DEPARTMENT OF SOILS

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TRIAXIAL TESTING APPLIED TO DESIGN OF FLEXIBLE PAVEMENTS

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SYNOPSIS

The triaxial-compression test can be used to determine the fundamental strength characteristics of materials used in the construction of flexible pavements. By determining the strength properties of surface, base course, subbase, and subgrade materials by this means, an opportunity is available to utilize these materials on a basis of resistance to strain and shear, comparable to the methods used for other structural materials, such as steel, concrete, and timber.

The purpose of this research project was to investigate the possible use of the triaxial tests for flexible-pavement design in Indiana. A review was made of the methods of application of results of triaxial tests to the design of flexible pavements as developed by various organizations engaged in the construction of pavements. A detailed study was made of the methods adopted by the highway departments of Kansas and Texas.

The theoretical required thicknesses of pavement, as determined by the results of triaxial tests on five typical soils of the drift-covered area of Indiana and on several soil-aggregate mixtures, were obtained and comparisons of the two methods were made. The effects of compaction and rate of application of load during test on the theoretical required thicknesses of pavement were investigated.

The effects of the variables, surface texture (roundness or angularity of particles), density, and gradation, on strength characteristics of base course materials were investigated.

It was concluded that the triaxial test offers a good means of evaluating pavement design. However, the need for further research before this procedure is adopted for pavement design in Indiana was indicated. Among the factors that must be evaluated are those of traffic and climate.

● Since the advent of motor-vehicle travel in 1904, engineers and the general public have become more and more concerned with the development and maintenance of local, state, and national highway systems. The rapid increase in amount of motor-vehicle travel and the accompanying demand for highways resulted in passage of the Federal-Aid Road Act in 1916. This marked the beginning of an accelerated program of development of a network of highways. At the present time, there are 350,000 mi. of main rural roads over which there are 160 billion vehicle-miles of travel annually (*1*).

Accompanying the development of our highways has been the rapid growth of the motor-freight industry. In 1949, 31 percent of the vehicle-miles traveled on our highways was by freight-carrying vehicles, and 89 billion ton-miles of freight were moved (*1*).

Prior to 1930, practically all flexible pavements were of macadam-type construction, that is, they were constructed of broken stone without particular regard to gradation except for limitations on maximum top-size. The pavements usually consisted of two component parts: a base course, usually waterbound macadam, and a surface course, usually asphalt-penetrated macadam with a seal coat.

The total thickness of the pavement was commonly determined by the general practice in the locality or area in which the pavement was to be built. Usually 6 in. total thickness was considered adequate; however, the thickness in some instances was increased to as much as 10 in. or more, depending on the opinion of the engineer. At that time the volume of traffic and the frequency of heavy axle loads above 20,000 lb. were small compared to present conditions; therefore, pavement thicknesses determined in this manner were generally satisfactory.

From 1930 to 1940, the new field of soil mechanics developed rapidly, but the volume of traffic and the intensity of heavy wheel loads had not yet become particularly destructive to our highways, and consequently, research was directed more toward the improvement of subgrade conditions and the gradation and cohesive properties of the base and surface courses than toward the development of methods for determining the required thickness of pavement for given subgrade soils and traffic conditions.

Just prior to World War II, the number of heavy axle loads per thousand vehicles began to rise sharply. By 1942 the intensity of heavy axle loads had tripled, and by 1948, the total number of heavy axle loads, 20,000 lb. or more, was 18 times the number in the 1936–37 period. Also, by 1948 the total number of gross loads of 40,000 lb. or more was 12 times as great and the frequency was 7 times as great as in the 1936–37 period (*2*, *3*).

The destructive action of these heavy wheel loads on our highway pavements has been discussed many times. The problem is being attacked from three angles: legal limitation, vehicle design, and pavement design.

The study reported in this paper dealt with certain phases of pavement design: the development of a laboratory method for the evaluation of the strength characteristics of materials used in the component parts of flexible pavements and a study of several variables affecting pavement thickness.

DESIGN PRINCIPLES AND METHODS

One of the earliest methods of calculation of the thickness of flexible pavements is the so-called Massachusetts Rule, published in the 1901 report of the Massachusetts Highway Commission (*4*). This method assumed that the wheel loads were distributed from the pavement surface through the surface course and base course, shape of a cone, at an angle of 45 deg. From simple geometry, this resulted in the following formula:

$t = 0.564 \sqrt{p/q}$, in which

t = thickness of pavement in inches

p = wheel load in pounds

q = bearing strength of subgrade in lb. per sq. in. (psi.)

In this same report, the bearing values of various soils were given as ranging from 4 psi. for impervious soils at their worst, to 20 psi. for sand and gravel. The method of determination of these values was not given.

The fundamental principal of the Massachusetts Rule is as sound today as when it was developed 50 years ago. However, complete agreement has not yet been reached on the exact distribution of wheel loads on horizontal and vertical planes in surface course, base course, and subgrade. Neither has complete agreement been reached on methods of measuring the strength of these component parts of a flexible pavement, though all current design methods recognize the fact that, under a given wheel load, less strength is required to prevent detrimental deformation, or failure, as the depth below the surface of application of load is increased (*4*).

No major changes from the Massachusetts Rule for determining the thickness of flexible pavements appeared until 1938, when A. T. Goldbeck of the National Crushed Stone Association assumed the contact area of tires on the pavement to be elliptical in shape and used bearing plates to determine the values of subgrade strengths (*4*). The allowable deformation of the subgrade soil was taken to be ½ in. However, the resultant formula had the general form of the Massachusetts Rule.

