Plate Bearing Tests and Flexible Pavement Design in Florida

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● FIELD PLATE BEARING TESTS have been performed since 1958 in conjunction with the flexible pavement design research program sponsored by the Florida State Road Department. The first field tests were run using the 3-sq in. (1.95-in. diamete piston of the California Bearing Ratio test. This was followed by the use of 4-, 6-, 8-, 10- and 12-in. diameter rigid plate tests. The tests were initiated to obtain the bearing values of highway base, subbase and subgrade materials as separate layers and as composite pavement sections. All bearing values were related to the deflectio of the plate and the corresponding pressure on the plate. Recent tests have dealt with the bearing value of composite sections, including an asphalt concrete wearing surfac

The review of plate bearing tests, performed in the State of Florida, has been subdivided into sections that are directly related to the various phases of the research pr gram, including (a) plate size and zone of stress, (b) variation of bearing values, (single layer relationships, (d) subgrade modulus as related to plate size, (e) two-la theory relationships, (f) thickness of wearing surface, and (g) repetitional loads.

PLATE SIZE AND ZONE OF STRESS

When a circular plate is loaded with a uniform load a zone beneath the plate is stressed. For a homogeneous semi-infinite mass, vertical stresses and maximum shearing stresses may be readily calculated by the use of equations developed by Jürgenson (1), Love and others. Of particular interest, when investigating the stresses associated with plate bearing tests on fle ible pavement layers, is the depth of the zone of significant stress as related to t diameter of the loaded area. The stress zone is often defined by a "pressure bulb which defines points of equal stress inte sity. Accurate pressure bulb or contou of stress diagrams may be found in man publications. Some excellent diagrams appear in HRB Bulletin 114.

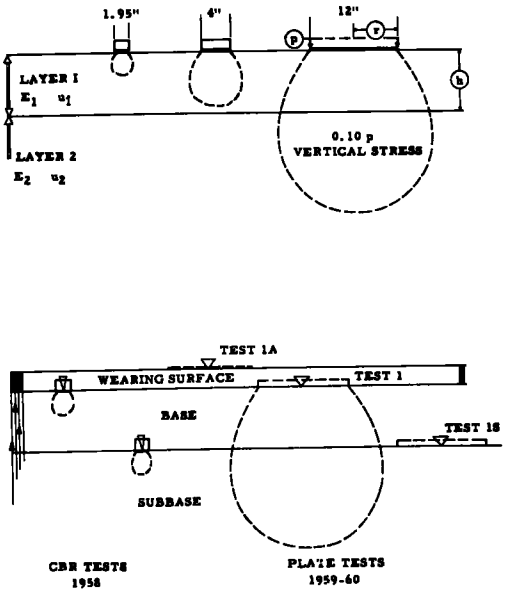

Figure 1. Plate size and pressure bulbs for plates and location of plate tests.

Figure 1a presents the pressure bulb corresponding to a vertical stress inten ty of 0.1 p for plates of 1.95-, 4-, and in. diameter. The depth of significant stress is about 1¾ times the diameter. For the maximum shearing stress of 0. p the depth of significant stress is abou 1¼ times the diameter. As can be seen the stressed zone increases in depth as the plate diameter increases. The CBR piston used for the original bearing test (1958) on base materials has a diamete of 1.95 in. Considering the stressed z under the piston it is obvious that the b ing value obtained is only a direct index

the strength of the base layer. Figure 1a also shows the stressed zone of the 12-in. diameter plate. It can be seen that for plate tests performed on the top of the base that a homogeneous mass of one layer does not exist throughout the stressed zone but a system of two layers is stressed. This system cannot be analyzed as a single layer but should be investigated as a layered system as was done by Burmister (2). Burmister's work is discussed later.

The effects of using a 1.95- and 12-in. diameter plate when testing a typical flexible pavement section are shown in Figure 1b. The advantages and limitations of each size plate are directly related to the depth and extent of the stressed zone. The small plate will give stress and displacement values of distinct and separate layers, whereas the large plate will give values of the layered section. Realizing most wheel loads have contact pressure areas that may be assumed circular, the use of the test data and theoretical stress computations for circular bearing areas may be used extensively for analysis.

TABLE 1

PERCENTAGE VARIATION OF BEARING VALUES; BASE STUDY, 1958

Material	Average of Maximum Values				Max. Values
	1.95-In. Plate (CBR)	4-In. Plate	8-In. Plate	12-In. Plate	1.95-In. Plate (CBR)
	(a) Standard CBR Tests				
nd clay	10	-	-	-	17
merock (N)	15	-	-	-	25
merock (S)	20	-	-	-	47
ab. shell	20	-	-	-	37
ell	25	-	-	-	49
	(b) Load Increment Tests, ASTM 1196-57				
ay sand (4)	35	25	20	15	35

The early studies conducted in the state were with the 1.95-in. piston. The bearing ts were run on all typical base materials throughout the state and on most subbase d subgrade materials. Results were presented in reports (3) issued in 1958. These rly tests established the strength characteristics of the individual layers and later, connection with other test data, led to the development of a modified CBR design thod. This was possible because if the properties of the distinct layers are known l/or specified for field construction, a system of layers may be proportioned empirlly which will have a known field performance. Later tests utilized 8- and 12-in. meter plates to develop the relationships of layered systems which were and are be- investigated experimentally and theoretically.

