VOL. 32, NO. 6 FEBRUARY 1963

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Bayshore Freeway (Bypass 101) showing 8 traffic lanes between Daly City and San Francisco, California, looking northeast from Industrial Street, Interstate Route 5.

PUBLISHED BIMONTHLY BY THE BUREAU OF PUBLIC ROADS, U.S. DEPARTMENT OF COMMERCE, WASHINGTON

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Vol. 32, No. 6 *February 1963*

Published Bimonthly

Muriel P. Worth, *Editor*

THE BUREAU OF PUBLIC ROADS

WASHINGTON OFFICE
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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 6, 1961.

IN THIS ISSUE

U.S. DEPARTMENT OF COMMERCE
LUTHER H. HODGES, Secretary

BUREAU OF PUBLIC ROADS
REX M. WHITTON, Administrator

Structural Behavior of Flexible Pavement

Analysis of Rigid-Plate Bearing Tests on Full-Size Test Sections

Y THE PHYSICAL RESEARCH DIVISION
UREAU OF PUBLIC ROADS

By[1] ALVIN C. BENKELMAN[2] and STUART WILLIAMS,
Chief, Flexible Pavement Section

This article presents the first comprehensive analysis of the data collected during an investigation of the structural behavior of flexible pavement conducted during the 1945–1950 period at Hybla Valley, near Alexandria, Virginia. Other articles have been published on the objectives of the investigation and the procedures, techniques, and equipment employed. Also, the refined data have been published and portions of these data have been analyzed by others.

The investigation at Hybla Valley, initiated by the Bureau of Public Roads, is believed to have been the most comprehensive experiment of this type ever conducted. Although the original objectives of the study were not attained in their entirety, the project has stimulated further research related to flexible pavement behavior and design. The new testing procedures and instrumentation developed were subsequently used by State highway departments and staffs of the WASHO and AASHO Road Test projects.

The comprehensive analysis presented in this article is expected to stimulate further interest in the application of the Hybla Valley test data to the design of flexible pavements.

Introduction

N ANALYSIS of data obtained in a five-year investigation of the structural havior of flexible pavements is presented this article. During the 1945–1950 period, gid-plate bearing tests were conducted on ll-size test sections at Hybla Valley near exandria, Va., by the Bureau of Public oads with the cooperation of the Asphalt stitute and the Highway Research Board. evelopment of data on fundamental re-ions between load and the thickness of xible pavement structures was one of the imary objectives of this investigation.

Information relative to this study of flexible pavements has been published in articles appearing in previous issues of PUBLIC ROADS magazine (*1*, *2*)[3] and publications issued by the Highway Research Board (*3*). Objectives of the investigation and the design and construction of the test sections, and the development of testing apparatus, techniques, and procedures were set forth in references *1* and *2*. Collected data on test findings have been presented in HRB Special Report 46 (*3*), to encourage independent analyses. These data included comprehensive tabulations of most of the refined plate-bearing test data, detailed descriptions of the test procedures, and descriptive summaries of the data refinement processes. Portions of these data have been analyzed in other publications (*4*, *5*).

Many different procedures can be used for plate-bearing tests and the data obtained may be analyzed in several ways (*3*). Most of the data obtained from use of three different test procedures have been analyzed for this article. These analyses—based on selected vertical movement criteria—show principally the load-bearing capacity of different combinations of pavement structure components on a given subgrade soil and of the subgrade itself. The data included in HRB Special Report 46 that have been analyzed for this article have not been repeated here; however, references have been made to tabulations of pertinent data at the time of their specific analysis.

For convenience, this article includes summaries of previously published information regarding the test facilities, the laboratory and field testing of materials and soil, and the test procedures used in the Hybla Valley investigation.

Findings

Flexible pavements to be adequate for carrying prevailing traffic must be designed to act in an essentially elastic manner. To meet this requirement each component of the pavement must possess (1) sufficient inherent stability to resist distortion within itself and (2) the density necessary to resist consolidation. The findings from the analysis of test data from structural behavior of pavement should be helpful to those concerned with designing and building flexible pavements.

Data from a series of Repetitional Tests showed that the thickness of pavement structure required to support a unit load of 80 p.s.i. varies approximately as the total load to the 0.4 power. At this required thickness (3 inches of AC surface plus granular base in this series of tests) no detrimental or nonelastic movement of the subgrade soil occurred.

