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The University of Illinois Pavement Test Track—A Tool for Evaluating Highway Pavements

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The behavior and failure modes of highway pavements are being studied through the use of a newly developed research tool, the University of Illinois Pavement Test Track. The test track is used to study behavior of highway pavements and pavement materials under controlled conditions that approximate service conditions. A description of the facilities is presented; the capabilities and limitations of the facilities are discussed; and the testing techniques and procedures employed are outlined. Emphasis is placed on the use of the serviceability-performance concepts in evaluating the rate of pavement deterioration and as a failure criterion. Typical test results of a research program illustrate potential use of the facilities.

• THE THICKNESS DESIGN of pavements is one of the most complex problems facing the engineering profession. The demand for new highways and the limited funds with which to construct them prohibits the inclusion of a large factor of safety in the design procedure. At the same time, highway pavements are subjected to a wide range of loading conditions and extreme exposure. To complicate the problem even further, pavements are constructed with a variety of paving materials ranging from cohesionless aggregates to high strength concrete over all types of subgrades. From all this the pavement designer must select the right materials in correct combination and thickness to give the maximum performance for the paving dollar.

The AASHO Road Test findings have emphasized the need for a greater understanding of factors that influence pavement performance. Current standard laboratory techniques used to measure the strength of paving materials do not, in general, give a satisfactory indication of the performance potential of the material. Some materials which exhibit good results in laboratory tests perform poorly in the field, whereas other materials which react poorly in the laboratory give a satisfactory performance in the field. This leads directly to the important questions of what factors influence the performance of a pavement and to what extent each factor influences this performance.

The ultimate answer to these questions will have to be found in a rigorous analysis of the pavement structure. This analysis must be based not on some assumed ideal properties of the paving materials but on the actual properties of the materials. It must be based on the observed behavior of these materials under realistic loading and climatic conditions. Unfortunately, the completion of the rigorous solution to this problem appears to be years away. In the meantime, a multibillion dollar program of highway construction continues.

Because a rigorous solution to the problem is not available, some procedure for determining the factors that influence pavement performance must be adopted. The procedure adopted must be based on test procedures that simulate the service conditions of a pavement as closely as possible. This reasoning obviously leads to more test roads. However, test roads are expensive and must extend over a long period of time to gather enough data so that all the variables can be sorted out and evaluated. Furthermore, because of the interaction between variables, it may not be possible to determine the extent of the influence of any one variable on pavement performance.

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To facilitate the evaluation of many of the variables, test pavements constructed and tested under rigidly controlled conditions can be employed. For example, if these pavements are tested under simulated traffic loads with the climatic factors held constant the influence of load on behavior and performance of pavements can be evaluated. After the effect of loads on pavement performance has been determined, the next step of evaluating the effect of environment and climate can be properly undertaken.

The University of Illinois pavement test track was developed to evaluate pavement performance and behavior under controlled conditions. It was designed to apply simulated traffic loads at a high frequency to the test pavements.

The idea of a pavement test track is not a new idea. Through the years a number of test track facilities have been developed and some of them are currently in use (1, 2, 3, 4). A survey of the literature during the design stages of the test track indicated that the descriptions presented in the literature were inadequate to evaluate the potential of these facilities. One of the primary objectives of this paper is to describe the facilities so that other engineers and investigators can evaluate its potential in their own terms.

From what limited information is available in the literature, it appears that the test track is unique in several ways. The width of the test pavements is much greater than in any of the other test tracks described. Because of this greater width, the effect of the boundary conditions on the performance of the pavement are held to minimum. The depth of subgrade used is also greater than for other test tracks described in the literature. Finally, it can be programed to distribute the load applications to give a desired load density histogram. This feature allows for more realistic loading than the single-path loading of other test track facilities.

Because a large number of loads can be applied to the test pavements in a short interval of time, and because all but a few of the variables in the test pavement can be held constant, the effect of a particular variable on pavement performance can be determined in a relatively short period of time and at a reasonable cost. The test track facilities are located close to a fully equipped laboratory, so that a program in the test track can be complemented with a thorough laboratory evaluation program for maximum benefit. In this manner the factors influencing pavement performances can be determined.

The test track can also be used effectively to measure the relative performance of a paving material. With new paving materials being introduced it is important that there be a procedure or tool that can make a preliminary evaluation of the material quickly and inexpensively. The test track effectively serves this function. By comparing the performance of a proposed material with the known performance of a standard material, the relative performance of the proposed materials can be determined.

SCOPE

It is the purpose of this presentation to illustrate how a tool such as the University of Illinois pavement test track can be used for evaluating the factors that influence pavement performance and in developing highway materials. This presentation describes the test track in detail, explains the concepts and limitations of its use, and illustrates through typical results the information that can be gained by the use of this too. The results of the test program presented are typical results but do not include all of the data gathered. Furthermore, the results presented have not been discussed and interpreted, as this is not the purpose of this paper. However, the results are typical and accurately reflect the type of information that can be obtained through the proper use of the facilities.

TEST TRACK FACILITIES

A quonset-type building with 2,400 sq ft of floor space was provided by the University of Illinois to house the test track. Figure 1 is a plan of the building showing the general layout of the testing facilities. Outside storage area has been provided for stockpiling the large quantities of materials required. Figures 2 and 3 show the test track and loading frame.

Physical Dimensions

A detailed cross-section of the test track is shown in Figure 4. The test pavements are placed in the form of an annulus with an outside diameter of 25 ft and an inside diameter of 9 ft, leaving a pavement width of 8 ft. The test pavements rest on a prepared subgrade having a minimum thickness of 3 ft. The center of the wheelpath has a diameter of 16 ft, placing it 3.5 ft from the inside edge and 4.5 ft from the outside edge of the test pavements.

The test track may be divided radially so that several test pavements may be evaluated concurrently. The test pavements that are tested simultaneously are designated

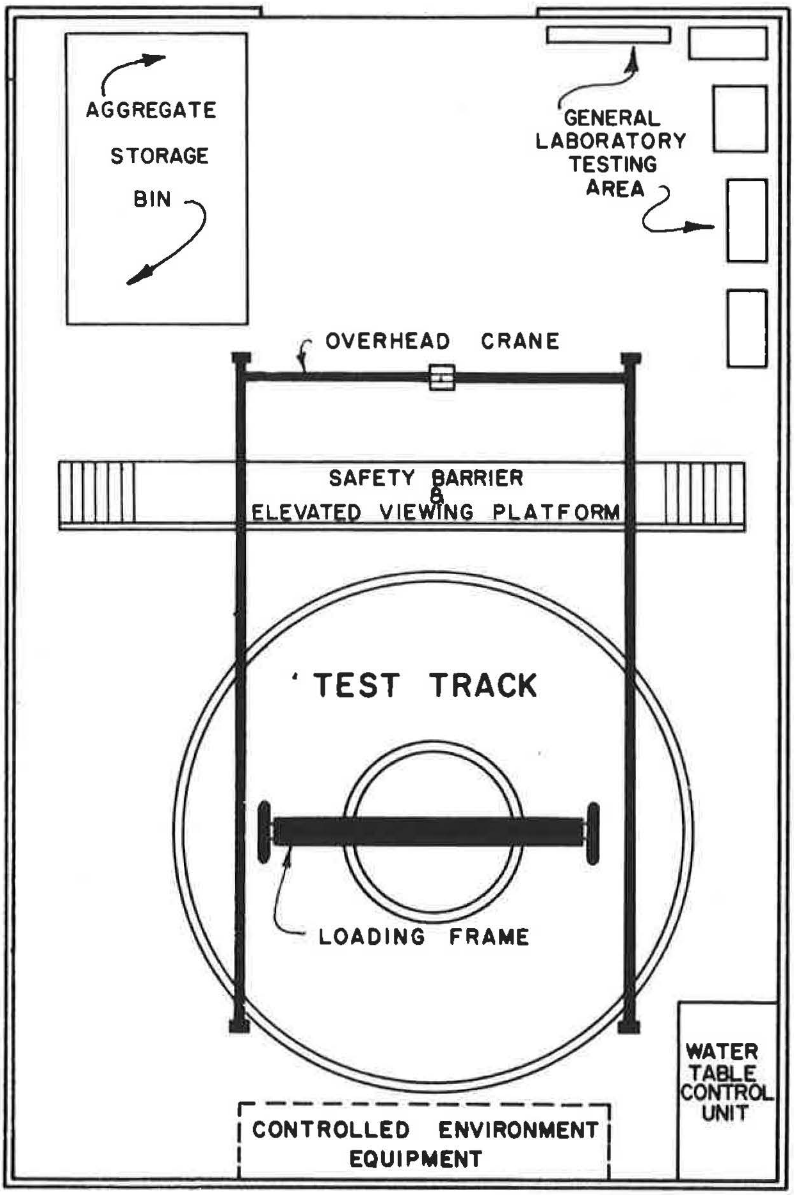

Figure 1. Test track, general layout.

as a set. Adjacent edges of test pavements are merged by the use of transition zones. Because of dimensional limitations discussed later, the maximum number of test pavements considered practical in one test set is six. Additional test pavements may be included in a set by replacing underdesigned test pavements which fail after a few applications of load with new test pavements.

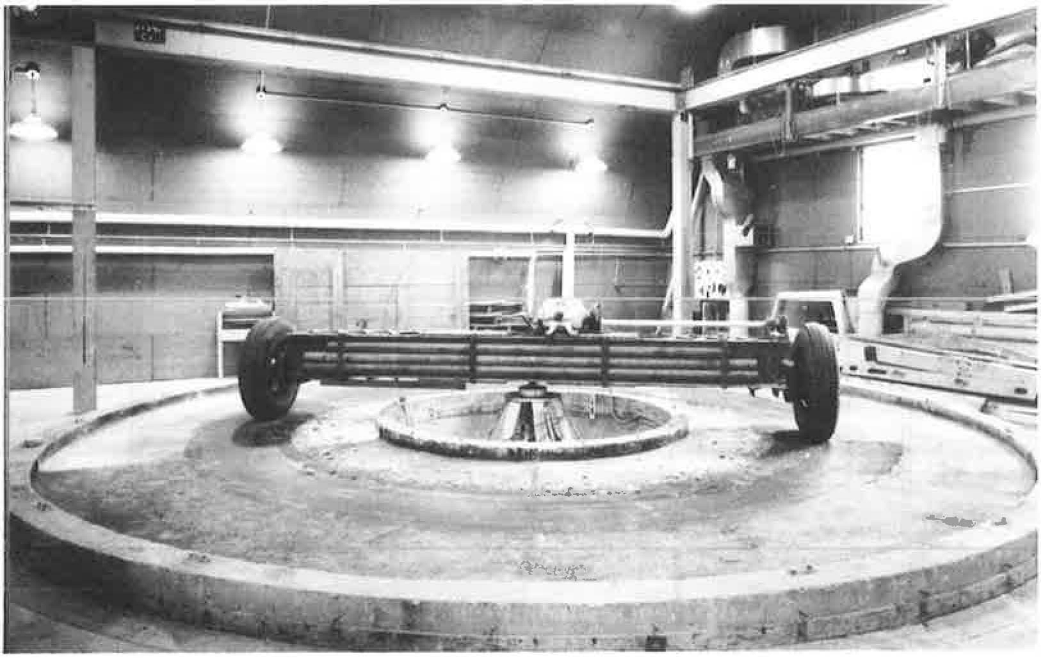

Figure 2. Test track.

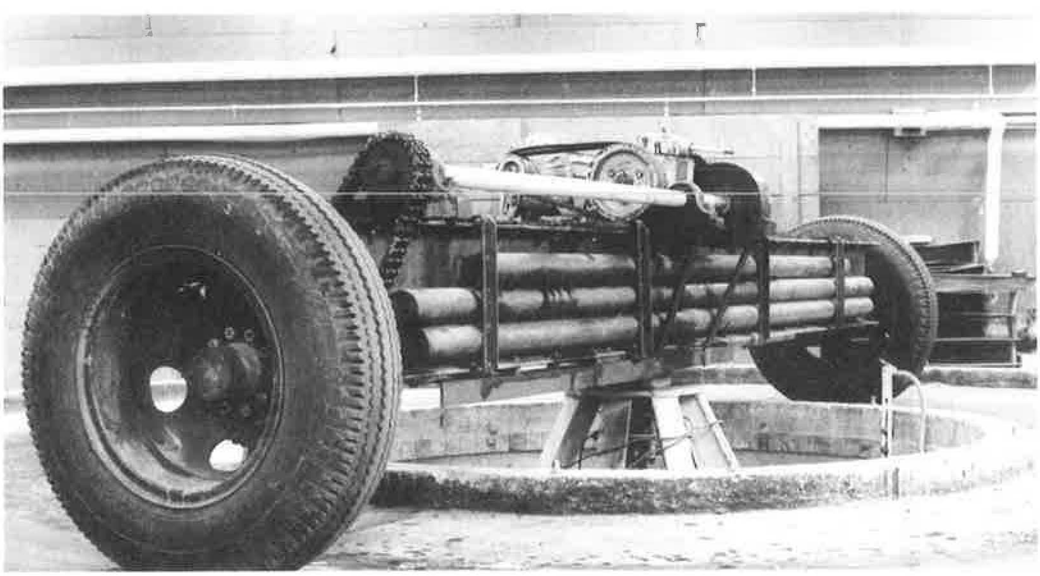

Figure 3. Test track loading frame.

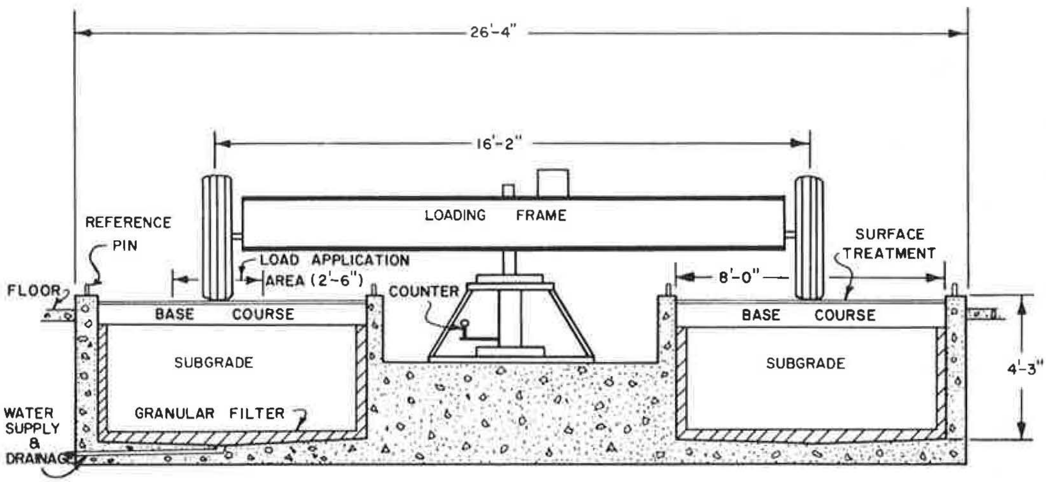

Figure 4. Test track, sectional view.

The base and the walls of the test track pit will affect the behavior of the test pavements because of their influence on the pavement boundary conditions. The dimensions of the test track are such that the effects of these boundary conditions are held to a minimum. A quantitative discussion of the effects of the boundary conditions is not possible because of the many factors that influence them. A quantitative discussion of the boundary conditions that seem most significant is given next.

The effect of boundary conditions on the behavior and performance of a pavement depends, among other factors, on the type of paving material and thickness of the pavement. For the purposes of this discussion the pavements are broken into two classifications: (a) pavements that distribute the load over a large area because of the ability of the paving material to develop relatively large tensile stresses (rigid pavements); and (b) pavements composed principally of cohensionless aggregates (flexible pavements). The terminology is arbitrary and does not necessarily connote the physical behavior of the pavement. Because the effect of the boundary conditions on the two classes of pavements is so different, the effects on each class of pavements is discussed separately.

The effects of the boundary conditions on the rigid pavements are dependent on the physical properties of the paving material as well as the pavement thickness. Therefore, it is convenient to discuss the significant dimensions of the test pavements in terms of a parameter that is a function of both material properties and pavement thickness. Such a parameter is Westergaard's (5) radius of relative stiffness denoted by the symbol L. The parameter L is given by

$$L = \sqrt[4]{\frac{Eh^3}{12\ (1-m^2)k}}$$

in which

E = modulus of elasticity of paving material;
h = thickness of pavement;
m = Poisson's ratio;
k = modulus of subgrade reaction.

The parameter L has a dimension of length and is usually given in inches.

To reduce the effects of the boundary conditions on pavement behavior, the load should be placed as far as possible from the edge, the distance being measured in terms of the relative stiffness L. The position of the load on the test pavements is controlled by the loading frame. Thus, to increase the effective distance of the load from the pavement edge, the L value for the pavement must be reduced. Conversely, to obtain the maximum effect of the boundary conditions the load should be placed relatively nearer the edge. The relative distance of the load in a fixed position from the edge of the pavement is at a minimum when the L for the pavement is at a maximum. Thus, for maximum effect of the boundary conditions the pavement with the greatest L value should be considered. Because the L of the pavement increases with pavement thickness, the maximum effect of the boundary conditions will occur with the thickest pavements.

Based on the AASHO Road Test findings and the maximum load used with the loading frame, the maximum anticipated thickness for a plain concrete test pavement is 4 in. With a subgrade having a modulus of subgrade reaction (k) of 100, the radius of relative stiffness for the 4-in. concrete pavement is approximately 21.5 in. The center of the wheelpath is 42 and 54 in. from the inside and outside edges, respectively, of the test pavements. Thus, the minimum distance from the center of the wheelpath to the edge of the test pavements is approximately 2L.

Meyerhof (6) in his analysis of the ultimate capacity of pavements has shown that the ultimate interior load capacity of a plain concrete slab would be developed if the load is placed a minimum distance of 2L from the edge. In other words, 2L is the minimum distance a load must be placed from the edge to develop the maximum interior loading capacity of the slab. Similarly, to develop the ultimate capacity of an edge-loaded pavement, the load must be placed a minimum distance at 2L from an intersecting edge.

Although the boundary conditions encountered in the test track may not influence the ultimate strength of the test pavements it would be premature to say that these same boundary conditions to not affect the stress in elastic slabs. That is, the stress in the slab before yielding may be influenced by the size of the slab even though the dimensions of the slab are great enough so that the ultimate strength is not influenced. The solution of a finite elastic slab on an elastic foundation is extremely complex. The analysis for a slab with the boundary conditions as imposed by the test track is not currently available.

The thickness of the elastic subgrade will affect the stress in the pavement. An analysis of 4-in. concrete pavement by means of the influence charts prepared by Pickett (7) et al., indicates that the stress in the pavement is reduced by less than 5 percent when the subgrade thickness is reduced from infinite thickness to a thickness of 2L. The depth of the subgrade in the test track under a 4-in. pavement is between 43 and 45 in. The L for the 4-in. concrete pavement with a relatively soft subgrade is between 21 and 25 in. Thus, the subgrade depth is approximately 2L, a depth that was shown to have an insignificant effect on pavement stress. Obviously, if the subgrade is assumed to be a dense liquid, the stress in the pavement is not a function of the subgrade depth.

The influence of the boundary conditions on the behavior and performance of flexible pavements is not known. This is mainly because the factors that influence the behavior and performance of the flexible pavements are not clearly established. There are some data, and theoretical justification, to support the theory that a cohesionless aggregate base will not distribute the load to any greater extent than predicted by the Boussinesq equation (8). If this is so, the Boussinesq equation can be used to estimate the influence of the test track pit on the behavior and performance of a flexible pavement.

The performance of a flexible pavement has been correlated with the pavement deflection under loads (9, 10, 11). Hence, a good indication of the effect of boundary conditions on behavior and performance of the test pavements would be the influence of the boundaries on the pavement deflection.

To illustrate the influence of the test track pit on the pavement deflection the bulb of pressure concept can be used. The bulb of pressure is defined by Terzaghi (12) as "the space within which the vertical normal stress in the subgrade is greater than one-fourth of the normal pressure on the surface of load application. The value of one-fourth has been selected because the major portion of the settlement of a loaded plate resting on a fairly homogeneous subgrade is due to the compression and deformation of the soil located with the space defined by this value."

In Figure 5, the bulbs of pressure for 12- and 30-in. plates are shown on a typical cross-section of the test track. The bulb of pressure for the 30-in. plate does not touch the bottom of the test track pit and for the 12-in. plate it reaches to less than the mid-depth of the pit. Also, the bulbs of pressure do not intersect the walls of the test track pit. The pressure bulbs shown are those as defined by Terzaghi and were calculated from the Boussinesq equations.

The test track pit is wide enough so that the log spiral failure plane proposed by McLeod (13) can form under all anticipated test conditions.

On the basis of the arguments just presented it is apparent that neither rigid nor flexible pavements will be significantly affected by the walls and base of the test track pit.

Water-Table Control Unit

The testing facility is equipped with a water-control unit so that the water table can be controlled to any desired level. A graded granular filter is provided on the bottom

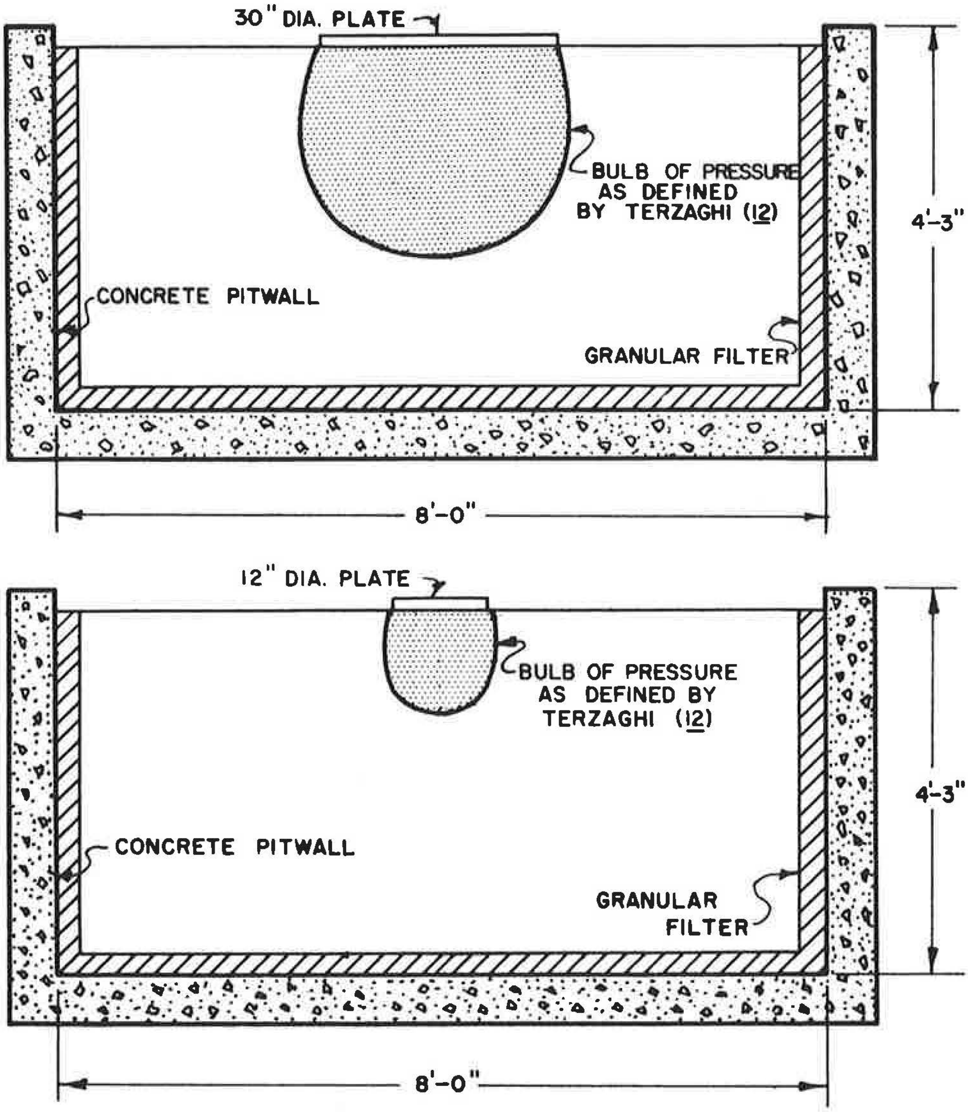

Figure 5. Bulb of pressure under rigid plates.

and along each vertical wall of the test track. A water supply and drainage system is connected to the granular filter. An automatic float system controls the position of the water table.

Controlled Environment Equipment

Controlled environment equipment has been incorporated into the plans for the test track. At the present time, the equipment is limited to producing temperatures on the surface of the track up to 140 F with a lower limit of ambient conditions. The relative humidity can be controlled in the range of 30 to 100 percent. Plans include the addition of refrigeration units so that lower temperatures can be produced. If environment conditions other than ambient are desired, a hood is placed over the test track and air of the desired temperature and humidity is circulated over the surface of the pavements being tested.

Loading Frame

The loading frame with appurtenances weighs approximately 3,700 lb. Provisions have been made for adding ballast to bring the total load to 6,500 lb. The total weight of the loading frame is carried by two wheels with the load evenly distributed between them. The loading frame is prevented from rotating about the wheel axles by a vertical guide which protrudes through the central portion of the loading frame. The frame is free to slide on the vertical guide and can rotate in a vertical plane about a horizontal axis perpendicular to the axles of the wheels, so that each wheel carries its proportionate share of the load at all times.

One of the loading frame wheels acts as a drive wheel, the other is a floating wheel which operates the oscillating mechanism on the frame. Power is supplied to the frame by means of a three-phase electric motor. A pulley system transmits the power from the motor to a four-speed gear box, through a drive shaft to the wheel. With the present pulley system and the four-speed gear box the drive wheel speed can be adjusted between 3 and 15 mph.

The oscillating mechanism controlled by the floating wheel causes the loading frame to oscillate radially as the frame rotates. The amplitude of the oscillation is controlled by a reversing mechanism. When the loading frame has moved to its most extreme position in one direction, a lever system activates the reversing mechanism causing the frame to start in the opposite direction. The amplitude of the oscillations can be adjusted by setting the stops which engage the lever on the reversing mechanism. The maximum amplitude of the loading frame is approximately 30 in. Approximately 100 revolutions of the frame, or 1 mi of wheel travel, is required for a complete cycle across the 30-in. path. By setting the stops and controlling the running time for each amplitude of the oscillation, various traffic patterns in the form of load density histograms can be produced. An automatic counter records the number of revolutions of the loading frame. Skewed distribution patterns can be obtained as conveniently as the symmetrical patterns.

Reference and Anchor Pins

In the design of the test track, permanent reference pins were included for use in measuring changes in the surface profile. These reference pins provide a base for accurate and expedient measuring of changes in the surface profile of the test pavement caused by the applied loads.

Anchors to which frames can be fastened were placed at various points in both concrete walls. These allow static bearing tests to be performed at any location in the track.

CONCEPTS AND LIMITATIONS

The design and analysis of a pavement must include a study of the soils and paving materials, their behavior under load, and the destructive effects of traffic and exposure. Few structures are subjected to as severe conditions of loading and exposure as highway pavements. The effects of both the loading conditions and exposure have been observed in all layers of highway pavements as well as in the supporting subgrade.

The performance of a pavement is the end result of the effects of a number of interrelated variables. Some of the variables that influence the performance of a pavement are traffic density, magnitude of load, load distribution, paving materials, subgrade soil, climatic conditions, drainage, etc. The quantitative effect of any of these variables on pavement performance has not been clearly established. This is mainly due to the difficulty in isolating the effect of any given variable under the fluctuating conditions to which a highway pavement is subjected. There is a danger of misinterpretating the results if one of a number of interrelated variables is isolated and studied independent of the others. If only one variable is studied and all others are suppressed, the results will not be the same as if all variables are acting. However, the variables that affect the pavement performance must be isolated and analyzed independently if the effects of each are to be put into proper perspective. Once the effects of the variables have been established, they can then be integrated into pavement design procedures.

The test track can be used as a tool for evaluating the performance and behavior of pavements. Loads that simulate traffic loads can be applied at a high frequency to the test pavements. Because of the controlled climate, subgrade, and loading conditions it is possible to isolate many of the variables and to study their effect on pavement performance without the confusing influence of other variables.

The test track can serve a useful purpose by spanning the gap between laboratory studies of material properties and field test roads. A logical and economical procedure for developing a design procedure for paving materials is as follows:

1. Conducting laboratory studies on the basic properties of the materials to be incorporated into the pavement.
2. Rationalizing the behavior of the pavement under service conditions and predicting the performance of the pavement.
3. Verifying and/or modifying the theory of behavior and performance by use of results from a test track.
4. Conducting road tests and studying the pavement under service conditions.
5. Observing the performance of the pavement over an extended period of time under actual service conditions.

All five steps must be used if new paving materials and new concepts for the use of paving materials are to be developed in an orderly, economical manner. Too much emphasis cannot be placed on following the sequence proposed. The results of each step must be carefully and completely analyzed before going to the next if maximum benefit is to be derived from a research and development program.

The need for a sound theory of pavement behavior and performance has long been recognized. This need becomes even greater as new materials are introduced. A rational theory is necessary for the orderly development of a testing program for paving materials. Without some theoretical basis it will be impossible to vary test parameters so as to obtain the maximum significance from the test results.

The test track results can be used as the first step for verifying or modifying the theory as applied to the paving materials. The test track has a number of advantages over both static tests and test roads. With the test track, loads that simulate traffic loads can be applied at a high frequency. As a result of the rapid build-up of load applications, the performance and behavior of the pavement under moving loads can be determined in a relatively short period of time. This reduces the time between initial testing of a material and its final incorporation into a highway pavement. The short period of time required for the load repetition build-up also reduces the cost of the test procedures. The total cost for testing a pavement in the test track is but a small fraction of the cost to test this same paving material in a test road.

It is possible to control many variables in a test track that cannot be controlled in a test road. In a test track the subgrade conditions can be either held constant or varied as desired by the investigator. Climatic conditions can be held constant at the test track to eliminate the effects of exposure. It is possible to vary the magnitude, frequency, and distribution of the loads on the test pavements in the test track.

The test track can be used to determine the relative performance of several highway pavements simultaneously. If the capabilities of one type of pavement are known from experience, the performance of another pavement can be compared with it. In this manner, results from the test track can be used to complement the experience and judgment of the highway engineer.

As with any testing facility, the test track has certain limitations. The results obtained from it are valid only for the conditions under which they were obtained. This holds true for all types of test results, including those from the test roads. The results from the test tack can be extrapolated to other conditions, but only on basis of sound engineering judgment, experience, and theoretical considerations.

The rapid accumulation of load applications listed as an advantage in testing a pavement can also be considered a limitation. It is not practical to consider the effects of time on pavement performance in the test track. This effect can best be studied in actual pavements.

At the present time it is not practical to study the effect of climate on the pavement performance in the test track. Facilities have been provided in the test track to install refrigeration equipment when desired. This, along with the heating and humidity control equipment, already present with the facilities, would make it possible to simulate certain climatic conditions on the test pavements. However, it is felt that for the present time the test track can be used to greater advantage in testing the behavior of pavements under load, leaving the evaluation of the effects of climatic conditions for a later phase of development.

It is the belief of all those who have had a close and knowledgeable association with test track that it, along with appropriate laboratory and theoretical studies, can provide useful information for the orderly evaluation of pavement materials.

TEST PROGRAM AND TYPICAL RESULTS

This section includes partial results from a research program currently in progress at the University of Illinois. The pavement test track is being employed as one of the tools for this study. The results included illustrate the type of data that may be obtained through the use of the test track. The authors have not presented a discussion or interpretation of the data as the sole purpose of including the data is to demonstrate the capabilities of the facility.

Included in this section of the paper is a description of the construction techniques employed in handling and placing the materials, a description of the materials used and data on the behavior and performance of several types of pavements. Performance and serviceability data are presented from typical sections of each type of pavement tested in the test track.

Materials and Construction Operations

The materials selected for use in the test program were selected by the project staff with the approval of an advisory committee. The materials selected were considered to be representative of materials in widespread use throughout the country.

Subgrade.—A total of 150 tons of selected subgrade material were taken from borrow pit No. 1 for the AASHO Road Test near Ottawa, Ill. Routine classification tests were made in the laboratory on samples of the subgrade material, which is a yellow-brown soil with an AASHO classification of A-6. The physical characteristics of the subgrade soil are summarized in Table 1. Additional information on soil from the same source is available in Highway Research Board publications (6, 7) relating to the AASHO Road Test.

Before placing the subgrade soil, a granular filter was placed on the bottom of the test track pit. The filter material was a graded aggregate with a range from ¾-in. through minus No. 200 sieve. The granular filter was compacted with a pneumatic tamper.

After the granular filter had been placed and compacted, the subgrade was placed over the filter material. Before placing the soil in the test track pit, vertical sheet metal separators were placed along both the interior and exterior walls of the test track pit so that the subgrade soil and the material for the vertical granular filter could be kept separated. The soil and the filter material were maintained at approximately the same level during placing. The filter material and the subgrade soil were first compacted around the vertical separators by hand. The vertical separators were removed before final compaction.

The subgrade soil was placed in the track and pulverized with a rotary hoe. Water was added to the soil during the pulverization to bring the material to the desired water content. The material was compacted in layers with 3-in. compacted thickness. Several methods of soil compaction were investigated to determine which would give the most uniform results. After considerable experimentation, it was found that the pneu-

TABLE 1

PHYSICAL CHARACTERISTICS OF SUBGRADE MATERIAL

Characteristic	AASHO Designation	Value
AASHO class.		A-6 (8)
Opt. moist. cont.	T99-57	13.0
Max. dry dens.	T99-57	120
Liquid limit (%)	T89-54	25
Plastic limit (%)	T90-54	14
Plastic index (%)	T91-54	11
Grain-size distr. (% passing sieve):	T88-57	
No. 4		98
No. 10		96
No. 40		92
No. 100		85
No. 200		79
0.02 mm		61
0.05 mm		39
0.002 mm		27

TABLE 2

COMPACTED CHARACTERISTICS OF SUBGRADE[a]

Characteristic	Test Set A	Test Set B	Test Set C
Dry density (pcf):			
At beginning of test	118.0	116.2	116.0
At end of test	122.0	119.8	118.8
Percent dry density[b]:			
At beginning of test	98.3	96.8	96.7
At end of test	101.7	99.8	99.0
Water content (%):			
At beginning of test	13.2	12.8	14.3
At end of test	12.8	12.1	13.4
Modulus of subgrade reaction (k)[c]:			
At beginning of test	164	163	58
At end of test	205	178	81

[a]Each value is average of six or more test values.
[b]Of standard (ASSHO T-99).
[c]At 0.05-in. deflection.

matic tampers gave the most uniform densities. Three to five passes of the tampers were required to bring the soil to the desired density. Alternate passes of the tamper were made in transverse directions to minimize directional densification of the subgrade.

During the process of subgrade placement, continuous testing was performed to control the moisture content and compacted density. After the soil was placed, plate bearing tests were made on the subgrade. The values of these tests are given in Table 2. At the end of each testing program, the base materials were carefully removed so that field density and plate bearing tests could again be made on the subgrade. The profile of the subgrade was carefully measured before and after each testing program.

At the completion of each test set, the subgrade material was removed to a depth of 1 ft or more. The removed soil was pulverized and replaced, as previously described, before placing the base courses for the next test set.

TABLE 3

PHYSICAL CHARACTERISTICS OF CRUSHED STONE BASE MATERIAL

Characteristic	AASHO Designation	Test Set A	Test Set C
Opt. moist. cont.	T99-57	6.8	6.2
Max. dry dens.	T99-57	139.0	144.6
Grain-size distr.			
(% passing sieve):	T88-57		
1-in.		100	100
3/4-in.		96	94
3/8-in.		65	60
No. 4		42	55
No. 10		25	41
No. 40		13	20
No. 200		7	16
Compacted dens.:			
Pcf		143.5	147.7
Percent of standard		103.2	102.1

TABLE 4

GRADATION OF GRAVEL FOR POZZOLANIC BASE

Sieve Size	Grain Size Distr.[a] (%)
3/4-in.[b]	100
3/8-in.	87
No. 4	73
No. 10	52
No. 40	23
No. 200	8
0.02 mm	4
0.05 mm	2
0.002 mm	1

[a]AASHO designation, T88-57.
[b]Material larger than 3/4 in. discarded.

TABLE 5

PROPERTIES OF FLY ASH FOR POZZOLANIC BASE

Property	Value (%)
Major constituent (approx):	
Silicon dioxide	41
Aluminum oxide	25
Ferric oxide	21
Calcium oxide	4
Sulfur trioxide	1
Loss on ignition	7.2
Grain-size distr.	
(passing sieve):	
No. 10	100
No. 40	98
No. 200	87
No. 325	79

During the initial construction phase a special soil planer was developed and was used to bring the various pavement layers to the desired elevation and thickness. The soil planer is capable of trimming the compacted soil to a tolerance of ± 0.03 in., and the compacted base materials to within 0.1 in.

Crushed Stone Bases.—The crushed stone bases used in the test program were designed and constructed to represent those used in typical highway pavements. The crushed stone was a limestone provided by stone producers from materials designated for use in the Illinois Highway Construction Program. The characteristics of the crushed stone are given in Table 3.

Before placing, the crushed stone was mixed on the job site in a concrete mixer, and water was added to bring the moisture content to the desired level. The materials were compacted with vibratory compactors and pneumatic tampers. The desired thickness was obtained by trimming the base with the soil planer.

Pozzolanic Bases.--The pozzolanic bases were composed of 82 percent gravel, 14 percent fly ash, and 4 percent lime. The gravel used for the pozzolanic bases came from a stockpile of subbase material used in the AASHO Road Test. It was the same material as was used for the cement treated and bituminous treated bases in the special base study at the AASHO Road Test. The grain-size distribution of the gravel is given in Table 4. The fly ash used in the pozzolanic base was obtained from the Public Service Electric and Gas Company, Sewarren, N.J. Properties of the fly ash are given in Table 5. The lime used in the pozzolanic bases was a monohydrated dolomitic lime supplied by the Marblehead Lime Company, Chicago, Ill. Properties of the lime are shown in Table 6.

An extensive laboratory investigation was conducted on pozzolanic base material before the repeated wheel load test in the test track. The general characteristics of the pozzolanic base material are given in Table 7. Figure 6 shows the general relationship between strength and age for the pozzolanic base mixtures used. The relationships shown are for specimens cured under ambient conditions. Specimens cured in moist sand for 28 days had a compressive strength of 710 psi, and those cured for 7 days in a sealed container at 130 F had a compressive strength of 1,360 psi. The pozzolanic base material exhibited no weight loss during the freezing and thawing or wetting and drying durability tests.

The fatigue characteristics of pozzolanic base material were measured and have been reported (16). The coefficient of thermal expansion was measured and found to be approximately 6×10^{-6}. The modulus of elasticity of cured material was found to vary between 1.6×10^6 and 2.5×10^6, depending on the age of the material (Fig. 6).

The pozzolanic base materials were proportioned and mixed at approximately optimum water content in a $1\frac{1}{2}$-cu ft pug mill mixer. The pozzolanic base was compacted with pneumatic tampers in the manner described for the subgrade material and the crushed stone. After compaction, the material was trimmed to the desired level with the soil planer.

Surfacing.—Several types of wearing surfaces were used on the test pavements reported. On test set A (Table 8) the wearing surface for the crushed stone was a sand-asphalt slurry seal, approximately $\frac{1}{8}$ in. thick. The wearing surface for the pozzolanic bases in test set B (Table 8) was a troweled fly ash mortar mixture $\frac{1}{16}$ to $\frac{1}{8}$ in. thick. The wearing surface provided a smooth initial profile and an opportunity to study the crack patterns at an early age in their development. The test pavements in test set C were made up of crushed stone bases covered with 1 to 4 in. of asphaltic concrete (Table 9). The engineering properties of the asphaltic concrete used in test set C are given in Table 9.

Experimental Test Pavements

The test track is divided radially into six test sections for experimental purposes. Each test set was initially composed of six test pavements in which the pavement thickness and/or materials were varied. Table 8 gives the pertinent test data for the test pavements.

Traffic Operations

The wheel loads were applied to the test pavements by the loading frame previously described. A wheel load of 3,200 lb was used for all tests. The load was applied to the pavements through 8.25 x 20 tires inflated to 75 psi.

All tests were conducted with a wheel speed of approximately 13 mph, unless surface roughness dictated that a lesser speed be used. A speed of 13 mph will provide approximately 22,000 load applications for each 8 hr of operation. Loading operations were suspended at regular intervals for routine tests, measurements, and maintenance.

The basic operation plan was to traverse the wheel across the loading path under a

TABLE 6

PROPERTIES OF LIME FOR POZZOLANIC BASE

Property	Value (%)
Major constituents (approx.):	4
Calcium carbonate	59
Calcium hydroxide	2
Magnesium hydroxide	33
Magnesium oxide	
Grain-size distr. (passing sieve):	
No. 30	100
No. 100	97
No. 200	90
No. 325	85

TABLE 7

GENERAL CHARACTERISTICS OF POZZOLANIC BASE MATERIAL

Characteristic	AASHO Designation	Value
Composition (% by wt):		4
Lime		4
Fly ash		14
Gravel		82
Max. dry den.	T99-49	135.4
Opt. moist. cont.	T99-49	7.8
Compacted dens:		
Pcf		132.2
Percent of standard		97.6

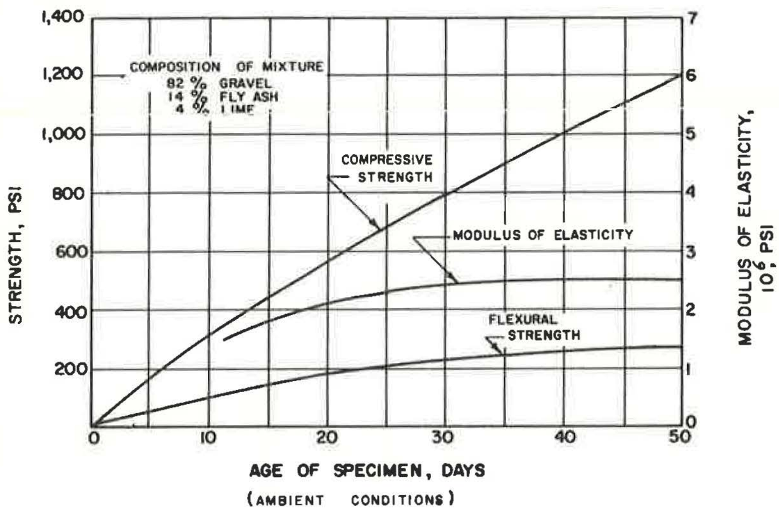

Figure 6. Strength-age relationships for pozzolanic base material.

controlled pattern that approximated the normal distribution of highway traffic. This was accomplished by adjusting the amplitude of oscillation of the loading frame at specified intervals. The resulting histogram of load density approximated the traffic distribution pattern determined by a Bureau of Public Roads study (17).

As testing progressed, it became apparent that the pozzolanic base was distributing load by slab action, and as such, should be considered as a rigid or semi-rigid base. A comparison of the fatigue characteristics of the pozzolanic base material (16) and the theoretical stresses produced during the build-up of a complete histogram of load applications showed that only 40 to 45 percent of the total applications would be effective in producing fatigue failure in the slab. The relatively high cohesive strength of the material also prevented rutting of the base. Therefore, to accelerate the test, tests on pavements with pozzolanic bases were conducted without the use of the traversing mechanism. All test loads applied to pavements with crushed stone bases were performed with the traversing mechanism in operation.

During loading operations, the surfaces of the test pavements were lubricated to reduce tire wear and to minimize horizontal forces created by a wheel moving in a circular path. Effectiveness of the surface lubrication in reducing the horizontal stresses is indicated by the tire wear. The 8.25 x 20 tires which were used in the testing program have traveled more than 5,000 mi with only nominal wear.

Whenever a test pavement failed, the section was declared out of test and pavement maintenance was conducted. The maintenance consisted of rebuilding the section with either asphaltic or portland cement concrete.

TABLE 8

TEST PAVEMENT DATA

Test Set	Test Pavement No.	Base Material	Base Thickness (in.)	Surfacing Material	Surface Thickness (in.)
A[a]	1	Crushed stone	8.0	Slurry seal	Nominal (1/8)
	2	Crushed stone	6.0	Slurry seal	Nominal (1/8)
	3	Crushed stone	10.0	Slurry seal	Nominal (1/8)
	4	Crushed stone	8.0[b]	Slurry seal	Nominal (1/8)
	5	Crushed stone	12.0	Slurry seal	Nominal (1/8)
	6	Crushed stone	10.0	Slurry seal	Nominal (1/8)
B[c]	1	Pozzolanic	4.3	Mortar	Nominal (1/16 to 1/8)
	2	Pozzolanic	4.8	Mortar	Nominal (1/16 to 1/8)
	3	Pozzolanic	5.3	Mortar	Nominal (1/16 to 1/8)
	4	Pozzolanic	5.8	Mortar	Nominal (1/16 to 1/8)
	5	Pozzolanic	4.8[b]	Mortar	Nominal (1/16 to 1/8)
	6	Pozzolanic	5.3[b]	Mortar	Nominal (1/16 to 1/8)
C	1[d]	Crushed stone	6.0	Asph. conc.	1.0
	2[d]	Crushed stone	3.0	Asph. conc.	1.0
	3[d]	Crushed stone	3.0	Asph. conc.	2.0
	4	Crushed stone	3.0	Asph. conc.	3.0
	5	Crushed stone	6.0	Asph. conc.	3.0
	6	Crushed stone	6.0	Asph. conc.	2.0
	1A[d]	Crushed stone	6.0	Asph. conc.	4.0
	2A[d]	Crushed stone	0.0	Asph. conc.	4.0
	3A[d]	Crushed stone	3.0	Asph. conc.	4.0

[a] Wheel load, 3,200 lb; tire pressure, 75 psi; for all test sets.
[b] Replica test pavement.
[c] Tests on pozzolanic bases began after 5 days during under ambient conditions.
[d] Sections 1, 2, and 3 were replaced after early failure with sections 1A, 2A, and 3A.

Load Distribution Behavior

The principal function of the base course in a highway pavement is to distribute the applied traffic loads to the underlying soil on which the pavement is built.

The manner in which a base course distributed the applied loads was studied by measuring the deflection of the pavement under moving wheel loads. A limited program was conducted on this phase of the research program.

The deflection of the pavement under the moving load was measured by means of linear variable differential transformers (LVDT) mounted in the pavement. The impulses from the LVDT's were transmitted to a Sanborn continuous recording device. With this system, the deflection at a particular point could be measured continuously. As the wheels moved on the pavement surface, a complete pattern of pavement deformation at a point due to the wheel load was measured. The LVDT's were mounted as shown in Figure 7.

TABLE 9

PROPERTIES OF ASPHALTIC CONCRETE SURFACE MATERIAL

Property	Value
Marshall stability (lb)	2,110
Marshall flow	8
Marshall density (pcf)	144.1
Marshall (% void)	3.5
Gradation[a] (% passing sieve):	
1/2-in.	100
3/8-in.	94
No. 4	71
No. 10	50
No. 40	30
No. 80	9
No. 200	3
Asphalt content (%)	5.3
In-place density (pcf)	137.7
Marshall density (%)	95.6

[a] By extraction.

The LVDT core was attached to a stainless steel (¼-in. diameter) anchor rod which was anchored to a base plate in the bottom of the test track pit. The casing of the LVDT was bonded to the base material. As the wheel load caused the base to deflect, the LVDT casing moved relative to the core and a change in potential was recorded on the Sanborn recorder. Each LVDT was individually calibrated before use.

The Sanborn-LVDT system provided a means of measuring the deflection of the pavement at a specific point, regardless of the position of the load. By observing the location and speed of the wheel, the deflection of the pavement at the LVDT was correlated with the wheel position. By the reciprocal theorem, the deflection at any point

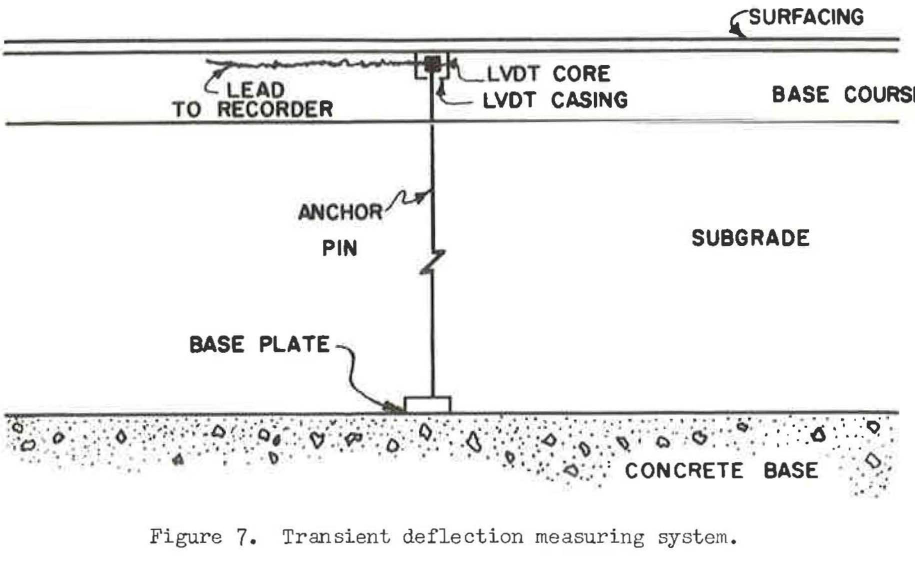

Figure 7. Transient deflection measuring system.

on the pavement as result of the wheel load over the LVDT can be obtained. Thus, the entire deflection pattern of the pavement due to a load at a specific point can be determined.

Figure 8 shows typical deflection patterns obtained with the pozzolanic and crushed stone base materials. Typical cross-sections of the deflection profiles are shown. The pozzolanic bases distributed the load over a larger area than than did the crushed stone bases and had less total deflection. Thus, the pozzolanic bases provide significant bridging action reducing subgrade stresses.

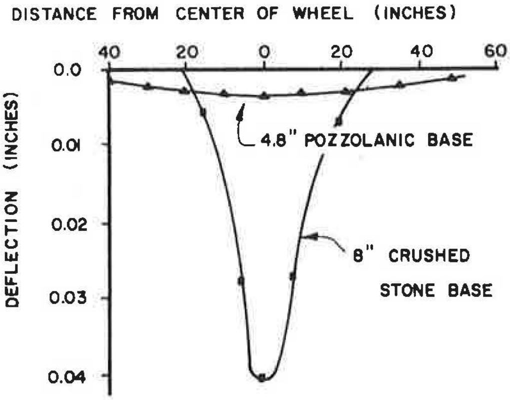

Figure 8. Typical base deflection pattern under moving load.

Pavement Serviceability and Performance

A major portion of the experimental study of pavement behavior involved a study of the relative performance of the test pavements in which the pavement thickness and/or materials were varied. The serviceability-performance concepts developed for the AASHO Road Test were used in this research program.

"The relative performance of various pavements is their relative ability to serve traffic over a period of time" (18). Present serviceability is defined as "the ability of the specific section of pavement to serve high-speed, high-volume, mixed (truck and auto) traffic in its existing condition" (18). The present serviceability index was developed as a mathematical combination of values obtained from certain physical measurements and so formulated as to measure the present serviceability of a pavement. The present serviceability index (PSI) corresponds to the following ratings of a pavement's ability to serve traffic at any given time:

4 - 5 Very good
3 - 4 Good
2 - 3 Fair
1 - 2 Poor
0 - 1 Very poor

The relative performances of various pavements may then be evaluated by a record of the present serviceability against number of load applications. A complete discussion of the serviceability-performance concept is given elsewhere (18).

The serviceability equations as presented in this reference are

for rigid pavement:

$$PSI = 5.41 - 1.78 \log (1+\overline{SV}) - 0.09 \sqrt{C + P}$$

for flexible pavement:

$$PSI = 5.03 - 1.91 \log (1+\overline{SV}) - 1.38 \overline{RD}^2 - 0.01\sqrt{C + P}$$

in which

$\overline{SV}$ = variance of slope along wheelpath;
$\overline{RD}$ = depth of rut in wheelpath under a 4-ft straight edge;
$C + P$ = major cracking and patching.

The parameters required for determining the present serviceability index of the test pavements in the test track were measured using procedures similar to those used in developing the equations.

The control reference pins made the measurement of the surface elevations for calculating slope variance a relatively simple task. A special frame was placed over the test pavements and rested on the reference pins. Dial indicators were used to measure

TABLE 10

PAVEMENT PERFORMANCE DATA

Test Set	Test Pavement No.	Base Material	Base Thickness (in.)	Surface Material	Surface Thickness (in.)	Approx. Thousands of Applications Before Serviceability Dropped to 4.0	3.0	2.0	Subgrade Conditions
A	1	Cr. stone	8.0	Slurry seal	Nominal	1	3	6	γ_d = 118.0 pcf;
	2	Cr. stone	6.0	Slurry seal	Nominal	1	2	3	w = 13.2%;
	3	Cr. stone	10.0	Slurry seal	Nominal	1	4	7	k = 164 pci
	4	Cr. stone	8.0	Slurry seal	Nominal	1	2	4	
	5	Cr. stone	12.0	Slurry seal	Nominal	2	7	18	
	6	Cr. stone	10.0	Slurry seal	Nominal	1	4	8	
B	1	Pozzolanic	4.3	Mortar	Nominal	210	320	350	γ_d = 116.2 pcf;
	2	Pozzolanic	4.8	Mortar	Nominal	--[a]	--[a]	--[a]	w = 12.8%;
	3	Pozzolanic	5.3	Mortar	Nominal	--[a]	--[a]	--[a]	k = 163 pci
	4	Pozzolanic	5.8	Mortar	Nominal	--[a]	--[a]	--[a]	
	5[b]	Pozzolanic	4.8	Mortar	Nominal	12	30	48	
	6	Pozzolanic	5.3	Mortar	Nominal	--[a]	--[a]	--[a]	
C	1	Cr. stone	6.0	Asph. conc.	1.0	--[c]	1	1	γ_d = 116.0 pcf;
	2	Cr. stone	3.0	Asph. conc.	1.0	--[c]	--[c]	--[c]	w = 14.3%;
	3	Cr. stone	3.0	Asph. conc.	2.0	--[c]	--[c]	1	k = 58 pci
	4	Cr. stone	3.0	Asph. conc.	3.0	1	3	5	
	5	Cr. stone	6.0	Asph. conc.	3.0	5	10	--[a]	
	6	Cr. stone	6.0	Asph. conc.	2.0	--[c]	2	5	
	1A	Cr. stone	6.0	Asph. conc.	4.0	1	3	23	
	2A	Cr. stone	6.0	Asph. conc.	4.0	2	5	14	
	3A[c]	Cr. stone	3.0	Asph. conc	4.0	--[c, d]	--[c, d]	--[c, d]	

[a] Serviceability of test pavement did not drop to this level.
[b] Removal of base after failure showed material segregation on bottom of base.
[c] Less than 500 applications.
[d] Crushed stone base inadvertently compacted to a density of 137.8 pcf compared with 147.7 pcf for other bases in test set C.

the surface elevation of the test pavements at 9-in. intervals both tangentially and radially. The surface elevation data from the wheelpath were used to compute the slope variance of the test pavements. The rut depth was obtained from the radial measurements of the surface elevations.

Surface irregularities will inevitably occur on any surface finished by normal construction procedures. These irregularities produce non-uniform values for the initial present serviceability indexes of the test pavements. These initial irregularities are not indicative of the performance of a given test pavement. Thus, to eliminate the effects of any initial surface irregularities, the change in slope due to the applied loads was used rather than the actual slope for determining the slope variance. The serviceability equation developed for rigid pavements was used for evaluating the performance of the pozzolanic bases, and the equation for flexible pavements was used in conjunction with the pavements with crushed stone bases. The serviceability record for each test pavement was plotted using a three-point moving average as a smoothing technique.

Table 10 summarizes the relative performance of the test pavements. The relationship between serviceability and the number of load applications for the 21 test pavements is shown in the Appendix.

CONCLUSIONS

A description of the test track and concepts of its use have been presented along with typical results to illustrate how the facility can be used to evaluate paving materials. Any conclusion regarding the trends in the data would necessarily require a discussion and interpretation of the results. Because the test program is not complete and only a portion of the available data has been presented herein, it would be both premature and unwise to discuss and interpret the data presented. It is anticipated that the entire testing program will be presented, and the results interpreted and discussed at some future date.

With respect to the test track proper, it has been shown that the facility was designed to keep the influence from the boundary conditions to a minimum while holding the volume of materials required to a reasonable amount. It was shown that the loading frame can apply a large number of loads distributed in a manner to simulate traffic loads in a relatively short period of time.

Typical test results were presented in this report. This should not be taken to mean that this is the only type of data that can be collected. On the contrary, data on many different phases of pavement behavior and performance can be gathered. The extent and type of data that can be obtained are limited only by the imagination of the personnel conducting the research.

The cost of evaluating a paving material with this facility will vary with the extensiveness of the program undertaken but will always be but a small fraction of the cost to evaluate the material in a test road.

ACKNOWLEDGMENTS

The authors wish to acknowledge the great number of individuals and organizations who have made the development of the University of Illinois pavement test track possible. The Federal Aviation Authority gratuitously supplied the test track loading frame, the University of Illinois and the National Science Foundation supplied the funds to construct and equip the test track, and several organizations from industry contributed the funds for the test program from which the typical results were taken.

The work covered in this report was carried out under the administrative supervision of W. L. Everitt, Dean of the College of Engineering; R. J. Martin, Director of the Engineering Experiment Station; N. M. Newmark, Head of the Department of Civil Engineering; and E. Danner, Director of the Illinois Cooperative Highway Research Program and Professor of Highway Engineering. The suggestions and advice given by the advisory committee to the project are gratefully acknowledged.

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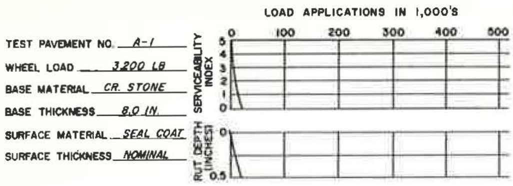
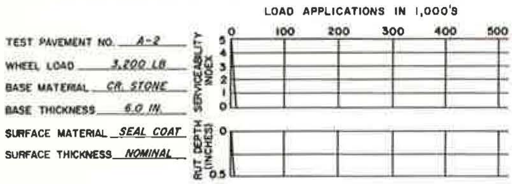
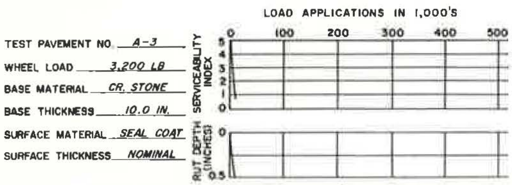
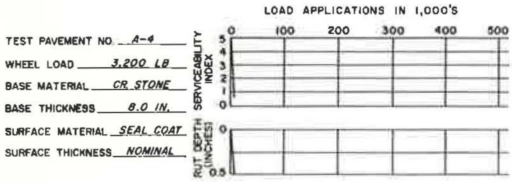
Appendix

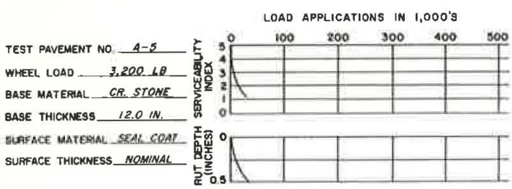
SERVICEABILITY/LOAD APPLICATIONS FOR TEST PAVEMENTS

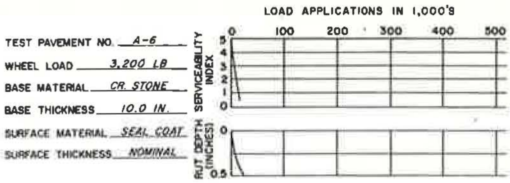


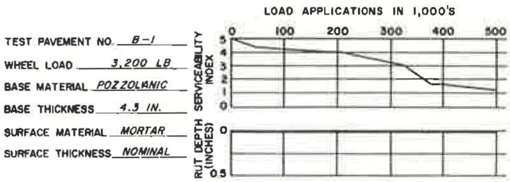


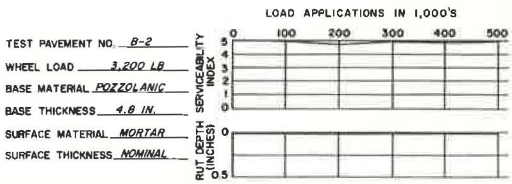


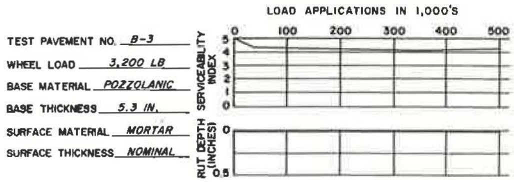


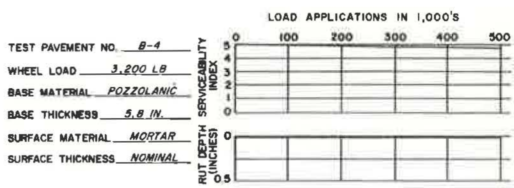




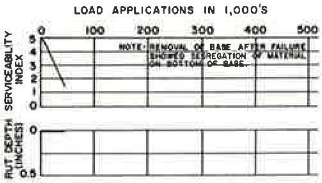




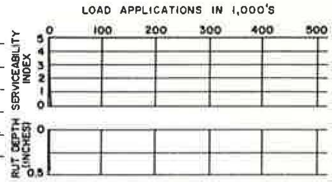




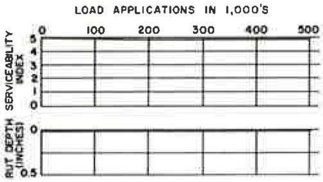
TEST PAVEMENT NO. B-5
WHEEL LOAD 3,200 LB
BASE MATERIAL POZZOLANIC
BASE THICKNESS 4.8 IN.
SURFACE MATERIAL MORTAR
SURFACE THICKNESS NOMINAL



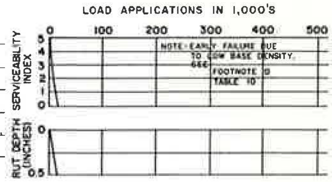
TEST PAVEMENT NO. C-3
WHEEL LOAD 3,200 LB
BASE MATERIAL CR. STONE
BASE THICKNESS 3.0 IN.
SURFACE MATERIAL ASP. CONC.
SURFACE THICKNESS 2.0 IN.



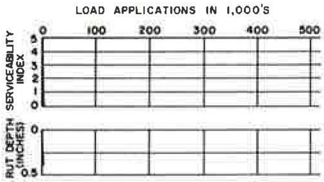
TEST PAVEMENT NO. B-6
WHEEL LOAD 3,200 LB
BASE MATERIAL POZZOLANIC
BASE THICKNESS 5.3 IN.
SURFACE MATERIAL MORTAR
SURFACE THICKNESS NOMINAL



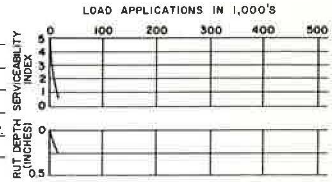
TEST PAVEMENT NO. C-3A
WHEEL LOAD 3,200 LB.
BASE MATERIAL CR. STONE
BASE THICKNESS 3.0 IN.
SURFACE MATERIAL ASP. CONC.
SURFACE THICKNESS 4.0 IN.



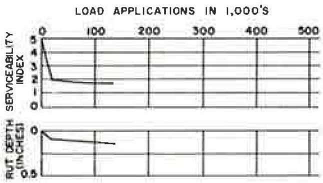
TEST PAVEMENT NO. C-1
WHEEL LOAD 3,200 LB
BASE MATERIAL CR. STONE
BASE THICKNESS 6.0 IN.
SURFACE MATERIAL ASP. CONC.
SURFACE THICKNESS 1.0 IN.



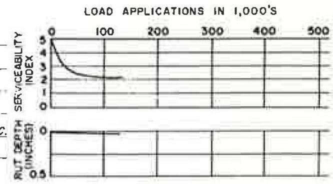
TEST PAVEMENT NO. C-4
WHEEL LOAD 3,200 LB
BASE MATERIAL CR. STONE
BASE THICKNESS 3.0 IN.
SURFACE MATERIAL ASP. CONC.
SURFACE THICKNESS 3.0 IN.



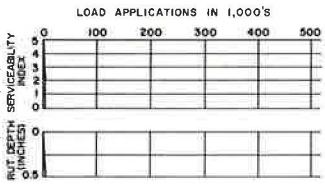
TEST PAVEMENT NO. C-1A
WHEEL LOAD 3,200 LB
BASE MATERIAL CR. STONE
BASE THICKNESS 6.0 IN.
SURFACE MATERIAL ASP. CONC.
SURFACE THICKNESS 4.0 IN.



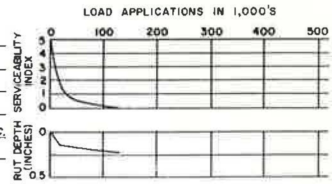
TEST PAVEMENT NO. C-5
WHEEL LOAD 3,200 LB
BASE MATERIAL CR. STONE
BASE THICKNESS 6.0 IN.
SURFACE MATERIAL ASP. CONC.
SURFACE THICKNESS 3.0 IN.



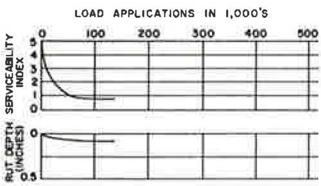
TEST PAVEMENT NO. C-2
WHEEL LOAD 3,200 LB.
BASE MATERIAL CR. STONE
BASE THICKNESS 3.0 IN.
SURFACE MATERIAL ASP. CONC.
SURFACE THICKNESS 1.0 IN.



TEST PAVEMENT NO. C-6
WHEEL LOAD 3,200 LB
BASE MATERIAL CR. STONE
BASE THICKNESS 6.0 IN.
SURFACE MATERIAL ASP. CONC.
SURFACE THICKNESS 2.0 IN.



TEST PAVEMENT NO. C-2A
WHEEL LOAD 3,200 LB.
BASE MATERIAL NONE
BASE THICKNESS 0
SURFACE MATERIAL ASP. CONC.
SURFACE THICKNESS 4.0 IN.



Model Study of Stresses in a Layered System

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This paper reports on a study of the variation of stresses in a layered system using thin metal plates of steel, copper, and aluminum, having different physical characteristics, arranged in stacks of different ways, using SR-4 strain gages of the rosette type, when loaded with a wheel load on a soil subgrade. The stacks of metal plates were loaded directly by means of a hard rubber wheel attached to the head of a universal testing machine.

From the strain measurements, the principal stresses together with the maximum shear stresses were evaluated. The studies show that the applied load being the same, the stress conditions in upper pavement layers are materially affected by variation of modulus of elasticity of the various layers of the pavement, the condition of the interface of the layers, and the direction of loading.

•THE HISTORY of development of highway transportation indicates the necessity for continuous highway improvements. To provide for the increasing volumes and the heavier wheel loads of highway traffic, in the most economical manner, the designer faces the following two essential problems: (a) correct assessment of the forces which the highway must resist, and (b) correct proportioning to resist those forces in the most effective and economical manner.

As far as is known to the author, there is very little information concerning experimental studies of stresses in a layered system.

The studies reported here were done primarily to establish the nature of stresses in the layered system of pavements. To accomplish this, the variation in stresses in thin metal plates of steel, copper, and aluminum were studied. The metal plates were used to study the change in stress due to four variables:

1. Varying arrangements of metal plates in stacks with regard to stiffness or modulus of elasticity.
2. Varying types of contact surfaces between the plates.
3. Variation in edge support conditions.
4. Variation in the direction of the applied load.

PROCEDURE

Model Studies

The pavement stress studies were made using metal plates as model pavement layers. Each plate was 23 in. wide, 25 in. long, and ⅛ in. thick. The research made comprises the following specific studies:

1. The effect of changes in plate arrangement. Three materials (steel, copper, and aluminum) were used with the following stacking arrangements from top to bottom: (a) steel, copper, aluminum; and (b) aluminum, copper, steel.
2. The effect of (a) polished, (b) oiled, and (c) roughened contact surfaces between the plates.
3. The effect of changes in the edge support condition. Plates were loaded with no

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edge restraint and with edge restraint provided on three sides at the rate of 200 lb per side.

4. The effect of load applied vertically and in an inclined manner.

Criteria for Choice of Metal Plates and Design of the Model

The metal plates were selected, because the materials have uniform physical properties and strain measurements can be made easily with the electric resistance gages. The system of metal plates studied does not satisfy the conditions of similitude. The length and width of the plates were fixed by the maximum size that could be accommodated in the testing machine. The thickness of $\frac{1}{8}$ in. for each plate was selected on the basis of obtaining reasonably high strains with relatively small loads in order not to overstress the subgrade. Results indicate that this decision on plate thickness was essentially correct. Not knowing the exact theoretical relations for stress and strain in the four pavement layers, it was not possible to set down the exact conditions that would be necessary for similar stresses and strains in model and prototype.

The plate system used is geometrically similar to various types of pavement-construction. As a model of particular pavement, the plate system is undoubtedly highly distorted. The contact conditions between the plates are also not similar to those existing in actual pavements. In spite of these deviations, it is considered that the stress patterns obtained will be similar to those existing in actual pavements.

Subgrade for Pavement Models

The model slabs and stacks of metal plates were supported on a soil subgrade of ML material (unified classification system) contained in a $25\frac{1}{2}$- by $23\frac{1}{2}$- by 23-in. wooden box, reinforced with an angle iron frame. The physical properties of the soil used for the subgrade are given in Table 1.

The top of the subgrade was covered with a layer of aluminum foil and the box was coated with bitumen inside to minimize moisture loss. A thin layer of less than $\frac{1}{4}$ in. of sand was placed over the aluminum foil to obtain good bedding for the strain gages placed on the bottom of the model pavement. The average moisture in the subgrade during the period of testing was found to be between 8 to 9 percent.

Manner of Applying Load

The load was applied to the metal plates in various arrangements by means of a small hard rubber wheel 4 in. in diameter attached to the head of a universal testing machine. A piece of sponge rubber 2 by 2 by $\frac{5}{8}$ in. was used under the loading wheel. In addition to the vertical loading, with a view to studying the effect of inclined loading on the distribution of surface stresses, the loading wheel attached to the head of the universal testing machine was deflected $11\frac{1}{2}°$ from the vertical by shims on one side of the seating plate of the loading wheel. Tests were run with the inclined wheel pointing towards the gage quadrant as well as pointing away from the gage quadrant both for free and restrained edge conditions. Figure 1 shows the subgrade box in the universal testing machine together with the loading arrangement. Figure 2 shows the relation of load to size of loaded area for the hard rubber wheel.

Method of Measuring Strains

Strains were measured in each case both at the top and bottom of the stack of metal plates by the use of SR-4 strain gages of the equiangular rosette type. The strain gages were mounted in one quadrant in each case. Figure 3 shows

TABLE 1

PHYSICAL PROPERTIES OF SUBGRADE SOIL

Property	Value
Liquid limit (%)	25.0
Plastic limit, (%)	21.8
Plasticity index (%)	3.2
Optimum moisture [a] (%)	13.6
Optimum density [a] (pcf)	109.1
California bearing ratio [b] (%)	40.0

[a]Standard Proctor. [b]At time of testing.

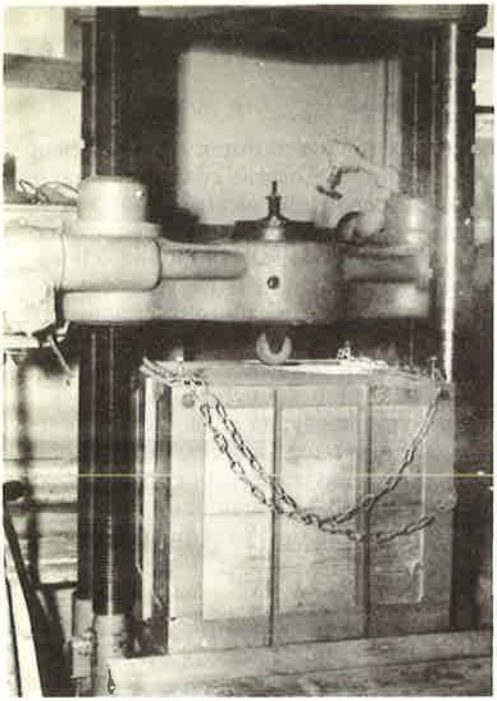

Figure 1. Subgrade box in universal testing machine together with loading wheel.