VARIATION OF BEARING VALUES

The use of small plates has been investigated and was reported (4), in 1959. Conerable economy could be realized by performing tests with small diameter (1.95-in.) tes; however, small plates tend to give erratic and somewhat inconsistent results n performing duplicate tests. Small plates are more sensitive to soil variations in ogenity, large particles, and to surface conditions.

The base study, noted previously, included data which is directly related to the varon of bearing values. The results of tests, repeated a minimum of three times, led he development of Table 1 which gives the average of the maximum percentage varin and the maximum percentage variation for the tests reported in 1958.

Collins (4) gives an indication of the maximum percentage variation of the 1.95-in. plate and, in addition, the variation of the plates of larger sizes may be estimated from the data. These data are also given in Table 1. The effect of larger plates in reducing the percentage variation is evident.

TABLE 2

NUMBER OF TESTS REQUIRED TO GIVE A MEAN WITHIN 10 PERCENT OF TRUE MEAN WITH 95 PERCENT CERTAINTY

	Number of Tests					
Material	CBR Plate	3-In. Plate	4-In. Plate	6-In. Plate	10-In. Plate	12-In Plate
	(a) Plates loaded rapidly (5)					
Clay	18	4	-	10	-	-
Silty clay	34	3	-	26	-	-
Sand	42	14	-	9	-	-
Grav. sand	9	10	-	9	-	-
	(b) Load Increment Tests, ASTM 1196-57 (4)					
Clay sand	32	-	10	-	10	8

The percentage variation varied with soil type. This is expected because, as mentioned previously, the scatter would be related to homogeneity, particle size and surface irregularities.

An attempt was made to analyze the data of Collins (4) using statistical methods. Sufficient data were not available for a reliable analysis; however, it is of interest to compare some preliminary calculations with those of Robinson and Lewis (5) who reported the results of a series of tests where 20 repetitions of each test were made to establish a true mean. The results of the study are given in Table 2. It may be noted that no definite curve exists relating required number of tests and plate size but that a trend does appear. The number of tests required for the 6-in. plate is significantly less than the number required for the 1.95-in. plate. The 3-in. plate test results are exceptional.

Using some of the data obtained in 1959, with a maximum of six repetitions of each test, the number of tests required for identical criteria are noted in Table 2. Many additional repetitive tests are necessary in this area of study to establish relationships between plate size, number of tests required, and soil type.

SINGLE-LAYER RELATIONSHIPS

A review of the single-layer theory as related to stress and deflection beneath a circular rigid plate was presented in previous reports (4, 6). The original problem of computing the stresses beneath a circular plate was solved by Boussinesq. Boussinesq obtained an equation for the deflection of a rigid plate located on a semi-infinite elastic body as follows:

$$w = \frac{\pi \ pr}{2E} (1 - \mu^2)$$

in which

w = deflection
π = 3.14
p = pressure
r = radius of plate

E = modulus of elasticity
μ = Poisson's ratio
for $\mu = 0.5$
$w = 1.18 \frac{pr}{E}$ .... (average deflection of a rigid plate) for a flexible plate

$$w = 1.5 \frac{pr}{E}$$

Terzaghi, 1943, noted that soils were not truly elastic, but did retain the concept of elasticity and essentially replaced E by a soil modulus, M, which was equal to Mo + az. The resulting deflection equation may be written as

$$w = K' \frac{pr}{Mo + az}$$

When a = o and if w is constant, the pressure required to produce a given settlement w' is

$$p = \frac{w' \, Mo}{K' \, r} = K \frac{1}{r}$$

in which

$$K = \frac{w' \, Mo}{K'}$$

The foregoing equation is that of a hyperbola. The equation $p = K \frac{1}{r}$ is a theoretical relationship between pressure for a given w and plate size. If the soil modulus, M, is varied, a family of curves may be constructed.

SUBGRADE MODULUS AS RELATED TO PLATE SIZE

Reference 4 presented a set of curves developed from experimental data (Fig. 2), relating subgrade modulus, k, and diameter for various soil types. Noting that k is equal to the pressure at a given deflection divided by the deflection, it is possible to superimpose some theoretical curves of the $p = K (\frac{1}{r})$ type on the experimental data (Fig. 2). The theoretical and test curves show good agreement for a = o. Three different soil modulus values have been plotted to present a typical family of curves.

