to the fact that there is a finite limit to the depth of material caused to move when a load is applied at the surface. Apparently all material is blessed with only a limited amount of ability to withstand pressure without failure. In the kind of materials under consideration this property is called stability.

Notice in Figures 1 and 2 that the outward sweep of the action lines is horizontal at some point along their length. If the forces along these lines are resolved into horizontal and vertical components it will be observed, that at this point, only a horizontal component exists, the vertical component being zero. Particles of the soil at this point can thus have only horizontal velocity The surface generated by connecting all of the points where this condition exists has therefore been called the zone of zero vertical velocity and it divides the soil mass into two parts The part inside the zone of zero vertical velocity is distinguished by the fact that all forces acting within it have downward and outward components only. Outside this zone the forces have only outward and upward components.

It is evident, therefore, that only part of the load applied at the surface is reacted upon by upward components of stress originating in the soil within the zone of zero vertical velocity. Some of the stress along the action lines is directed into the surrounding soil and must take its reaction from horizontal components and vertical components developed there. Since the magnitude of downward reaction is limited by the amount of material above any point in this region, the resolution of motion

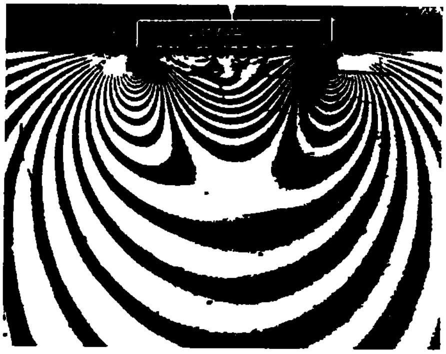

Figure 3. Photo-Elastic Study Showing almost Ideal Development of Pressure Bulbs

is bound to be upward toward the surface. Any downward movement is opposed by the soil mass below

Thus, according to demonstration, the analysis leads to the general conclusion that when a soil is stressed by means of a load on the surface, horizontal and upward movement of the soil particles occurs in the region outside the zone of zero vertical velocity when the magnitude of horizontal stress components is sufficiently great to overcome the natural stability of the soil and the weight of its overlying mass

This leads to the interesting proposition of what will happen if the forces along the action lines emerging at the surface are opposed by an equal and opposite force, as by an overlying asphalt surface. Obviously, if the surface confines the soil without upward movement, considerably more load can be applied to the bearing area than before and the soil will undergo mostly cubical compression reacting primarily as an elastic body. By the same token, if the load is not increased, the intensity of stress on the soil within the zone of zero vertical velocity would be less by the amount of upward reaction against the underside of the pavement adjacent to the bearing area The depth of soil stressed would be in proportion to the amount of load reaching the contact plane beneath the bearing area and between the soil and surface course

THE FUNDAMENTALS OF DESIGN

The analysis up to this point has been a discussion of the distribution of stress in a soil mass which can be considered the subgrade and base for purposes of design. The next

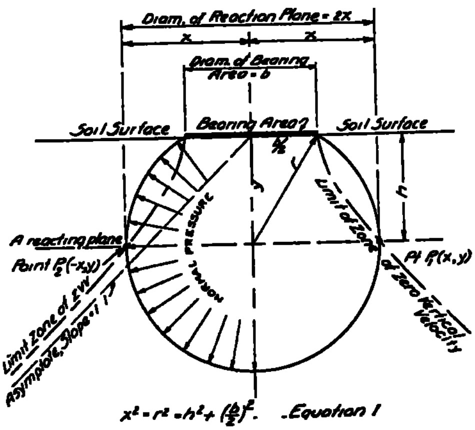

Figure 4. Geometry of Spherical Pressure Bulb

portion of this paper will be devoted to evaluation of the design including an asphalt surface course. The nomenclature used is given in Table 1 and the mathematical derivation of all equations has been included in Table 2. Reference should be made to these tables whenever indicated

Figure 4 pictures a spherical pressure bulb with radius $= r$ under a bearing area of diameter $= b$. Points $P_1(x, y)$ and $P_2(-x, y)$ are in the zone of zero vertical velocity. In the two-dimensional representation, tracing out the locus of all such points on the pressure bulbs of every size will define the zone of zero vertical velocity.