Some of the other more important findings are given in the following paragraphs.

- When the subgrade soil was tested by having the load applied directly through rigid plates, the same degree of elastic deflection was not developed in the subgrade soil as when the load was distributed to the subgrade through the overlying pavement structure.

[1] Presented at the International Conference on Structural sign of Asphalt Pavement, University of Michigan, Ann bor, Mich., August 1962.

[2] Mr. Benkelman is now retired. During the study sented in this article he was employed by the Bureau of blic Roads as a Highway Research Engineer and later ved as the Flexible Pavement Research Engineer for the .SHO Road Test.

[3] References indicated by italic numbers in parentheses have been listed on page 140.

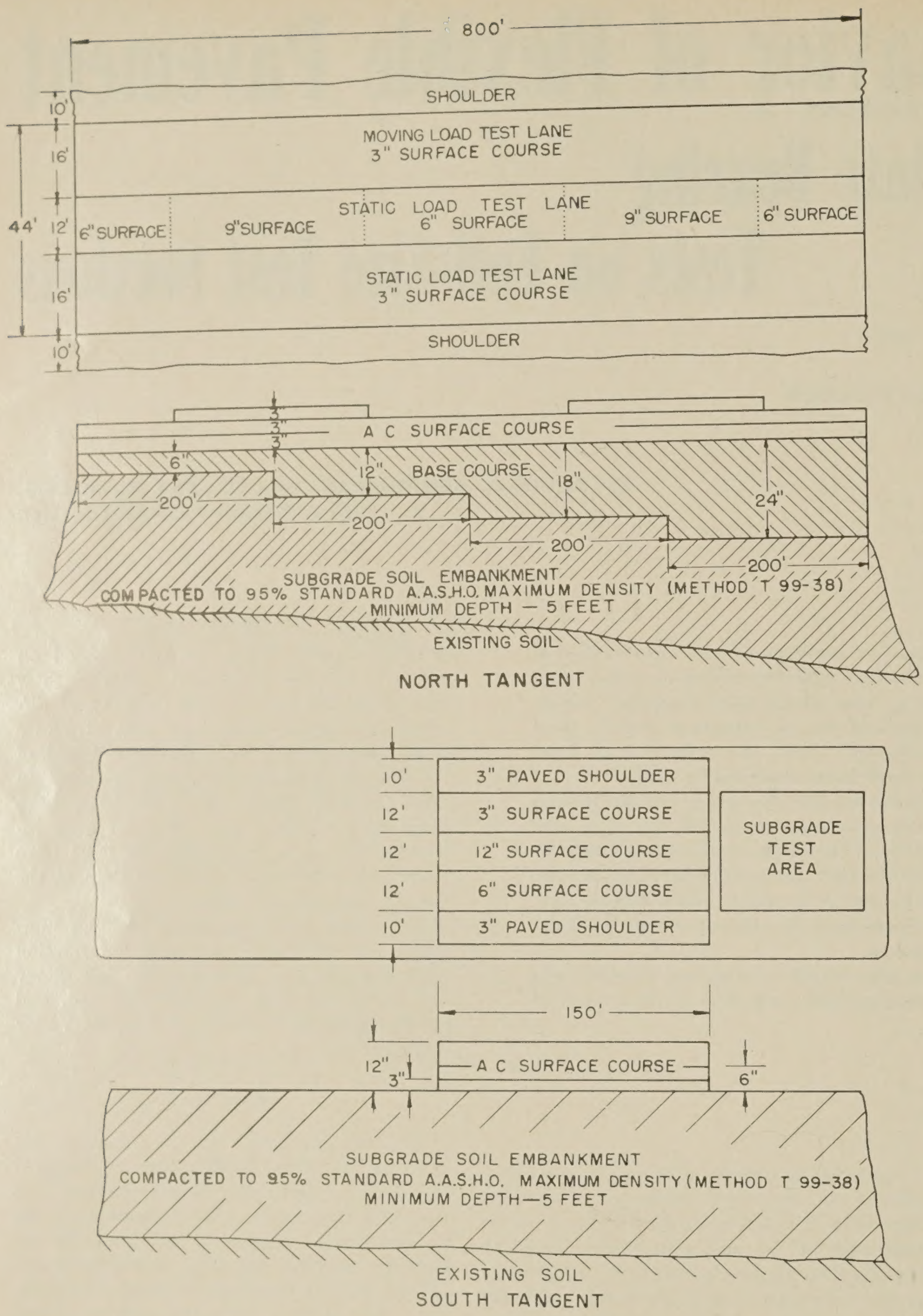


Figure 1.—Plan and profile of north and south tangents of test track.