An even greater variation from the Massachusetts Rule was presented in 1940 by E. S. Barber and L. A. Palmer (*4*). Their design formula made use of the Boussinesq formulas for determining the distribution of stresses resulting from wheel loads, and the triaxial compression test was utilized to determine the strengths of the component parts of the flexible pavement. Their formula is:

$t = \dfrac{a}{\sqrt[3]{C_p/C_t}} \sqrt{(P/Q)^2 - 1}$, in which

t = thickness of pavement (in.)

a = radius of circular loaded area, (in.)

C_p/C_s = ratio of stress-strain moduli of the pavement and subgrade

P = average pressure on loaded area (psi.)

Q = allowable bearing pressure on subgrade (psi.)

$= \dfrac{C_s d_s}{1.5a}$

d_s = allowable displacement of subgrade (in.)

The Boussinesq formulas expressing the stress components caused by a perpendicular, point, surface force at points within an elastic, isotropic, homogeneous, semi-infinite mass may be found in many standard reference books on soil mechanics or applied elasticity (*5*, *6*). They were first published in 1885, and are based on the mathematical theory of elasticity. The Boussinesq expressions for a point load were integrated over a circular area by A. E. H. Love (*7*) and the results were published in 1929. The respects in which these expressions depart from the true distribution of stresses, of course, depend upon the amount of variance which exists between the actual conditions and those assumed for the development of the formulas. Soil authorities are in general agreement that actual soils are not truly elastic, homogeneous, or isotropic. At the same time, the Boussinesq equations are commonly used for computing stress distribution in problems involving settlement of soils (*8*). While these values for pressure distribution are not exact, they are reasonably close estimates, particularly when strains are small and failure by shear is not imminent.

The Palmer-Barber formula makes use of an empirical relation to make allowance for the ability of relatively stiffer pavement to effect greater lateral distribution of load to the subgrade. This relation of stiffness factor, $\sqrt[3]{C_p/C_s}$, was proposed by K. Marguerre on the basis of the stiffness factor for slabs and was against the elastic displacement due to a point load on a two-layer system (*9*, *10*).

The problem of distribution of stresses in a two-layer system has been solved rigorously by Burmister (*10*) for two conditions: perfect continuity at the interface between two layers, and no friction at the interface. However, it has been found that considerable labor is involved, including triple integration, in the calculation of usable influence values (*11*). In the absence of a mathematical method of determining the distribution through layered systems that is feasible, the use of the Marguerre stiffness factor by Palmer and Barber may be justified on the basis of its simplicity and is most certainly a step in the right direction.

The Kansas State Highway Commission began investigation of the use of triaxial testing for design of flexible pavements in 1941 (*12*, *13*). After five years of development, Kansas adopted a procedure for triaxial testing of subgrade soils, base course materials, and bituminous mixtures. By the Kansas method, the required thickness of pavement is based on the stress-strain relationship of each component part of the pavement as determined by the triaxial test. One confining, or lateral, pressure of 20 psi. is used. It is considered by the state highway commission that 20 psi. is comparable to the lateral support, or horizontal resistance, which is normally provided by the adjacent similar material under field conditions. Their design thickness of flexible pavements is obtained by using a modification of the Palmer-Barber formula and is as follows:

$$T = \left[\sqrt{\left(\frac{3Pmn}{2\pi C_s}\right)^2 - a^2}\right]\left[\sqrt[3]{\frac{C}{C_p}}\right],$$

in which

T = thickness required (in.)
C_p = modulus of deformation of surface (psi.)
C = modulus of deformation of subgrade or subbase (psi.)
P = wheel load (lb.)
m = traffic coefficient
n = saturation coefficient
a = radius of area of tire contact (in.)
s = permitted deflection of surface (in.)

When a combination of several layers are desired:

$$t_b = (t - t_p)\sqrt[3]{\frac{C_p}{C_b}}, \text{ in which}$$

t_b = thickness of base (in.)
t = thickness of surface required on soil (in.)
t_p = thickness of surface desired in combination (in.)
C_p = modulus of deformation of surface (psi.)
C_b = modulus of deformation of base (psi.)

The modifications are a traffic coefficient based on volume of traffic, and a saturation coefficient based on amount of annual rainfall. It should be noted that these modifications are empirical, and represent an attempt to correlate theoretical design with variations in pavement performance due to volume of traffic and amount of rainfall.

The Texas Highway Department, in 1946, presented a method of flexible pavement design utilizing the triaxial compression test to determine the Mohr's rupture line of subgrade, subbase, and base course materials (*14*, *15*). The test procedure involves use of a pressure cell to apply the confining pressure. The Texas pressure cell is quite simple in design and is, therefore, more susceptible to field use than the usual types of triaxial testing devices.

From the triaxial-compression tests, the Texas Highway Department obtains stress-strain curves, values of internal friction and cohesion, and a Mohr's rupture line. In applying these results, the state highway department plots the Mohr's rupture line on a classification chart which then places the material in one of six classes. The thickness of pavement required and the suitability of the material for base course depends upon the classification of the material (Table 1). Thus, values from a rational test are used in connection with a classification chart based on a correlation of field performance with test data.