Thus,

$$x^2 = h^2 + \left(\frac{b}{2}\right)^2 \qquad \text{.Equation 1}$$

See Tables 1 and 2

This equation defines the radius, x, of a circle representing the intersection of the zone of zero vertical velocity with a horizontal plane at a depth, h, below the bearing area having a radius—$\frac{b}{2}$. All dimensions are in inches.

With this equation it is possible to determine the relation between a load applied on a

TABLE 1—NOMENCLATURE

W = wheel load, lb.
A = area of equivalent circle of contact of load-inflation quotient $= \frac{W}{p}$ sq in
p = average unit stress on bearing area or tire inflation pressure
= design load-stress, psi
r = radius of spherical pressure bulb, in
b = diameter of bearing or contact area, in
x = radius of circle defined by the intersection of the reaction plane and zone of zero vertical velocity, inches
h = depth of reaction plane beneath plane of bearing area on unconfined subgrade, in , also depth to subgrade when unconfined base equals h, in , thickness
h' = thickness of base course, in
p_z = average stress on A on top of base, psi also bearing strength of base course of depth h or h'.
p_0 = average stress on A on top of subgrade, psi , also bearing strength of subgrade
p'_0 = bearing strength of sub-base or compacted subgrade, psi
p'_s = a parameter = pressure on contact plane between asphalt surface and base, psi
f = force equivalent of punching shear distributed over area A, psi
wt = unit weight of asphalt surface, psi
S = shear strength of asphalt surface, psi
t = thickness of asphalt surface, in
p_d = distributed pressure of confined base against annulus of surface adjacent to bearing area, psi
R = load reaction coefficient, a dimensionless function.

bearing area resting on the surface of a soil and the reacting stress distributed uniformly over the surface of a circular area on any plane within the zone of zero vertical velocity. Such a relationship is expressed as

$$p_s = p_0\left[\left(\frac{2h}{b}\right)^2 + 1\right] \quad \text{. . . . Equation 2}$$

See Tables 1 and 2

With this equation, the average stress, p_s, on the bearing area can be computed for any known stress, p_o, on a reaction plane at a depth, h.

Written in another form

$$h = \frac{b}{2}\sqrt{\frac{p_s}{p_0} - 1} \quad \text{. . . Equation 3 and 3a}$$

See Tables 1 and 2

TABLE 2
MATHEMATICAL DERIVATION OF ALL EQUATIONS

Geometric Properties of Pressure Bulbs

In the right triangle beneath the bearing area:

$$r^2 = y^2 + \left(\frac{b}{2}\right)^2$$

but $r = x$ and $y = h$

So,

$$x^2 = h^2 + \left(\frac{b}{2}\right)^2 \ldots \qquad \ldots \text{Equation 1}$$

Stress Distribution inside Zone of Zero Vertical Velocity

Equating load on bearing area to the stress on the reaction plane at depth h:

$$p_s\left(\frac{b^2}{4}\right) = p_0\left(\frac{4x^2}{4}\right)$$

$$p_s\left(\frac{b^2}{4}\right) = p_0(x)^2$$

$$= p_0\left(h^2 + \frac{b^2}{4}\right) \quad \text{from Eq. 1}$$

and

$$p_s = p_0\left\{\left(\frac{2h}{b}\right)^2 + 1\right\} \quad . \quad \text{Equation 2}$$

Notice also that when $p = p_s$

$$x = \frac{b}{2}\sqrt{\frac{p}{p_0}} \ldots \qquad \ldots \text{Equation 2a}$$

Also

$$h = \frac{b}{2}\sqrt{\frac{p}{p_0} - 1}\,. \qquad \text{Equation 3}$$

and

$$h' = \frac{b}{2}\sqrt{\frac{p_s}{p_0} - 1} \qquad \text{Equation 3a}$$

Pressure on Contact Plane Between Asphalt Surface and Base

1. Pressure on the base

$$p_s' = p - f + wt$$

where

$$f = \frac{P}{A}\,St = \frac{4St}{b}$$

$$p_s' = p - \frac{4St}{b} + wt \qquad \ldots\ \text{Equation 4}$$

2. Bearing capacity of the base

$$p_s' = p_s + p_d + wt$$

According to Eq. 3a,

$$p_s = p_0\left\{\left(\frac{2h'}{b}\right)^2 + 1\right\}$$

$$p_d = St\left\{\frac{\pi b + 2\pi x}{\pi x^2 - \pi\frac{b^2}{4}}\right\}$$

$$= 2St\left(\frac{x + \frac{b}{2}}{x^2 - \left(\frac{b}{2}\right)^2}\right)$$

$$= 2St\left\{\frac{1}{x - \frac{b}{2}}\right\} = \left\{\frac{4St}{2x - b}\right\}$$

