

Plate Bearing Tests for Design of Flexible and Rigid Pavements

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SYNOPSIS

This paper presents the results of a study of pavement design based on the plate bearing test. The report supplements previous papers on pavement design, particularly the study reported in 1952 on the use of the triaxial test for design of flexible pavements.

A series of tests was conducted on a test section of the Joint Highway Research Project Test Road. These tests were conducted by loading a 30 inch diameter circular plate on various depths of compacted gravel to evaluate the effect of depth of base on subgrade reaction.

The data obtained from the testing program indicated that the variation of subgrade reaction with depth of base and as determined by the testing program was nearly the same as that determined by the mathematical analysis proposed by Prof. D. M. Burmister of Columbia University. It was indicated that even relatively shallow depths of base course greatly improve subgrade supporting capacities.

The effects of density of reinforcing layer were studied and evaluated. It was concluded that the density of the granular material is important in that increases in compaction result in greatly increased subgrade support.

It was concluded that the plate bearing test offers a good means for evaluating pavements. However, the need for further research before this design procedure can be adopted for pavement design in Indiana was indicated. Among the factors that must be evaluated are the effect of repeated loads, and climate.

PLATE BEARING TESTS FOR DESIGN OF FLEXIBLE AND RIGID PAVEMENTS

INTRODUCTION

During the summer and fall of 1951 a test section was constructed at the Joint Highway Research Project Test Road in connection with a special research project. This test area was built up of gravel to an

average depth of 36 inches. The gravel was placed in six lifts, with each layer of material compacted by means of a sheep's foot, as well as a smooth wheeled roller.

A short series of special tests was made at the time of construction of the test section; however, other than changes in physical state due to climatic variations, a large portion of the gravel was not disturbed up to the time of the plate-bearing testing program.

The gravel which was used for construction of the test site consisted of relatively well graded material containing on the average slightly more than 10 per cent finer than a No. 200 mesh sieve. The maximum size aggregate was one inch.

The first step in the construction of the granular base was to clear the top soil from the test site to a depth of about six inches. This was done by means of a motor grader. After stripping, the subgrade was lightly rolled with a small five ton roller. The natural soil at the site is designated pedologically as Crosby soil and consists of about six inches of "A" horizon resting on a more plastic "B" horizon. The soil immediately under the granular base after the stripping operation was, therefore, the "B" horizon having a liquid limit of 31 per cent and a plasticity index of 14 per cent.

The test site was approximately 50 by 180 feet. Varying compactive effects were used during construction in an attempt to obtain three different density conditions of the base material. During construction, field C. B. R. tests were made on the gravel at various depths and for each density condition. Tables 1 and 2 show the results of the tests made during the construction period. The average of the dry densities of the gravel varied from 137.2 lb./cu. ft. for the high density section at the west end to 130.1 lb./cu. ft. for the low density at the east end. The average intermediate density was 134.1 lb./cu. ft. The field C. B. R. varied from 61 to 7 per cent.

The results of the tests made on the subgrade are shown in Table 2. The field C. B. R. varied to some extent with depth, ranging from 4 to 8 per cent. It can be seen that the dry densities were 100.8 lb./cu. ft. at the surface (immediately under the base) and 92.6 lb./cu. ft. at a depth of twelve inches.

PURPOSE OF PLATE-BEARING TESTS

As previously mentioned, the major portion of the test site was not disturbed during the original testing program. It was felt, therefore, that the granular base offered an excellent opportunity to study the effect of density and depth of base on the structural capacity of the base as measured by plate-bearing tests. It was therefore decided

TABLE 1
Summary of Tests on Gravel Base as Constructed During September 1951

Depth*	West End			Center			East End				
	Density (#/cu.ft.)	Field CBR (%)	w (%)	Depth* (in)	Density (#/cu.ft.)	Field CBR (%)	w (%)	Depth* (in)	Density (#/cu.ft.)	Field CBR (%)	w (%)
12	136.0	4.6	6	129.2	2.8	6	129.4	5.1
18	137.6	4.5	12	135.8	6.2	18	129.9	3.4
24	138.0	15	4.5	18	137.0	3.7	24	131.5	3.8
36	61	24	133.8	13	6.4	30	131.0	5.7
				30	134.9	4.2	36	129.0	7
Average	137.2	4.2		134.1	4.6		130.1	4.5

* Depth of gravel above top of subgrade

to perform a series of tests during various seasons of the year to evaluate the effect of seasonal variations on structural capacity. However, it was found that in order to perform just one series of plate tests, it was necessary to disturb a large area making more than one series of tests impractical. As a result, a testing program was set up to include the following variables.

TABLE 2
Summary of Tests on Subgrade as Constructed During September 1951

Depth	Density	Field CBR	w
(in)	(#/cu.ft.)	(%)	(%)
24	4	20
12	92.6	4	20
surf.	100.8	8	17

1. Effect of depth on the subgrade reaction "k"
2. Effect of surcharge on the subgrade reaction "k"
3. Effect of density on the subgrade reaction "k"

In addition to studying the above variables, it was a principal objective of the program to study the applicability of the Burmister method for estimating the effect of depth of base on subgrade support (1).

PROCEDURES

Procedures adopted for the testing program were patterned after those recommended by the Corps of Engineers for determination of the "k" value for soils (2). For this procedure a series of stacked plates was used with a thirty inch diameter plate in contact with the material to be tested. Reaction for the load on the plate was supplied by means of the test truck operated by the Bureau of Materials and Tests of the Indiana Highway Department (see Figure 1).