TABLE 1

TABLE FOR INTERPRETATION OF CLASSIFICATION SUBGRADE AND FLEXIBLE BASE MATERIAL (15)

Class of Material	General Description of Material	Depth of pavement (Base and Surfacing) Inches.					
		8000-lb. Wheel Load		12000-lb. Wheel Load		16000-lb. Wheel Load	
		High E[a] Base Course	Low E[a] Base Course	High E Base Course	Low E Base Course	High E Base Course	Low E Base Course
1	Good flexible base material	Good—Light bituminous surfacing acceptable.					
2	Fair flexible base material	One to four inches of bituminous surfacing or a stable layer of class 1 material covered with a good light surfacing.					
3	Borderline base and subbase materials	3–8	4–10	4–10	5–12	4–12	6–14
4	Fair to poor subgrade	8–13	10–16	10–15	12–20	12–18	14–23
5	Weak subgrade	13–17	16–21	16–21	20–26	18–24	23–30
6	Very weak subgrade	17+	21+	21+	26+	24+	30+

[a] E = Young's Modulus. In these computations High E was approximately 20,000-psi. and Low E was approximately 6,000-psi. The table is not strictly applicable to materials of considerably different characteristics. At stopsigns, additional base depth of two to four inches plus heavy surfacing is indicated.

PURPOSE AND SCOPE

From current engineering literature, it was noted that these later two state highway departments, Kansas (*12*, *13*) and Texas (*14*, *15*), were using the triaxial-compression test on the materials used in flexible pavements, and the required thickness of pavement was determined from these test results. It was the purpose of this project to make a detailed study of the procedures and equipment used by these two highway departments, and to make a study of the application of triaxial testing to base course materials by testing a series of soil-aggregate mixtures having surface texture, gradation, and density as variables. The end point was to determine the applicability of the triaxial test for pavement design in Indiana. The variation in the fundamental engineering properties of these mixtures, internal friction, cohesion, and stress-strain relationships, was determined,

This study was not directed toward the problem of frost action and spring breakup and no attempts were made to determine the required pavement thickness from the standpoint of those factors.

MATERIALS AND PROCEDURE

Base-Course Materials

The variable of surface texture was introduced by using an angular aggregate (crushed limestone) and a predominantly rounded aggregate (pit gravel with 32 percent of the particles having one or more appreciable crushed faces).

The variable of gradation was introduced by preparing mixtures of each material at the upper limit of coarseness, the medial gradation, and the lower limit of coarseness—based on AASHO specification B-1, Materials for Stabilized Base Course (AASHO Designation: M56–42). Figure 1 shows the gradation curves for the base course materials. Warsaw soil, A horizon, was used for the finer portions of the gravel mixtures, and limestone dust was used for the finer portions of the crushed stone mixtures.

The variable of density was introduced by molding one specimen of each of the above mixtures using a compactive effort equivalent to that required for approximately 97 percent standard Proctor density—designated as Type A compaction, and molding duplicate specimens to 100 percent standard Proctor density—designated as Type B compaction.

The base-course materials were tested using the pressure cell developed by the Texas Highway Department and shown in Figure 2. There is developed during this test an undetermined amount of friction between the sides of the specimen and the surface of the heavy rubber membrane of the pressure cell. However, the Texas pressure cell does not require machined parts and is, therefore, much less expensive than the conventional equipment used for the triaxial testing of specimens containing aggregates up to 1 in. in diameter. The specimens were molded in a split-mold using a mechanical compactor.

Each test specimen of base-course material was molded at its particular optimum moisture content. Free moisture was present in all the base-course materials at their respective optimum moisture contents.

Duplicate specimens of each type material, gradation, and compaction were tested in triaxial compression using 7, 14, and 20 psi. confining, or lateral, pressures respectively.

Subgrade Soils

Five subgrade soils, typical of the drift covered areas of Indiana were selected for tests. The laboratory numbers assigned to these soils, the textural types, and the locations from which they were sampled are listed below. The textures are based on U. S. Bureau of Chemistry and Soils Classification.

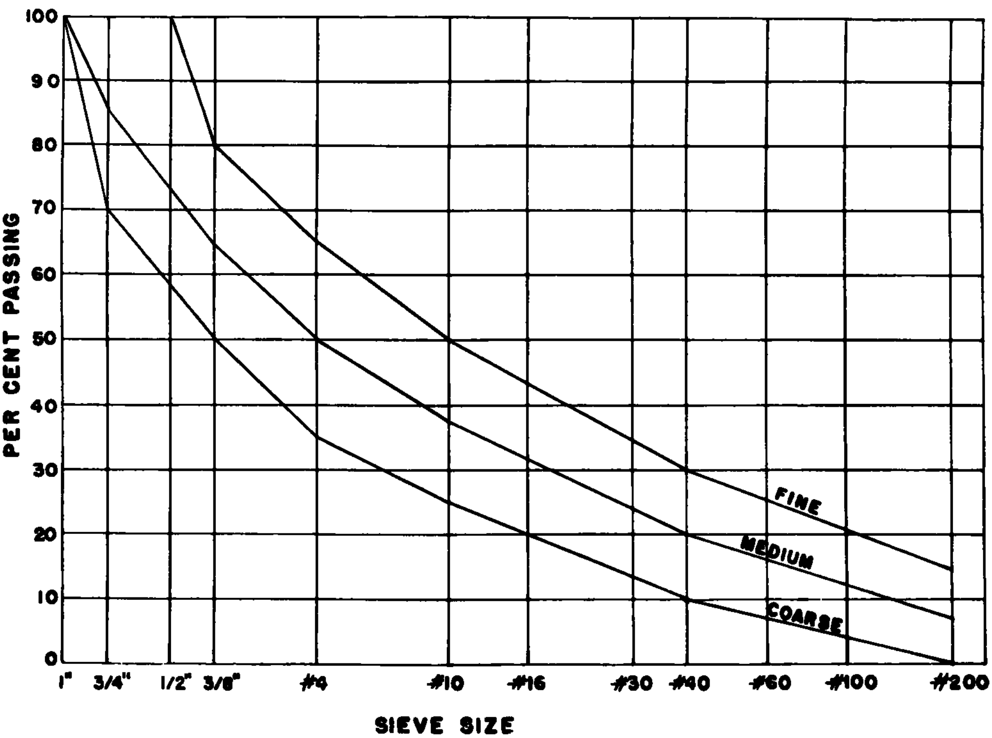

Figure 1. Grading of base course materials.