Thus,

$$p_s' = p_0\left\{\left(\frac{2h'}{b}\right)^2 + 1\right\} + \frac{4St}{2x - b} + wt. \qquad \text{Equation 5}$$

The General Equation for Design of Flexible Pavements

Combining Equations 4 and 5 gives an expression in terms of all the reacting stresses and dimensions Thus,

$$p - \frac{4St}{b} + wt = p_0\left\{\left(\frac{2h'}{b}\right)^2 + 1\right\} + \frac{4St}{2x - b} + wt$$

and

$$p = p_0\left\{\left(\frac{2h'}{b}\right)^2 + 1\right\} + St\left\{\frac{4}{b} + \frac{4}{2x - b}\right\} \qquad \ldots\text{Equation 6}$$

Simplifying the General Equation

Notice that the last function in Equation 6 may be written

$$\frac{4St}{b}\left\{1 + \frac{1}{\frac{2x}{b} - 1}\right\}$$

Consider the dimensionless function in the above brackets:

Let

$$1 + \frac{1}{\frac{2x}{b} - 1} = R$$

Simplifying

$$R = \frac{\frac{2x}{b}}{\frac{2x}{b} - 1} = \frac{1}{1 - \frac{b}{2x}}$$

TABLE 2—*Concluded*

SIMPLIFYING THE GENERAL EQUATION—*Concluded*

and

$$R = \frac{x}{x - \frac{b}{2}}$$

Evaluating x in terms of stress

$$R = \frac{\frac{b}{2}\sqrt{\frac{p}{p_0}}}{\frac{b}{2}\sqrt{\frac{p}{p_0}} - \frac{b}{2}} \quad \text{so} \quad R = \frac{\sqrt{\frac{p}{p_0}}}{\sqrt{\frac{p}{p_0}} - 1}$$

and finally

$$R = \frac{1}{1 - \sqrt{\frac{p_0}{p}}}$$

Thus equation 6 may be written

$$p = p_s + \frac{4StR}{b} \qquad \text{Equation 7}$$

EQUATION FOR THICKNESS OF ASPHALT SURFACE

By rearranging the terms in Equation 7, the following equation for thickness of the asphalt surface is obtained:

$$t = \frac{p - p_s}{\frac{4SR}{b}} \qquad \text{Equation 8}$$

from which the depth, h, of a plane carrying a stress, p_o, can be calculated when the average stress, p_s, on the bearing area is known.

This series of equations will evaluate the relationships of applied stress, reacting stress and dimensions within the loaded soil when the soil mass is of ample depth as in a subgrade but unconfined by any overlying strata such as surface and base courses.

The objection may be raised at this point that Equations 2 and 3 are based on the assumption that the stress, p_o, is uniformly distributed over the area of the reaction plane and stress, p_s, is uniformly distributed over the bearing area. This objection has nothing to do with the determined position, h, of the reaction plane but is concerned rather with the fact that by any analysis or measurement there is a region at the middle of the reaction plane where the stress is in excess of the assumed uniform value, p_o.

Again we may emphasize that while this is technically true, the uniform distribution of stress is a practical assumption to make since the consequent error is on the safe side in a design. Consider first the higher stress at the middle of the reaction plane. This concentration of stress is directly beneath the bearing area and over the most stable position in the underlying soil. The only result it can have is to show greater support available for the applied load than under the assumed condition. This might in some instances tend to place the reaction plane at a slightly shallower depth. Ignoring this refinement is a conservative conclusion and quite in line with avoiding the use of a micrometer scale when a foot rule will serve a more practical purpose.

As a next step, the effect of confinement on the stress reactions in the soil should be considered Figure 5 depicts a pavement comprising an asphalt surface over a base course and subgrade. The various load and reaction stresses are shown.

First consider a subgrade soil having a bearing strength of p_o. If the design load is placed upon the bearing area resting on the unconfined subgrade soil, the soil would be stressed to a depth, h, according to Equation 3. The radius of the reaction plane according to Equation 2a would be x.