For the first series of tests (those with surcharge), the base material was excavated to the desired depth but to only sufficient width to



Fig. 1. General view of test truck.

permit seating of the plates. For series No. 2 and No. 3 (those without surcharge), the base was excavated to the desired depth, but in this case the open excavation was approximately 12 feet in diameter.

After excavation, the first step was to level the test surface by means of a flat shovel and then to place a layer of sand to a depth of about $\frac{1}{4}$ inch as shown in Figure 2. The purpose of the sand layer was to facilitate seating of the plate.

After the plates were in place, firmly seated, and level, three deflection dials were placed at the outer rim of the 30-inch diameter plate. These dials were securely fastened to a steel beam which rested on small blocks well outside of the test pit (see Figure 3).

A seating load of one pound per square inch was first applied in 30 seconds and then released immediately, after which initial dial readings were recorded. A load of 7,070 pounds (10 psi) was then applied in 10 seconds and maintained until the rate of deformation decreased to less than 0.002 inches per minute.

Next, a load of 14,140 pounds, or 20 psi, was applied and maintained until the rate of deformation decreased to the desired amount. Additional loads in increments of 7,070 pounds were applied by the above procedure. The maximum load applied in each case was 35,350 pounds, or that which resulted in 0.2 inch deflection, whichever occurred first. After the maximum load was applied, the load was released in increments of 7,070 pounds.



Fig. 2. Placing sand layer on base in preparation to seating the plate.

Grain size distribution, in place density, and moisture content, determinations, using standard procedures (3), were made on the material immediately beside the outer rim of the plate after the completion of each plate test.

RESULTS

A summary of the test results obtained during this testing program is shown in Figure 4, as well as in Tables 3 and 4. Test series No. 1 was performed at the western end of the test section on the high density base, series No. 2 on the intermediate density section in the center, while series No. 3 was performed at the eastern end of the test site on the low density gravel.

Effect of Depth of Base

Figure 4 shows the complete load-rebound curves for each test. It can be seen that the depth of base materially affected the results in that (except for series No. 1) an increase in depth resulted in increased bearing. However, tests made at the surface of the gravel in series

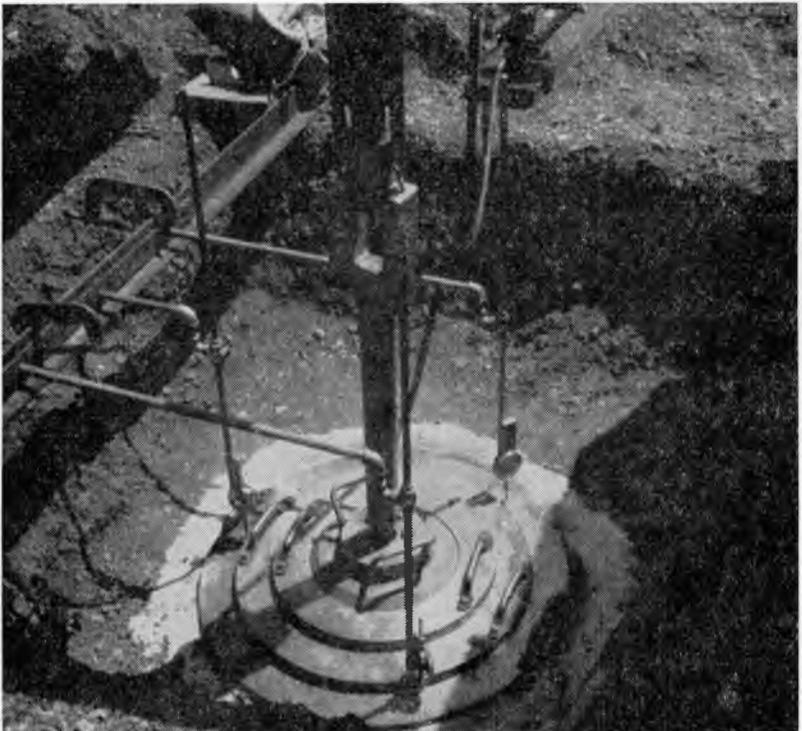


Fig. 3. Deflection dial and plate arrangement.