SO-2846 Clay soil from Valparaiso Morainal area, U. S. 30, Lake County, C horizon.

SO-2847 Sand, from Kankakee sand plains area, U S. 35, Starke County, C horizon.

SO-2848 Sand, Dune Sand, U. S. 35, Starke County.

SO-2852 Loam, Crosby soil, C horizon, Tipton till plain, Tippecanoe County.

SO-2853 Sandy loam, Illinoian Drift, Indiana 59, Clay County.

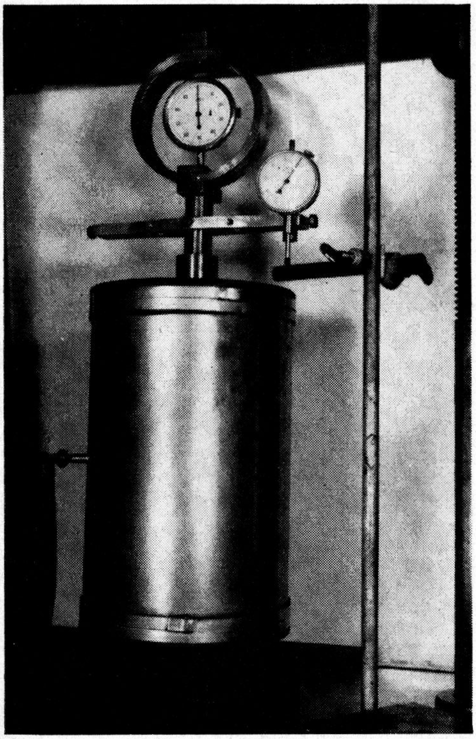

Figure 2.

Proctor density—designated as Type C compaction, and the other at approximately 95 percent of maximum Proctor density—designated Type D compaction.

The Kansas State Highway Testing Laboratory uses a rate of application of load of 0.005 in. per min. (*13*). In order to check the Kansas method of design, and to determine the effect of rate of application of loading, one specimen of each soil at each density was tested using a rate of loading of 0.005 in. per min. and a duplicate specimen was tested using a rate of loading of 0.05 in. per min. The lateral pressure was 20 psi in both tests.

The triaxial-compression tests on soils were conducted using the conventional equipment shown in Figure 3. For the sandy soils, SO-2847 and SO-2848, specimens were formed inside a split mold and a vacuum was applied through a porous stone fixed in the base of the testing device. By this means, the atmospheric pressure on the outside of the specimen acted as a confining pressure, thereby preventing the specimen from slumping prior to being placed in the compression chamber.

Duplicate specimens of each soil and each density were tested using 7-, 14-, and 20-psi. lateral pressure and at a rate of loading of 0.05 in. per min., in order to obtain sufficient data for drawing Mohr's diagrams.

All specimens were molded at saturation moisture content for two reasons: to test the soils in their weakest condition (for a given density), and to test all soils in a common condition.

TABLE 2
SUBGRADE SOIL CHARACTERISTICS

Soil No.	Liquid Limit	Plastic Limit	Textural Classification	Gravel + 2.0 mm.	Coarse Sand 2.0–0.25 mm.	Fine Sand 0.25–0.05 mm.	Silt 0.05–0.005 mm.	0.005 mm.	B.P.R. Classification	
									Group	Group Index
	%	%								
2846	33	15	Clay[a]		7	15	38	40	A-6	10
2847		N.P.	Sand		37	41	7	5	A-2	0
2848		N.P.	Sand		51	46	1	2	A-3	0
2852	30	13	Loam	2	15	28	36	19	A-6	9
2853	31	14	Sandy-loam	5	16	37	25	17	A-6	6

[a] Based on U. S. Bureau of Chemistry and Soils Classification.

The liquid limit, plasticity index, grain size, moisture-density relations, and BPR classification and group index for each of these soils are shown in Table 2.

Duplicate specimens were tested, one at approximately 100 percent of maximum

RESULTS

Base-Course Materials

The results of the triaxial tests indicated that the angles of internal friction of the base-course materials were not influenced to a marked degree by either compactive effort, gradation, or surface texture, (see Table 3). For the crushed stone specimens, the medium gradation, had the lowest value of internal friction, 47 deg., and the fine gradation had the highest value, 55 deg. The medium gradation of crushed stone had fewer large-size angular pieces than the coarse gradation, and, probably, had fines in insufficient quantity to completely fill the voids. Both of these factors would tend to lower the value of angle of internal friction. On the other hand, the specimens of fine gradation of crushed stone had slightly less density than those of medium gradation. This would indicate that the voids were over-filled, permitting the sharp, angular limestone fines to become fully effective in the development of frictional resistance to displacement. There was, in general, an increase in the angle of internal friction with increase in compactive effort.

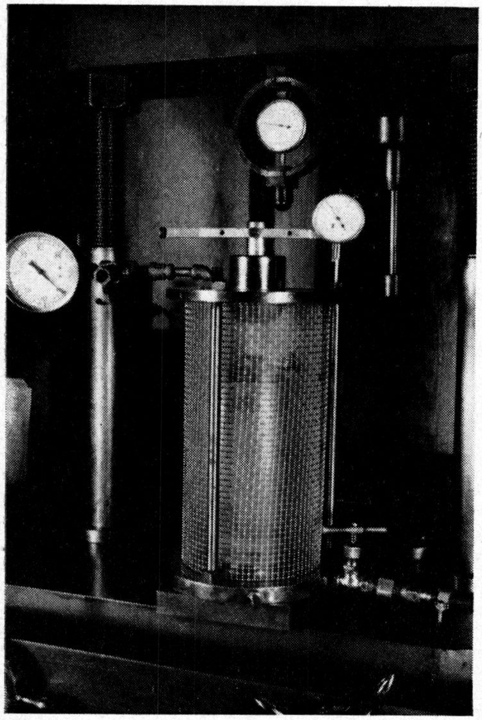

Figure 3.