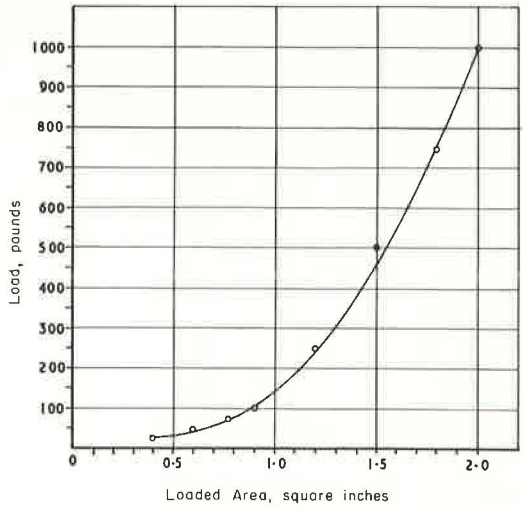

Figure 2. Relation of load to size of loaded area for hard rubber wheel.

the steel and aluminum plates on either side with mounted gages together with the scored copper plate in the middle.

Conversion of Strain Data to Unit Strain

The strain data were reduced to principal stresses and shear stresses by a graphical method described by Bossart and Brewer (1).

MODEL PAVEMENTS WITH THEIR PROPERTIES

The 25- by 23- by ⅛-in. metal plates were cut from rolled sheets of steel, copper, and aluminum. The modulus of elasticity and Poisson's ratio of the material in the plates were determined on standard tension test specimens prepared in accordance with ASTM procedure E 8 - 52 T, "Tension Testing of Metallic Materials." The modulus of elasticity, E, and Poisson's ratio, u, were calculated from measured longitudinal and lateral strains obtained with SR-4 electrical resistance strain gages during the tension tests. The values of modulus of elasticity and Poisson's ratio obtained from the various metal plates are given in Table 2. Figure 4 shows the specimens used for determining the modulus of elasticity and Poisson's ratio.

All the contact faces of the plates were

Figure 3. Steel and aluminum plates on either side with mounted gages; scored copper plate in middle.

TABLE 2

VALUES OF MODULUS OF ELASTICITY AND POISSON'S RATIO OBTAINED FOR THE VARIOUS METAL PLATES

Type of Plate	Modulus of Elasticity E (psi)	Poisson's Ratio
Aluminum	10.3×10^6	0.34
Copper	15.9×10^6	0.28
Steel	28.3×10^6	0.23

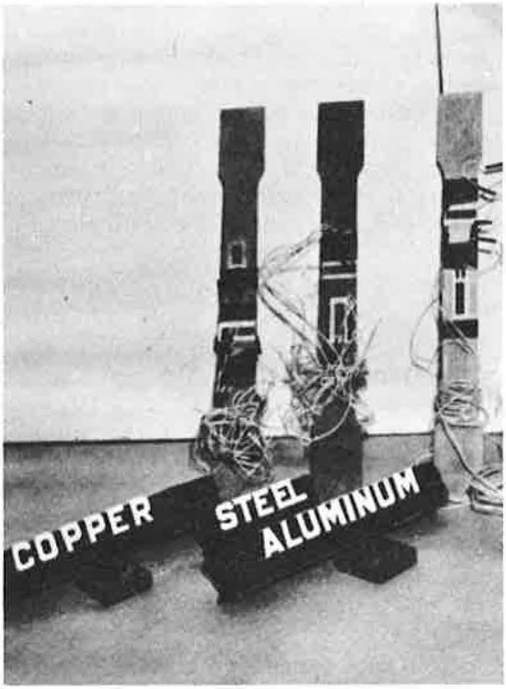

Figure 4. Metal specimens used for determining modulus of elasticity and Poisson's ratio.

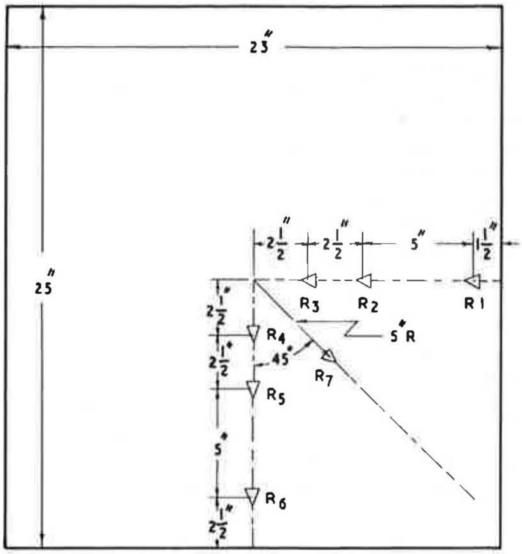

Figure 5. Location of SR-4 gages on aluminum plate.

polished to a high degree of surface smoothness for the initial tests, using emery cloth. For later tests with roughened surfaces, the contact surfaces of the plates were scored in two perpendicular directions, using a hard, pointed steel file. Figure 2. shows the scored copper plate in the middle.

APPLICATION OF SR-4 STRAIN ROSETTES

In this study, SR-4 equiangular rosettes of AR4-1 type were used for all strain studies on model pavements. In all the tests, the gages were located in one quadrant of the plates under test. The gages were

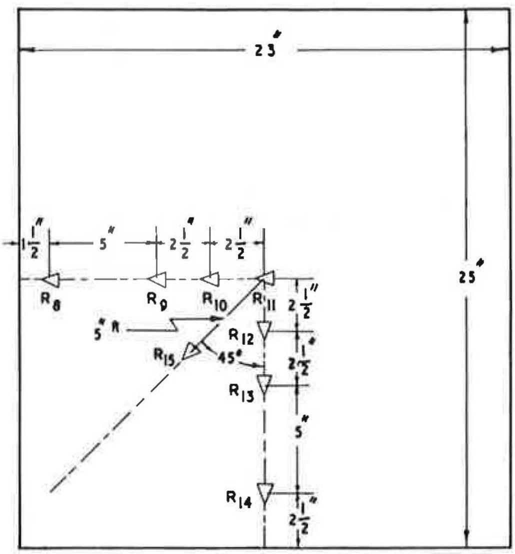

Figure 6. Location of SR-4 gages on steel plate.

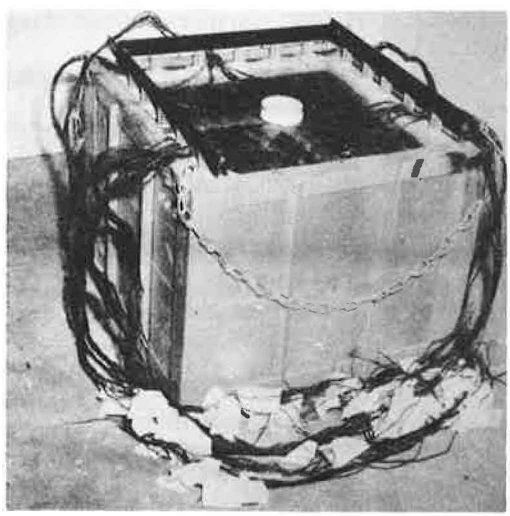

Figure 7. Model pavement in subgrade box with edges restrained on three sides.

placed along lines at the edges and along the diagonal of the quadrant. Typical locations of gages are shown in Figures 5 and 6. For the tests on the various stacks of metal plates, a central rosette was used only on the bottom plate, and it was necessary to install and remove this gage as the stacking arrangement was varied. For the remainder of the gage locations, the rosette gages as mounted at the beginning were used again and again for the different stacking arrangements tested.

The SR-4 gages were mounted on the metal plates adhering to the recommended procedure by the manufacturers for mounting the gages. The gages were waterproofed by applying hot petrocene wax over them. The steel and aluminum plates with gages mounted and ready for test are shown in Figure 2.

TEST PROCEDURE

The model pavement comprising the metal plates with mounted gages was carefully seated over the prepared subgrade and, at the same time, the leads of the gages on the bottom face were drawn through holes provided in the side.

For the restrained edge condition, precalibrated springs were used for restraining the edges on three sides of the model pavements. The edges were restrained by providing the required deflection in the calibrated springs by screwing down the nuts on the angle iron frame set over the springs. Figure 7 shows a model pavement in the subgrade box with edges restrained on three sides to simulate edge condition of a road pavement. The subgrade box was then loaded into the universal testing machine and carefully centered.

The following series of eight tests were conducted:

1. Aluminum plate on top and steel plate at bottom with copper plate in between, polished surfaces, vertical loading, tested once with free and once with restrained edges.
2. Steel plate on top and aluminum plate at bottom with copper in between, polished surfaces, vertical loading, tested once with free and once with restrained edges.
3. Steel plate on top and aluminum plate at bottom with copper plate in between, oiled surfaces, vertical loading, tested once with free and once with restrained edges.
4. Aluminum plate on top and steel plate at bottom, with copper plate in between, oiled surfaces, vertical loading, tested once with free and once with restrained edges.
5. Aluminum plate on top and steel plate at bottom, with copper plate in between, roughened surfaces, vertical loading, tested once with free and once with restrained edges.
6. Steel plate on top and aluminum plate at bottom, with copper plate in between, roughened surfaces, vertical loading, tested once with free and once with restrained edges.
7. Steel plate on top and aluminum plate at bottom with copper plate in between, roughened surfaces, tested with inclined load at $11\frac{1}{2}°$ to vertical (pointing away from gage quadrant), once with free and once with restrained edges.
8. Steel plate on top and aluminum plate at bottom with copper plate in between, roughened surfaces, tested with inclined load at $11\frac{1}{2}°$ to vertical (pointing towards the gage quadrant), once with free and once with restrained edges.

The model pavement slabs were loaded directly by means of a hard rubber wheel attached to the head of a universal testing machine.

The strain measurements were made from the SR-4 gages of equiangular rosette type with an SR-4 strain indicator. The active gage is mounted on the stressed model and the compensating gage is mounted on unstressed piece of the same material. The active and compensating gages are located close together so that both are subjected to the same temperature and the strains undergone by the various gages are obtained directly from the strain indicator.

RESULTS

The principal stresses and the maximum shear stresses for all the strains measured were calculated and plots for 750-lb load with a loaded area of 1.8 sq in. for the

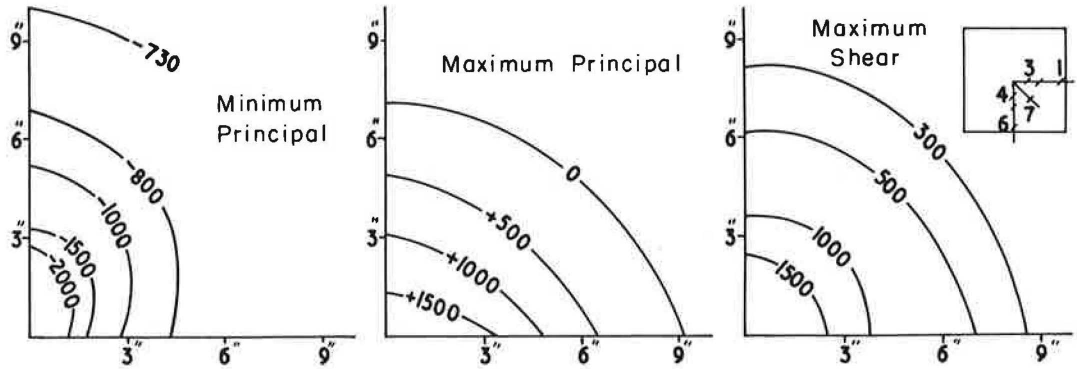

Figure 8. Test 1: stress contours (psi) on top aluminum plate with free edges.

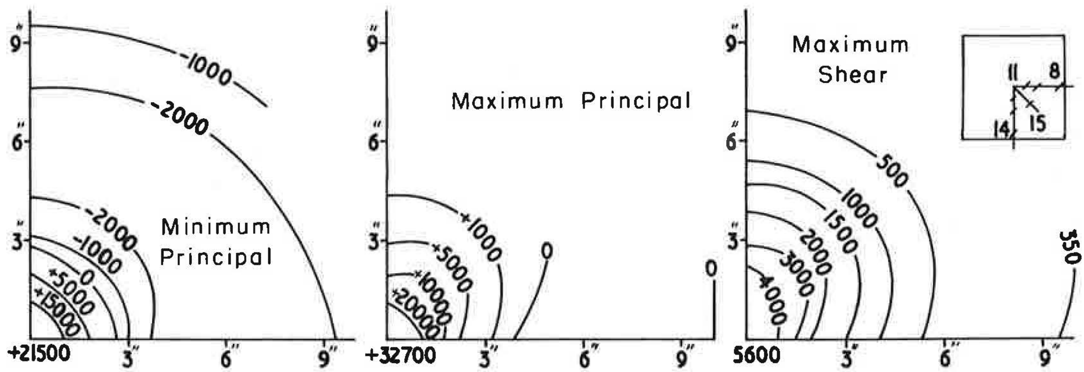

Figure 9. Test 1: stress contours (psi) on bottom steel plate with free edges.

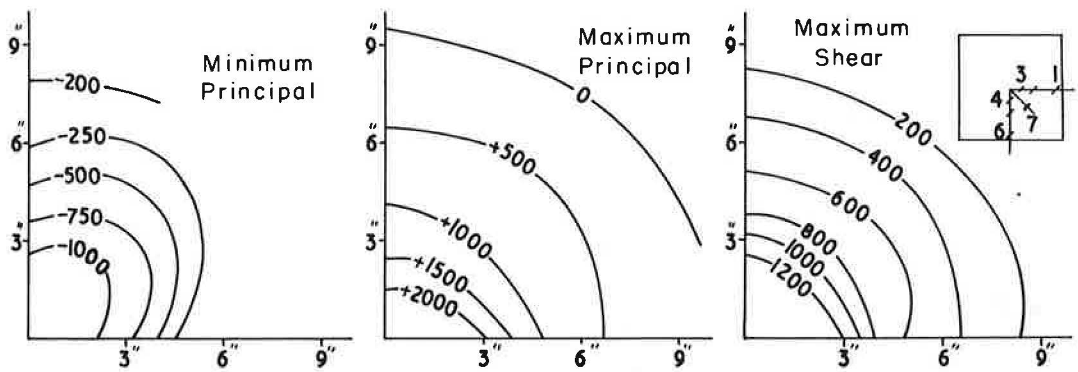

Figure 10. Test 1: stress contours (psi) on top aluminum plate with restrained edges.

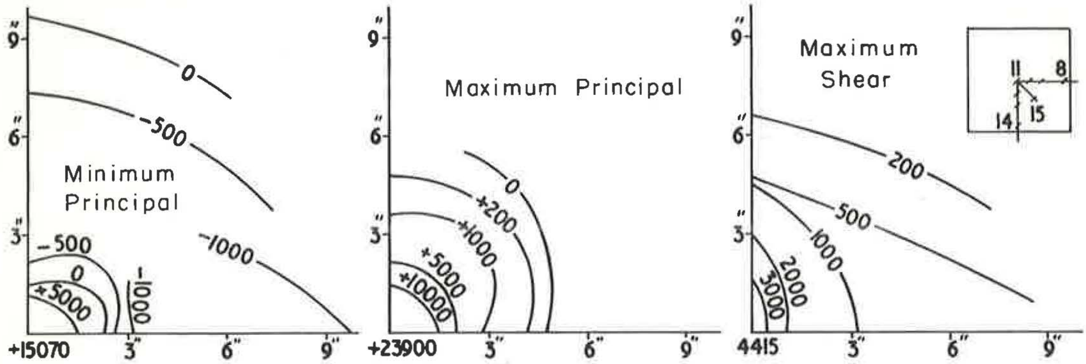

Figure 11. Test 1: stress contours (psi) on bottom steel plate with restrained edges.

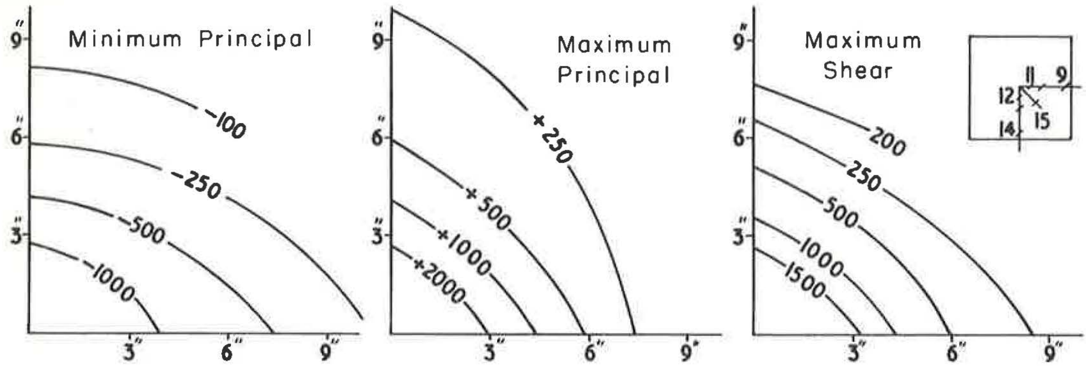

Figure 12. Test 2: stress contours (psi) on top steel plate with free edges.

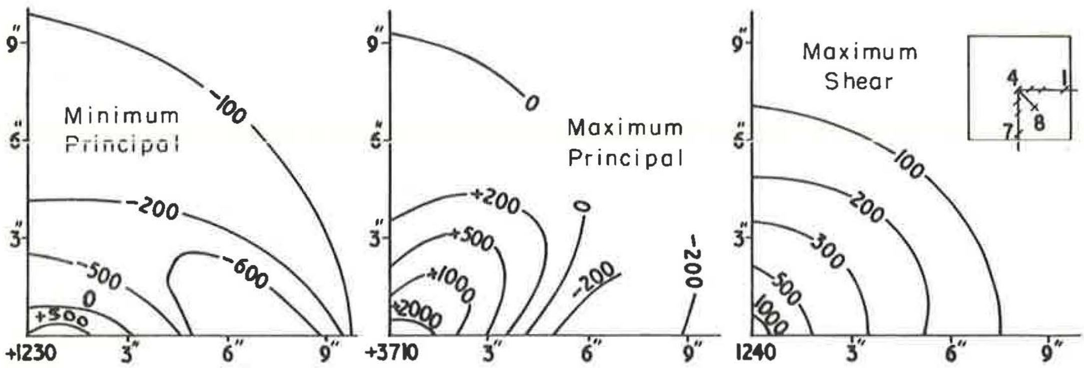

Figure 13. Test 2: stress contours (psi) on bottom aluminum plate with free edges.

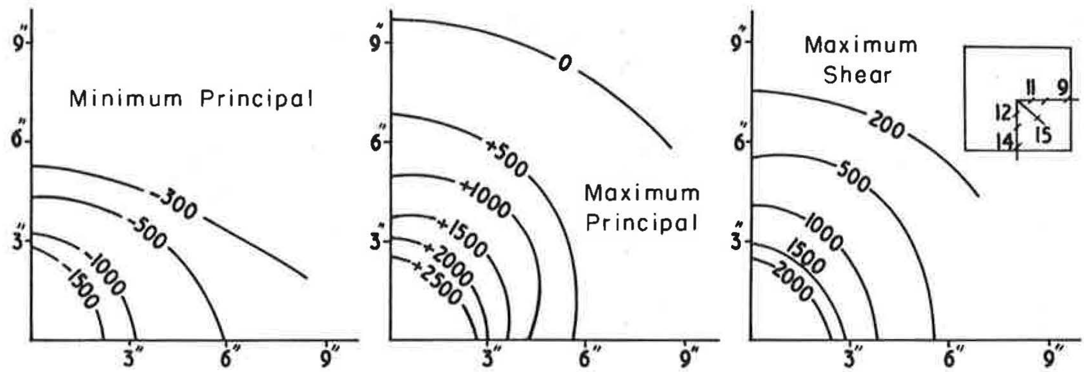

Figure 14. Test 2: stress contours (psi) on top steel plate with restrained edges.

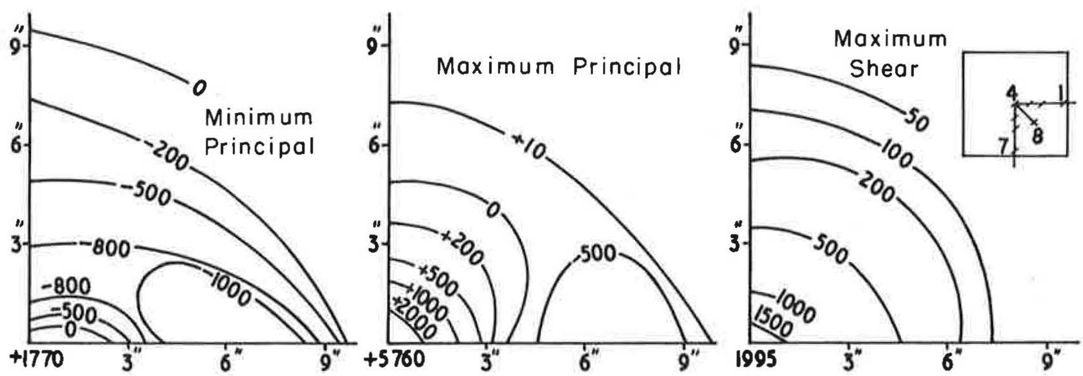

Figure 15. Test 2: stress contours (psi) on bottom aluminum plate with restrained edges.

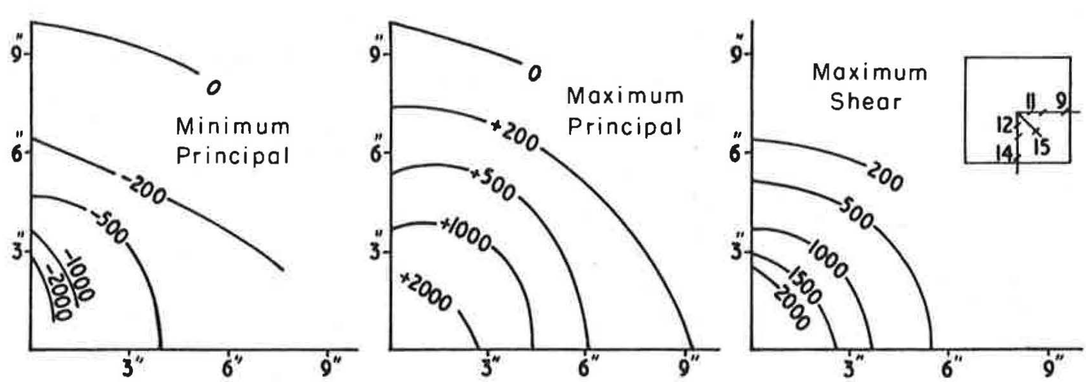

Figure 16. Test 3: stress contours (psi) on top steel plate with free edges.

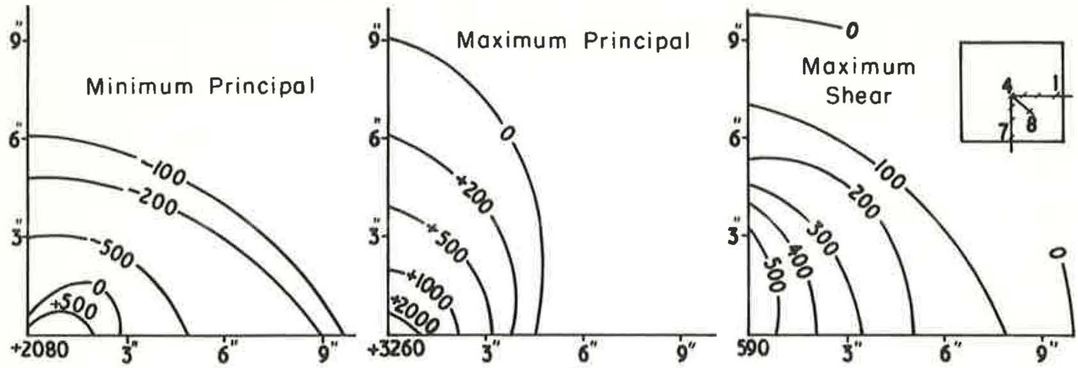

Figure 17. Test 3: stress contours (psi) on bottom aluminum plate with free edges.

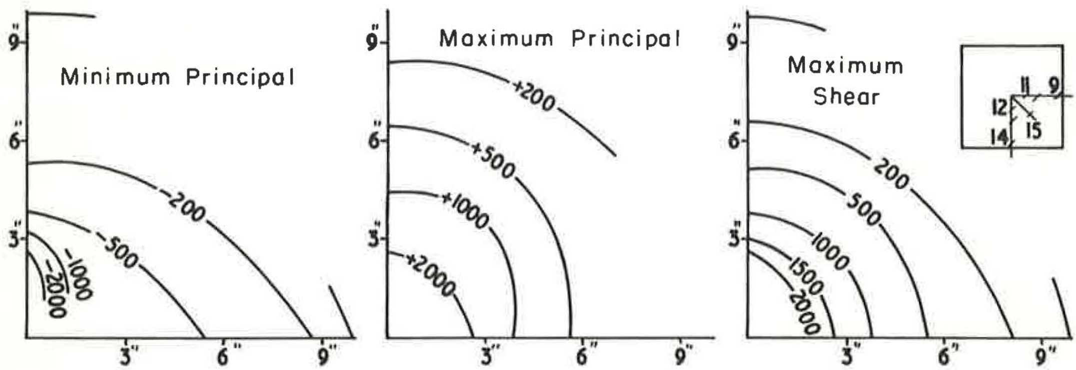

Figure 18. Test 3: stress contours (psi) on top steel plate with restrained edges.

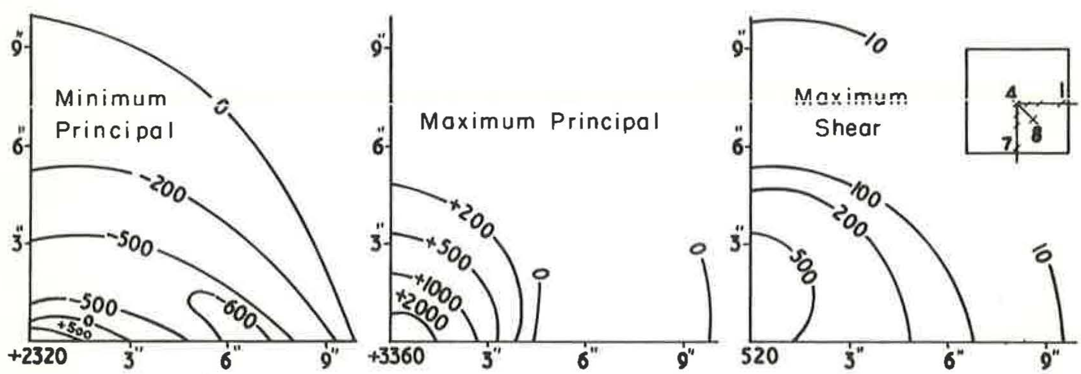

Figure 19. Test 3: stress contours (psi) on bottom aluminum plate with restrained edges.

maximum principal stress, minimum principal stress, and maximum shear stress for tests 1 to 8 are shown in Figures 8 to 39 for the condition of loading as detailed in the respective figures. Cross-hatching was used to show the restrained edge condition. The location of the gages mounted in each test is indicated in the small square on the right hand side for Figures 8 to 31.

The stresses measured in the center of the bottom plate can be directly compared to establish the fundamental relationships. Theoretically, for a circular loaded area, the maximum and minimum principal stresses at this point should be equal and the shear stress should be zero. The stress values in Figures 8 to 31 show material differences in the principal stresses at the center of the bottom plate. This indicates that the bending of the bottom plate is not symmetrical.

If the maximum principal stress at the center of the bottom plate is taken as the most significant stress then the general relationship between the applied load and this stress can be expressed by

$$S_b = CP\left(\frac{E_b}{E_t}\right)^n \tag{1}$$

in which

S_b = maximum principal stress at center of bottom plate (psi);
C = a constant;
P = applied load (lb);
E_b, E_t = moduli of elasticity of top and bottom plate, respectively;
n = exponential constant.

For the vertical loading condition, the three variations in roughness of contact surface, and the free and restrained edge conditions, the stress relations (computed for the 750-lb load) are found to be as follows:

1. Smooth Contact Surfaces

Free edges $$S_b = 14.7\ P\left(\frac{E_b}{E_t}\right)^{1.06} \tag{2}$$

Restrained edges $$S_b = 15.6\ P\left(\frac{E_b}{E_t}\right)^{0.70} \tag{3}$$

2. Oiled Contact Surfaces

Free edges $$S_b = 8.45\ P\left(\frac{E_b}{E_t}\right)^{0.68} \tag{4}$$

Restrained edges $$S_b = 8.2\ P\left(\frac{E_b}{E_t}\right)^{0.6} \tag{5}$$

3. Scored Contact Surfaces

Free edges $$S_b = 5.5\ P\left(\frac{E_b}{E_t}\right)^{0.44} \tag{6}$$

Restrained edges $$S_b = 5.0\ P\left(\frac{E_b}{E_t}\right)^{0.68} \tag{7}$$

Irrespective of the stacking arrangement of the metal plates, Figures 9, 11, 13, 15, 17, 19, 21, 23, 25, 27, 29, 31, 34, and 38 show that both the maximum and minimum principal stresses at the center of the bottom face of the bottom plate are tension for both vertical and inclined loading. Proceeding from the center to the edge, the maximum principal stress changes from tension to compression and then back to tension. At 2½ in. distance from the center, the maximum stress is tension in most of the cases; at 5 in. distance from the center, the maximum principal stress is compression. This indicates that the point of contraflexure on the bottom plate lies between 2½ and 5 in. from the center.

Because the gages were not operative when placed directly under the load, no stress measurements were made at the center of the top plate. Theoretically, both principal

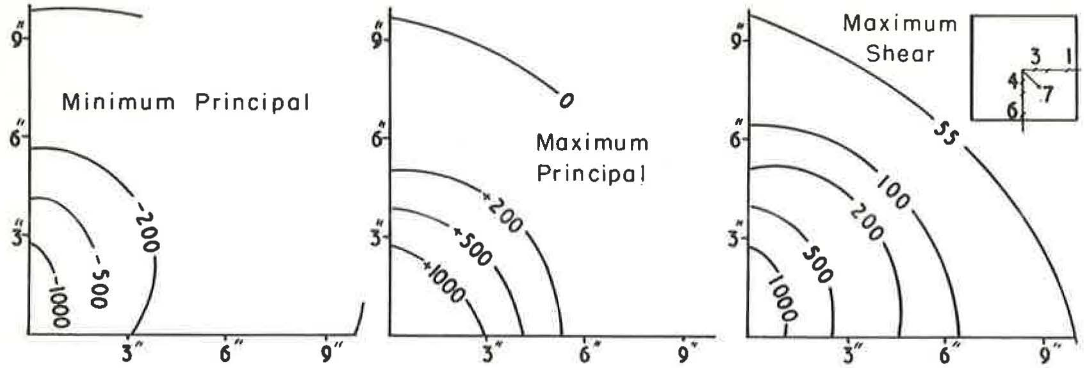

Figure 20. Test 4: stress contours (psi) on top aluminum plate with free edges.

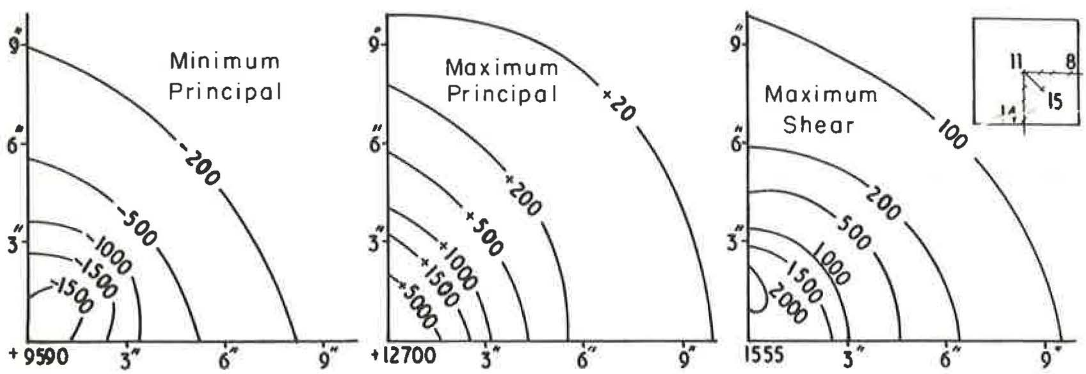

Figure 21. Test 4: stress contours (psi) on bottom steel plate with free edges.

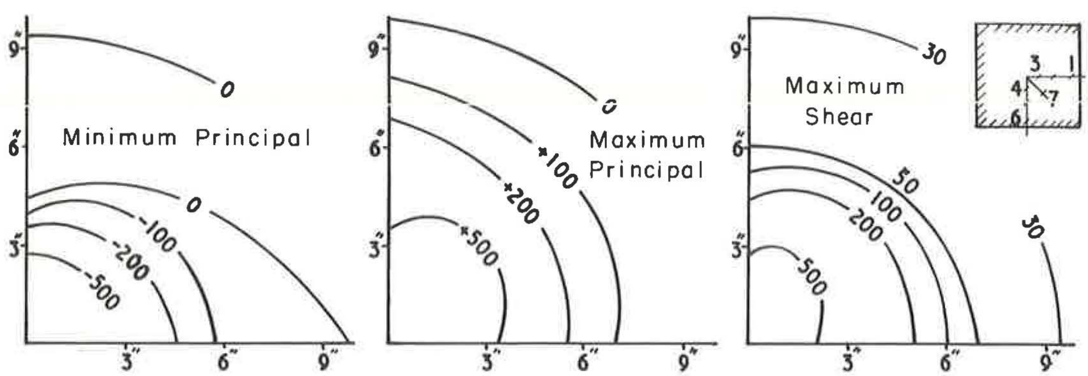

Figure 22. Test 4: stress contours (psi) on top aluminum plate with restrained edges.

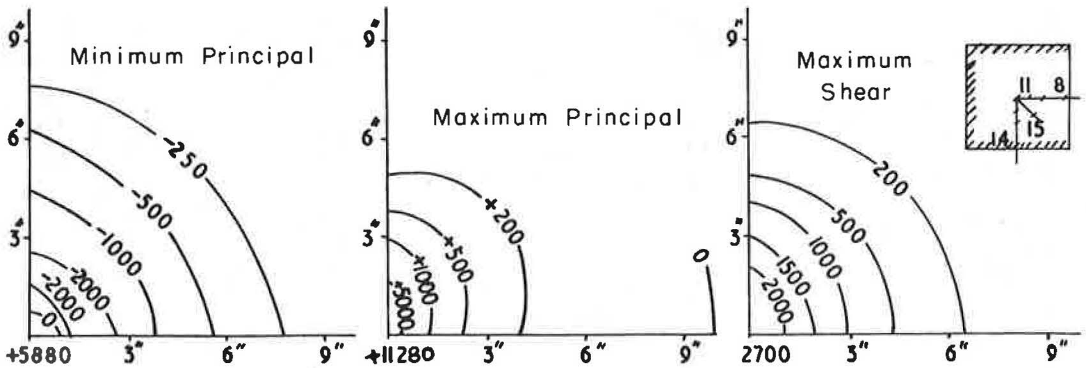

Figure 23. Test 4: stress contours (psi) on bottom steel plate with restrained edges.

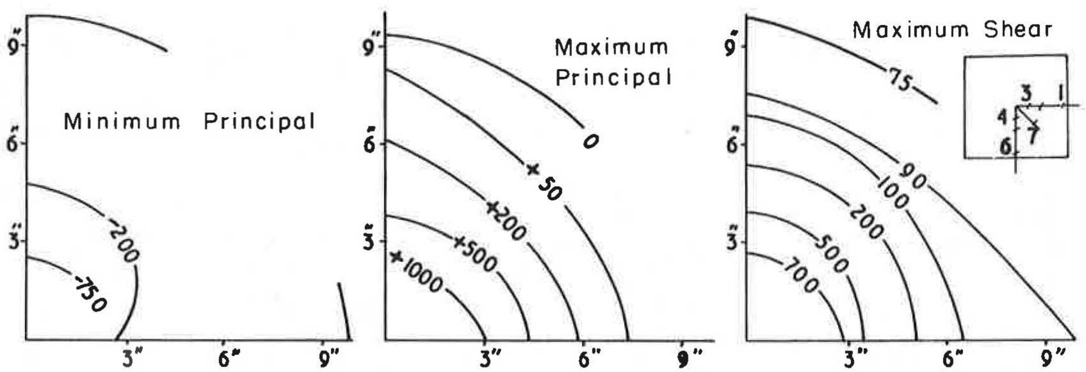

Figure 24. Test 5: stress contours (psi) on top aluminum plate with free edges.

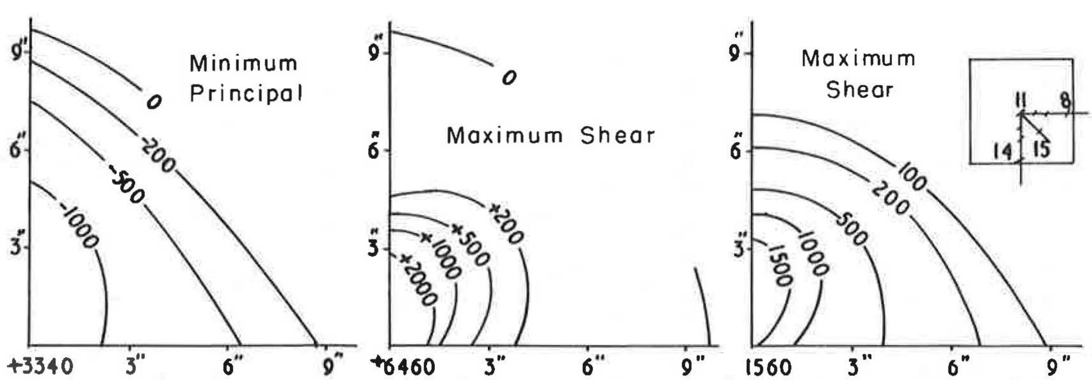

Figure 25. Test 5: stress contours (psi) on bottom steel plate with free edges.

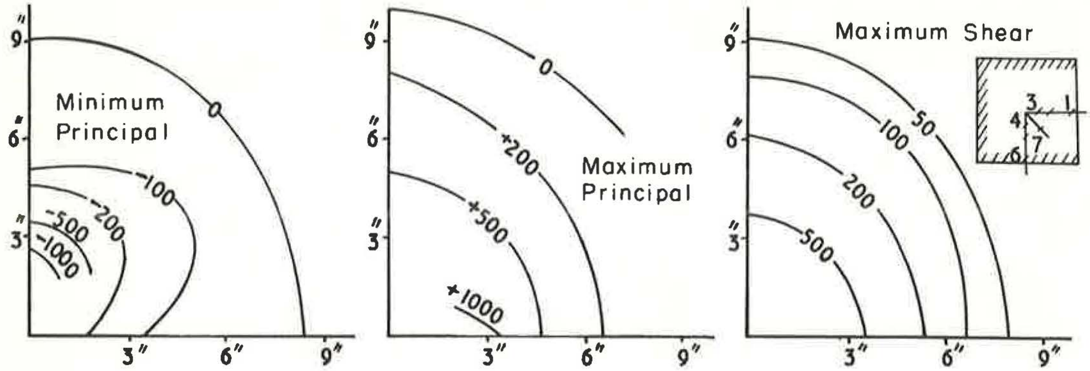

Figure 26. Test 5: stress contours (psi) on top aluminum plate with restrained edges.

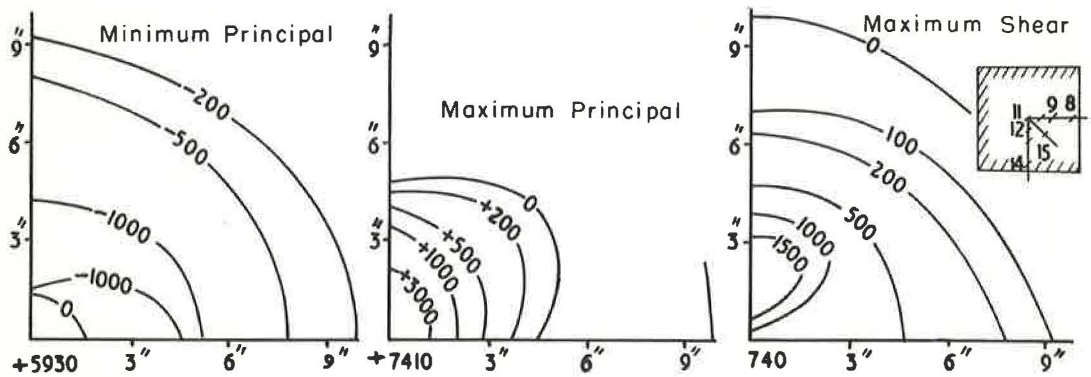

Figure 27. Test 5: stress contours (psi) on bottom steel plate with restrained edges.

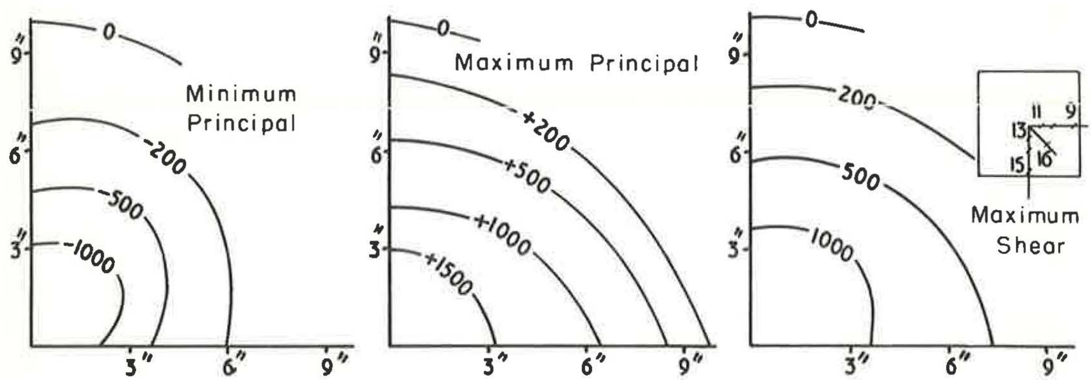

Figure 28. Test 6: stress contours (psi) on top steel plate with free edges.

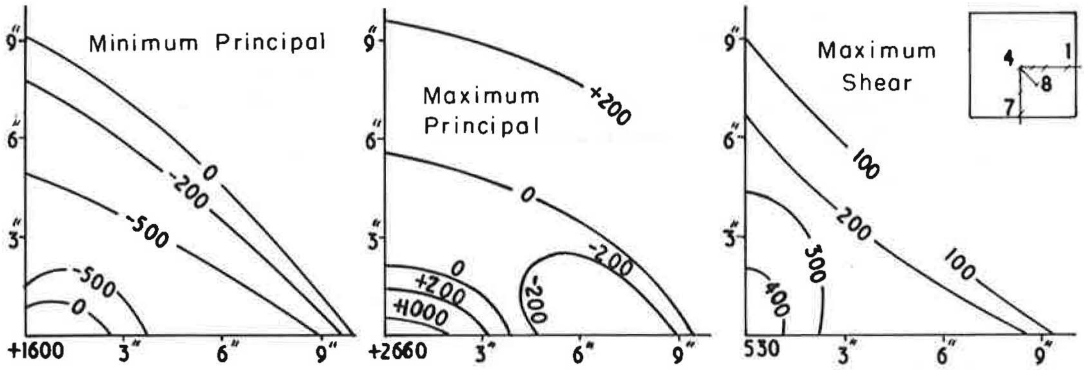

Figure 29. Test 6: stress contours (psi) on bottom aluminum plate with free edges.

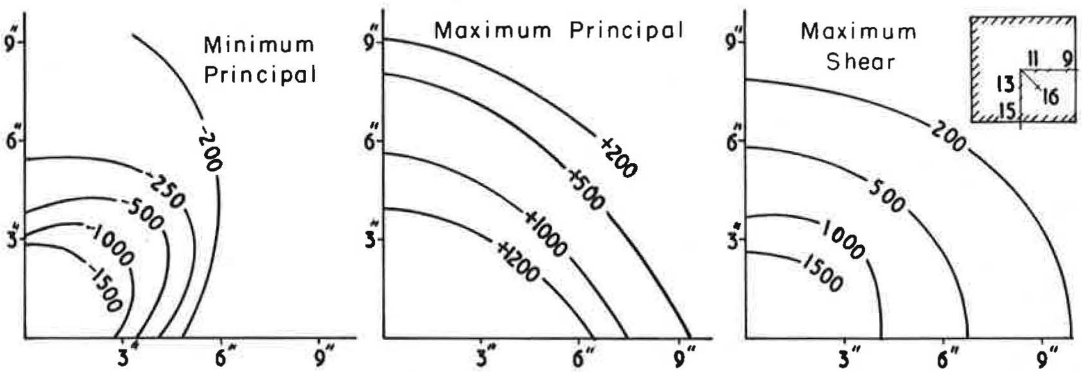

Figure 30. Test 6: stress contours (psi) on top steel plate with restrained edges.

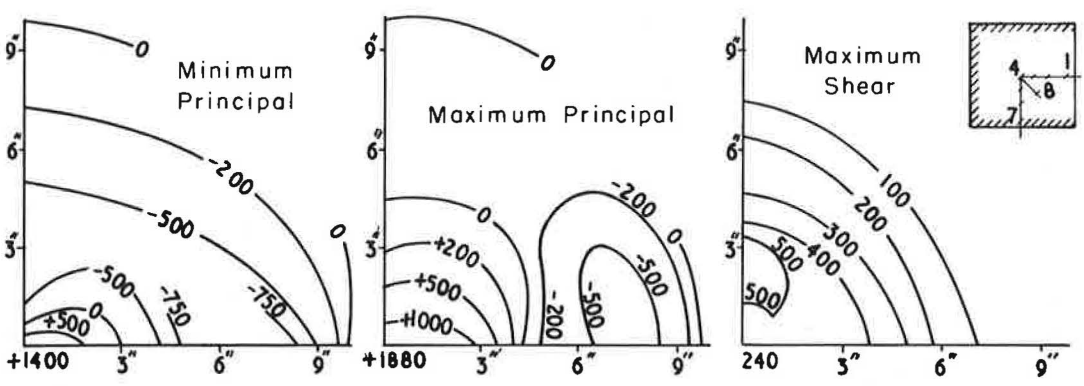

Figure 31. Test 6: stress contours (psi) on bottom aluminum plate with restrained edges.

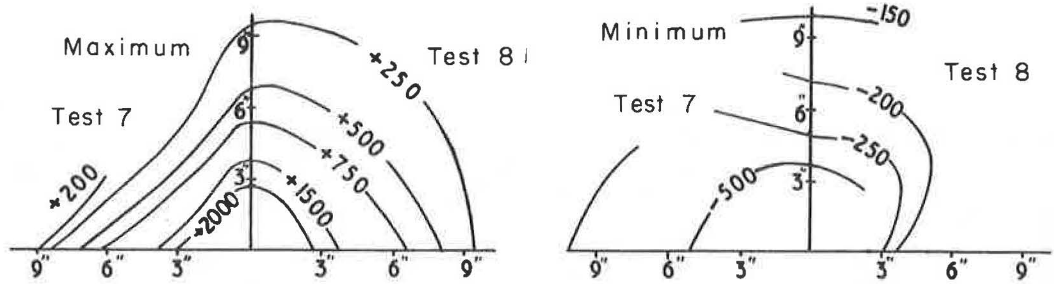

Figure 32. Tests 7 and 8: principal stress contours (psi) on top steel plate with free edges.

stresses in the top plate at the center will be compression and observation of plate deformations indicated that this was the stress condition existing. Figures 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32 and 36 show that, regardless of the nature of contact surfaces and edge conditions, the maximum principal stress at $2\frac{1}{2}$ in. from the center was tension in most cases. At a distance of 5 in. from the center, the maximum principal stress in the top plate was tension in all cases. This indicates that the point of contraflexure in the top plate was in most cases less than $2\frac{1}{2}$ in. from the center. The point of contraflexure on the top plate is thus closer to the center than the point of contraflexure on the bottom plate. This is in agreement with theory because the radius of curvature of the bottom plate will be greater than that of the top plate.

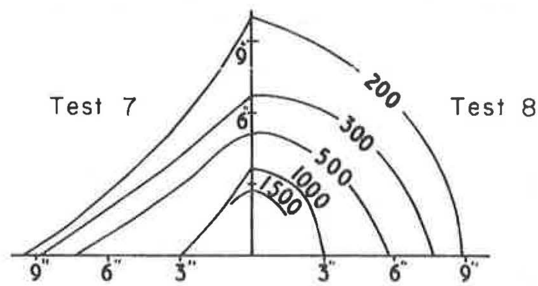

Figure 33. Tests 7 and 8: maximum shear stress contours (psi) on top steel plate with free edges.

Figures 8 to 38 show that tensile stresses may occur extensively throughout continuous pavement layers near the surface. This fact is of considerable importance in the design of the upper pavement layers because the paving materials normally used have very low resistance to tensile stresses.

The maximum shear stress is found to increase in the metal plates from the edge towards the center in all cases. It is possible, however, that shear stresses in excess of those measured may occur between the center and the gage $2\frac{1}{2}$ in. from the center. Because gages could not be mounted closer together, it was not possible to measure the shear stresses in this area.

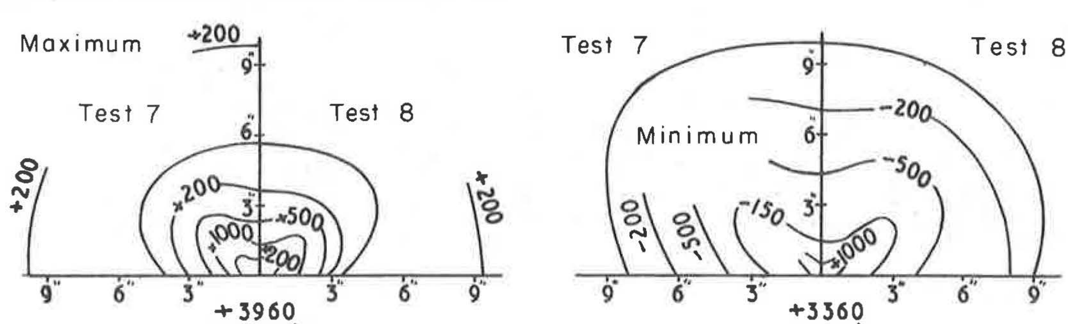

Figure 34. Tests 7 and 8: principal stress contours (psi) on bottom aluminum plate with free edges.

Effect of Edge Restraint

The restrained edge condition represents the condition existing in continuous pavement layers. Examination of Eqs. 3, 5, and 7 indicates that for the plates used and the restrained edge condition, the maximum principal stress at the center of the bottom plate is given very closely by

$$S_b = CP\left(\frac{E_b}{E_t}\right)^{0.66} \tag{8}$$

for all three types of contact surfaces. The exponents for the restrained edge condition vary between 0.60 and 0.70 and for the normal range of values of modulus of elasticity ratio, the use of the average value of 0.66 will not materially affect the stress values obtained. The constant, C, is variable with the contact surface. The restrained edge condition apparently insures a definite deformation pattern and a nearly fixed variation in stresses with changes in stiffness. This assumes that the effect of the center copper plate and the effect of the subgrade is the same for all loading conditions. The change in contact surfaces produces a material variation in the maximum bottom principal stress since, C, changes from 15.6 for smooth surfaces to 5.0 for scored surfaces.

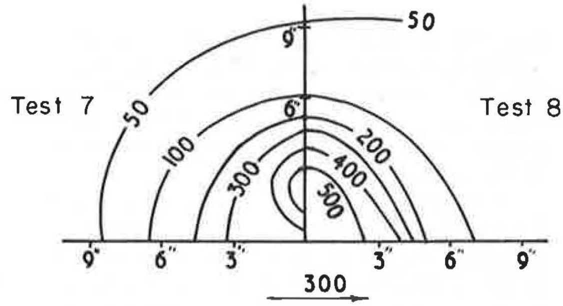

Figure 35. Tests 7 and 8: maximum shear stress contours (psi) on bottom aluminum plate with free edges.

Examination of Eqs. 2, 4, and 6 shows that for the plates used and the free edge condition, both variation in stiffness and variation in contact surface have variable effects on the maximum principal stress at the center of the bottom plate.

The maximum shear stresses at the various gages do not show any consistent variation with edge restraint except that near the edges of the plate, the shear stresses are smaller for the restrained edge condition.

The pattern of principal stresses near the edges of the plate is different for restrained and free edges as would be expected. Corresponding values are decreased in magnitude or change in sign for the restrained edge condition.

Effect of Variation in Contact Surfaces

Eqs. 2 through 7 show that the stress conditions in upper pavement layers are materially affected by variation in contact surface condition. The ability of the layer system to transmit stress across the contact surfaces between the layers is an important factor in fixing stress magnitude. Considering the fixed edge condition and a given ratio of modulus of elasticity, Eqs. 3, 5, and 7 show that for smooth, oiled, and scored surfaces the maximum principal tensile stress on the center of the bottom

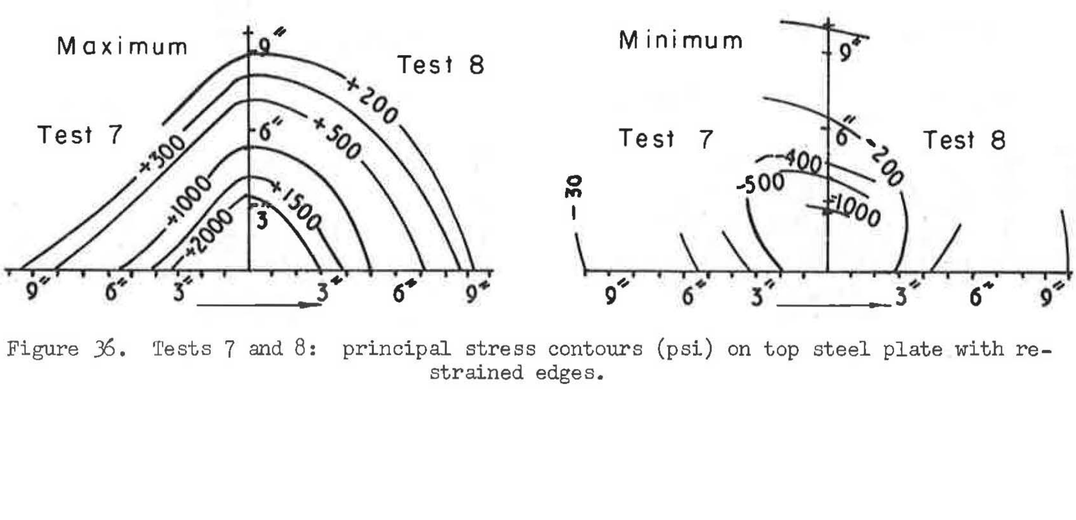

Figure 36. Tests 7 and 8: principal stress contours (psi) on top steel plate with restrained edges.

layer will vary in the order 15.6, 8.2, and 5.0, respectively. Hence, for continuous pavement layers, the conditions existing on the contact surfaces are very important in fixing the critical stresses in the layers.

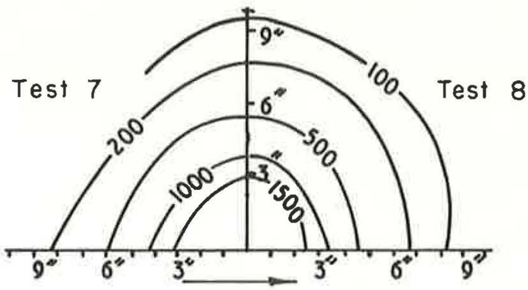

Figure 37. Tests 7 and 8: maximum shear stress contours (psi) on top steel plate with restrained edges.

The maximum shear stresses for both free and restrained edge conditions with smooth contact surfaces are consistently higher than for either oiled or scored contact surfaces. For the oiled contact surfaces and the scored contact surfaces both free and restrained edge conditions, there are no significant differences in maximum shear stresses except at central gages and gages 2½ in. from the center. Oiled contact surfaces showed higher maximum shear stresses at these gages near the center for both edge conditions. This indicates that surface contact conditions have an important effect on maximum shear stresses, particularly at the locations close to the load.

Effect of Variations in Stiffness

Figures 8 to 31 show that the plate with the highest modulus of elasticity, which is the stiffer plate, has the highest significant stresses regardless of the arrangement of the plates. This is true for all contact surfaces and both free and restrained edges. Stiffness is therefore of fundamental importance as would be anticipated from theoretical calculations. For free edge conditions, Eqs. 2, 4, and 6 show that the effect of stiffness varies as the contact surface conditions change whereas Eqs. 3, 5, and 7 show that the effect is nearly independent of the contact surfaces for the restrained edge condition.

Effect of Inclined Loading

Figures 32 to 39 inclusive show the stress patterns obtained with the inclined loading condition. Figures 28 to 31 inclusive show the stress patterns for the same plate arrangement for vertical load only. The vertical load in each case is 750 lb. The inclined loading condition also imposes a horizontal load equal to 0.2 of the vertical load or 150 lb.

When Figures 32 to 39 are compared with Figures 28 to 31, the major effect of the inclined loading shows an increase in the maximum principal stresses in areas near

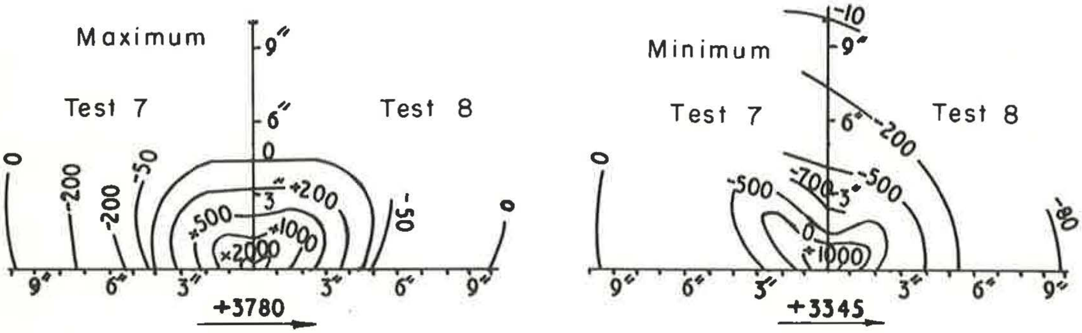

Figure 38. Tests 7 and 8: principal stress contours (psi) on bottom aluminum plate with restrained edges.

the load for both the free edge and restrained edge condition. The increase in the principal tensile stress at the center of the bottom plate is about 50 percent for the free edge condition and 100 percent for the restrained edge condition.

The maximum principal stresses for areas near the load on top of the top plate are increased in magnitude (greater tension) whereas the minimum principal stresses are decreased in magnitude (smaller compression) for the inclined load condition and both free and restrained edges. On the other hand, for the bottom of the bottom plate both maximum and minimum principal stresses increase in magnitude for the inclined load condition with both free and restrained edges. The stress patterns for maximum and minimum principal stresses are about the same for vertical and inclined loading.

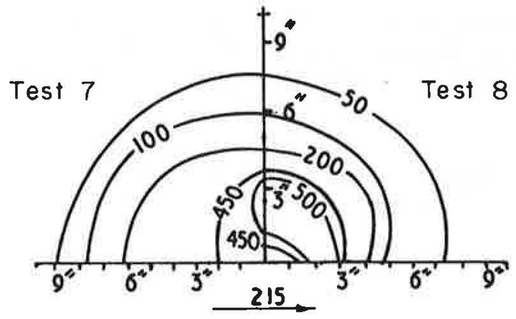

Figure 39. Tests 7 and 8: maximum shear stress contours (psi) on bottom aluminum plate with restrained edges.

The maximum shear stresses in the top plate are about the same in magnitude for both vertical and inclined loading and for free and restrained edges. The shear stress patterns for the top plate and the inclined loading condition bulge in the direction of the horizontal component. The maximum shear stresses on the bottom plate are increased in the direction of the horizontal component for both free and restrained edges.

The increased tensile and shear stresses due to inclined loading on actual pavement layers are a probable cause of excessive pavement deformation at locations such as street intersections, where much braking of vehicles occurs.

CONCLUSIONS

The studies indicate the following conclusions:

1. The general relationship between the applied load and the maximum principal stress at the center of the bottom layer in a layered system, with no variation in thickness of layers, can be expressed by

$$S_b = CP\left(\frac{E_b}{E_t}\right)^n$$

in which

S_b = maximum principal stress at center of bottom plate (psi);
C = constant, which is a variable with contact surface and other factors;
P = applied load;
E_b, E_t = moduli of elasticity of bottom and top plate, respectively (psi);
n = variable exponential constant.

2. The stress conditions in upper pavement layers are materially affected by variation in contact surface condition. The maximum stresses in the upper layers are higher for smooth contact surfaces than for rough contact surfaces for the same load.

3. In general increased tensile and shear stresses are noticed in the case of inclined loading when compared to the vertical loading, the total load being the same in both the cases.

ACKNOWLEDGMENTS

This research was a part of the dissertation submitted by the author to the Graduate School of the Agricultural and Mechanical College of Texas, in partial fulfillment of the requirements for the Degree of Doctor of Philosophy. The author wishes to express his gratitude to Fred. J. Benson, Dean of Engineering, under whose expert guidance the work was carried out. The author is grateful to S. R. Wright, Head of the Department of Civil Engineering; John B. Page, Dean of the Graduate School; and S. R. Mehra, Director, Central Road Research Institute, New Delhi, India, for according permission to publish this paper.

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Discussion

R. G. AHLVIN, Special Assistant, Soils Division, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss. — Mr. Subbaraju has an interesting approach to the study of pavement behavior and one that should shed light on certain of the effects of relative stiffness of upper pavement layers and on lateral tractions between these layers.

The author mentions a paucity of information relative to studies of stresses in layered systems. Though this is true, the Corps of Engineers has for quite a number of years been conducting research on the action and use of metal mats as expedient pavement elements. Certain of the information accumulated during these studies, and particularly some of the theoretical studies conducted in 1955, should be applicable to the study reported. Various references are included in the bibliography hereto, but particular reference is made to Waterways Experiment Station Technical Memorandum 3-418, "Theoretical Landing Mat Studies," October 1955. This report summarizes several separate research efforts directed toward gaining knowledge of the action of metal landing mats on soil subgrades. The report treats work by Gerald Pickett (7, 8, 9) on analytical developments involving thin layers on both elastic and Westergaard subgrades. In includes results of small-scale model tests of thin steel plates on a rubber subgrade (1, 2) which were carried out at the Corps of Engineers' Ohio River Division Laboratories. Also, this report presents results of plate load tests on instrumented metal landing mat on a heavy clay subgrade which were conducted at the Corps of Engineers' Waterways Experiment Station.

The analytical developments by Pickett might well be used to provide theoretical stresses for comparison with those measured by Mr. Subbaraju. The tests on steel plates on a rubber subgrade and on instrumented steel landing mats on a clay subgrade provide some directly comparable information. Some of this information is presented herein.

Figures 40 and 41 show strain in the top and bottom of a circular steel plate 0.018 in. thick and 12 in. in diameter on a rubber subgrade 12 in. thick loaded with 1-in. diameter circular loads. The data shown are to a degree directly comparable with those presented in the author's paper in Figures 8 and 9, 12 and 13, 16 and 17, 20 and 21, 24 and 25, and 28 and 29. The most direct comparisons possible are those with respect to the author's major principal stresses in both top and bottom of his stacked plates. These compare with the strains shown in Figures 40 and 41. The plots show tension in the bottom fiber to be about twice that in the top. The author's paper shows ratios between top and bottom fiber stresses other than 2 to 1, but differences are apparently due to differential stiffness in top and bottom plates as well as to variations in frictional restraints between plates.

Instrumented landing mats are shown in Figure 42, and some of the results of load testing are shown in Figures 43 and 44. Again, patterns here are in reasonable agreement with those developed by the author in regard to his major principal stresses. In this case, as in the author's case, bending is not symmetrical with respect to the mats or plates being loaded. The landing mat is geometrically irregular, whereas the author's stacked plates are, collectively, nonhomogeneous.

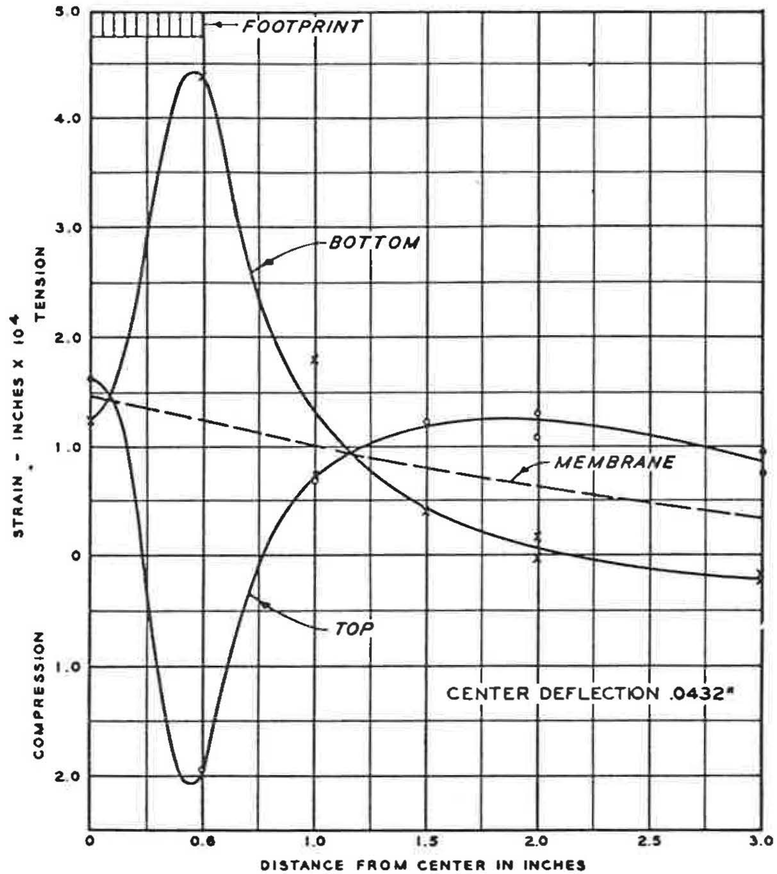

Figure 40. Load 28.8 lb applied at center of mat by rigid plate footprint 1 in. in diameter (strains in steel mat 0.018 by 12 in. in diameter).

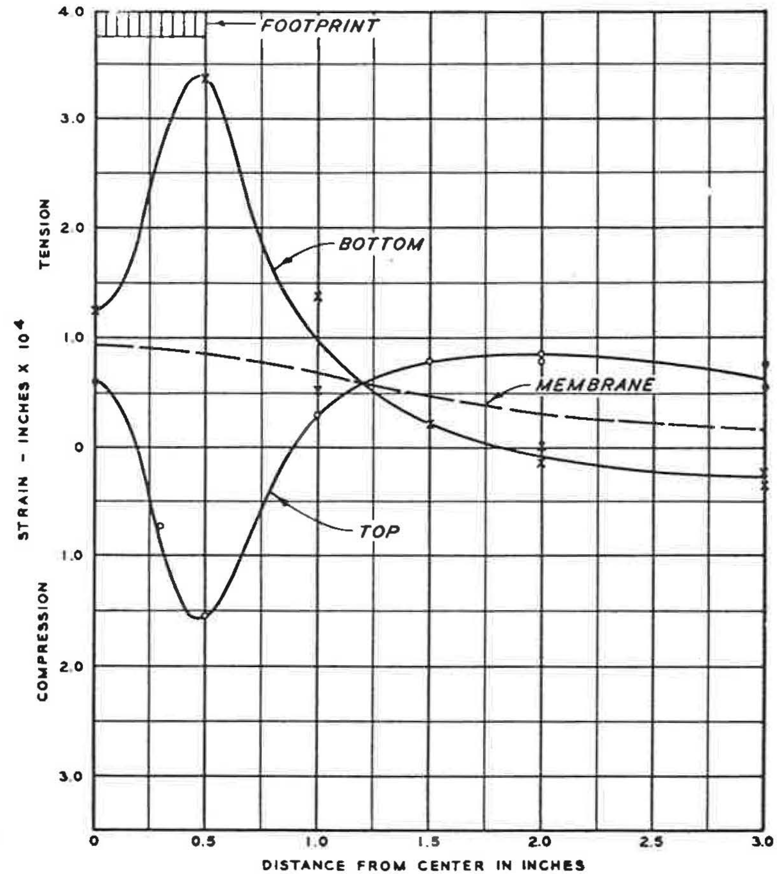

Figure 41. Load 18.8 lb applied at center of mat by tennis ball footprint 1 in. in diameter (strains in steel mat 0.018 by 12 in. in diameter).

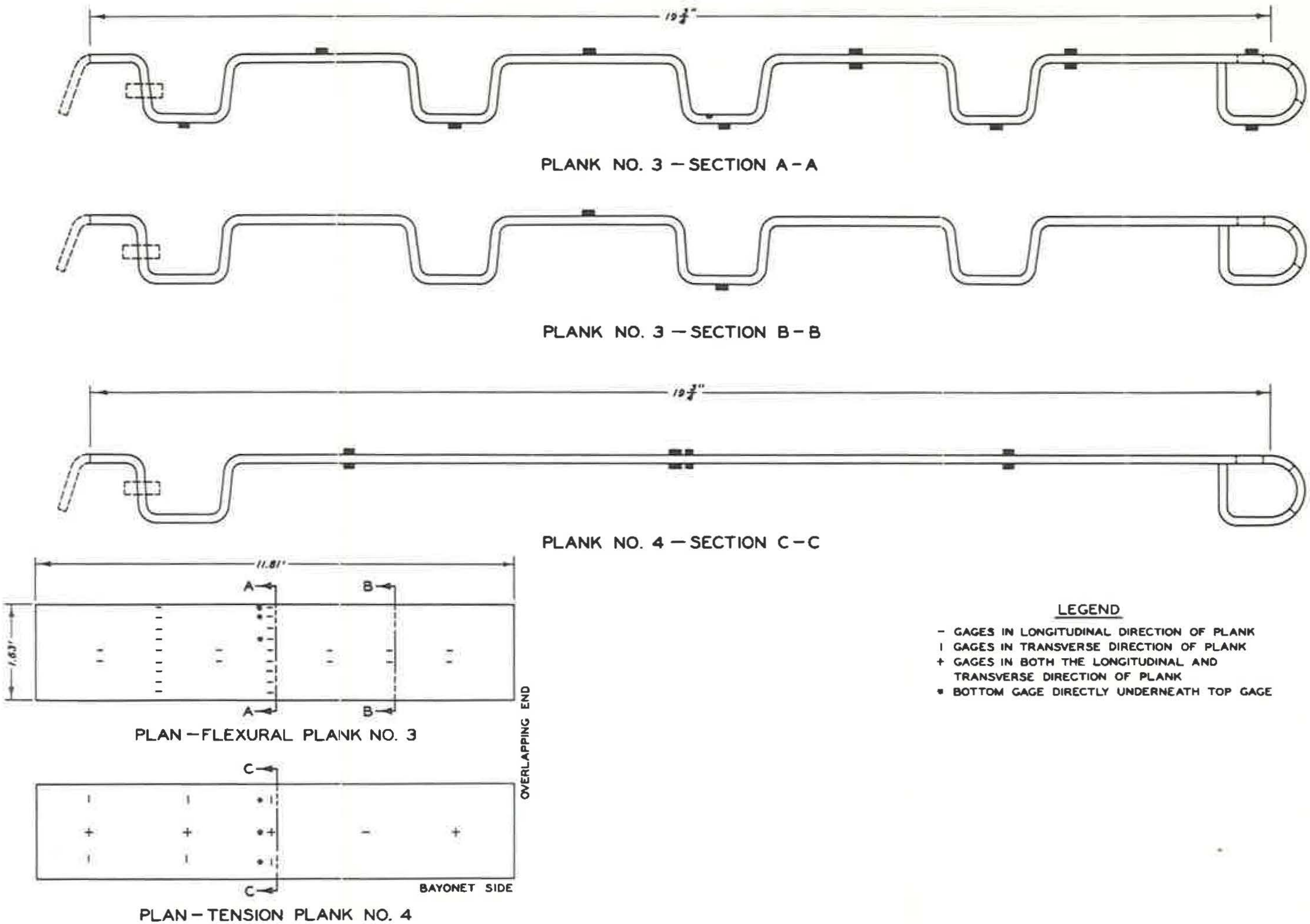

Figure 42. Typical cross-sections and plans of special M-8 mat showing location of strain gages.