• Test data showed a constant and orderly interrelation of the effect of unit load, diameter of test plate, and thickness of pavement structure.

• When average surface course temperatures exceeded 75° F., the granular base course appeared to be more effective in supporting load applied through a rigid plate than an equal thickness of AC only. But, when the temperatures of the surface course fell below 75° F., the AC surface was more effective than the granular base in supporting the load applied through a rigid plate.

• The ability of the pavement structure to support load was appreciably greater when the average pavement temperature was between 40° F. and 50° F. than when the pavement temperature was between 75° F. and 95° F.

• The surcharge provided by the AC surface course appeared to have had little effect upon the ability of the base course to support load applied through rigid plates. Likewise, the surcharge provided by the surface plus base course had little effect on the load-supporting capacity of the subgrade.

• The unit load supported by the subgrade soil at a given deflection decreased at a diminishing rate as the size of the loaded area was increased up to that of a bearing plate 84 inches in diameter—the maximum size of plate used in these tests.

Table 1.—Thickness of test sections

Test section	Thickness—Surface course	Base course	Total structure
NORTH TANGENT			
Number	*Inches*	*Inches*	*Inches*
1	3	6	9
2	3	12	15
3	3	18	21
4	3	24	27
5	6	6	12
6	6	12	18
7	6	18	24
8	6	24	30
9	9	6	15
10	9	12	21
11	9	18	27
12	9	24	33
SOUTH TANGENT			
13	3	0	3
14	6	0	6
15	12	0	12
16	0	0	0

Test Facilities

The tests on structural behavior of flexi[ble] pavements were conducted on two 800-f[oot] tangents of an oval test track. The plan a[nd] profile of the north and south tangents of t[he] test track are shown in figure 1. A tabulati[on] of the thicknesses of the test sections is giv[en] in table 1. The test sections located on t[he] north tangent consisted of either 3-, 6-, or [9-]inch thicknesses of bituminous concrete s[ur]faces each laid respectively on 6-, 12-, 18-, a[nd] 24-inch granular base courses that had be[en] constructed on a uniform clay-soil emba[nk]ment. The test sections on the south tang[ent] consisted of either 3-, 6-, or 12-inch bitumin[ous] concrete courses laid directly on the unifo[rm] clay-soil embankment, which was of the sa[me] type as the clay soil in the embankment [of] the north tangent. Tests were also m[ade] directly on the subgrade soil of the so[uth] tangent in an area reserved for these spe[cial] tests. Test sections were each a minimum [of] 100 feet in length and had a minimum wi[dth] of 12 feet.

The subgrade was constructed of a unif[orm] A-7-6 soil in 4-inch layers compacted t[o a] minimum of 95 percent of the maxim[um] density obtained by *Standard Laborat[ory] Method of Test for The Compaction [an]d Density of Soil, AASHO Designation: T 99-[38]*, in *Highway Materials*, Part II, 1947, [pp.] 212–213. The minimum height of emba[nk]ment was 5 feet. The base course wa[s a] uniform dense-graded mixture of soil, sa[nd], and gravel conforming closely to grading [ ]-1 of *Standard Specifications for Materials [fo]r Stabilized Base Course, AASHO Designati[on] M 56–42*, in *Highway Materials*, Part I, 1[9]7, pp. 35–36. It was constructed in 3-i[nch] compacted layers to a minimum, in-place [dr]y density of 136 pounds per cubic foot (p.c[f]).