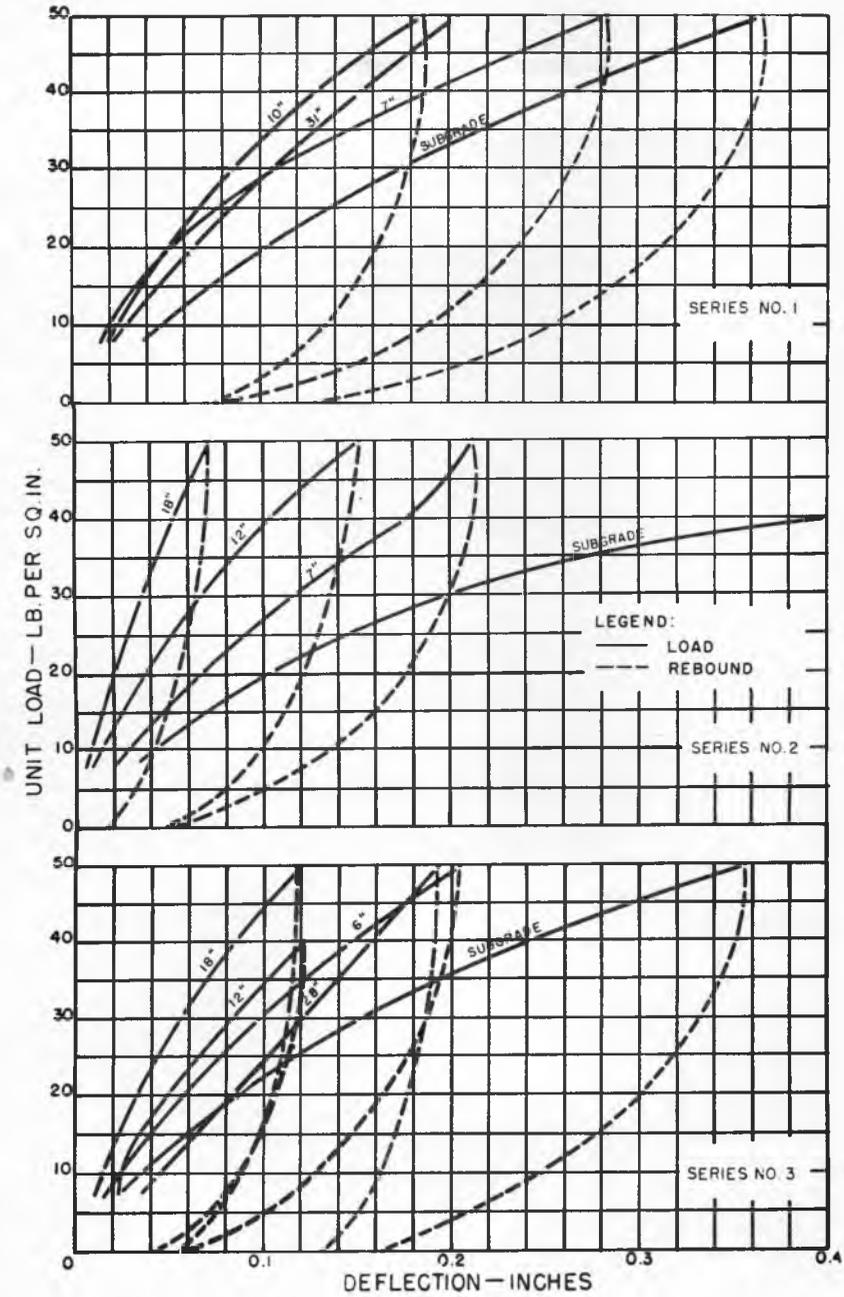


Fig. 4. Load-rebound curves, Series No. 1, 2, and 3.

TABLE 3
Summary of Tests on Gravel Base During May 1954

		FIELD TESTS										GRAIN SIZE (Finer Than)					
Section	Test No.	Depth* (in)	Density (#/cu.ft.)	w (%)	k @ 10 psi (#/cu.in.)	1" (%)	½" (%)	#4 (%)	#40 (%)	#100 (%)	#200 (%)						
West End Series #1	4	7	140.9	4.6	525	100	83.5	63.0	19.6	13.9	12.4						
	2	10	145.6	4.4	435	100	84.2	61.7	17.5	13.5	11.8						
	1	31	134.4	4.3	370	100	76.9	53.6	21.3	15.5	13.8						
	subgrade	285												
Center Series #2	5	7	134.2	4.9	389	100	83.7	61.1	18.2	13.4	11.6						
	12	12	141.2	4.0	590	100	84.7	62.0	16.3	11.6	9.0						
	6	18	132.1	4.8	1250	100	87.7	61.6	25.4	17.8	15.4						
	subgrade	225												
East End Series #3	10	6	135.5	5.3	415	100	83.3	60.8	18.3	12.8	11.6						
	13	12	127.5	5.8	415	100	84.4	61.2	15.7	10.1	8.6						
	8	18	126.5	5.9	715	100	79.8	59.1	30.7	26.1	24.5						
	9	28	124.0	4.1	225	100	85.9	63.3	26.6	19.1	16.5						
subgrade	310													

* Depth of base above subgrade

TABLE 4
Summary of Tests on Subgrade During May 1951

Field Tests		Limits		Grain Size (Finer Than)		
Depth	W	L.L.	P.L.	#4	#40	#200
(in)	(%)	(%)	(%)	(%)	(%)	(%)
36"	23	31	17	100	98	96
24"	24	32	20	100	99	98
12"	24	43	20	100	97	91
surf.	21	33	19	100	97	92

No. 1 and No. 3 showed extremely low bearing values. This is believed to be due to lower densities at the surface which resulted from freezing-and-thawing and other climatic variations. The dry densities at the surface (series No. 1, 31 inch depth, and series No. 3, 28 inch depth) were about eleven pounds per cubic foot less than the maximum recorded for each section.

It is of interest to note that the moisture content of the gravel base did not change appreciably from that during construction. The average moisture content of the gravel at time of construction at the west end was 4.2 per cent compared to 4.4 per cent 2½ years later at the time of the plate testing program. The values for the center section were 4.6 per cent in both cases, while for the low density section the test values were 4.5 per cent and 5.3 per cent.

In the far right hand column of Table 3 it is seen that the gravel contained from 8.6 to 24.5 per cent finer than a No. 200 mesh sieve. The modulus of subgrade reaction varied from 225 lb./cu. in. to 1,250 lb./cu. in. for the gravel. The modulus for the subgrade varied from 225 lbs./cu. in. to 310 lb./cu. in.