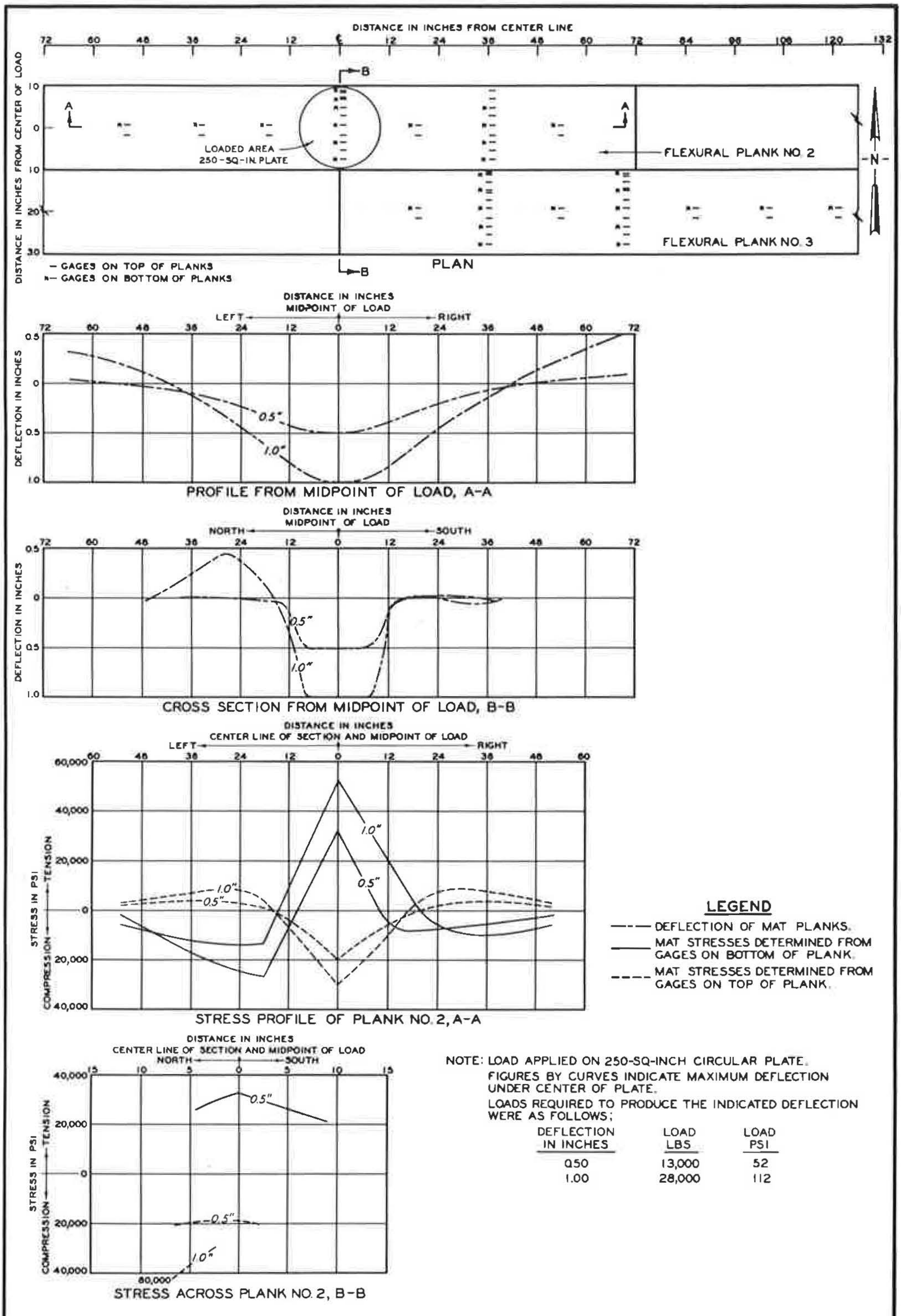

Figure 43. Mat strain and deflection data (mat in loose state).

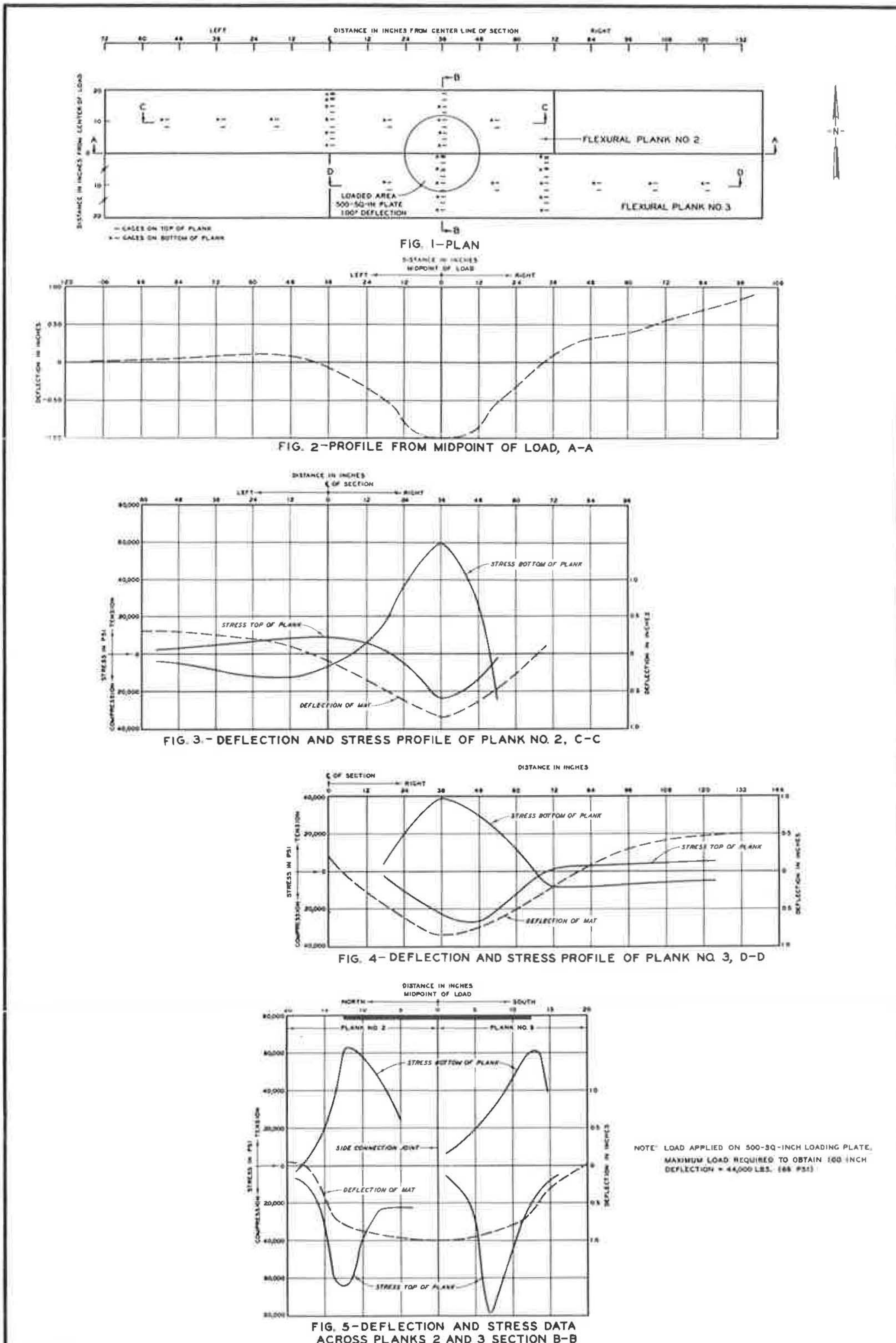

Figure 44. Mat strains and deflection data (mat in loose state).

It is not meant to infer in this discussion that there is a need to modify the author's analysis, but it is hoped that the author will find the data and references of value in his research.

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Deflections as an Indicator of Flexible Pavement Performance

F. P. NICHOLS, Jr., Highway Research Engineer, Virginia Council of Highway Investigation and Research, Charlottesville

•DEFLECTION MEASUREMENTS by means of the Benkelman beam have become increasingly important in evaluating the strength and load-carrying capacity of flexible pavements. The following report summarizes the results of tests performed in the spring of 1962 on 45 pavements in service in Virginia and of tests on 8 other pavements performed a year or two earlier. All but one of the 53 pavements reported were tested during the spring season, the period when subgrades are considered to be weakest. Also the paper presents a critical analysis of the effectiveness of certain commonly employed pavement design features in preventing excessive deflection and in improving performance.

An 18,000-lb single-axle load is employed in the measurement of deflections in Virginia. In the procedure now used, the rebound, or recovery from deflection, is measured rather than the deflection itself. At the start of the test, "the probe" (the tip of the lever arm) is inserted between the tires to a point exactly 2 ft ahead of the loaded wheel. The truck then moves forward slowly so that the maximum extensometer dial reading may be recorded as the load passes the point of measurement. Additional dial readings are made at intermediate points when the load is 2, 4, 6, and 9 ft beyond the probe. A final dial reading is taken after the test load has moved completely out of range of any possible effect on the measuring device. Figures 1 and 2 show the measurement procedure.

The value of total rebound deflection or recovery from deflection thus becomes the difference between the maximum dial reading and the final dial reading (multiplied by 2 to account for the mechanical advantage of the lever arm). The other values recorded are the differences between the dial readings when the load is at the various intermediate points, and the final dial reading when the load is out of range (again multiplied by 2). These values serve to define the approximate diameter of the "basin" deflected by the load, and indicate, in a qualitative sense at least, the degree to which the load is distributed to the underlying layers.

Using the preceding procedure, it is possible to make measurements in both wheelpaths at a great many sites in a single day. Test sites usually are spaced 50 ft apart in groups of five, thus covering a 200-ft length of highway per group. These groups are spaced at variable intervals, generally at least 1,000 ft apart; the number and spacing of groups on a given project are governed largely by the length of the project, by sight distances available to oncoming traffic, and by the frequency of superelevated curves. From 10 to 14 groups of test sites are established on a typical project and their locations are marked at the pavement edge with spray paint. Subsequent measurements on the same project are made at the exact same locations, insofar as it is possible to relocate the sites.

The familiar term, "deflection," still used frequently in the text, in all cases refers to the rebound value or recovery from deflection, determined in the manner just described.

DEFLECTION TEST RESULTS

The data obtained from both the 1962 measurement program and those of prior years are summarized in tabular form in terms of total project averages and ranges in group averages. These tables also show structural thicknesses, construction costs,

Paper sponsored by Committee on Flexible Pavement Design.

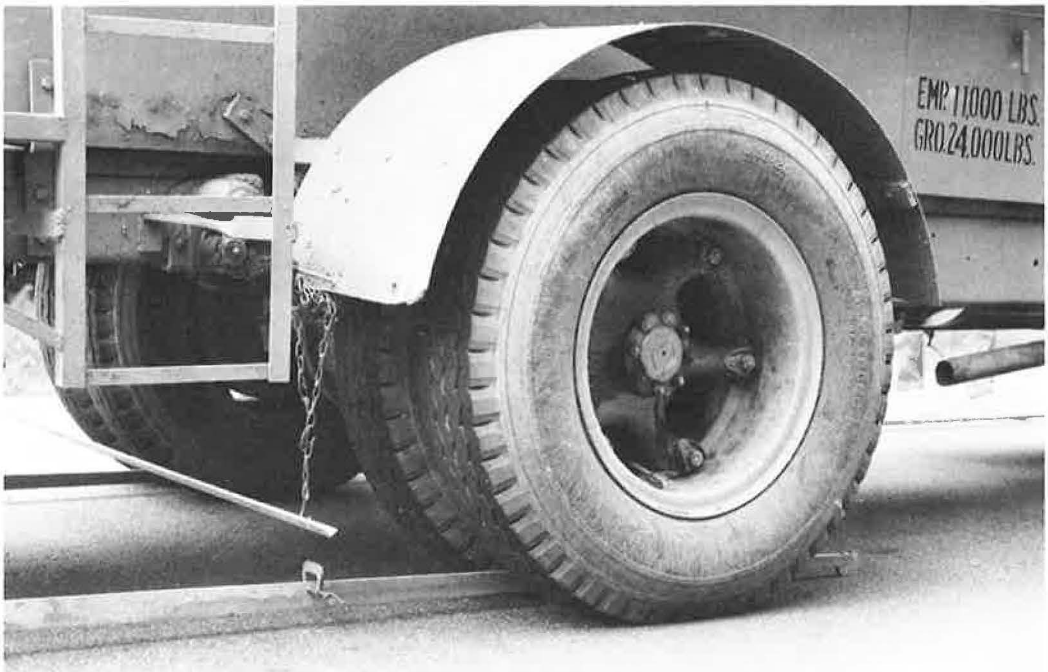

Figure 1. Test truck in initial position; points of measurement exactly 2 ft ahead of load wheels as indicated by clamps on beams.

the year the project was opened to traffic, and general remarks. The cost figures are discussed later.

Appendix A includes cross-section details for each project tested. Identification is provided by the code number corresponding to that shown in the first column of the tables.

In the "remarks" column is found first the average daily volume of trailer trucks and busses (TT & B) using the pavement in both directions, as reported in the Traffic and Planning Division's 1960-61 summary. Next is shown the soil area number, as defined in Appendix B. In general, these broad areas were numbered in the approximate order of suitability of the predominant soil types for highway subgrades, as seemed evident from analysis of condition survey data taken after the spring break up of 1948. Finally under "remarks" are found brief comments describing the performance of the pavement to date, including mention of average rebound deflection values which may have been determined in prior years.

Projects have been grouped for tabulation purposes in accordance with certain characteristics of their pavement designs such as the presence or absence of "black bases" or of lime or cement stabilization in either the subgrade or one of the structural components. Appendix C describes typical Virginia paving materials including the very popular black base mixes.

The first group of projects is distinguished by the inclusion in their designs of black bases, without any cement or lime stabilization in underlying layers. All the designs in this group include more than 6 in. total thickness of hot-mixed asphaltic concrete or sand asphalt. The essential data are summarized in Table 1.

The second group consists of pavements with untreated aggregate or water bound macadam bases and, again, no stabilization within the structure or in the subgrade. Though some of these have up to 3 in. of the H-3 (1) mix normally considered as "black base," the total thickness of asphaltic concrete is never as great as 6 in. The data for this group are summarized in Table 2.

Figure 2. Truck moving ahead and stopping with load wheels several feet ahead of point of measurement.

The third group consists of projects falling into the "black base" category (6 in. or greater total thickness of asphaltic concrete), but distinguished from those in the first group by the presence of a cement or lime stabilization of the subgrade. Data for this third group are given in Table 3. Total structural thicknesses include the stabilized subgrade layer, usually 6 in. thick. Only on project III-6 in this group was lime used.

Project average deflections in Table 3 are noticeably lower than most of those in Tables 1 and 2. Also the percent of deflection remaining as the load moves away is generally higher, indicating reduced bending of the surface layers and more favorable distribution of the load to the roadbed soil.

A fourth group is similar to the second in that the asphaltic concrete is less than 6 in. thick; it is similar to the third group in that the use of cement or lime stabilization of either subgrade or base is incorporated. Data for this group of projects may be found in Table 4. Even though some of these pavements were relatively inexpensive to construct, the effect of the cement or lime stabilization is indicated by the low deflections and good load distribution.

Still another listing is offered in Table 5 to summarize deflection data from the two experimental projects, one on Route 58 in Halifax County and the other on Route 360 in Charlotte and Prince Edward Counties. The variables on the first project have been described in other reports (1, 2) and are detailed again in the Appendix; essentially they are related to the thickness of asphaltic concrete in designs of the same total thickness, and no stabilization of subgrade or base is included. In the second project, comparisons

TABLE 1

FLEXIBLE PAVEMENT DEFLECTION SUMMARIES[a]

When Project Tested	Code	Proj. No.	District	Date Tested	Temp. Range (°F)	Rebound Deflection (thousandths in.) OWP/IWP[b] Proj. Avg.	Rebound Deflection Range	% Defl. Remaining at Indicated Distance Average[b] 2 Ft	6 Ft	9 Ft	Structural Thickness (in.) A. C.	Total	Construction Costs ($/lin ft) Actual	Adjusted	Year Opened to Traffic	TT & B	Soil Area	Remarks
In 1962	I-1	0081-011 -001	2	May	55-60	13/13	11-17/11-16	31/23	8/0	0/0	9.5	24.5	16.09	18.31	1960	828	7	Deflections measured in 1961 averaged 18/15, no defects noted.
	I-2	0029-071 -014-015	3	May	48-51	36/31	21-46/20-43	42/39	8/10	3/3	8.0	13.5	10.38	9.47	1955	269	11	Cracking and rutting became pronounced until 1½" resurfacing applied in 1959; few cracks noted since.
	I-3	0029-071 -022	3	May	79-83	37/41	28-44/27-57	32/39	5/2	3/0	7.0	15.0	-	10.45	1958	234	11	No defects noted.
	I-4	0029-071 -023-024	3	May	72-76	32/30	26-46/21-44	31/30	6/7	0/3	7.0	15.0	-	10.45	1958	234	11	No defects noted.
	I-5	0058-041 -028-032	3	March	76-79	49/38	32-65/25-54	39/34	2/3	0/0	7.0	13.0 -37.0	10.51 -12.10	10.04 -16.01	1958	986	6	Deflections measured in 1958 averaged 72/67. Cracking and rutting became pronounced until 1½" resurfacing applied in 1959. Few minor cracks again evident.
	I-6	0360-073 -002	3	April	64-68	25/20	14-39/12-27	56/45	4/5	0/0	9.0	15.0	9.55	11.89	1956	1,263	6	A few odd cracks, apparently not caused by traffic.
	I-7	0360-073 -009-010	3	April	58-60	70/66	20-173/23-148	51/44	14/12	0/2	9.0	17.0	14.35	13.83	1958	1,263	6	Badly cracked in places. Part resurfaced 1962.
	I-8	0060-020 -007	4	April	65-68	24/24	13-38/14-42	25/29	4/4	0/0	8.0	14.0	6.60	10.48	1956	190	2	Edges cracked; OK otherwise.
	I-9	0095-074 -001	4	April	83-85	20/17	17-24/14-19	30/35	0/0	0/0	9.5	21.5	15.03	19.09	1961	1,328	1	No defects noted.
	I-10	0301-074 -004	4	April	63-65	20/13	8-36/7-18	40/46	5/0	0/0	7.0	13.0	5.23	9.38	1956	1,328	1	Slippage cracks on original surface followed by general transverse cracking necessitated two complete re-surfacings by 1960, total 3". Transverse cracking still evident.
	I-11	0360-020 -013	4	May	85-88	62/58	27-124/30-98	21/34	2/4	0/2	9.0	15.0	8.44	10.91	1954	1,036	2	Pronounced cracking general except where resurfaced in 1961.
	I-12	0360-020 -019-027	4	April	83-85	42/38	26-67/21-60	26/26	5/5	0/0	9.0	15.0	10.39	11.38	1956	1,123	3	Occasional pronounced cracking noted.
	I-13	0017-030 -010	7	May	92-96	46/39	24-60/26-63	22/26	0/3	0/0	7.0	16.0 -22.0	11.86	12.82	1957	122	6	Cracking and rutting became pronounced until 1½" resurfacing applied in 1961.
	I-14	0029-023 -005	7	May	83-87	62/52	40-81/30-71	16/19	2/2	0/0	7.0	13.0	11.62	10.21	1959	319	11	Cracking and rutting became pronounced until 1½" resurfacing applied in 1961. Minor cracks again noted.
	I-15	0029-030 -002	7	May	73-76	40/35	29-48/25-40	30/34	2/3	0/0	9.0	12.0	9.54	10.62	1955	435	11	Part resurfaced in 1961. Balance generally cracked, some pronounced. Little effect on deflections noted from resurfacing.
Before 1962	I-16	0081-077 -001	2	Oct. 60	65-83	19/19	11-26/12-25	28/28	4/3	0/0	9.5	24.5	16.68	21.53	1960	1,119	7	No defects noted.
	I-17	0081-077 -008	2	Oct. 60	65-83	24/22	19-30/16-31	24/25	4/2	0/1	9.5	24.5	19.86	21.53	1960	1,119	7	No defects noted.
	I-18	0301-048 -002	6	March 1961	50-58	9/10	5-13/6-13	73/60	15/10	3/1	8.5	16.5 -18.5	5.52	8.00 -8.29	1951	984	1	Remarkable performance; practically no defects after 11 winters.
	I-19	0081-082 -021	8	Apr. 60	63-75	13/12	6-19/7-18	42/-	4/-	0/-	9.5	27.5	16.67	21.11	1960	907	7	No defects noted.
	I-20	0081-082 -017	8	Apr. 60	63-75	22/22	17-22/11-25	37/-	4/-	0/-	9.5	27.5	17.88	21.11	1960	956	7	No defects noted.

[a]Black bases—no stabilization in subgrade or subbase.

[b]Figures to left of slash for outer wheelpath, to right for inner.

TABLE 2

FLEXIBLE PAVEMENT DEFLECTION SUMMARIES[a]

Code	Project No.	District	Date Tested	Temp. Range (° F)	Rebound Deflection (thousandths in.) OWP/IWP[b]		% Defl. Remaining at Indicated Distance Average[b]			Structural Thickness (in.)		Construction Costs ($/lin ft)		Year Opened to Traffic	TT & B	Soil Area	Remarks
					Proj. Avg.	Range	2 Ft	6 Ft	9 Ft	A. C.	Total	Actual	Adjusted				
II-1	0460-035 -012	2	May 1962	60-63	24/25	14-38/16-35	25/24	4/4	0/0	1.0 (M. I. P.)	13.0	5.52	6.21	1956	29	7	Deflections measured in August 1959 averaged 25/21; in May 1960, 26/30. General surface deterioration until 1½" resurfacing applied in 1960. No defects since.
II-2	0058-041 -014	3	April 1962	46-52	52/42	20-78/16-63	37/26	4/2	0/0	2.0	37.0	8.88	9.84	1956	986	6	Deflections measured in 1957 averaged 37/35; in 1958, 56/47. General pronounced cracking in both lanes. Has been resurfaced since 1962 tests made.
II-3	0058-041 -015	3	March 1962	45-80	48/37	16-69/14-62	35/35	4/5	0/0	2.5	14.5 -30.5	9.08 -9.75	9.72 -11.96	1957	986	6	Deflections measured in 1957 averaged 65/62; in 1958, 52/49. Pronounced alligator cracking over most of project soon after completion; 1½" resurfacing applied in 1957. Considerable cracking has reappeared but riding quality not impaired appreciably.
II-4	0017-030 -015	7	May 1962	60-65	60/61	32-111/24-92	17/24	2/2	0/0	4.5	23.5 -29.5	11.58 -12.55	11.57 -13.03	1958	122	6	General minor alligator cracking, occasionally pronounced. Some rutting.
II-5	0020-068 -010-013	7	May 1962	68-70	35/33	27-50/25-48	31/33	6/6	3/3	1.0 (M. I. P.)	13.0	6.52	5.31	1957	60	6	General pronounced surface distress after first winter. 1½" resurfacing applied in 1958. Many patches and areas of pronounced cracking again evident.
II-6	0020-068 -101, C-501	7	May 1962	84-87	38/35	21-76/14-68	45/40	5/6	3/3	4.0	12.0 — +	--	10.61	1962	60	6	New project. No defects.
II-7	0029-056 -102, C-1	7	May 1962	90-92	53/47	31-68/28-66	23/23	4/4	2/2	4.5	24.5 -48.5	8.92 -10.26	8.73 -17.07	1961	352	6	Deflections measured in 1961 averaged 44/41. General minor alligator cracking, some surface ravelling; spotty texture.
II-8	0017-030 -002	7	May 1962	88-91	44/44	24-64/22-65	32/32	5/2	2/0	2.0	11.0 -27.0	7.18 -8.59	6.85 -8.29	1952	271	6	Surface cracking became pronounced until 1½" resurfacing applied in 1958. Cracks reappeared until seal applied in 1961.
II-9	0029-076 -007;-030-001	7	May 1962	67-70	26/20	21-31/15-32	19/25	4/5	0/0	3.0 (Pen. Mac.)	13.0 -27.0	8.47 -9.57	8.21 -10.93	1953	435	11	Original M. I. P. surface developed minor cracking and raveling until 1½" resurfacing applied in 1958. Few defects since.

[a]Untreated aggregate or water-bound macadam bases; no stabilization in subgrade or subbases.
[b]Figures to left of slash for outer wheelpath, to right for inner.

TABLE 3

FLEXIBLE PAVEMENT DEFLECTION SUMMARIES[a]

When Project Tested	Code	Proj. No.	District	Date Tested	Temp. Range (°F)	Rebound Deflection (thousandths in.) OWP/IWP[b]		% Defl. Remaining at Indicated Distance Average[b]			Structural Thickness (in.)		Construction Costs ($/lin ft)		Year Opened to Traffic	TT & B	Soil Area	Remarks
						Proj. Avg.	Range	2 Ft	6 Ft	9 Ft	A. C.	Total	Actual	Adjusted				
In 1962	III-1	0015-019 -101, C-2 (Heavy)	3	April	40-43	16/13	12-19/11-14	50/38	6/0	0/0	7.0	19.0	13.73	12.92	1960	1,211	9	Deflections measured in 1961 averaged 16/18. No defects noted.
	III-2	0017-030 -003, C-501	7	May	72-74	26/24	18-34/15-32	54/42	8/4	4/0	7.0	21.0	16.21	15.89	1962	271	11	No defects.
	III-3	0017-030 -003, C-502	7	May	65-68	20/18	15-25/15-21	50/44	10/6	0/0	7.0	21.0	18.91	15.89	Incomp.	271	11	No defects; portion of project not open to traffic.
	III-4	0017-030 -008	7	May	80-82	18/15	7-30/8-24	50/47	11/13	6/4	7.0	21.0	15.56	16.02	1959	271	11,6	Deflections measured in 1961 averaged 16/14. No defects noted.
	III-5	0066-029 -101;-076-101	7	May	85-88	22/19	16-30/12-25	27/26	5/5	0/0	9.5	21.5	16.77	21.35	1962	367	11	No defects.
	III-6	0050-034 -101, C-501	8	June	65-70	18/21	15-21/17-24	27/25	3/3	0/0	7.0	17.0 -23.0	10.69 -12.21	11.82 -14.09	1962	229	7,4	Deflections reduced only slightly by addition of lime treatment to subgrade on parts of project.
Before 1962	III-7	0220-044 -030	2	April 1960		35/32	20-51/22-44	58/—	28/—	12/—	7.0	23.0	14.40	13.75	1959	797	8	No defects noted.
	III-8	0123-029 -012	7	March 1961	52-55	33/23	23-42/17-30	56/56	14/18	4/9	7.0	21.0	14.75	14.40	1960	123	6	No defects noted.
	III-9	0236-029 -007	7	March 1961	58-61	15/14	10-18/10-16	55/54	13/15	3/3	7.0	21.0	13.90	15.30	1960	647	6	No defects noted.

[a] Black base with stabilization in subgrade.
[b] Figures to left of slash for outer wheelpath, to right for inner.

TABLE 4

FLEXIBLE PAVEMENT DEFLECTION SUMMARIES[a]

Code	Project No.	Dis-trict	Date Tested	Temp. Range (°F)	Rebound Deflection (thousandths in.) OWP/IWP[b] Proj. Avg.	Range	% Defl. Remaining at Indicated Distance Average[b] 2 Ft	6 Ft	9 Ft	Structural Thickness (in.) A. C.	Total	Construction Costs ($/lin ft) Actual	Ad-justed	Year Opened to Traffic	TT & B	Soil Area	Remarks
IV-1	0460-035 -101, C-1, C-2	2	May 62	60-63	14/17	10-20/14-22	29/24	7/0	0/0	4.0[c]	20.0[d]	8.99	9.94	1961	29	7	No defects noted.
IV-2	0117-080 -002, C-1	2	May 62	70-73	19/18	13-34/14-27	42/39	5/5	0/0	5.5	15.5[e]	12.53	13.09	1961	161	7	No defects noted.
IV-3	0017-080 -002, C-502	2	May 62	74-77	19/17	13-28/10-28	46/47	5/8	0/2	5.5	15.5[f]	13.67	13.09	Incomp.	178	7	No defects. No appreciable difference between deflections on regular and special design sections.
IV-4	0015-019 -101, C-2 (Light)	3	April 62	40-43	20/14	13-28/6-19	50/43	10/7	0/0	4.5	16.5	11.22	10.80	1960	55	9	No defects noted.
IV-5	0015-058 -101, C-501	4	April 62	52-58	20/13	15-24/9-15	30/31	5/8	0/0	4.5	16.5	14.05	13.40	1962	55	9	No defects noted.
IV-6	0058-071 -020	3	April 62	72-75	23/22	19-32/16-32	35/45	9/14	4/5	4.0	19.0	9.42	10.25	1961	245	6, 11	No defects noted
IV-7	0060-746 HS-1, IS-1	3	April 62	51-55	18/17	10-42/6-31	67/65	17/24	6/6	2.5	14.5[g]	6.42	7.63	1948	182	6	Only noticeable defect has been general transverse shrinkage cracking, becoming pronounced after some years. Pavement sealed in 1949, given 1½" resurfacing in 1959.

[a] Other than black base; cement or lime stabilization in subgrade or subbase.
[b] Figures to left of slash for outer wheelpath, to right for inner.
[c] Penetration macadam.
[d] Includes 6-in. stabilization of subgrade with lime.
[e] Includes 5-in. CTB.
[f] Includes 5-in. CTB, except on special 2,000-ft section where 10-in. lean concrete base was substituted.
[g] Two 6-in. layers of soil-cement.

TABLE 5

FLEXIBLE PAVEMENT DEFLECTION SUMMARIES[a]

Code	Proj. No.	District	Date Tested	Temp. Range (°F)	Rebound Deflection (thousandths in.) OWP/IWP[b] Proj. Avg.	Range	% Defl. Remaining at Indicated Distance Average[a] 2 Ft	6 Ft	9 Ft	Structural Thickness (in.) A.C.	Total	Construction Costs ($/lin ft) Actual	Adjusted	Year Opened to Traffic	TT & B	Soil Area	Remarks
V-1A	0058-041 -012-033 Design A	3	March 62	59-82	81/72 TL	55-155/47-94	30/26	1/3	0/0	9.0	25.0	12.87	14.89	1959	986	6	From previous deflection measurements in 1959, 1960, and 1961 it was noted that deflections in the traffic lane in Design A have been consistently higher than in other designs, except in the first series of measurements made soon after the project was opened to traffic in 1959. (See Progress Reports 1 and 2, "Experimental Flexible Pavements.") Alligator cracking and rutting now evident in traffic lane in all designs, but most notably in Design A.
					44/48 PL	37-48/34-60	36/48	2/2	0/0								
V-1B	Design B	3	March 62	59-82	42/39 TL	39-48/28-49	26/26	0/0	0/0	7.0	25.0	11.89	13.67				
					34/36 PL	26-42/22-50	29/39	0/0	0/0								
V-1C	Design C	3	March 62	59-82	53/57 TL	39-73/40-68	23/25	2/0	0/0	5.0	25.0	10.90	12.46				
					50/52 PL	44-60/45-56	26/29	2/4	0/0								
V-1D	Design D	3	March 62	59-82	52/47 TL	36-82/26-76	23/19	0/0	0/0	4.0	25.0	10.50	11.82				
					44/44 PL	31-61/31-52	23/32	2/2	0/0								
V-2A	0360-019 -002; -073-008 Design A	3	April 62	40-60	39/31 TL	31-50/24-42	33/29	3/6	0/0	7.0	23.0	12.47	13.66	Incomp.	1,136	9	New project, partly open to traffic, lacked final surface course when tested. Following items common to all 4 designs: 6" treatment of the native soil subgrade with 10% cement; 6" layer of local select borrow from same pit used in earlier projects Code I-6 and I-7. Deflections in Designs A and D of this project are highest of any measured where cement-treated subgrade used.
					39/42 PL	32-48/34-46	23/26	5/2	0/0								
V-2B	Design B	3	April 62	40-60	19/11 TL	16-22/8-12	63/64	11/9	0/0	4.5	23.0[c]	13.11	13.77				
					22/16 PL	18-28/14-20	50/63	5/6	0/0								
V-2C	Design C	3	April 62	40-60	29/14 TL	18-38/12-17	45/50	7/14	0/0	3.0	23.0[d]	12.28	12.38				
					26/16 PL	18-37/12-20	38/56	4/6	0/0								
V-2D	Design D	3	April 62	40-60	60/42 TL	36-82/25-59	18/17	2/2	0/0	4.5	23.0	11.60	12.25				
					46/48 PL	28-50/26-62	20/21	4/4	0/0								

[a]Special experimental projects.
[b]Figures to left of slash for outer wheelpath, to right for inner.
[c]Includes 6-in. CTB.
[d]Includes 4-in. CTB.

are made between asphaltic concrete and crushed aggregates, both treated and untreated, as base types on a cement-treated subgrade.

This second project was not complete when the deflections were measured, which may at least partially account for the relatively high deflections recorded for Designs A and D. A subsequent series of tests made in December 1962, produced substantially lower values, particularly in the outer wheelpaths, but part of the reduction may have been due to the different season of the year. Still further tests are planned for spring 1963.

SUMMARY OF RESULTS

The data tabulated here tend to confirm what has seemed evident throughout the seven years of deflection testing in Virginia: that pavements built in certain soil areas are much more likely to exhibit high deflections than pavements built in other soil areas. The poorest soils areas from a deflection standpoint are in the Piedmont section, in the Culpeper, Lynchburg, and Richmond Districts.

Piedmont Virgina soils tend to be quite heterogeneous, ranging in BPR classification from A-2-4 (0) to A-7-6 (20). The types most frequently found are A-4, A-5, and A-7-5, and the one characteristic most commonly associated with these soils is the presence of substantial percentages of mica.

In this report, 13 projects are listed that are not located in the Piedmont; none of these produced deflections higher than 0.025 in., regardless of pavement type or thickness. Of these 13 projects, 3 (Codes I-9, I-10, and I-18) are in the Coastal Plains and 10 (Codes I-1, I-16, I-17, I-19, I-20, II-1, III-6, IV-1, IV-2, and IV-3) are in the Valley and Ridge Province. Soils in the Coastal Plains (Area 1) usually contain high percentages of sand, whereas the Valley soils (Areas 4 and 7) most commonly are heavy clays or shales. In these areas, the magnitude of deflection apparently has little to do with the problems of pavement behavior.

It is in consideration of the data from the forty projects in the Piedmont that the maximum value can be obtained from these deflection studies. Accordingly, Table 6 summarizes the data from these projects only. In this table, the data from the experimental sections of the projects on Routes 58 and 360 from Table 5 have been worked into the summary in the proper pavement category described previously in connection with Tables 1, 2, 3, and 4.

It is recognized that the significance of some of the differences between corresponding figures in Table 6 may be debatable. Simple averages and ranges of values are often influenced to a major extent by extreme values for individual projects. Though all measurements were made as soon as possible after the frost was known to be out of the ground, obviously there were differences in temperature and natural ground moisture between projects tested early in the program and those tested later. These differences could have had an appreciable effect on readings. No positive conclusions can be advanced, therefore, regarding the relative merits of black base pavements and non-black base pavements under similar subgrade conditions.

There does seem, however, to be a marked difference between the figures for pavements that include no stabilization (Types I and II) and those that do include cement

TABLE 6

SUMMARY FOR PAVEMENTS IN PIEDMONT SECTION ONLY

Type Code	No. of Projects	Black Base	Lime or Cement Stabilization	Rebound Deflection[a] (thousandths in.)		% Deflection Remaining at Indicated Distances[a] (grand avg.)		
				Grand Avg	Range	2 Ft	6 Ft	9 Ft
I	14	Yes	No	46.2/41.6	13-173/12-148	31.9/34.6	3.9/4.6	0.5/0.7
II	10	No	No	46.1/42.3	16-111/14-92	28.6/27.8	3.6/3.2	1.0/0.8
III	9	Yes	Yes	24.2/21.0	7- 51/8 -44	44.6/41.9	8.1/9.8	2.0/2.4
IV	7	No	Yes	27.0/19.0	10- 82/6 -59	44.0/45.0	8.7/11.1	1.4/1.5

[a]Figures to left of slash for outer wheelpath, to right for inner.

stabilization in either the subgrade or the base or both (Types III and IV). (In considering only the projects in the Piedmont, no lime stabilization is included. The only two projects with lime stabilization (III-6 and IV-1) are located in the Valley and Ridge Province.) These differences are consistent across the board, so to speak, in that the averages, the minimum single group values, and the maximum single group values for Types III and IV are seldom much more than one-half the corresponding values for Types I and II. Furthermore, the distribution of the loads to the subgrade seems to be noticeably better, generally, on projects of Types III and IV; this observation is based on the higher average values of percentage of deflection remaining after the test load has moved certain specified distances away from the point of measurement.

The use of cement-treated subgrades thus seems to be providing a most effective solution to the problem of fatigue failures caused by high deflections in the Piedmont soil areas.

Deflection vs Performance

In further summary, the 53 separate pavements in this report have been classified with respect to (a) traffic volume and (b) average rebound deflection value in an attempt to learn what maximum deflection can be withstood under various conditions. Traffic volumes are classified as light (less than 200 TT & B daily), medium (200-699 TT & B daily), and heavy (700 or more TT & B daily). The following remarks summarize the findings:

1. Pavements exhibiting very low average deflections (less than 0.020 in.). Many of the 18 pavements in this group are new or nearly so. Among the older ones, only two have required appreciable upkeep expenditure. Both of these relatively inexpensive pavements (I-10 with local sand asphalt base and IV-7 with a soil cement base) have required resurfacing on account of transverse cracks which seem unrelated to deflection. Five pavements (I-1, I-16, I-18, I-19, and III-1) have carried heavy traffic without distress for some time now, one for a period of 11 years.
2. Pavements exhibiting low average deflections (0.020 to 0.030 in.). Four of these 10 pavements are less than two years old. One of the older ones (II-1) developed numerous areas of distress in the original mixed-in-place surface, but has performed well since being resurfaced. None of the others have required appreciable maintenance, although three carry traffic classified as heavy.
3. Pavements exhibiting medium average deflections (0.030 to 0.040 in). Most of the 10 pavements in this group are from 2 to 6 years old. Three carry heavy traffic: one of these has developed occasional pronounced alligator cracking (V-1B); another (III-7) shows no defects yet, but the deflections are well distributed; the third is new (V-2A). Four carry medium traffic: the two older ones (I-2 and I-15) have both required resurfacing due to development of pronounced cracking and rutting; the two newer ones show no defects after four winters. Three carry light traffic: the oldest of these (II-5) has been resurfaced once and is in distress again for reasons that are not clear in view of the light traffic; no defects have appeared on the other two.
4. Pavements exhibiting high average deflections (0.040 in. and higher). Fifteen pavements make up this group which would naturally be expected to display considerable distress. As expected, nearly all have developed pronounced distress, including two that carry only light traffic (I-13 and II-4). On seven, the distress has been severe enough to warrant at least partial resurfacing with asphaltic concrete. On two (I-12 and II-7), the distress seems to be developing surely but perhaps more slowly than might be expected. One pavement was not yet open to traffic when tested (V-2D).

In view of the foregoing, it is felt that the observations made previously (1, p. 21) are still justified. Briefly, it was stated that flexible pavements whose average deflections under an 18,000-lb axle load exceed 0.036 in. and which are subjected to heavy or medium heavy traffic may be expected to develop early distress in the form of alligator cracking and rutting.

General Observations

Data from a number of specific projects, if singled out and subjected to scrutiny, may be found of considerable interest. On such a basis, the following observations are offered:

1. The use of soil cement or cement-treated aggregates for base courses seems to be quite effective in lowering deflections. (IV-2, IV-3, IV-7, V-2B, and V-2C). There are drawbacks, however:

(a) These more rigid bases may not be able to stand as high deflections as can more flexible bases, especially if such deflections occur with considerable frequency.

(b) The presence of higher percentages of cement immediately beneath the surface often leads to shrinkage cracks which are reflected through the surface and produce something of a maintenance problem. Close observation of the performance of the cement-treated aggregate bases on Route 117 (IV-2 and -3) and Route 360 (V-2 designs B and C) may show how much of a cracking problem can develop from this type of construction.

2. Relatively high deflections, in comparison with other projects whose designs include subgrade stabilization, are recorded for projects III-7 and III-8 (Route 220, Henry County; and Route 123, Fairfax) and for experimental designs V-2A and V-2D (Route 360, Charlotte and Prince Edward). A noticeable difference exists, however, in that deflections on III-7 and III-8 are better distributed, indicating that the entire structure is behaving like a slab and deflecting on a resilient layer beneath the stabilized subgrade. On pavements V-2A and V-2D, the distribution is poorer, indicating perhaps that much of the deflection originates within the structure itself, probably above the stabilized subgrade layer.

3. Referring further to the experimental project on Route 360 (V-2), every design includes a layer of crushed stone (either treated or untreated) and a layer of local select material. In designs B and C the crushed stone is treated with cement which has tended to minimize the deflections. It has been suspected that resilience in the local material may have caused the high deflections measured in designs A and D. However, a nearby pavement (I-6), which includes local material from the same pit but no crushed stone, has performed well and shows moderately low deflections. At the same time, still another nearby pavement (I-7), built more recently and including both the crushed stone (untreated) and the same local material, exhibits very high deflections and has performed very poorly. There is reason therefore to suspect that the crushed stone rather than the local material may be to blame.

There is an urgent need in Virginia for a laboratory method of measuring the potential resilience of materials proposed for use in pavements or their subgrades, so that the disastrous effects of high deflections on expensive pavements may be avoided. The CBR test falls far short of answering this need.

4. The addition of overlays of the usual thickness of 1½ in. has had an uncertain effect on deflections. One pavement (I-14) is observed to be deflecting more since being overlaid than before; another, partly resurfaced when tested (I-15), deflected no less where resurfaced than where the original cracked surface remained. Still other projects seem to have been greatly improved by overlays (I-2, I-5, and II-3).

PAVEMENT COST ANALYSIS

It has been noted that two columns in the tabulations are included to indicate "actual" and "adjusted" construction costs per linear foot per roadway. These costs include all operations performed after completion of what is classed as "regular excavation, " and includes materials imported to build shoulders. "Actual" costs were computed from actual contract unit prices; "adjusted" costs were determined by substituting the same typical assumed unit costs into the computation for each pavement. The unit costs used for this purpose were the following:

1. $7.00 per ton for asphaltic concrete binder or surface course materials.
2. $6.00 per tone for H-3 (1) asphaltic concrete base course material. Where actual bid prices were on a square yard basis, a figure of 130 lb per sq yd per in. of depth was used for the necessary conversion.
3. $5.50 per ton for hot-mixed black base materials with aggregates obtained from local pits.
4. $5.50 per cu yd for aggregate base materials of all types produced by commercial quarries. (Cubic yard units usually measured as finally compacted in place; no allowance made for thickness in excess of that shown on plans.)
5. $4.70 per cu yd for aggregate subbase materials of all types produced by commercial quarries.
6. $3.00 per cu yd for select material Type I, CBR 20 or higher, produced by commercial quarries or traveling crushers.
7. $2.75 per cu yd for aggregate base or subbase materials available from local pits.
8. $2.00 per cu yd for select material or select borrow, CBR 20 or higher, available from local pits.
9. $1.50 per cu yd for any borrow blanket material available on the job or within very close haul.
10. $5.00 per bbl for cement used in stabilization.
11. $25.00 per ton for hydrated lime used in stabilization.
12. $0.35 per sq yd for manipulation involved in road-mix stabilization operations.

These unit costs were selected after study of statewide averages from all construction bids, prepared by the Traffic and Planning Division, and study of typical Interstate job prices. They may be low, if applied to secondary or small primary projects, or somewhat high if applied to very large Interstate projects. The one estimated price most often higher than the corresponding actual bid price is for the item of borrow available within close haul; the $1.50 price makes the adjusted cost of pavements on some projects or parts of projects seem unreasonably high. All in all, however, the adjusted cost approach makes cost comparisons between different pavements much more reasonable.

These cost computations were included to permit careful study of the relative cost of various pavements built for similar conditions of traffic, soil, and climate. They will admit some insight into the benefits in relation to the costs involved, of such costly features of many recent pavement designs as the following, for example:

1. "Black base" construction.
2. Full roadway width construction of commercial aggregate base, subbase, and select borrow materials.
3. Stabilization of subgrades and bases with cement and lime.

Black bases cost from two to more than four times as much per inch of thickness as untreated aggregate bases. But at the AASHO Road Test it was found that the asphaltic concrete used in that installation had over three times the load supporting power of the crushed stone base material and four times that of the gravel subbase material (3, p. 89). If this relationship were universally true, then the greatest economy should result from designs that would include nothing but asphaltic concrete.

The superiority of black bases over aggregate bases was rather generally proclaimed at the International Conference on the Structural Design of Asphalt Pavements at Ann Arbor, Mich., in August 1962 (4, 5). The ratios of superiority or equivalent factors, varied markedly, and even when computed from the same data from the AASHO Road Test, the factors ranged from 2.6 to 6.7, depending on the method of analysis used.

In view of the preceding, it is surprising to note in the "remarks" column of Table 1 that seven black base projects built between 1954 and 1959 have developed serious distress necessitating at least partial resurfacing (I-2, I-5, I-7, I-11, I-13, I-14, and I-15). In addition, pavement V-1A of the experimental project on US 58, the design which included 9 in. total asphaltic concrete thickness, has not performed as well as pavement V-1D, which included only 4 in. in the same total structural thickness. Although the advantages of a moderately thick bituminous mat in providing cohesion and resistance to surface shear stresses are well recognized, it is felt that Virginia's experiences tend to minimize these advantages and should be reported.

The second costly feature of many recent pavement designs, ditch-to-ditch construction with densely-graded aggregate subbase materials, is more difficult to evaluate. Barber (6) has pointed out that the densely-graded bases often have permeabilities less than that of the surface. Particularly, when a subbase is densely graded and is also covered by a penetrating prime treatment, it tends to pond water in the more open-graded black base above. The whole subject of structural section drainage is a complex one and is not within the scope of this report.

There is evidence, however, that a properly stabilized subgrade that cannot be softened by free water from above combined with a system of properly compacted granular materials of good quality can produce good performance without extensive efforts at subdrainage. An example of this is furnished by project I-18, built over 10 years ago by the then-standard trench design. On the day the deflection measurements were made on this project, the shoulder material was so saturated it would not support a passenger automobile. Other examples are afforded by projects I-3 and III-1; on both of these projects excavation at the edge of the pavement on the day after a heavy rain resulted in a lively flow of free water from the saturated "black base," and yet performance has been good on these projects through four and two winters, respectively.

The effect of the adoption of both black base and full width subbase construction as the standard for Interstate designs has been quite marked. Costs of this type of construction, using the "adjusted" unit price scale, have exceeded $21.00 per linear foot, and in view of the most recent bid prices on Select Material Type I, estimated costs probably should be higher yet. Performance of the few projects of this type now open to traffic (I-1, I-16, I-17, I-19, I-20) has been good, but none of these projects is located in the Piedmont; therefore, none is subjected to high deflections.

There is evidence that performance comparable to that afforded by present Interstate designs can be obtained at substantially less cost. The pavement design of project III-1, for example, has a subbase only 26 ft wide, has 2½ in. less asphaltic concrete than the Interstate designs, but does include a cement-treated subgrade. The total actual cost of construction was only $13.73 per linear foot. Substitution of a surface-treated soil cement shoulder pavement for the untreated crushed aggregate shoulder surfacing should not add more than $1.25 per linear foot, resulting in a total cost still less than $15.00. Facts that should not be overlooked in considering the wisdom of using such designs are (a) that the saving involved would more than defray the cost of the first three 150-lb per sq yd resurfacings, and (b) that at least one such resurfacing can be programed initially to be financed as a final stage in two stage construction.

Deflection and performance studies to date have indicated that the use of subgrade stabilization has been well worth the modest cost involved. The benefits received from the other two features are still open to question.

CONCLUSIONS AND RECOMMENDATIONS

An obvious conclusion from study of the tabulated deflection data and the foregoing summaries is that fatigue failures resulting from repeated high deflections are a major cause of flexible pavement distress in Virginia, especially in the Piedmont section.

A further conclusion might be copied from a paper prepared earlier by the author for presentation at the International Conference on Structural Design of Asphalt Pavements. This paper, prepared to meet a publication deadline of February 1, 1962, included none of this year's deflection data. Nevertheless, the following conclusion was expressed.

> Flexible pavement performance is affected to a greater extent by the degree of support offered by the underlying layers than by the thickness of asphaltic concrete in the upper portion of the structure; strength and resistance to deflection can be improved appreciably through better

> control over base and subbase compaction, but more significantly through stabilization of the subgrade with lime or cement.

This conclusion seems even further justified now.

A third conclusion, relating to the technique of deflection measurement with the Benkelman beam, is that the procedure described herein produces accurate data in adequate detail at a maximum rate of accomplishment. The fact that on most pavements an appreciable percentage of the deflection still remains when the test load has advanced a distance of 6 ft indicates clearly that the old WASHO method of attempting to measure deflection was often in error because both the point of measurement and the forward supports were within the area influenced by the load when the initial reading was taken (each being only 4½ ft away). The process of measuring deflection by backing the truck over the point of measurement and pulling forward again is tedious and time consuming, and graphical recording of the data by means of the Helmer apparatus is considered to produce greater than necessary detail.

The final, but by no means the least significant, conclusion is that many relatively low cost pavements resist deflection as well, or practically as well, as many others that carry a very high price tag.

In view of the foregoing, the following recommendations are made:

1. Laboratory tests used in the design of flexible pavements should include some measure of the potential resilience of roadbed soils. (Various test methods are being considered, and pilot studies to evaluate at least two such methods on Virginia soils are scheduled to get under way soon.)
2. Efforts should be made to develop workable procedures for "proof testing" subgrades and bases to discover and correct areas of high deflection during construction before application of the more expensive black base and surfacing elements. It is believed that the Benkelman beam can be used effectively for this purpose and that the tests can be made rapidly enough to avoid unnecessary delays in construction schedules.
3. Stage construction should be programed more often for flexible pavements, with a considerable portion of the more expensive asphaltic concrete applied from one to several years after the initial stage has been opened to traffic.

The stage construction concept advanced in the last recommendation points out one of the principal advantages offered by flexible pavement designs over rigid designs. It is the author's considered opinion that it is a mistake to attempt to construct in a single paving contract any type of pavement that would be expected to last indefinitely, or even for as long as ten years, without some likelihood of its needing a renewal of the surface course. A far more economical approach involving a minimum of risk is one in which a design such as III-1, mentioned earlier, or even IV-6, still less expensive, would be programed as the initial stage of what would ultimately become a two-stage construction project. A few years later, then, after any weak spots have shown up and been corrected, after the apparently inevitable settlements around drainage structures have occurred, and after the entire road structure has become comfortable in its environment, a new asphaltic concrete riding surface would be placed to iron out all irregularities. The total cost of such two-stage construction, even including cement stabilization and surface treatment on the shoulders if desired, would still be substantially below that of most rigid pavement designs and many flexible designs commonly used for single-stage construction in Virginia.

ACKNOWLEDGMENTS

Each spring for the past several years selected flexible pavements in various parts of the State have been tested for deflection, and it is felt that a continuation of such a program will result in a more thorough understanding of pavement behavior. But such a test program would be impossible without the splendid cooperation received from the field forces in the immediate area, who furnish the test truck and, usually, all except one man of the test crew. The individuals in the 16 residencies involved in the last two years who have helped with this work are too numerous to be listed in full, but the efforts of each, from Resident Engineer down, are sincerely appreciated.

The one representative of the Research Council present for each series of deflection tests was R. W. Gunn, of the Pavement Evaluation Section. His efforts in organizing and instructing green crews in the test procedure, in pushing the testing to completion, and in summarizing the results for publication, have given the Department of Highways the benefit of a maximum amount of information at a bare minimum of cost. His interest and enthusiasm in his work deserve special mention here.

This report is the first issued since initiation of the Pavement Design and Performance Study being conducted in cooperation with the Bureau of Public Roads. The financial assistance from HPS funds and the interest and encouragement of Stuart Williams, Supervisory Highway Research Engineer, are gratefully acknowledged.

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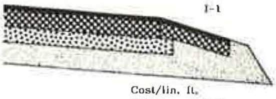
1. Nichols, F. P., Jr., "Progress Report No. 2, Experimental Flexible Pavements." Virginia Council of Highway Investigation and Research (July 1961).
2. Nichols, F. P., Jr., "Flexible Pavement Research in Virginia." HRB Bull. 269, pp. 35-50 (1960).
3. "The AASHO Road Test, Report 5." HRB Special Report 61E (1962).
4. Dorman, G. M., "The Extension to Practice of a Fundamental Procedure for the Design of Flexible Pavements." Proc., Internat. Conf. on the Structural Design of Asphalt Pavements, Univ. of Mich. (1962).
5. Shook, J. F., and Finn, F. N., "Thickness Design Relationships for Asphalt Pavements." Proc., Internat. Conf. on the Structural Design of Asphalt Pavements, Univ. of Mich. (1962).
6. Barber, E. S., "The Properties of Granular Materials." Proc., 10th Highway Materials Conf., Virginia Council of Highway Investigation and Research, p. 7 (Sept. 1962).

Appendix A

PRINCIPAL PAVEMENT SECTIONS—PROJECTS IN TABLE 1

Surface ½" F-1 (Mod)
Binder 1" H-2
Base 7½" H-3 (1)
Subbase 6" Cr. Aggr.
Subbase 9" Select B.

I-1



Cost/lin. ft.
Actual $16.09
Adjusted 18.31

Surface 1" I-3
Binder 1½" H-2
Base 5½" H-3 (1)
Subbase 4" Cr. Aggr.
Resurface 1½" I-3

I-2

Cost/lin. ft.
Actual $10.38
Adjusted 9.47

Surface 1½ I-3
Base 5½" H-3 (1)
Subbase 8" Cr. Aggr.

I-3

Cost/lin. ft.
Actual
Adjusted $10.45

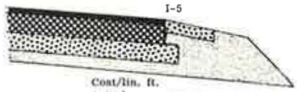
Surface: 1½" I-3
Base : 5½" H-3 (1)
Subbase: 8" Cr. Aggr.

I-4

Cost/lin. ft.
Actual
Adjusted $10.45

Surface : 1½" I-3
Base : 5½" H-3
Subbase : 5" Cr. Aggr.
Subbase : 0"-24" Select B. CBR 12
Resurface: 1½" I-3 8-6-59

I-5



Cost/lin. ft.
Actual $10.51 - 12.10
Adjusted 10.04 - 16.01

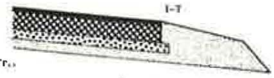
Surface: 1½" F-1
Base : 7½" H-3 (1)
Subbase: 6" Subgr. Tr. (Local Aggr.)

I-6

Cost/lin. ft.
Actual $9.55
Adjusted 11.89

Surface: 1½" F-1 Mod.
Base : 7½" H-3 (1)
Subbase: 4" Cr. Aggr.
Subbase: 4" & Var. Subgrade. Tr. (Local Aggr.)

I-7



Cost/lin. ft.
Actual $14.35
Adjusted 13.83

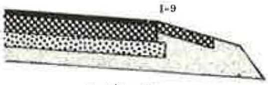
Surface: 1" F-1
Base : 7" H-3 (1)
Subbase: 6" & Var. Select B. CBR 28

I-8

Cost/lin. ft.
Actual $6.60
Adjusted 10.48

Surface: 2" F-1 Mod.
Base : 7½" H-3 (1)
Subbase: 6" Cr. Aggr.
Subbase: 6" Select B. CBR 30

I-9



Cost/lin. ft.
Actual $15.03
Adjusted 19.09

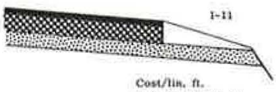
Surface : 1½" F-1
Base : 5½" F-1 (mod.)
Subbase : 6" Subgr. Tr. (Local)
Resurface: 1½" F-1 1955 & 1956
Resurface: 1½" F-1 1960

I-10

Cost/lin. ft.
Actual $5.23
Adjusted 9.38

Surface: 1½ F-1
Base : 7½ H-3 (1)
Subbase: 6" Subgr. Tr. (Local Aggr.)

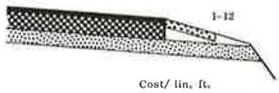
I-11



Cost/lin. ft.
Actual $8.44
Adjusted 10.91

Surface 1½" F-1
Base 7½" H-3 (1)
Subbase 6" Subgrade Tr. (Local Aggr.)

I-12



Cost/ lin. ft.
Actual $10.39
Adjusted 11.38

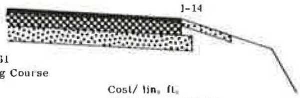
Surface 1½" I-3
Base 5½" H-3 (1)
Subbase 3" Subgr. Tr.
Subbase 6" & 12" Select B.
Resurface 1½" I-3 October 1961

I-13

Cost/ lin. ft.
Actual $11.86
Adjusted 12.82

Surface 1½" I-3
Base 5½" H-3 (1)
Subbase 6" Cr. Aggr.
Resurface 1½" I-3 June 1961
H-2 Leveling Course

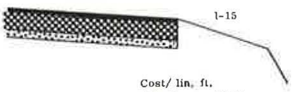
I-14



Cost/ lin. ft.
Actual $11.62
Adjusted 10.21

Surface 1½" I-3
Base 7½" H-3 (1)
Subbase 3" Cr. Aggr.

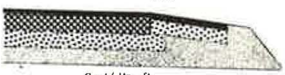
I-15



Cost/ lin. ft.
Actual $9.54
Adjusted 10.62

Surface 1/2" F-1
Binder 1 1/2" H-2
Base 7 1/2" H-3 (1)
Sub-base 6" Soil Aggr.
Sub-base 9" CBR 30

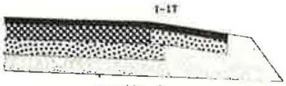
I-16



Cost/ lin. ft.
Actual $16.68
Adjusted 21.53

Surface 1/2" F-1 Mod
Binder 1 1/2" H-2
Base 7 1/2" H-3(1)
Sub-base 6" Soil Aggr.
Sub-base 9" CBR 30

I-17



Cost/ lin. ft.
Actual $19.86
Adjusted 21.53

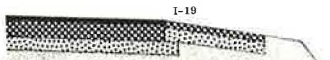
Surface : 2½" F-1
Base : 6" Loc. Agg. Bit. Conc.
Sub-base: 8" & 10" Local M.

I-18

Cost/ lin. ft.
Actual $5.52
Adjusted 8.00 - 8.29

Surface 1/2" F-1
Binder 1 1/2" H-2
Base 7 1/2" H-3(1)
Sub-base 6" Soil Aggr.
Sub-base 12 Select B (CBR 30)

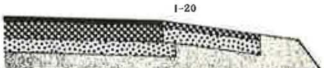
I-19



Cost/ lin. ft.
Actual $16.67
Adjusted 21.11

Surface 1/2" F-1
Binder 1 1/2" H-2
Base 7 1/2" H-3(1)
Sub-base 6" Soil Aggr.
Sub-base 12" Select B. (CBR 30) Crushed Stone

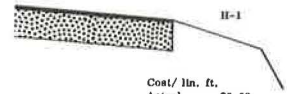
I-20



Cost/ lin. ft.
Actual $
Adjusted 21.11

PRINCIPAL PAVEMENT SECTIONS—PROJECTS IN TABLE 2

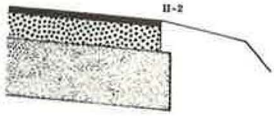
II-1



Surface 1" M.I.P.
Base 12" Cr. Aggr.
Resurface 1½" I-3 1960

Cost/ lin. ft.
Actual $5.52
Adjusted 6.21

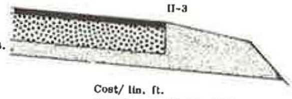
II-2



Surface: 2" H-2
Base: 11" Graded Aggr.
Subbase: 24" Select B.

Cost/ lin. ft.
Actual $8.88
Adjusted 9.84

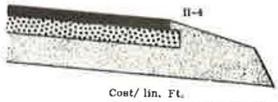
II-3



Surface : 2½" H-2
Base : 12" Graded Aggr.
Subbase : 6", 10" & 16" Select B.
Resurface: 1½" F-1 July 1957

Cost/ lin. ft.
Actual $9.08 - 9.75
Adjusted 9.72 - 11.96

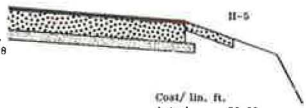
II-4



Surface 1½" I-3
Binder 3" H-3 (1)
Base 7" Cr. Aggr.
Subbase 12" & 18" Select B.

Cost/ lin. Ft.
Actual $11.58 - 12.55
Adjusted 11.57 - 13.03

II-5



Surface 1" M.I.P.
Base 8" Cr. Aggr.
Subbase 4" Subgr. Tr.
Resurface 1½" I-3 1958

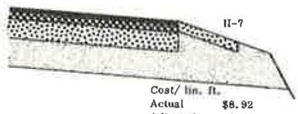
Cost/ lin. ft.
Actual $6.52
Adjusted 5.31

II-6

Surface 1" I-3
Binder 3" H-3 (1)
Base 8" Cr. Agge.
Subbase Var. Select B.

Cost/ lin. ft.
Actual -
Adjusted $10.61

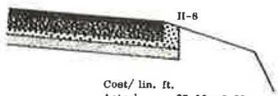
II-7



Surface 1½" I-3
Binder 3" H-3 (1)
Base 8" Cr. Aggr.
Subbase 12" Select B.

Cost/ lin. ft.
Actual $8.92
Adjusted 8.73

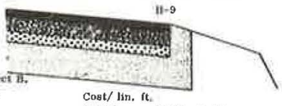
II-8



Surface 2" F-1
Base 8" W. B. Macadam
Subbase 3" Subgr. Tr.
Subbase 0", 4", 13", 16" Select B.
Resurface 1" I-3 1958
Resurface Seal 1961

Cost/ lin. ft.
Actual $7.18 - 8.59
Adjusted 6.85 - 8.29

II-9

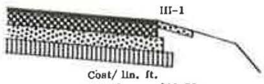


Surface 1" Surface Tr.
Binder 2" Bit. Pentr.
Base 6" Macadam
Subbase 4" Cr. Aggr.
Subbase 0", 14", 35" Select B.
Resurface 1½" I-3 1958

Cost/ lin. ft.
Actual $8.47 - 9.57
Adjusted 8.21 - 10.93

PRINCIPAL PAVEMENT SECTIONS—PROJECTS IN TABLE 3 AND 4

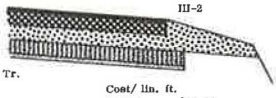
III-1



Surface 1½" I-3
Base 5½" H-3 (1)
Subbase 6" Cr. Aggr.
Subbase 6" Cement Tr.

Cost/ lin. ft.
Actual $13.73
Adjusted 12.92

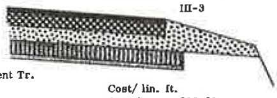
III-2



Surface 1½" I-3
Base 5½" H-3 (1)
Subbase 6" Cr. Aggr.
Subbase 8" Select B with Cement Tr.

Cost/ lin. ft.
Actual $16.21
Adjusted 15.89

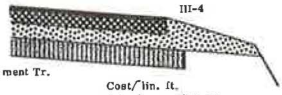
III-3



Surface 1½" F-1
Base 5½" H-3 (1)
Subbase 6" Cr. Aggr.
Subbase 8" Select B with Cement Tr.

Cost/ lin. ft.
Actual $18.91
Adjusted 18.85

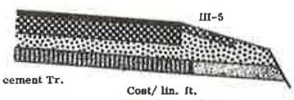
III-4



Surface 1½" I-3
Base 5½" H-3 (1)
Subbase 6" Cr. Aggr.
Subbase 8" Select B. with Cement Tr.

Cost/ lin. ft.
Actual $15.56
Adjusted 16.02

III-5



Surface ½" F-1
Binder 1½" H-2
Base 7½" H-3 (1)
Subbase 6" Cr. Aggr.
Subbase 6" Select B with cement Tr.

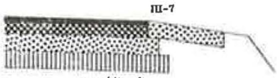
Cost/ lin. ft.
Actual $16.77
Adjusted 21.35

III-6

Surface ¾" F-4
Binder 1½" H-2
Base 5½" H-3(1)
Subbase 10" Select B.
Subbase 6" Lime Tr.

Cost/ lin. ft.
Actual $12.69 - 12.21
Adjusted 11.82 - 14.09

III-7



Surface 1 1/2" I-3
Base 5 1/2" H-3 (1)
Sub-base 8" Soil Aggr.
Sub-base 8" Cement Stab.

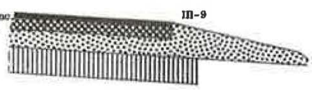
Cost/ lin. ft.
Actual $14.40
Adjusted 13.75

III-8

Surface : 1½" I-3
Base : 5½" H-3 (1)
Prime : MC-0
Sub-base: 6" Cr. Aggr.
Sub-base: 8" Cement Tr.

Cost/ lin. ft.
Actual $14.75
Adjusted 14.40

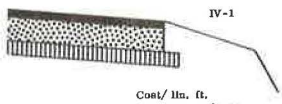
III-9



Surface : 1.5" I-3 Bit. Conc.
Base : 5.5" H-3 (1) Bit. Conc.
Prime : MC-0
Sub-base: 6" Subgrade Tr. Matl. C or D
Sub-base: 8" Cement Tr.

Cost/ lin. ft.
Actual $13.90
Adjusted 15.30

IV-1



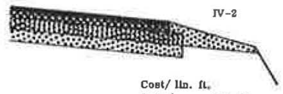
Surface 1" M.I.P.
3" Pentr. Macadam
Base 10" Cr. Aggr.
Subbase 6" Lime Tr.

Cost/ lin. ft.
Actual $8.99
Adjusted 9.94

PRINCIPAL PAVEMENT SECTIONS--PROJECTS IN TABLE 3 AND 4 (Continued)

Surface ¼" F-4
Binder 1¼" H-2
Binder 4" H-3 (2)
Base 5" Cement Tr. Cr. Aggr.
Subbase 5" Cr. Aggr.

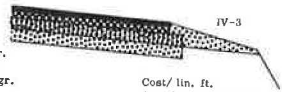
IV-2



Cost/ lin. ft.
Actual $12.53
Adjusted 13.09

Surface ¼" F-1
Binder 1¼" H-2
Binder 4" H-3 (2)
Base 5" Cement Tr. Cr. Aggr.
Subbase 5" Cr. Aggr.

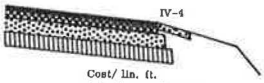
IV-3



Cost/ lin. ft.
Actual $13.67
Adjusted 13.09

Surface: 1½" I-3
Binder : 3" H-3 (1)
Base : 6" Cr. Aggr.
Subbase: 6" Cement Tr.

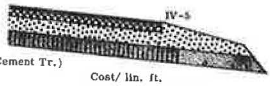
IV-4



Cost/ lin. ft.
Actual $11.22
Adjusted 10.80

Surface: 1½" I-3
Binder : 3" H-3 (1)
Base : 6" Cr. Aggr.
Subbase: (6 " Select Mat'l Cement Tr.)

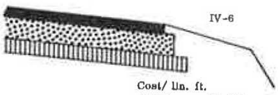
IV-5



Cost/ lin. ft.
Actual $14.05
Adjusted 13.40

Surface 1½" I-3
Binder 2½" H-2
Base 9" Cr. Aggr.
Subbase 6" Cement Tr.

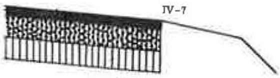
IV-6



Cost/ lin. ft.
Actual $9.42
Adjusted 10.25

Surface : 1" F-1
Binder : 1½" H-2
Base : 6" Select Material Cement Tr.
Subbase : 6" Cement Tr.
Resurface: 1½" I-3, Oct. 1959

IV-7

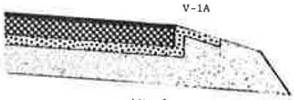


Cost/ lin. Ft.
Actual $6.42
Adjusted 7.63

PRINCIPAL PAVEMENT SECTIONS—PROJECTS IN TABLE 5

Surface 1½" I-3
Base 7½" H-3 (1)
Subbase 4" Cr. Aggr.
Subbase 12" Select D. (CBR 12)

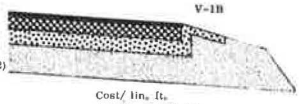
V-1A



Cost/ lin. ft.
Actual $12.87
Adjusted 14.89

Surface 1½" I-3
Base 5½" H-3 (1)
Subbase 6" Cr. Aggr.
Subbase 12" Select B. (CBR 12)

V-1B



Cost/ lin. ft.
Actual $11.89
Adjusted 13.67

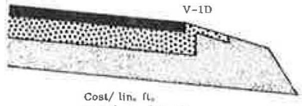
Surface 1½" I-3
Binder 3½" H-3 (1)
Base 8" Cr. Aggr.
Subbase 12" Select B (CBR 12)

V-1C

Cost/ lin. ft.
Actual $10.90
Adjusted 12.46

Surface 1½" I-3
Binder 2½" H-2
Base 9" Cr. Aggr.
Subbase 12" Select B (CBR 12)

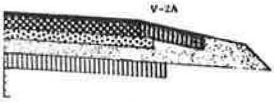
V-1D



Cost/ lin. ft.
Actual $10.50
Adjusted 11.82

Surface 1.5" I-3
Base 5.5" H-3 (1)
Subbase 4" Cr. Aggr.
Subbase 6" Select M.
Subbase 6" Cement Tr.

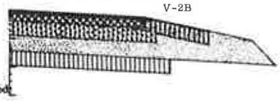
V-2A



Cost/ lin. ft.
Actual $12.47
Adjusted 13.66

Surface 1.5" I-3
Binder 3" H-3 (1)
Base 6" Cr. Aggr. Cement Treated
Subbase 6.5" Select M.
Subbase 6" Cement Treated

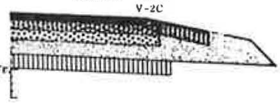
V-2B



Cost/ lin. ft.
Actual $13.11
Adjusted 13.77

Surface 1.5" I-3
Binder 1.5" H-2
Base 6" Cr. Aggr. with top 4" Cement Tr.
Subbase 6" Select M.
Subbase 6" Cement Tr.

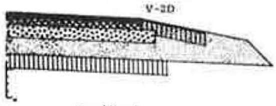
V-2C



Cost / lin. ft.
Actual $12.28
Adjusted 12.38

Surface 1.5" I-3
Binder 3" H-3 (1)
Base 6" Cr. Aggr.
Subbase 6.5" Select M.
Subbase 6" Cement Tr.

V-2D



Cost/ lin. ft.
Actual $11.60
Adjusted 12.25

Appendix B

GENERAL SOIL AREAS OF VIRGINIA

Figure 3 shows a map of Virginia divided into twelve general soil areas. This map was originally published in a paper by Stevens, Maner, and Shelburne, "Pavement Performance Correlated with Soil Areas" in the Highway Research Board Proceedings of 1949. General soil areas were selected on the basis of geological formations and past experience, and were numbered in the approximate order of suitability of the predominant soil types as highway subgrades, as seemed evident to the authors from their analysis of condition survey data from the spring break-up of 1948.

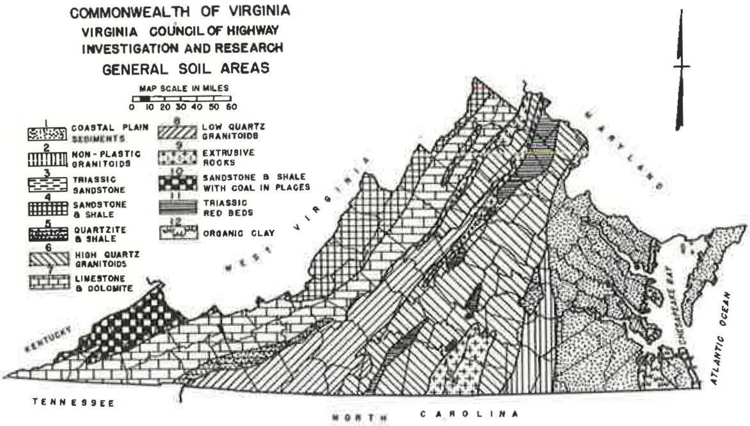


Figure 3. Map of Virginia showing general soil areas.