Obviously this load cannot be carried on the unconfined soil since all of the soil beneath and adjacent to the bearing area to the depth, h, will be overstressed and therefore moving. One thing that can be done then is in effect to replace the overstressed soil in this layer with a more stable soil and, if the soil selected is stable enough, the load can be carried without causing the underlying subgrade to fail. In effect this creates and describes a base course. However, the depth required for heavy loads on a poor subgrade under these conditions (i e., soil unconfined) is generally large. Furthermore, such a design does not in any way utilize the advantage of confinement obtained from an asphalt surface on top of a base.

It is apparent from previous discussion that if an asphalt surface of adequate strength is included in the design, the base course can be made appreciably thinner without overstressing the subgrade Figure 5 illustrates the principles adopted for a balanced design.

An asphalt surface course is proposed having sufficient strength to supply the necessary shear reactions to resist punching through by the bearing area and the upward thrust of the adjacent soil against the surrounding annulus of surface. The inside diameter of the annulus is the diameter of the bearing area. The outer diameter is equal to $2x$, or the diameter of the zone of zero vertical velocity on the reaction plane for the unconfined subgrade This limits the total upward reaction of the soil against the underside of the asphalt surface to an area equal to that on the reaction plane for the unconfined soil.

All of the dimensions and relations of the variables for a complete design can now be derived by equating the pressures above and below the contact plane between asphalt surface and base course The pressure on the contact plane is designated as p'_s and is the parameter for equating the applied stress to reacting stresses

The downward pressure, p'_s, according to Figure 5 is the pressure, p, on the bearing area, minus the pressure factor, f, representing a

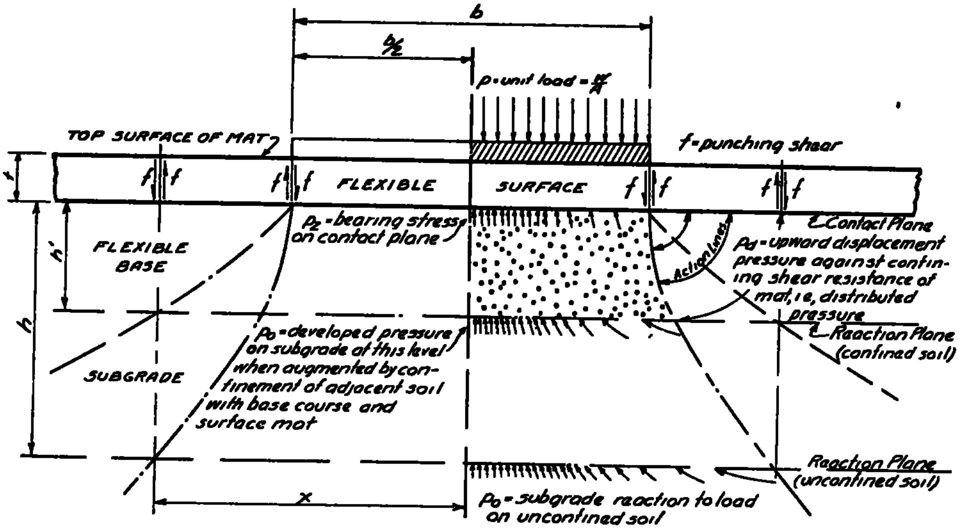

Figure 5. Distribution of Stresses in a Flexible Pavement System

force required to punch a hole in the asphalt surface by punching shear, plus the unit pressure (wt) from the weight of the mat, i e.,

$$p'_s = p - f + wt$$

Evaluating f in terms of the shear strength of the asphalt surface, S, thickness of the surface, t, and dimensions of the bearing area in terms of the perimeter area ratio $\frac{P}{A} = \frac{4}{b}$ gives

$$p'_s = p - \frac{4St}{b} + wt \quad \text{Equation 4}$$

See Tables 1 and 2

The bearing power of the base is the pressure, p'_s, on the contact plane equal to the pressure, p_s, that can be developed as bearing stress within the zone of zero vertical velocity due to subgrade bearing strength, p_o, on the reaction plane at depth, h', plus the distributed pressure, p_d, due to the upward thrust against the confining influence of the asphalt surface, plus the unit pressure (wt) from the weight of the surface, i.e.,

$$p_s' = p_s + p_d + wt$$

Evaluating p_s according to Equation 2 and p_d according to the derivation in Table 2, this becomes

$$p_s' = p_0\left[\left(\frac{2h'}{b}\right)^2 + 1\right] + \frac{4St}{2x - b} + wt \text{....Equation 5}$$