Effect of Surcharge

In Table 3, series No. 1, it will be noted that as the depth of base increased the modulus of subgrade reaction decreased. This is believed due to the surcharge effect of the unexcavated material near the area of the plate. This effect, however, was noticeable only at low deflections since the curves for the seven and ten inch depth were reversed for deflections greater than 0.05 inches. It should be pointed out also that the seven inch depth test in series No. 1 (surcharged) showed lower bearing at high deflection than a comparable depth in series No. 2.

Effect of Density of Base

The effect of the dry density on the bearing values of the granular base material is apparent in Figure 4 as well as Table 3. The data are not consistent regarding this factor and are also influenced by the subgrade underlying the gravel. The reaction "k" for the subgrade was just 225 lb./cu. in. for series No. 2 compared to 310 lb./cu. in. for series No. 3. Nevertheless, in series No. 2 the "k" values for the 12 inch and 18 inch depths were considerably higher than in series No. 3. For the lesser depth gravel series No. 2 showed a lower "k" value than did series No. 3.

Data appearing in Table 3 illustrate a significant principle. For relatively thin bases the bearing values are influenced to a large degree by the underlying soil; however, as the depth of base increases the quality of the base plays a more prominent role.

THEORETICAL CONSIDERATIONS

Methods have been available for several years for calculating stresses in earth masses which result from various types of loads at the surface. By and large these methods have been developed from the basic Boussinesq equations and depend upon Poisson's ratio and the modulus of elasticity of the soil, and assume a single layer system (4, 5). For a uniformly loaded flexible plate, the Boussinesq equation takes the following form:

$$S_L = \frac{prF}{E} \dots \dots \dots (1)$$

- Where: S_L = Settlement of plate in inches
 p = Unit load on the plate in pounds per square inch
 r = Radius of the plate in inches
 E = Modulus of elasticity of the soil in pounds per square inch
 F = Settlement factor, dimensionless, which depends on the ratio of the depth (h) considered, to the radius of the plate r .

The above expression has been utilized by the Kansas Highway Department in the design of flexible pavements (6). In their method use is made of the triaxial test for evaluating the modulus of elasticity, or the modulus of deformation, as it is sometimes called. One distinct disadvantage of equation No. 1 is that it is assumed that the subgrade consists of a single layer of homogeneous, isotropic material. In addition, the settlement is for a soil mass existing at a depth h , and downward to infinity. It follows then that the material from the surface to a depth h is incompressible or that its compression must be neglected.

Where h/r is equal to zero, F becomes 1.5 in equation No. 1

$$S_L = 1.5 \frac{pr}{E} \dots \dots \dots (2)$$

In this case, the settlement is given for a material of infinite depth and for a circular load at the surface.

In pavement design the engineer deals with a layered system which consists of the pavement, and/or pavement and base, as well as the subgrade soil. For this case Burmister has developed a solution for the two layer system (1).

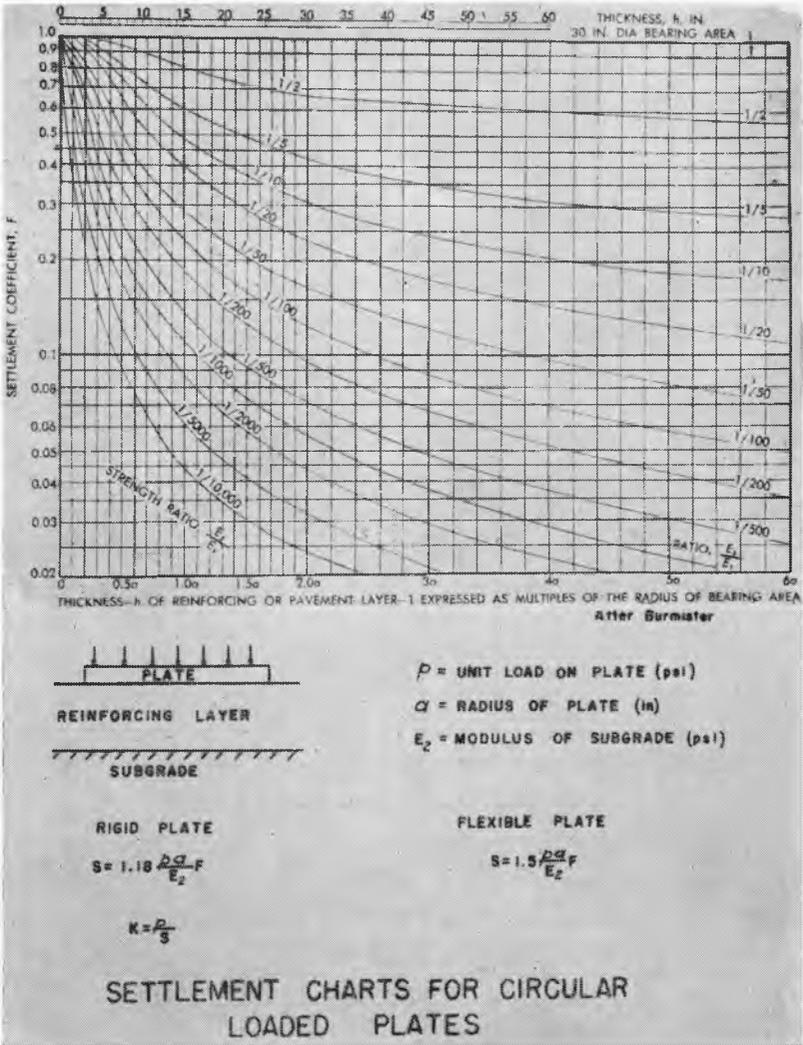


Fig. 5. Settlement charts for circular loaded plates.