Appendix C

SPECIFICATIONS AND PROPERTIES OF TYPICAL VIRGINIA PAVING MATERIALS

Asphaltic Concretes

Type	Gradation Tolerance (% passing)							Asphalt (avg. %)
	1½-In.	1-In.	½-In.	No. 4	No. 10	No. 40	No. 200	
F-1 surface			100	75-90	60-80	15-35	2-10	7.0
F-4 surface				100	95-100	40-95	2-8	9.0
I-3 surface			100	50-70	35-50	10-25	2-10	6.2
H-2 binder		100		40-60	30-40			5.5
H-3 (1) base	90-100			30-45	20-35		0-5	4.5
F-2 base (local)		100	85-100	75-100	60-95	20-50	0-10	4.5

Compaction Requirements.—Compaction of completed pavements generally required to produce density not less than 90 percent of the calculated density of voidless mixture composed of same materials in like proportions. Exceptions made for local sand mixes of types F-2 and F-3; density requirements less rigid.

Aggregate Base and Subbase Materials

Material	Gradation Tolerance (% passing)					
	2-In.	1-In.	$^3/_8$-In.	No. 10	No. 40	No. 200
Base grading A	100	50-80	30-65	15-40	10-20	4-10
Base grading B	100	65-90	50-75	25-45	12-30	4-15
Base grading C	100	90-100	50-85	25-50	12-30	5-15
Base grading D		100	60-100	30-65	20-40	5-15
Base grading E		100		40-100	20-50	6-20
Base grading F		100		55-100	30-70	8-25
Subbase grading 1	100	80-100	50-90	30-70	10-40	4-15
Subbase grading 2	100	-	-	40-100	25-75	0-25

Type I Base Material. —Crushed stone, slag, or gravel, maximum liquid limit 25, maximum P. I. 3, grading A, B, C, or D.

Type II Base Material. —Maximum liquid limit 25, maximum P. I. 6, grading C, D, E, or F.

Type III Base Material (Graded Aggregate). —Crushed stone, slag, or gravel premixed with soil mortar fraction in pug mill or other approved plant. Grading B only, otherwise same as Type I.

Subbase Materials. —Maximum liquid limit 25, maximum P. I. 3.

Los Angeles abrasion loss on plus No. 10 fraction. —45 percent maximum, all base types.

Compaction Requirements. —Compaction of completed base or subbase required to produce density not less than 100 percent of the maximum theoretical density "D" calculated as described in paper "Suggested Compaction Standards for Crushed Aggregate Materials Based on Experimental Field Rolling," by F. P. Nichols, Jr., and H. D. James, HRB Bull. 325 (1962). Modified standards suggested in above paper not applicable to any of the pavements in this report.

Select Materials

Required properties variable from job to job, specified in Special Provisions attached to individual contracts. Typical requirements:

Maximum aggregate size—3 in.
Maximum passing the No. 200 sieve—25-40 percent.
Maximum liquid limit—25-40.
Maximum laboratory CBR—10-30.
Compaction requirements same as for bases and subbases.

Stabilized Subgrades or Subbases

Granular materials or friable soils generally stabilized with cement; percentages 5 to 12 percent by volume. Heavy clays stabilized with hydrated lime, usually 5 to 6 percent. Layer thicknesses usually 6 in. compacted, maximum 8 in. in friable soil. Compaction of completed stabilized layer required to produce density of 100 percent of the density of the same material when tested in accordance with AASHO Method T-134, with tolerance of 5 pcf.

Some Notes on Pavement Structural Design

NORMAN W. McLEOD, Asphalt Consultant, Imperial Oil Limited, Toronto, Ontario, Canada

The design equation, $T = K \log (P/S)$, resulting from the Canadian Department of Transport's investigation of airport runways in Canada by means of plate bearing tests, is briefly reviewed, is shown to be an elastic equation, and is found to be related mathematically to the elastic equations developed by Burmister for his layered system approach to pavement design. The pavement factor K is shown to be equal to $T/-\log F_W$, where F_W is Burmister's deflection factor.

Analysis of the Hybla Valley plate bearing test data indicates that the value of the pavement factor K goes through a minimum for each size of bearing plate when K is plotted versus pavement thickness T. It is shown that when K is evaluated for any bearing plate size by means of the Burmister theoretical equations for a two-layer elastic system, the values of K determined on this purely theoretical basis also go through a minimum when plotted against T.

Design charts presented enable different combinations of pavement elastic modulus E_1 and pavement thickness T to be selected, all of which are equally capable of carrying a specified wheel load and traffic volume over a given subgrade.

Analysis of the Hybla Valley load test data indicates that for each bearing plate size employed there is an optimum thickness of granular base (pavement material) at which the load supporting value per inch thickness of granular base reaches a maximum, and that this optimum thickness is roughly equal to the diameter of the loaded area. From strictly theoretical calculations based on the equation $T = K \log (P/S)$, and on the Burmister equations for a two-layer elastic system, it is shown that for pavement materials ordinarily employed for flexible pavement structures there is an optimum thickness for each pavement material at which the load supporting value per inch thickness of pavement attains a maximum. Furthermore, for ordinary flexible pavement design these calculations indicate that this optimum thickness of pavement ranges from approximately 1.5 times the radius of the loaded area for pavements on strong subgrades, to approximately 2.0 times the radius of the loaded area for pavements on weak subgrades.

The current conventional approach to flexible pavement design ordinarily calls for additional depth of granular material when load carrying capacity must be increased. The findings presented in this paper question the technical utility of this solution to the problem of achieving greater bearing capacity. It is shown that the added thickness is often well down on the curve of diminishing returns with regard to strength increase, and that the supporting value per inch of thickness of the added depth of granular material may be relatively low.

There is need to conserve the gradually diminishing deposits of good natural aggregates and to upgrade the quality of inferior

Paper sponsored by Committee on Flexible Pavement Design.

granular materials. This could be achieved by treatment of the entire depth of pavement material above the subgrade to increase its elastic modulus, so that a pavement structure of much smaller thickness (a two-layer elastic pavement system) would have the necessary overall strength required for the wheel load and traffic volume to be carried.

Increasing the elastic modulus of the pavement component requires upgrading the strength characteristics of granular or soil materials by the application of what are usually called stabilization processes, and particularly by the incorporation of bituminous binders.

To measure the degree of improvement achieved by upgrading the elastic strength characteristics of pavement materials requires the development or perfection of simple, reliable, precise methods for measuring the elastic moduli of these materials. This also applies to subgrade soils.

• AT THE 26th (1946) Annual Meeting of the Highway Research Board the author presented the results of an investigation of existing airport runways in Canada by the Canadian Department of Transport (1). The investigation included strength measurements made on the subgrade, the base course, and the pavement surface by means of load tests on bearing plates that were usually 12, 18, 24 and 30 in. in diameter. From analysis of many load test data, the following equation was developed for thickness requirements for flexible pavements (Fig. 1):

$$T = K \log (P/S) \tag{1}$$

in which

- T = required thickness, in inches;
- P = wheel load to be carried (single tire);
- S = subgrade support measured for the same loaded area and deflection that pertain to P; and
- K = pavement factor, an inverse or reciprocal measure of the increase in strength provided by the first unit of thickness of pavement placed on the subgrade.

For the first part of this paper, it is necessary to examine the background and significance of the pavement factor K in some detail. All symbols are defined where they first appear, and for ease of reference are also listed in the Appendix.

Figure 2 illustrates one of the principal findings of the Canadian Department of Transport's investigation (1); namely, the increase in load supporting value provided by any specified thickness $\bar{T}$ of base course material varies directly with the strength of the subgrade on which it is placed. It follows that when successive layers of a given base course are of equal thickness (Fig. 3) the second layer of base provides a greater increase in load supporting value than the first layer of base, and so on, because the second layer of base rests on a stronger "subgrade" (the subgrade plus the first layer of base course) than the first layer of base, which rests on the subgrade.

Figure 4 shows how this finding is developed into the design equation (Eq. 1). From the geometry of Figure 4, as described in detail in an earlier paper (1), the following relationship can be established,

$$T = [1/\log (P_1/S)] \log (P/S) \tag{2}$$

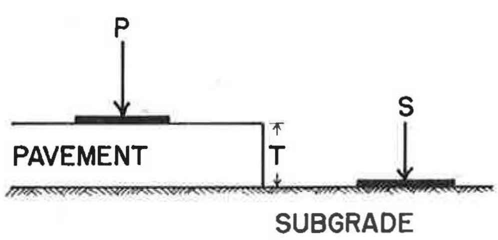


Figure 1. Layout of load tests for Canadian Department of Transport's investigation of airport runways.

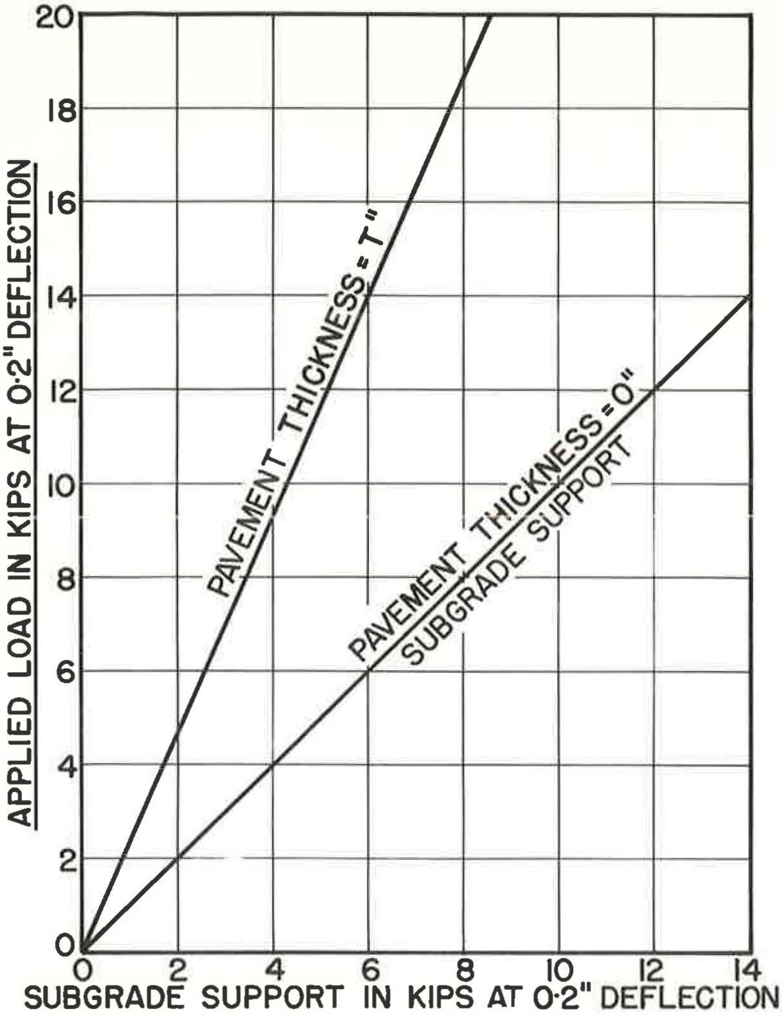


Figure 2. Typical graph of Canadian Department of Transport load test data.

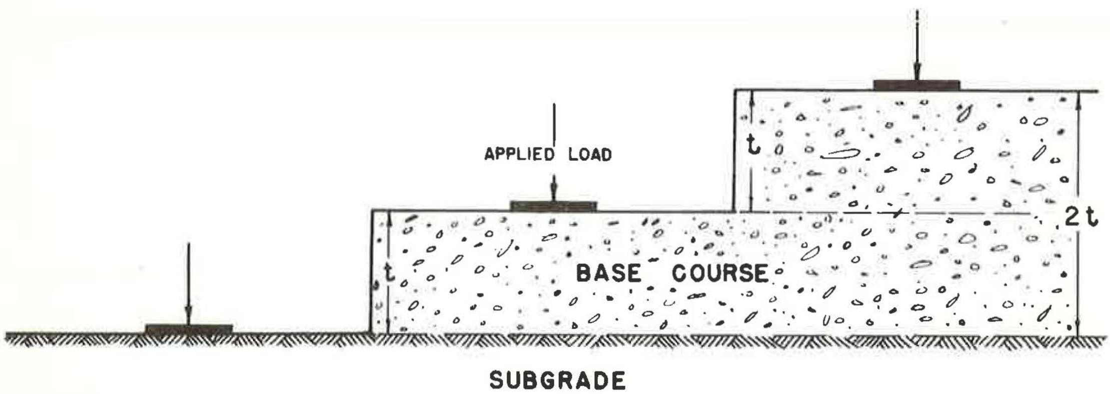


Figure 3. Sketch of applied loads on successive base course layers of uniform thickness.

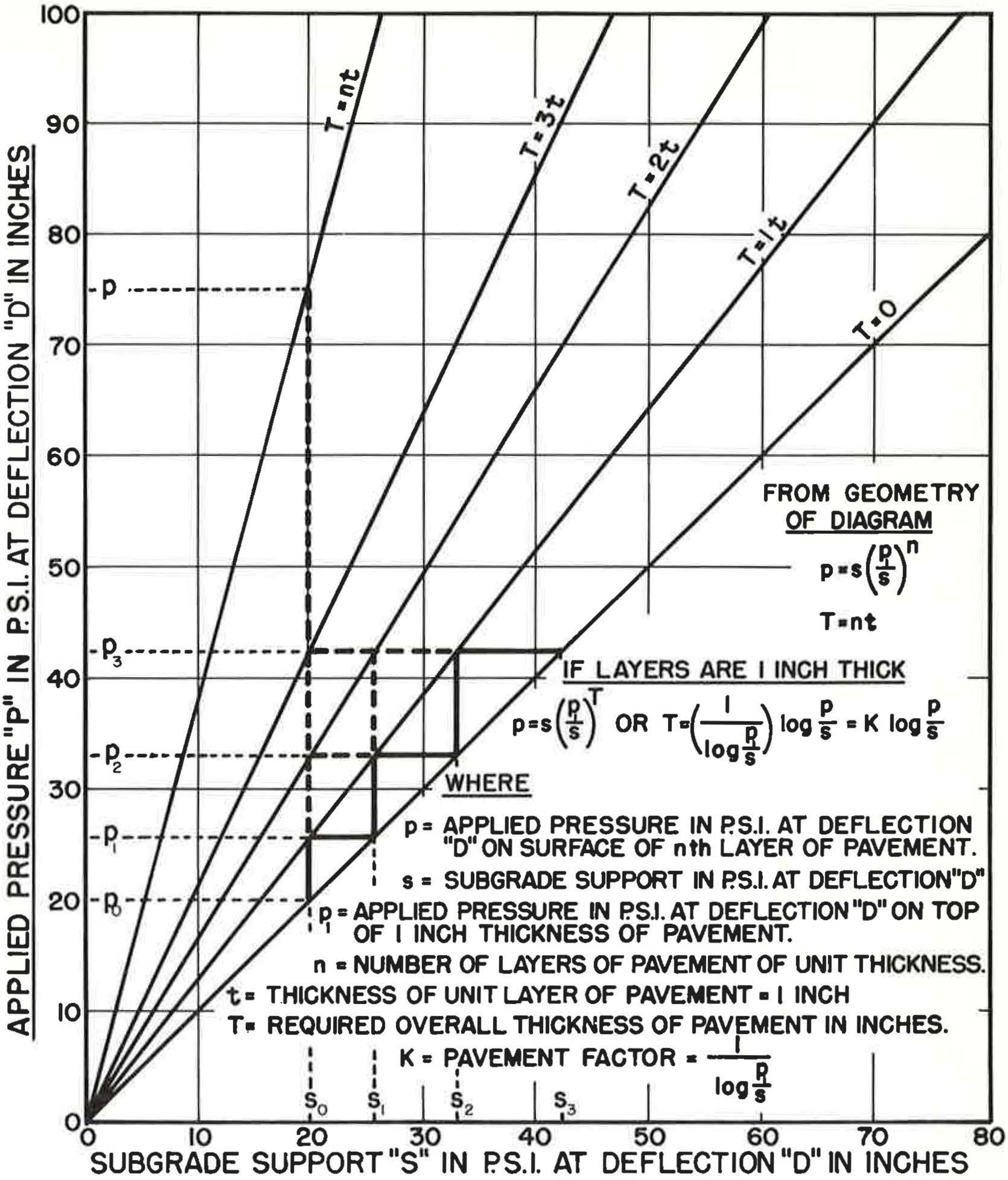


Figure 4. Development of the design equation, T = K log (P/S).

in which T, P, and S are as previously defined and P_1 is the load supported on the surface of the first 1-in. thickness of the pavement material on the subgrade for the same deflection and loaded area that pertain to P and S.

For any given type of pavement material (for example, crusher run gravel) and for any specified loaded area, the load test data indicated that for the range of flexible pavement thicknesses ordinarily employed the expression $\frac{1}{\log (P_1/S)}$ was substantially constant in value; that is,

$$\frac{1}{\log (P_1/S)} = K \tag{3}$$

in which K is the pavement factor already described and assumed to be a constant for any given pavement material and size of loaded area.

Consequently, for ordinary design purposes Eq. 2 can be rewritten as Eq. 1.

The pavement factor K was evaluated by means of load test data. The best empirical values that could be determined for K are indicated by Figure 5, which shows that the value of K varies with size of loaded area.

Figure 6 shows that Eq. 1 and Figure 5 can be utilized to provide a flexible pavement thickness design chart. The general shape of the curves, and the thicknesses indicated, approximate those provided by other approaches to the problem of flexible pavement thickness requirements.

Although it may not be apparent as a first impression, it must be emphasized that Eq. 1 belongs to the elastic theory category of approaches to pavement design. Both P and S in Eq. 1 are measured for the same loaded area and at the same deflection. Consequently, the ratio P/S in Eq. 1 is actually the ratio of a secant modulus of elasticity of the pavement structure as a whole, to a secant modulus of elasticity of the subgrade.

Inasmuch as the load on the pavement and the load on the subgrade are measured for the same loaded area and at the same deflection, it should be clear that the ratio P/S is equal to the ratio p/s, where P is the total load (pounds or kips) and p is the unit pressure (psi) on the pavement, and S is the total load (pounds or kips) and s is the unit pressure (psi) on the subgrade. Consequently, Eq. 1 can be written either

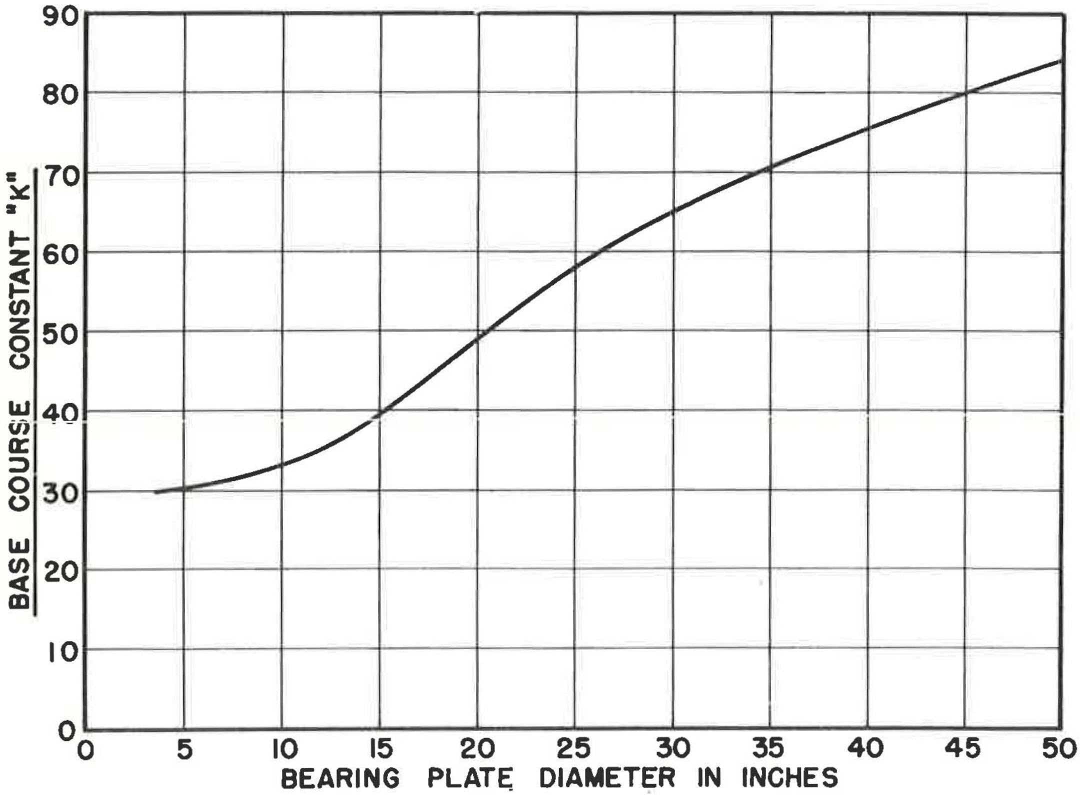


Figure 5. Influence of bearing plate diameter on value of K in flexible pavement design equation, T = K log (P/S).

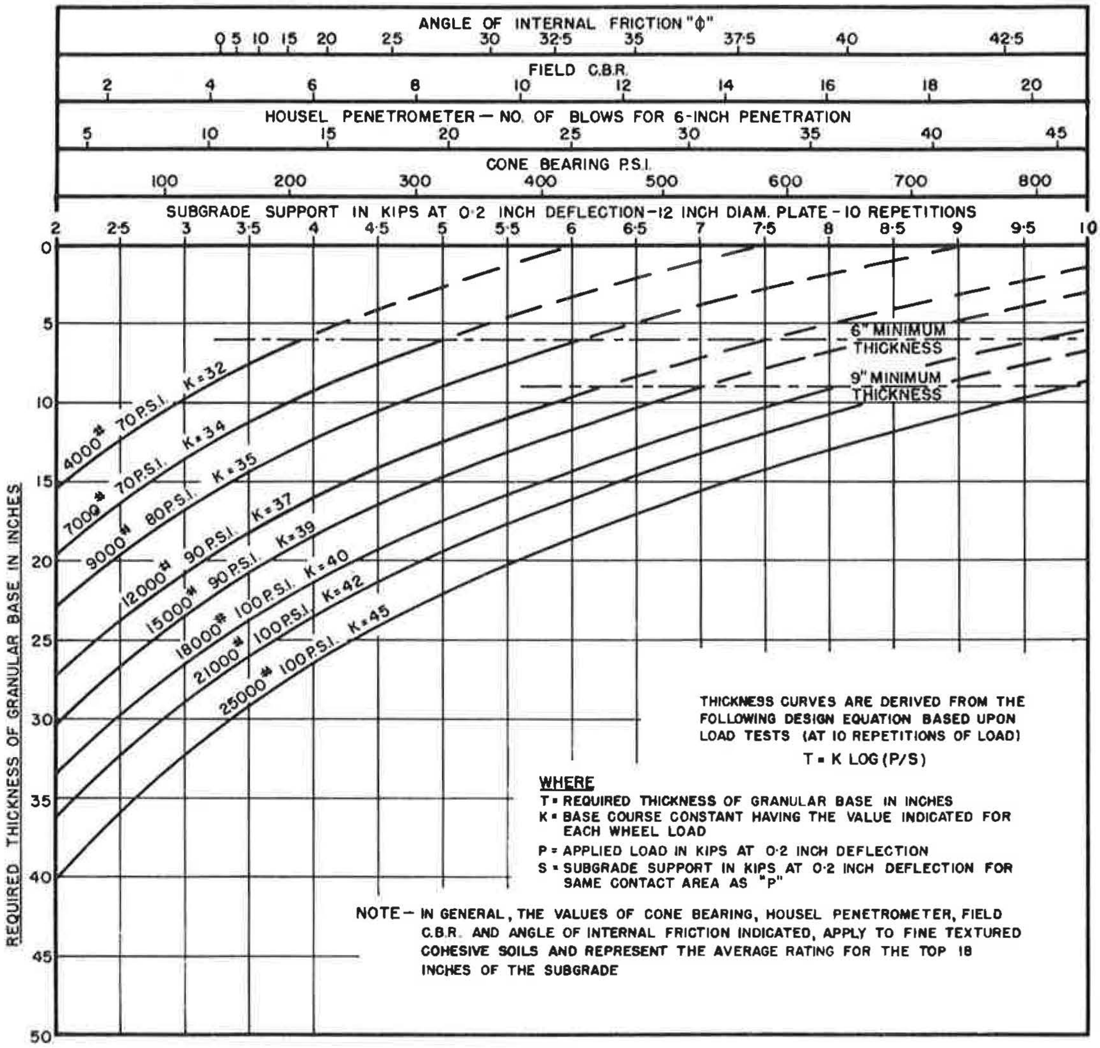


Figure 6. Flexible pavement thickness requirements for highways carrying maximum traffic volume (full load on single tire).

in the form previously given or as

$$T = K \log (p/s) \qquad \text{(1a)}$$

without change in significance, as long as the proper units of load are used for P, p, S, and s.

VARIATION OF K WITH PAVEMENT THICKNESS

It should be emphasized that although Eq. 3 shows that the value of K depends specifically on the logarithm of the ratio of the load P_1 supported on the first inch of pavement (base course) directly on the subgrade, to the load S carried by the subgrade for the same loaded area and at the same deflection, in actual practice K is evaluated on the basis of the load P supported on a normal thickness of pavement (base course), and by substituting measured values for P, S, and T in Eq. 1.

Analysis of the data from hundreds of plate bearing tests on paved runways, using bearing plates ranging in diameter from 12 to 42 in., indicated that the pavement factor K in Eq. 1 varies with plate diameter (Fig. 5).

This finding, that for any given pavement material K is a constant which varies only with size of loaded area (Fig. 5), appears to be reasonably satisfactory for the range of flexible pavement thicknesses ordinarily required for highway and airport wheel loadings.

Nevertheless, even when the size of the loaded area is kept constant, it was recognized from the beginning (1) that above some thickness of flexible pavement the value of K must increase with thickness. Otherwise, use of Eq. 1 with the K-values of Figure 5 would indicate that large thicknesses of flexible pavement could support loads in excess of the crushing strength of the aggregate particles.

Because the thickness of flexible pavement on the runways at any given airport was usually more or less constant, the necessary load test data were not available to establish with the necessary degree of precision how the value of K varies with flexible pavement thickness. However, the Hybla Valley project near Washington, D. C., provides reliable data for this purpose (2). The subgrade soil was processed and compacted to a depth of 5 ft to obtain a homogeneous uniform subgrade. Depths of 6, 12, 18, and 24 in. of uniform and well-compacted granular base course were laid on the subgrade and surfaced with 3, 6, and 9 in. of asphaltic concrete. The strengths of the subgrade, the different thicknesses of granular base, and the finished pavement, were measured by means of plate bearing tests employing rigid steel plates 12, 18, 24, and 30 in. in diameter.

Eq. 1 can be rearranged as

$$K = \frac{T}{\log (P/S)} \tag{4}$$

From the Hybla Valley project, measured values are available for S, P, and T, from load tests on the subgrade, and on granular base course thicknesses of 6, 12, 18, and 24 in., and for bearing plate diameters of 12, 18, 24 and 30 in. When these measured values are substituted in Eq. 4, values for the pavement factor K can be calculated for these various base course thicknesses and bearing plate diameters.

Figure 7, resulting from Hybla Valley data for load tests at 0.2-in. deflection, plots K versus thickness of granular base for each of four bearing plate diameters. It indicates that the value of K varies with base course thickness as well as with the diameter of the bearing plate. Furthermore, for each bearing plate diameter the value of K goes through a minimum, which occurs at a thickness that seems to lie somewhere between the radius and the diameter of the loaded area.

Eq. 3 indicates that for any given subgrade and pavement material, the value of K depends on the magnitude of P_1, the load supporting value of the first inch of pavement material in contact with the subgrade (Fig. 8). Consequently, Figure 7 implies, as indicated by the curved arrow in Figure 8, that P_1 increases to a maximum and then decreases in value as the pavement thickness gradually increases from very small to very large. Correspondingly, as indicated by Eq. 3 and shown by Figure 7, the value of K decreases to a minimum and then increases as the pavement thickness increases from approximately zero to a great depth. In effect, for any given pavement material P_1 changes with thickness in such a way that the geometric arrangement of Figure 4 (which led to the derivation of Eq. 1) is able to duplicate the measured value of P associated with each different pavement thickness.

RELATIONSHIP BETWEEN EQ. 1 AND THE BURMISTER EQUATIONS FOR A LAYERED SYSTEM

In 1943, Burmister (3) published the results of a purely theoretical investigation of pavement design, based on the assumption that a pavement consists of layers of materials with strictly elastic properties.

Figure 9 illustrates a two-layer elastic system studied by Burmister, for which the elastic modulus of the pavement is E_1 and the elastic modulus of the subgrade is E_2. Perfect continuity is assumed to exist across the interface between the pavement and the subgrade.

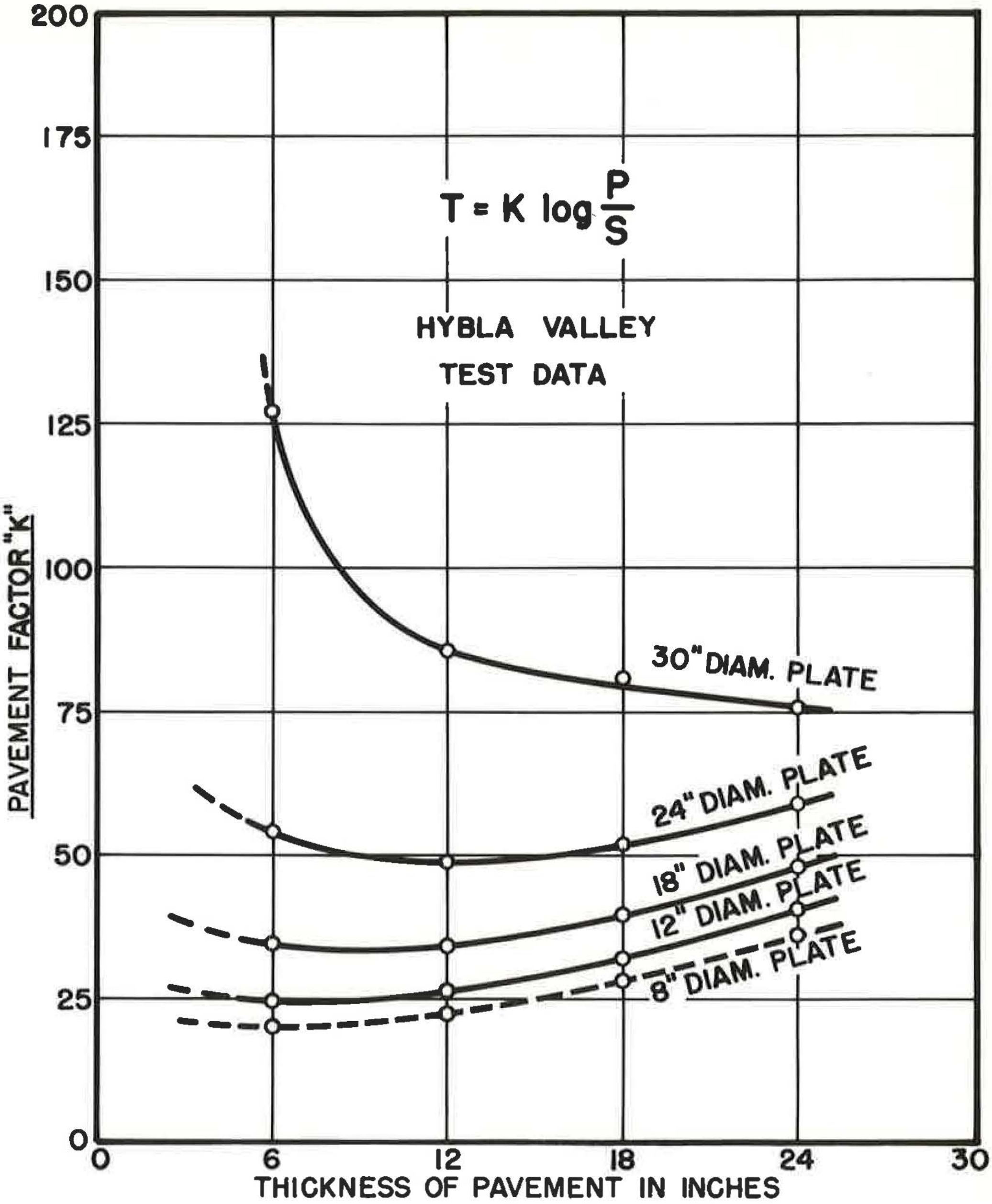


Figure 7. Influence of pavement thickness and bearing plate diameter on pavement factor K (Hybla Valley).

For a rigid plate bearing test on the subgrade (Fig. 9) the Boussinesq equation is

$$w_S = \frac{1.18\ s\ r}{E_2} \tag{5}$$

and for a load test made with a rigid bearing plate on the surface of the pavement layer (Fig. 9) Burmister has developed the equation

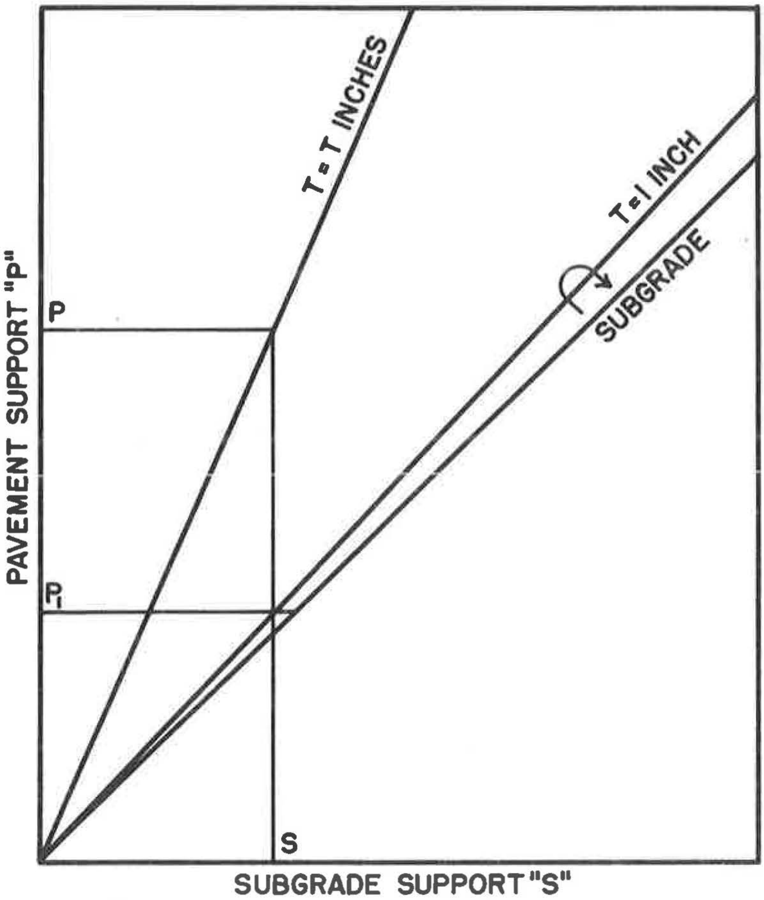


Figure 8. Reason for variable values for pavement factor K.

$$w_p = \frac{1.18\,p\,r}{E_2}\,F_w \tag{6}$$

in which

w_s = deflection at the surface of the subgrade;
w_p = deflection at the surface of the pavement;
s = applied load, in psi, on a rigid bearing plate on the subgrade;
p = applied load, in psi, on a rigid bearing plate on the pavement;
r = radius of bearing plate;
E_2 = elastic modulus of the subgrade;
and
F_w = deflection factor which varies with T, r, and E_1/E_2 (Fig. 10).

When utilizing the Burmister method, load test data are obtained for a given bearing plate diameter, and for a specified deflection, by means of plate bearing tests on both pavement and subgrade.

When w_s = w_p, Eqs. 5 and 6 can be equated to give

$$F_w = s/p \tag{7}$$

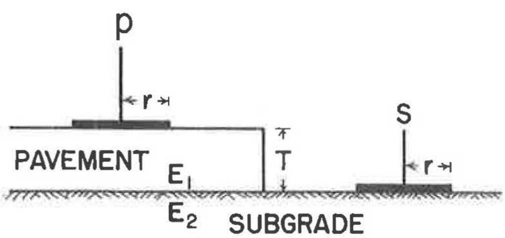


Figure 9. Burmister's layered system elastic theory approach to pavement design.

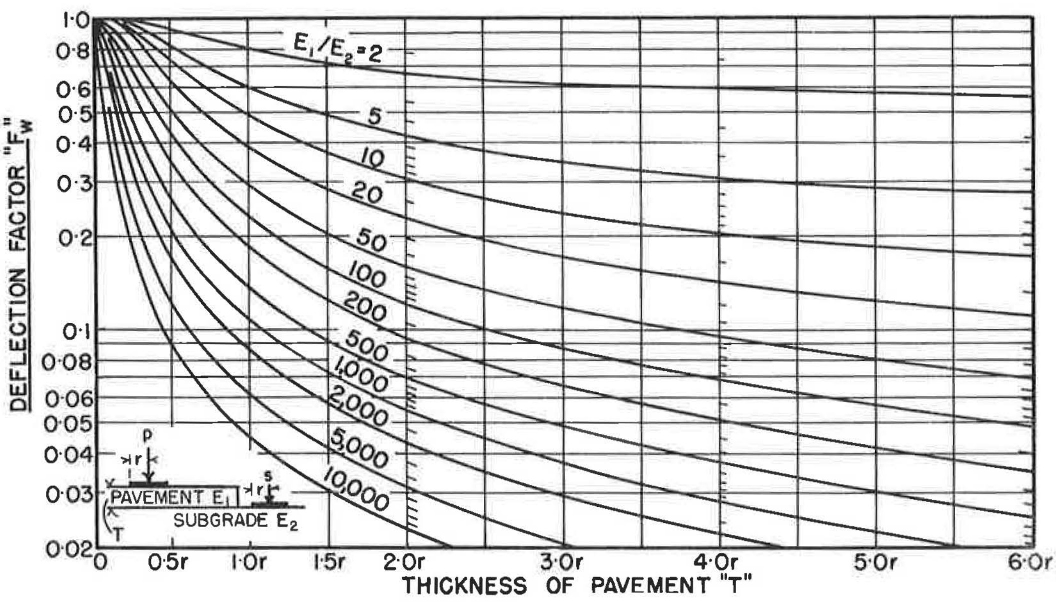


Figure 10. Relationship between deflection factor, pavement thickness, and E_1/E_2 ratio, for a two-layer elastic system.

Rearrangement of Eq. 7 and substitution in Eq. 4 gives

$$K = \frac{T}{-\log F_W} \tag{8}$$

which provides a mathematical bridge between Eq. 1, developed empirically from load test data measured on airport runways by the Canadian Department of Transport, and Eqs. 5 and 6, derived by Burmister from purely theoretical considerations based on the elastic properties of a layered system.

Burmister (3) has published a chart (Fig. 10) showing the relationship between values of F_W, T, r, and E_1/E_2. By utilizing Eq. 8 and Figure 10, the theoretical relationship between the pavement constant K (from Eq. 1) and pavement thickness T can be established (Fig. 11).

An example of the calculations required for Figure 11 is given in Table 1, using $E_1/E_2 = 10$. In keeping with common usage in soil mechanics, both K and T are expressed as multiples of the radius r of the loaded area.

The resemblance of Figure 11, resulting from this purely theoretical approach, to Figure 7, based entirely on load test measurements made at Hybla Valley, is striking. In both Figures 7 and 11, the value of K goes through a minimum when K is plotted versus pavement thickness. Figure 11 indicates that the pavement thickness at which the minimum value of K occurs, varies with the ratio E_1/E_2.

SOME MISCELLANEOUS RELATIONSHIPS

Because they are required for some of the developments presented later, several useful miscellaneous relationships resulting from studies of the Canadian Department of Transport's load test and other data are summarized in this section.

Analysis of the large number of plate bearing tests made by the Canadian Department of Transport has resulted in Figure 12, the best average curve for load versus deflection for both subgrades and pavements.

Figure 13 shows the results of another analysis of a large number of load tests made on subgrades at Canadian airports with bearing plates of different diameters. It is based on a pressure of one unit per square inch on a 30-in. bearing plate at 0.2-in.

TABLE 1

DATA FOR ESTABLISHING RELATIONSHIP BETWEEN PAVEMENT FACTOR K AND PAVEMENT THICKNESS T; $E_1/E_2 = 10$

Pavement Thickness, T	Deflection Factor, F_W	$-\log F_W$	$K = \frac{T}{-\log F_W}$
0.25r	0.900	0.0458	5.46r
0.5r	0.760	0.1192	4.00r
1.0r	0.492	0.3080	3.27r
1.5r	0.373	0.4283	3.50r
2.0r	0.308	0.5114	3.91r
2.5r	0.269	0.5686	4.40r
3.0r	0.242	0.6144	4.88r
4.0r	0.208	0.6819	5.87r
5.0r	0.183	0.7376	6.82r
6.0r	0.173	0.7620	7.85r

deflection for 10 repetitions of load. If the supporting value of the subgrade has been measured at one deflection for one size of bearing plate, the load supported on the same or any other size of bearing plate at any deflection from 0 to 0.7 in. can be quickly determined. Consequently, Figure 13 makes it possible to obtain a maximum of load test information from a minimum of load testing effort.

Figure 14 is similar to Figure 13, but is based on a pressure of one unit per square inch on a 12-in. diameter bearing plate at 0.2-in. deflection for 10 repetitions of load. Because it is concerned with larger loaded areas, Figure 13 can be employed for airport pavement thickness design, whereas Figure 14 is more useful for thickness design for highway pavements.

Throughout the world, the CBR test is the most commonly used method for rating and expressing subgrade strengths. A relationship between plate bearing and CBR values is therefore desirable. During the Canadian Department of Transport's investigation of airport runways, in-place CBR and plate bearing tests were made on the subgrade at a large number of test locations. Figure 15 indicates the best average relationship that could be established between these in-place CBR ratings and load test values for 30-in. and 12-in. diameter bearing plates for each of six plate bearing deflection values.

Figures 12, 13, 14 and 15 provide useful relationships that have resulted from the analyses of hundreds of plate bearing tests made on existing airport runways by the Canadian Department of Transport. These relationships were developed because they enable a maximum of useful information to be obtained from a minimum of load test or similar measurements. It should be emphasized, however, that these relationships apply specifically to Canadian conditions and may require modification for use elsewhere.

CRITICAL PAVEMENT DEFLECTIONS

Pavements appear to fail, at least in part, by fatigue. Evidence of this is provided by the results of many Benkelman beam studies on actual highways (and airports) in recent years, which have shown that the critical deflection employed for the structural design of a pavement must be reduced as traffic volumes in terms of a given wheel load or its equivalent increase. For example, Benkelman beam studies made by the Committee on Pavement Design and Evaluation of the Canadian Good Roads Association (4) have shown that when designing for heavy highway traffic, 98 percent of the Benkelman beam measurements made with an 18,000-lb axle load (9,000 lb on dual

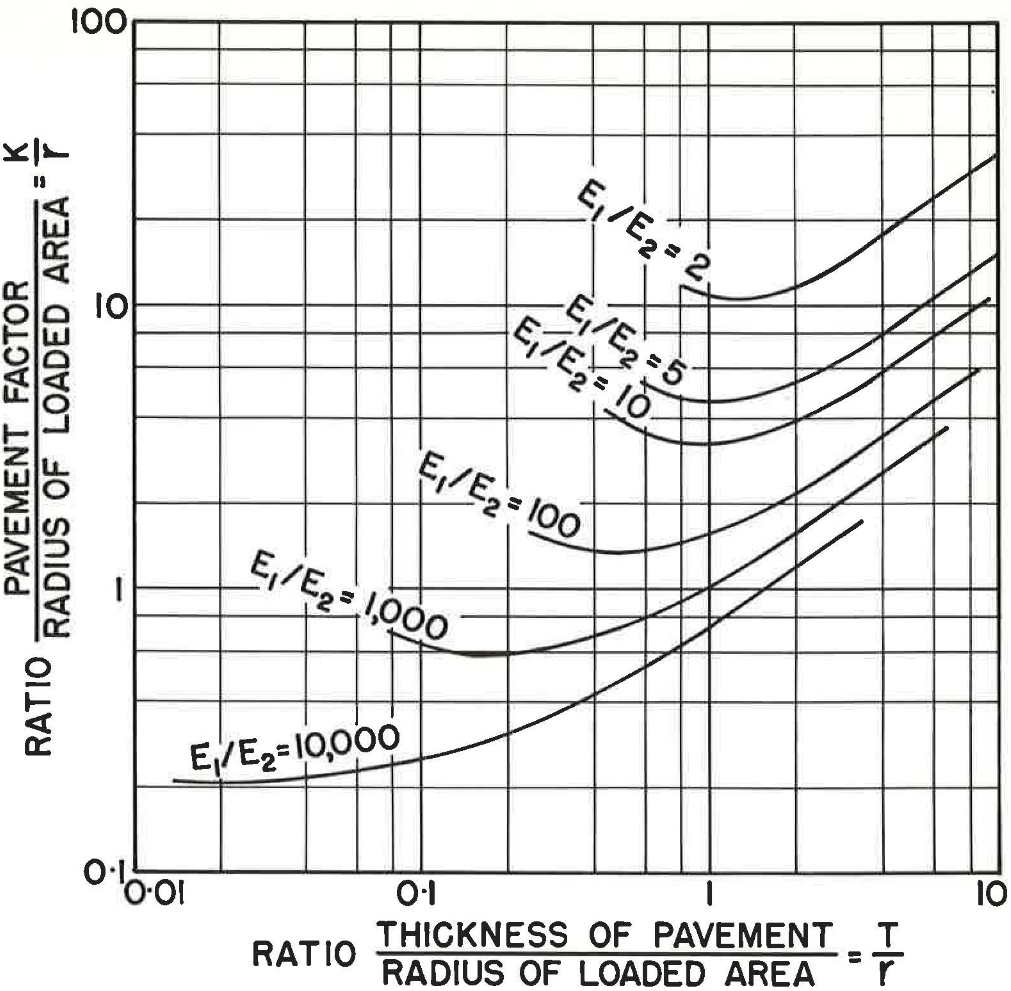


Figure 11. Theoretical relationship between pavement factor K and pavement thickness T.

tires) on a section of flexible pavement should show a rebound deflection less than 0.05 in. On the other hand, similar Benkelman beam studies have shown that lighter highway traffic can be carried on flexible pavements with rebound deflections of 0.1 in. or more.

Further evidence that pavements can fail primarily by fatigue when the pavement deflection that is critical for any particular combination of wheel load and traffic volume is exceeded, is provided by the WASHO Road Test report (5), which states: "Failures on the WASHO test were primarily due to bending and flexing of the pavement over a resilient soil and not attributable to plastic deformation resulting in displacement of the soil."

Because of the part played by fatigue in pavement failure as traffic volumes in terms of a given wheel load or its equivalent increase, the critical deflection employed for the structural design of the pavement must be decreased (Fig. 16). This reduces the amplitude of vertical movement at the surface of the pavement as vehicles pass over it, and thereby decreases the tendency for fatigue failure under the traffic volume expected.

In Figure 16, which pertains to pavements on a weak subgrade for which the corresponding CBR rating would be 3, all the doad-deflection curves are based on Figure 12.

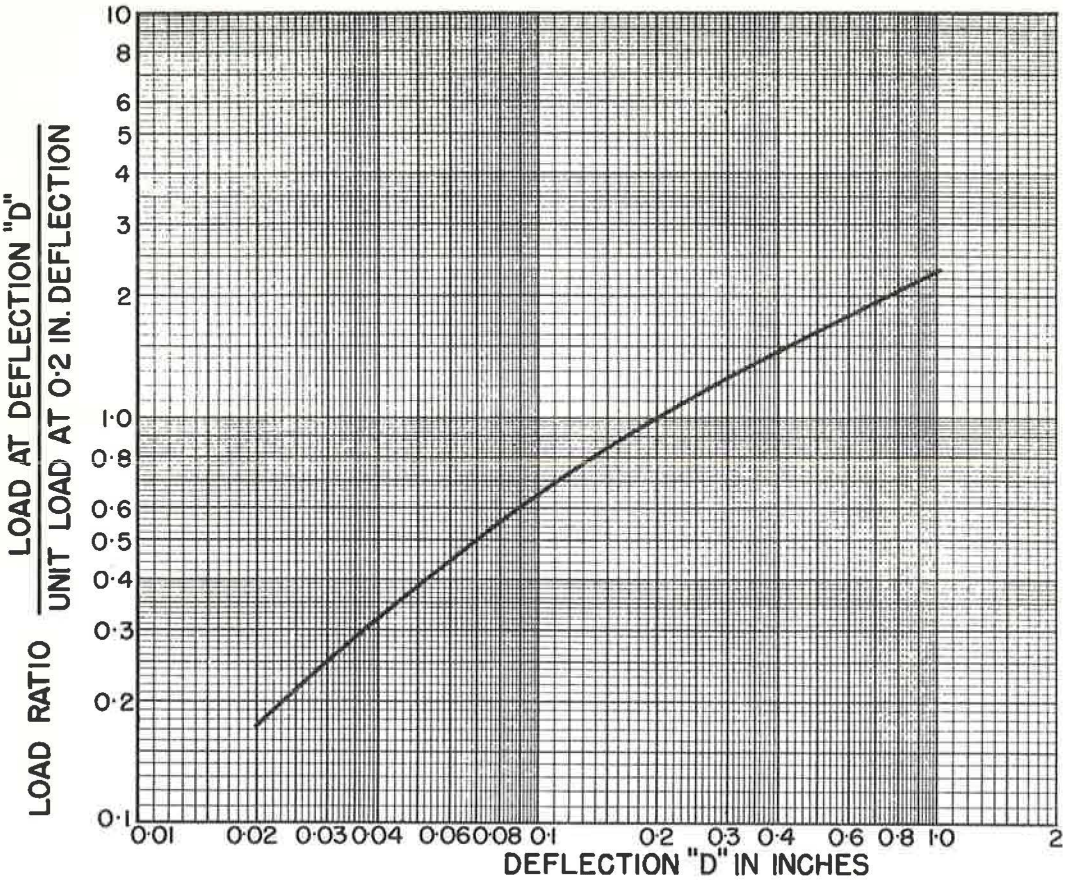


Figure 12. Average relationship between load and deflection for rigid bearing plates.

Figure 16 shows the influence that the deflection which is selected as being critical can have on pavement structural design. It will be observed from the subgrade curve that as the critical deflection is reduced, the corresponding subgrade supporting value may be decreased quite drastically; for example, from 4,640 lb on a 12-in. bearing plate at 0.5-in. deflection, to only 460 lb on the same bearing plate at 0.02-in. deflection. Consequently, if the design wheel load remains constant (for example, 9,000 lb) as the critical deflection is decreased from 0.5 in. to 0.02 in., either the pavement thickness or the elastic modulus of the pavement material, or both, must be increased (Fig. 9).

Incidentally, as a first impression Figure 16 might appear to indicate that a pavement designed on the basis of a certain traffic volume of 9,000-lb wheel loads or equivalent and a deflection of 0.1 in., could support traffic by a wheel load of about 14,000 lb at 0.2-in. deflection, or a wheel load of 22,000 lb at 0.5-in. deflection. However, if 0.1-in. deflection is a critical criterion for pavement design for the anticipated traffic volume of 9,000-lb wheel loads or equivalent, the pavement would obviously be expected to fail under the same traffic volume of 14,000 lb associated with 0.2-in. deflection or 22,000 lb associated with 0.5-in. deflection.

Figure 16 indicates that for a 9,000-lb wheel load, if 0.1-in. deflection is critical for a high traffic volume, 0.2-in. deflection would be satisfactory for a much smaller traffic volume. Figure 16 might also seem to suggest that a pavement capable of supporting a high volume of traffic of 9,000-lb wheel loads or equivalent at 0.1-in. deflection would also support a 14,000-lb wheel load at the much smaller traffic volume

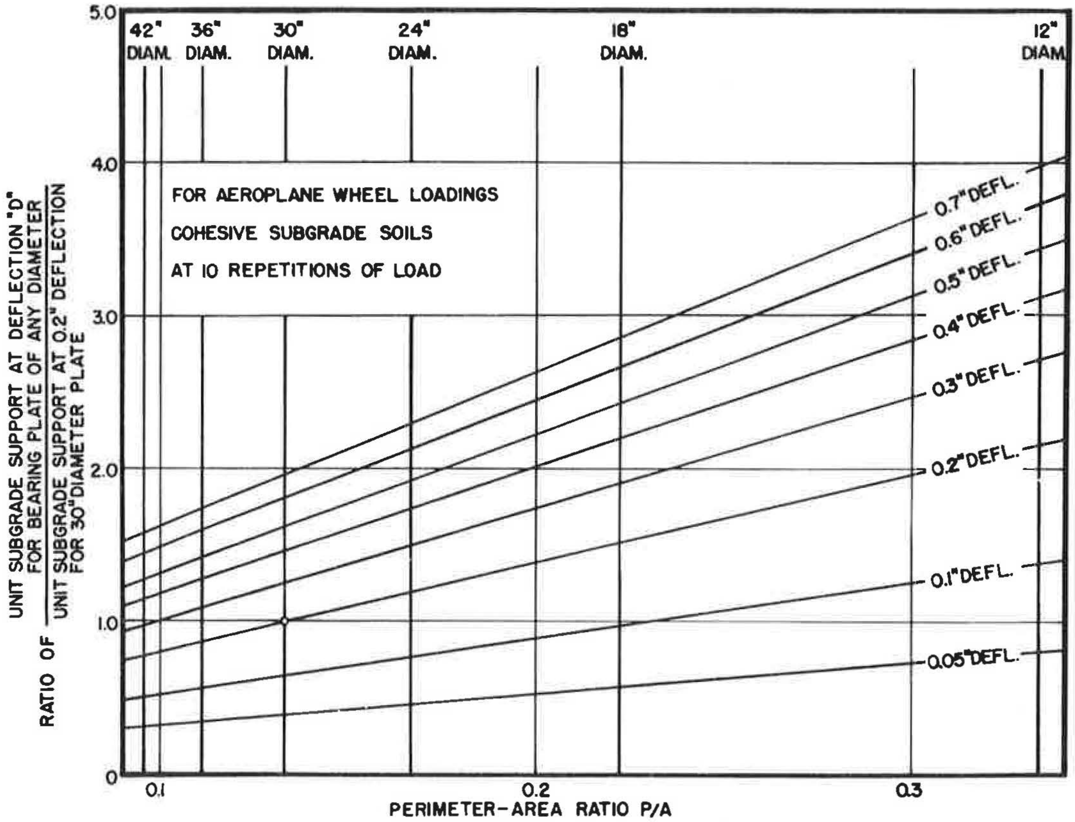


Figure 13. Ratio of subgrade support at deflection D for bearing plates of any diameter over subgrade support at 0.2-in. deflection on 30-in. diameter plate versus perimeter-area ratio.

associated with 9,000-lb wheel loads at 0.2-in. deflection. However, this requires substantiation. Nevertheless, experience has shown that a pavement designed for a given wheel load and traffic volume can support a limited volume of heavier wheel loads.

In addition to the effect of traffic volume for a given wheel load, the critical pavement deflection can be expected to be influenced by substantial differences in the size of the loaded area; for example, the larger contact areas of airport versus highway wheel loads, and probably in some cases at least by the radius of curvature.

EVALUATING THE ELASTIC MODULI E_1 AND E_2

Before Burmister's equations for a two-layer system (Eqs. 5 and 6) can be employed for pavement design, it is necessary to evaluate the pavement elastic modulus E_1 and the subgrade elastic modulus E_2 for each of the various pavement and subgrade materials available at any given project.

Each of the several test methods currently available for measuring values of E_1 and E_2 appear to be subject to serious criticisms. In part, this is due to the fact that for many subgrade and pavement materials the values of E_1 and E_2 obtained by static or very slow rates of loading are much lower than when the same materials are loaded dynamically at a high rate of loading. On the other hand, this does not seem to apply to purely granular materials. Figure 17 is a Mohr diagram for a purely granular material which possesses angle of internal friction but no cohesion (no intercept on the ordinate axis). Soil mechanics teaches that the Mohr envelope (and angle of internal

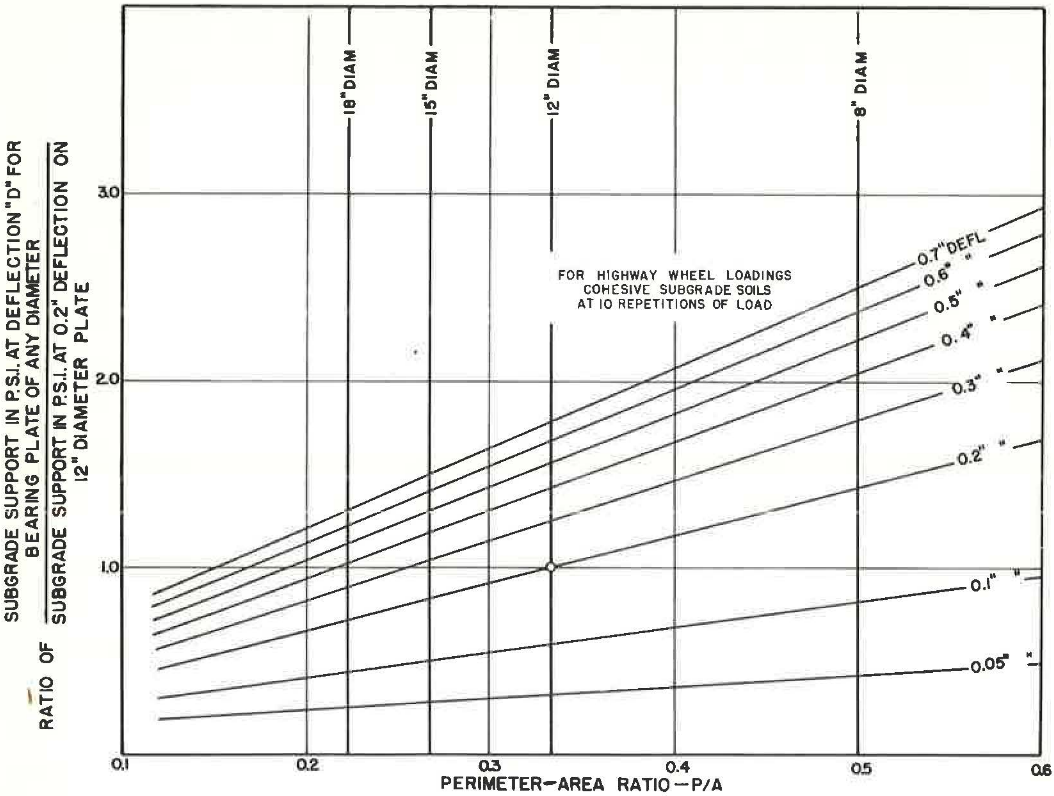


Figure 14. Ratio of subgrade support at deflection D for bearing plate of any diameter over subgrade support at 0.2-in. deflection on 12-in. diameter plate versus perimeter-area ratio.

friction) of such a purely granular material is substantially unaffected by the rate of loading. Consequently, for any purely granular material the static and dynamic values for the elastic modulus are assumed to be approximately equal.

Figure 18 is a Mohr diagram for a material that possesses both cohesion and angle of internal friction, such as an asphalt paving mixture. Many investigators have shown that although the angle of internal friction of this material is not noticeably influenced by the rate of loading, the cohesion is greatly affected. Figure 18 demonstrates that the cohesion is much greater for dynamic or a fast rate of loading than for static or a slow loading rate. The location of the Mohr envelope is also affected by the rate of loading. Therefore, as indicated by Figure 18, if under static loading the bearing capacity developed by the material is P_1, then under dynamic loading the bearing capacity developed will be the much higher value P_2. The difference between P_1 and P_2 can range from small to very large, depending on whether the dynamic loading rate is relatively slow or very fast, and on the material itself. The corresponding value for the elastic modulus of the material can also vary over a wide range, depending on the rate of loading.

When designing pavements for parking or other areas where vehicles will be stationary for a time, the values of E_1 and E_2 for static loading conditions are required. On the other hand, for the main pavements for highways and streets, and for airport runways except at the ends, values of E_1 and E_2 for dynamic conditions of loading should be used. Consequently, the same subgrade and pavement materials could have different values of E_2 and E_1, respectively, assigned to them for design purposes,

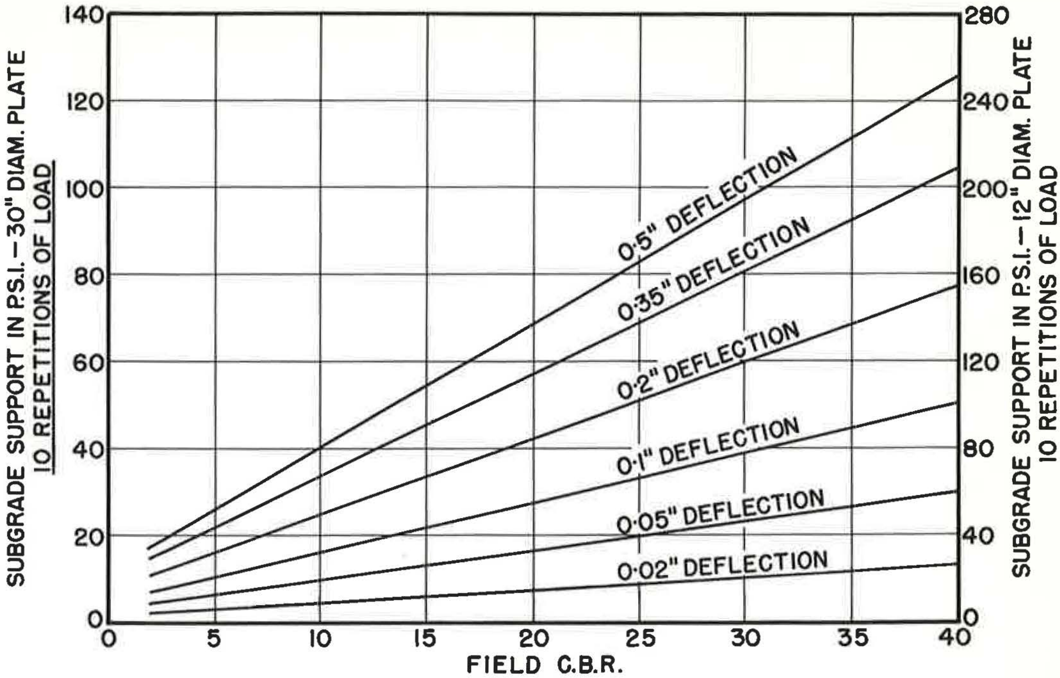


Figure 15. Relationships between plate bearing and in-place CBR subgrade values.

depending on whether the pavement is expected to support stationary or moving wheel loads.

In spite of much excellent work by a number of investigators (6, 7, 8, 9), there does not yet appear to be an entirely satisfactory method for measuring the dynamic modulus of elasticity of subgrade and pavement materials. Some of the criticisms of the various test methods tried are as follows: the inability of the test method to simulate the conditions of stress within the pavement structure that occur under normal moving traffic; the frequency of vibration employed results in a higher rate of loading than that of a vehicle traveling at ordinary high speed on a pavement, which leads in turn to measured moduli of elasticity that are too high; the complications introduced in the case of asphalt pavements by temperature variations, nature and quantity of the asphalt binder, paving mixture composition, method and degree of compaction, etc.; and the inability of the test method to pinpoint the value of the dynamic modulus of elasticity, so that only approximate values with a wide margin of uncertainty are provided.

Hybla Valley plate bearing tests (2) provide a great deal of useful information concerning values of what are usually referred to as "static" moduli of elasticity. As previously pointed out, the Hybla Valley load test data are particularly useful for this purpose because of the care taken to obtain uniformity of subgrade, base course, and asphalt pavement materials, and to achieve uniformity of construction, and because of the wide range of base course thicknesses employed (6, 12, 18, and 24 in.) and the substantial range of bearing plate diameters (12, 18, 24, and 30 in.) used for the load tests. It should be noted that all of the load test values in Figures 19, 20, 21, 22 and 23, are for a deflection of 0.2 in.

Figure 19 plots unit load versus size of bearing plate for load tests made at the surface of 6, 12, 18 and 24 in. of granular base course from which 3 in. of asphalt concrete surface course were removed just prior to each test. Each plate size is expressed in terms of its perimeter-area (P/A) ratio, which is numerically equal to 2/r, where r is the radius of the bearing plate. Figure 19 shows that for all four thicknesses of granular base a straightline relationship occurs, a relationship clearly established for plate load tests many years ago by Housel (10) and confirmed by analyses of many

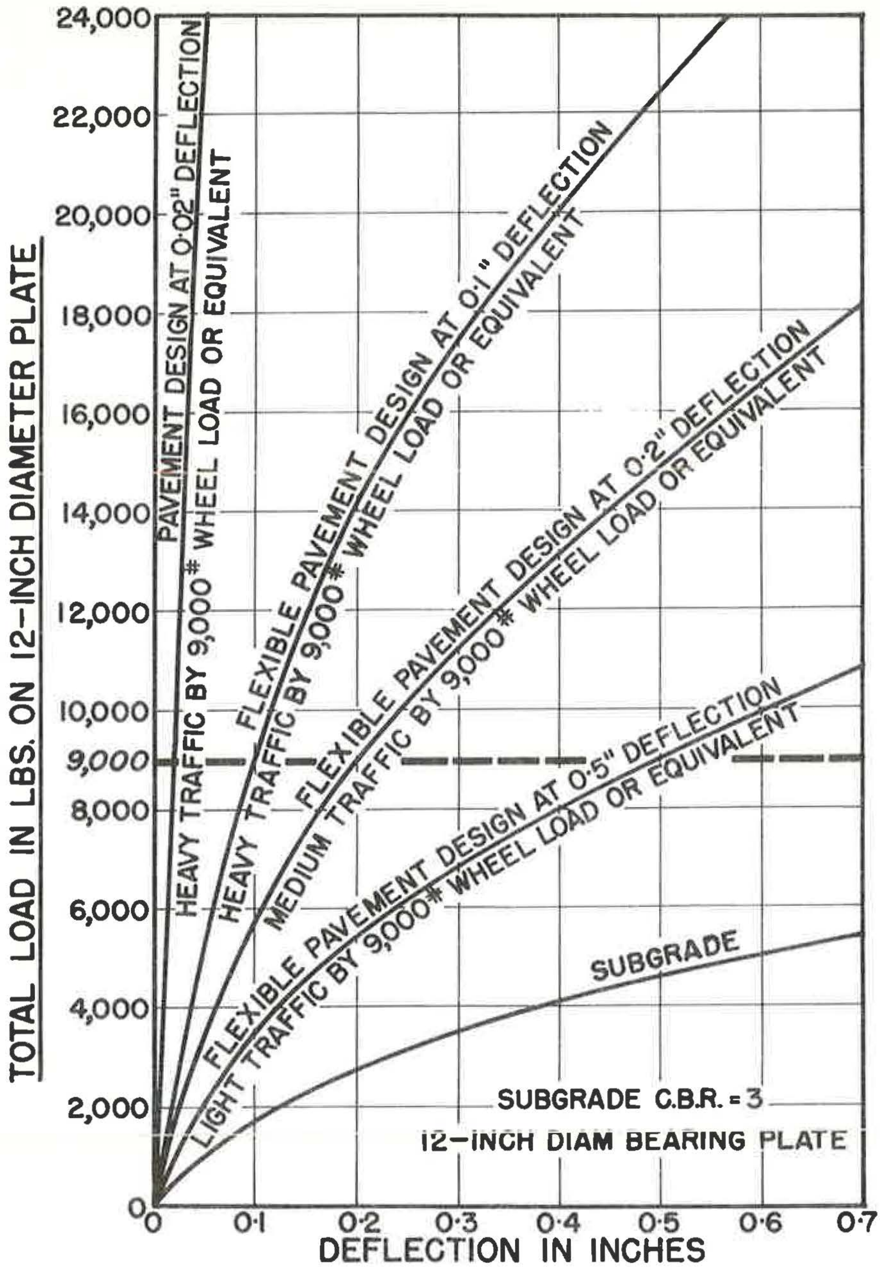


Figure 16. Influence of critical pavement deflection on pavement design.

hundreds of plate bearing tests made on airport runways by the Canadian Department of Transport.

Figure 20 plots unit load versus size of bearing plate for load tests made on the Hybla Valley subgrade after overlying thicknesses of 6, 12, 18 and 24 in. of granular base plus 3 in. of asphalt concrete surface course were removed just prior to each test. Figure 20 indicates that possibly as a result of the compaction of the superimposed base course and asphalt surface during construction, a somewhat stronger subgrade appeared to have been developed under the thicker than under the thinner depths of overlying base and surface material.

Analysis of the data has shown that more plate bearing tests on the subgrade during the Hybla Valley project would have been desirable. However, although this would have increased the precision of the findings based on these data, it would not have changed the general conclusions presented in this section of the paper.

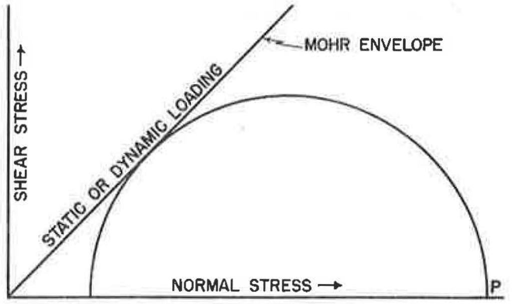


Figure 17. Mohr diagram for purely granular material.

Figure 21, taken from Figures 19 and 20, illustrates unit load versus the P/A ratios for four bearing plate sizes for load tests made on the surface of 12 in. of granular base, and on the surface of the underlying subgrade. Similar graphs can be prepared from the load test data contained in Figures 19 and 20 for the other combinations of granular base course thickness and corresponding underlying subgrade support illustrated.

The Hybla Valley load test data (Figs. 19, 20, and 21) can be analyzed by Burmister's layered system theory (Eqs. 5 and 6) to provide values for the elastic modulus E_1 of the granular base course material (Table 2), and for the elastic modulus E_2 of the subgrade (Table 3).

Table 2 shows that in spite of the great care taken to obtain a uniform granular base, and the unusual precautions and precision employed when making the load tests, the value of E_1 for the granular base course may not be a constant independent of thickness of base and size of loaded area, as is usually assumed when applying the Burmister layered system theory. Table 2 shows that E_1 may vary with bearing plate diameter when thickness is held constant (for example, from 7,390 psi for a 30-in. plate to 13,590 psi for an 18-in. diameter plate when the thickness of granular base was 12 in.) The data also seem to indicate that E_1 may vary somewhat with thickness of granular base when the bearing plate diameter is kept constant. On the other hand, the largest variation in E_1 with bearing plate size occurs for the 6-in. base course thickness, whereas the E_1 values are almost constant regardless of bearing plate diameter for the base course thickness of 24 in. In view of the inevitable scatter of data associated with plate bearing tests, it is not possible to decide on the basis of Table 2 whether the variation in E_1 values is real, or is due to normal experimental error. This factor requires further investigation.

Table 3 demonstrates that the elastic modulus of the subgrade E_2 can vary over a considerable range depending on the diameter of the bearing plate, and on the thickness of the overlying base course and surfacing materials. For example, E_2 increased from 1,580 psi for the 12-in. plate to 2,290 psi for the 30-in. plate when the overlying thickness of granular base removed was 24 in. Also, when measured with the 18-in. bearing plate, E_2 increased from 1,350 psi when the removed overlying thickness of granular base was 6 in. to 1,820 psi when the overlying base thickness removed was 24 in. In the latter case, part of the increase in E_2 might be explained as an increase in subgrade strength resulting from the compaction employed for the greater depth of overlying base.

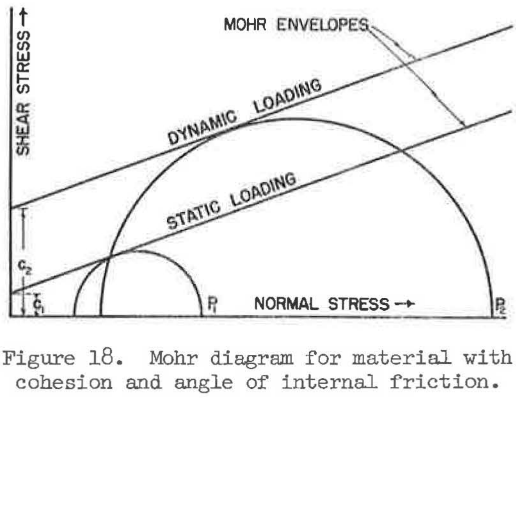


Figure 18. Mohr diagram for material with cohesion and angle of internal friction.