The bituminous concrete surface course [wa]s a dense-graded, hot, plant-mix type confo[rm]ing to specifications of the Asphalt Insti[tute] (*6*). Mix No. IV was used for all 3-i[nch]

Table 2.—Summary of principal features of test procedures

	Accelerated Test—		Repetitional Test—		Incremental-Repetitional Test
	Part *a*	Part *b*	Part *a*	Part *b*	
Number of different loads	3		4	1	4
Successive deflections, approximately ..... inches	0.20 0.30 0.40	(1)	(1)		0.125 0.250 0.375 0.500
Load ..... p.s.i.	(1)	(1)	16 32 48 64	80	(1)
Number of load applications or release.	1	Continuous to end of test.	1	75	5
Duration of each load application or release.	Until movement decreased to 0.001 inch per 15 sec.		Until movement decreased to 0.001 inch per 15 sec.	1 minute	Until rate of movement decreased to 0.001 inch per minute.
Plate diameter ..... inches	12 18 24 30	12 18 24 30	6 to 42 in 3-inch increments.	6 to 42 in 3-inch increments.	12 18 24 30

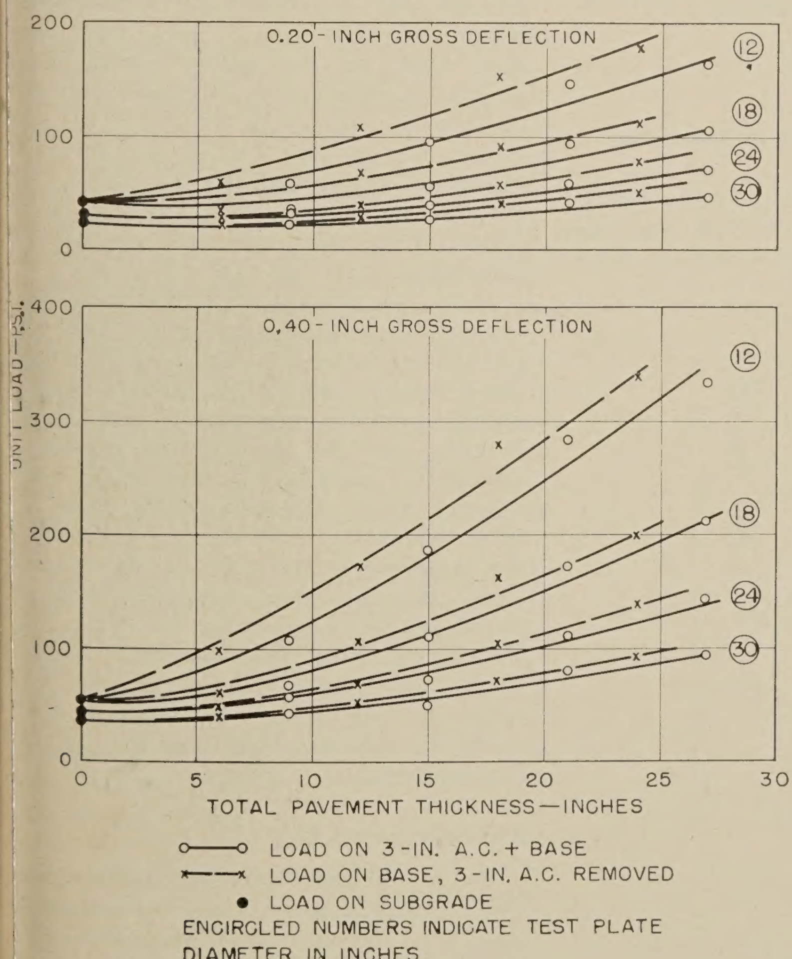
[1] Depended upon test conditions.

urface courses and for the top 3 inches of the -, 9-, and 12-inch surface courses. Mix No. I was used for the lower portions of the 6-, -, and 12-inch surface courses. The test ections were built with great care, extreme recautions being taken to ensure uniformity n composition and compaction of the subgrade, and in composition, compaction, and thickness of the pavement structure components. Detailed descriptions of the test facilities—including the layout of test sections—and materials and construction methods used are contained in previous publications (*1*, *2*, and *3*).

Laboratory and Field Testing of Materials

In connection with the loading tests, representative samples of the pavement structure components and the subgrade were obtained periodically from most of the test sections for laboratory analysis. In addition, in-place density and moisture determinations were made of the base course, and undisturbed cores of the subgrade were obtained for density and moisture content determinations.