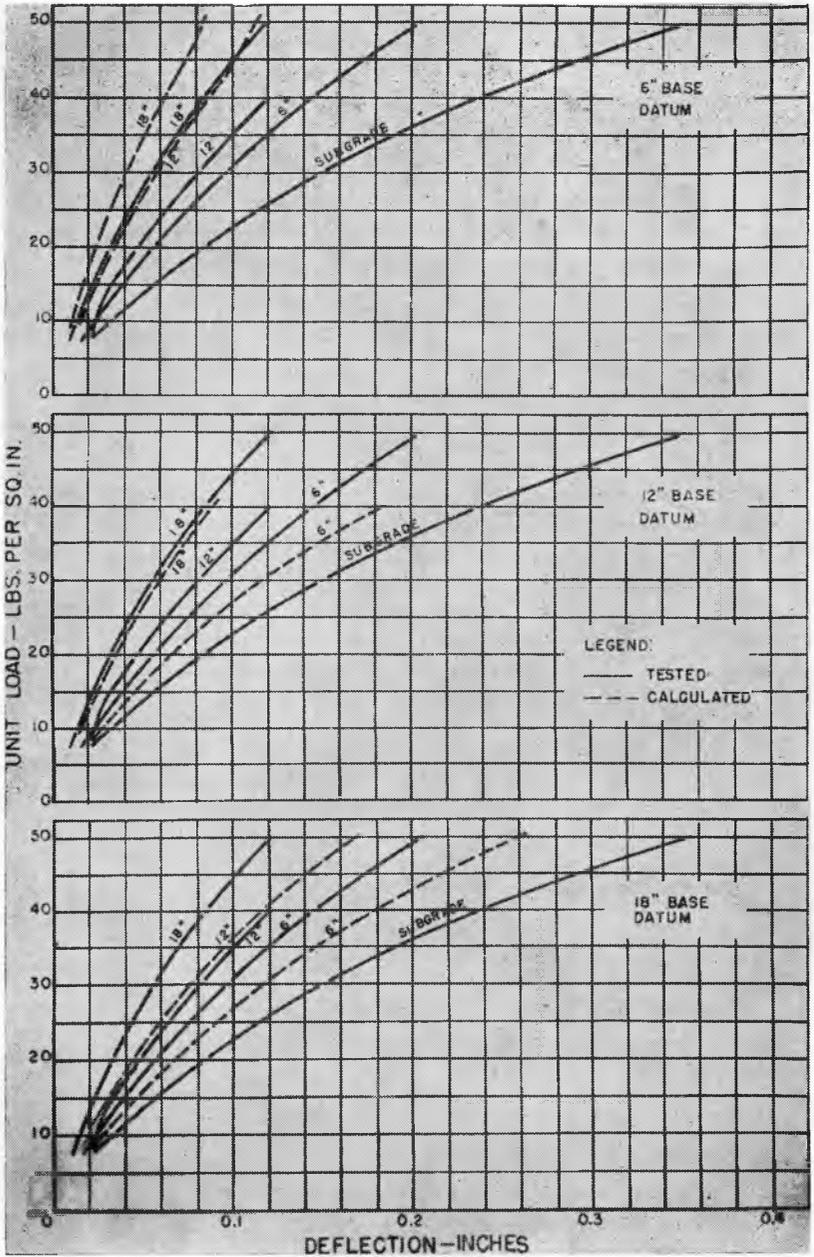


Fig. 6. Comparison of tested and calculated load-deflection curves, Series No. 3.

In his analysis Burmister makes the following assumptions:

1. Both layers are homogeneous, isotropic, elastic, and obey Hoake's Law.
2. The reinforcing layer is assumed to be weightless, infinite in extent in the horizontal direction but of finite depth.
3. The surface of the reinforcing layer is free of normal and shearing stresses outside the loaded area.
4. The stresses in the lower layer at infinite depth are zero.
5. The two layers are in constant contact and act together as an elastic medium.

The basic settlement equations then take the following forms:

$$\text{For flexible plates: } S_L = 1.5 \frac{pr F_2}{E_2} \dots \dots \dots (3)$$

$$\text{For rigid plates: } S_L = 1.18 \frac{pr F_2}{E_2} \dots \dots \dots (4)$$

In the above expression E_2 is the modulus of elasticity of the subgrade, p and r are the contact pressure and radius of the loaded area respectively, and F_2 is a dimensionless factor depending on the depth of reinforcing layer, radius of the plate, and the moduli of elasticity of the subgrade and reinforcing layer. Figure 5 contains influence curves for this settlement factor F_2 .

To utilize this method of analysis it is necessary to perform a test on the subgrade to evaluate the modulus of elasticity of the subgrade E_2 , and also to perform a test on any depth of gravel to evaluate E_1 . From this, the unit load which any depth base of the same type will support is calculated using equation No. 4. Generally, a six inch base is used to evaluate E_1 , but a twelve inch base is preferred (7).

Calculated Load-Deflection Curves

Figure 6 shows comparisons of tested and calculated load-deflection curves for series No. 3. The calculated curves were developed using procedures which are illustrated in the appendix. These curves were developed from those of the subgrade and one depth of base. For example, in Figure 6 the upper set of curves are based on the tested values for the subgrade and six inch depth base. In this case the twelve and eighteen inch depth curves were calculated. In the center set of curves the subgrade and twelve inch base were used as the basis for the calculations. The lower curves show the data using the subgrade and eighteen inch base course data.