The relationship between subgrade modulus and plate size may be expressed in many ways in mathematical form. Because the relationship between CBR (Load Increment Test) and larger plate sizes is of primary interest in Florida and is essentially one of subgrade modulus and plate size, the following equation is presented. CBR, plate size and pressure are related by:

$$p = K \frac{1}{r} \tag{1}$$

When w = 0.1 in., where 0.1 in. is the deflection of Standard CBR, the "CBR" Equation (Load Increment Test) becomes

$$p = \frac{10(CBR)}{r} \tag{2}$$

which

p = pressure, in psi;
CBR = ratio at 0.10-in. penetration; and
r = radius, in inches.

Eq. 2 may be used to relate CBR, p, and r until additional test data are available. Reasonable agreement exists between test and theory for CBR values greater than 10. Additional testing is necessary to relate the results of the Standard CBR Test and the Load Increment Test (ASTM, 1196-57 and (4)).

TWO-LAYER THEORY RELATIONSHIPS

The analysis of the two-layer system was developed and presented by Burmister in 1943 and most recently discussed in 1958 (7). An investigation related to the two-layered system was conducted in 1960 and reported (6). Before discussing the results of the recent tests some comments about the layered system may be desirable to visualize the action of a typical system. A typical two-layer system is shown in Figure 1a. This system represents much closer agreement to the actual problem that exists when pavement sections are loaded either by wheel loads or plates. The effectiveness of spreading load or reducing vertical stress, when a reinforcing layer with a modulus E_1 is used over a second layer with a modulus E_2 less than E_1, has been discussed and illustrated by Burmister (7). The reduction is significant and the effectiveness of reduction increases as E_1/E_2 increases. Another factor of importance in the two-layer system is that of an increase of vertical stress gradient toward the interface, which in turn causes a shearing stress buildup. The shearing stress, as mentioned by Burmister, is much more important than in the Boussinesq case and must be sustained at the interface for continuity between the layers. Shearing stress could lead to failure

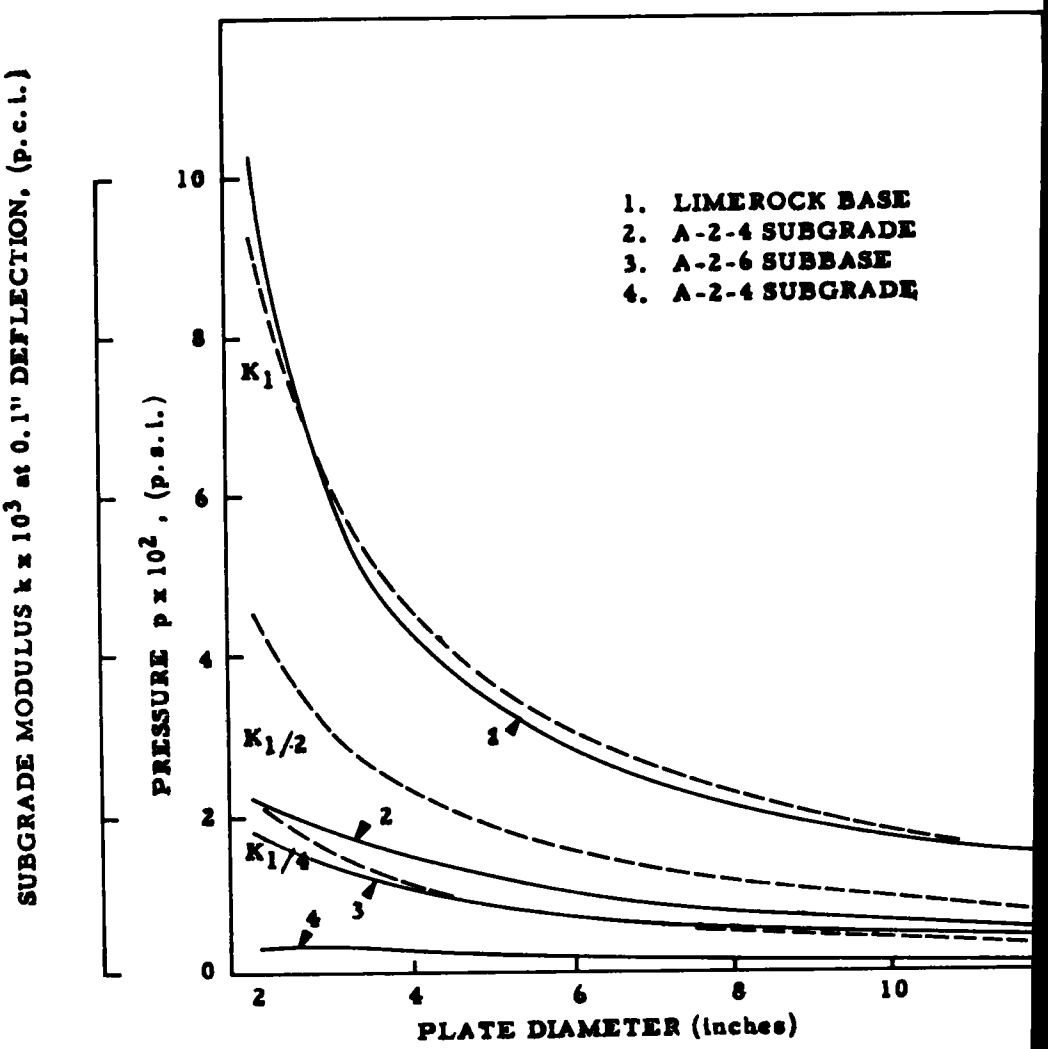

Figure 2. Subgrade modulus-diameter curves for some soils and theoretical curves.

due to excessive shearing strain; therefore, the deflection at the surface must be limited so that the shear stresses and strains are not critical. This limiting deflection may be about 0.05 in. for high-type flexible pavements constructed in Florida.