Results of preliminary tests emphasized the importance of the effect of temperature of the bituminous (AC) surface course at the time of testing on the ability of the different sections to support load. Consequently, thermocouples were installed at several different depths in the pavement structure of representative test sections, and temperature measurements were recorded routinely. Precise control of the temperature of the bituminous surface courses during a series of loading tests was of course impossible. However, by limiting the testing season from mid-May to mid-September of each year and by shading the pavement in the vicinity of the test site, the average temperature of the bituminous surface could be controlled within what seemed to be reasonable limits of about 75° F. and 95° F. This procedure was followed for all tests in which it was desired



igure 2.—Load-thickness curves of the pavement and base course based on gross deflection of the medium tested—3-inch AC surface sections.

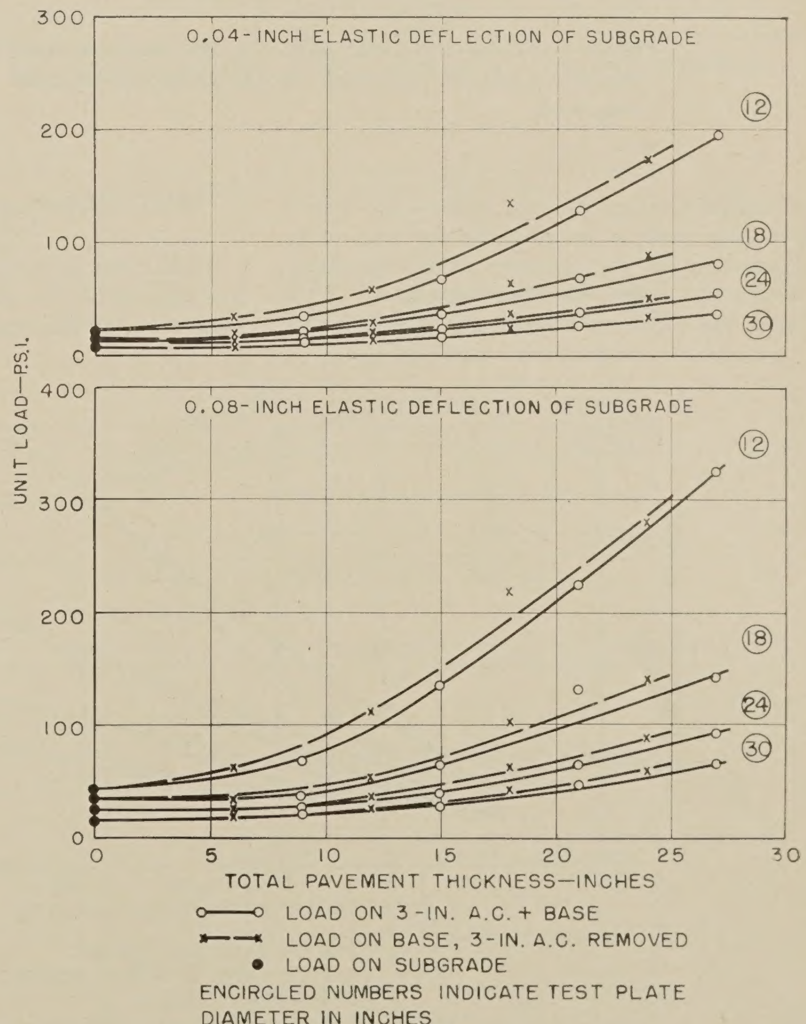


Figure 3.—Load-thickness curves of the pavement and base course based on elastic deflection of the subgrade—3-inch AC surface sections.