Observation of these curves reveals that the calculated curves follow the same general pattern as those developed from actual tests. In every case the curves which were developed using the Burmister

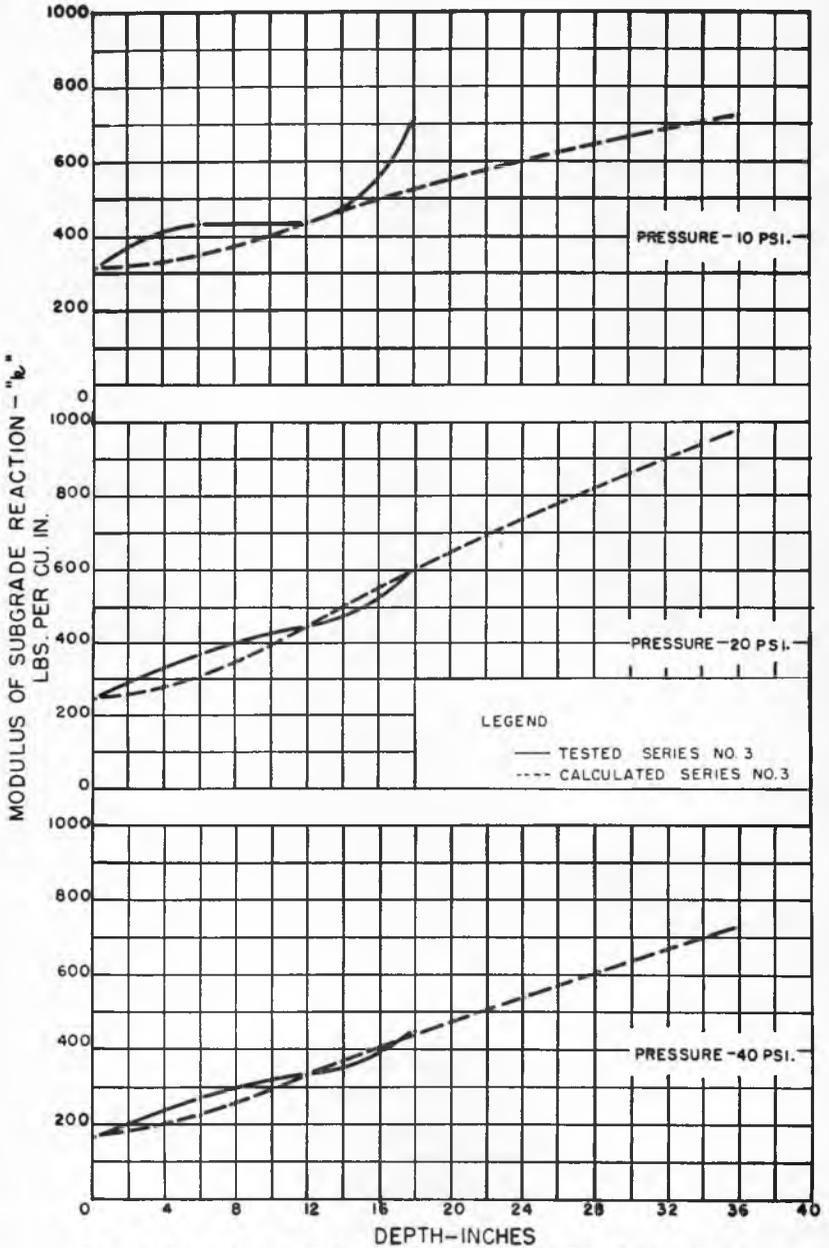


Fig. 7. Comparison of tested and calculated "k", Series No. 3.

procedure resembled the curves which were used as the base curves to a large degree. Furthermore, in several cases the calculated data were for all practical purposes nearly the same as that determined by test. The data indicated that use of reinforcing layers in excess of six inches are preferable for evaluating E_1 . The twelve inch depth appeared to be best for this particular series of tests.

Effect of Depth of Base on Subgrade Reaction

In Figure 7 is shown the variation of modulus of subgrade reaction "k" as determined by test as well as by the theoretical procedures. For sake of simplicity the data shown are for series No. 3 only and for a plate pressure of 10, 20, and 40 pounds per square inch. The solid curves are those determined by test while the broken curves were determined by calculation. In each case the twelve inch depth gravel test was used as the basis of the calculations.

In general, the calculated values are consistent with the tested values. The exception to this in Figure 7 is the calculated curves at 10 psi. In this particular case the discrepancy is probably due to irregularities in the field test made on the twelve inch base. The remaining data agree very closely with the tested values.