The modulus of deformation was determined by the straight-line portion of the stress-strain curves. For all gravel specimens, there was a marked increase in the value of modulus of deformation with increased density and increasing coarseness of gradation (Table 3). The values ranged from 16,000 psi. for the stone, coarse gradation, Compaction B, to 1,100 psi. for the gravel, fine gradation, Type A.

The classification system used by the Texas Highway Department consists of plotting the Mohr's rupture line for a soil or base-course material, obtained by means of triaxial tests, on the empirical classification chart (*14*, *15*). The position of Mohr's envelope of failure when plotted on the classification chart, determines the class number of the material. The classes are numbered from 1 to 6 for angles of internal friction ranging from over 50 deg. to 0 deg., and the description accompanying these classes range from good flexible base materials to very weak subgrade, as shown in Table 1. The range of required

TABLE 3
RESULTS OF TRIAXIAL TESTS ON BASE COURSE MATERIALS

	Type of Compaction	Type of Gradation					
		Coarse		Medial		Fine	
		Crushed Stone	Gravel	Crushed Stone	Gravel	Crushed Stone	Gravel
Angle of internal friction (degrees)	A	50	54	48	50	53	41
	B	50	54	47	52	55	50
Average density of test specimens (*/cu. ft.)	A	131.1	137.4	138.6	141.7	137.7	135.7
	B	137.3	140.8	142.2	144.4	141.4	137.7
Percent equivalent proctor density	A	95.4	98.8	98.3	98.8	97.7	98.7
	B	99.9	100.7	100.9	100.7	100.2	100.2
Texas classification	A	1	3	1	3	1	4
	B	1	2	1	2	1	3

thicknesses of pavement for each of the six classes for various wheel loads and two types of base-course materials varies according to the class.

According to the Texas classification, all gradations of crushed stone, using either type of compactive effort, fall into Class 1—good flexible base material, light bituminous surfacing acceptable. The gravel mixtures range from Class 2—fair flexible base material, for coarse and medium gradations—Type B compaction, through Class 3—borderline base and subbase materials, for coarse and medium gradations—Type A compaction, and Class 4 fine gradation—Type A compaction.

TABLE 4
RESULTS OF TRIAXIAL TESTS ON SOILS

Soil No.	Type of Compaction	Angle of Internal Friction	Cohesion	Dry Density	Percent Proctor	Texas Class
		deg.	*psi.*	*lb. per cu ft.*	%	
2846	C	3	3.6	108.0	100	5
	D	3	3.6	103.5	95	5
2847	C	38	3.0	114.5	100	4
	D	35.5	1.5	108.8	95	4
2848	C	34.5	4.0	107.0	100	4
	D	34.5	2.0		95	4
2852	C	13	3.8	113.5	100	5
	D	3	3.0	102.0	95	6
2853	C	8	12.5	114.3	100	4
	D	5	5.0	108.5	95	5

Subgrade Soils

The results of the triaxial tests are shown in Table 4 and indicate that the angles of internal friction of the soils vary from 3 deg. to 38 deg.

The effect of compaction is noted in that, in general, the Mohr rupture line had a steeper slope when the soils were compacted by the higher compactive effort. In two instances (SO-2846 and SO-2848) no difference was found between the two compactive efforts. Values of cohesion also varied with compactive effort but not to the same extent as angle of internal friction.

The modulus of deformation was determined by the procedures described by the State Highway Commission of Kansas (*13*). This modulus, which is essentially the secant modulus of elasticity, was found to vary consistently with compaction and soil type. Values ranged from 666 to 5870 psi. The Texas classification for the soils varies from Class 4 to Class 6.

Design Thickness

Figures 4 through 9 show the required pavement thickness as calculated from the formula adopted by Kansas. In all cases, for the sake of simplicity, a 9,000 lb. single wheel load was assumed, rainfall and traffic coefficients were taken as unity, and limiting deflection was taken as 0.1 in. A 2-in. wearing

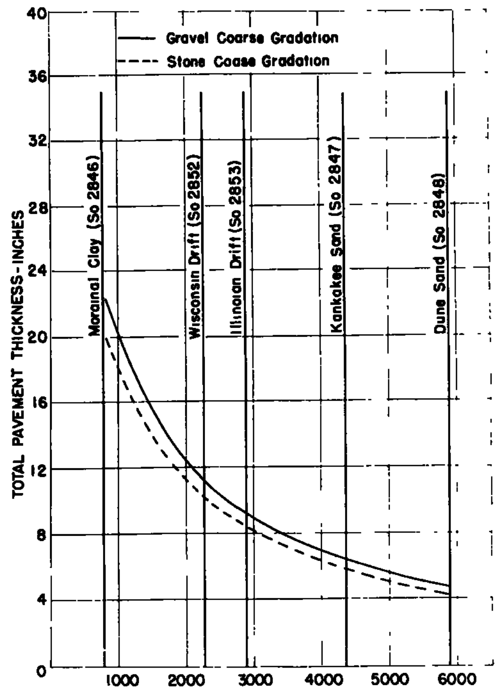

Figure 4. Comparison of required total pavement thickness for several soils.

course having a modulus of deformation of 15,000 psi. was assumed. These curves are not presented as design curves, but rather to illustrate several variables as measured by the triaxial test. It is to be kept in mind that the absolute values of pavement are not important, but that relative values are desired.