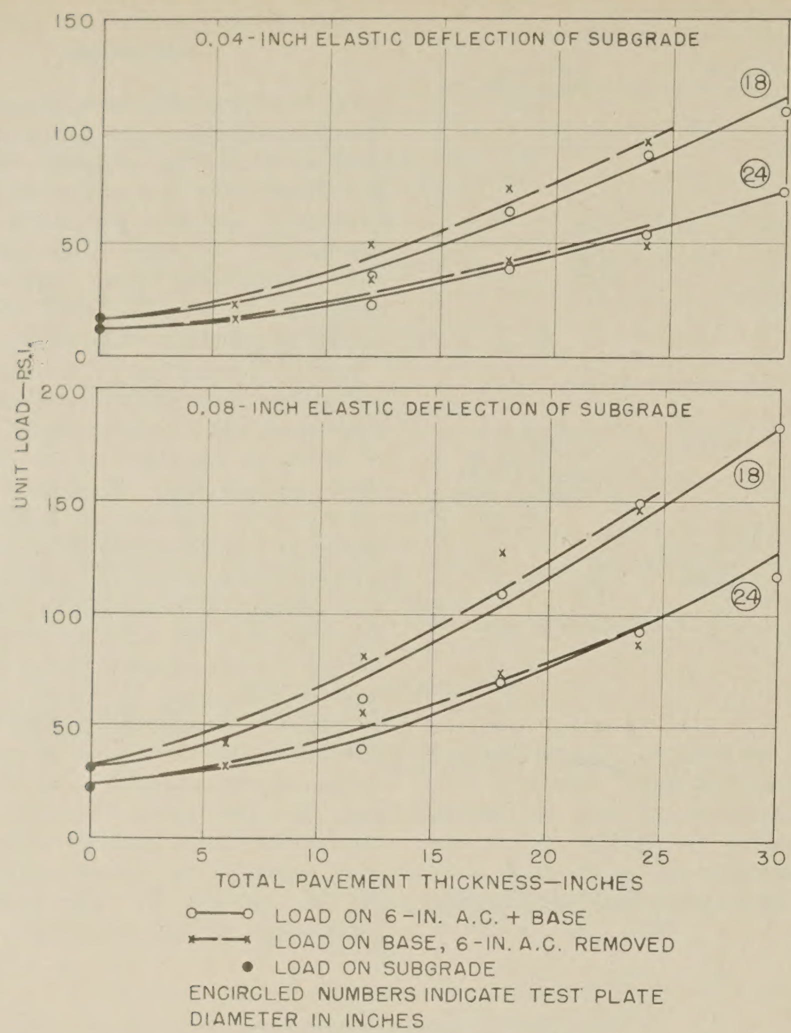


Figure 4.—Load-thickness curves of the pavement and base course based on elastic deflection of the subgrade—6-inch AC surface sections.

to control the temperature within these limits. For tests made on the granular base and on the subgrade, the temperature of the materials within the range obtaining during any of the tests was not considered a significant factor.

Plate-Bearing Test Procedures

Definitions of pertinent terms and a detailed description of the test procedures used have been reported previously (*1*, *2*, and *3*). However, to facilitate comprehension of this article, a summarization has been included. The following listed definitions apply.

Pavement.—The entire structure including the surface course, and the base course when present. Movements of the pavement included those occurring in the subgrade.

Surface course.—The asphaltic concrete (AC) course that rested on the base course or on the subgrade.

Base course.—The granular aggregate course that rested on the subgrade.

Subgrade.—The soil embankment underlying the base or surface course.

Gross deflection.—The total vertical movement, both elastic and inelastic, of the pavement, of the base plus subgrade, or of the subgrade only caused by the application of a single load or more than one load.

Settlement.—The permanent or inelastic vertical displacement caused by the application of a single load or the cumulative permanent displacement caused by the application of more than one load.

Elastic deflection.—The portion of the gross deflection of the pavement, the base plus subgrade, or the subgrade only that was recovered upon release of the load.

Early in the investigation it was realized that the data could be greatly augmented and the analysis enhanced or broadened by obtaining measurements of the vertical movements, not only on the surface being loaded but also simultaneously at other levels in the pavement structure. After considerable experimental work, a reliable and accurate method was developed (*3*). Consequently for all the tests reported here, when tests were made on the surface course simultaneous measurements of movement were made at the surface, at the top of the base course, and at the top of the subgrade. When tests were made on the base course simultaneous measurements were made on the top of the base course and the top of the subgrade.

At the time of this investigation no standard or generally accepted procedure for conducting rigid-plate bearing tests had been developed. Consequently, one of the objectives of a rather elaborate preliminary field testing program was the development of such a procedure. Most of the data obtained and reported herein were obtained by either one or the other of two newly developed procedures, the Accelerated Test and the Repetitional Test. However, some additional data obtained by a third procedure, the Incremental Repetitional Test, also have been included. The principal features of each of these tests are shown in table 2. The standards developed for conducting these tests were meticulously adhered to.