Figure 8 shows the effect of depth of base course, and quality of base course on the subgrade reaction "k" for an assumed E_2 value of 1,500 psi. These curves illustrate that for high quality bases, the

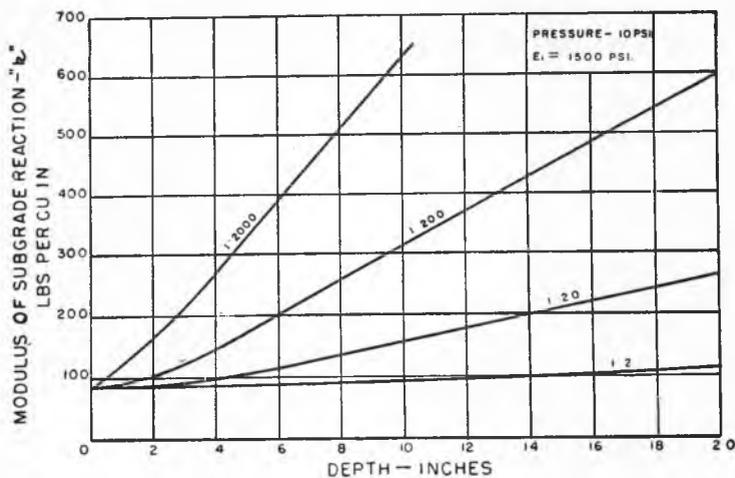


Fig. 8. Variation of calculated "k" with depth for various values of $E_1 : E_2$

resulting effect of depth is quite pronounced. In cases where bases of low E are used increased depth has little or no effect on the subgrade reaction.

Comparison of Laboratory and Field Tests

During the construction of the test sections a controlled series of laboratory C.B.R. tests were made. These data are shown in Figure 9 along with the field C.B.R. and plate loading data. Tests made on the subgrade are not included in this figure. It is evident that for the gravel used in the test section, the laboratory C.B.R. values are much higher than the field values for comparable densities. This is no doubt due to the confining effect of the C.B.R. mold.

In addition to the above, it is apparent that no direct correlation exists between C.B.R. (either laboratory or field) and modulus of subgrade reaction on the basis of density. This is not surprising inasmuch as the plate load test is greatly affected by depth of base while the C.B.R. test is not. Further, the plate test is not greatly affected by small surface irregularities while the C.B.R. test is greatly affected by the condition of the surface.

APPLICATION OF ANALYSIS TO DESIGN OF PAVEMENTS

Data of the type developed in this paper have been used by the U. S. Department of the Navy in the design of flexible pavements for several years (7). In this method of design the Burmister solution is used to estimate the required thickness of flexible pavement to with-

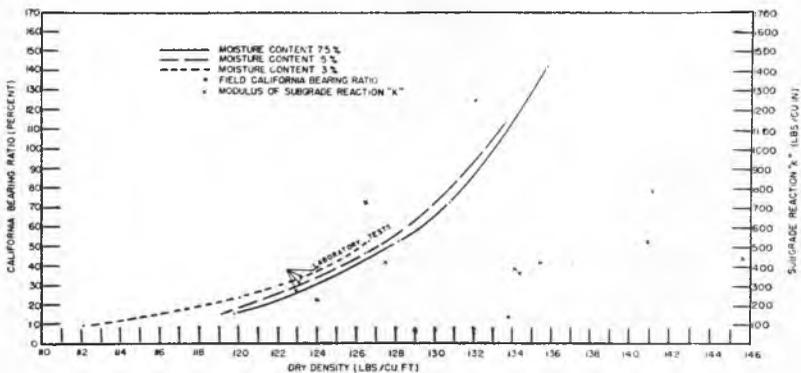


Fig. 9. Variation of California bearing ratio and subgrade reaction with density.

stand a unit pressure equal to that of the contact pressure between air-plane tires and the pavement surface.

The close agreement between the calculated values and those obtained by test suggest further that this method of analysis could be used to good advantage in flexible pavement evaluation. This method would make it possible to extrapolate data obtained in performance surveys to other pavements in similar situations but of different depths.

Several inherent disadvantages of the plate bearing test have been recognized for many years by engineers. Among these can be included the high initial cost of the equipment itself. Also, the cost of performing the test from the standpoint of time alone can make the test prohibitive, especially for routine design work.

For the design phase of flexible pavements, the Department of the Navy uses the theoretical analysis only for the purpose of estimating the thickness of the required pavement. A test is first made on the subgrade and on a small test section of the reinforcing layer from which estimates of E_1 and E_2 are made. Equation No. 3 is then used to calculate the required pavement thickness. After a reasonable estimate of the pavement thickness is made, three test sections are built, the first two-thirds of the theoretical thickness, the second 1.5 times the theoretical thickness and the third section of the estimated theoretical thickness. Plate tests are then made on each of these sections, and the design thickness is that which will result in 0.02 inches deflection under the design load and tire pressure.

In both the Navy and Corps of Engineers procedures corrections are applied to the test data to account for subsequent loss in strength due to saturation of the subgrade. For the Navy procedure unconfined compression tests are made on subgrade samples, compacted at optimum moisture content, and at two per cent greater than optimum moisture content. The deflections are then corrected by the ratio of the two strength values.

$$\text{Corrected } S_L = \frac{P_1}{P_2} (S_L) \dots \dots \dots (5)$$

P_1 = Unconfined strength at optimum

P_2 = Unconfined strength at optimum plus 2%

S_L = Plate settlement

The correction factor used by the Corps of Engineers is obtained by testing two samples of soil in the consolidometer. The first specimen is an undisturbed sample at field moisture content, while the other

sample is a duplicate of the first but saturated. The correction then is as follows:

$$k_u = k_f \frac{(S_1)}{S_2} \dots\dots\dots (6)$$

- k_u = Corrected modulus of subgrade reaction
- k_f = Field test modulus of subgrade reaction
- S_1 = Deflection in consolidometer at field moisture
- S_2 = Deflection in consolidometer after saturation

In both of the above cases the size of test plates is considered. For the Navy procedure a 30-inch diameter plate is used for the original evaluation of E_1 and E_2 , and then a plate with a radius equal to an equivalent circle of tire contact is used while testing the three test sections. The radius of contact is calculated by means of equation 7.

$$r = \sqrt{\frac{P}{p\pi}} \dots\dots\dots (7)$$

- r = Radius of equivalent circle of contact
- P = Gross wheel load
- p = Contact pressure

For determination of "k" a 30-inch diameter plate is always used by both of the above agencies since the size of the loaded area will greatly affect the resulting deflection. For a given deflection a plate will sustain a unit pressure as given in equation No. 8.

$$S = n + \frac{mP}{A} \dots\dots\dots (8)$$

- S = Unit stress
- P = Perimeter of the plate
- A = Area of the plate

m and n are constants depending on the soil or base course. Values of m and n have been established for several types of materials by McLeod (9) and others.