In Figure 22, the solid lines (1) and (2) represent load test data for the Hybla Valley subgrade measured with several bearing plate sizes. The broken lines

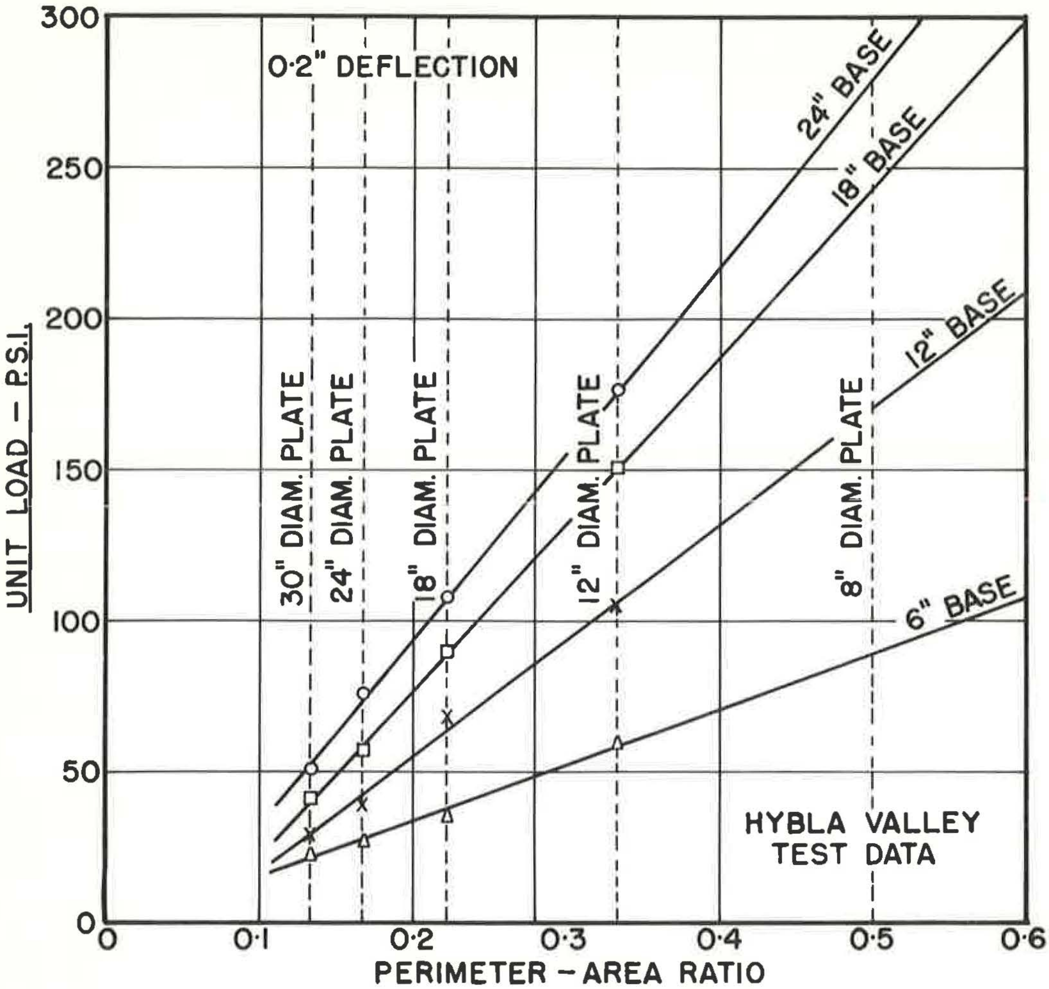


Figure 19. Influence of bearing plate diameter and thickness of granular base on load supporting values.

represent supporting values for the same bearing plate sizes, as calculated by means of the Boussinesq elastic equation (Eq. 5) employed in the Burmister method of design.

Solid line (1) in Figure 22 represents subgrade supporting values measured with four rigid plates (12-, 18-, 24-, and 30-in. diameter) on the subgrade underlying 12 in. of granular base (Fig. 21). Solid line (2) represents subgrade supporting values measured at another location on the Hybla Valley project with seven rigid plates (12-, 18-, 24-, 30-, 42-, 60-, and 84-in. diameter) on subgrade underlying 8 in. of granular base.

The broken lines radiating from the origin in Figure 22 provide subgrade load supporting values for different sizes of bearing plates, and represent different values for the subgrade elastic modulus E_2 given by Eq. 5 after rearranging to

$$s = \frac{w_S E_2}{1.18r} = \frac{w_S E_2}{2.36} \times \frac{2}{r} = \frac{w_S E_2}{2.36} \times \frac{P}{A} \qquad (5a)$$

in which P is the perimeter of the loaded area, A is the area of the loaded area, and the other symbols are as already defined.

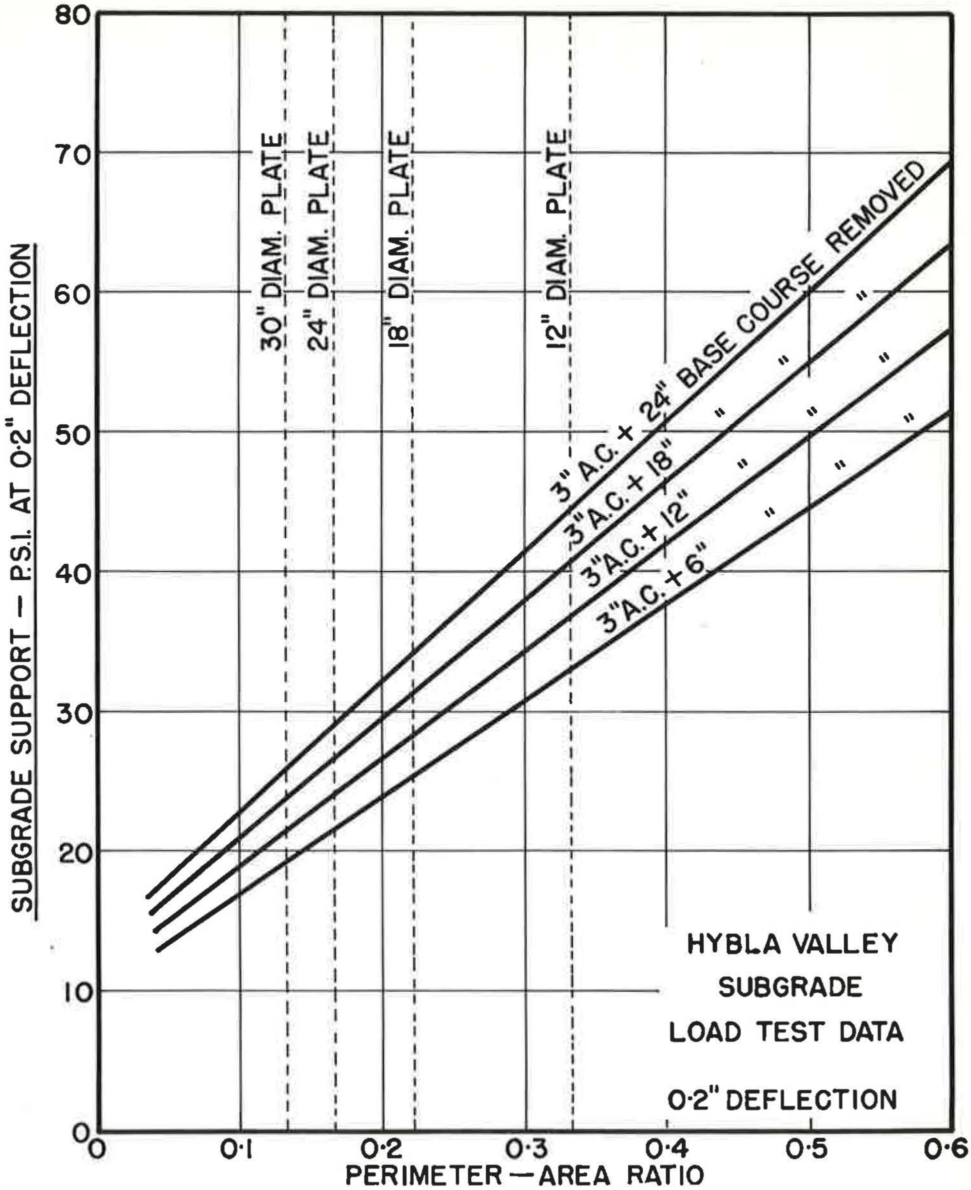


Figure 20. Hybla Valley subgrade load test values.

In Figure 22, the solid lines representing values of subgrade support measured with different sizes of bearing plates on the Hybla Valley subgrade have a flatter slope than the broken lines representing subgrade supporting values calculated by means of the Boussinesq equation for the same bearing plate sizes and for different values of E_2. Furthermore, although the latter lines go through the origin, extensions of the former toward the left would make positive intercepts with the ordinate axis.

Figure 23 compares typical Canadian Department of Transport subgrade load test values for different bearing plate sizes with those provided by the Boussinesq equation.

Figures 22 and 23 demonstrate that actual load test data obtained with different bearing plate sizes on these subgrades do not agree with those given by the Boussinesq equation. Because the lines representing measured subgrade supporting values inter-

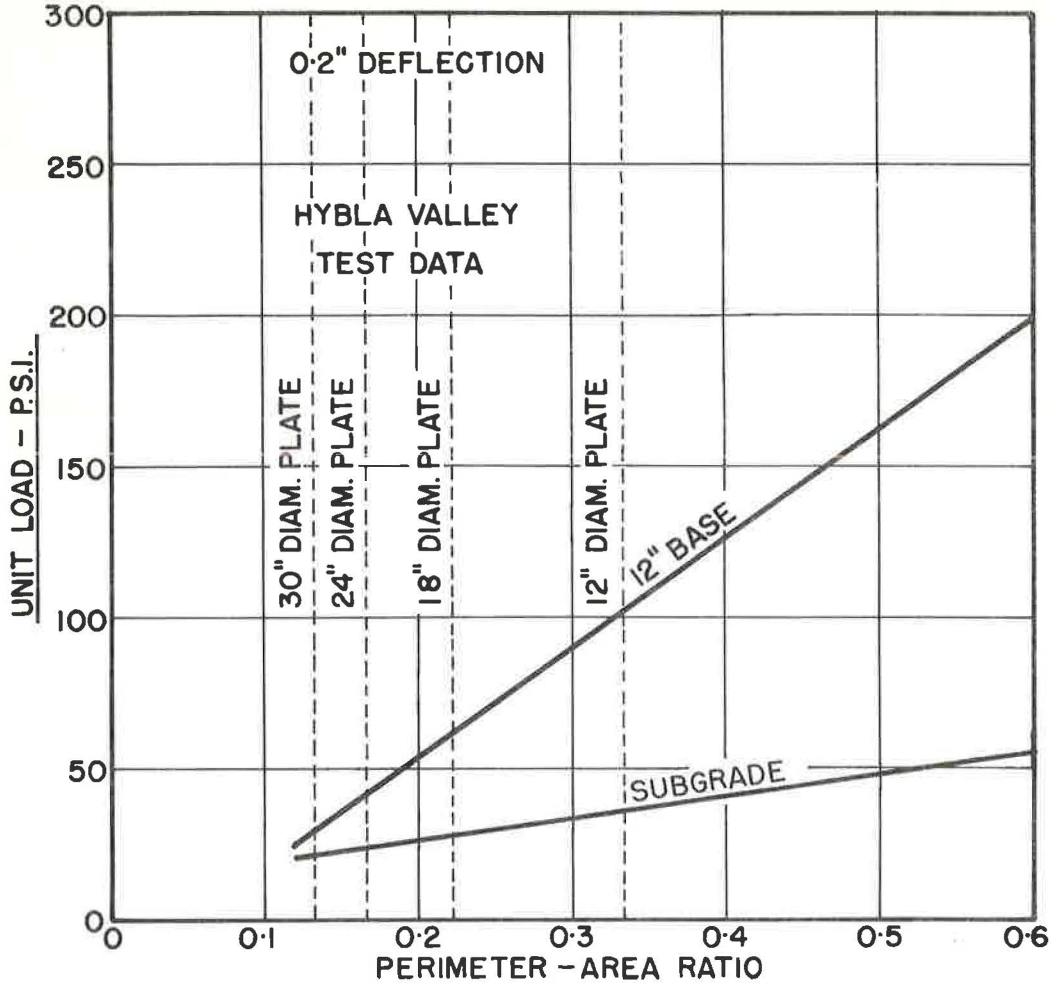


Figure 21. Unit load versus bearing plate diameter for subgrade and 12-in. thickness of granular base course.

sect those provided by the Boussinesq equation, each of these actual subgrades has not one value of E_2, but a different value of E_2 for each size of bearing plate employed.

The Hybla Valley load test data (2) enable the influence of deflection on the values of E_1 and E_2 to be investigated. Figure 24 shows load-deflection curves for the Hybla Valley subgrade and base course obtained by plotting the published data for a 30-in. diameter bearing plate. Figure 25 provides similar information based on data from 12-in. diameter plate bearing tests. For both Figures 24 and 25, the base course load tests were made on the surface of 18 in. of granular base immediately after removing 3 in. of asphalt concrete; the subgrade load tests were performed on the subgrade immediately after removing the overlying 18 in. of base course plus 3 in. of asphalt concrete.

From Figures 24 and 25 it is evident that the shape of the load-deflection curves for both subgrade and base course is concave downward. This is typical. The subgrade elastic modulus E_2 determined for any given deflection from the subgrade load test curves in Figures 24 and 25 is a secant modulus. Consequently, because the slope of the secant becomes steeper E_2 becomes increasingly greater as the deflection for which it is calculated becomes smaller. For any given size of bearing plate, there-

TABLE 2

INFLUENCE OF THICKNESS OF GRANULAR BASE AND OF BEARING PLATE DIAMETER ON THE VALUE OF THE ELASTIC MODULUS E_1 OF THE GRANULAR BASE COURSE MATERIAL (HYBLA VALLEY)

Plate Diam. (in.)	E_1			
	6-In. Base	12-In. Base	18-In. Base	24-In. Base
12	7,490	10,380	11,670	11,200
18	10,900	13,590	12,930	12,160
24	12,890	10,870	12,000	12,310
30	6,790	7,390	8,580	10,760

TABLE 3

INFLUENCE OF BEARING PLATE DIAMETER AND THICKNESS OF OVERLYING GRANULAR BASE COURSE ON THE VALUE OF THE SUBGRADE ELASTIC MODULUS E_2 (HYBLA VALLEY)

Plate Diam. (In.)	Value of E_2			
	6-In. Base	12-In. Base	18-In. Base	24-In. Base
12	1,170	1,310	1,440	1,580
18	1,350	1,500	1,660	1,820
24	1,520	1,700	1,880	2,050
30	1,700	1,900	2,090	2,290

fore, the value of E_2 for each subgrade depends on the subgrade deflection selected for its evaluation.

On the basis of the load-deflection curves for the Hybla Valley subgrade and base course obtained with a 30-in. bearing plate (Fig. 24), values for E_2 and E_1 have been calculated (Table 4) by means of the Boussinesq and Burmister equations (Eqs. 5 and 6) and Figure 10. Similar information based on a 12-in. bearing plate and Figure 25 are given in Table 5.

It is clear from Tables 4 and 5 that, as expected from the increasing slope of the secant, for any given bearing plate size E_2 increases as the deflection for which it is calculated decreases. In Table 4, for example, E_2 increases from 1,660 psi to 3,520 psi as the deflection decreases from 0.5 to 0.02 in. It is also apparent from Tables 4 and 5 that, for any given subgrade deflection, the value of E_2 increases as the size of the bearing plate employed for its determination is increased. For a deflection of 0.1 in. for example, Table 4 indicates that E_2 = 3,100 psi when determined with a 30-in. bearing plate, whereas Table 5 shows that E_2 = 1,800 psi when obtained from a load test made with a 12-in. plate.

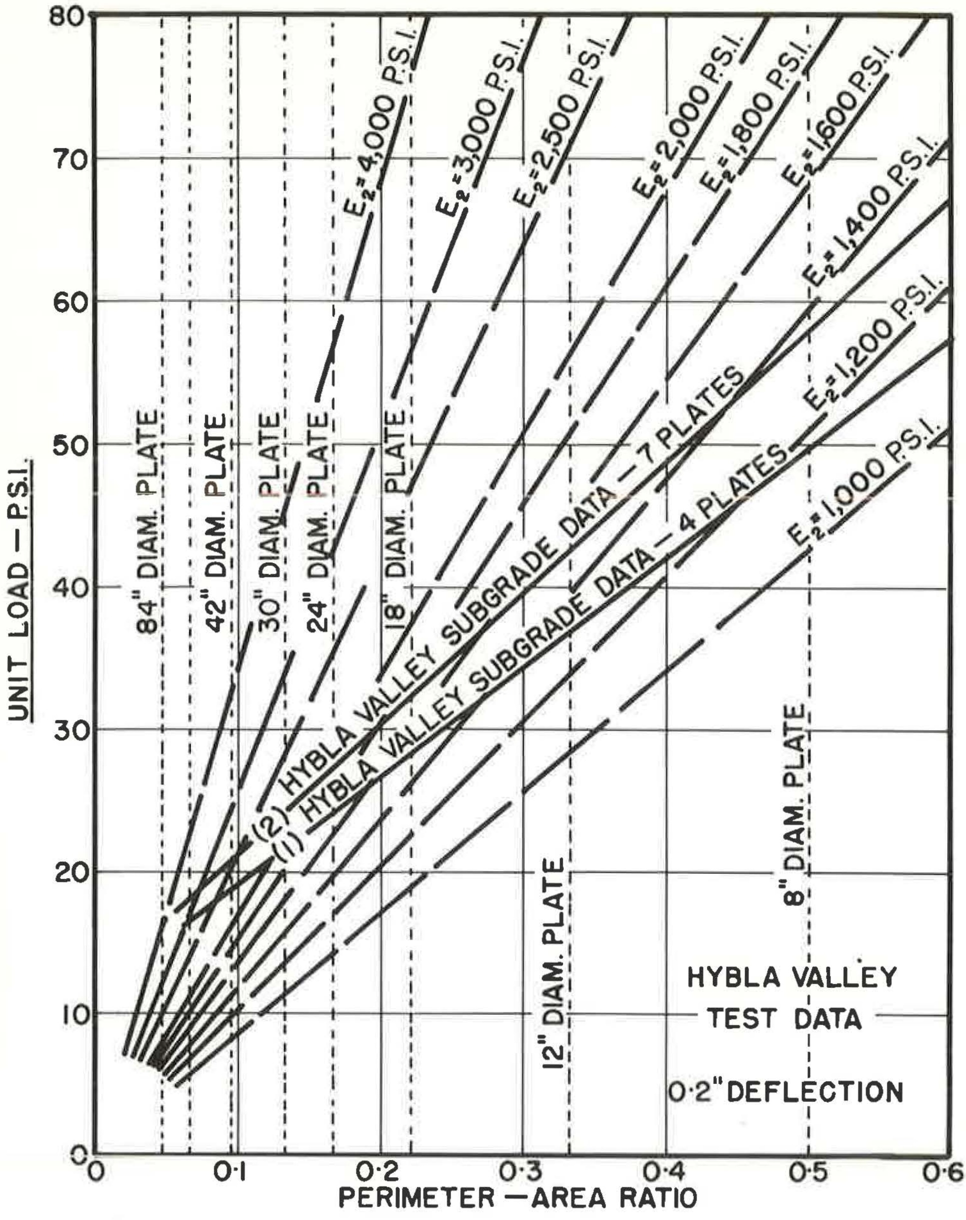


Figure 22. Comparison of subgrade supporting values calculated from Boussinesq equation with actual values measured for the Hybla Valley subgrade.

Tables 4 and 5 tend to indicate that the value of the pavement elastic modulus E_1 may become gradually smaller as the deflection used for its determination is decreased. With the 30-in. bearing plate (Table 4), E_1 decreases from 11,600 psi to 4,925 psi as the deflection is decreased from 0.5 in. to 0.02 in. However, Table 5 shows that the reduction in values of E_1 for the same decrease in deflection is much less for a 12-in. plate. For each of the larger deflections (0.4 and 0.5 in.) Tables 4 and 5 indicate that the value of E_1 tends to be almost the same when determined with either the 12-in. or 30-in. bearing plates. For the smaller deflections, on the other hand, the value of E_1 provided by the 30-in. bearing plate is much smaller than that given by the 12-in. plate.

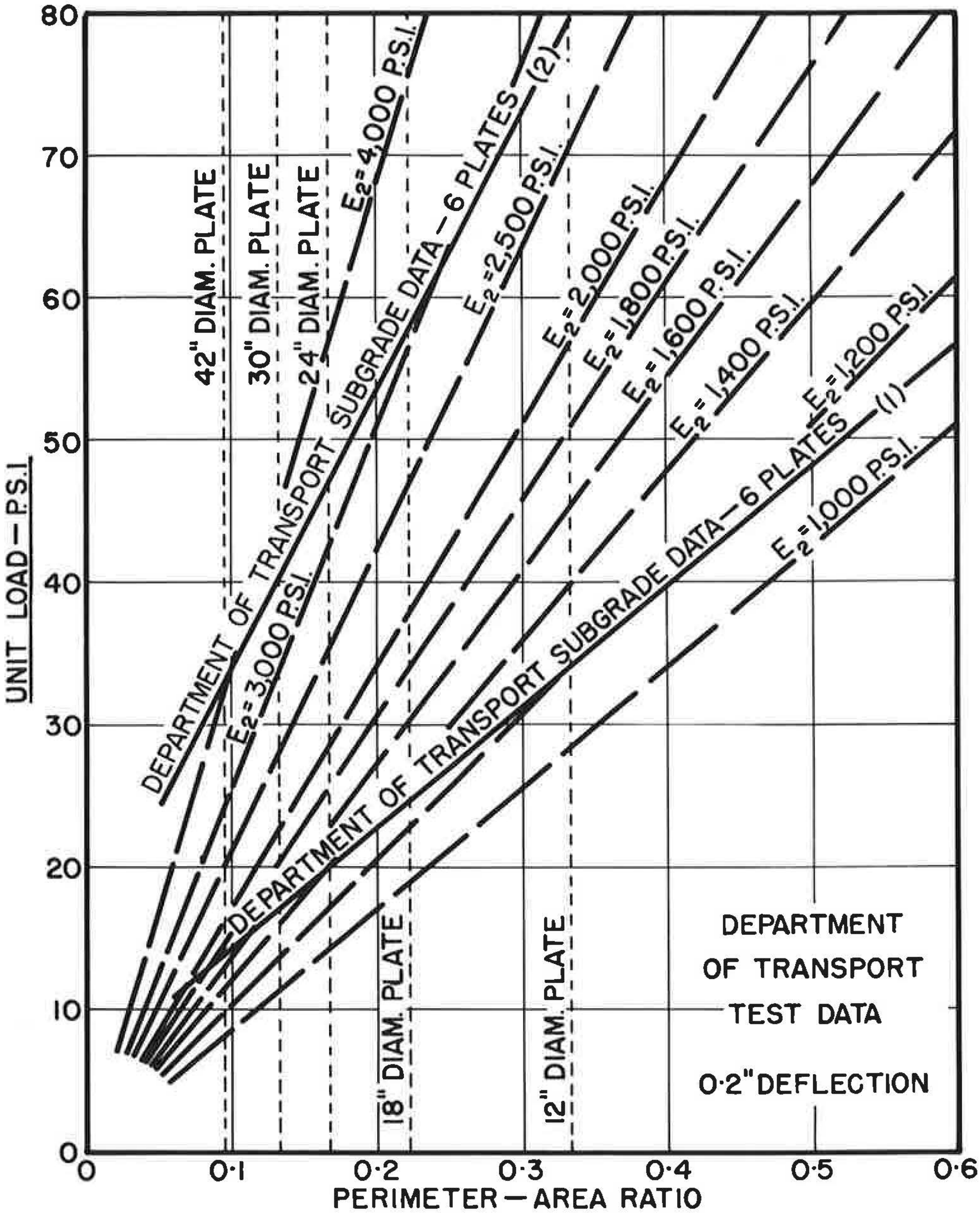


Figure 23. Comparison of subgrade supporting values calculated from Boussinesq equation with actual values measured by the Department of Transport.

At 0.1-in. deflection, for example, Table 4 shows that E_1 is 5,580 psi when obtained by means of a 30-in. diameter plate, whereas Table 5 indicates that E_1 is 10,350 psi when determined by means of a 12-in. diameter plate.

When considering the differences between E_1 values in Tables 4 and 5, it is again necessary to keep in mind the considerable scatter of data ordinarily associated with the most carefully conducted load tests. Consequently, additional investigation is required to establish how much of the reduction in E_1 values with corresponding decrease in deflection indicated by Table 4, for example, actually occurred, and how much, if any, is due to normal experimental error.

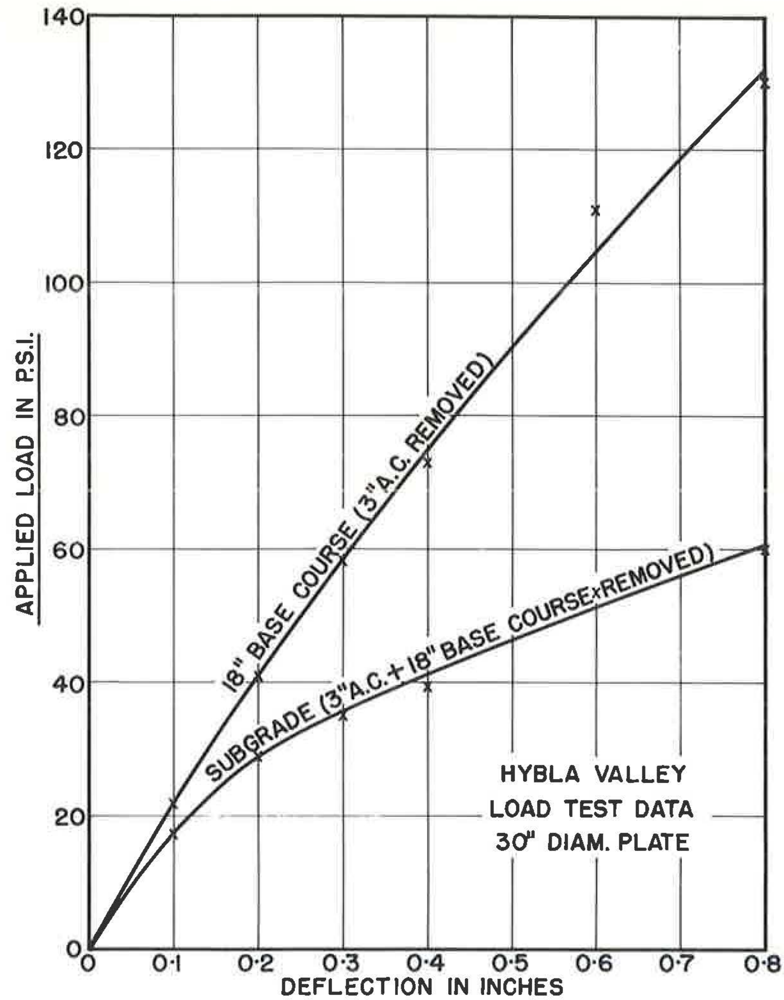


Figure 24. Load-deflection curves for Hybla Valley subgrade and base course (30-in. bearing plate).

In summarizing this section on the determination of values for the pavement elastic modulus E_1 and the subgrade elastic modulus E_2, the difference between values provided by static and dynamic methods of testing is recognized. A simple, rapid, precise method (or methods) for determining representative values of E_1 and E_2 for the most critical condition to which the pavement and subgrade materials will be subjected in service, still remains to be developed.

Plate bearing tests can be employed to establish static elastic modulus values for E_1 and E_2. Depending on the nature of the subgrade and pavement materials, these static values could be much too conservative for moving load conditions. Plate bearing test data indicate that the static value of E_2 for any given subgrade can vary over a wide range, depending on the bearing plate size and the deflection employed for its determination. The static value of E_1 for any given pavement material, as determined by means of plate bearing tests, may also vary with bearing plate size and with deflection. In addition, depending on the characteristics of the pavement materials, the value of E_1 may change with temperature, moisture content, degree of compaction, etc.

When using plate bearing tests to evaluate E_1 and E_2, therefore, the measurements

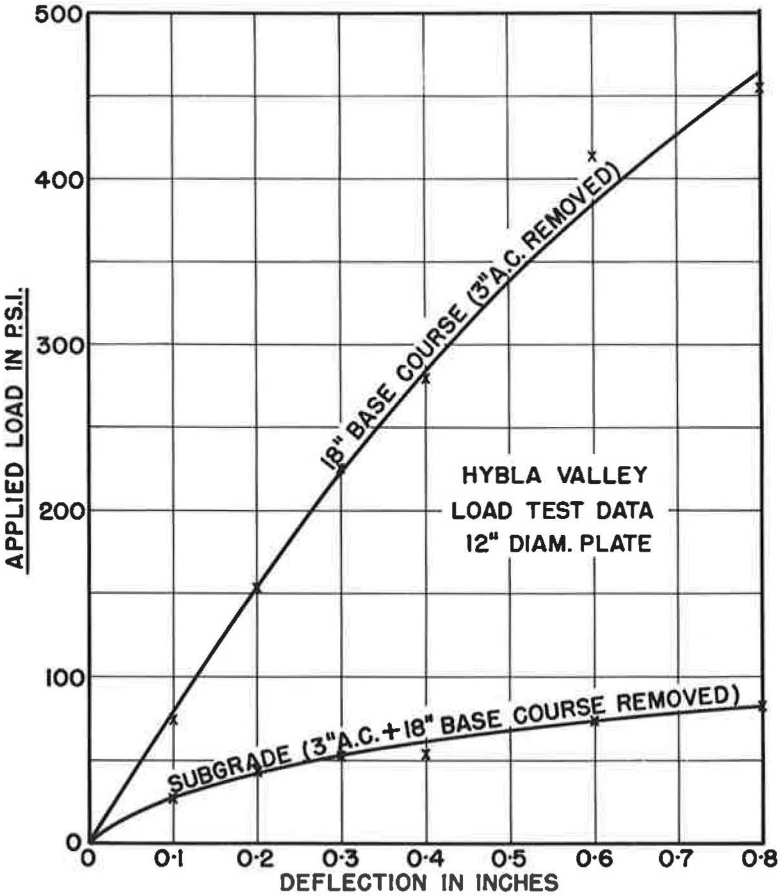


Figure 25. Load-deflection curves for Hybla Valley subgrade and base course (12-in. bearing plate).

should be made at the deflection specified or selected for pavement design, with the pavement and subgrade materials in the condition likely to be most critical in service, and with a bearing plate equal in size to the contact area of the design wheel load.

To obtain more realistic working values for the static moduli of elasticity E_1 and E_2, it is recommended that the plate bearing test employed be of the repetitive type (for example, ASTM Method D1195, and The Asphalt Institute's "Soils Manual," Chapter 9).

PLATE LOADING VERSUS WHEEL LOADING

The usefulness of plate bearing tests for pavement design and evaluation is sometimes questioned. The usual criticism implied is that a pavement is stressed differently by a bearing plate than by a wheel load on a pneumatic tire.

Some recent measurements made by the Canadian Department of Transport on airport runway pavements (Fig. 26) seem to provide an answer to this criticism. Sebastyan (11) conducted Benkelman beam measurements with a wheel load of 9,000 lb and load tests with a 30-in. diameter bearing plate at the same locations on pavement surfaces at a number of airport runways. Twelve Benkelman beam measurements were made and averaged at each plate bearing test location. The flexible pavement thicknesses at the test locations ranged from 6 to 50 in., the pavement deflections provided by the Benkelman beam measurements varied from about 0.01 to about 0.4 in.,

TABLE 4

INFLUENCE OF DEFLECTION OF LOADED 30-IN. BEARING PLATE ON E_1 AND E_2 VALUES; HYBLA VALLEY LOAD TEST DATA

$$\left(\begin{array}{c}\text{Base course thickness, 18 in.}\\ \frac{T}{r} = \frac{18}{15} = 1.2\end{array}\right)$$

Deflection (in.)	Surface Load, P (psi)	Subgrade Load, S (psi)	Deflection Factor, $F_W = S/P$	E_1/E_2	E_2 (psi)	E_1 (psi)
0.02	4.6	4.0	0.87	1.4	3,520	4,925
0.05	11.1	9.5	0.86	1.5	3,360	5,040
0.1	21.7	17.5	0.81	1.8	3,100	5,580
0.2	41.8	28.9	0.69	3.0	2,560	7,780
0.3	61.2	36.1	0.59	4.2	2,130	8,950
0.4	79.0	42.0	0.53	5.5	1,860	10,200
0.5	94.2	46.9	0.50	7.0	1,660	11,600

TABLE 5

INFLUENCE OF DEFLECTION OF LOADED 12-IN. BEARING PLATE ON E_1AND E_2 VALUES; HYBLA VALLEY LOAD TEST DATA

$$\left(\begin{array}{c}\text{Base course thickness, 18 in.}\\ \frac{T}{r} = \frac{18}{6} = 3.0\end{array}\right)$$

Deflection (in.)	Surface Load, P (psi)	Subgrade Load, S (psi)	Deflection Factor, $F_W = S/P$	E_1/E_2	E_2 (psi)	E_1 (psi)
0.02	17	6.5	0.38	4.3	2,300	9,900
0.05	40	15.0	0.37	4.6	2,110	9,700
0.1	78	26.5	0.34	5.5	1,880	10,350
0.2	153	42.0	0.27	8.0	1,490	11,900
0.3	225	56.0	0.25	9.5	1,320	12,500
0.4	291	73.0	0.25	9.5	1,290	12,250
0.5	350	87.0	0.25	9.5	1,230	11,700

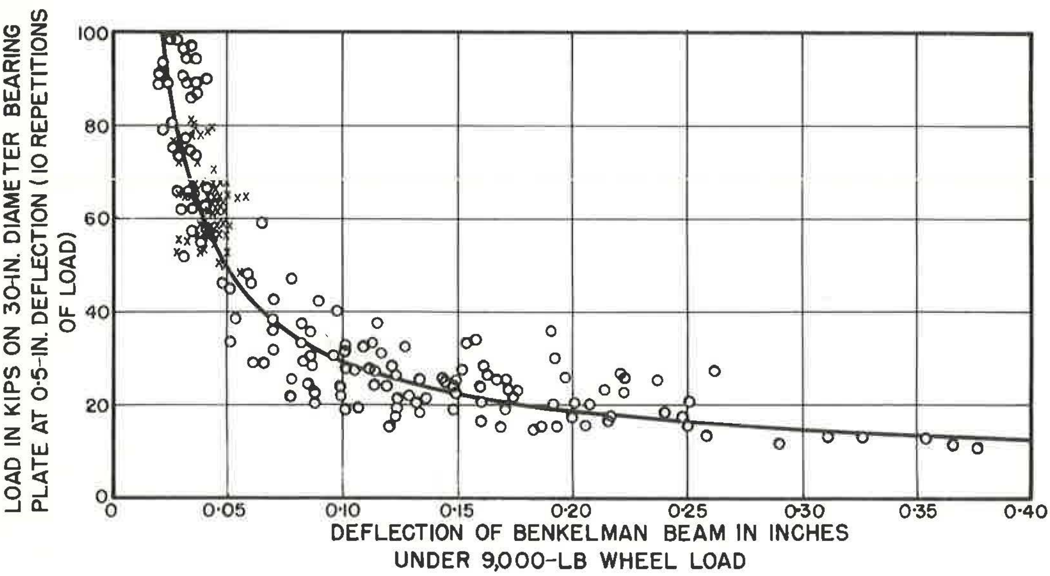


Figure 26. Relationship between Benkelman beam deflection for 9,000-lb wheel load and load on 30-in. diameter bearing plate at 0.5-in. deflection for 10 repetitions of load.

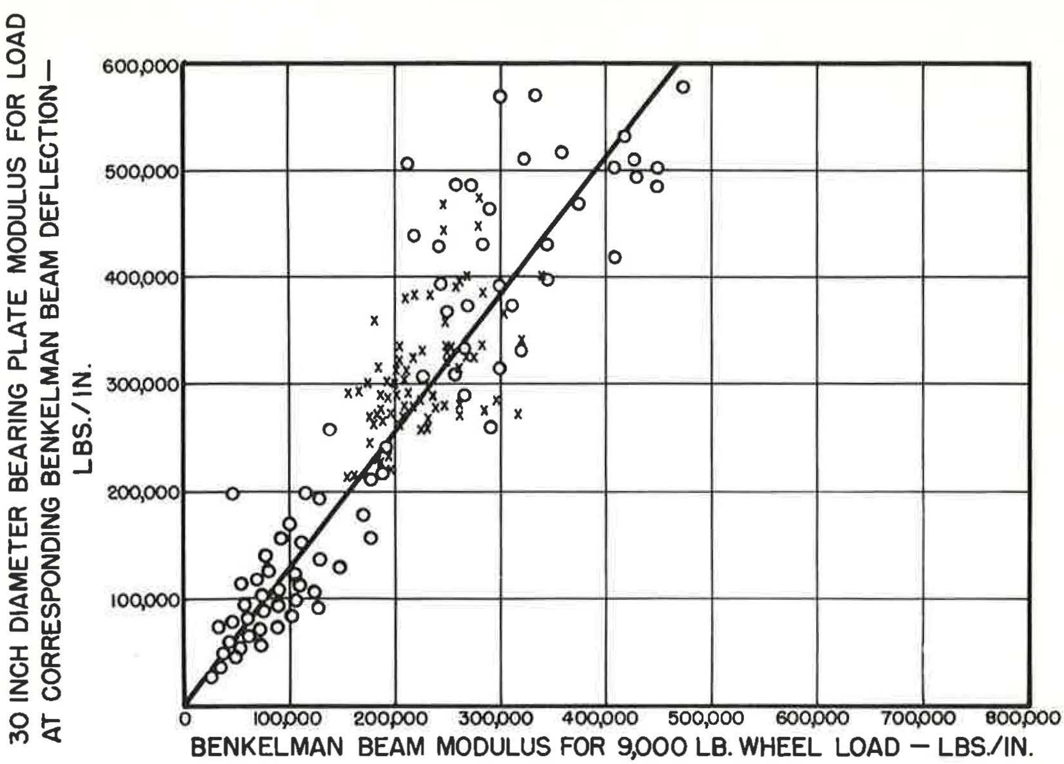


Figure 27. Relationship between Benkelman beam modulus for 9,000-lb wheel load and 30-in. diameter bearing plate modulus for load at the corresponding Benkelman beam deflection.

and the load on the 30-in. bearing plate for 0.5-in. deflection (10 repetitions of load) ranged from 10,000 to 100,000 lb (Fig. 26).

The general shape of the curve of Figure 26 suggested that a relationship might exist between secant moduli for the plate bearing data and the corresponding secant moduli provided by the deflections from the Benkelman beam measurements. For each point in Figure 26, by means of Figure 12 the load on the 30-in. bearing plate at 0.5-in. deflection was converted to its corresponding 30-in. plate bearing load at the Benkelman beam deflection indicated by the same plotted point. The secant modulus for the load on the 30-in. plate at this deflection was expressed as total load in pounds per 1-in. deflection. Similarly, for the same point, the corresponding Benkelman beam secant modulus was obtained by dividing the wheel load of 9,000 lb by the Benkelman beam deflection, and was expressed as total load in pounds per 1-in. deflection. Analysis of the data of Figure 26 yielded Figure 27, in which the line of best fit was established by the method of least squares. A linear relationship appears to exist between corresponding secant moduli for plate bearing and Benkelman beam ratings for the various test locations.

Transfer of the straightline relationship of Figure 27 back to Figure 26 results in the curved line, which represents the plotted data quite accurately.

Figures 26 and 27 indicate a close correlation between Benkelman beam and plate bearing measurements. Furthermore, the linear relationship in Figure 27 between the secant moduli for corresponding plate bearing and Benkelman beam ratings seems to imply that a flexible pavement structure reacts in the same way insofar as its stress-strain characteristics are concerned, whether the load is applied by dual pneumatic tires (Benkelman beam), or by a steel bearing plate.

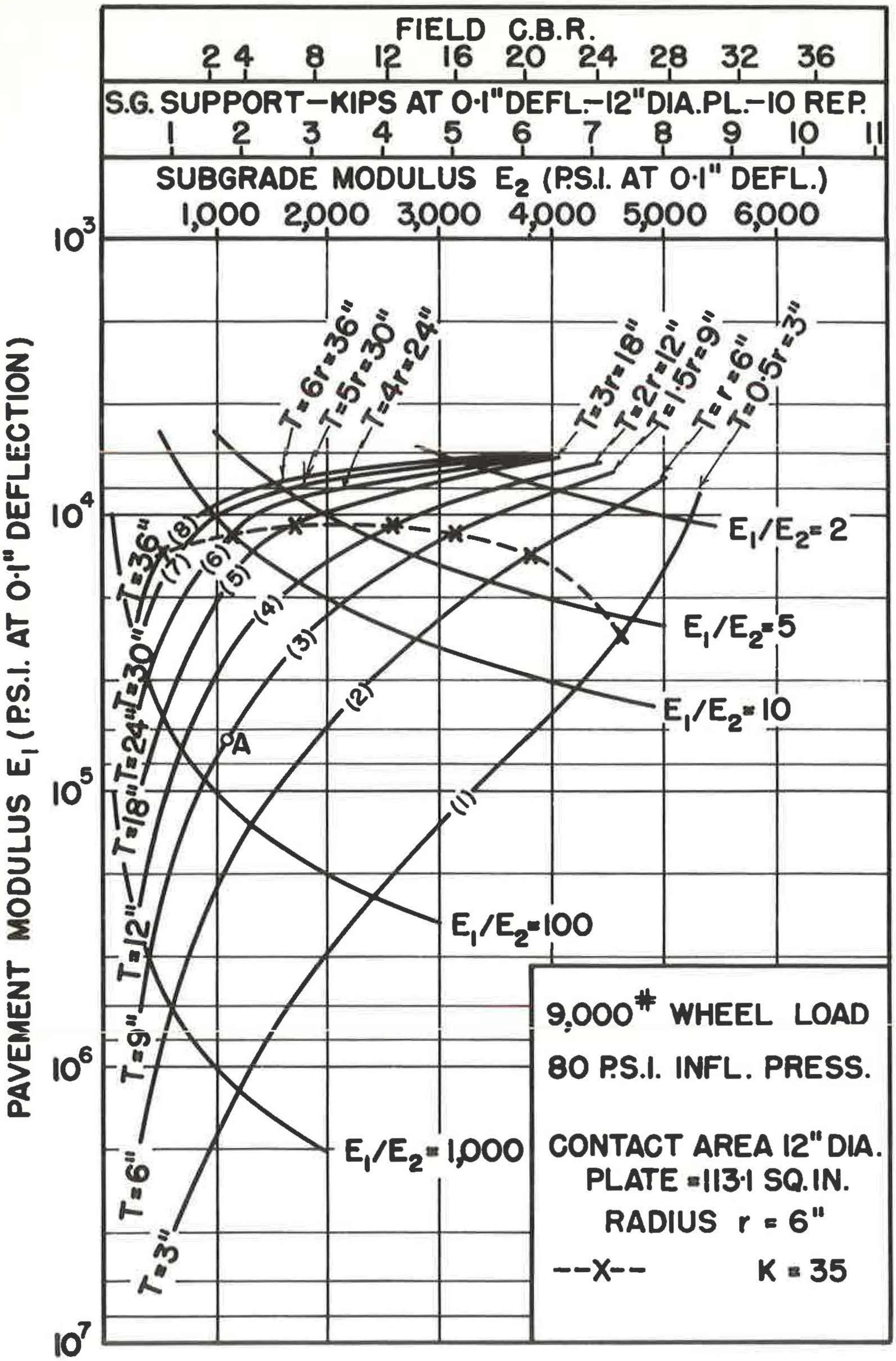


Figure 28. Influence of pavement modulus E_1 and subgrade modulus E_2 on flexible pavement design for a 9,000-lb wheel load (critical deflection 0.1 in.).

TABLE 6

CORRESPONDING VALUES FOR E_1, E_2, AND T FOR PAVEMENT DESIGN[1]

Single Wheel Load, P (lb)	Subgrade Support, S (lb)	Subgrade Elastic Modulus, E_2 (psi)	Deflection Factor, $F_W = S/P$	Values of E_1/E_2 for Pavement Thickness of							
				0.5r	1.0r	1.5r	2.0r	3.0r	4.04	5.0r	6.0r
9,000	500	310	0.055	-	6,600	2,250	950	350	170	103	77
9,000	1,000	630	0.111	6,300	950	280	135	70	37	24	19.5
9,000	2,000	1,250	0.222	900	115	42	21	11.8	8.7	7.2	6.8
9,000	3,000	1,880	0.333	240	37	13.5	8.6	5.5	4.5	4.0	3.85
9,000	4,000	2,500	0.444	92	14	6.6	4.5	3.4	3.02	2.8	2.65
9,000	5,000	3,130	0.555	37	7	3.75	2.85	2.35	2.25	2.15	2.00
9,000	6,000	3,760	0.666	17.5	3.8	2.4	2.0	1.75	1.70	1.65	1.60
9,000	7,000	4,380	0.777	8.2	2.4	1.7	1.52	-	-	-	-
9,000	8,000	5,010	0.888	3.1	1.48	-	-	-	-	-	-
9,000	8,500	5,320	0.944	1.6	-	-	-	-	-	-	-

[1]
Single wheel load = 9,000 lb
Tire inflation pressure = 80 psi
Traffic volume = heavy
Tire contact area = equiv. to 12-in. diam. bearing plate
Radius of contact area = 6 in.
Critical pavement defl. = 0.1 in.

FLEXIBLE PAVEMENT STRUCTURAL DESIGN

Figure 28 shows one way in which either Eq. 1, derived empirically from Canadian Department of Transport load test data, or Eqs. 5 and 6, resulting from the Burmister elastic theory approach based on a layered system (Eq. 8 provides a direct mathematical connection between these two methods), can be employed to design a flexible pavement for heavy traffic consisting of single-wheel loads of 9,000 lb or equivalent. The tire inflation pressure is 80 psi and the tire contact area is assumed to be equal to that of a 12-in. diameter bearing plate. The flexible pavement structure is assumed to be a two-layer elastic system (Fig. 9) consisting of a layer of homogeneous pavement of thickness T and elastic modulus E_1 resting on a homogeneous subgrade of semi-infinite depth and elastic modulus E_2. It is also assumed that the two layers are

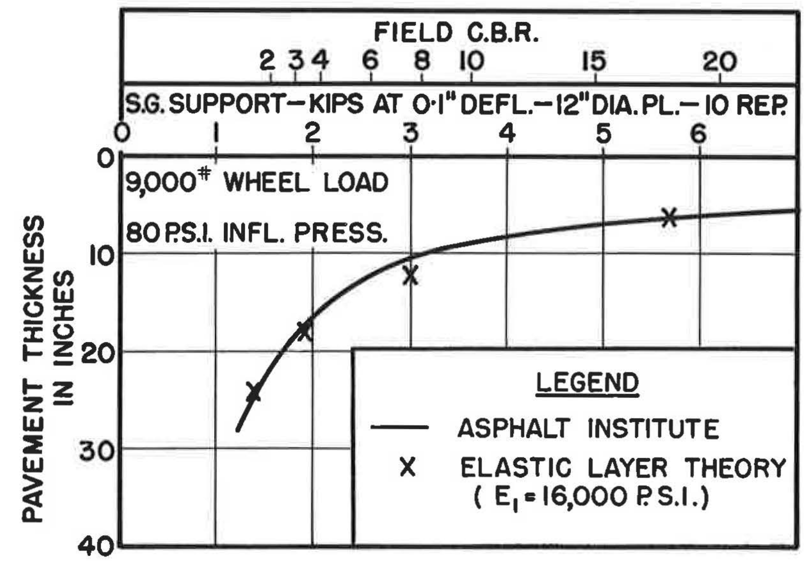


Figure 29. Comparison of Asphalt Institute and elastic layer theory thickness requirements for a 9,000-lb wheel load.

in continuous contact at all points and that the interface between them is perfectly rough. Representative data on which Figure 28 is based are given in Table 6. The relationship between in-place or field CBR values and subgrade support on a 12-in. bearing plate was obtained from Figure 15.

It should be noted that the relationships between in-place CBR, plate bearing, and E_2 values shown at the top of Figure 28, and in all later diagrams, assume that E_2 represents the static modulus of elasticity. For many subgrade soils, if the same elastic modulus scale were employed for dynamic elastic modulus values of E_2, the corresponding scales for in-place CBR and plate bearing values would have to be shifted to the right.

Figure 28 indicates the minimum value of E_1 required for any given combination of pavement thickness and E_2 when designing a pavement for heavy traffic by a single wheel load of 9,000 lb or equivalent, and a tire inflation pressure of 80 psi. For example, when the subgrade elastic modulus E_2 is 1,000 psi for a deflection of 0.1 in. (CBR = 2), and the pavement thickness is 18 in., Figure 28 indicates that the minimum required E_1 is 21,000 psi.

The broken-line curve near the top of Figure 28 represents a value of 35 for the pavement factor K (Eq. 1 and Fig. 5). The points on this curve were obtained by substituting the given values for T, P, and K in Eq. 1 and solving for S.

Over the range of flexible pavement thickness between T = 2r = 12 in. and T = 5r = 30 in. (corresponding subgrade CBR ratings from about 16 to 2) currently employed for flexible pavements for heavy traffic of 9,000-lb wheel loads, it will be noted from Figure 28 that a value of 35 for the pavement factor K corresponds to an almost constant value of the pavement elastic modulus E_1, the actual range in E_1 values being only from 11,000 to 12,500 psi. (This is a narrower range of E_1 values than the reproducibility of any available current method for evaluating E_1.) This illustrates why in the application of Eq. 1 the use of a constant value of K for each size of loaded area (Fig. 5) provides flexible pavement (granular base) thickness requirements that conform to those in actual use (Fig. 6) and that are very close to those indicated by

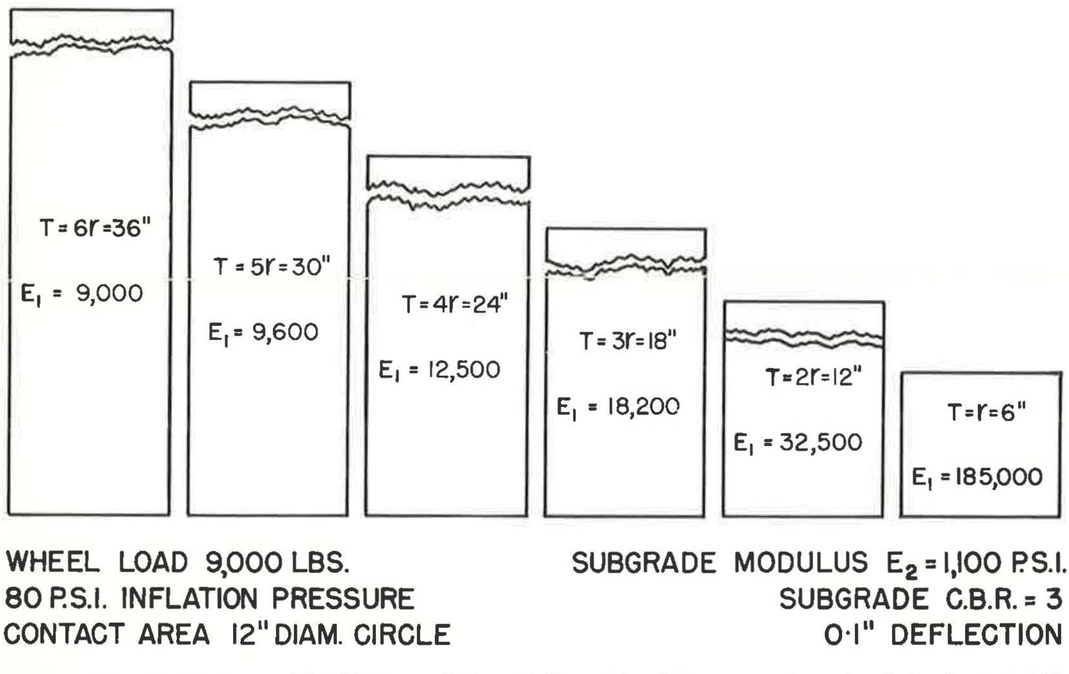


Figure 30. Various combinations of E_1 and T required to support a wheel load of 9,000 lb on a pavement over a CBR 3 subgrade.

other empirical approaches to flexible pavement design. However, Figure 28 also shows that for very large and quite small thickness requirements of a given pavement material, the use of a constant value for K would result in inadequate pavement thicknesses, and the value of K for these conditions would have to be increased.

Figure 29 indicates flexible pavement thickness requirements over subgrades with a wide range of CBR values, as currently recommended by The Asphalt Institute (12) for very heavy traffic consisting of wheel loads of 9,000 lb (single-axle load 18,000 lb). The points represent pavement thicknesses of 6, 12, 18, and 24 in. (thicknesses equal to r, 2r, 3r, and 4r), taken from Figure 28 for corresponding subgrade strength values measured with a 12-in. bearing plate at 0.1-in. deflection for 10 repetitions of load and for a pavement material having E_1 = 16,000 psi. It is apparent from Figure 29 that for a pavement elastic modulus E_1 of 16,000 psi the pavement thickness requirements for a 9,000-lb wheel load given by Figure 28, based on the elastic properties of a two-layer system, are almost identical with those specified by The Asphalt Institute for the same wheel load and subgrade strengths. (The Institute's curve is similar to that obtained by the Corps of Engineers on the basis of the CBR test.) Similar good agreement can be shown with Asphalt Institute thickness requirements for lighter traffic volumes, for which combinations of subgrade strengths measured by a 12-in. plate at deflections greater than 0.1 in. (larger critical pavement deflections) with pavement materials having elastic moduli E_1 less than 16,000 psi can be utilized.

Figure 30, obtained directly from Figure 28, shows six widely different combinations of pavement thickness T and corresponding minimum values of pavement elastic modulus E_1, all of which are capable of carrying heavy traffic by a single wheel load of 9,000 lb or equivalent at 80-psi tire inflation pressure, over a weak subgrade having a CBR rating of 3, and a related elastic modulus E_2 of 1,130 psi (0.1-in. deflection). These combinations range from 36 in. of pavement with E_1 = 9,000 psi, to 6 in. of pavement having E_1 = 185,000 psi.

Having the wide variety of choice for combinations of pavement thickness T and corresponding minimum pavement elastic modulus E_1 for heavy traffic by a 9,000-lb wheel load (Fig. 28 for each of a broad range of subgrade strengths, and Fig. 30 for a CBR 3 subgrade), one can select the best combination of T and corresponding E_1 for the technical and economic conditions associated with each project.

Figures 29 and 30 are both based on Figure 28; for all three figures the traffic criteria are heavy traffic by a 9,000-lb wheel load or equivalent, and a tire inflation pressure of 80 psi. For Figure 30, the subgrade elastic modulus E_2 was held constant at 1,130 psi (CBR of 3), and the influence of change in the pavement elastic modulus E_1 on the required minimum pavement thickness T was investigated. For Figure 29, on the other hand, the pavement elastic modulus E_1 was held constant at 16,000 psi and the crosses illustrate the influence of changing subgrade strength (change in subgrade elastic modulus E_2 or in CBR value) on the required minimum pavement thickness T.

Figure 29 shows a typical result of the currently employed empirical methods for determining flexible pavement thickness requirements. Even though they make no reference to the elastic moduli of pavement or subgrade materials, these empirical methods tend to be based on accumulated experience with average granular materials for which some more or less average value for the pavement elastic modulus E_1 would probably be representative.

Figure 28, which provides precise information on flexible pavement design in terms of corresponding requirements for E_1, E_2, and T for heavy traffic of 9,000-lb wheel loads or equivalent, is based on a critical deflection of 0.1 in. for a load of 9,000 lb on a 12-in. bearing plate. Earlier, and as illustrated by Figure 16, it was pointed out that the permissible critical pavement deflection must be reduced as the traffic volume of a given wheel load or equivalent is increased.

Not enough information is available to establish precisely the relationship between critical pavement deflection measured with a steel bearing plate and volume of traffic. This is undoubtedly influenced by the composition of the pavement and other factors, as well as by traffic volume. However, it would appear that a critical deflection of 0.1 in. for a load of 9,000 lb on a 12-in. diameter bearing plate is not in serious error as a basis of design for heavy traffic of 9,000-lb wheel loads or equivalent when using

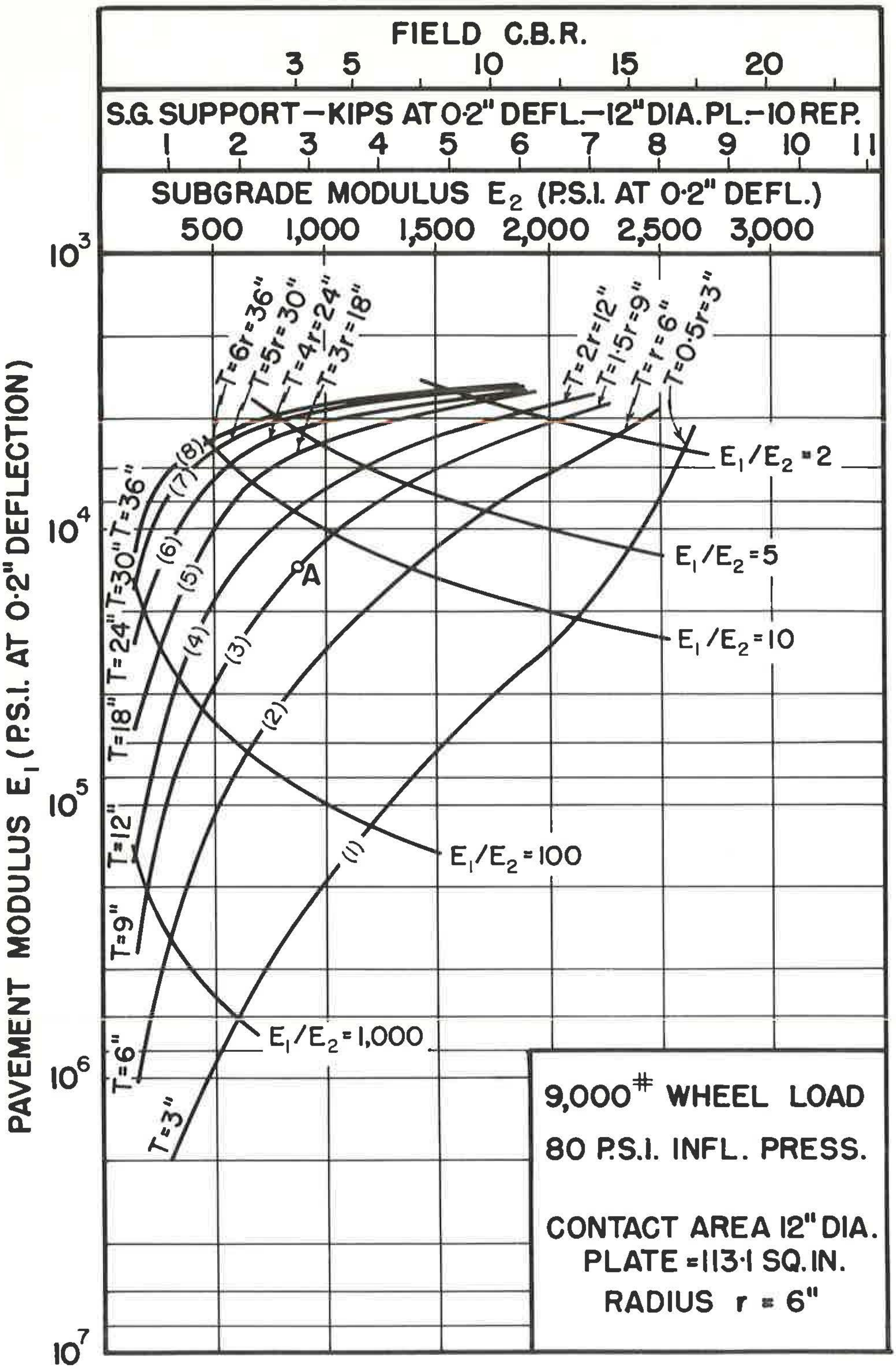


Figure 31. Influence of pavement modulus E_1 and subgrade modulus E_2 on flexible pavement design for a 9,000-lb wheel load (critical deflection 0.2 in.).

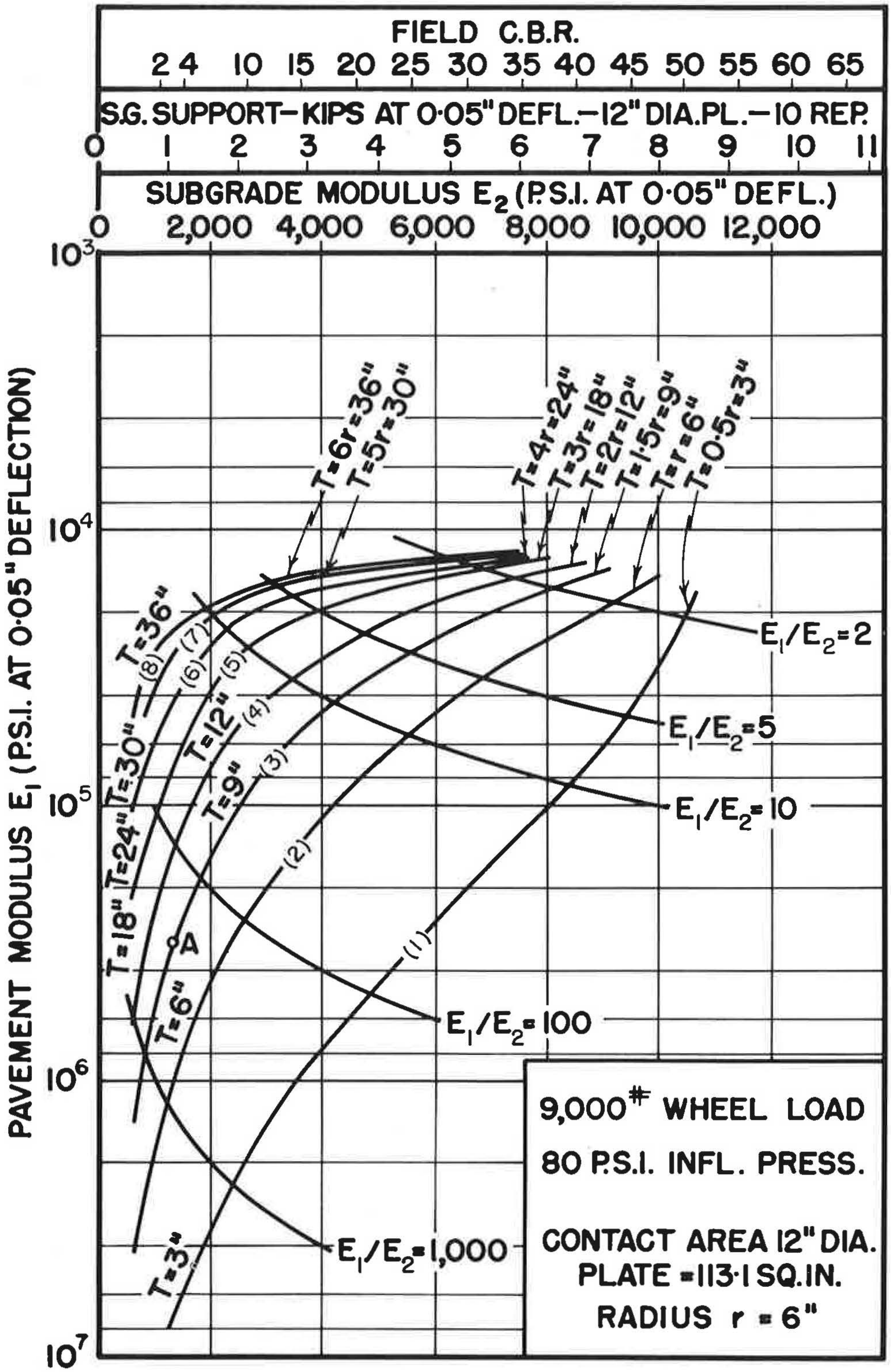


Figure 32. Influence of pavement modulus E_1 and subgrade modulus E_2 on flexible pavement design for a 9,000-lb wheel load (critical deflection 0.05 in.).

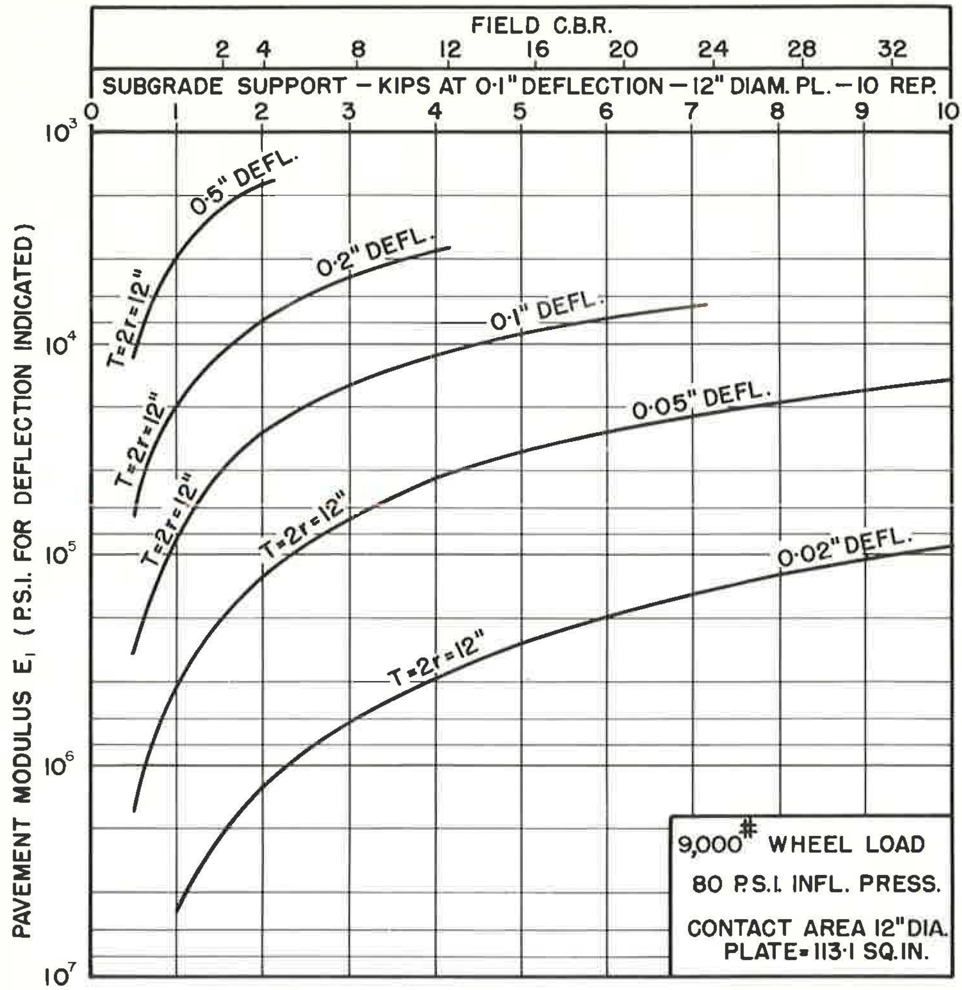


Figure 33. Relationship between critical pavement deflection and pavement modulus E_1 when the pavement thickness required for a 9,000-lb wheel load is kept constant.

current pavement materials. For light to medium traffic, a critical deflection of 0.2 in. would be in the right direction. Similarly, for very heavy traffic of 9,000-lb wheel loads or equivalent, consisting of capacity traffic containing a high percentage of heavy trucks, the critical pavement deflection would be less than 0.1 in., but probably never less than 0.05 in. for properly designed pavement materials. One of the advantages of this method of design is that it is not tied to a single deflection. A wide range of permissible deflections is available, from which the particular deflection that is considered to be critical for any given combination of pavement material and traffic volume to be carried can be selected.

Figure 31 is similar to Figure 28, but is based on a critical deflection of 0.2 in., which is considered to be adequate for light to medium traffic volume of 9,000-lb wheel loads or equivalent. Figure 32, for which the critical pavement deflection is 0.05 in., is also similar to Figure 28. However, Figure 32 would be employed for pavement design only for capacity traffic containing a very high percentage of heavily loaded trucks, and therefore an unusually high traffic count of 9,000-lb wheel loads or equivalent.

Figure 33, based in part on Figures 28, 31, 32, and 37, demonstrates the great

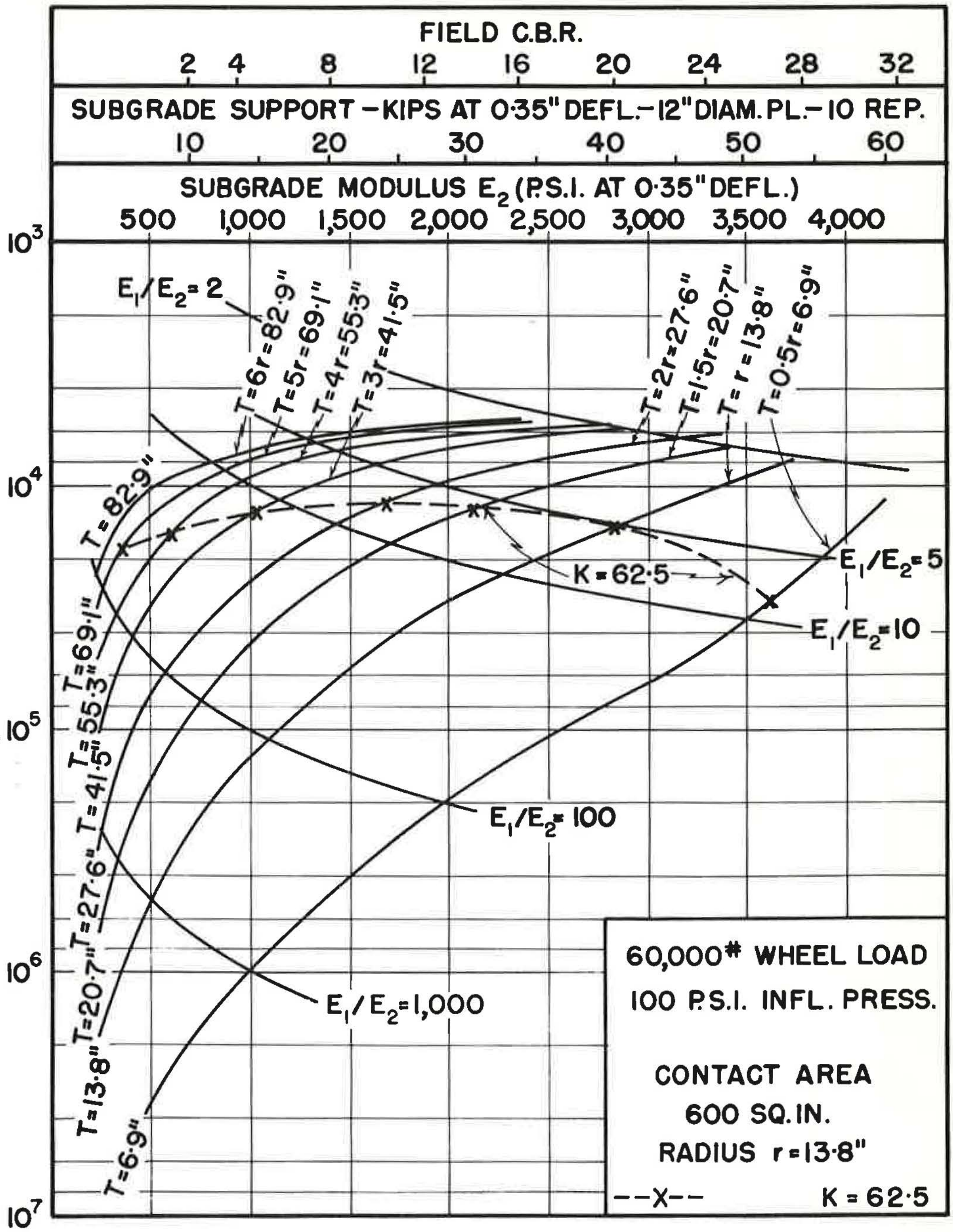


Figure 34. Influence of pavement modulus E_1 and subgrade modulus E_2 on flexible pavement design for a 60,000-lb wheel load (critical deflection 0.35 in.).