See Tables 1 and 2

The general equation for design in terms of all the reacting stresses is now obtained by equating the conditions described in Equations 4 and 5 about the contact plane with the following result:

$$p = p_0\left[\left(\frac{2h'}{b}\right)^2 + 1\right] + St\left[\frac{4}{b} + \frac{4}{2x - b}\right] \text{....Equation 6}$$

See Tables 1 and 2

Equation 6 may be simplified as follows:

$$p = p_s + \frac{4StR}{b} \text{.. Equation 7}$$

See Tables 1 and 2

where

$$R = \frac{x}{x - \frac{b}{2}}$$

$$\text{or } R = \frac{\sqrt{\frac{p}{p_0}}}{\sqrt{\frac{p}{p_0} - 1}} \text{ ...See Tables 1 and 2}$$

The dimensionless function R is apparently a load-reaction coefficient approaching the value $R = 1$ for very weak subgrades or extremely heavy loading.

Expressing thickness of the asphalt surface in terms of all the stresses, Equation 7 becomes

$$t = \frac{p - p_s}{\frac{4SR}{b}} \text{......Equation 8}$$

The following calculations for an airport runway surface will suffice to illustrate the use of the various formulae:

AIRPORT RUNWAY DESIGN

Heavy loading, Poor subgrade.

Data:

$W = 60000$ lb. $\quad b = 31.90$ in. $\quad \frac{4}{b} = .125$

$p = 75$ psi $\quad \frac{b}{2} = 15.95$ in. $\quad S = 50$ psi.

$p_o = 15$ psi

Calculations:

$\frac{p}{p_o} = 5.00 \quad \sqrt{\frac{p}{p_o}} = 2.24 \quad \sqrt{\frac{p}{p_o} - 1} = 2.00$

$h = 15.95 \times 2.00 = 31.90$ in. $\quad x = 15.95 \times 2.24 = 35.7$ in. $\quad R = \frac{35.7}{19.75}$

$h' = \frac{h^2}{2x} \quad x - \frac{b}{2} = 19.75$ in $\quad = 1.81$

$h' = \frac{(31.90)^2}{2\ (35.7)} \quad \frac{p_s}{p_o} = \left(\frac{14.3}{15.95}\right)^2 - 1 \quad p_s = 15 \times 1.801$

$= 14.3$ in. $\quad = 1.801 \quad = 27.0$

$$t = \frac{75 - 27}{.125\ (50)\ 1.81} = 4.24 \text{ in.}$$

A typical cross-section will have a total thickness of 18¼ in obtained from the sum of $h' + t = 14$ in. + 4¼ in. = 18¼ in. The thickness, h', of the base should always be rounded off to the nearest inch and the thickness of asphalt surface to the nearest one-quarter inch.

The asphalt surface mixture must have a shear strength, S, in this case equal to or greater than the 50 psi. assumed in the example. This would most probably be a hot-mix asphaltic concrete laid in two courses. The 14-in. base must be made of suitable aggregate or soil combination having a bearing strength of $p_s = 27$ psi. Considering that the subgrade itself has a bearing strength $p_o = 15$ psi, the aggregate for the base would no doubt be some selected top soil or other local material Better yet, and certainly more economical would be a 14-in base comprising two 7-in courses. In this case, p_s for the upper layer (base) would be 27 psi. and for the bottom 7-in. (sub-base) the strength by Equation 2 would be

$$27 = p_o'\left[\left(\frac{7}{15.95}\right)^2 + 1\right]$$

$$p_o' = \left(\frac{27}{1.193}\right) = 22.6 \text{ psi.}$$

In still another case, if the compacted subgrade soil can be used, suppose its bearing strength is found to be $p_o' = 23.5$ psi The thickness of the two courses can be evaluated as follows. From Equation 3a:

$$h'(\text{base}) = 15.95\sqrt{\frac{27}{23.5} - 1}$$

$$= 15.95\sqrt{1.150 - 1}$$

$$= 15.95(.387) = 6.18 \text{ in., or 6 in. approx.}$$

and h'(sub-base)

$$= 14 \text{ in.} - 6 \text{ in.} = 8 \text{ in.}$$

Thus, the base would comprise 8-in. compacted subgrade and a 6-in. aggregate base course of 27 psi. bearing strength.