Included in the appendix are sample calculations which suggest means by which the Burmister equations may be used for the design of both flexible and rigid pavements. For these illustrations it was assumed that the modulus of elasticity of the subgrade, E_2 , was 1,500 psi. It was further assumed that a dual wheel truck load of 10,000 pounds was to be investigated. For these conditions the calculations indicated 16 inches of base for a flexible pavement, while for a 9 inch uniform concrete pavement a base of 4 inches was indicated.

In the case of the flexible pavement mentioned above, the calculations are similar in most respects to those recommended in the Navy manual (7). For the concrete pavement use was made of Figure 8,

which was developed in this paper. The use of this procedure for concrete pavements may appear to be unjustified to the casual reader since concrete pavement design charts, generally, assume the subgrade to act as a viscous liquid. On the other hand, the settlement equations are based on the theory of elasticity. It is pertinent, however, that Figure 8 merely indicates the change in subgrade reaction which results when a base course is added to the subgrade.

The analysis presented in this paper is not necessarily dependent on the performance of plate-bearing tests. Evaluation of the elastic properties of the subgrade and reinforcing layers can be accomplished by means of triaxial test.

ACKNOWLEDGMENTS

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APPENDIX

SAMPLE CALCULATIONS

Calculation of Load-Deflection Curves

To illustrate the use of the settlement expressions, (equations 3 and 4), assume that the unit load on a rigid 30 inch diameter plate, resting on the subgrade is 30 psi and that the resulting deflection is 0.2 inch. Assume in addition that when the test is repeated on a six inch base course constructed over the same subgrade the unit load at 0.2 inch deflection is 45 psi.

From equation 4 the modulus of elasticity of the subgrade is calculated.

$$S_L = 1.18 \frac{pr}{E_2} F_2 \dots \dots \dots (4)$$

$$0.2 = 1.18 \frac{(30)(15)}{E_2}$$

$$E_2 = \frac{1.18 (30) (15)}{0.2}$$

$$E_2 = 2,670 \text{ \#/sq. in.}$$

For the six inch depth base

$$0.2 = 1.18 \frac{((45)(15))}{2,670} F_2$$

$$F_2 = 0.67$$

Therefore, in Figure 5, for F_2 equal 0.67 and r/h equal to $\frac{15}{6}$ or 2.5, the ratio of E_2 to E_1 is 1 to 35.

If it is desired to calculate the unit pressure a 30 inch diameter plate will withstand at 0.2 inch deflection, but on 24 inches of gravel base, the ratio of $\frac{r}{h}$ is now $\frac{15}{24}$ or 0.625. For this condition the settlement factor is 0.24. Thus, the unit pressure is calculated as follows:

$$0.2 = 1.18 \frac{((p)(15))}{2,670} 0.24$$

$$p = 125 \text{ psi}$$

It should be noted that in general the unknowns in this expression are the deflection S_L , the unit load p , and the settlement factor. Therefore, any two values may be assumed and the solution made for the remaining unknown. In the usual case the limiting deflection is assumed and the design unit pressure is known. This leaves the settlement factor F_2 as the only unknown. Then, after F_2 is determined, the depth of reinforcing layer to sustain the design pressure at the limiting deflection can be determined.

Calculations for Pavement Design

Assume that the modulus of elasticity of the subgrade was determined by a plate load test to equal 1,500 psi and that $E_1/E_2 = 20$.

Flexible Pavement

Calculate the depth of base to withstand a 10,000# dual wheel load with radius of contact area equal to 8 inches (see ref. 8) and at a limiting deflection equal to 0.1 inch. For this case use can be made of equation 3 for a flexible plate.

$$S_L = 1.5 \frac{((p)(r))}{E_2} F_2 \dots\dots\dots(3)$$

$$0.1 = 1.5 \frac{((50)(8))}{1,500} F_2$$

$$F_2 = 0.25$$

From Figure 5, $r/h = 0.50$

$$\text{Therefore } h = \frac{8}{0.50} = 16''$$

Rigid Pavement

Calculate the depth of base to withstand a 10,000# dual wheel load with radius of contact area equal to 8 inches, and assume that the concrete pavement is 9 inches uniform depth with a modulus of rupture of 550 psi.

From Figure 5 of reference No. 8 and using a factor of safety of 2 for the working concrete flexural stress the required modulus of subgrade reaction is equal to 100#/cu. in. From Figure 8 the required base depth is then equal to slightly more than 4 inches.

It should be emphasized that in these examples, test values were assumed merely for illustration purposes. Furthermore, this method deals only with structural capacity of the pavement base structure. Likewise, the important factor of repetition of load is completely neglected. In the case of the flexible pavement the structural capacity of the wearing surface was also neglected. In addition no corrections were applied to the test values to account for saturation or frost action.