Figure 4 shows a comparison of pavement thickness required for the coarse-gravel and crushed-stone mixtures. The soils and base materials were compacted to approximately 100 percent standard Proctor density.

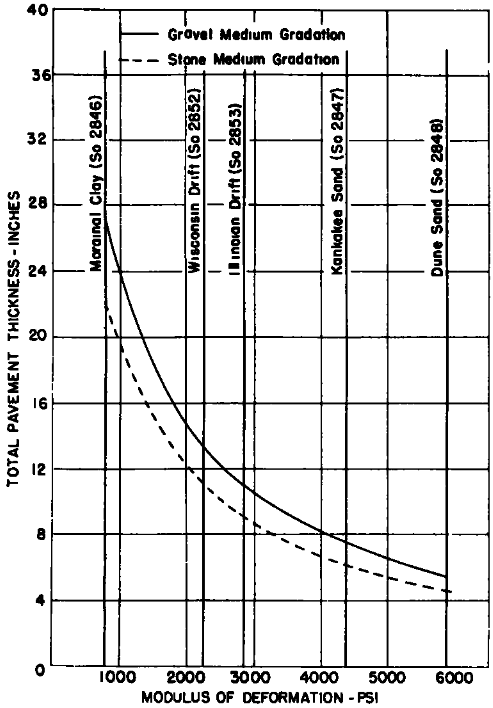

Figure 5. Comparison of required total pavement thickness for several soils.

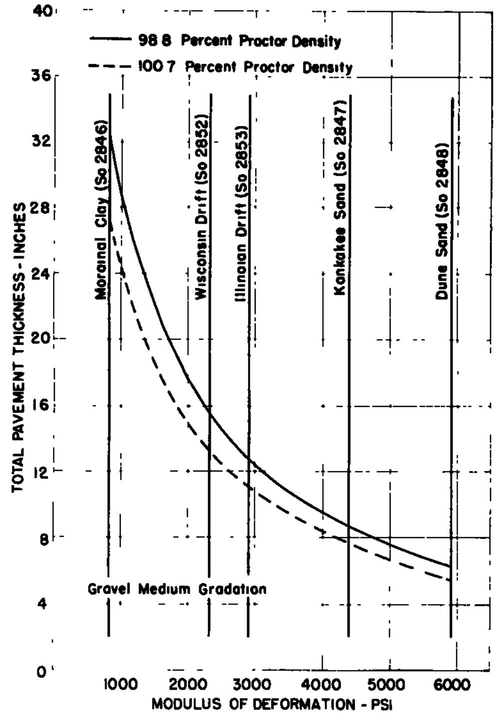

Figure 7. Variation of required total pavement thickness with compaction of base.

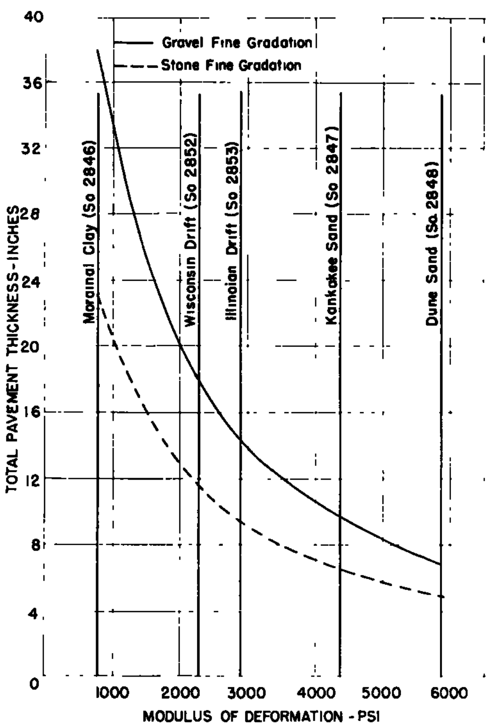

Figure 6. Comparison of required total pavement thickness for several soils.

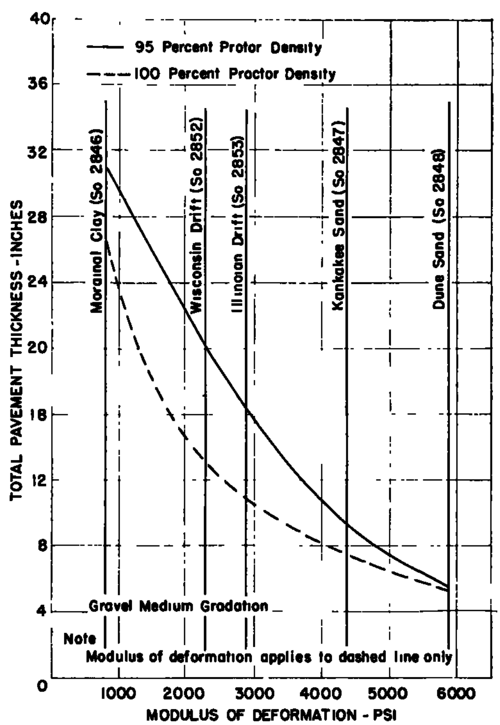

Figure 8. Variation of required total pavement thickness with compaction of soil.

It will be noted that the gravel base course used on the plastic clay (Valparaiso Moraine clay) required the greatest thickness, requiring 22½ in. Also the crushed stone used with this clay required 20 in. It is further noted that for the better subgrade soils the difference between the required depth of gravel base and required depth of stone base decreases. For the plastic clay the difference is 2½ in. and for the dune sand, 1 in.