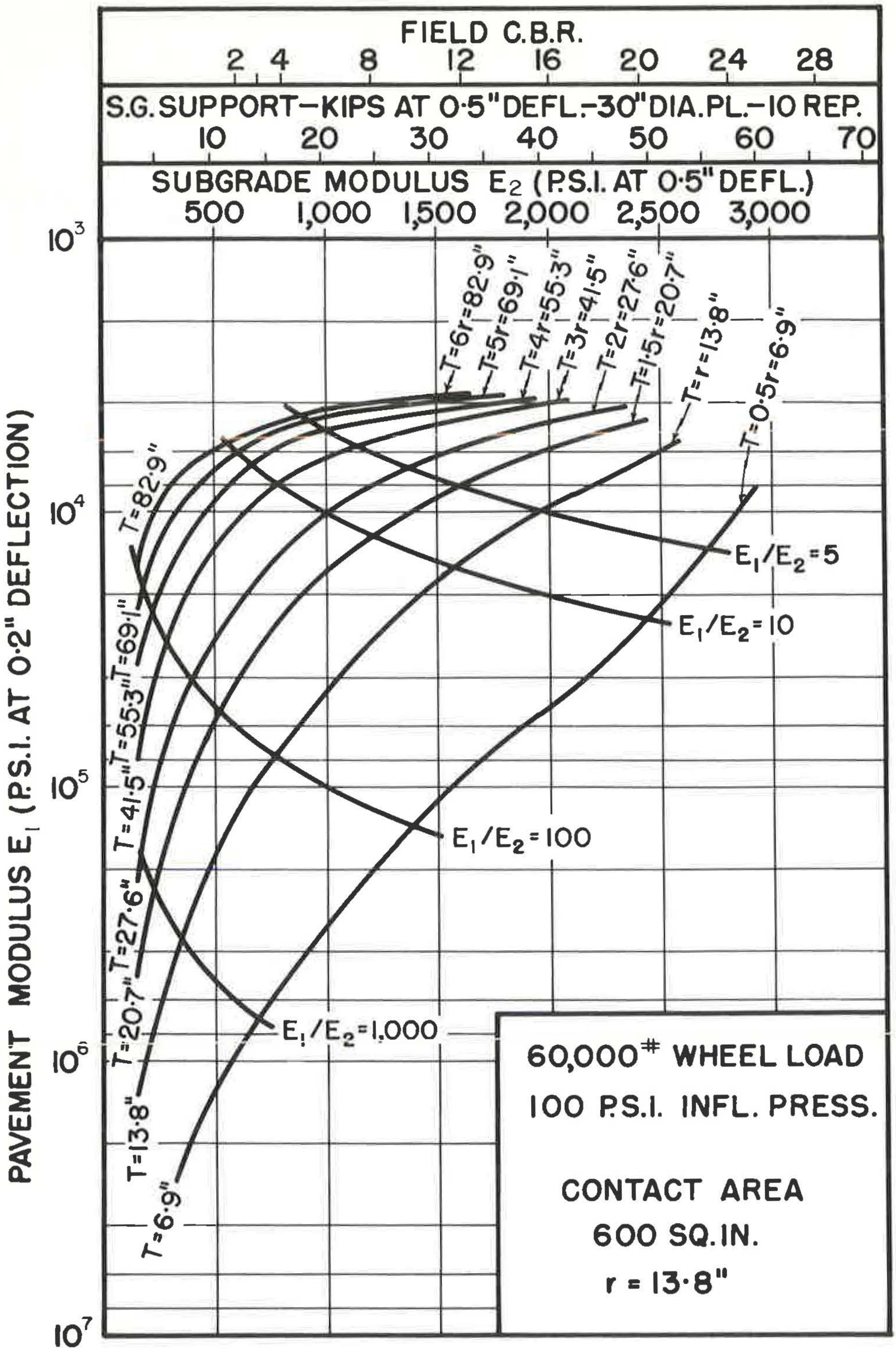


Figure 35. Influence of pavement modulus E_1 and subgrade modulus E_2 on flexible pavement design for a 60,000-lb wheel load (critical deflection 0.5 in., tire inflation pressure 100 psi).

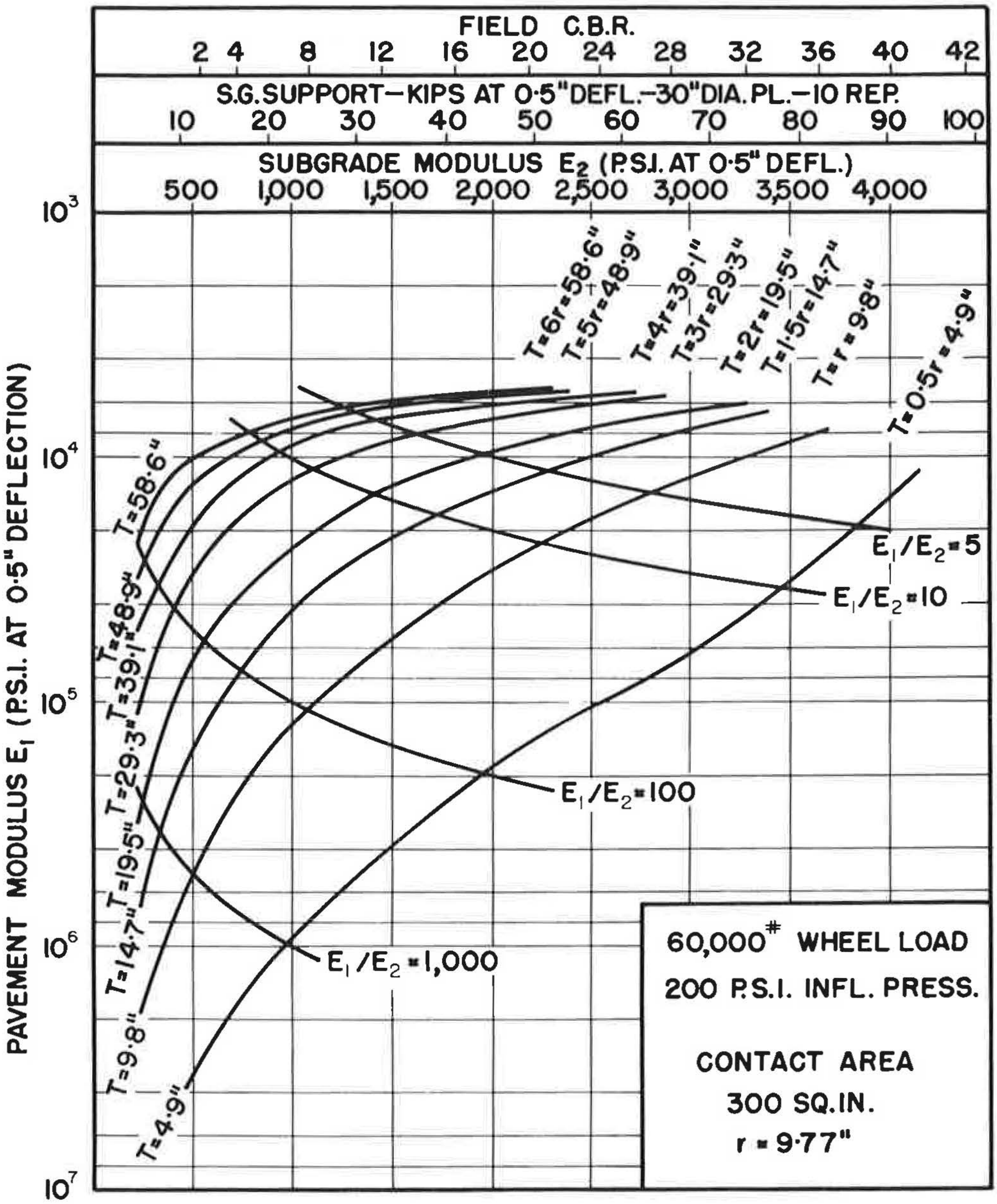


Figure 36. Influence of pavement modulus E_1 and subgrade modulus E_2 on flexible pavement design for a 60,000-lb wheel load (critical deflection 0.5 in., tire inflation pressure 200 psi).

influence that the critical pavement deflection exerts on pavement design requirements for a 9,000-lb wheel load or equivalent. In all cases, the pavement thickness is held constant at T = 2r = 12 in. For a CBR 3 subgrade, for example, it can be seen from Figure 33 that for critical pavement deflections of 0.05, 0.1, 0.2, and 0.5 in. the corresponding minimum requirements for pavement elastic modulus E_1 are 160,000, 32,500, 9,000, and 2,000 psi, respectively. Consequently, when all other factors are equal, pavement design requirements are greatly affected by the critical pavement deflection specified or selected.

Figure 33 also demonstrates the important advantages that would be achieved if it were possible to design pavements of much greater flexibility that could withstand larger deflections even when carrying capacity traffic. A worthwhile research program could be undertaken to determine the requirements for pavement design, the characteristics of binder materials that are either in current use or that should be developed, etc., that would provide this greater pavement flexibility. At the same time, it must always be remembered that properties of pavement materials other than the elastic modulus E_1, such as durability and resistance to deformation and to cracking, are also vitally important to good pavement performance.

Figures 34, 35, and 36 are similar to Figure 28, but illustrate airport pavement design in terms of T, E_1, and E_2 for a two-layer elastic system for aircraft with a single wheel load of 60,000 lb or equivalent. Figures 34 and 35 demonstrate the influence of the critical pavement deflection on pavement design requirements. Figures 35 and 36 provide similar information on the influence of differences in tire inflation pressure.

Figures 34 and 35 are based on critical pavement deflections of 0.35 and 0.5 in., which are assumed to be adequate, respectively, for taxiway and for runway airplane traffic of 60,000-lb wheel load or equivalent. The tire inflation pressure is kept constant at 100 psi. It will be noted that the pavement requirements are greatly influenced by the critical deflection. For example, when the pavement thickness is maintained constant at T = 2r = 27.6 in. over a CBR 3 subgrade, a pavement elastic modulus E_1 of 33,000 psi is required when the critical pavement deflection is 0.35 in. (Fig. 34, taxiways), whereas a minimum E_1 value of 18,000 psi is adequate when the critical deflection is 0.5 in. (Fig. 35, runways).

Figures 35 and 36 show the large effect of increasing the tire inflation pressure from 100 to 200 psi on pavement design requirements for runways when the critical pavement deflection of 0.5 in. and the airplane single wheel load of 60,000 lb are kept constant. For example, if the pavement thickness remains constant at T = 2r = 27.6 in. over a CBR 3 subgrade, a pavement elastic modulus E_1 of 18,000 psi is required for a tire inflation pressure of 100 psi (Fig. 35), but when the tire inflation pressure is doubled to 200 psi (Fig. 36) a minimum pavement elastic modulus E_1 of 25,000 psi is indicated (T = 27.6/9.77 = 2.83r).

In Figure 34, curve K = 62.5 (taken from Fig. 5) indicates the design requirements for a single wheel load of 60,000 lb at 100-psi tire pressure as given by Eq. 1. For the range of thickness between T = r = 13.8 in. and T = 4r = 55.2 in. (corresponding subgrade CBR values from about 20 to 2) normally specified for flexible pavements on airports for a single wheel load of 60,000 lb at 100-psi tire pressure, Figure 34 indicates that a value of K = 62.5 for Eq. 1 corresponds to a relatively constant value of the pavement elastic modulus E_1, the actual range of E_1 values being from 12,000 to 16,000 psi. Consequently, as already pointed out in connection with Figure 28, the use of a constant value for K for each size of loaded area (Fig. 5) when applying Eq. 1 in the past has resulted in flexible pavement thickness requirements that are in accordance with those indicated by field experience and by other commonly employed empirical methods of design. Nevertheless, to achieve more accurate design requirements the values of K provided by Figure 11 are recommended when Eq. 1 is employed for pavement structural design.

For rigid pavements built in North America, typical values for the pavement elastic modulus E_1 are within the range of 3,000,000 to 4,000,000 psi (3, 13). Furthermore, on poor subgrade soils it is not uncommon to specify a rigid pavement thickness of 9 in. for highways carrying heavy traffic.

Figure 37, like Figure 28, pertains to pavement design for high-volume highway traffic by a single wheel load of 9,000 lb or equivalent. In this figure, however, the critical pavement deflection is assumed to be 0.02 in. for 10 repetitions of load. Westergaard's subgrade modulus k, as determined by a 30-in. diameter bearing plate at 0.05-in. deflection, also is shown.

Of particular interest is point A, which is located at the intersection of curve (3), representing a pavement thickness of 9 in. (1.5r), with the in-place subgrade CBR rating 3, and a corresponding Westergaard subgrade modulus k of 90 (Fig. 15). Point A

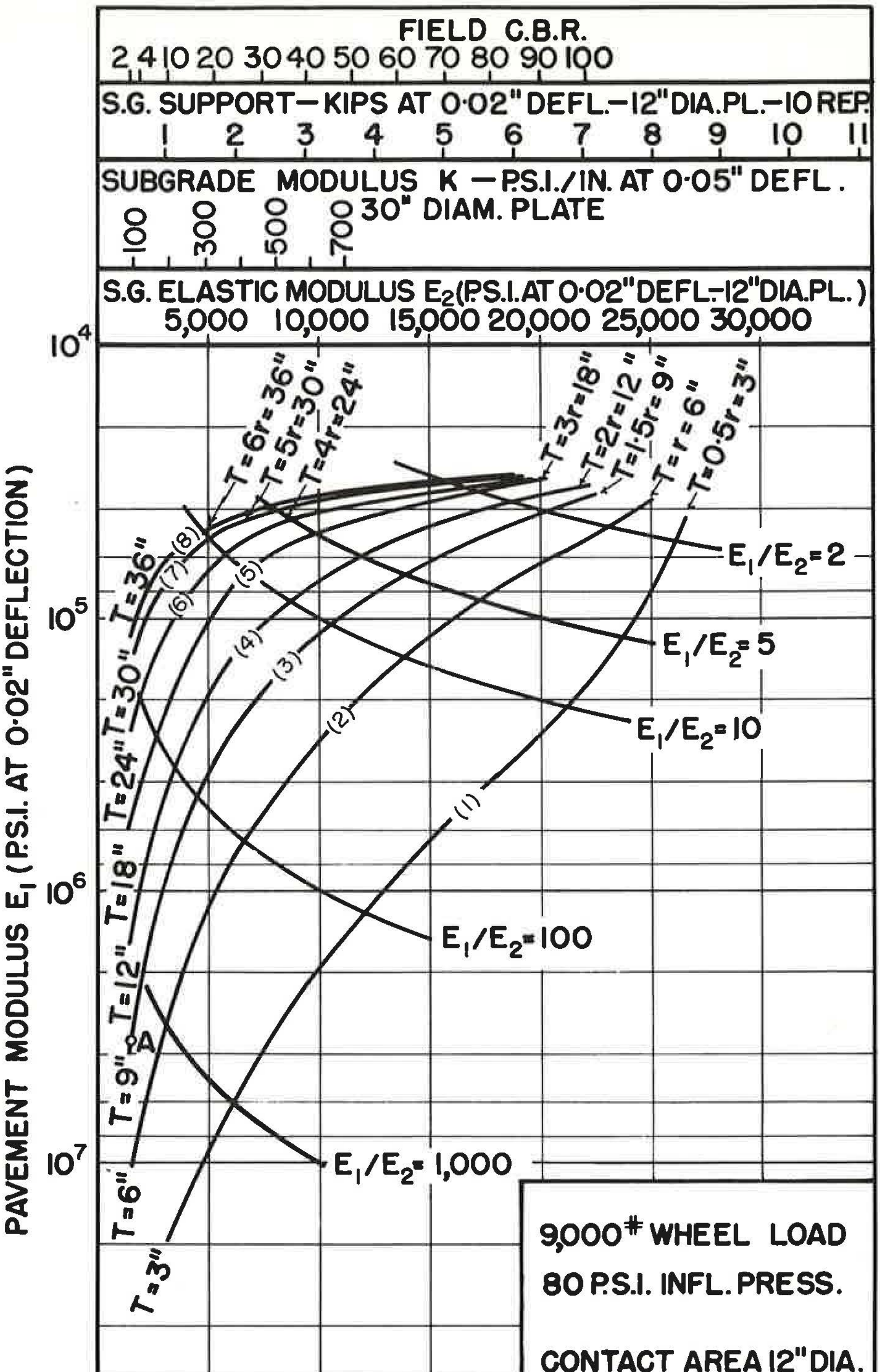


Figure 37. Influence of pavement modulus E_1 and subgrade modulus E_2 on flexible pavement design for a 9,000-lb wheel load (critical deflection 0.02 in.).

demonstrates that for a pavement thickness of 9 in. on this weak subgrade the required minimum pavement elastic modulus E_1 is 3,500,000 psi. A critical deflection of 0.02 in. would appear to be not unreasonable for rigid pavement design for high-volume highway traffic. Consequently, point A of Figure 37 represents common current rigid pavement design requirements for capacity highway traffic of a 9,000-lb wheel load or equivalent, consisting of 9 in. of good quality portland cement concrete over a subgrade for which Westergaard's subgrade modulus k is approximately 100, and the corresponding in-place or field subgrade CBR is 3.

To illustrate the important influence that the critical pavement deflection exerts on pavement design requirements, point A has been marked on Figures 28, 31, and 32, as well as on Figure 37. In each case, point A is for a pavement thickness of 9 in. (1.5r) and a subgrade CBR of 3. Figures 37, 32, 28 and 31, in that order, demonstrate that for the same subgrade strength (in terms of CBR value), and for the same pavement thickness (9 in.), the required minimum value of the pavement elastic modulus E_1 decreases from 3,500,000, to 320,000, to 65,000, and to 14,000 psi, as the permissible critical pavement deflection increases from 0.02, to 0.05, to 0.1, and to 0.2 in., respectively. This means, for example, that when placed on the same properly constructed CBR 3 subgrade, a 9-in. flexible pavement with E_1 = 65,000 psi, which at a critical deflection of 0.1 in. can support an unlimited number of vehicles of 9,000-lb wheel load, is just as capable of carrying capacity highway traffic of this wheel load or equivalent as a 9-in. rigid pavement with E_1 = 3,500,000 psi for which the critical pavement deflection is 0.02 in.

It is apparent from Figures 37, 32, 28, and 31, that for a given wheel load and traffic volume, the thickness of flexible pavement (having the necessary flexibility characteristics) needed for adequate design could be even less than the required thickness of rigid pavement.

This illustrates the need for research in the design of paving mixtures containing asphalt or specially developed flexible binders, and in flexible pavement design in general, to discover the factors that would contribute to greater pavement flexibility for any given moderately high value of the pavement elastic modulus E_1, thereby enabling employment of a higher critical pavement deflection for pavement design for any specified traffic volume from light to capacity or unlimited. However, it would be unwise to become so enthusiastic about designing asphalt paving mixtures with high E_1 values that the factors that contribute to pavement durability and to freedom from cracking and distortion are neglected. This would merely repeat in another form the serious mistake that has so often been made in the past, when high stability has been considered the most important asphalt pavement design requirement, and the other factors that contribute to good pavement service performance have been disregarded.

An explanation for the well-recognized effectiveness of asphalt concrete overlays on old rigid pavements is provided by the influence of pavement critical deflection on pavement design requirements illustrated by Figures 37, 32, 28, and 31. When cracks occur in a rigid pavement, it is understood that the load supporting capacity of the pavement becomes dependent on interlock between the faces of the fractured concrete on opposite sides of each crack. However, if 0.02 in. is the critical deflection for an uncracked rigid pavement for capacity highway traffic (Fig. 37), it is apparent that an appreciably greater pavement deflection, which is dependent on the amount and nature of the cracking and in many cases probably approaches or exceeds 0.1 in., is required to provide enough vertical movement to develop the full pavement strength by interfacial interlock across the cracks.

If the critical pavement deflection for uncracked rigid pavement is 0.02 in. (Fig. 37), it is clear that a portland cement concrete overlay is going to develop only a small portion of the strength of an underlying cracked pavement at 0.02-in. deflection, if a deflection of 0.1 in., for example, is required to mobilize the full bearing capacity of the old cracked pavement. Consequently, the rigid pavement overlay itself must be designed to carry most of the applied traffic load. On the other hand, if the critical deflection of well-designed asphalt concrete is 0.1 in., an overlay of asphalt concrete is very effective because it develops the full load carrying capacity that the underlying cracked rigid pavement can provide at any deflection up to 0.1 in. plus the additional subgrade strength that is mobilized at the higher deflection. For this reason, most of the applied traffic load is still carried by the underlying old rigid pavement and the subgrade on which it rests, and the asphalt concrete overlay itself needs to be designed to provide only a fraction of the load supporting capacity required. This appears to be in agreement with the observed performance of asphalt concrete and portland cement concrete overlays on old rigid pavements.

In connection with the elastic layered system approach to pavement design, it must be recognized that there is some value of the pavement elastic modulus E_1 below which failure of the pavement material in shear would occur, because the wheel load would exceed the pavement's ultimate strength. Consequently, for design charts such as those of Figures 28, 31, 32, 34, 35, 36, and 37, minimum permissible values of the pavement elastic modulus E_1 should be indicated as soon as they have been determined by further investigation.

For dual-wheel and multi-wheel arrangements on trucks and aircraft, determination of the radius r of the equivalent circular loaded area for use with the pavement design charts of Figures 28, 31, 32, 34, 35, and 36, may present some difficulty. On the other hand, the equivalent single wheel load concept worked out by the Corps of Engineers may provide an acceptable solution to this problem.

INCREASE IN LOAD SUPPORT PROVIDED BY PAVEMENT LAYER THICKNESS

For its report entitled "Highway Research in the United States," a committee of the Highway Research Board selected 19 highway problems on which research is most urgently needed, and recommended that $34 million be expended for this research over a period of 4 to 5 years (14). Of this total, it was recommended that $10 million be spent on research on just one of these problems, improvement of knowledge of aggregates and soils, which includes conservation of aggregates and upgrading the quality of poor aggregates.

When using current conventional methods of flexible pavement design, the required thicknesses of pavement materials, and particularly the required thicknesses of granular bases, can vary over a wide range depending on subgrade strength, wheel load to be carried, etc. (Fig. 6). Consequently, it is worthwhile to investigate the average increase in load supporting value provided by the pavement layer per inch of thickness as the pavement thickness, size of loaded area, pavement material, etc., are varied.

From Figure 1 or Figure 9, for a given size of loaded area and a given deflection, it is clear that the increase in load supporting value provided by the pavement layer is P - S, and the average per inch of thickness is (P -S)/T. This leads to either

$$I = \frac{P - S}{T} \tag{9}$$

or

$$i = \frac{p - s}{T} \tag{10}$$

in which

- I = average increase in pavement load supporting value per inch of thickness, in lb per in. of pavement thickness;
- P = load, in lb, supported at surface of pavement on a given loaded area and at a given deflection;
- S = load, in lb, supported by the subgrade for the same loaded area and same deflection as pertain to P;
- T = thickness of pavement layer, in in.;
- i = average increase in pavement load supporting value per inch of thickness, in psi per in. of pavement thickness;
- p = load, in psi, supported at surface of pavement on a given loaded area and at a given deflection; and
- s = load, psi, supported by the subgrade for the same loaded area and same deflection as pertain to p.

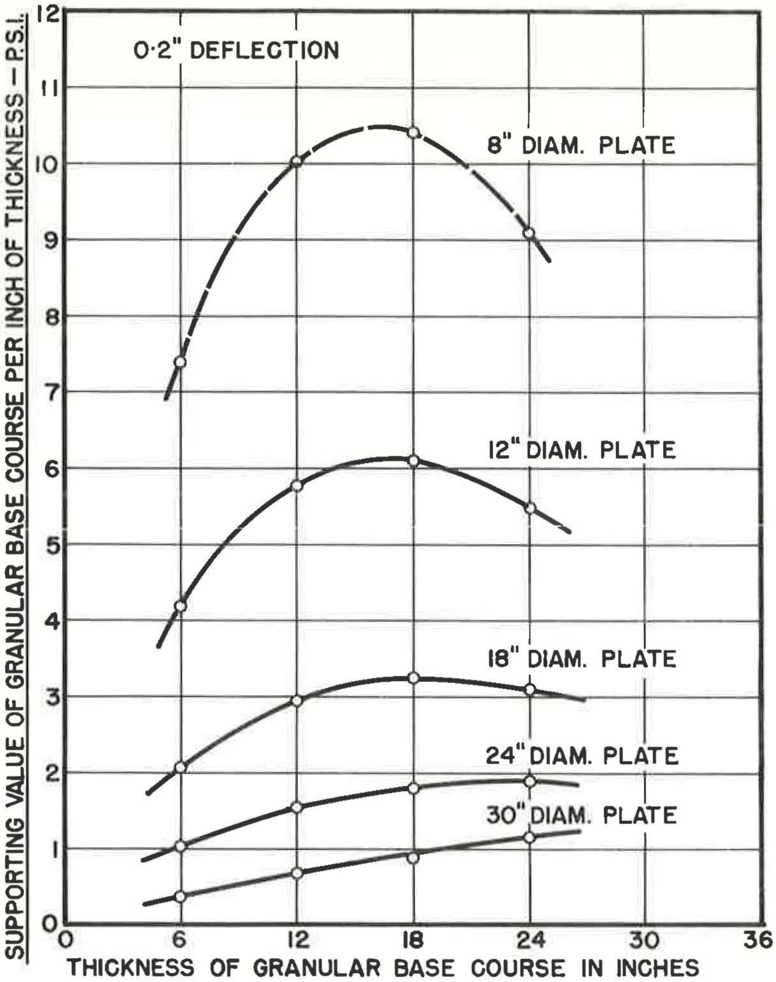


Figure 38. Influence of granular base course thickness and bearing plate diameter on average load supporting value per inch of thickness.

Load test data from the Hybla Valley project (2) are useful for investigating the average increase in load supporting value provided by the pavement layer per inch of thickness, because of the uniformity of the subgrade and pavement materials, the substantial differences in thickness of granular base (6, 12, 18, and 24 in.), and the wide range of bearing plate diameters (12, 18, 24, and 30 in.) employed. For example, Figure 21 shows the results of load tests made at the surface of 12 in. of granular base and on the underlying subgrade with bearing plates 12, 18, 24, and 30 in. in diameter, for a deflection of 0.2 in. For each bearing plate size, by substituting corresponding values for s and p (from Fig. 21), and for T (12 in.), in Eq. 10, a value is obtained for i, the average increase in supporting value (psi), provided by the granular base per inch of thickness. This calculation can also be applied to the Hybla Valley load test data (Figs. 19 and 20) for the other thicknesses of granular base (6, 18, and 24 in.) employed for that project, and for the underlying subgrade. The results are summarized in Figure 38.

Although the granular base at Hybla Valley was very uniform, Figure 38 shows that when appropriate data for T, p, and s, from Figures 19, 20, and 21, are substituted in Eq. 10 the average increase in load supporting value provided by the granular base

TABLE 7

EXAMPLE OF INFLUENCE OF PAVEMENT THICKNESS ON AVERAGE INCREASE IN SUPPORTING VALUE PER INCH THICKNESS OF PAVEMENT[1] AND ON PAVEMENT SUPPORTING VALUE[2,3]

Pavement Thickness, T (mult. of radius)	Pavement Thickness, T (in.)	Pavement Factor, K	$10^{\frac{T}{K}}$	$10^{\frac{T}{K}}-1$	$\frac{S}{T}$, $\frac{1,810}{T}$	I[1] (lb/in.)	P[2] (lb)
0.5r	3.0	4.00r	1.316	0.316	603	191	2,380
1.0r	6.0	3.27r	2.033	1.033	302	312	3,680
1.5r	9.0	3.50r	2.681	1.681	201	338	4,850
2.0r	12.0	3.914	3.247	2.247	151	335	5,880
2.5r	15.0	4.40r	3.704	2.704	121	326	6,730
3.0r	18.0	4.88r	4.115	3.115	101	313	7,480
4.0r	24.0	5.87r	4.808	3.808	75	287	8,700
5.0r	30.0	6.824	5.464	4.464	60	269	9,890
6.0r	36.0	7.85r	5.780	4.780	50	240	10,460

[1]Eq. 12
[2]Eq. 11
[3]$E_1/E_2 = 10$
Radius of bearing plate = 6 in.
Critical pavement deflection = 0.1 in.
Subgrade supporting value, S = 1,810 lb (CBR = 3).

course (pavement) per inch of thickness, i, varies considerably, depending on the size of the loaded area and the thickness of the granular material. For any thickness of granular base from 6 to 24 in., the value of i is much greater for a 12-in. than for a 30-in. diameter bearing plate. Also, when measured with any given size of bearing plate the value of i is seen to vary with the thickness of granular base. Furthermore, at some critical base course thickness roughly equal to the diameter of the loaded area, the value of i reaches a maximum, and it falls off for base thicknesses that are either less or greater than this critical depth. Consequently, Figure 38 indicates that, for Hybla Valley conditions at least, for each size of loaded area there was a critical or optimum thickness of granular base at which the use of this particular base course material would be most effective in terms of i, the average increase in load supporting value provided by the base course per inch of thickness.

It is worth pursuing this matter further, and to endeavor to learn if an optimum thickness at which the average increase in load supporting value per inch thickness of pavement (i or I) is a maximum, is a basic characteristic of pavement materials in general. It will be shown that it is for the materials ordinarily employed in flexible pavement structures.

Eq. 1 can be rearranged as

$$P = 10^{(T/K)} S \tag{11}$$

Substitution of Eq. 11 in Eq. 9 gives

$$I = \frac{10^{T/K} S - S}{T} = \frac{S(10^{T/K} - 1)}{T} \tag{12}$$

and substitution of Eq. 8 in Eq. 12 gives

$$I = \frac{S}{T}\left(10^{-\log F_W} - 1\right) \tag{13}$$

Similarly,

$$i = \frac{s}{T}\left(10^{-\log F_W} - 1\right) \tag{14}$$

Eqs. 13 and 14 are derived primarily from Eq. 1, but equivalent forms can also be derived from the Boussinesq and Burmister equations (Eqs. 5 and 6).

It was shown previously that Eqs. 5 and 6 lead to Eq. 7, from which

$$p = s/F_W \tag{15}$$

and

$$P = S/F_W \tag{15a}$$

Substituting Eq. 15 in Eq. 10 gives

$$i = \frac{1}{T}\left(\frac{s}{F_W} - s\right) = \frac{s}{T}\left(\frac{1}{F_W} - 1\right) \tag{16}$$

Similarly,

$$I = \frac{S}{T}\left(\frac{1}{F_W} - 1\right) \tag{17}$$

Table 7 gives data calculated by means of Eq. 12 that are required to provide the relationship between I and T when the ratio of the elastic moduli for pavement and subgrade $E_1/E_2 = 10$, the loaded area is a 12-in. diameter bearing plate, and the deflection is 0.1 in. for 10 repetitions of load. Table 8 demonstrates that for identical conditions the same relationship between I and T is provided by calculations based on Eq. 17. Similar tables have been prepared for additional values of E_1/E_2, with all other factors remaining constant.

Figure 39 shows relationships between I and T for E_1/E_2 values of 1.5, 2.0, and 5.0. Figure 40 provides similar information for E_1/E_2 values of 10 (Tables 7 and 8), 50, and 100, and Figure 41 for E_1/E_2 values of 500 and 1,000. For Figures 39, 40, and 41, the loaded area is a 12-in. diameter bearing plate (9,000-lb wheel load at 80-psi tire inflation pressure), the pavement deflection is 0.1 in. for 10 repetitions of load, and the subgrade elastic modulus E_2 is 1,130 psi, which is equivalent to a field or in-place subgrade CBR rating of 3.

The ratios of E_1/E_2 from 1.5 to 1,000 selected for Figures 39, 40, and 41 are representative of the range of pavement materials ordinarily employed for flexible pavement construction. As can be seen from Figure 41, there is some irregularity about the location of the points on the curves for E_1/E_2 ratios of 500 and 1,000, because of

TABLE 8

EXAMPLE OF INFLUENCE OF PAVEMENT THICKNESS ON AVERAGE INCREASE IN SUPPORTING VALUE PER INCH THICKNESS OF PAVEMENT[1] AND ON PAVEMENT SUPPORTING VALUE P[2,3]

Pavement Thickness, T (mult. of radius)	Pavement Thickness, T (in.)	Deflection Factor, F_W	$\frac{1}{F_W}$	$\frac{1}{F_W}-1$	$\frac{S}{T}$, $\frac{1,810}{T}$	$I = \frac{S}{T}(\frac{1}{F_W}-1)$ (lb/in.)	$P = \frac{S}{F_W}$ (lb)
0.5r	3.0	0.76	1.316	0.316	603	191	2,380
1.0r	6.0	0.492	2.033	1.033	302	312	3,680
1.5r	9.0	0.373	2.681	1.681	201	338	4,850
2.0r	12.0	0.308	3.247	2.247	151	335	5,880
2.5r	15.0	0.269	3.704	2.704	121	326	6,730
3.0r	18.0	0.242	4.115	3.115	101	313	7,480
4.0r	24.0	0.208	4.808	3.808	75	287	8,700
5.0r	30.0	0.183	5.464	4.464	60	269	9,890
6.0r	36.0	0.173	5.780	4.780	50	240	10,460

[1]Eq. 17.
[2]Eq. 15(a).
[3]$E_1/E_2 = 10$
Radius of bearing plate = 6 in.
Critical pavement deflection = 0.1 in.
Subgrade supporting value, S = 1,810 lb (CBR = 3).

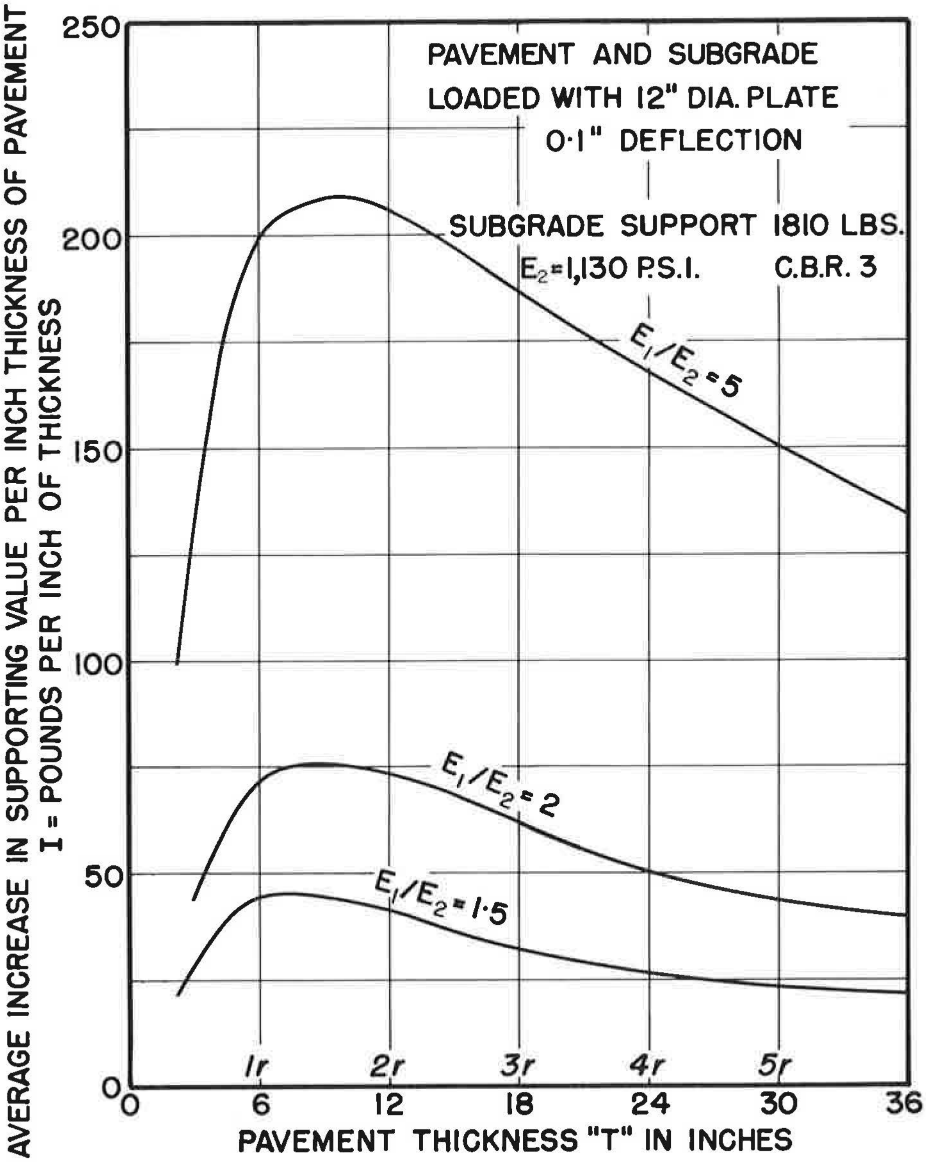


Figure 39. Influence of pavement thickness on the average increase in load supporting value per inch thickness of pavement for E_1/E_2 = 1.5, 2.0, and 5.0 (12-in. diam. plate at 0.1-in. deflection).

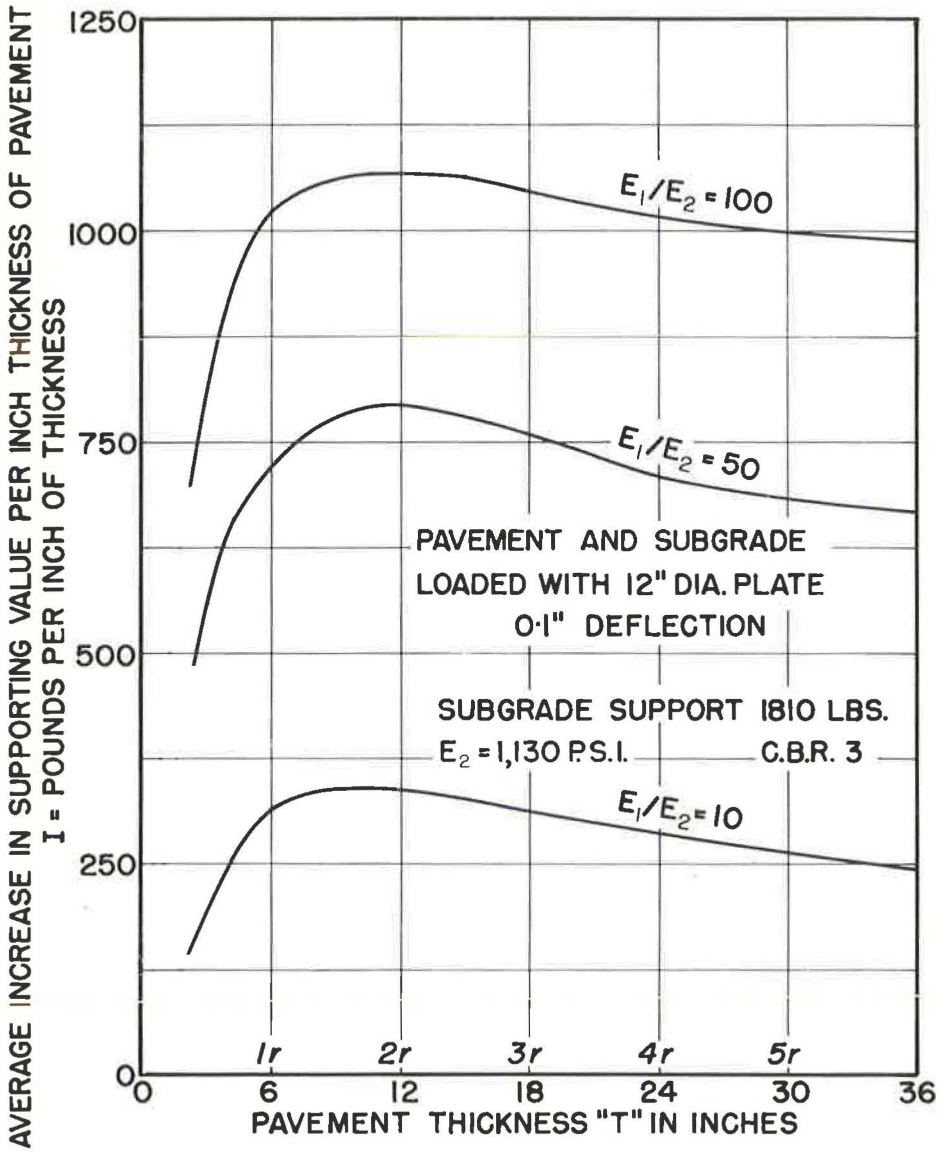


Figure 40. Influence of pavement thickness on the average increase in load supporting value per inch thickness of pavement for E_1/E_2 = 10, 50, and 100 (12-in. diam. plate at 0.1-in. deflection).

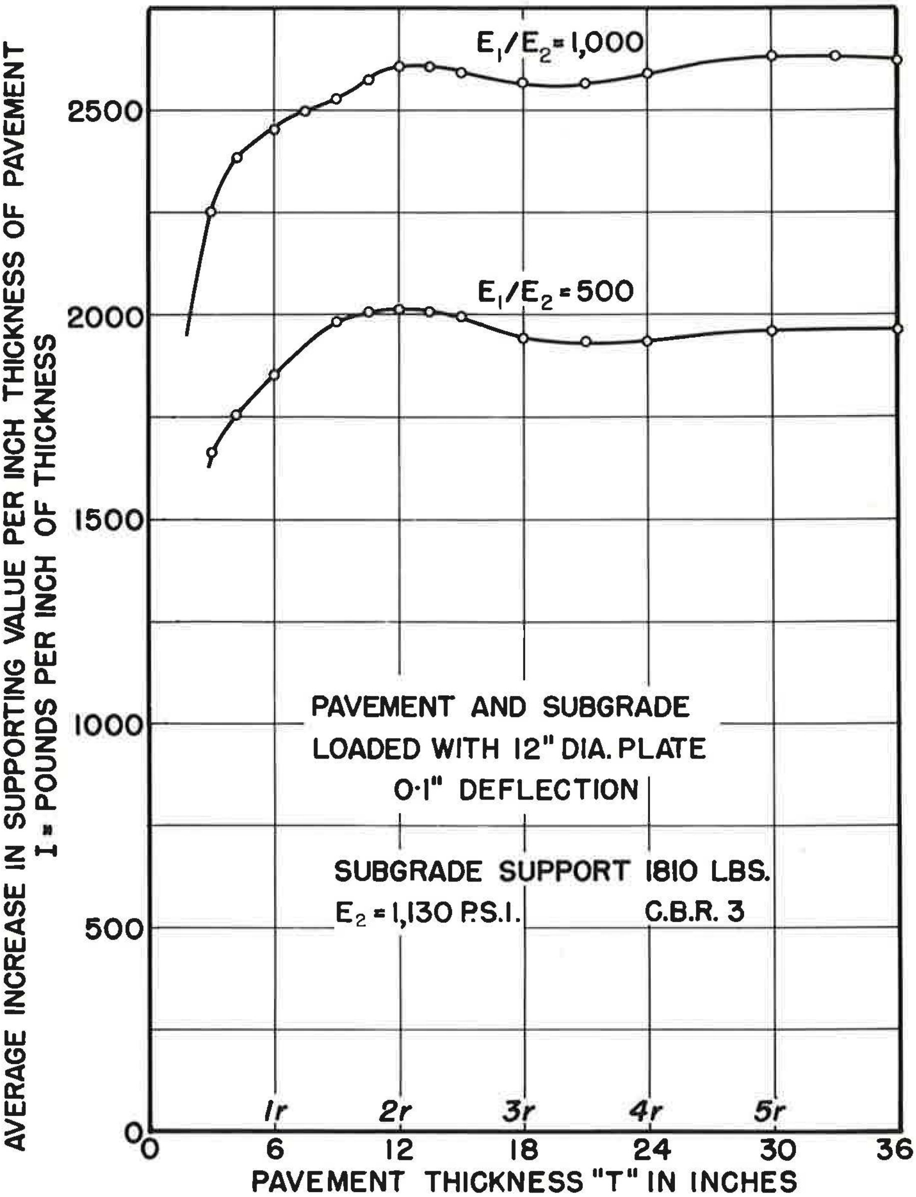


Figure 41. Influence of pavement thickness on the average increase in load supporting value per inch thickness of pavement for E_1/E_2 = 500 and 1,000 (12-in. diam. plate at 0.1-in. deflection).

the uncertainty of interpolation in the upper left corner of Figure 10. To a lesser extent this also applies to Figures 39 and 40 because of the limitations of accuracy of Figure 10. This demonstrates the need for recalculating the data required for Figure 10 to a high degree of accuracy by means of an electronic computer, redrafting Figure 10 with a maximum of precision, and enlarging the upper left corner of the chart to make more accurate interpolation possible.

From the curve for each value of the ratio E_1/E_2 illustrated in Figures 39 and 40 (and possibly 41), it is apparent that there is an optimum or critical pavement thickness T (peak of the curve), at which I is a maximum. Consequently, Figures 39 and 40 demonstrate that Eqs. 5 and 6, based on the elastic properties of a two-layer pavement system, are capable of duplicating, on a purely theoretical basis, the empirical findings shown in Figure 38. Both the theoretical equations and the analysis of actual load tests have shown that there is an optimum pavement thickness at which the pavement material provides a maximum average increase in load supporting value per inch thickness of pavement. For the balance of this paper, this optimum pavement thickness (peak of the curve) is designated T_o and the maximum average increase in load supporting value per inch thickness of pavement (peak of the curve) is designated I_m or i_m.

Values for T_o taken from the peaks of the E_1/E_2 curves of Figures 39, 40, and 41, have been plotted versus E_1/E_2 in Figure 42. The optimum pavement thickness is seen to increase gradually from about 1.25r for $E_1/E_2 = 1.5$ to slightly more than 2r for $E_1/E_2 = 1,000$. The irregularity of some of the plotted points in Figure 42 illustrates again the uncertainty of interpolation in Figure 10, which was previously referred to.

Figure 43, based on either Table 7 or Table 8, illustrates the change in I as the pavement thickness is varied from 6 to 36 in. (r to 6r), when the pavement and subgrade materials remain unchanged throughout. The subgrade support is kept constant at 1,810 lb on a 12-in. bearing plate at 0.1-in. deflection (corresponding $E_2 = 1,130$ psi, and CBR = 3), and $E_1/E_2 = 10$. Consequently, the elastic modulus E_1 of the pavement material is 11,300 psi. Figure 43 shows that under these conditions, I_m for this pavement material is 340 lb per inch of thickness and T_o is about 10.5 in. or approximately 1.75r.

Figure 43 also shows the load P (12-in. diameter bearing plate at 0.1-in. deflection) that can be supported at the surface of each thickness of pavement. The values of P are taken from Tables 7 and 8. This curve shows that for $T_o = 10.5$ in., P = 5,350 lb. Also, T = 25 in. is required for P = 9,000 lb, and T = 36 in. is needed for P = 10,500 lb.

Figure 43 shows that I decreases substantially for pavement thicknesses either less than or more than the optimum thickness, $T_o = 10.5$ in.; for example, from 340 lb per inch at T = 10.5 in. to only 250 lb per inch at T = 34 in. Table 9 shows that the load supporting effectiveness of successive increments of pavement thickness above the optimum thickness T_o is even less than Figure 43 seems to indicate. Also that the average increase in load supporting value per inch of pavement thickness for each of several successive 5-in. increments of pavement thickness above the optimum thickness decreases rapidly with increasing thickness. For example, the average increase in load supporting value for the 5-in. increment between 25 and 30 in. of pavement is only 160 lb per inch of thickness, which is less than one-half of I_m (340 lb per inch of pavement thickness). For the next 5-in. increment, between 30 and 35 in., the average increase in load supporting value is only 120 lb per inch of thickness, which is about one-third of I_m. Pavement thicknesses between 25 and 35 in., consisting largely of granular subbase and base course material, are well within the range of flexible pavement thicknesses currently required for high-volume highway truck traffic over weak subgrades. However, Figure 43 and Table 9 indicate that these great thicknesses represent very inefficient use of pavement materials.

Figure 44 shows the change in I for a wide range of thicknesses of a given pavement material on subgrades varying from weak to strong. The thickness of pavement on each subgrade is just capable of supporting 9,000 lb on a 12-in. diameter plate at 0.1-in. deflection. The pavement material has an elastic modulus $E_1 = 10,000$ psi; the subgrade elastic moduli E_2 vary from about 1,000 to 5,000 psi, with corresponding

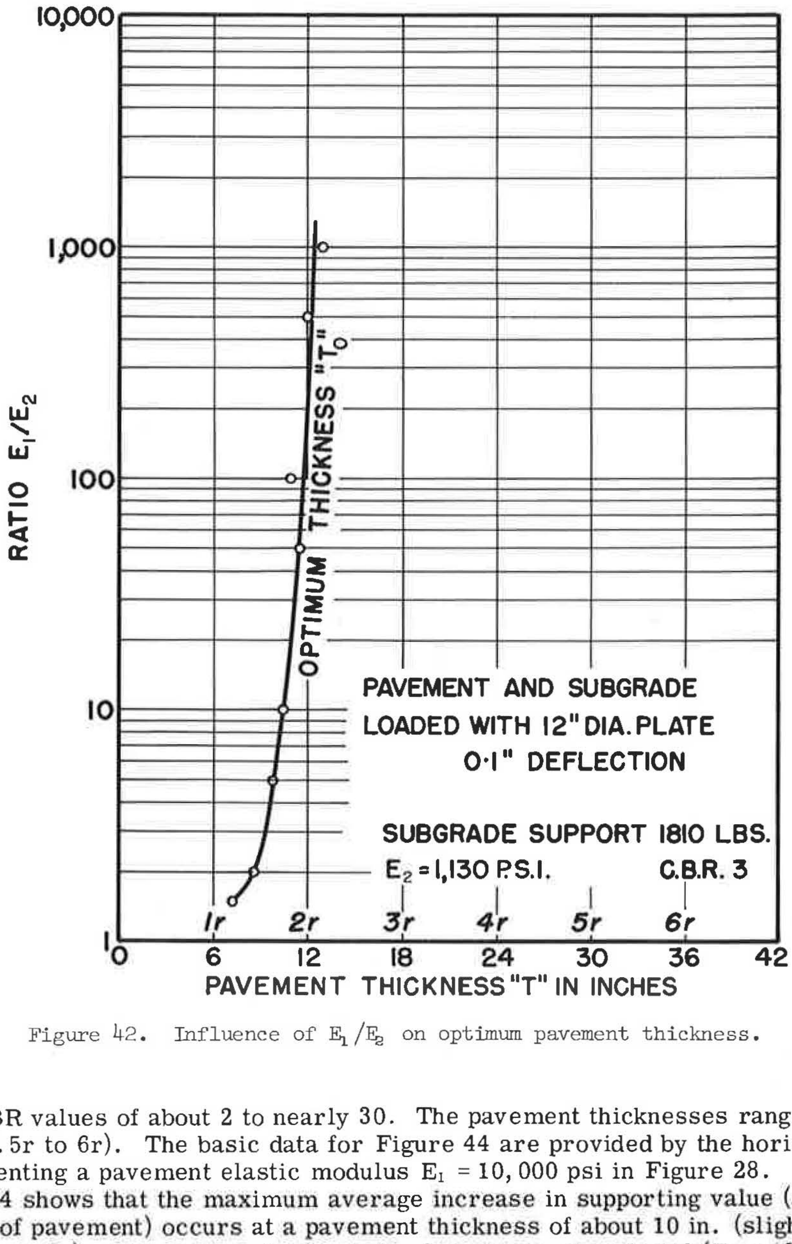


Figure 42. Influence of E_1/E_2 on optimum pavement thickness.

in-place CBR values of about 2 to nearly 30. The pavement thicknesses range from 3 to 36 in. (0.5r to 6r). The basic data for Figure 44 are provided by the horizontal line representing a pavement elastic modulus E_1 = 10,000 psi in Figure 28.

Figure 44 shows that the maximum average increase in supporting value (about 380 lb per inch of pavement) occurs at a pavement thickness of about 10 in. (slightly greater than 1.5r). Ten inches of this particular pavement material (E_1 = 10,000 psi) is just adequate to support 9,000 lb on a 12-in. bearing plate at 0.1-in. deflection when placed on a subgrade with E_2 = 3,250 psi (corresponding in-place CBR rating of about 16). Figure 44 demonstrates that for the conditions on which it was based, thicknesses of pavement substantially greater or less than 10 in. because they have been placed on weaker or stronger subgrades, respectively, provide much smaller values for I. For example, for a pavement thickness of 30 in. on a CBR 3 subgrade, the value for I is only about 240 lb per inch thickness of pavement. This is about 140 lb per inch of pavement thickness less than the value for I_m (380 lb per inch of pavement thickness) for the optimum thickness, T_o, of about 10 in. Consequently, Figure 44, like Figure 43, indicates that great thickness requirements represent an inefficient use of pavement materials.

Figure 44 results from a purely theoretical investigation of the pavement design

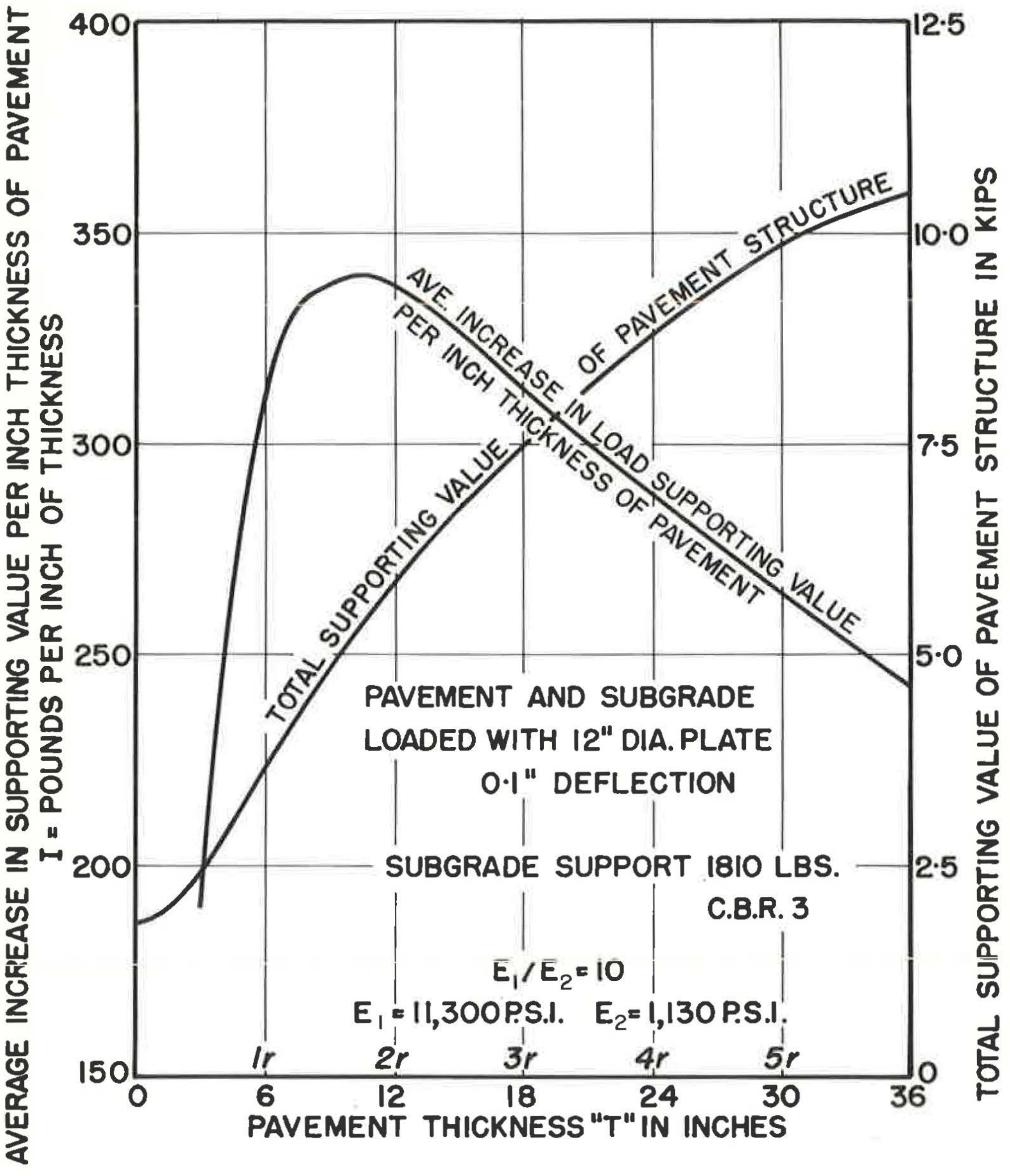


Figure 43. Example of effect of pavement thickness on the average increase in supporting value per inch thickness of pavement (constant subgrade strength, CBR 3).

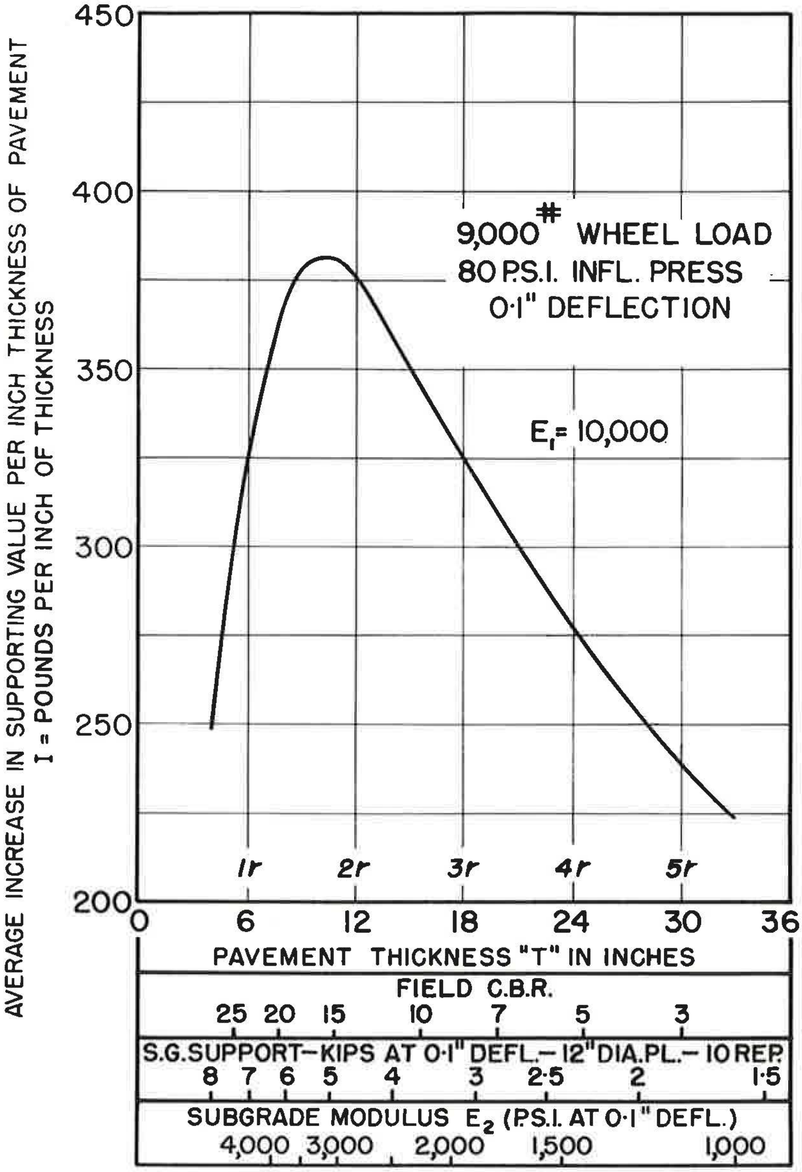


Figure 44. Example of effect of pavement thickness on the average increase in supporting value per inch thickness of pavement (variable subgrade strength, CBR 2 to 30 and constant total load).

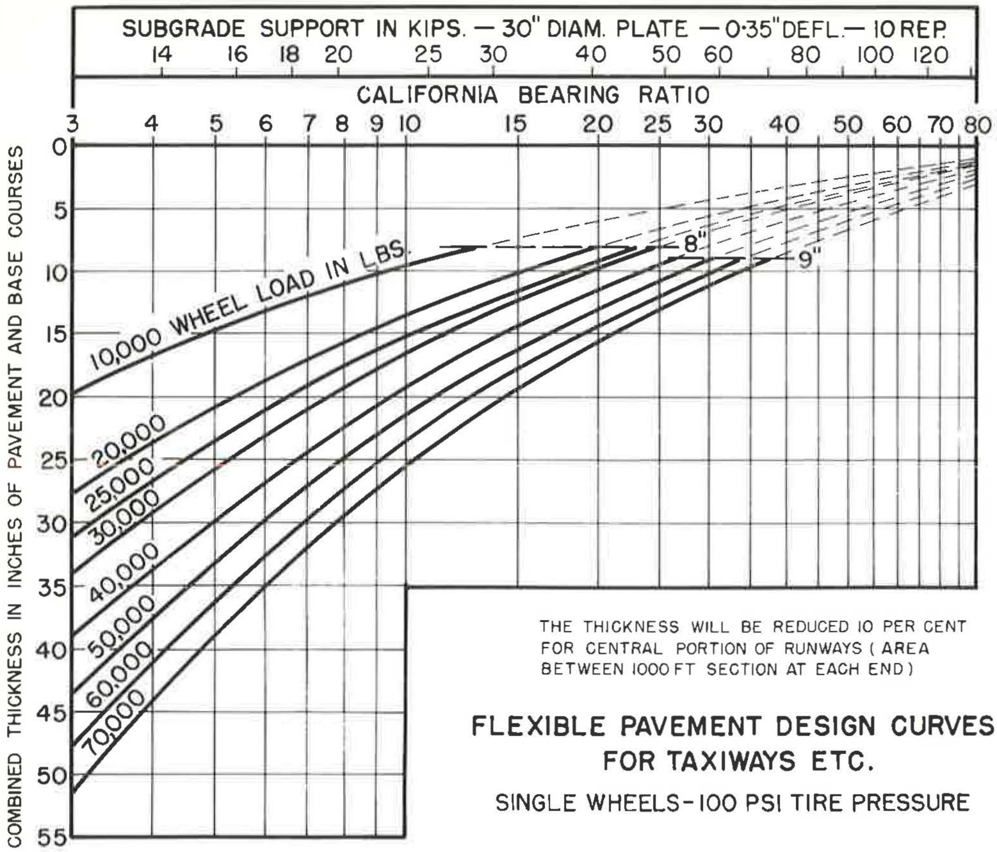


Figure 45. Corps of Engineers flexible pavement design for airport pavements.

problem on the basis of the elastic properties of a two-layer system. Nevertheless, its implications can be verified by a simple analysis of currently used empirically derived thickness design charts like Figure 45, which shows the flexible pavement design requirements of the Corps of Engineers for airport pavements (15). This verification is provided by Tables 10 and 11, and Figure 46. To Figure 45, which is based exclusively on CBR subgrade ratings, the corresponding subgrade supporting values in terms of load on a 30-in. diameter bearing plate at 0.35-in. deflection have been added. Figure 15 was employed for this correlation.

Table 10 is based on the curve for the single wheel load of 70,000 lb in Figure 45. Any of the other curves would have served equally well. The resemblance of Figure 46, based on empirically obtained data, to Figure 44, which resulted from a purely theoretical approach to pavement design, is striking. Figure 46 shows that the maximum average increase in load supporting value per inch thickness of pavement occurs at an optimum pavement thickness of about 19 in.

A single wheel load of 70,000 lb at a tire inflation pressure of 100 psi is usually assumed to have a tire contact area of 700 sq in., which is nearly equal to the area of a 30-in. diameter bearing plate (707 sq in.). Consequently, the optimum pavement thickness T_0 = 19 in. given by Figure 46 corresponds to about 1.3r.

Figure 46 and Table 10 show that the maximum average increase in load supporting value per inch thickness of pavement (1,960 lb) which occurs at optimum pavement thickness T_0 = 19 inches over a CBR 16 subgrade, is 76 percent higher than the average increase (1,120 lb per inch thickness) provided by a pavement thickness of 52 in. over a CBR 3 subgrade. Although the 52 in. of pavement thickness required for a 70,000-lb wheel load over a CBR 3 subgrade (Fig. 45) is ordinarily constructed of

TABLE 9

EXAMPLE OF INCREMENTAL AVERAGE INCREASES IN SUPPORTING VALUE PER INCH THICKNESS OF PAVEMENT VERSUS INCREMENTAL INCREASES IN PAVEMENT THICKNESSES[1]

Increments of Pavement Thickness (in.)	Load Supporting Value	
	Increment (lb)	Average Increase (lb/in.)
0 - 5	3,220 - 1,810 = 1,410	282
5 - 10	5,210 - 3,220 = 1,990	398
(0 - 10.5)	(5,380 - 1,810 = 3,570)	(340)[2]
10 - 15	6,720 - 5,210 = 1,510	302
15 - 20	7,920 - 6,720 = 1,200	240
20 - 25	9,000 - 7,920 = 1,080	216
25 - 30	9,800 - 9,000 = 800	160
30 - 35	10,400 - 9,800 = 600	120

[1] E_1/E_2 = 10
Radius of bearing plate = 6 in.
Critical pavement deflection = 0.1 in.
Subgrade supporting value, S = 1,810 lb (CBR = 3)
[2] 10.5 in.

TABLE 10

AVERAGE SUPPORTING VALUE PER INCH THICKNESS OF PAVEMENT, AND PAVEMENT ELASTIC MODULUS, DERIVED FROM CORPS OF ENGINEERS DESIGN CURVE FOR FLEXIBLE PAVEMENT[1]

Subgrade CBR	Pavement Supporting Value, P (lb)	Subgrade Supporting Value, S (lb)	P - S (lb)	Pavement Thickness, T (in.)	$I = \frac{P-S}{T}$ (lb/in.)	$F_w = \frac{S}{P}$	$\frac{T}{r}$	E_2 (psi)	E_1/E_2	E_1 (psi)
3	70,000	11,800	58,200	52	1,120	0.169	3.47	843	18.0	15,200
5	70,000	15,200	54,800	39	1,410	0.217	2.6	1,090	15.5	16,900
7	70,000	18,500	51,500	32	1,610	0.264	2.13	1,320	13.5	17,800
9	70,000	21,900	48,100	28	1,720	0.313	1.87	1,560	11.0	17,200
11	70,000	25,200	44,800	24	1,870	0.36	1.60	1,800	9.9	17,800
13	70,000	28,500	41,500	21.5	1,930	0.407	1.43	2,040	9.0	18,360
15	70,000	31,900	38,100	19.5	1,950	0.456	1.30	2,280	8.0	18,240
17	70,000	35,200	34,800	18.0	1,930	0.503	1.20	2,520	7.0	17,640
19	70,000	38,600	31,400	16.5	1,900	0.551	1.10	2,760	6.0	16,560
21	70,000	41,900	28,100	15.0	1,870	0.599	1.00	3,000	5.1	15,300

[1] Airplane single wheel load = 70,000 lb
Tire inflation pressure = 100 psi
Contact area = 700 sq in.
Radius of equiv circular contact area = 15 in.
Critical pavement deflection = 0.35 in.

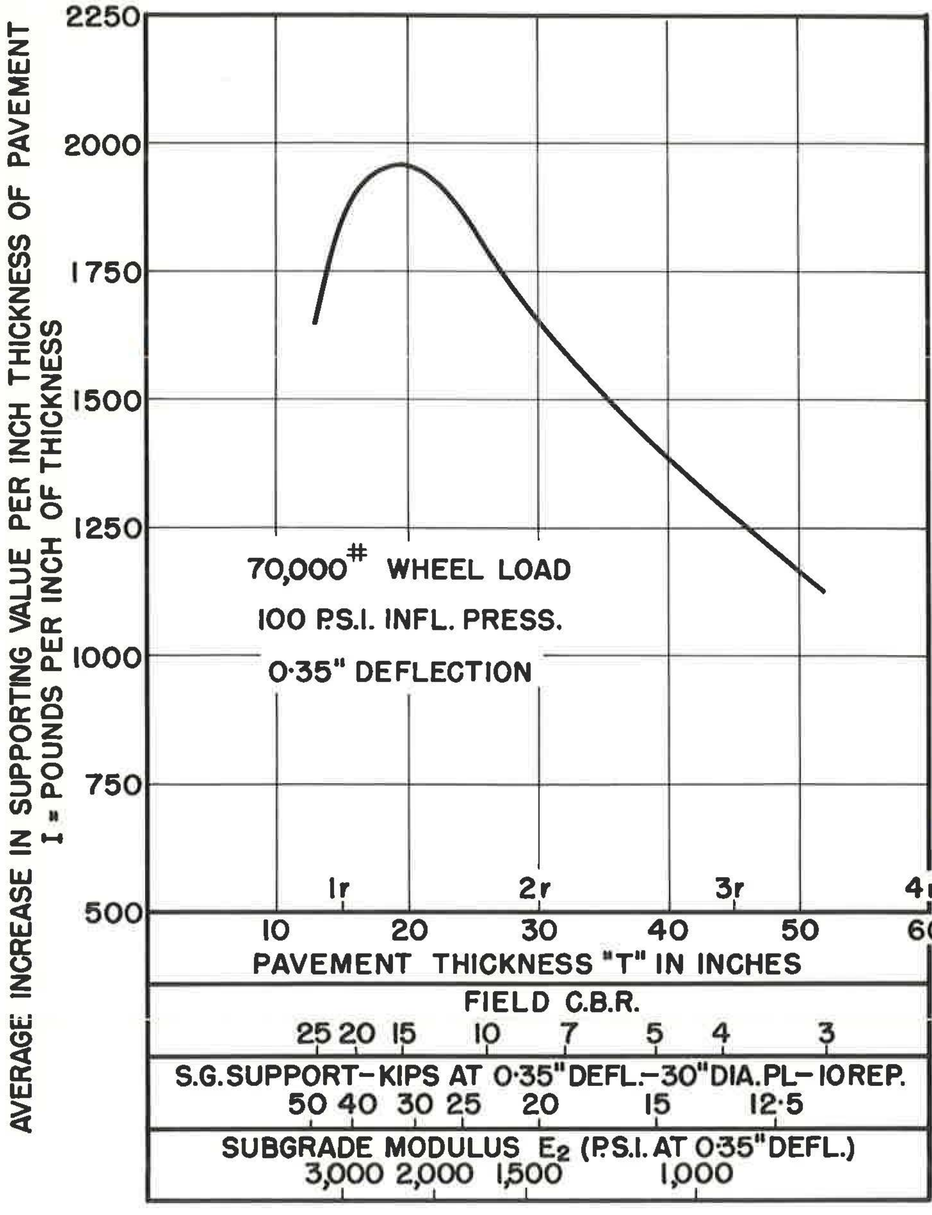


Figure 46. Relationship between I and T for the CBR design curve (Corps of Engineers) for an airplane single wheel load of 70,000 lb.