The possibilities of this design method and its flexibility provide for the greatest use of local aggregate in the base courses. The balance between asphalt surface course and soil bases is economically and mechanically good. Various asphalt mixtures can also be used, of course In Table 3 is a list of typical average shear strengths of popular asphalt surface mixtures and the usual thicknesses that they are laid It is assumed in any design that acceptable specifications will be used with proper construction methods on the job.

TABLE 3
LIST OF AVERAGE SHEAR STRENGTHS AND PRACTICAL THICKNESS FOR SOME OF THE MOST POPULAR ASPHALT SURFACES

See Notes	Kind	Type		Average shear strength, psi	Range of practical thickness, in
1	Asphaltic Concrete	Hot	Plant Mix	50	3–9
2	Sheet Asphalt	"	" "	45	2–5
3	Topeka Mix (Stone-filled sheet)	"	" "	40	2–4
4	Hot Penetration Macadam	"	Penetration with A C.	35	2½–6
5	Dense Graded Aggregate Mix	Cold	Plant or Road Mix	20	2–4
6	Macadam Aggregate Mix	"	Plant or Road Mix	15	2–4
7	Light Surface Seal	"	Plant or Road Mix	10	½–1
8	Heavy Surface Treatment	"	Road Application	7	¾–1½
9	Light Surface Treatment	"	Road Application	5	¼–½

Notes
1 Minimum single course 3 in Minimum base course 3 in and minimum surface course 2 in
2 Binder and top proportional between 1¼ in binder—¾ in top for 2 in surface to 3 in binder—2 in top for 5 in. surface.
3 One course 2 in –4 in
4 Maximum single course 4 in
5 " " " 3 in
6 " " " 3 in
7 Fine mix maximum size aggregate 95–100 per cent through No 10 sieve
8 Double or triple treatment
9 Single treatment

TESTS

Shear Strength of Asphalt Mixtures

Evaluating the shear strength of the asphalt mixtures is generally a simple matter. Almost any device that will test a cylinder or beam of asphalt mixture for direct shear is suitable Any device that will measure punching shear on slab or disc-like specimens will serve. The control of these tests is simple Loads should be applied gradually until the test specimen fails. Record the maximum shear stress in pounds per lineal inch per inch of specimen thickness In other words, pounds per square inch of shearing planes This gives the value, S, in the formulae While different methods and various techniques will give somewhat varied results, these variations are not as great as the variations observed in a single job with one testing apparatus The tests should preferably be made on specimens at 140 F. and, in the case of cold-mixes, after a suitable curing period.

Bearing Strength of Subgrade

Considerable doubt is carried in some people's minds regarding suitability of actual bearing tests for evaluating the bearing capacity of soil for flexible pavement design. I have seen much data obtained by elaborate, heavy field testing equipment that certainly does much to convince the uninitiated that the bearing test is temperamental This is unfortunate because in most cases like this, the field load bearing test has been misused and the data misunderstood

I have listened to, and been assailed with, the shortcomings of the rigid bearing plate Much of the data flouts a negative Housel n-value or, in other words, indicates a negative developed pressure with an excessive concentration of stress on the perimeter of the bearing area. In fact, I believe data I presented in a paper at the 10th Annual Asphalt Paving Conference in New Orleans in 1932 first called this phenomenon to Mr Housel's attention. Since that time his studies have led to the conclusion that some flexible loading device might avoid this complication He points out, however, that his insistence on the use of rigid plates was justifiable in the "simplification of settlement measurements and the fact that secondary boundary effects has caused no difficulty in load tests for building foundations" He states further, "This latter observation can be traced to the fact that natural soils are comparatively compressible and uniform bedding of the plates can be obtained without more than negligible stress concentration at the edge" And further, "By the same line of reasoning it appears that paving surfaces fall into a different category and these effects should be eliminated"