The deflection of a layered system as related to vertical stress and shear stress may be summarized by Burmister's influence curves of settlement coefficient, F_W (Fig. 3). The deflection equation for the layered system rigid plate is:

$$w = 1.18 \frac{pr}{E_2} \quad F_W \left[\frac{r}{h}, \frac{E_2}{E_1} \right] \tag{3}$$

or

$$w = 1.18 \frac{pr}{E_2} \quad F_W \tag{4}$$

Eq. 4 is in the same form as the Boussinesq equation for one layer and reduces to this case for a one-layer system. In the two-layer system the settlement coefficient curves are related to r, h and E_2/E_1. The effect of these variables will be shown by the curves, both test and theoretical, that follow.

The state conducted numerous tests on layered systems consisting of a typical Ocala limerock base material and a clay-sand subbase. The base thickness was varied from to 11 in. in controlled sections over a 600-ft test area, to study the effect of the thickness, h, of the reinforcing layer. Plates having diameters of 1.95, 4, 8, and 12 in. were used on the different base course thickness to study the effect of radius of plate r and thickness of layer h. The results of these tests are presented in some detail in an earlier report (6).

Figure 4 shows the equipment used for some of the field testing, performed in conjunction with the recent plate bearing test studies.

The data obtained in recent studies have been re-evaluated and important parts are summarized and discussed.

To calculate the theoretical deflection, w, of the layered system, accurate values of the moduli, E_2 and E_1, are necessary. It has been found (8) that a minimum of thickness of soil at least 1.5 times the diameter of the loaded plate is necessary for calcula-

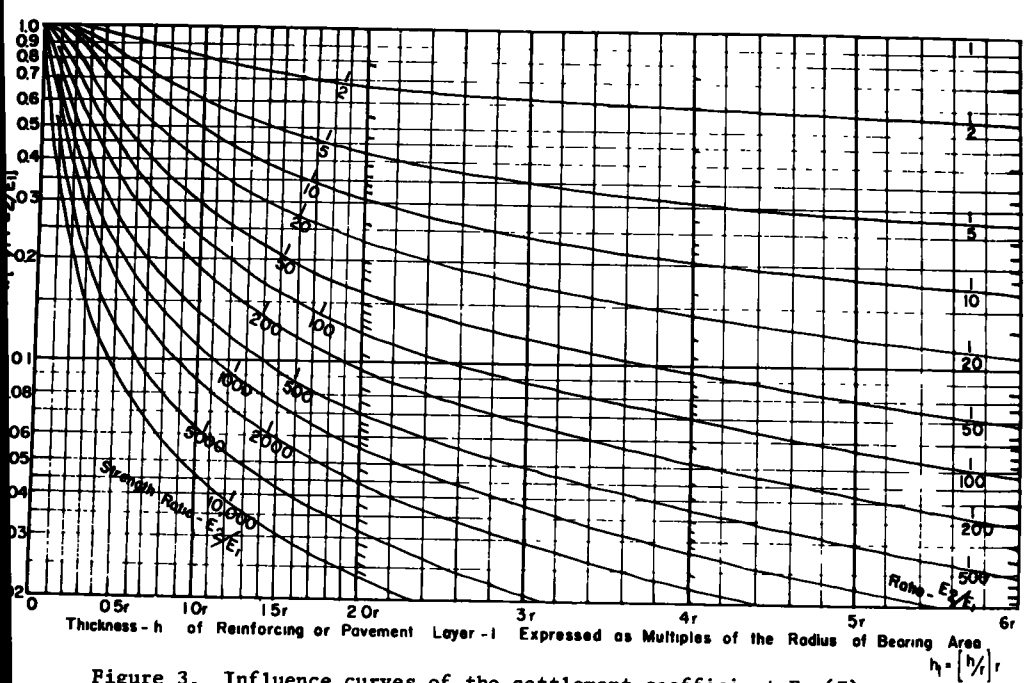

Figure 3. Influence curves of the settlement coefficient F_W (7).