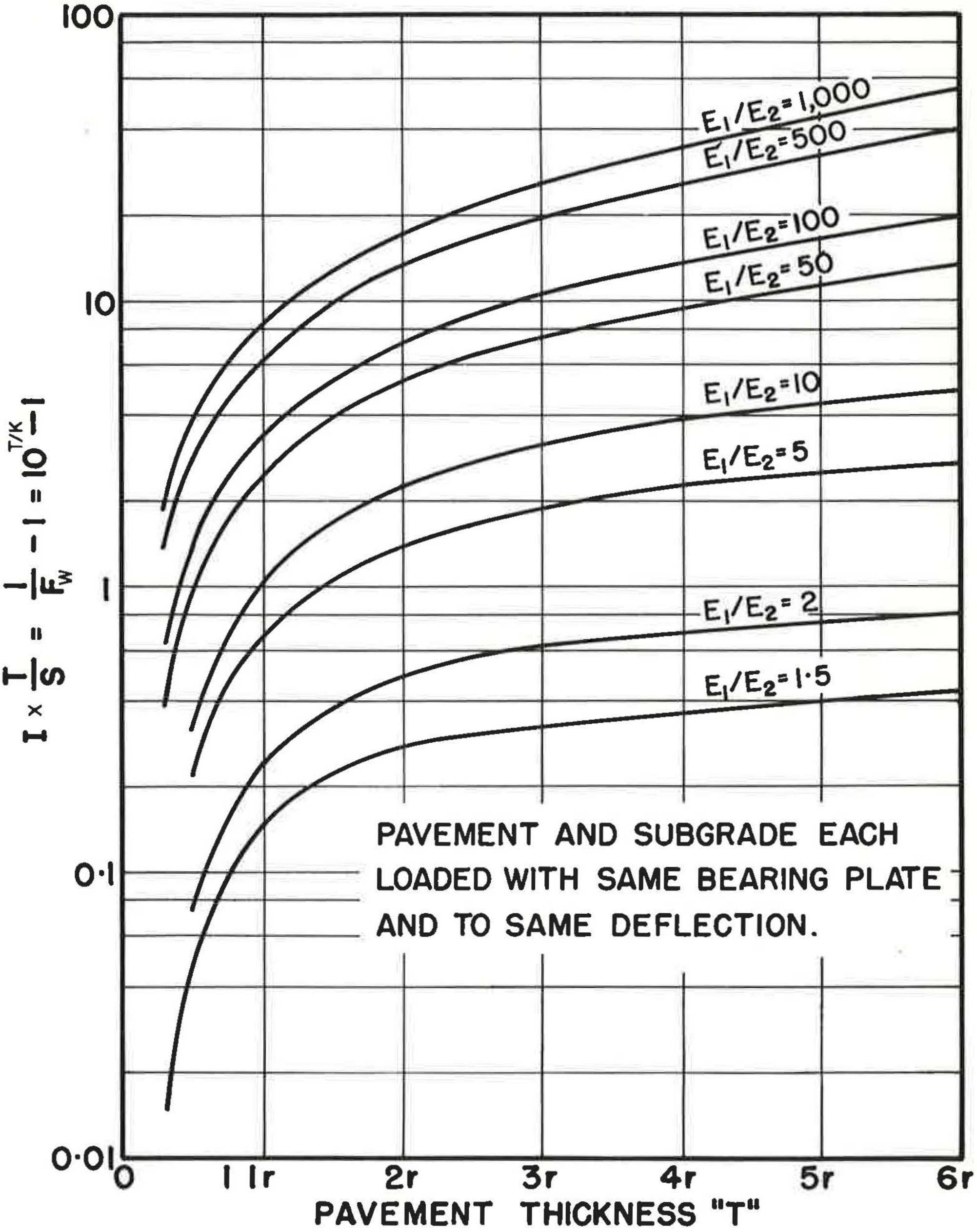


Figure 47. General chart for determining average increase in supporting value per inch thickness of pavement.

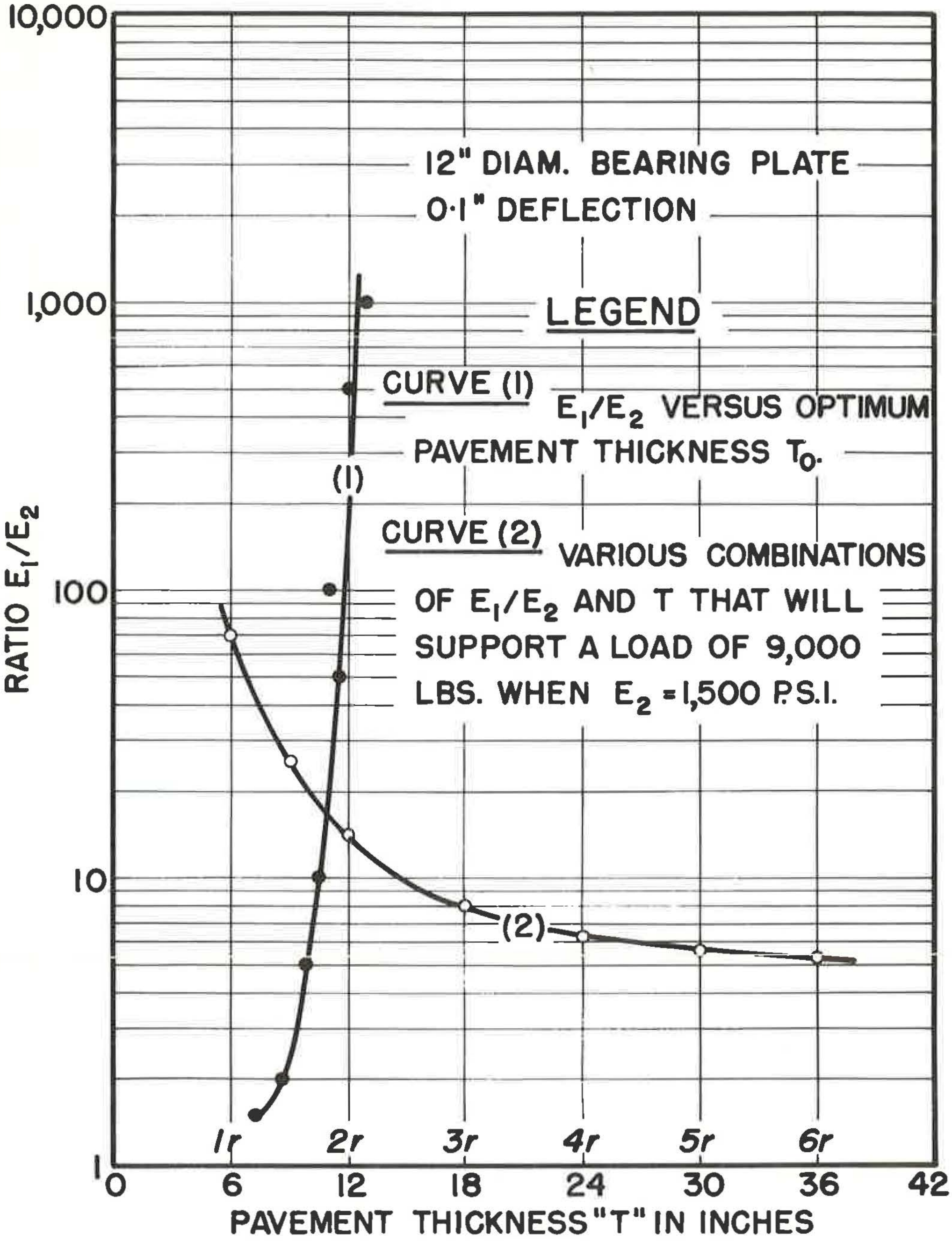


Figure 48. Example of a useful plot of data for pavement structural design.

TABLE 11

AVERAGE SUPPORTING VALUE PER INCH THICKNESS OF PAVEMENT, AND PAVEMENT ELASTIC MODULUS, DERIVED FROM CORPS OF ENGINEERS' DESIGN CURVE FOR FLEXIBLE PAVEMENT[1]

Subgrade CBR	Pavement Supporting Value, P (lb)	Subgrade Supporting Value, S (lb)	P - S (lb)	Pavement Thickness, T (in.)	$I = \frac{P-S}{T}$ (lb/in.)	$F_w = \frac{S}{P}$	$\frac{T}{r}$	E_2 (psi)	E_1/E_2	E_1 (psi)
3	20,000	4,760	15,340	27.5	5,580	0.238	3.46	641	9.0	5,760
5	20,000	6,100	13,900	21.0	6,690	0.305	2.64	823	7.5	6,160
7	20,000	7,440	12,560	17.0	7,400	0.372	2.14	1,000	6.1	6,100
9	20,000	8,780	11,220	14.5	7,750	0.439	1.82	1,180	5.2	6,140
11	20,000	10,120	9,880	12.5	7,900	0.506	1.57	1,366	4.5	6,150
13	20,000	11,460	8,540	11.0	7,760	0.573	1.38	1,547	3.9	6,030
15	20,000	12,800	7,200	10.0	7,200	0.640	1.25	1,730	3.3	5,700
17	20,000	14,140	5,860	9.0	6,510	0.707	1.13	1,910	2.9	5,540
19	20,000	15,580	4,420	8.5	5,200	0.779	1.07	2,100	2.2	4,620

[1]Airplane single wheel load = 20,000 lb
Tire inflation pressure = 100 psi
Contact area = 200 sq in.
Radius of equiv. circular contact area = 7.98 in.
Critical pavement deflection = 0.35 in.

layers of successively stronger materials, the Corps of Engineers specifies that no reduction in pavement thickness will be permitted even if high quality granular material must be used throughout the full depth (16). After analyzing the results of traffic tests conducted by the Corps of Engineers, Turnbull (17) has indicated that this requirement for high quality materials for full pavement depth appears to be justified, and that the pavement thickness may have to be increased if lower quality materials are substituted for the lower part of the pavement. This is essentially the equivalent of preferring uniform pavement material of adequately high elastic modulus throughout the full depth, and of requiring that the pavement thickness be increased by way of compensation, if material of smaller elastic modulus is selected for the lower layers of the pavement.

To demonstrate that Table 10 and Figure 46, which are based on the 70,000-lb wheel load curve of Figure 45, are by no means unique, Table 11 presents information provided by the 20,000-lb wheel load curve of Figure 45. Table 11 also shows that there is an optimum thickness (T_o = 12.5 in., or slightly more than 1.5r) at which I is a maximum (7,900 lb) on a CBR 11 subgrade. It also shows that I_m = 7,900 lb is 41 percent higher than the average increase in load supporting value per inch thickness of pavement (I = 5,580 lb) that is associated with a required pavement thickness of 27.5 in. over a CBR 3 subgrade.

It appears reasonable to assume that the design curves for the different wheel loads (Fig. 45) are based on the use of similar pavement (granular) materials. Employing this assumption, calculated values of E_1 for the different pavement thicknesses for the 70,000-lb wheel load are given in Table 10, and for the 20,000-lb wheel load in Table 11. It is apparent that in both cases the values of E_1 are fairly constant. However, the E_1 values for the 70,000-lb wheel load average roughly 2-½ times those for the 20,000-lb wheel load. This seems to indicate that the value of E_1 for any given pavement material is influenced consistently and very substantially by the size of the loaded area. The effect of the size of the loaded area on the value of E_1 for any given pavement material appears to be very much greater on the basis of Tables 10 and 11 than was previously indicated by Tables 2, 4, and 5 based on a similar analysis of Hybla Valley load test data.

Figures 39, 40, and 41 include a wide range of values for E_1/E_2, but are limited to a 12-in. diameter bearing plate at 0.1-in. deflection and a CBR 3 subgrade. Because both I and T are expressed in terms of the radius of the loaded area, Figure 47 is universally applicable to any particular combination of E_2, bearing plate diameter, E_1/E_2, and T. Consequently, Figure 47 enables particular relationships between I and T, such as those of Figures 39, 40, and 41, to be determined for specified combinations of subgrade elastic modulus pavement thickness, E_1/E_2, and size of loaded area.

Tables 7 to 11, and Figures 39, 40, 43, 44, and 46, demonstrate that the current standard approach to flexible pavement thickness design, which merely calls for greater thicknesses of pavement as wheel loads, traffic volumes, etc., increase, is in serious conflict with the need for the conservation and more effective use of both low quality and high quality aggregates emphasized by the Highway Research Board report already referred to (14). Table 9, for example, shows that pavement thicknesses of 4r, 5r, and 6r (24, 30, and 36 in., respectively) for a single wheel load of 9,000 lbs at 80 psi tire inflation pressure are far down on the curve of diminishing returns insofar as concerns the increase in load supporting value per inch of thickness that flexible pavement materials can provide. For the conditions of Table 9, the incremental average increase in load supporting value per inch thickness of pavement over the thickness range of 30 to 35 in. (5r to 6r) is about one-third that of the same pavement material when used at its optimum thickness of 10.5 in.

Figure 30 shows that many combinations of T and E_1 are capable of carrying a specified wheel load and traffic volume over a given subgrade. Furthermore, that within this range of suitable combinations as E_1 is increased the pavement thickness can be reduced. Figure 30 makes it evident, therefore, that one way in which both low quality and high quality aggregates could be conserved and used more effectively would be to upgrade their load carrying capacities by increasing their E_1 values. This would enable the design wheel load to be adequately supported by pavement thicknesses approaching the optimum thickness, or even somewhat less.

Methods employing additives to increase the load carrying capacity or to otherwise improve the quality of inferior aggregates and soils are ordinarily referred to as soil stabilization. The soil stabilization processes most widely used at present involve the incorporation of either bituminous materials or portland cement.

The effectiveness of these binders for upgrading the quality of inferior aggregates has been dramatized by the results of the special base investigations at the AASHO Road Test (18). The sandy gravel subbase material used on this project was mixed with 85/100 penetration asphalt cement as one stabilization treatment, and with portland cement as another. These stabilized mixtures were laid as base course materials, and were compared with each other and with crushed stone and gravel bases by subjecting them to the test traffic.

On the basis of their performance under all AASHO Road Test traffic, it was established that 1 in. of the asphalt-stabilized sandy gravel base had the same traffic carrying capacity as 1.3 in. of the portland cement-treated sandy gravel base, 3 in. of high-quality crushed stone base, and 4 in. of the untreated sandy gravel subbase. Consequently, incorporation of either asphalt binders or portland cement is an effective means for upgrading the quality and load supporting capacity of inferior aggregates.

For the range of E_1 values normally required for flexible pavement design, bituminous materials seem to have a number of desirable characteristics as binders for increasing the E_1 values of aggregates and soils. By incorporating a bituminous binder into an aggregate, the resulting mixture has been waterproofed. Thoroughly compacted dense-graded bituminous mixtures are impervious to water and do not have to be drained. They are unaffected by frost action. They are not attacked by salts that are present in high concentration in the soil and groundwater in some areas or that are applied to the paved surface for snow and ice control. They develop a high elastic modulus E_1 under rapidly moving loads. Because they are cold they have very high load supporting value during spring breakup, and this tends to compensate for loss of subgrade support during this period. Because the binder itself is flexible, a well-designed asphalt-treated aggregate can adjust itself substantially within limits to the strains imposed by load and environment without cracking.

By how much the E_1 value of an aggregate material should be increased by incorporating a suitable binder, will be determined partly by technical and partly by economic considerations. As is well-illustrated by Figure 30, for a given wheel load the higher the E_1 value achieved the smaller is the pavement thickness required. Consequently, there will be an optimum combination of improved E_1 value attained by processing the aggregate with a binder, associated with a corresponding reduced thickness requirement,

TABLE 12

RELATIONSHIP BETWEEN PAVEMENT ELASTIC MODULUS AND PAVEMENT THICKNESS PROVIDED BY THE SOLUTION TO AN ELASTIC 2-LAYER FLEXIBLE PAVEMENT DESIGN PROBLEM[1]

Pavement Thickness, T (mult. of radius)	Pavement Thickness, T (in.)	Pavement Elastic Modulus, E_1 (psi)	Subgrade Elastic Modulus, E_2 (psi)	E_1/E_2
1.0r	6.0	105,000	1,500	70.0
1.5r	9.0	38,000	1,500	25.3
2.0r	12.0	21,000	1,500	14.0
3.0r	18.0	12,000	1,500	8.0
4.0r	24.0	9,500	1,500	6.3
5.0r	30.0	8,400	1,500	5.6
6.0r	36.0	7,800	1,500	5.2

[1] Wheel load = 9,000 lb
Tire inflation pressure = 80 psi
Radius of equiv. circular contact area = 6 in.
Critical pavement deflection = 0.1 in.
Subgrade elastic modulus, E_2 = 1,500 psi (CBR = 5.5)

that will result in the lowest pavement cost. Where aggregate materials of satisfactory quality but lower E_1 values are available in almost unlimited quantities, economy may indicate the use of a greater thickness of the untreated aggregate. On the other hand, in areas where aggregates are scarce and costly, or require upgrading because of their inferior quality, an economic study may show that treatment of the aggregate for the full depth above the subgrade with sufficient bituminous or other binder to provide a high E_1 value along with the correspondingly smaller thickness would be the most economical solution to the flexible pavement design problem.

The steps required to design a flexible pavement in accordance with the approach just described are as follows:

Step 1.—Prepare a design chart like that of Figure 28, or Figure 34, for the particular combination of wheel load, tire inflation pressure, and critical pavement deflection involved. For example, suppose that for the anticipated traffic volume of a single wheel load of 9,000 lb or equivalent at 80-psi tire inflation pressure, the critical pavement deflection is 0.1 in (Fig. 28) (corresponding circular bearing plate diameter = 12 in.).

Step 2.—Measure or otherwise determine the value of the elastic modulus E_2 of the subgrade on which the pavement is to be placed. For example, suppose E_2 = 1,500 psi (CBR = 5.5).

Step 3.—Using Figure 42 as a model, make a semi-logarithmic plot of E_1/E_2 versus T_0 for the 12-in. diameter bearing plate at 0.1-in. deflection (curve 1, Fig. 48).

Step 4.—Enter Figure 28 at E_2 = 1,500 psi, and read off the E_1 values for pavement thicknesses T = 6r, 5r, 4r, 3r, 2r, 1.5r, and r, and plot the corresponding calculated values for E_1/E_2 (Table 12) and T (Fig. 48).

Step 5.—Draw a smooth curve through the points representing the corresponding values for E_1/E_2 and T (curve 2, Fig. 48).

Step 6. —The intersection of the two curves in Figure 48 gives the ratio of E_1/E_2 corresponding to the optimum pavement thickness T_O, which is the pavement thickness at which the average increase in load supporting value per inch thickness of pavement is a maximum, I_m. From Figure 48, optimum thickness $T_O = 1.8r = 10.8$ in.; E_1/E_2 at optimum thickness = 17.0; E_2 = 1,500 psi. Therefore, E_1 at optimum thickness = 1,500 × 17 = 25,500 psi.

Step 7. —For each of the aggregates available in the vicinity of the project, determine the cost of the binder material and processing to upgrade it to various values of E_1 indicated by Table 12 and curve 2 of Figure 48.

Step 8. —Determine from curve 2 of Figure 48 which combination of pavement elastic modulus E_1 and pavement thickness will provide the required pavement at lowest cost, while being adequate in other respects.

When the entire thickness of pavement material above the subgrade has been treated with sufficient binder to approach the characteristics of a typical paving mixture, the E_1 value will usually tend to be high, and the corresponding pavement thickness required will be relatively low. In this case, only the top 1.5 to 2.0 in. of the pavement would consist of a standard asphalt concrete surface course mixture.

On the other hand, where considerations of technical adequacy and economy indicate that a greater thickness of pavement of lower E_1 value should be placed on the subgrade, the pavement material may be treated with a smaller quantity of binder, or may be untreated. In this case, the usual minimum requirements for a sufficient thickness of high quality material for the top of the pavement would govern and the standard minimum thickness of asphalt concrete base and surface course would be specified.

That there appears to be an optimum pavement thickness, within the approximate limits of one to two times the radius of the loaded area, at which a pavement material develops its maximum average increase in load supporting capacity per inch thickness of pavement, is a basic principle that could be usefully applied to certain foundation and other problems encountered in general soil mechanics.

In concluding this section, it is reasonable to ask why there should be an optimum pavement thickness for flexible pavement materials. A quantitative mathematical explanation will probably be forthcoming. Qualitatively, it appears that for thicknesses up to the optimum pavement thickness, the differences in strength characteristics between the subgrade and pavement materials are important. However, for increasing thicknesses above the optimum pavement thickness, the strength performance of the pavement structure gradually approaches what would be expected by placing additional pavement layers on the surface of a semi-infinite depth of the pavement material itself. Expressed somewhat differently, for thicknesses of pavement up to the optimum the pavement layers are being placed on a subgrade material that can differ substantially from the pavement material in strength characteristics. Above the optimum thickness successive layers of pavement are being placed on a structure that is becoming more and more like the pavement material itself.

PAVEMENT TRENCH CROSS-SECTION

This paper is primarily concerned with the case of a two-layer pavement system consisting of a pavement layer placed directly on the subgrade. By increasing the elastic modulus E_1 of the pavement material, the design wheel load can be supported by quite modest thicknesses of pavement, as shown by Figures 28, 30, 34, etc.

The usual approach to flexible pavement design at the present time calls for substantial thicknesses of granular subbase and base course materials, particularly over weaker subgrade soils. In areas subject to deep frost penetration, the use of frost blankets, consisting of granular material several feet thick, is frequently specified. Because these thick layers of granular material must be drained, it has become common practice to extend them from ditch slope to ditch slope in order that they will be self-draining. This added width has the serious disadvantage of adding greatly to the cost of flexible pavements, particularly in the case of multilane highways. In addition, experience has shown that many of these granular subbases and bases are not self-draining as supposed, and they actually become water reservoirs beneath the pavement.

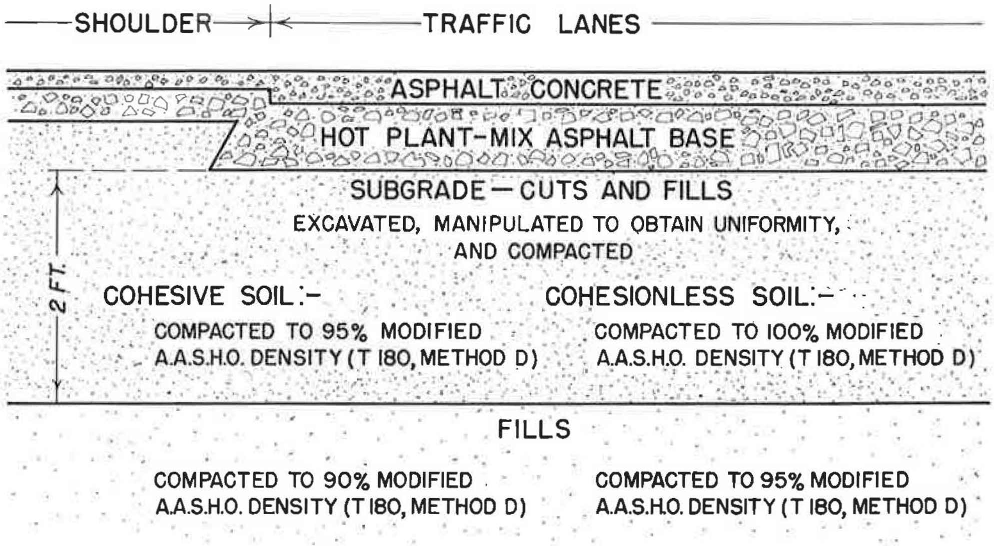


Figure 49. Suggested asphalt pavement cross-section.

By using the much smaller thicknesses of flexible pavement of higher E_1 values as described in this paper, combined with better management of the subgrade soil, the more economical pavement width trench cross-section can be adopted for flexible pavements (Fig. 49). By treating the aggregate with a bituminous binder to provide these high E_1 values, the pavement material is waterproofed. This waterproofed material can be successfully laid as a trench cross-section in most locations without any provision for subdrainage, because no water requiring drainage will accumulate in the pavement.

The cost of the greatly increased quantities of untreated base course and subbase aggregates required for ditch slope to ditch slope as compared with pavement width trench cross-section, or the cost of the special drainage measures needed when these untreated granular materials are used for trench cross-section, can be substantial. These added costs should be credited as savings when considering the expense of processing aggregates for the full depth above the subgrade with bituminous binders to increase their E_1 values, in order to utilize the much thinner pavement and the trench type of cross-section that this treatment makes possible.

LAYER EQUIVALENCIES

One of the most valuable findings of the AASHO Road Test (19) concerned the relative capabilities of equal thicknesses of the various base course and pavement materials employed to provide load carrying capacity. This is usually referred to as "layer equivalency."

The test data from the AASHO Road Test, including those from the special base sections, indicated that for the particular materials employed for this project, 1 in. of hot-mix asphalt-treated base was equivalent in load carrying capacity to 1.3 in. of portland cement-treated base, 3 in. of high quality crushed stone base, and 4 in. of sandy gravel subbase. However, similar materials incorporated in road or airport pavement structures elsewhere could show quite different layer equivalencies, depending on their gradation, particle size and shape, composition, nature of the asphalt binder, relative degrees of compaction, etc.

It is usually assumed that, as at the AASHO Road Test, carefully constructed test sections and controlled or uniform test traffic are required to obtain representative values for layer equivalencies for the many types and compositions of subbase, base course, and wearing course materials in common use, and for the various conditions to which they are subjected in service. Although this can be a reliable method when properly employed, it is costly and time-consuming.

Figures 28 and 30 demonstrate that the layered system elastic theory approach to pavement design is capable of providing values for layer equivalencies. For example, when the subgrade elastic modulus E_2 is 1,130 psi (CBR 3 subgrade), it is clear from Figure 28 that a 9-in. thickness of pavement material with an elastic modulus E_1 of 65,000 psi is just as capable of supporting heavy traffic by a 9,000-lb wheel load or equivalent, as 36 in. of a pavement material with an elastic modulus E_1 of 9,000 psi. For this example, the layer equivalency of the former is 36/9 = 4 in terms of the latter pavement material. Also, Figure 30, taken from Figure 28, shows that when placed on the same CBR 3 subgrade (E_2 = 1,130 psi), 12 in. of pavement material with an elastic modulus E_1 of 32,500 psi has the same 9,000-lb wheel load carrying capacity as 30 in. of a pavement material with an elastic modulus E_1 of 9,600 psi. In this case, the layer equivalency of the first pavement material is 30/12 = 2.5 when expressed in terms of the second; that is, 1 in. of the first pavement material is equivalent in load carrying capacity to 2.5 in. of the second.

It is apparent, therefore, that if representative values for the elastic moduli E_1 for the different pavement materials are available, and if the corresponding pavement thickness requirement can be determined for the conditions associated with each particular paving project (for example, Fig. 30), the layer equivalencies between these various pavement materials can be easily and quickly established.

INFLUENCE OF ENVIRONMENT ON PAVEMENT DESIGN

The characteristic of a highway that the average motorist probably values above all others is smoothness of ride. Consequently, the ultimate objective of pavement design and construction should be the attainment of pavements that are initially smooth and that remain smooth riding throughout their service life. The AASHO Road Test staff made a major contribution to highway engineering by focusing attention on this objective when they adopted loss of smoothness of ride (present serviceability rating or present serviceability index) as the basis for measuring the amount and rate of deterioration of the test sections at the Road Test (20).

Highway engineering, like most other technological activities, progresses by stages from one plateau of achievement to the next. Since the greatest era of road building in history began about 35 years ago on the North American continent, the introduction and gradual improvement of strength concepts of pavement design and construction has occupied one of these plateaus of development in highway engineering. Only recently have highway engineers acquired sufficient knowledge and experience to build pavements of approximately adequate strength which avoids serious over- or under-design, and there is still much to learn. This is indicated by the fact that it was one of the objectives of the recently completed AASHO Road Test to provide information that would increase the precision of pavement strength design. It is reasonable to ask, therefore, what the next stage or plateau of development in pavement design and construction will be.

Figures 50 and 51 show the results of extensive road roughness investigations directed by Housel (21, 22) on a large mileage of both rigid and flexible pavements in Michigan over a period of years. In spite of the fact that these pavements were designed to have adequate load carrying capacity, Figures 50 and 51 demonstrate that both the rigid and flexible pavements have lost smoothness of ride at an average rate of about 4.5 in. per mile per year in terms of the road roughness scale employed. The road roughness index is the sum of the vertical heights of all the bumps large and small that occur in a mile of highway, and is expressed as inches per mile. When the accumulated roughness reaches 150 in. per mile (equivalent to a present serviceability rating of 2.5), the pavement is no longer safe for high volume high-speed traffic and should be resurfaced. Because the pavements represented by Figures 50 and 51 are considered to be structurally adequate, Housel points out that the average increase of 4.5 in. of roughness index per mile per year must be due to environmental factors.

Figure 52 represents somewhat similar results from Benkelman beam studies on

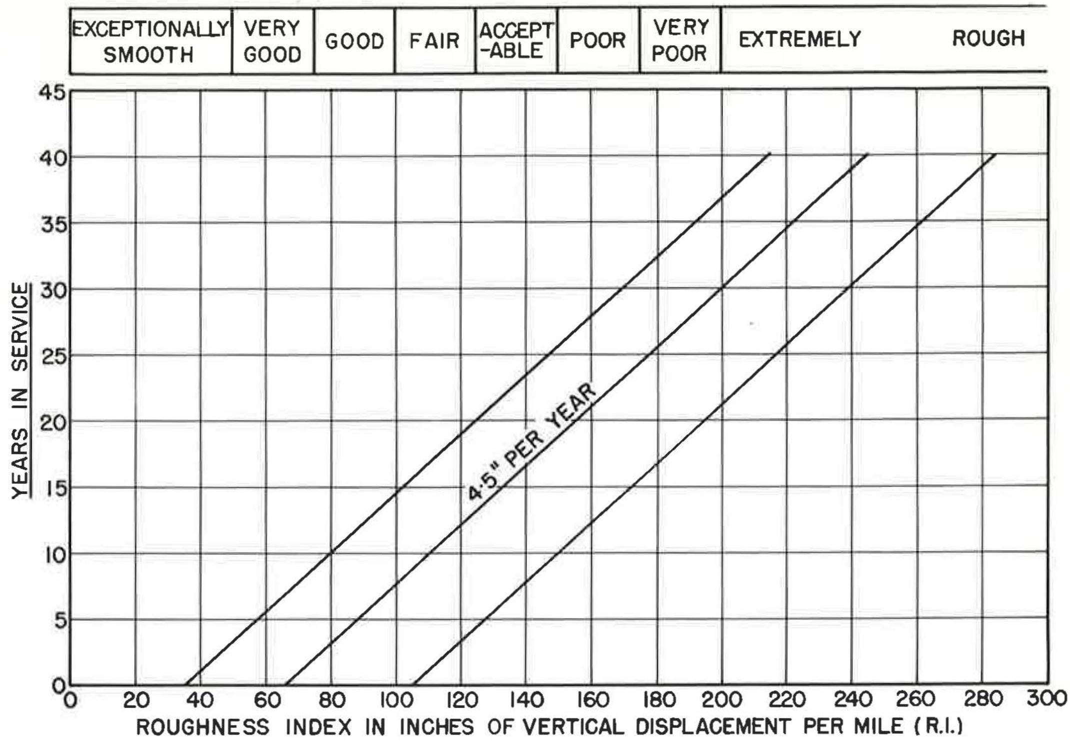


Figure 50. Roughness index versus years in service, rigid pavements in Michigan.

flexible pavements in Canada, reported by the Committee on Pavement Design and Evaluation of the Canadian Good Roads Association (4). The broken lines refer to low traffic volume, and the solid lines to relatively high traffic volume. Three degrees of pavement strength are represented—weak, medium, and strong—having corresponding Benkelman beam deflections of 0.075, 0.05, and 0.025 in., respectively. The ordinate scale shows present serviceability ratings from 0 to 5 as employed for the AASHO Road Test. Comparison of the lowest pair of broken and solid lines (light and heavy traffic, 0.075-in. deflection by Benkelman beam), which diverge noticeably, shows that when the pavement is weak it loses smoothness of ride with age faster under heavy traffic than under low traffic volume. On the other hand, the top pair of broken and solid lines (light and heavy traffic, 0.025-in. deflection by Benkelman beam), are close together and indicate that for pavements of adequate strength the rate of increase of pavement roughness with age is approximately the same for either low or high traffic volume. Like Housel, the Canadian Good Roads Association's Committee on Pavement Design and Evaluation interprets this latter observation as evidence of the strong influence of environmental factors on the rate of deterioration of pavement smoothness with time.

It was pointed out earlier that progress in technical fields usually occurs by stages, and that during the past 30 years or so, highway engineers have been primarily concerned with learning how to build pavements of adequate load carrying capacity. However, just as this objective appears to be within reach, the concept of the present serviceability rating introduced by the AASHO Road Test staff, and the data on rate of deterioration of pavement smoothness with age provided by measurements made on thousands of miles of in-service highways by Housel and the CGRA Committee on Pavement Design and Evaluation, indicates that building pavements of adequate strength is not enough. The contributions of these organizations have made it evident that the next stage of development in the pavement field must be learning how to design and

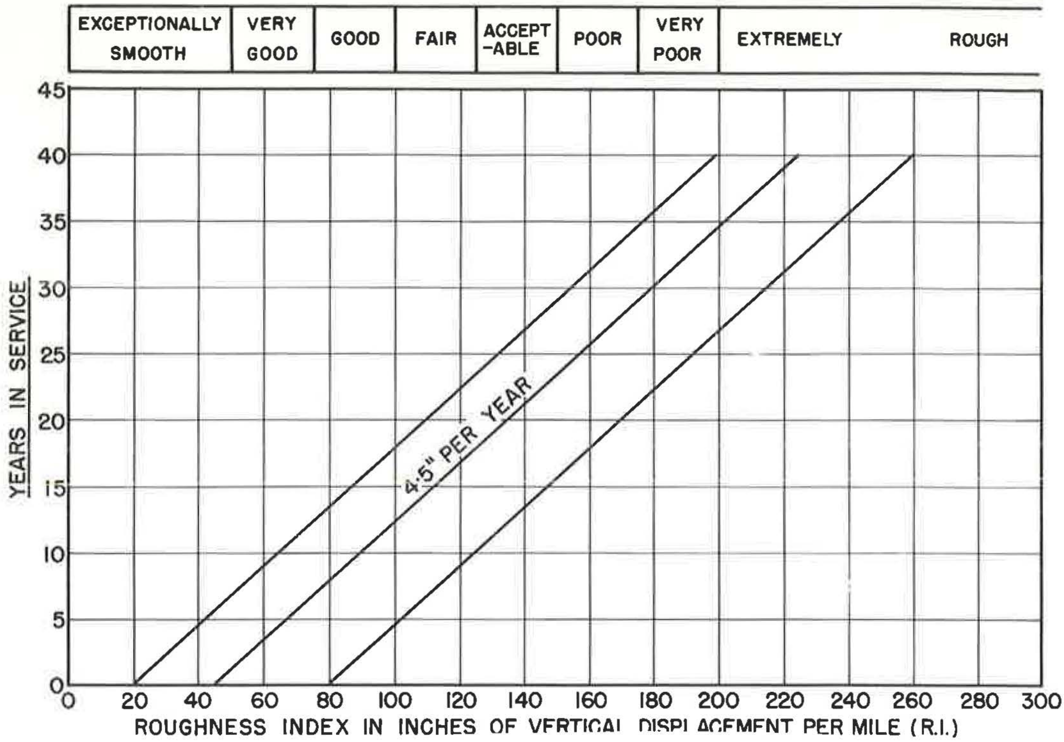


Figure 51. Roughness index versus years in service, flexible pavements in Michigan.

construct pavements that not only will have adequate load carrying capacity, but also will at the same time remain smooth riding throughout their service life in spite of the forces of the environments in which they are located.

An example of what the highway engineer faces in this respect can be taken from the field of structural engineering. Like the highway engineer, the structural engineer has been plagued with the problems of environment throughout history. On the basis of the knowledge of the strength of materials, and of the principles of stress analysis that have been developed during the past 100 years or so, it is not difficult for a structural engineer to design and construct a multi-storied building that will ordinarily be quite safe for any specified floor loading. Nevertheless, many of these structures have disintegrated when subjected to an earthquake, a powerful environmental force.

In the case of the Imperial Hotel in Tokyo it has been demonstrated that by applying special principles of design which took the local environment into account, a building could be constructed to withstand the destructive forces of even severe earthquakes, as witnessed by the fact that the Imperial Hotel is still in service in spite of disastrous earthquakes that have leveled other large buildings in its vicinity during the 40-odd years since its completion.

Highway engineers may claim that through the use of drainage measures, frost blankets of granular materials, elevated grade lines, moisture control, compaction, etc., they have for many years been taking the environment into account in pavement design and construction. In general, however, the principal purpose of these measures in the past has been to achieve and to maintain adequate pavement strength. The investigations of Housel and of the CGRA Committee on Pavement Design and Evaluation have shown that although design and construction procedures adopted in the past with respect to the influence of environmental forces may have been able to preserve pavement strength, they have been unable to maintain surface smoothness. Consequently, considerable modification of current design and construction procedures, and probably

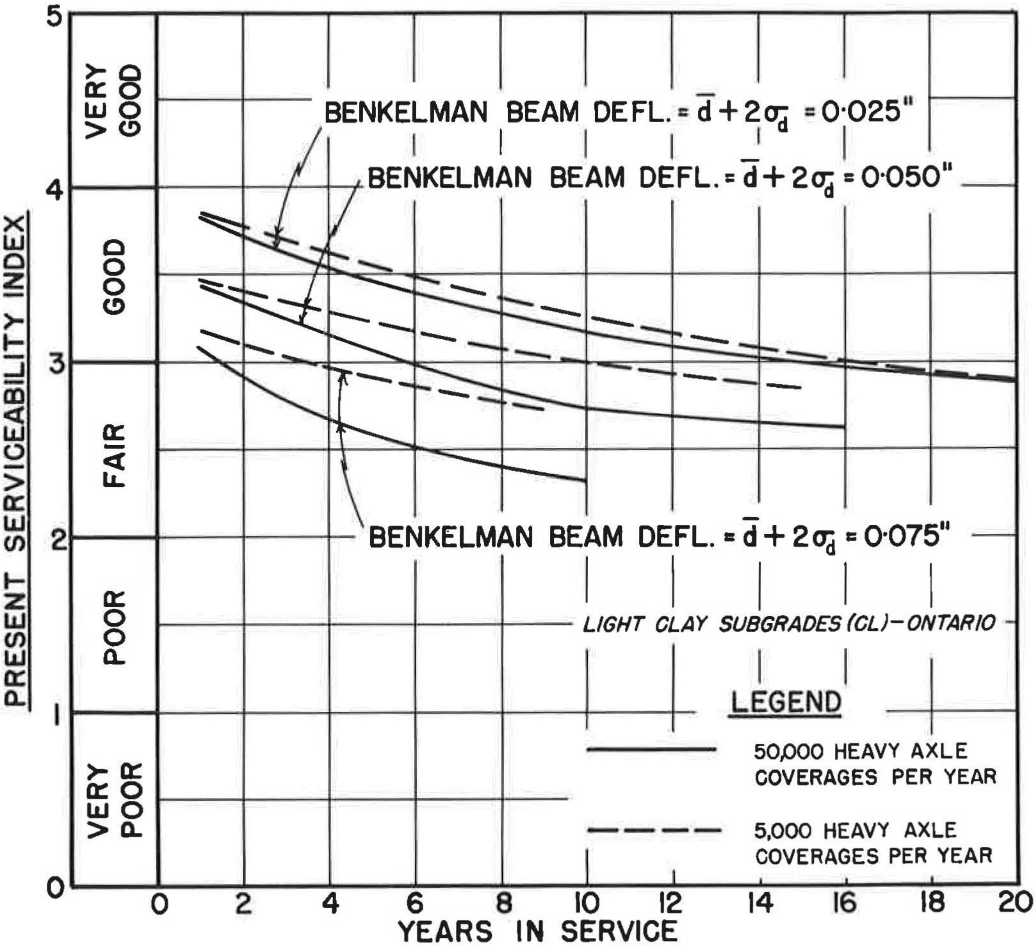


Figure 52. Influence of pavement strength and traffic volume on the service performance of weak, medium, and strong pavements in Ontario, Canada.

some completely new approaches, are required if the environmental forces which detrimentally influence pavement strength and pavement smoothness are to be eliminated or adequately controlled.

At present, the various environmental influences, and the mechanisms through which they operate to the detriment of pavement strength and pavement smoothness, are not all known or understood. However, some of the more obvious environmental factors are subgrade soils with their variable texture, structure, density and general lack of homogeneity; local drainage characteristics, including depth to water table; topography, with its influence on grade line in cuts and fills; rainfall; climate; existence or absence of frost; and prevalence or scarcity of granular materials suitable for construction.

In colder climates, three major destructive environmental factors are nonuniform soil, poor drainage, and frost. With respect to frost action, it is generally recognized that frost heaving requires a combination of three basic conditions—a frost-susceptible soil, a readily available source of groundwater, and prolonged freezing weather. The freezing weather must be accepted; but the availability of groundwater can be controlled by adequate drainage, which may often be simplified by raising the grade line. Because the frost-susceptible soil frequently occurs in cuts or at the junction of cut and fill in the form of pockets or layers of silt or fine sand, its frost susceptibility can often be practically eliminated by excavating the complete soil, mixing it to obtain uniformity, and recompacting it into place.

Figure 49 represents an attempt to provide a more favorable pavement environment, primarily by obtaining a homogeneous and adequately compacted subgrade on both cut and fill sections. This involves the complete removal of soil from the top 2 ft of cut sections, mixing it thoroughly to obtain a uniform texture, and recompacting it into place. With modern earthmoving equipment, adequate mixing of the soil is usually achieved by excavating it in thin layers to the depth specified, followed by replacement and compaction in thin lifts. If in spite of the homogeneity obtained, the subgrade soil still tends to be frost-susceptible due to availability of groundwater, the grade line can be elevated to provide more positive drainage. On most projects soil is the cheapest construction material available and considerable manipulation, together with any additional volume of soil needed for a higher grade line, can usually be provided at less cost than the greater depth of granular base materials otherwise required. Although Figure 49 indicates that a 2-ft depth of cut is to be handled in this manner to obtain homogeneity, the actual depth to be treated will vary with soil type and local conditions of climate and drainage, and could range from about 1.5 to about 3.0 ft.

In many areas granular materials are becoming scarce and more costly. Consequently, granular subbases and base courses, and the use of considerable depths of granular materials as frost blankets over the subgrade, have also been eliminated from the pavement cross-section of Figure 49.

It is customary in highway engineering to specify granular subbases and base courses to obtain load carrying capacity for flexible pavements and to require considerable thicknesses of granular materials as frost blankets over subgrades of so-called frost-susceptible soils in cold climates. These frost blankets are applied partly with the intention of controlling differential frost heaving, and partly to provide added load supporting capacity when the subgrade is weakened during the spring breakup period. The use of various depths of granular materials has become conventional, partly because they have usually been reasonably plentiful, and partly because they have provided a relatively inexpensive means of obtaining the necessary load supporting capacity. Consequently, because it has eliminated granular subbases and base courses, the pavement cross-section of Figure 49 represents a break with traditional methods employed for flexible pavement construction. However, the pavement cross-section of Figure 49 appears to be justified by the performance of asphalt-treated bases at the AASHO Road Test and elsewhere, and its advantages have already been described.

It should be noted that frost blankets as such were not specified for either the AASHO or the WASHO Road Tests, although both projects were located in areas where the subgrade is frozen for several months. The subgrade soil at the AASHO Road Test was an A-6 clay soil, and was a highly frost-susceptible A-4 silt loam at the WASHO Road Test. Furthermore, some highway departments in cold climates do not employ frost blankets, usually in areas where granular materials are scarce and too costly to use in this way. Experience of these highway departments, along with that at the WASHO and AASHO Road Tests, has shown that pavements of adequate strength to carry full traffic even during the spring breakup period can be built without the need for several feet of granular material as frost blankets.

Investigations by the Frost Effects Laboratory of the U. S. Army Corps of Engineers have demonstrated that granular materials are poor insulators against frost penetration into the subgrade, and that the amount of loss of subgrade strength during the spring breakup or thaw period appears to be independent of the depth of subgrade frost penetration.

Observation has also shown that granular subbases and bases thought to be free-draining are not always capable of acting in this way. In some cases, these saturated or partly saturated bases actually contribute to the frost action they are intended to prevent.

As long ago as 1930, following a survey of frost action in highways in Michigan, Burton and Benkelman (23) reported that 80 percent of the frost heaves studied occurred in cuts, 18 percent at the transition from cut to fill, and only 2 percent on fills.

Experience elsewhere has usually been somewhat similar.

The soils in the fills studied by Burton and Benkelman in most cases came wholly or in at least in part from the cuts that showed a high percentage of frost heaves. This high percentage of frost heaves in cuts is usually due to lack of soil uniformity, and to the presence of pockets or layers of soil of frost heave texture, particularly silts and fine sands. The transportation and handling procedures required to place these soils in fills would result in reasonable homogeneity. In addition, fills tend to be better drained than cuts.

One important inference from these findings of more than 30 years ago is that the soil in cuts should be completely excavated, manipulated to attain uniformity, and then thoroughly recompacted into place (Fig. 49). As previously indicated, the depth of cut to be treated in this way varies with soil type and local conditions of climate and drainage, and could range from about 1.5 to about 3.0 ft.

Housel's (21) pavement roughness survey showed that the most variable pavement performance in Michigan occurred in pavements placed over subgrades consisting of well-drained sands and gravels. These natural granular subgrades were assumed to be homogeneous at the time the pavements were built, and the subgrade soil in cuts was not disturbed. The variable pavement performance over these undisturbed soils gives further emphasis to the commonly observed fact that soils in nature are seldom uniform.

As a result of these findings, the Michigan State Highway Department was reported to be planning to excavate soils in cuts to a depth of 18 in., then mix and recompact them into place. This was intended to eliminate the differences in texture, structure, and density of the natural soil that are considered responsible for the wide variations in the rate of loss of pavement smoothness demonstrated by the pavement roughness survey.

It is recognized that, initially at least, the pavement cross-section of Figure 49 may not always result in the desired degree of improvement in retention of pavement smoothness. However, experience can be a skillful if not always efficient teacher. Furthermore, in some areas and under certain conditions some other pavement cross-section may provide better overall pavement performance. In general, this is due to the fact that there is still much to be learned about environmental forces and how they act.

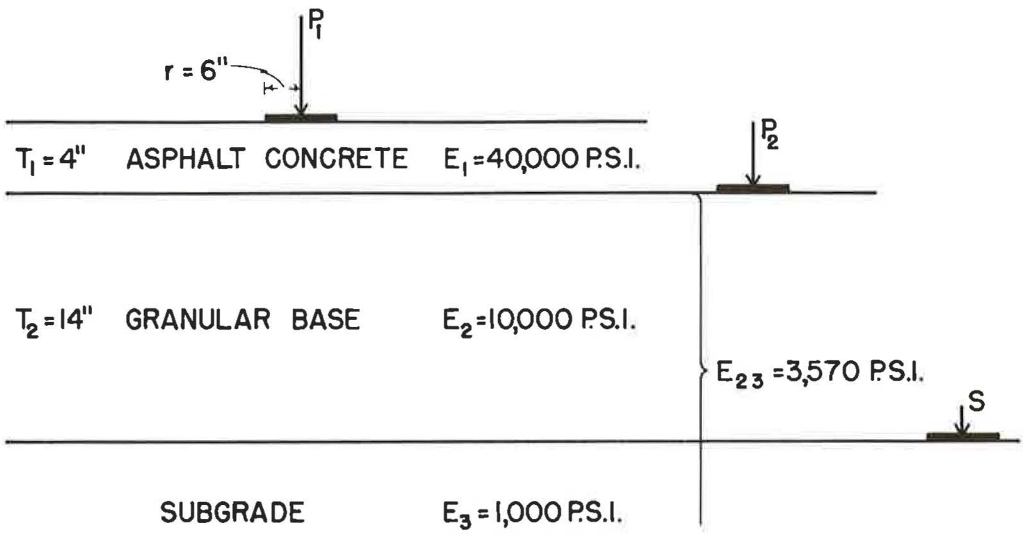


Figure 53. Example of design of an elastic three-layer pavement for a 9,000-lb wheel load at 0.1-in. deflection when the elastic moduli E_1, E_2, and E_3 are given.

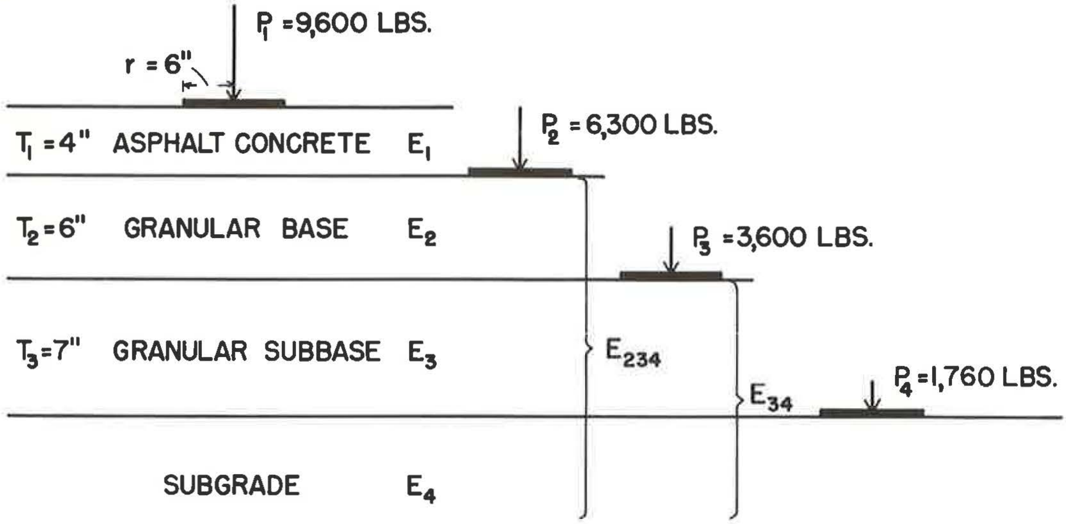


Figure 54. Example of the use of plate bearing test data to evaluate the elastic moduli E_1, E_2, E_3, and E_4 of an elastic four-layer pavement.

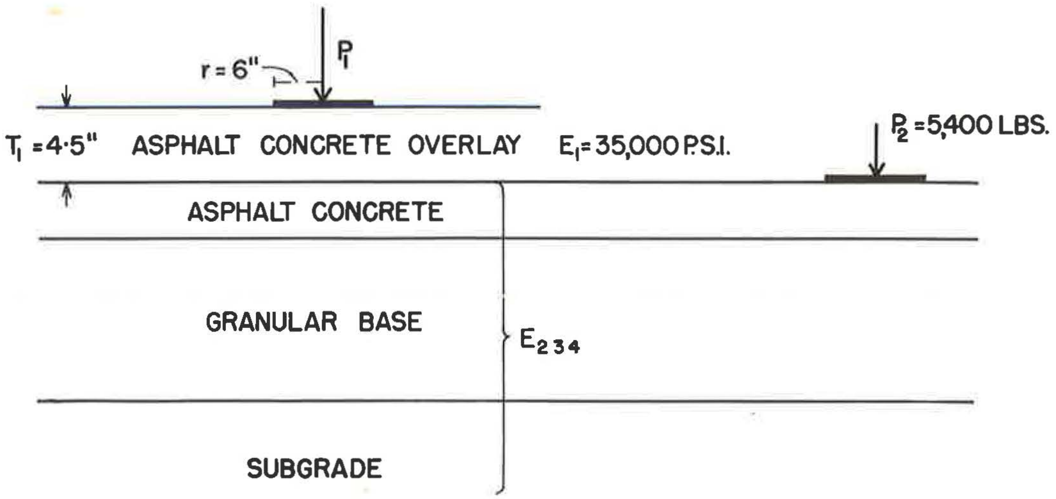


Figure 55. Example of the design of an asphalt concrete overlay for an existing pavement on the basis of the elastic modulus E_1 of the overlay material.

In view of current incomplete knowledge concerning the nature of the various environmental forces and of the mechanisms through which they operate to influence pavement performance, a program of planned stage construction has much to recommend it. Construction of a new road upsets whatever equilibrium nature has gradually achieved over thousands of years within the landscape it crosses. Experience has shown that it requires time to reach a new equilibrium under the additional forces introduced by the presence of the road. In spite of the adoption of the most advanced currently known design and construction procedures, differential movements within the subgrade and pavement are inevitable while the adjustments required to establish the new equilibrium are taking place. These adjustments usually tend to occur rapidly at first, but decrease in both rate and magnitude with time. Consequently, if a program of planned stage construction were adopted, the subgrade would be properly built initially, but only sufficient pavement would be laid to provide adequate load carrying capacity for the traffic anticipated during, say, the first five years. Within this period, the major adjustments required to attain the new equilibrium will ordinarily have taken place. By placing the balance of the pavement at the end of this time, the subsequent rate of loss of pavement smoothness will be much slower, and its useful service life will be substantially increased. Consequently, for the same total capital expenditure, a program of planned stage construction should provide longer pavement life and more satisfactory pavement performance.

DESIGN AND EVALUATION OF MULTI-LAYER ELASTIC PAVEMENT SYSTEMS

This paper is concerned primarily with two-layer elastic pavement systems for several reasons. From a purely technical point of view, thinner two-layer pavements of adequate strength, having optimum thicknesses within the range of 1.5 to 2r (Fig. 42), represent a much more efficient use of aggregate materials than the much greater thicknesses required by traditional flexible pavement design. They make the economies of trench construction possible (Fig. 49). They have the other advantages described earlier. Nevertheless, in regions where suitable granular materials are plentiful and inexpensive, conventional flexible pavement cross-sections consisting of granular subbase, granular base, and asphalt surface course, can be economically attractive.

This section, therefore, considers the design and evaluation of multi-layer elastic pavement systems, such as those shown by Figures 53, 54, and 55. The approximate method employed, which in effect converts a multi-layer elastic pavement step by step to an equivalent two-layer pavement, is illustrated by three sample calculations involving a three-layer pavement (Fig. 53), a four-layer pavement (Fig. 54), and the design of an asphalt concrete overlay to strengthen an existing flexible pavement (Fig. 55).

It should be noted that in each of these cases the values shown for pavement and subgrade elastic moduli are provided by or related to plate bearing tests. Consequently, they are the conservative static elastic moduli.

Case 1—A Three-Layer Pavement (Fig. 53)

Given: $E_1 = 40,000$ psi, $E_2 = 10,000$ psi, $E_3 = 1,000$ psi, $T_1 = 4$ in., $T_2 = 14$ in., and critical pavement deflection = 0.1 in.

Problem: Will this pavement support heavy traffic consisting of a wheel load of 9,000 lb or equivalent at a tire inflation pressure of 80 psi?

Solution: r = Radius of equivalent circular contact area = 6 in., $E_2/E_3 = 10,000/1,000 = 10$, $T_2/r = 14/6 = 2.333$ and, from Figure 10, $F_W = 0.28$. Eq. 5 can be rewritten in terms of total load S, rather than unit pressure s, as

$$E_2 = \frac{1.18\, s\, r}{w} = \frac{0.376\, S}{w\, r} \tag{18}$$

Therefore, $S = E_2\, w\, r/0.376 = (1,000 \times 0.1 \times 6)/0.376 = 1,600$ lb and, from Eq. 15a, $P_2 = S/F_W = 1,600/0.38 = 5,710$ lb.

Consequently, a two-layer system consisting of 14 in. of granular base of $E_1 = 10,000$ psi on this subgrade will support a load to 5,710 lb on a 12-in. bearing plate at 0.1-in. deflection.

The assumption is now made that an equivalent homogeneous soil of elastic modulus E_{23} will also support a load P_2 = 5,710 lb at 0.1-in. deflection, from which, employing Eq. 18, $E_{23} = (0.376\ P_2)/(w\ r) = (0.376 \times 5{,}710)/(0.1 \times 6) = 3{,}580$ psi. Then E_1/E_{23} = 40,000/3,580 = 11.2, T_1/r = 4/6 = 0.667, and, from Figure 10, F_W = 0.63. Also, from Eq. 15a, $P_1 = P_2/F_W$ = 5,710/0.63 = 9,070 lb.

Therefore, the pavement in Figure 53 will support heavy traffic by a wheel load of 9,000 lb or equivalent.

Layer Equivalency.—The thickness of the granular base material in Figure 53 that would be required to provide the same load supporting value can be obtained as follows: $F_W = S/P_1$ = 1,600/9,070 = 0.177, E_2/E_3 = 10,000/1,000 = 10, from Figure 10 T/r = 5.6, and T = 5.6r = (5.6 × 6) = 33.6 in.

Therefore, the layer equivalency of the asphalt concrete in terms of the granular base material is (33.6 - 14)/4 = 19.6/4 = 4.9; that is, each inch of asphalt concrete has the supporting value of 4.9 in. of the granular base material.

Case 2—A Four-Layer Pavement (Fig. 54)

Given: S = 1,760 lb, P_3 = 3,600 lb, P_2 = 6,300 lb, P_1 = 9,600 lb, T_3 = 7 in., T_2 = 6 in., T_1 = 4 in., r = radius of loaded area = 6 in., and critical pavement deflection = 0.1 in.

Problem: To evaluate the elastic moduli E_4, E_3, E_2, and E_1.

Solution: From Eq. 18, $E_4 = (0.376\ S)/(w\ r) = (0.376 \times 1{,}760)/(0.1 \times 6)$ = 1,100 psi, T_3/r = 7/6 = 1.167; from Eq. 15a, $F_W = S/P_3$ = 1,760/3,600 = 0.489; and from Figure 10, E_3/E_4 = 7.8. Therefore, E_3 = 7.8 × 1,100 = 8,600 psi. From Eq. 18, $E_{34} = (0.376\ P_3)/(w\ r) = (0.375 \times 3{,}600)/(0.1 \times 6)$ = 2,250 psi; from Eq. 15a, $F_W = P_3/P_2$ = 3,600/6,300 = 0.57 and T_2/r = 6/6 = 1; and from Figure 10, E_2/E_{34} = 6. Therefore, E_2 = 2,250 × 6 = 13,500 psi. From Eq. 18, $E_{234} = (0.376\ P_2)/(w\ r)$ = $(0.376 \times 6{,}300)/(0.1 \times 6)$ = 3,940 psi and T_1/r = 4/6 = 0.677; from Eq. 15a, F_W = P_2/P_1 = 6,300/9,600 = 0.66; and from Figure 10, E_1/E_{234} = 9. Therefore, E_1 = 9 × 3,940 = 35,460 psi.

Case 3—Asphalt Concrete Overlay (Fig. 55)

Figure 55 illustrates an existing pavement that was originally constructed for a 9,000-lb wheel load, but for only light to medium traffic volume. It must now be strengthened to carry a high traffic volume of 9,000-lb wheel loads or equivalent.

Given: r = Radius of loaded area = 6 in., critical pavement deflection = 0.1 in., P_2 = load supported by existing pavement = 5,400 lb, elastic modulus of asphalt concrete overlay E_1 = 35,000 psi, and T_1 = thickness of overlay = 4.5 in.

Problem: Will this overlay increase the load supporting value of the pavement to 9,000 lb?

Solution: From Eq. 18, $E_{234} = (0.376\ P_2)/(w\ r) = (0.376 \times 5{,}400)/(0.1 \times 6 = 3{,}380$ psi, E_1/E_{234} = 35,000/3,380 = 10.4, and T_1/r = 4.5/6 = 0.75; from Figure 10, F_W = 0.59; and from Eq. 15a, $P_1 = P_2/F_W$ = 5,400/0.59 = 9,150 lb. Therefore, the existing pavement plus the overlay will support high volume traffic consisting of a wheel load of 9,000 lb or equivalent, at a tire inflation pressure of 80 psi.

The solutions to the problems of Cases 1, 2, and 3 were obtained by means of Eqs. 18 and 6 and Figure 10. Solutions to these problems could also have been obtained by employing Eqs. 18 and 1 and Figure 11. This can be illustrated by applying them, for example, to the first step of Case 1, and to the final step of Case 3.

Case 1, First Step

E_2/E_3 = 10,000/1,000 = 10; T_2/r = 14/6 = 2.333; from Figure 11, K/r = 4.2 and K = 4.2 × 6 = 25.2; and from Eq. 18, $S = (E_2\ w\ r)/0.376 = (1{,}000 \times 0.1 \times 6)/0.376$ = 1,600 lb. Substituting in Eq. 1 gives 14 = 25.2 log P_2/1,600, from which, P_2 = 5,740 lb.

Within the accuracy of interpolation in Figures 10 and 11, this value of P_2 is in close agreement with the 5,710 lb previously obtained.

Case 3, Final Step

From Eq. 18, $E_{234} = (0.376\ P_2)/(w\ r) = (0.376 \times 6{,}300)/(0.1 \times 6)$ = 3,940 psi and T_1/r = 4/6 = 0.667; from Eq. 1, 4 = K log 9,600/6,300, or K = 21.88 and K/r = 21.88/6 = 3.647. From Figure 11, E_1/E_{234} = 9; therefore, E_1 = 9 E_{234} = 9 × 3,940 = 35,460 psi, which is identical with the previous result.

NEEDED RESEARCH ON THE ELASTIC LAYERED SYSTEM APPROACH TO PAVEMENT STRUCTURAL DESIGN

Some of the problems concerning the elastic layered system approach to pavement structural design on which research is urgently needed are as follows:

1. A computer analysis to calculate F_W values with greater precision, followed by redrafting of Figure 10 to a maximum of accuracy and including more lines for E_1/E_2 values. In addition, a greatly enlarged accurate diagram of the upper left corner of Figure 10 is needed, to enable required values in this region to be read more precisely.
2. The development of a rapid, simple, precise laboratory method or methods for measuring both the static and dynamic elastic moduli of various pavement and subgrade materials, that will provide values for elastic moduli equal to those developed by the same subgrade and pavement materials under actual traffic loads on highways, streets, and airports.
3. An investigation to establish the influence of deflection, size of loaded area, traffic speed, pavement thickness, temperature, degree of compaction, moisture content, paving mixture composition, etc., on the value of the elastic modulus E_1 of different pavement materials, and also the influence of many of these factors on the elastic modulus E_2 of subgrade soils.
4. If for any given wheel load and traffic volume a constant pavement thickness is specified, then the larger the permissible pavement deflection (critical pavement deflection) the smaller is the value of the pavement elastic modulus E_1 required. Consequently, how can flexible pavements be made more flexible to enable higher critical pavement deflections to be specified, and thereby take advantage of the lower pavement elastic modulus E_1 values that could then be employed?
5. What minimum permissible values of the pavement elastic modulus E_1 should be specified for various combinations of wheel load and traffic volume to avoid pavement failure by shear or plastic flow?
6. Identification of the environmental factors and of the various mechanisms through which they operate to influence pavement strength and loss of pavement smoothness with age, and the development of practical methods to control, completely eliminate, or minimize the detrimental effects of these environmental forces.

SUMMARY AND CONCLUSIONS

1. The empirically derived equation $T = K \log (P/S)$, resulting from the analysis of load test data obtained by the Canadian Department of Transport on airport pavements in Canada, is reviewed and is shown to belong to the elastic theory category of pavement design methods.
2. It is shown that the pavement factor K in this empirically derived equation is equal to $T/-\log F_W$, in which F_W is the deflection factor from Burmister's theoretical elastic layered system approach to pavement design. Consequently, this entirely empirical equation for flexible pavement design, and Burmister's purely theoretical equations, are mathematically related.
3. Analysis of the Hybla Valley load test data shows that for any given size of loaded area, the pavement factor K varies with pavement thickness and has a minimum value at some thickness roughly between the radius and the diameter of the loaded area.
4. Calculations and a diagram based on Burmister's theoretical equations are presented which demonstrate that these equations could have been used to predict that the pavement factor K would vary with pavement thickness, and that the value of K would go through a minimum.
5. Evidence indicating that for any given wheel load the permissible pavement deflection must be reduced as the traffic volume is increased, is reviewed.
6. On the basis of Hybla Valley load test data, the influence of variables such as size of loaded area, pavement thickness, and deflection, on values of the pavement elastic modulus and the subgrade elastic modulus is investigated, and the need for a rapid, simple, precise method for evaluating E_1 and E_2 for both dynamic and static loading conditions is emphasized.

7. Evidence is presented that a flexible pavement structure appears to react in the same way insofar as its stress-strain characteristics are concerned, whether load is applied by a pneumatic tire or by a steel bearing plate.

8. Pavement design charts for a 9,000-lb highway wheel load have been prepared on the basis of an elastic two-layer pavement system, showing acceptable corresponding relationships between subgrade elastic modulus, pavement thickness t, and pavement elastic modulus. These charts demonstrate that for any specified wheel load, traffic volume, and subgrade supporting value, there are many combinations of corresponding values for pavement thickness and pavement elastic modulus that are equally capable of providing the load carrying capacity required. This enables the best selection to be made from the pavement materials available, on the basis of both technical and economic considerations.

9. Similar design charts for a 60,000-lb airplane wheel load are included. These provide useful information; for example, when wheel load, traffic volume, subgrade support, and pavement thickness are kept constant, these charts demonstrate the increase in pavement elastic modulus required for a tire inflation pressure of 200 psi, when compared with that needed for a tire inflation pressure of 100 psi.

10. Data from the Hybla Valley project are particularly useful for study because of the great care taken to ensure uniformity of materials and construction procedures. It is significant, therefore, that analysis of the Hybla Valley load test data shows that for a given loaded area the average increase in load supporting value per inch thickness of granular base course varies with base course thickness. Furthermore, the average increase in load supporting value per inch thickness of base course reaches a maximum at a base course thickness roughly equal to the diameter of the loaded area.

11. Tables and diagrams based on either Eq. 1, or Burmister's theoretical equations for an elastic two-layer system are included. They show that this elastic theory approach to pavement design could have predicted that for each pavement material (at least among those normally used for flexible pavements) the average increase in load supporting value per inch thickness of pavement varies with pavement thickness. In addition, these equations indicate that there is an optimum pavement thickness, usually within the range of 1.5 to 2.0 times the radius of the loaded area, at which the pavement material develops its maximum average increase in load supporting value per inch thickness of pavement.

12. Consequently, both the empirical load test data from Hybla Valley and the theoretical equations based on an elastic two-layer pavement system indicate that there is a critical or optimum pavement thickness at which a given pavement material provides the highest average increase in load supporting capacity per inch thickness of pavement of which it is capable. For thicknesses either greater or smaller than this optimum thickness the pavement material is less efficient with respect to load supporting value and provides a gradually decreasing average increase in supporting value per inch thickness of pavement.

13. A table is provided to demonstrate the serious loss of efficiency in supporting value per inch thickness of pavement that can occur with increasing pavement thickness above the critical or optimum thickness. For a CBR 3 subgrade, a uniform pavement material with an elastic modulus E_1 = 11,300 psi, a 12-in. diameter loaded area, and a critical pavement deflection of 0.1 in., it is shown that at the optimum pavement thickness of 10.5 in. the average increase in supporting value per inch thickness of pavement is 340 lb. On the other hand, the 5-in. increment of pavement thickness between 30 and 35 in. increases the overall load supporting value of the pavement structure by only 120 lb per inch thickness of pavement.

14. It is generally recognized that there is need to conserve the remaining deposits of good natural aggregates in many regions. There is also need to upgrade the quality of inferior aggregate materials by soil stabilization processes, among which the incorporation of asphalt binders was shown by the AASHO Road Test to be particularly effective.

Data presented in this paper suggest that the conventional method of flexible pavement design, which calls for great thicknesses of granular subbase and base course materials over poorer soils, tends to be wasteful of the diminishing aggregate resources because of the inefficiency, in terms of load supporting capacity, of pavement thicknesses well above the optimum.

On the other hand, from a purely technical point of view one of the most effective methods of conserving aggregate materials would be to upgrade their quality (increase their load carrying capacity) by the incorporation of asphalt binders, or by other means, until at the optimum pavement thickness or even less they would have adequate supporting capacity for the design wheel load and traffic volume to be carried. However, economic considerations will usually establish the permissible degree of upgrading of quality, with its corresponding reduction in pavement thickness, in comparison with the cost of the much greater thickness of untreated granular material required.

15. When all of the material above the subgrade is asphalt treated, the pavement thicknesses required are relatively small and the advantages and economies of trench construction become possible.

16. It is shown that the elastic theory layered system approach to pavement design provides a simple method for establishing layer equivalencies between different pavement materials.

17. It is pointed out that, up to the present, highway engineers have been gradually learning how to build pavements of adequate strength. Data presented indicate that the next stage of development in pavement design and construction will involve learning how to build pavements that not only are of adequate strength, but which will remain smooth-riding for many years. To achieve this objective, highway engineers must learn how to eliminate or adequately control the environmental forces that detrimentally influence both pavement strength and pavement smoothness. It is shown that planned stage construction can be usefully employed to attain this objective.

18. In many areas, conventional flexible pavement design, consisting of a granular subbase, granular base course, and asphalt wearing surface, will continue to be economically attractive. Consequently, an approximate method for the design and evaluation of elastic multi-layered pavement systems is reviewed and illustrated.

19. A list of some of the problems concerning the elastic layered system approach to pavement structural design on which research is urgently needed, is included.

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Appendix

GLOSSARY OF SYMBOLS

E = elastic modulus of the pavement material or of the subgrade;

F_w = Burmister's deflection factor;

i = average increase in load supporting value per inch thickness of pavement, psi;

i_m = maximum average increase in load supporting value per inch thickness of pavement, in psi (occurs at T_o);

I = average increase in load supporting value per inch thickness of pavement in lb;

I_m = maximum average increase in load supporting value per inch thickness of pavement, in lb (occurs at T_o);

K = pavement factor;

p = unit load applied to the pavement, in psi;

P = total load applied to the pavement, in lb;

P/A = perimeter/area ratio of the loaded area;

r = radius of loaded area;

s = unit load supported by the subgrade for the same loaded area and deflection that pertain to p, in psi;

S = total load supported by the subgrade for the same loaded area and deflection that pertain to P, in lb;

T = pavement thickness;
T_o = optimum pavement thickness; and
w = deflection under load, in in.

Thickness of Flexible Pavements by the California Formula Compared to AASHO Road Test Data

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For the past several years, the State of California has been using a pavement structural design method based on test road data and on observed performance of pavement structures. The original formula, containing factors for traffic, supporting power of the soil, and slab strength of the pavement and base layers, has been modified at times as better information became available.

This paper describes not only the design formula but also modifications suggested from a study of the AASHO Test Road data. Correlation with test track data is shown.

•SOILS and granular materials have been used in building construction, for walls, floors, and pavements, for many thousands of years. Obviously, the ancients must have had a great amount of practical knowledge about the use of such materials. When the designers and builders of ballistae, catapults, and similar engines of war turned their attention to other forms of construction, precise methods for estimating the potential behavior of materials began to emerge. The need to design stable earthworks was probably most pressing on the military engineers and one of these, Charles Augustin Coulomb (1736-1806), was among the first to propose a formula by means of which the stability of earthwork embankments might be computed. Nevertheless, in spite of the long history of engineering works involving earthy materials, formulas for calculating the bearing capacity of soils have not been as reliable or perhaps not as well understood as are formulas for bridge members and other structures.

Engineering is a profession that requires an understanding of several sciences and disciplines but which depends primarily on a knowledge of materials and how the materials will perform or "stand up" under given conditions. The typical engineer has a working knowledge of physics, mechanics, mathematics, and is acquainted with a collection of somewhat inexact numbers and values optimistically referred to as "the strength of materials." The strength concept seem to be reasonable, sound, and "common sense." However, it is deceptively simple and can be misleading. A layman knows that a 12- by 12-in. timber beam will sustain a greater load than will be 2- by 4-in., and can also grasp the idea that a steel beam will support a greater load than a wooden beam of the same dimensions. Carpenters, millwrights, masons, and even architects have designed and constructed some fairly elaborate structures without very much in the way of recognizable engineering training. However, though the strength properties of wood, stone, or iron may be reasonably well appraised by experience or intuition, this approach has been less successful in estimating the ability of soils and foundations to sustain loads.

A great deal of the difficulty may be ascribed to the lack of means for identifying and measuring the important properties of the materials involved. Although the "strength idea" is accepted almost spontaneously and instinctively and presents no serious difficulties when applied to such things as steel, timber, and reinforced concrete, it does become a little blurred and the image rather fuzzy when one tries to apply this term to the properties of soils. It becomes even more elusive when applied

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to cohesionless sand and fails completely to describe the properties of liquids such as water.

Webster's dictionary defines "strength":

> Power to resist force; solidity or toughness; the quality of bodies by which they endure the application of force without breaking or yielding; a measure of the cohesion of material; firmness; coherence; as the strength of bone, beam, wall, rope, et cetera.

The word "strength" obviously has many meanings and shadings, and it does not mean the same thing when applied to different materials and circumstances. One may speak of a strong wind or a strong current of water but what is meant is that when either a gas or a liquid is in motion it can exert considerable force. A "strong man" may also be able to exert considerable force but he cannot necessarily withstand as much as a "weak woman." At least women have shown that they often have great powers of resistance! One speaks of a strong steel cable or a nylon rope, and such strands are strong in the sense of the dictionary definition meaning "cohesion." For most engineering materials, the word strength actually denotes only tensile strength, but materials such as soils can "endure the application of force" and yet possess little or no tensile strength. It, therefore, appears that a more precise general term for these properties is "resistance." This term is explicit and may be applied without confusion to a variety of materials. Thus, a strong steel wire or a cable requires a considerable force to overcome its resistance to breaking. A column of stone blocks or a dry rubble wall exerts considerable resistance to compressive forces. Even more pertinent to this discussion, the common materials of the earth's crust (rock, sand, gravel, soil, or mud) can all be shown to offer measurable degrees of resistance to applied forces. But these materials have little cohesion and hence little or no "strength" unless combined with an artificial binder such as asphalt or portland cement, and even the tensile strength of concrete is not very great compared to steel, for example.