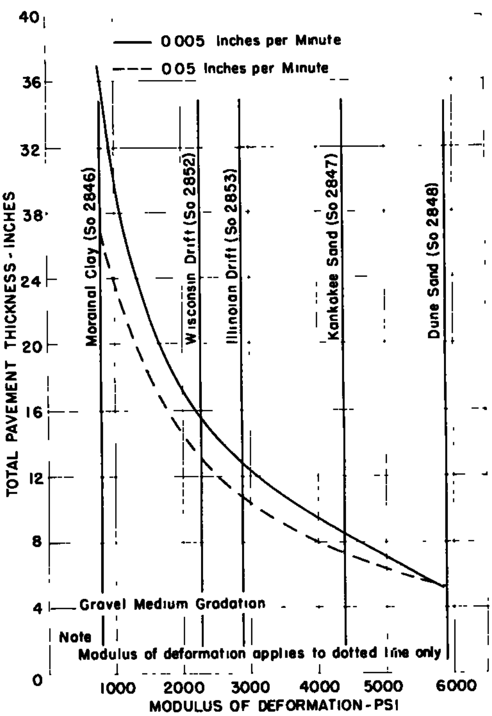

Figure 9. Variation of required total pave ment thicknes with rate of loading.

The corresponding thickness of pavement by the Texas method varied from 13 to 17 in. for an 8,000-lb. wheel load on Soil 2846 to 8 to 13 in. on Soils 2847, 2848, and 2853. Since exact pavement thickness is not important from the standpoint of this study, the principal difference between the two methods appears to be the degree of differentiation between soils. Where small differences in pavement thickness were noted by the Kansas method, the soils were placed into just two groups by the Texas method. It is to be noted, however, that ranges in thickness are permitted within any one group by the latter method.

Figure 5 shows the required thickness for the two base materials of medium gradation. The required thickness varies from a maximum of 27½ in. for the gravel base on the plastic clay soil to 4½ in. of stone and surface on the dune sand. Here again, the greatest difference between the two base materials resulted when used on the plastic clay (a difference of 5½ in.).

Figure 6 shows the required thickness for the two base materials of fine gradation. In this instance the required thickness varies from 38 in. for the gravel plus surface, to approximately 5 in. of stone and surface when used on sand.

Comparison of Figures 4, 5, and 6 indicates that, of the materials tested, the gravel required the greatest thickness and also that the difference between the stone and gravel was greatest for the fine-gradation base materials.

For example, the greatest difference in required thickness for the two materials of comparable gradation (when used on the plastic clay) resulted from the fine gradation (a difference in thickness of 13 in.), while for the medium gradation the difference was 4½ in., and for the coarse, 2½ in. In other words, the difference between the merit of stone and gravel was more noticeable for the finer limits of gradation.

A comparison of pavement thickness for the medium-graded gravel under conditions of two densities of base is shown in Figure 7. It will be noted that a very small decrease in the density (2 percent) resulted in an appreciable increase in pavement thickness, an increase of 5 in. on the plastic soil, and 1 in. on the sand. The difference is seen to be about the same for all soils.

Figure 8 shows a comparison of thickness of base (gravel, medium gradation) and surface under different degrees of compaction of the soils. The greatest difference was again shown by the more plastic soils. The thickness of pavement required on the dune sand was the same irrespective of the compactive effort, (95 percent or 100 percent). The greatest difference was for the Wisconsin drift (7 in.).

To determine the affect of rate of loading during the triaxial list, several tests were made on the soil at varying rates of loading. Figure 9 shows that the principle difference again was shown by the plastic soil. No difference was shown in thickness on the dune sand.

CONCLUSIONS

From this laboratory work and the study of the application of triaxial testing to determine the strength characteristics of subgrade soils and base course materials, and to design of flexible pavements, the following conclusions are indicated:

1. For base-course materials the values of modulus of deformation were influenced consistently and to a great extent by the variables, surface texture, gradation, and density.
2. The pressure cell functioned quite satisfactorily. However, the rubber membrane is heavy and, undoubtedly, provides a frictional force against the sides of the specimen and some restriction to lateral deformation.
3. The method adapted by Kansas for design of flexible pavements appears to be a good one. It utilizes a rational test that can easily be adopted. The matter of rainfall, traffic, frost action, rate of loading, etc., however, warrants further study to modify the basic formula for use in Indiana. The principal difference between the Texas method and the Kansas method is the degree of refinement that can be realized.
4. The results indicated that on the basis of the formula for determining pavement thickness, the structural qualities of the base has a marked influence on the depth of pavement required above a soil of given strength.
5. The crushed stone was better than the round-grained gravel (as affecting pavement thickness). The difference between the two materials varied directly with gradation, from fine to coarse.
6. Degree of compaction of the soil influenced the required depth of pavement to a more marked degree than degree of compaction of the base materials.
7. All the variables were most pronounced when pavements on plastic soils were considered. Generally speaking, the thickness of pavement required on the sand subgrade was not influenced to any appreciable degree by density, rate of loading, or type of base course.

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DISCUSSION

W. M. HAAS, *Instructor in Civil Engineering, University of Wisconsin*—The purpose of the research project described in the paper was to investigate the possible use of triaxial tests for flexible pavement design in Indiana. Various base-course materials were evaluated in addition to natural subgrade soils. If the tests of both subgrade and base-course materials can be properly interpreted, all elements of a flexible pavement may be designed intelligently.

The general conclusion was reached that triaxial tests are a good means of evaluating flexible-pavement design but that further research is needed before this design procedure could be adopted in Indiana. Factors that must be explored further include traffic, climate, and especially frost action and spring break-up.