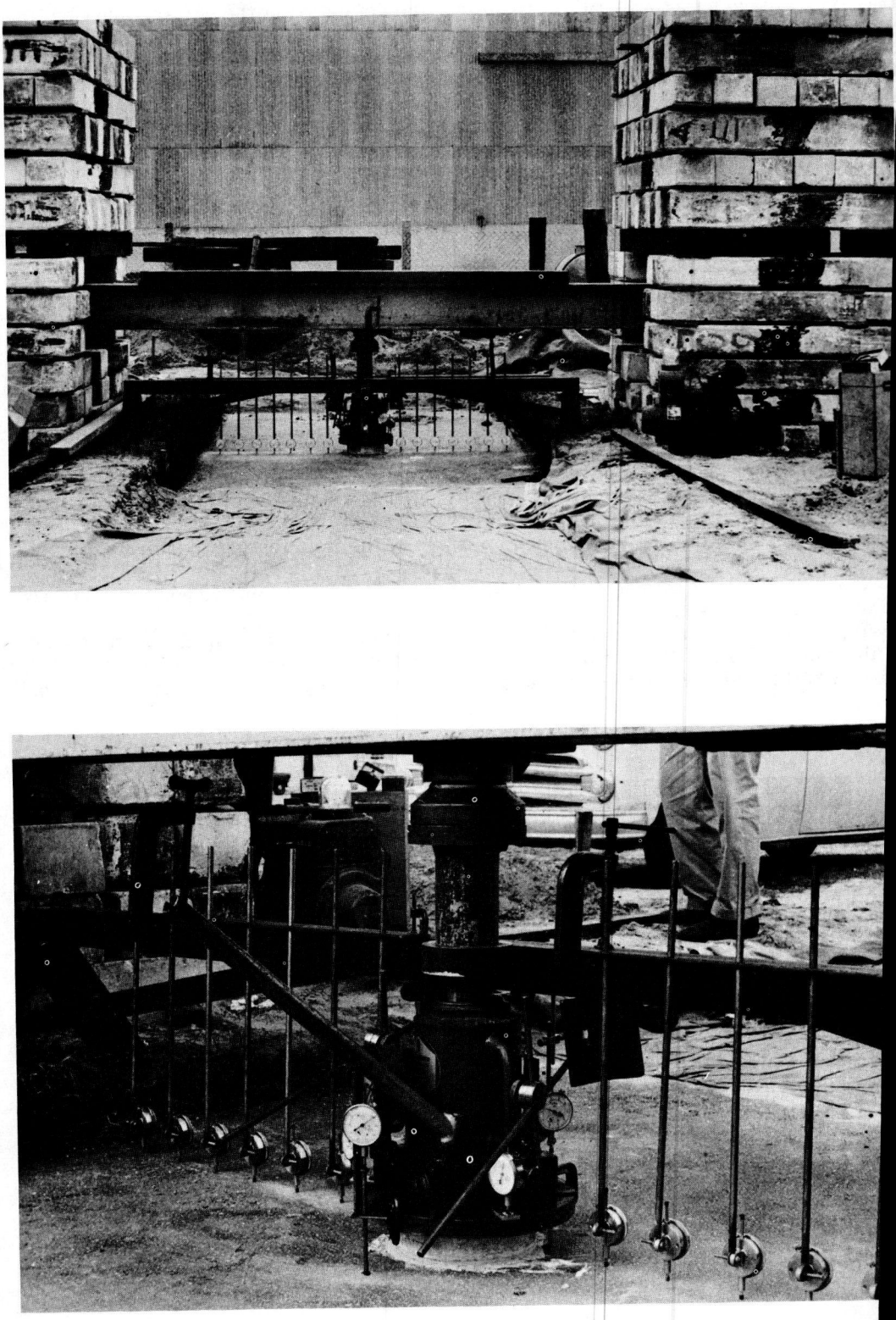

Figure 4.

tion of the modulus values and a thickness of twice the diameter is recommended.

Examination of numerous pressure deflection curves indicated that a straight line relationship did not extend much beyond a deflection of 0.05 in. and this was selected for the calculations that follow. Early work used a deflection of 0.10 in., which appears to be too high for all materials, particularly limerock. Using a deflection of 0.05 in., the modulus E_2 may be calculated for a typical subbase as follows:

$$E_2 = 1.18 \frac{pr}{w} F_w \tag{5}$$

$$= 1.18 \frac{(220)}{0.05} (1) = 5,200 \text{ psi}$$

For this study, pr equaled the average product of the pressure (from ASTM 1196-57 and (4)) times the radius for the 4-, 6-, 8-, 10- and 12-in. diameter plates. The depth of soil tested was in all cases equal to or greater than 4r, and E_1 = 20,000 psi. (Tests are being performed during the summer of 1960 to evaluate E_1 for different base materials. Bearing tests are run in a 7-ft x 7-ft pit using a 12-in. diameter plate. Base thicknesses are increased from 4 to 24 in., the latter thickness being used to compute E_1. Using this technique, the modulus value as well as the effect of varying the thickness, h, may be investigated. Two-layer influence curves are being prepared for typical systems.)