THE PAVEMENT PROBLEM

All pavements, regardless of type, rest upon the materials of the earth's surface, and though there are a few examples of relatively solid rock subgrades, the vast majority of highway pavements are supported by soils or related granular materials having low cohesive strength. Nevertheless, a wide variety of soils have "what it takes" to support pavements if the pavement structure is "properly designed." This means that soils possess some pertinent property other than cohesive strength and this property is easily identified as interparticle friction. The importance of both friction and cohesion was recognized by Coulomb, and values for each appear in his formulas.

To apply the principles of engineering to the structural design of a pavement, the engineer must know what properties of materials are involved. Lack of reliable tests has been on of the greatest stumbling blocks. Many of the tests that have been applied to soils and paving materials do not provide measures of fundamental properties. For example, if one wishes to measure the tensile strength of steel, a carefully prepared specimen is attached to the jaws of a testing machine and the force required to pull the specimen apart is measured. This is a direct measurement of an important property. If the strength of concrete is involved, a carefully prepared test cylinder or cube is subjected to a direct compression loading. However, even though steel and concrete are often combined to produce reinforced concrete structures, one rarely attempts to measure the properties in combination. The individual strength properties are evaluated by separate tests. Unfortunately, in the case of soils and other granular materials, a number of test methods are affected by the two distinct properties acting simultaneously.

Many tests provide no means for differentiating between such radically different attributes as friction and the cohesive resistance. Though the resistance to deformation or displacement due to friction is fairly well defined (if not well measured), the cohesive "strength" or resistance is generally defined as "that portion of the resistance to sliding that is not affected by the pressure." This is a negative definition and differs from the dictionary definition of cohesive strength. In effect then, the soil mechanics definition of cohesion does not define what cohesion is, it merely says what it is not. The other element of confusion arises from the use of such devices as the Mohr circle analysis in which the intercept of the Mohr envelope on the vertical scale is defined as "cohesion." Tests on certain obviously cohesionless materials have shown a definite value for the intercept which would therefore be defined as "cohesion." Finally, a great many have been "thrown off the track" by the substitution of such terms as "shear strength" which by itself is not a property of materials; the total resistance to shear being again composed of variable portions of frictional and cohesive resistance. The resistance due to each of these dissimilar properties combines to produce the total resistance in an endless variety of combinations. The use of tests such as the CBR test, several varieties of direct shear tests, or unconfined compression tests, all tend to reflect or summarize some arbitrary combination of friction and cohesion. The relative proportions depend on the geometry of the test specimen and speed of loading which usually differ considerably from the conditions on an actual roadway.

Both geologists and agronomists have studied fragmentary stone and the finer decomposition products called "soil" and each group has developed classification schemes and names for the numerous varieties of rock, gravel, sand, and soil types. These classifications have their uses and have proved helpful to the engineer but none are directly fitted to the engineer's problem. As stated by Feld, "an adequate soil classification scheme for engineers should be based upon engineering properties." All this leads up to the point that soil, sand, gravel, and other naturally occurring mineral materials possess a number of properties and characteristics and can be variously described according to geologic origin, petrographic classification, grain size, soil texture, mineralogical composition, or even the chemical compounds involved. These classifications may or may not indicate the suitability of the material or the best means of treatment for engineering purposes.

As with all the other sciences concerned with soils, the engineer needs to know what properties are important to him and what determines the ability of the soil to support loads, and having identified these properties he must then know what test methods to use to measure them. This is a step that must be made first as no reliable or valid mathematical formula for structural design can be developed unless it includes numerical values to express real and essential properties of the materials involved.

In 1948 a design formula for calculating the thickness of pavements (1) was reported which includes an expression for the measured resistance value of various soil or granular layers and for the tensile strength or cohesive resistance of all elements composing the pavement structure. The basic data for the relationships developed were derived from a small but full-scale project known as the Brighton test track constructed by the California Division of Highways in 1940. For an expenditure of less than $100,000, it was possible to construct and operate a test track that included eight different types of base material varying in thickness from 3 to 18 in. resting on the same saturated silty clay soil having a CBR value of about 3 or an R-value of approximately 17. The track was subjected to a loaded truck and at the end of the operation it was evident that the thickness required for the various types of base did not show any consistent relationship to the CBR value or the resistance value for the base material itself, but there was an orderly and consistent trend with the tensile strength of the materials as measured by the cohesiometer. This test track made it possible to assign tentative values to some of the variables such as the effects of wheel load and repetition. Though the underlying soil on the test track was uniform throughout and gave no range of value, some additional check points were obtainable from observations on the State highway system. A few scattered examples where the pavement thickness had been varied over different types of soils made it possible to establish a relationship. The establishment of a scale of values for soil support was greatly simplified by the fact that the thickness of pavement structure required bears a linear relationship to the resistance value of the soil as measured by the stabilometer. There was no opportunity to introduce a variation in tire pressure so the effects of this variable were not established. The equation developed at the time was

$$T = \frac{(K\, P \sqrt{a} \log r)\,(P_h/P_v - 0.10)}{\sqrt[5]{C}} \tag{1}$$

in which

T = thickness of cover (base and pavement) (in.);
K = 0.0175 for best correlation but without any factor of safety (for design purposes, it is suggested that $K = 0.02$);
P_h = transmitted horizontal pressure in the stabilometer test (psi);
P_v = applied vertical pressure in the stabilometer test (typically 160 psi);
P = effective tire pressure (psi);
a = effective tire area (sq in.);
r = number of load repetitions; and
C = tensile strength of the cover material as measured by the cohesiometer in grams per square inch (approximately equals modulus of rupture $\times$ 45.4).

Eq. 1 was simplified by reducing the effects of load and repetition to an expression termed the traffic index and by reducing the stabilometer data to a resistance value R. Eq. 1 then becomes

$$T = 0.095\ \frac{(\text{traffic index})\,(90-R)}{\sqrt[5]{\text{cohesion value}}} \tag{2}$$

in which

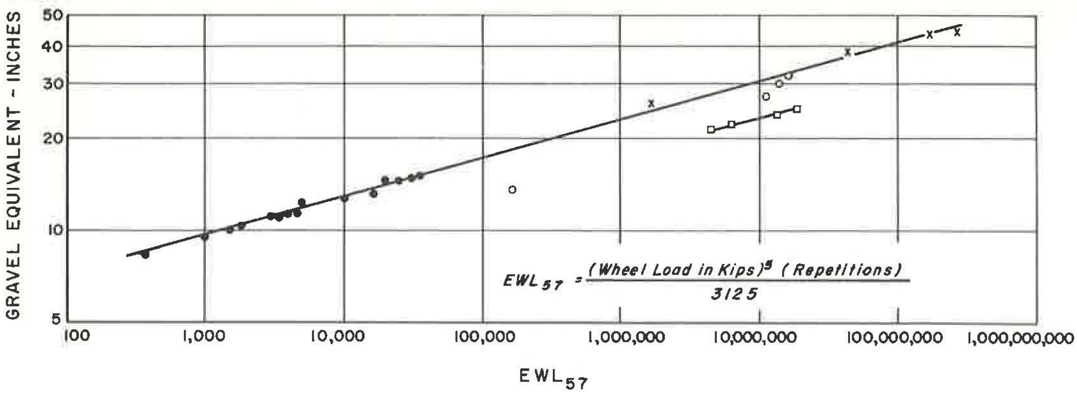
T = required thickness of cover; and
R = resistance value by stabilometer.

This equation was used for the design of pavements, and any discrepancies that became apparent between prediction and performance were noted and modifications in the testing and design procedure were introduced as seemed to be warranted.

On the completion of the WASHO test road in Idaho, attempts were made to check the California formula by comparison with the performance on the WASHO test road. Unfortunately, the design of this project was such that only a very few definite points could be established. Although the usable data from the WASHO road agreed with the predictions of the formula, they were insufficient to confirm its validity over any substantial range (Fig. 1).

The tremendously larger AASHO test road in Illinois furnishes a great deal more data and gives a much wider range of values for checking a previously established structural design formula. To make a comparison between calculated values and test road data, the various materials, basement soils, granular base, subbase, and asphaltic pavement were tested and evaluated according to the California procedures. The wheel loads and number of trips were converted through the equivalent wheel load calculation to the traffic index number. With values derived by laboratory tests of the Illinois materials and calculations for the traffic, it is possible to arrive at a design thickness based on the California formula (1957 Model). The calculated thicknesses may then be compared with the actual thickness reported to be necessary on the test road. The correlation is shown later in Figure 5. The statistical values showing a standard error of estimate of ± 2.7 in. and a coefficient of correlation of 0.87 (Appendix B) seem to confirm the ability of the California design formula to predict the thickness of pavement required for a wide variety of traffic loads and materials.

The test road data, however, neither prove nor disprove the applicability of the California formula to other types of soil or granular base materials. The test road pavement structures were supported by only one type of basement soil. Because of this lack of variables on the AASHO project, it is not possible to develop a design formula by using the test road data alone. Also, the statistical-type formulas developed by the road test staff have no terms or identities that permit application to soils differing in properties and ability to support loads from those used on the test road. The test road formula does not identify or indicate means for measuring the properties or physical



TEST TRACK DESIGNATION	WHEEL LOAD	TYPE OF SOIL SUPPORT
□ WASHO		17 Avg. CBR Soil
• Brighton	6 KIP	4 Avg. CBR Soil
○ Stockton	25 KIP	5 Avg. CBR Soil
x Stockton	40 KIP	

Figure 1. Correlation of EWI$_{57}$ with gravel equivalent.

conditions that account for the performance of the subbases, bases, and asphalt pavement types.

FACTORS TO BE CONSIDERED IN A DESIGN FORMULA

A design formula for the structural elements of a pavement should embody all the important factors that affect the ability of the pavement structure to sustain vehicle loads over a substantial period of years. There have been many formulas proposed. M. S. Kersten (2) has listed 22 different ones. Some of these were based on theoretical concepts, others were completely empirical, and some represented a mixture of the two approaches. The factors that influence the over-all performance of a pavement are so numerous and the desirable attributes of a pavement are so diverse that it seems impossible or highly improbable that all of these variables can ever be included in a single formula, or if such a formula were constructed, only a highly sophisticated electronic calculator could hope to reach a solution. Even then, a certain allowance would be needed for the inability to do a perfect job.

Figure 2 is included to show the variables that can affect the performance of an asphalt pavement. At least 30 items have been identified. However, design formulas rarely need to cover every factor, and many of the variables shown in the figure can be ignored or combined into a single element in the formula.

As an example of the simplification that is possible and quite practicable, an adequate structural design might be described as one that produces an economical or efficient pavement that will neither crack nor deform under the assumed traffic during the design life of the pavement. (Guarding against disintegration types of failure is primarily a question of mixture design and quality of materials rather than a structural design problem.) Column 3 of Figure 2 shows that there are three primary factors; namely, the effects of traffic, the strength of the pavement, and the ability of the foundation to support the load. The primary factors have the following relationship:

$$T = \frac{KD\,(90-R)}{S} \tag{3}$$

in which T = thickness; K = constant; D = destructive effect of traffic; R = resistance value of support; and S = strength of pavement structure.

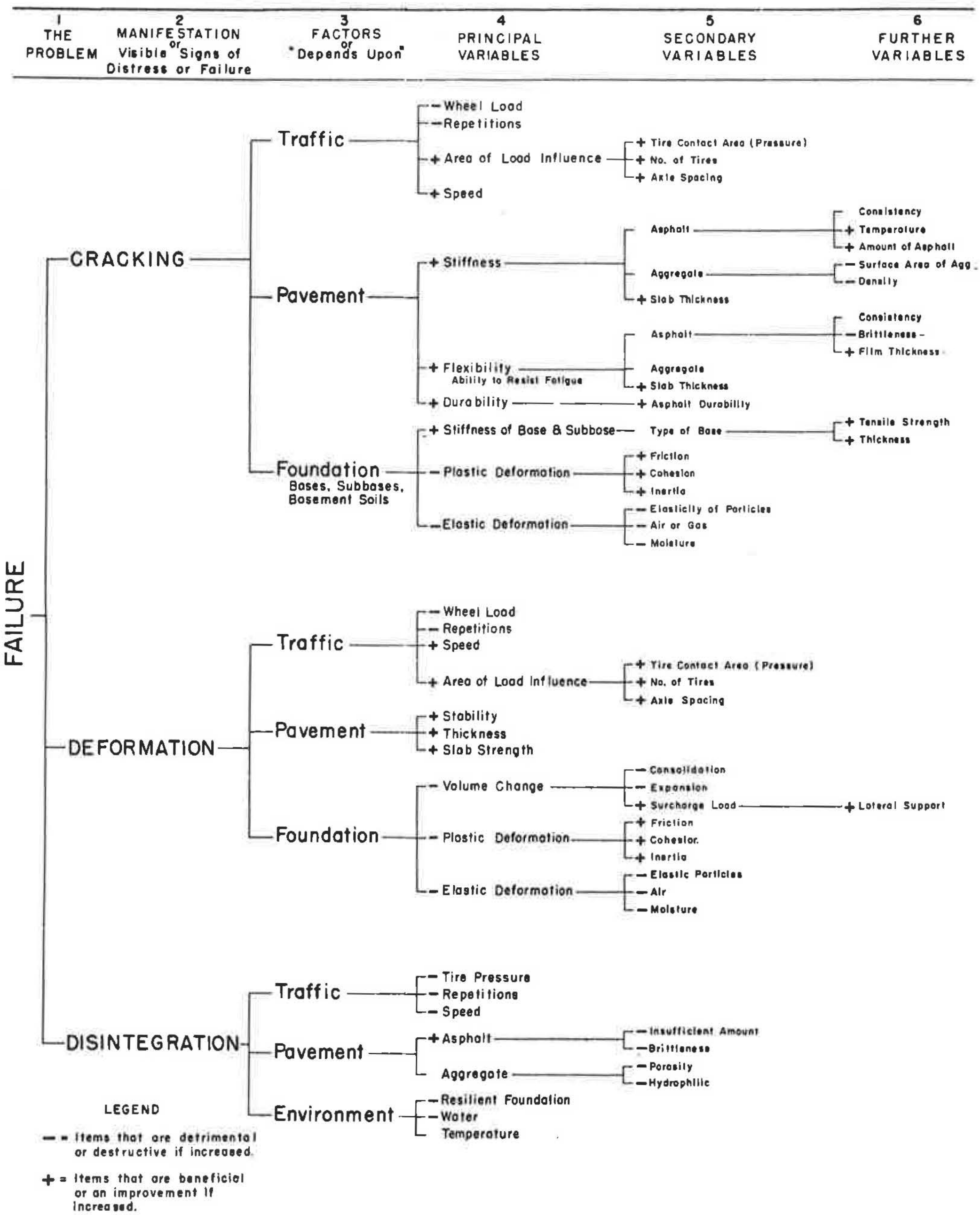


Figure 2. Analytical chart showing variables that must be evaluated for structural design of asphalt pavements.

To derive a number to express the effect of traffic, it is necessary to consider Columns 4 and 5 which list some of the subdivisions that make up the traffic load effect or "the destructive effect of traffic." The principal variables are the total wheel load in contact with the pavement and the number of times this load passes over the pavement. The area of load influence is a factor but the problem has thus far been simplified for highway traffic as the maximum tire pressure on most motor trucks is in the order of 70 or 80 psi for the heavier vehicles. The axle spacing or "the proximity factor" is confined to only two typical configurations; namely, single axles some 15 ft apart or tandem axles (2 axles within 4 ft). Although the comparative effects of tandem axles vs single axles differ markedly as those between flexible pavements and rigid pavements, nevertheless, it is possible to convert these two types of axle spacings to a common denominator for each type of pavement.

Examining all the available data which include the Brighton test track, the Stockton track (constructed by the Corps of Engineers), the WASHO and the AASHO projects, it appears that the relative effects of traffic may be expressed as follows for flexible pavement design:

$$TI = 1.30 \left(\frac{W}{5}\right)^{0.050} r^{0.119} \tag{4}$$

in which

TI = traffic index;
W = wheel load in kips for tandem axles (W = 1.10 individual wheel load); and
r = number of load applications.

This equation assumes a tire pressure in the range of 50 to 100 psi but does not provide for effects of extreme variation in tire pressure as there are insufficient data available to indicate how variation in tire pressure may affect the performance of a road structure.

Figure 2 also shows there are a number of factors that compose the over-all properties of the pavement. Primarily, there is a question of stiffness or the resistance to bending. The term "stiffness" has been borrowed from a report by L. W. Nijboer and C. van der Poel (3). Nijboer computes stiffness from

$$S = \frac{F_p}{X_p} (12) \tag{5}$$

in which

F_p = force acting on pavement in newtons [limits of F_p between 10^4 newtons (1 ton) and 2×10^4 newtons (2 tons), respectively]; and
X_p = deflection of the pavement in microns.

Therefore, the term "stiffness" bears a simple mathematical relationship to the deflection of the pavement, and as used by Nijboer, "stiffness" implies the resistance of all components including the pavement, bases, subbases, and the underlying soil. For design purposes it seems preferable to associate the concept of stiffness with the pavement and base structures alone, in which case there will not be a consistent relationship between "stiffness" and "deflection" as the character of the supporting soil will then represent a variable: resilience.

Stiffness of a "flexible" pavement is influenced by the thickness, the type and amount of asphalt, and the temperature. This means that an asphalt pavement has a high degree of stiffness during cold weather and it also means that the lower courses provide greater stiffness in warm weather than the same mixture in the surface layer exposed directly to the sun. The stiffness of all materials can be expected to increase with the thickness of the layer but in the case of asphalt pavements the effect is enhanced by the lower temperatures in the bottom courses, especially where the pavement is of substantial depth. Flexibility is more or less the opposite number, or complement, of stiffness. This is a property not easily measured but it may enable a pavement to survive the flexing over resilient or springy foundations. It is a difficult value to include in a simple design formula.

The word "stiffness" is also not entirely applicable or adequate to express the manner in which a pavement structure functions. The concept of "stiffness" is readily visualized in the case of a thick asphalt pavement. It is even more descriptive of a portland cement concrete slab, but a substantial layer of crushed stone or gravel will have the same effect, within the limits of its own resilience, in reducing deflections. Precisely speaking, the term stiffness hardly seems appropriate for a bed of cohesionless material. Nevertheless, in the absence of a better term, a thick layer of sand or gravel may be said to have "stiffness." The question of pavement stability and resistance to water action are properties that fall into the area of mix design and need not ordinarily be considered in a structural design formula.

The process of assigning strength or resistance values to foundation materials must resolve a great many variables due to the wide variety of materials that may be involved. The treated bases and subbases may possess properties similar to that of the pavement layer, whereas granular bases and underlying soils are generally low or completely lacking in tensile strength or cohesive properties. As inferred in the preceding, a great deal of the so-called fundamental or theoretical approach to the design problem has focused attention on the elastic properties but for the most part it is the plastic properties of soils, subbases, and granular bases that have caused the most trouble. Again, one must recognize the very dissimilar response of friction and cohesion to most tests or loads.

The stabilometer furnishes a means for measuring the internal friction or granular materials under load. When solid particles such as stone or sand grains are coated with asphalt or wet clay, a lubrication effect is introduced as soon as a sufficient quantity of the lubricant has been added. Obviously, the amount needed and effects produced may vary considerably. Rough crushed stone particles are difficult to lubricate, whereas smooth polished gravel and sands will tolerate only small amounts of asphalt or wet clay additions. The problem of stability of asphalt pavements or the ability of granular bases and subbases to support a pavement depends very largely on the friction or the degree to which the friction has been reduced or lost by lubrication. Thus, the designer of bituminous mixtures or clay-bound stone bases is confronted with the fact that the very materials added to increase the cohesion (strength) will also reduce the friction through lubrication whenever sufficient amounts have been added.

When the cohesive effect is provided by a viscous liquid such as asphalt it becomes impossible to summarize the two unlike properties except under some specific condition of load area and speed of loading. Furthermore, the two properties are individually important and each is most effective in certain regions or zones of the pavement structure. A bed of cohesionless crushed stone, gravel, or sand will support traffic provided the surface is covered with an adequate thickness of material that does possess some cohesion. A surface treatment or seal coat on a gravel road is an example, but to be successful, a certain depth of the gravel must have some coherence or cementing action furnished by a soil binder. In contrast, a thin seal coat would be completely ineffective on a bed of clean beach sand. There is ample evidence therefore to show that an adequate pavement structure must provide an upper layer of material having some coherence or tensile strength, and the thickness of this layer must increase with increasing wheel loads. Beyond this critical depth, a completely cohesionless gravel or sand will serve quite well and will often prove to be less critical and give more lasting service than will base and subbase layers cemented with natural materials. Natural materials may consist of soil including clay or fines produced by degradation of the aggregate. Figure 3 shows the regions in the pavement structure where cohesion and friction are most influential or important. Figure 4 is an alignment chart suggesting the depths of pavement and/or cohesive base layer that is required over a completely cohesionless material.

For various magnitudes of wheel loading, the AASHO test road furnishes examples that supplement observations on the performance of actual highways. On Loop 2, the thin bituminous surface treatment resting directly on the soil gave a better performance and sustained a greater number of trips before failure than did the same thickness of surface resting on the gravel, yet the soil had a lower CBR and a lower R-value, and would be considered to be far less adequate by most methods of evaluation thus far

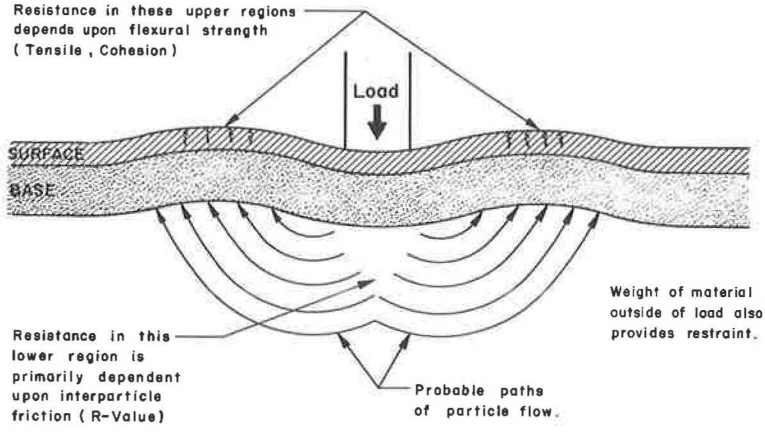


Figure 3. Plastic flow phenomena in soils supporting a pavement.

developed. Referring to Loop 5, the wedge sections 457, 458, 467, and 468 also demonstrate that the failure of the pavement was due to the gravel base as it failed as readily with 15 in. of base depth as with 5 in.

In the California formula, one of the factors that reflects the effect of pavement thickness is the (90 - R) factor which in effect states that a material of 90 R-value would be of sufficient strength to support any highway traffic load. This expression was developed from early data when the formula was devised and was based on extrapolations from rather light traffic. Furthermore, the factor appeared to correlate with California experience.

The data from the AASHO Road Test would indicate that a more rational approach to determining thickness of pavement for heavy traffic would be to use a factor of (100 - R). This would provide adequate thickness over most of the sections that appeared to fail because of inadequate base cover. This adjustment in the (90 - R) factor is made possible through more accurate information on the effect of traffic and also by adjustment of the cohesion factor in the formula. Again, in the gravel base wedge sections in Loop 5 of the Road Test, 4 ½ in. of asphalt concrete, in lieu of the 3 in. provided would have been required over the base material if any of the base thicknesses were to have survived the Test Road traffic. Likewise, in Loop 4, for the same wedge of gravel base, it would appear that 3¾ in. in lieu of 3 in. would be required for the traffic of Loop 4 to have been satisfactorily carried over the wedge for the duration of the project. These increased thicknesses of surfacing over these cohesionless gravel materials would have allowed the effect of gravel thickness to have been measured in a uniform and consistent manner, with the principal variable being thickness of base.

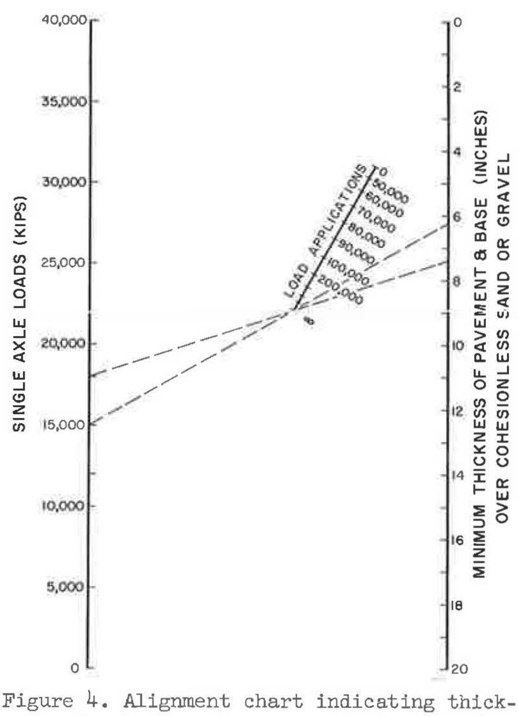


Figure 4. Alignment chart indicating thickness of cohesive layer required over cohesionless sand or gravel.

Figure 5 shows the correlation between the thickness computed by the California method (1957 revision) (4) and the actual minimum thickness found to be adequate on the AASHO test road. It appears that the greatest discrepancy between the predictions of the California formula and the actual performance is in the bituminous base sections, and it is therefore evident that the assumed cohesive strength value that has been used for California asphalt pavements is not adequate to account for the performance of the thick asphalt section on the test road.

To evaluate the Test Road performance of thicker asphalt concrete sections properly, it was necessary to revise the scale of cohesion values used in the California formula. The original formula assumed there was a cohesion of 100 for gravel and no materials would be less than 100. However, in trying to evaluate the AASHO test road, it became evident that the gravel base, for example, had far less than 100 cohesion. Actual tests performed on this material indicated a cohesion of only 20 g per lineal inch. Cohesions on the crushed rock base material had a value of only 30. To obtain more accurate definition with the design formula, it appeared expedient to change the basic cohesion for cohesionless materials (such as the AASHO subbase) from 100 to 20 and to use a value of 30 for crushed rock bases. The use of a more cohesive material (such as asphalt) or a cementitious material (such as portland cement) has a greater effect in the reduction of thickness of section. An evaluation of the effect that bituminous bases have on performance of the wedge sections of the Test Road provides some information. Table 15 of AASHO Road Test Report 5 gives information showing the equivalencies in terms of inches of gravel for both the bituminous-treated and the cement-treated bases. From the AASHO information, the equivalencies in Table 1 were developed.

TABLE 1

EQUIVALENCIES OF TREATED BASES

App. (1,000)	Base Type[a]	Equivalency (in. of stone base per inch of treated base)							
		Loop 3		Loop 4		Loop 5[b]		Loop 6[c]	
		12K-S	24K-T	18K-S	32K-T	22.4K-S	40K-T	30K-S	48K-T
100	CTB			1.8	2.2			1.9	2.0
	BTB	2.9	3.2			2.4	2.3	3.0	2.9
300	CTB			1.7	1.8			1.6	1.6
	BTB	2.8	2.9			2.2	2.3	1.9	2.3
500	CTB			1.7	1.6			1.5	1.5
	BTB	2.7	3.0			2.3	2.4	1.7	2.1
700	CTB			1.6	1.6			1.5	1.5
	BTB	3.1	3.1			2.2	2.4	1.7	1.8
900	CTB			1.6	1.6			1.5	1.5
	BTB	3.3	3.1			2.2	2.4	1.7	1.7
1,114	CTB			1.6	1.6			1.5	1.5
	BTB	3.4	3.1			2.2	2.4	1.7	1.7
Average	CTB			1.7	1.7	(1.65)	(1.65)	1.6	1.6
	BTB	3.0	3.1			2.3	2.4	2.0	2.1

[a] CTB = cement-treated base; BTB = bituminous-treated base.
[b] Because there was no stone base wedge section in Loop 5, the average equivalency for CTB (1.65) from Loops 4 and 6 was assumed to be correct for Loop 5 also, and this value was used for comparison with the BTB sections. Data for Loop 5 are, therefore, interpolations.
[c] For Loop 6, 4 in. of subbase was replaced by 3.5 in. of stone base for comparison purposes.

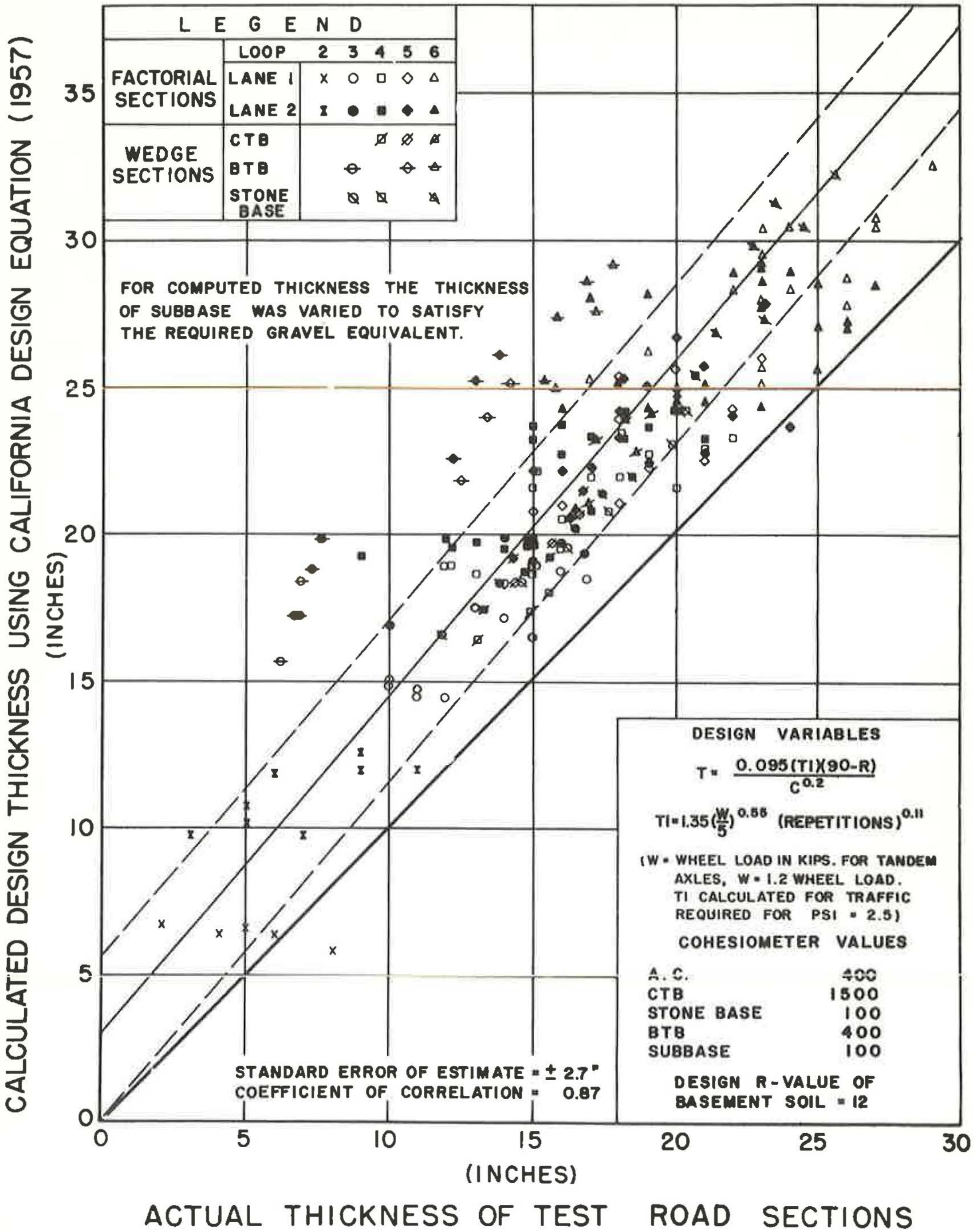


Figure 5. Thickness of test sections at AASHO Test Road vs calculated design thickness using California design equation (1957).

Table 1 would indicate that cement-treated bases have an equivalency of 1.65 in. of gravel to 1 in. of base. This agrees quite favorably with California experience in which a factor of 1.75 to 1 is currently being used.

From the information on bituminous bases, on the other hand, it is apparent that the magnitude of load has a marked effect on the equivalency of bituminous bases. It is suspected that there is also an effect due to depth of layer and the number of repetitions. However, in these latter two cases, it was not possible to isolate the variables by means of the information available. It is possible that one effect offsets the other.

A study of air temperature data at the AASHO test road and corresponding pavement temperature data indicated that an approximate average pavement temperature of about 72° would represent the over-all condition of the test road pavement. Cohesion (tensile strength) tests were made on AASHO pavement cores tested at various temperatures. The results are shown in Figure 6. At 72°, the cohesion value of the AASHO mix is 5,000 g per lineal inch. The recovered penetration of the asphalt in these cores was 37. (Cohesion test is performed by breaking a 2½- by 4-in. diameter test specimen by bending. Cohesion value equals the grams per lineal inch to break specimen when the load is applied on a 30-in. lever arm (6).)

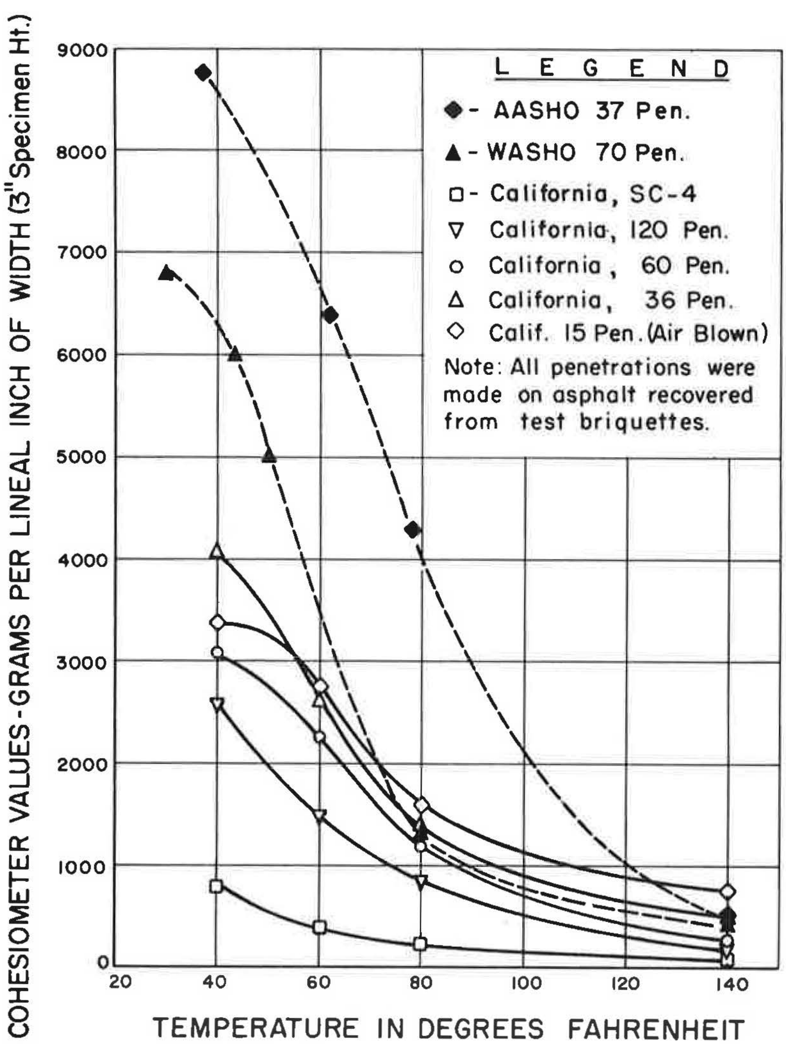


Figure 6. Cohesiometer values of bituminous pavement at various temperatures.

To compare a normal California mix using a good crushed California aggregate and asphalt manufactured on the Pacific Coast the remaining series of tests shown in Figure 6 were performed. For these California mixes, the cohesion at 72° would be only 2,000 g per lineal inch.

Most observers would agree that the equivalency of a rigid layer of material, in terms of inches of gravel, should be directly related to its tensile strength and its depth of section. A somewhat different situation exists in the case of bituminous layers, for in this case, strength is related not only to composition but also to temperature (Fig. 6). A bituminous mix varies in temperature from top to bottom, consequently there is a variation in that portion of its strength that is dependent on the viscosity of the asphalt binder.

To evaluate the property of cohesion, an empirical formula was developed to fit AASHO conditions:

$$C = \text{cohesion at } 72^{0} \left(\frac{8}{W+2}\right)^{2.5} \tag{6}$$

in which

C = equivalent cohesion; and
W = applied wheel load in kips (≥ 6).

Also, for gravel equivalency (GE),

$$GE = \left(\frac{C}{\text{cohesion of gravel}}\right)^{0.2} \tag{7}$$

Figure 6 indicated that mixes in themselves have widely divergent tensile strength characteristics; in ordinary highway design problems, an equivalency correction for wheel load would not be a simple matter because mixed traffic is involved and the weight of individual axles is rarely known, except on a statistical basis. However, assuming that lightly traveled roads will generally be designed for light loads, and heavy industrial roads will be subjected to heavy loads, a general relationship between equivalency and traffic index can be established.

Figure 7 is an empirical development from AASHO test road data which provides a means of adjusting equivalency for mixes that do not have the tensile strength characteristics of the AASHO asphalt concrete. These reductions in equivalency are necessary and need to be considered if flexible pavements are to be designed with the assurance of an adequate life. In California, therefore, it is proposed that a series of equivalencies be used that are based on the predicted traffic.

The proposed equivalencies taken from Figure 7 are given in Table 2. It covers a complete range of traffic currently using California streets and highways.

The coefficients in Table 2 would appear to challenge the validity of the coefficients D_1, D_2, and D_3 which were developed in the formula explaining the performance of the AASHO Road Test. These coefficients were obtained by statistical analysis of the factorial sections and most surely expressed what happened at the AASHO Road Test, yet there are the wedge sections and they, by this analysis at least, do not necessarily agree with the factorial sections. If the evidence reported by the British Road Test (8) that 6 in. of bituminous base is equivalent to 10 in. of gravel is added to this, as well as Nichols' report from Virginia (9) concerning distress of a number of asphalt base projects in which the total base and surfacing equaled 9 in., it would appear that there are other factors to consider before a single standardized ratio of equivalencies can be established for use under all conditions and all geographical areas. In Table 2 there is an attempt to indicate the ranges of equivalencies that might be encountered due to varying traffic conditions or varying quality of the asphalt concrete layer itself.

In 1957, the method for calculating traffic index in the California formula (4) was revised. The formula, based on test road data and experience available at that time, was

$$TI = 1.35 \left[\left(\frac{W}{5}\right)^{5} \text{repetitions} \right]^{0.113} \tag{8}$$

in which

TI = traffic index, a number directly proportional to the required thickness of structural section.

The AASHO Road Test data were reviewed to determine the validity of the exponents in the formula. The number of applications at present serviceability index (PSI) = 2.5 was plotted vs the gravel equivalent of the individual sections. The plots on log log paper yielded the slopes given in Table 3.

Table 3 shows that the use of different base materials results in different deterioration rates due to applications of a given load. However, the estimating of future traffic for purposes of design is, at best, only an approximation. Therefore, refinements in the exponent due to base type are not justified until methods of traffic prediction are greatly improved. To encompass all reasonable possibilities, it appears that the exponent of 0.119 would provide a reasonably satisfactory value.

Using the same procedure as the preceding, a tabulation was made for the same test sections in which curves of wheel load vs gravel equivalent were plotted for the indicated number of applications. The slopes are determined for the wheel load exponent. The tabulation is given in Table 4 and typical curves are shown in Figure 8.

In Table 4 the factorial sections were omitted because sufficient data were not available to interpolate exact thicknesses for given numbers of repetitions.

The average value of 0.48 is sufficiently close to a theoretical value of 0.50 to justify the use of the latter figure. Using the value of 0.50 the formula for thickness becomes

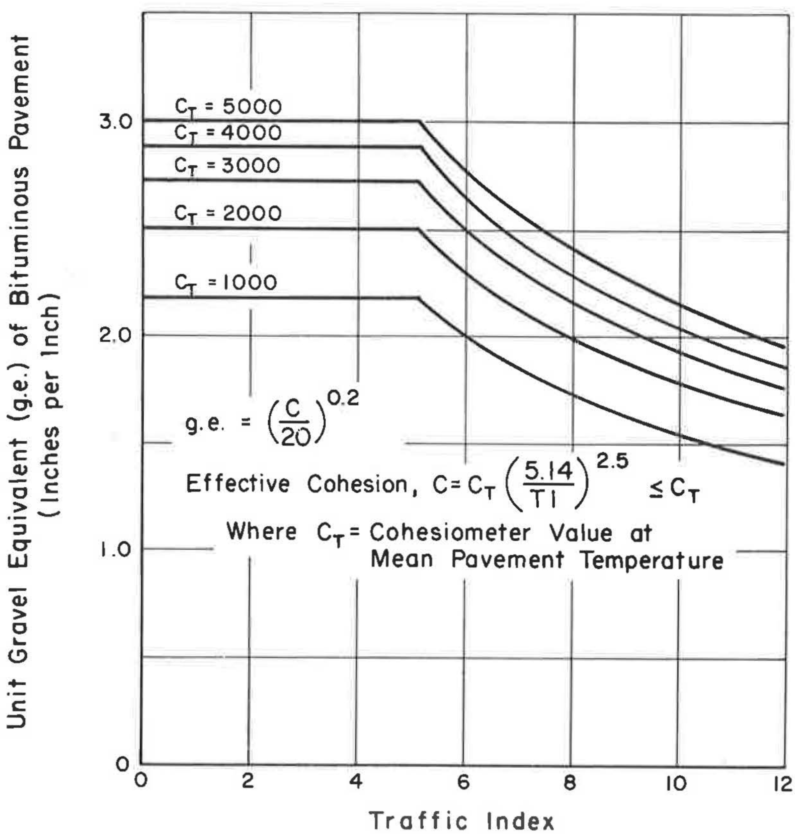


Figure 7. Gravel equivalent of bituminous pavement based on AASHO Test Road analysis.

TABLE 2

PROPOSED EQUIVALENCIES FOR BITUMINOUS MATERIALS SHOWING THICKNESS OF GRAVEL LAYER REQUIRED TO EQUAL 1 IN. OF ASPHALT CONCRETE PAVEMENT

Class of Road	Traffic Index Range	Gravel Equivalency (in.) AASHO Mat'l.	Calif. Mat'l.
Heavy industrial	12	2.0	1.6
	11	2.1	1.7
Heavy truck traffic	10	2.2	1.8
	9	2.3	1.9
Medium truck traffic	8	2.4	2.0
	7	2.6	2.1
Light truck traffic	6	2.8	2.3
Residential streets	5	3.0	2.5
	4	3.0	2.5

TABLE 3

SLOPE VALUES OF APPLICATION VS GRAVEL EQUIVALENT CURVES

Loop	Lane	Slope of Application vs Gravel Equivalent Curve: All Factorial Sections	BTB Wedge	CTB Wedge	Stone Wedge
3	1	0.118	0.088		0.137
	2		0.099		0.111
4	1	0.146		0.100	0.067
	2	0.141		0.127	0.064
5	1	0.093	0.100	0.082	
	2	0.103	0.080	0.103	
6	1	0.097	0.162	0.086	0.046
	2	0.090	0.161	0.099	0.044
Avg.		0.112	0.115	0.099	0.078

$$T = \text{constant } W^{0.50}\, r^{0.119} \qquad (9)$$

in which

T = thickness;
W = wheel load; and
r = repetitions.

From Eq. 9, wheel load constants may be calculated which may be applied to mixed traffic:

$$\frac{T_1}{T_2} = \left(\frac{W_1}{W_2}\right)^{0.50} \left(\frac{r_1}{r_2}\right)^{0.119} \qquad (10)$$

If $T_1 = T_2$; $W_1 = 5{,}000$ lb; and r_2 = one repetition of load W_2, then

$$r_1 = \left(\frac{W_2}{5}\right)^{4.2} \text{ equivalent 5-kip wheel loads (EWL)} \qquad (11)$$

TABLE 4

SLOPE VALUES OF WHEEL LOAD VS GRAVEL EQUIVALENT CURVES

Applications of Load	Slope of Wheel Load vs Gravel Equivalent Curve: BTB Wedge	CTB Wedge	Stone Wedge	All Wedges
100,000	0.504	0.411	0.563	
300,000	0.535	0.476	0.488	
500,000	0.595	0.431	0.455	
700,000	0.636	0.394	0.380	
900,000	0.653	0.349	0.349	
1,114,000	0.668	0.359	0.347	
Avg.	0.599	0.403	0.430	0.48

The constants are called EWL_{62} to differentiate from previous EWL calculations made by the California Division of Highways.

The details of using this method to obtain constants applicable to mixed traffic are outlined by Sherman (4). Briefly, the method consists of a statistical sample of traffic as weighed at various loadometer stations throughout the State. The development of the method is given in Table 5 where axle weights have been grouped together to show variations within classes of trucks (such as 2-, 3-, 4-, 5-, or 6-axle trucks. In the table, wheel load factors for the 3-, 4-, 5-, and 6-axle trucks show a variation within a given wheel load group. This is due to allowance for tandem effect. Based on road test data, a 10 percent effect was allowed for each pair of tandems included. The number of tandem vehicles for each class of truck is estimated, using tables published in House Document 91, 1st Session, 86th Congress. This document contains a large sample of truck combinations and loadings for various geographical areas of the United States. It contains sufficient information to establish the percentage of single- and tandem-axle combinations for each load group. These percentages were applied to the loadometer tables of the California Division of Highways to determine the average wheel load factor for each class of truck and for each loading.

Table 6 gives the totals arrived at in Table 5 and develops the EWL_{62} constants for computing average daily traffic.

Because California traffic counts are reported as the total vehicles in two directions, the truck constants developed in the last column of Table 6 are for these bidirectional counts. Further the constants in Table 6 are based on 1959 traffic, and any increase in allowable load limits will result in higher constants. These constants multiplied by the estimated number of trucks of each axle grouping will total to the design equivalent 5,000-lb wheel loads (EWL). Constants could also be determined quite readily for equivalent 9,000-lb wheel loads.

TABLE 5

CALCULATIONS TO DETERMINE YEARLY ADT CONSTANTS FOR TRUCK GROUPS BASED ON 1959 STATEWIDE LOADOMETER SURVEY[a]

		2-Axle Trucks			3-Axle Trucks			4-Axle Trucks			5-Axle Trucks			6-Axle Trucks		
Axle Group (kips)	Wheel Load (kips)	EWL Per Axle	No. Axles	EWL	EWL[b] Per Axle	No. Axles	EWL	EWL[b] Per Axle	No. Axles	EWL	EWL[b] Per Axle	No. Axles	EWL	EWL[b] Per Axle	No. Axles	EWL
2- 8	2	0.02	1,939	39	0.02	931	14	0.02	1,104	21	0.02	3,313	55	0.01	153	3
8- 9	4¼	0.51	115	59	0.45	241	108	0.48	108	51	0.39	859	331	0.44	31	13
9-10	4¾	0.81	77	63	0.72	212	153	0.73	90	65	0.62	492	302	0.73	36	26
10-11	5¼	1.23	64	79	1.08	157	170	1.10	53	58	0.93	253	235	1.10	27	29
11-12	5¾	1.80	49	88	1.58	105	165	1.51	54	82	1.37	261	357	1.55	19	29
12-13	6¼	2.54	45	114	2.25	76	212	2.16	50	108	1.99	290	578	2.22	11	25
13-14	6¾	3.52	34	120	3.16	71	224	3.00	56	168	2.93	409	1,198	3.07	23	70
14-15	7¼	4.75	28	134	4.24	105	444	4.05	64	259	3.97	515	2,041	4.15	20	83
15-16	7¾	6.3	28	177	5.6	114	641	5.3	55	290	5.2	667	3,494	5.5	12	66
16-17	8¼	8.2	15	123	7.3	66	483	6.9	53	365	6.8	675	4,611	7.2	8	57
17-18	8¾	10.5	29	305	9.4	38	358	8.8	39	342	8.7	615	5,362	9.1	6	55
18-19	9¼	13.2	15	198	11.9	16	190	11.2	26	290	11.0	276	3,039	11.5	1	12
19-20	9¾	16.5	3	50	14.8	1	15	13.9	9	126	13.8	40	551	—	—	—
20-22	10½	22.6	4	90	20.1	—	—	18.8	4	76	18.6	11	205	—	—	—
22-24	11½	33	1	33	32	—	—	28	1	28	27	9	245	—	—	—
24-26	12½	47	—	—	45	—	—	39	—	—	39	3	117	—	—	—
Total No. Axles			2,446			2,133			1,763			8,688			347	
Total EWL				1,672			3,177			2,329			22,721			468

[a]Percentage of single and tandem axles extracted from House Document No. 91, 86th Congress, 1st Session, March 2, 1959.
[b]Based on tandem effect (i.e., one tandem = one single 10 percent heavier than tandem wheel load).

TABLE 6

TABLE OF AVERAGE DAILY TRUCK CONSTANTS FOR VARIOUS CLASSES OF TRUCKS

No. of Axles Per Truck	Total No. Axles	Total EWL	EWL per Axle	EWL per Truck	EWL for 365 Days[a]	EWL/Year for One Truck in One Direction[b]
2	2,446	1,672	0.684	1.368	499	250
3	2,133	3,177	1.489	4.467	1,630	815
4	1,763	2,329	1.321	5.284	1,929	965
5	8,688	22,721	2.615	13.075	4,772	2,385
6	347	468	1.349	8.094	2,954	1,475

[a]Constants when traffic counts cover traffic in one direction only.
[b]Constants when traffic counts include bidirectional traffic.

The EWL may be converted to traffic index by

$$TI = 1.30\ (EWL_{62})^{0.119} \tag{12}$$

A typical traffic index calculation is shown in Appendix A.

Those who are familiar with and have used the California method previously will note a substantial reduction in the EWL constants. However, the relation between constants (i.e., the ratio of 2-axle to 5-axle or 3-axle to 6-axle vehicles) has not greatly changed. Also, for a given traffic situation the new EWL constants will result in virtually the same traffic index. For example, in Appendix A, the EWL_{57} would have a traffic index of 10.7, whereas the new 1962 constant will yield a traffic index of 10.9.

Having re-evaluated the various factors of the design formula in light of the AASHO data, it would appear the formula should be changed to read

$$\text{Thickness} = \frac{0.070\ (\text{traffic index})\ (100 - \text{resistance value})}{(\text{cohesion})^{0.2}} \tag{13}$$

Also in Appendix A is a typical example showing the pavement thickness calculation using the nomograph (Fig. 11) that solves the suggested new formula. This calculation illustrates how each layer may be evaluated, one on top of the other, to give the most economical thickness of cover material. Naturally, when applying this formula on a broad-scale highway system, some additional factors of safety may be allowed, especially when the traffic factor cannot be accurately estimated. It is, of course, uneconomical to change structural sections too often on a single project so that some "rounding off" in sections is needed. For these reasons, may States provide design standards for minimum thicknesses of pavement and base for certain traffic conditions and allow only the subbase layer to be varied. In the example shown in Appendix A, however, the thickness determined by formula is shown.

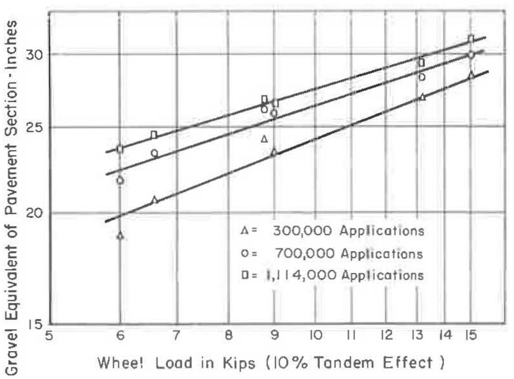


Figure 8. Log gravel equivalent of pavement section vs log wheel load.

By introducing an expression for an increased tensile strength allowance,

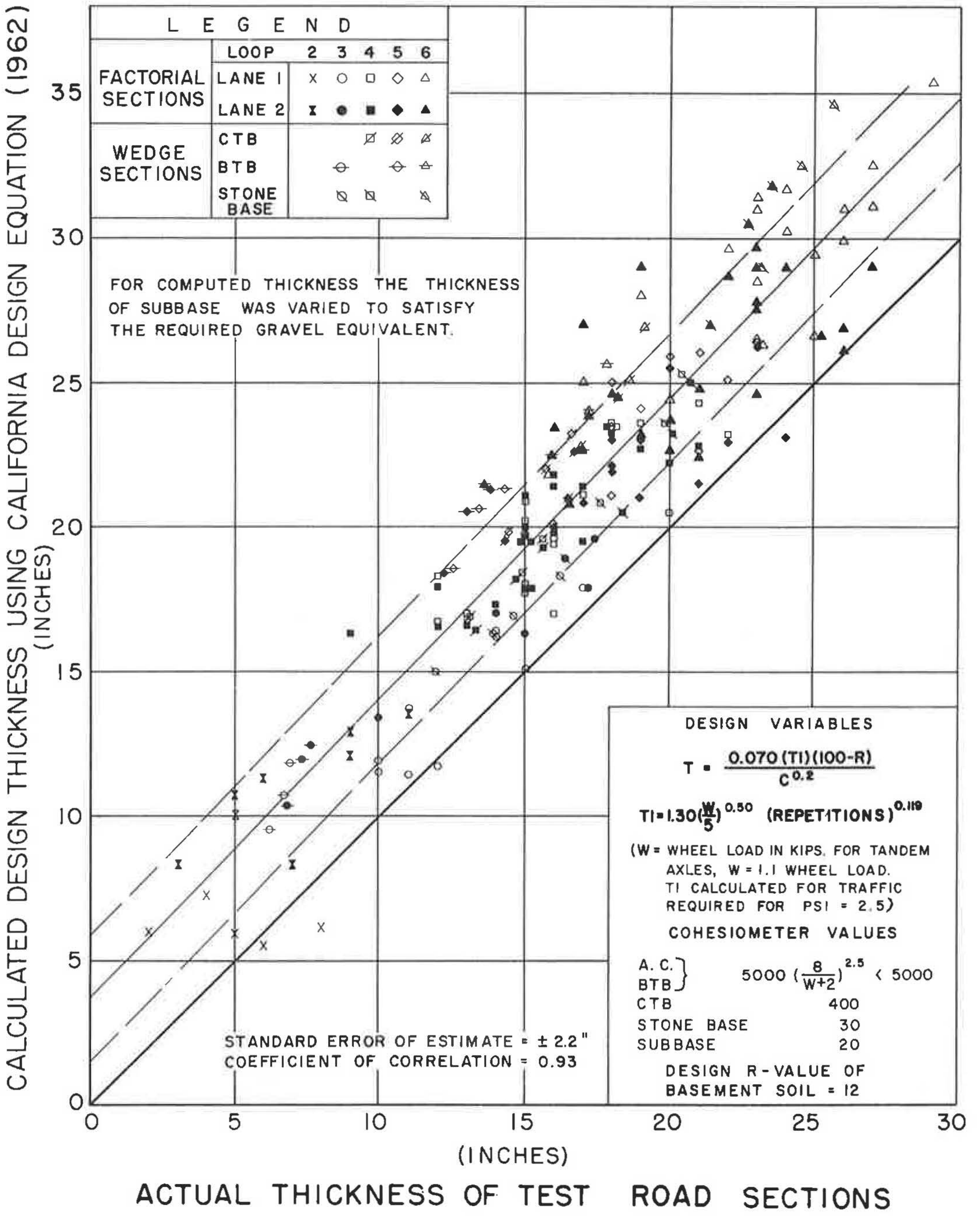


Figure 9. Thickness of test sections at AASHO Test Road vs calculated design thickness using California design equation (1962).

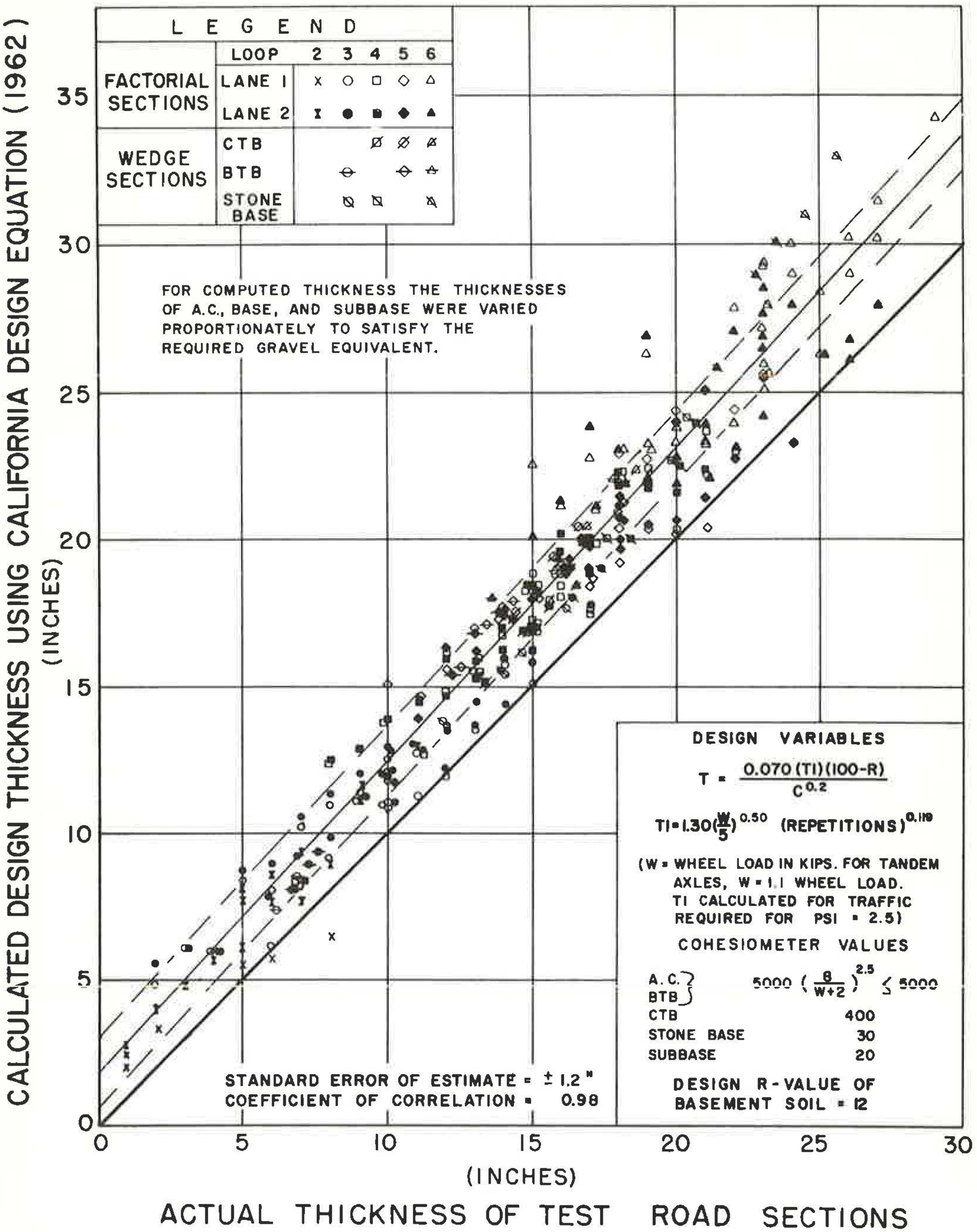


Figure 10. Thickness of test sections at AASHO Test Road vs calculated design thickness using California design equation (1962) (equivalent sections).

coupled with a readjustment of the load and repetition exponents, a better correlation with the test road data is obtained (Fig. 9). The improved correlation is measured numerically by the reduction in the standard error of estimate from ± 2.7 in. shown in Figure 5 to ± 2.2 in. and the increase in coefficient of correlation from 0.87 to 0.93.

Figures 5 and 9 contain the statement that the sections that failed during the first spring thaw are omitted. The reason for doing this was lack of time to study all of the sections on the flexible pavement portions of the Test Road, and because most of the highway mileage in California is in frost-free areas, an analysis was made first on those sections that survived the first spring thaw. These two charts (Fig. 5 and 9) report the results of this study.

Also, in Figure 9 all the error is being placed in the subbase layer. This gives a maximum error of estimate and a minimum coefficient of correlation when such things are evaluated in terms of thickness of section. The reason for this is obvious in that the error between actual and calculated thickness must be determined first in terms of gravel equivalent thickness, then converted to inches of surface, base, and subbase. Subbase, having the lowest equivalency, gives the greatest error. Surface material, having the highest equivalency, will give the lowest error.

An example of how the correlation factors might be changed is shown by Figure 10 which shows the data for all sections on the road test. This represents the same plot as that in Figure 9 except that the difference in gravel equivalent was prorated by thickness of layer to surface, base, and subbase. When this is done, the error of estimate ± 2.2 in. in Figure 9 becomes ± 1.2 in. and the coefficient of correlation raises to 0.98.

SUMMARY

Figures 9 and 10 serve to illustrate the influence of the method used to judge the efficiency of a design formula. These figures also show that the thicknesses computed by means of the California formula (based on measured properties of the basement soil, the subbase, base, and surface, also the effects of traffic expressed by the traffic index) are in nearly all cases equal to or greater than the thickness indicated in the serviceability index of 2.5 on the test road. A similar relationship could be shown for 2.0 or 1.5 serviceability index. This is the only relationship that can be justified, as a design formula should provide a structure stronger than any section known to fail. In other words, no portions are expected to show failure within the design life of the project. It may be argued that this provides too great a factor of safety and that the theoretical thickness, in many cases, would be excessive compared to the depths reported as just adequate on the test road. In judging the validity of a pavement design formula by comparing the calculated thickness with test road data, the following facts must be considered:

1. Every effort was made to secure a high degree of uniformity on the test road, and no such uniformity of performance can be expected on a highway constructed by ordinary methods.
2. Traffic was continued on the test road for a period of only two years. This means that the test road did not undergo the large number of cycles ranging from high to low temperature and from wet to dry which affects the performance of a highway over a period of many years.
3. The asphaltic pavements and bases on the test road were only two years old at the end of the test. Virtually all asphalts harden to some degree and become brittle with age. One could not assume an equally good performance over a long period of time on the average highway.

Taking these considerations into account, any design formula should be on the conservative side and provide some factor of safety over the thickness and strength of pavement which appeared to be barely adequate on the test road. The following are primary and important advantages of the California formula:

1. The California procedure utilizes numerical values derived from physical tests of the basement soil, the subbase, base and pavement.

2. The California method provides a logical means for converting miscellaneous traffic wheel loads to a single number—the traffic index. This number bears a direct linear relationship to the thickness of pavement structure required.

3. The California method has been in use for approximately 13 years and has demonstrated that it can accommodate wide variations in the type of soil, type of base, and type of pavement as well as variations in wheel loads and in the number of load repetitions.

ACKNOWLEDGMENTS

In addition to the many persons who have worked on the California pavement design formula for previous papers, the authors wish to acknowledge the help of several whose combined efforts made this paper possible. The review of the AASHO Road Test data was carried out by Parks M. Adams, David W. Eckhoff, Carl R. Sundquist, and Robert O. Watkins. Correlation tests were performed under the general direction of Ernest Zube. Clyde Gates, Daniel Howe, and Merle Nelson all contributed to the testing program and analysis of results.

The authors also wish to thank William H. Yttrup, Robert Souza, Agnes Lyon, Mary Martin, and their staffs, for the exacting work of preparing the figures, reproductions, and manuscript typing.

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Appendix A

TYPICAL EXAMPLE OF PAVEMENT THICKNESS DESIGN

Given the resistance value of a basement soil = 20, as measured by the Hveem stabilometer, cohesion of gravel = 20, cohesion of crushed stone base = 30, cohesion of asphalt concrete = 2,000, and the average daily truck traffic shown in Table 7, the number of trucks counted in each class is multiplied by the yearly EWL constants to determine the annual EWL.

TABLE 7

Truck Class by Axle	Average Daily No. of Trucks[a]	EWL (5,000-lb) Yearly Constants	Yearly EWL
2	679	250	169,750
3	344	815	280,360
4	295	965	284,675
5	1,539	2,385	3,670,515
6	113	1,475	166,675
Total			4,571,975

[a]Two-directional count.

Assuming that in 10 years the traffic will have increased 50 percent, the average annual design EWL is $\frac{1.0 + 1.5}{2}$ (4,571,975) = 5,715,000 EWL. The total design EWL for 10 years is 10 (5,715,000) = 57,150,000 EWL.

Traffic index (TI) is calculated from the EWL by Eq. 12:

$$TI = 1.30\,(EWL)^{0.119}$$

For the preceding example TI = 10.9; therefore, 11.0 should be used.

Pavement Thickness Calculation

The required gravel equivalent GE is determined by

$$GE = \frac{0.070\ (\text{traffic index})\ (100 - \text{resistance value})}{(\text{cohesiometer value of gravel})^{0.2}} \qquad (14)$$

For the example, GE = 33.8 in.

Surface Thickness

To determine the thickness of asphalt concrete required, the nomograph in Figure 11 is used. The California specifications require a crushed aggregate base to have an 80 R-value minimum. With a straightedge, Scale E is intersected at 80 R-value and Scale F at 11.0 traffic index. The intersection of this line with Scale G is the thickness of gravel equivalent required. Using this value of 8.5 in. gravel equivalent as a turning point, Scale H is intersected at the appropriate value of cohesion for the AC. This cohesion value is found from

$$C = C_T\left(\frac{5.14}{TI}\right)^{2.5} \leq C_T \qquad (15)$$

in which C_T = 2,000 and TI = 11.0; therefore, C = 300. The intersection of this line with Scale I gives 4.9 in. of asphalt concrete required. In design, 5 in. should be used.

Base Thickness

Using California Standard Specifications of 60 R-value minimum for subbase materials (this value can be and is modified in the Special Provisions to fit local aggregate conditions), Figure 11 shows a gravel equivalent of 16.9 in. needed over the subbase materials. Because the 5-in. AC is equivalent to 8.6 gravel equivalent inches, 8.3 in. remains to be satisfied by the base material. A cohesion of 30 for a good crushed rock product would indicate 7.5 in. to be satisfactory. Therefore, 8.0 in. of base material should be used.

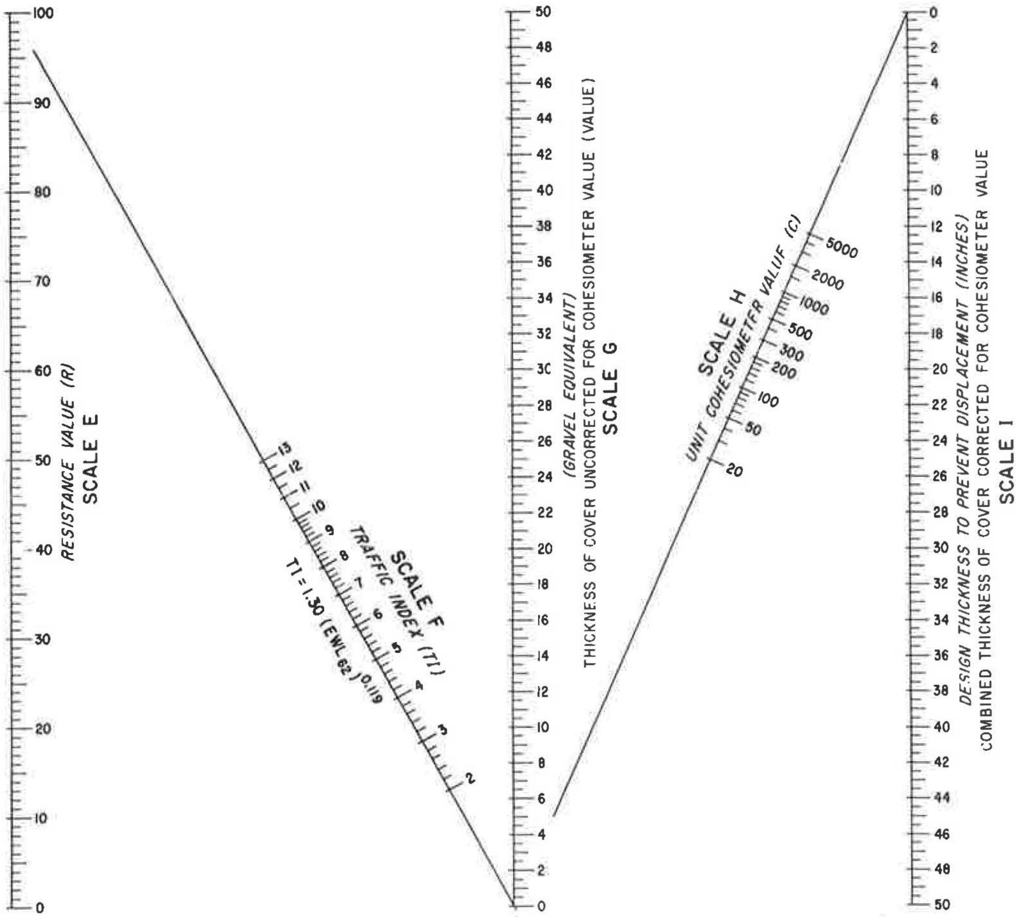

Figure 11. Nomograph for solving $T = \frac{0.070\ (TI)\ (100 - R)}{C^{0.2}}$

Subbase Thickness Design

Figure 11 also shows that a 20 R-value basement soil with TI = 11.0 requires a gravel equivalent of 33.8 in. The GE of surface and base is 5-in. surface = 8.6-in. GE, and 8-in. base = 8.8-in. GE; and the total GE = 17.4 in.

Required thickness of subbase is, therefore, 33.8 - 17.4 = 16.4 in. Thus, 17.0-in. subbase should be used.

The minimum allowable structural section over 20 R-value basement soil for very heavy truck traffic is 5-in. AC, 8-in. Class I aggregate base, and 17-in. 60 R-value subbase, for a total thickness of 30 in.

Various other structural sections that might also be found satisfactory for the preceding traffic and soil conditions would be 5-in. asphalt concrete, 8-in. cement-treated base, 11-in. subbase, for a total of 24 in.; and 5-in. asphalt concrete, 8-in. bituminous-treated base, 12-in. subbase for a total of 25 in.

Appendix B

DEFINITION OF STATISTICAL TERMS

Coefficient of Correlation

Linear correlation is used to determine whether a relationship exists between two variates. There may be a direct, an inverse, or no relationship between variates.

Pearson's coefficient of correlation for ungrouped data has theoretical limits of ±1. A value of r approaching +1 indicates a direct relationship between the variates, whereas a value approaching -1 indicates an inverse relationship. A value of r tending toward 0 indicates that no relationship exists between the variates.

$$r = \frac{\frac{\Sigma xy}{N} - \frac{\Sigma x \Sigma y}{N^2}}{\sigma_x \sigma_y} \tag{16}$$

in which

$$\sigma_x = \sqrt{\frac{\Sigma x^2}{N} - \frac{(\Sigma x)^2}{N^2}};$$

$$\sigma_y = \sqrt{\frac{\Sigma y^2}{N} - \frac{(\Sigma y)^2}{N^2}};$$

x = actual thickness (inches);
y = computed thickness (inches);
N = number of points;
σ = standard deviation; and
r = coefficient of correlation.

Line of Regression

If the plotted data indicate a linear relationship between the variates, then a straight line that best fits the data is called a line of regression. The general equation is expressed as y = mx + b and the values of m and b are found by using the method of least squares.

$$y = mx + b \tag{17}$$

in which

$$m = \frac{N\Sigma xy - \Sigma x \Sigma y}{N\Sigma x^2 - (\Sigma x)^2};$$

$$b = \frac{\Sigma y \Sigma x^2 - \Sigma x \Sigma xy}{N\Sigma x^2 - (\Sigma x)^2};$$

N = number of points;
y = computed thickness (inches);
x = actual thickness (inches);
m = slope; and
b = y-intercept.

Standard Error of Estimate

Standard error of estimate (7) measures the concentration of the points clustered about the line of regression. A zone drawn parallel to the line of regression on either side at a vertical distance S_y will include approximately 67 percent of the points. A vertical distance $2S_y$ will include approximately 95 percent of the points.

$$S_y = \sigma_y \sqrt{1-r^2} \tag{18}$$

in which

σ = standard deviation;
r = coefficient or correlation; and
S_y = standard error of estimate (inches).

Additional Flexible Pavement Design Paper

The Flexible Pavement Design Committee also sponsored a paper by B. B. Broms, "The Bearing Capacity of Flexible Pavements Subject to Frost Action." The subject matter of this paper is such that it was felt to be more appropriate that it be published in the issue of Highway Research Record dealing with Stresses in Soil and Layered Systems.