A research project conducted by the discussor at the University of Wisconsin had a parallel objective: to investigate the possible use of the Kansas triaxial method to design flexible pavements that would stand up under Wisconsin climatic conditions. It is of interest here that somewhat similar conclusions were reached: that the Kansas method had definite possibilities but would have to be modified to yield satisfactory designs. The most significant conclusion was that the triaxial method as developed by the Kansas Highway Department would lead to thicknesses that would not be adequate for the more severe frost and spring break-up conditions in Wisconsin.

Briefly, the investigation at Wisconsin consisted of triaxial tests of subgrade soils from three current highway projects. From the results of these tests, the thickness of flexible pavement required above each subgrade was determined using the Kansas procedure and design curves. The thickness of bituminous surfacing was assumed the same as that selected by the Wisconsin Highway Commission, and the modulus of deformation was assumed as 15,000 psi. The modulus of deformation of the base-course material was assumed to be 6,000 psi. Thus the thickness of base course required varied only with the quality of the subgrade.

The thicknesses thus determined were compared with those selected by the Wisconsin Highway Commission. Two of the Wisconsin designs included a subbase, also known locally as a ballast course or sand lift. In every case, the Kansas method indicated less thickness required. There was a greater variation between the two methods on the poorer subgrades than on the better soils (see Fig. A).

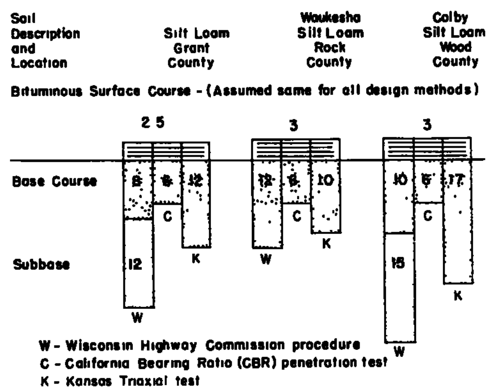

Figure A

A brief description of the Wisconsin Highway Commission procedure for designing flexible pavements is given for an understanding of the principles involved. This method may appear somewhat arbitrary to the casual observer, yet it reflects the actual performance experience of all Wisconsin highways over a period of years. The thicknesses of the various layers are determined by two main considerations: (1) the average annual 24-hr. traffic count, and (2) the "roadway design class." The traffic-count data are simple to apply, and simple to obtain, as traffic-count data are available for all state and federal highways in the state. In general the thicknesses of bituminous surfacing and of base course increase with the traffic volume but do not vary with the roadway design class (see Table A).

The roadway design class determines whether a subbase shall be used, and if it is indicated, how thick it shall be. The mechanical analysis and other pertinent properties of base and subbase are covered by appropriate specifications.

In some respects, the roadway design class is similar to the "class of material" used in the Texas method. However, the Texas material Classes 1 and 2 would correspond to the Wisconsin roadway design Class 1, the Texas material Class 3 would be similar to the Wisconsin Class 2, and so on, with the Texas Class 6 representing the same general quality of material that would indicate a Wisconsin design Class 5, or the worst subgrade conditions. The method of determining the Wisconsin roadway design class is quite different from that used for determining the Texas class of material, however.

TABLE A

FLEXIBLE-PAVEMENT DESIGN THICKNESS

Wisconsin Highway Commission Procedure

Annual 24-hr. average traffic volumes		Less than 100	100 to 500	500 to 1000	1000 to 2000	2000 to 3000	Over 3000
Surface course thickness in inches		1 or less	2	2½	3	3	3
Roadway design class	Ballast thickness required	REQUIRED THICKNESS OF BASE COURSE					
	in.	*in.*	*in.*	*in*	*in*	*in.*	*in.*
1	NONE	5	6	8	10	12	12
2	NONE	6	8	10	12	14	14
3	9	5	6	8	10	12	12
4	12	5	6	8	10	12	12
5	15	5	6	8	10	12	12

The roadway design class is, in effect, a general measure of the suitability of a given subgrade soil. The poorer the soil, the more subbase required. However, quality of soil is not determined by the usual laboratory soil tests. It is determined from the pedological classification of soils, which indicates both the geologic origin of the soil material, and its textural classification, which is a rough sort of mechanical analysis. Other factors that enter into the roadway design class are the topographic and geologic position of the soil. For instance, a coarse soil in a high, well-drained position would indicate a low-numbered roadway design class, and accordingly, a thin subbase or perhaps none at all. On the other hand, a fine-grained soil in a relatively poorly-drained area would indicate a thick subbase. Besides the above factors, which can be determined from soil and topographic maps, other known characteristics are taken into account. For instance, if a certain soil type is known to be susceptible to frost heave, a thicker subbase is indicated, other factors being equal. Thus the roadway design class is selected on the basis of actual highway performance records on the soil under consideration, subject to the judgment of the designing engineer.

With regard to the comparisons being reported here, if the designs arrived at by the use of the Wisconsin Highway Commission procedure are assumed as satisfactory for Wisconsin conditions, it would then appear that designs on the basis of the Kansas procedure would be inadequate for Wisconsin conditions.

The difference probably is best explained by a comparison of the severity of frost action in the two states, as each method was developed by correlating with actual experience. Kansas has an average frost penetration depth varying from 15 in. in the south to 25 in. in the north, while in Wisconsin the range is from slightly less than 30 in. in the southeast to nearly 50 in. in the northwest. This would indicate the necessity for thicker pavements in Wisconsin, other conditions being equal.

One must concur with Yoder and Lowrie that climatic factors, especially frost action and spring break-up, must be correlated or integrated into any design based on soil tests, but it appears that the traffic factor has been reasonably well taken into account in the Kansas method.