Having evaluated E_2 and E_1 and knowing the geometry of the section to be studied, values of deflection, w, or pressure, p, for a given deflection may be computed from Burmister's equation:

$$w = 1.18 \frac{pr}{E_2} F_w \tag{6}$$

in which F_w is obtained from Figure 3.

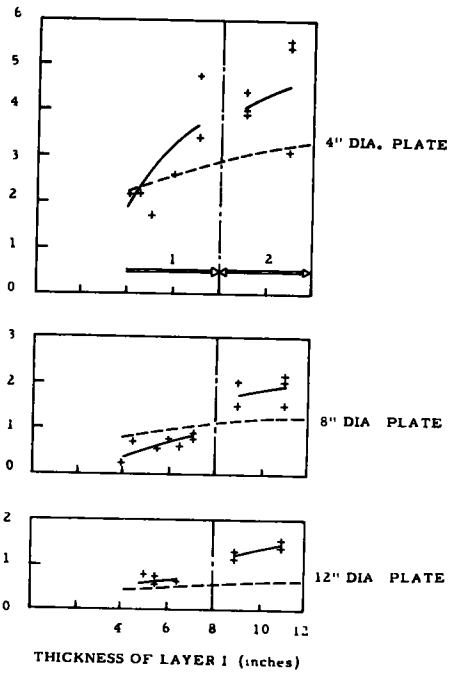

Figure 5. Experimental and theoretical curves—US 441—2-layer system study (layer I, E_1 = 20,000 psi, layer II E_2 = 5,200 psi).

As part of the two-layer study, tests were conducted in a test pit as well as on US 441. Tests in this series were performed with 4-, 8- and 12-in. diameter plates and followed a procedure similar to ASTM Standard 1196-57.

The effect of varying plate size as well as base thickness is shown in Figure 5. The curves have been developed from US 441 test data. Agreement between theory and test is fair. The 4- and 8-in. plate test curves cross the theoretical curve showing minus and plus variation. This may be attributed to the fact that when the thicker base sections were constructed in two layers (2 in Fig. 5), density may have increased which would increase E_1. Increasing E_1 from 20,000 to 25,000 psi for the test on the double lift base sections would result in reasonably good agreement between theory and test. This magnitude of increase is definitely possible.

The results of field tests conducted up to the present time indicate that the use of layered theory is quite promising. Some adjustment of the constants used in the Burmister theory may be necessary to predict the exact results obtained in the field. This is expected inasmuch as the

degree with which real conditions may agree with the idealized conditions is one of the major problems associated with the use of the theoretical equation of Burmister.

THICKNESS OF WEARING SURFACE

The most recent work completed, dealt with the effect of increasing the wearing surface thickness and studying the effects on the strength and deformation characteristics of a two-layer system. The complete section was then subjected to repetitional loads. A Type-I asphaltic concrete surface was used in the research study and was tested as described in a recent report (8).

The effect of adding layers of wearing surface of 1.5-, 3-, and 4.5-in. total thickness to a limerock base 24 in. thick did generally follow the layered system concepts. The data indicate that the actual experimental deflection values are almost equal to the predicted values and the variation that exists between the different thicknesses of surfacing is within the range of experimental error. It appears that the two-layer theory is reasonable for predicting the behavior of the system investigated. Table 3 gives a comparison of experimental and theoretical data. Deflection values are given for a

TABLE 3

EXPERIMENTAL AND THEORETICAL DATA OBTAINED FROM 8-IN. DIAMETER PLATE TESTS PERFORMED ON ASPHALTIC CONCRETE SURFACES OVER LIMEROCK BASE, 1960

Condition	Deflection of Plate (in.) for Surface Thickness of 1.5 In.	3.0 In.	4.5 In.
Experimental	0.055	0.058	0.053
Theoretical[1]	0.053	0.051	0.049

[1]Surface thickness as noted;
24-in. limerock base;
8-in. diameter rigid plate; p = 200 psi;
E_2 = 17,000 psi; and
$E_2/E_1 = 1/1.6$, $\mu_1 = \mu_2 = 0.5$.

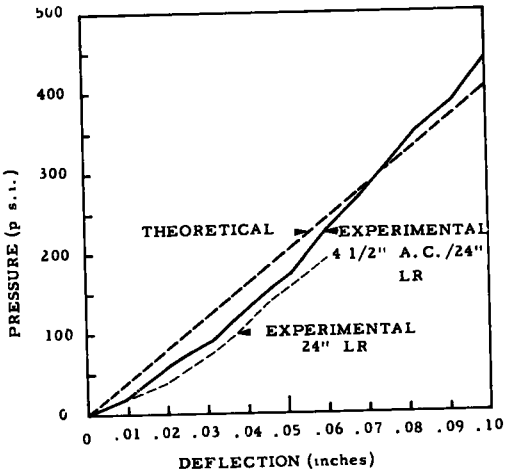

Figure 6. Pressure deflection curves for $4\frac{1}{2}$-in. asphaltic concrete, type I over limerock (LR) base, pit tests (E_2 = 17,000 psi, $E_2/E_1 = 1/1.6$, $u_1 = u_2 = 0.5$).