I cannot agree with all of this because of its general implication Whether I do or not, I can be assured, however, that we do agree that the bearing test with rigid plates gives proper results at least for subgrades and material for soil bases, which for the purpose of the present design is sufficient. Personally, I do not expect as much improvement from the use of flexible bearing areas as some believe will result My interpretation of the stress concentration showing up on the perimeter when testing flexible pavements at low deflection would be that this phenomenon is rather due to the fact that with the first inclination of the asphalt surface to bend the base or subgrade is stressed over the annulus surrounding the bearing area. Since the area of the annulus is greater than that of the bearing plate, the reaction is bound to overemphasize the factor, m, of perimeter shear. Accordingly I expect the results with flexible plates to be not much different than those with the rigid plates and certainly the technique and apparatus will be immeasurably complicated

Therefore, for measuring the bearing capacity of the subgrade, p_o, and the bearing capacity of the base course materials, p_s, p_o', etc, this method will be satisfied with data obtained from bearing tests with rigid plates. Whatever the error, it will keep the design conservative since the rigid plates will merely result in lower bearing values for large areas than the flexible plates, if my belief is proven wrong.

Bearing tests of the subgrade and potential base course materials run in the field should be interpreted for the effect of moisture and consolidation on the basis of corresponding laboratory bearing tests with suitably prepared samples of the soil. Equipment for such testing with bearing plates 1, 2, 4 and 6 sq. in. area has been described in the 1932 paper referred to earlier, as well as by Messrs. Housel and Berry in their 1935 A S T M paper. Small portable equipment for use of small bearing plates could be used equally well in the field

The bearing value to use in the formulae should be calculated from the data for the size of bearing area designated by the load-inflation quotient in every case. The general equation for this is obtained from the best straight line through the data when the bearing value at a given deflection is plotted against the perimeter-area ratio of the bearing plates used

Some investigators have decided that the bearing strength at 0 10-in. deflection is a proper criterion for design. Others have argued for 0 20 in, 0 25 in. and Provost Hubbard of the Asphalt Institute has offered data to show a 0.5-in deflection of the surface is all that should be tolerated. Personally, we believe a design is safe if the bearing strength of the subgrade, p_s, is selected at 0 10 to 0.20 in and the bearing strengths of base and sub-base materials on such basis that the total deflection of the whole pavement does not exceed Mr. Hubbard's figure The laboratory test on finite depths of soil for the base courses should supply the data

It is necessary in all bearing tests either to correct the load-deflection data for the effect of seating the plates and consolidation of the soil or use some method of repetitive loading suggested by Hubbard in Asphalt Institute Research Bulletin No. 9, or use the yield-value method suggested by M. Housel in the 1942 *Proceedings*, Association of Asphalt Paving Technologists.

These many thoughts have been discussed to emphasize that the approach to bearing tests should not be hampered by too many refinements. It will not affect the ultimate design a great deal to call a 20-psi. bearing value subgrade an 18 or 22-psi. subgrade. When the amount of deflection to be tolerated in the pavement has been decided upon, the bearing value measured will not be out of line by any careful technique as much as the variations to be encountered in soil conditions and construction in the field.

One last thought· bearing strength determined by other methods than bearing tests may be used in their proper place in the formulae for design. If the bearing value of a soil, determined by some other test such as the triaxial shear test, can be evaluated in terms of the size bearing area and load required in the design, such values may be taken for p_o, p_o' and p_s without changing the ultimate answer a great deal.

CULVERT DESIGN IN CALIFORNIA[1]

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SYNOPSIS

After a 20-year period of comparatively moderate storms in California, the heavy runoff of 1937–38 caused such expensive damage to highways throughout the state that the Division of Highways of California, Department of Public Works, authorized an exhaustive departmental study of the performance of culverts and survey of culvert practice.

It was found that conduit sizes were usually determined directly from formulae, such as Talbot's depending only on drainage area and a runoff factor, and that the runoff factor was modified to fit local conditions as determined by experience Consequently, there was a tendency toward under design after a period of drought and toward over design after a period of floods It was also evident that headwalls, endwalls, and other appurtenances had not been designed from any rational application of hydraulics

From this need there was developed a new concept for design of culvert conduits, combining two principles; (one) design to pass a 10-year flood without static head on crown of conduit at the entrance, (two) balanced design of barrel and appurtenances to pass a 100-year flood without serious damage to the highway

These two principles are applied successively for each problem For the typical case with free outfall, the outlet is far from full when the entrance is just full. If gradient of flow line exceeds a neutral slope (say 0 8 per cent for smooth and 3

[1] The conclusions and recommendations expressed are those of the writers and do not necessarily reflect policies of the California Department of Highways, Division of Highways