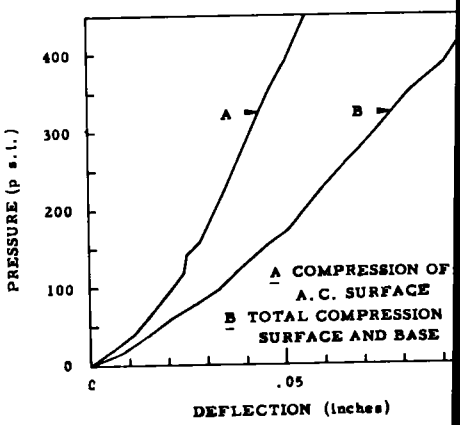

Figure 7. Compression of asphaltic concrete surface ($4\frac{1}{2}$ in.) and of limerock base (24 in.), 8-in. diameter plate (E_2, $E_1 = 1/1.6$).

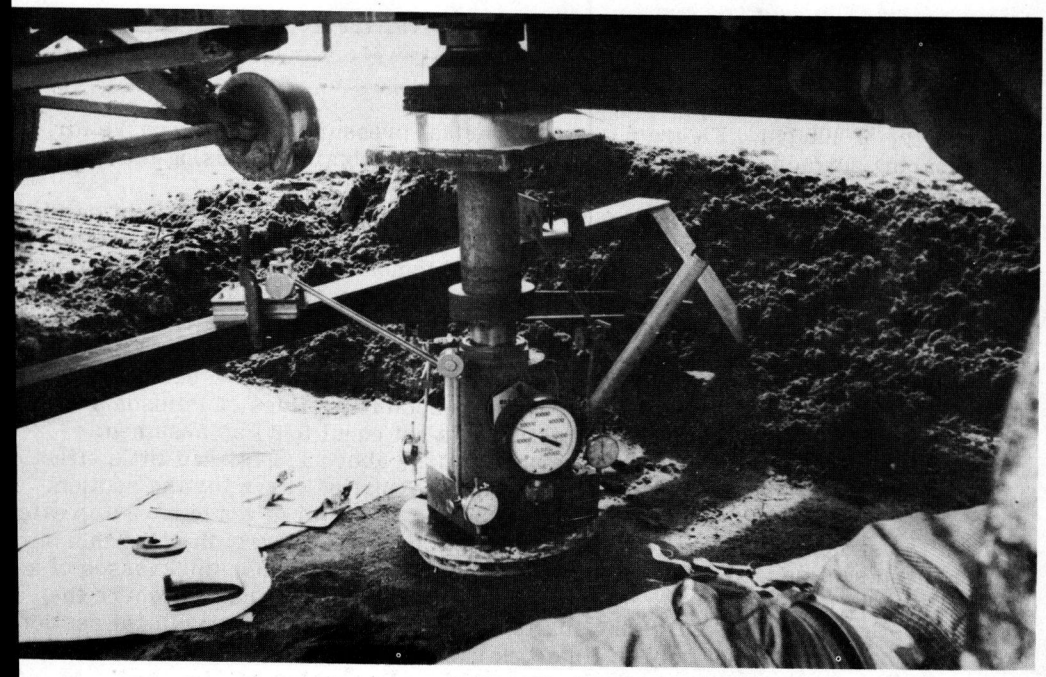

Figure 8.

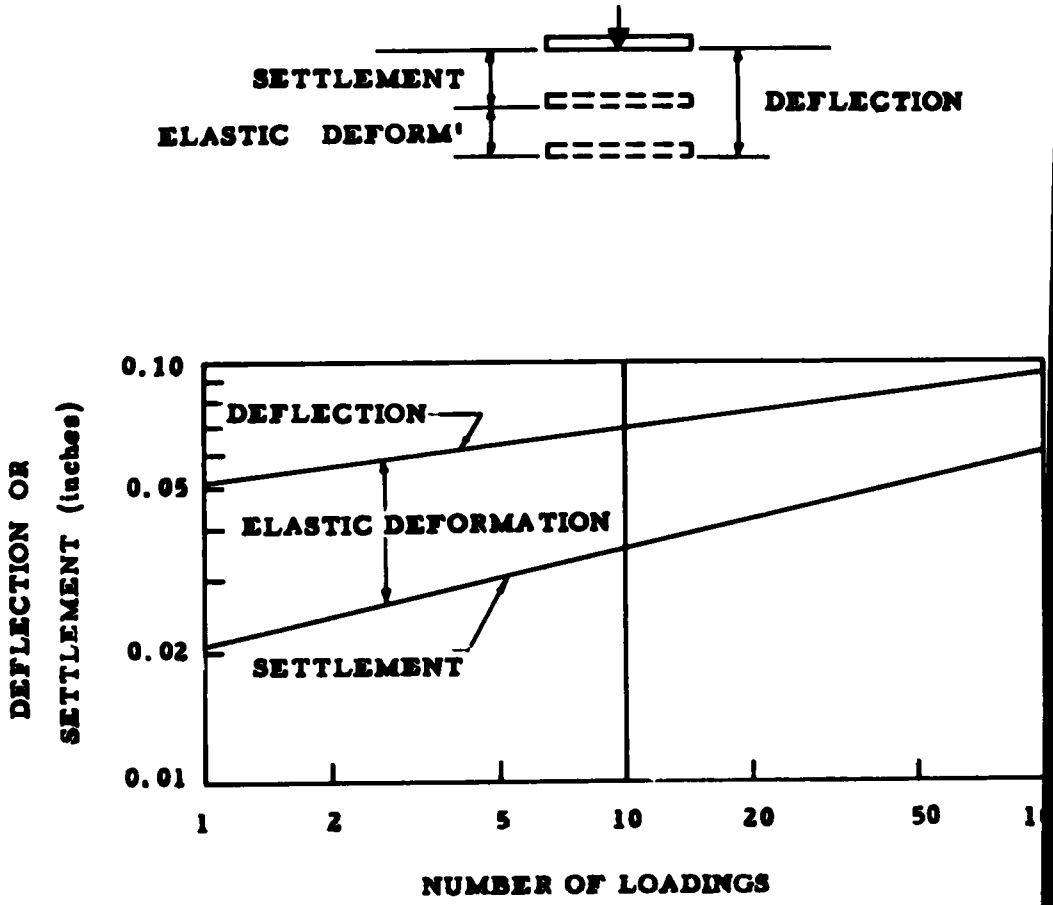

Figure 9. Increase in deflection and settlement with repetitions of a 12,000-lb load 8-in. diameter plate (8).

pressure, p, of 200 psi. Figure 6 shows the actual pressure deflection curve for 4. in. of wearing surface as well as the theoretical curve obtained by using the two-laye theory.

This study clearly indicated the need for precise measurements of deflection wher conducting this type of experiment. Accurate evaluation of the variables affecting th action of the layered system is also necessary to compare test results and theory. The theoretical computations were based on an estimate of the ratio of E_2/E_1 obtaine from modified CBR tests. This is at best only an estimate and more exact values of the modulus of asphaltic concrete are necessary before any definite conclusions can made with regard to the use of layered theory for predicting the real behavior of wea ing surfaces. Experimentation is also needed to establish values of Poisson's ratio, μ, for asphaltic concrete as in all probability μ is not equal to μ_2 as assumed.

An increase in the thickness of the surface course above 1.5 in. had little effect the slope of the straight line portion of the load deformation curve for the section. However, it is probable that the thicker wearing surface course would have a greate ultimate resistance and resist the shearing stress more effectively than the thin sur face and base section. Additional tests are necessary with thinner thicknesses of as phaltic concrete (0.75 in. to 2.5 in.). Additional tests are also needed where the range of E_2/E_1 is varied to cover the limits encountered on typical pavement section throughout the state and not only on limerock bases. Where three-layer systems ar encountered, analysis similar to those presented by Burmister will be used.

The action of the combined section of asphaltic concrete and limerock base was that of a layered system. Figure 7 shows the deflection both of the upper surface of the asphaltic concrete and of the surface of the limerock. Proportional amounts of deflection exist throughout the deformation range tested.

Measurements were also made of the surface deflections surrounding the 8-in. diameter plate when subjected to a pressure of 80 psi. The deflected surface was typical of a layered system and extended outward from the center of the load a distance of about four diameters. The deflection curve was almost parabolic and was similar to the surface deflection curves obtained when performing Benkelman beam tests on similar pavement sections.

Figure 8 shows the field test arrangement used to obtain the compression and deflection data for this study.

REPETITIONAL LOADS

One of the major problems is that of limiting the accumulated settlements associated with repetitional loads. Extensive studies have been made and discussed by McLeod) on the effects of repetitional loads on settlements. As part of one of the bearing ate investigations a preliminary testing program was completed where 30 repetitions load (stress = 234 psi) were applied to a pavement section consisting of 4.5 in. of sphaltic concrete over 24 in. of limerock base. The results are shown in Figure 9. he findings agreed with those obtained by McLeod. The relationships of deflection, ttlement and elastic deformation are summarized in this figure for an 8-in. diameter ate.

The extrapolation of the curves beyond the 30 repetitions to 100 appears to be justied. Extrapolation beyond this range into the higher numbers of repetitions cannot be ade or justified at this time. Additional tests must be made in the range of 1,000 d 10,000 repetitions to establish the settlement relationships. Plans for building petitional load testing equipment have been made and tests should be initiated in 1961. The repetitional load equipment will permit evaluation of accumulated settlement der repetitional loads as well as the effective soil modulus. The use of repetitional d information along with layer system analysis should lead to a more realistic meth- of analysis of flexible pavements.

ACKNOWLEDGMENTS

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