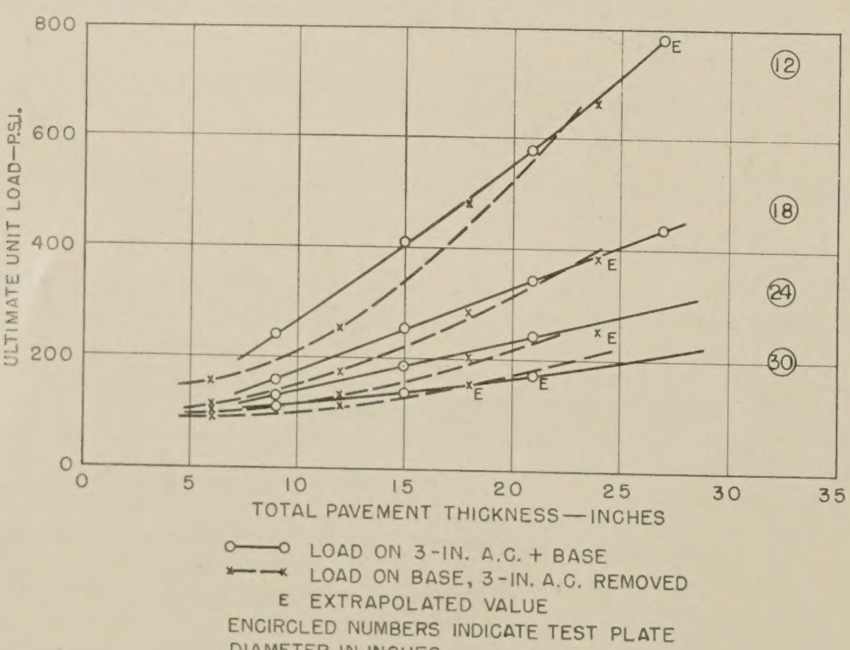


Figure 5.—Ultimate load-thickness curves of the pavement and base course—3-inch AC surface sections.

The Accelerated Test

The Accelerated Test consisted of two parts, the incremental (part *a*) and the accelerated (part *b*). The incremental part provided for the application and release, once each, of three individual loads of increasing magnitude. The period of application or release was maintained until the rate of movement slowed to 0.001 of an inch in 15 seconds. The magnitudes of the loads were selected by estimation so as to produce gross deflections of approximately 0.20, 0.30, and 0.40 of an inch for each of the three loads, respectively. Since the loads were necessarily estimated, it was intended that the gross deflections mentioned should be considered merely goals that were sought rather than specific gross deflections that were to be attained.

After release of the third load and after the movement-time criterion of the incremental part of the test had been satisfied, the accelerated part of the test was begun. It consisted of the continuous application of a load of increasing magnitude, the rate of load application being controlled so as to produce a rate of vertical movement of the surface under test of 0.5 of an inch per minute. The application of the load was continued until: (1) the material was unable to support a further increase in the load, (2) the gross deflection exceeded 2.0 inches, or (3) the total reaction load had been utilized. In nearly all of the tests, condition (1) was reached first; the maximum load applied to reach this condition was called the ultimate load. The rate of load application in the incremental part of the Accelerated Test was comparatively high, the total elapsed time required to cause the selected maximum gross deflection of 2.0 inches being only 4 minutes. Also, the maximum gross deflection in the incremental part of this test was much greater than in any of the other types of tests.

The Repetitional Test

The Repetitional Test was developed in an effort to determine whether the elastic action of a flexible pavement structure might serve as an acceptable criterion of its load-supporting capacity. Like the Accelerated Test, it consisted of two parts, part *a* being similar to the first part (incremental) of the Accelerated Test. In part *a* of the Repetitional Test, 16-, 32-, 48-, and 64-p.s.i. loads were applied and released once each, and the resultant vertical movements were measured. The movement-time criterion was the same as that of the Accelerated Test. Part *b* required the application and release 75 times of a constant unit load of 80 p.s.i. For each repetition, the load was applied and removed alternately for 1-minute periods of time.

The principal objective of the Repetitional Test was to determine the magnitude of total load for a given unit load or contact pressure under which the subgrade would act in an essentially elastic manner when the load was applied repetitiously to the surface. Since it was desired that the magnitude of the unit test load approximate the tire contact pressures of the heavier commercial vehicles, a unit load of 80 p.s.i. was used as standard. Thus the magnitude of the total applied load varied directly as the area of the bearing plate. Plates ranging from 6 to 42 inches in diameter in 3-inch increments were used on the different test sections. The range in total applied load was, therefore, from 2.26 to 110.8 thousand pounds deadweight load (kip).

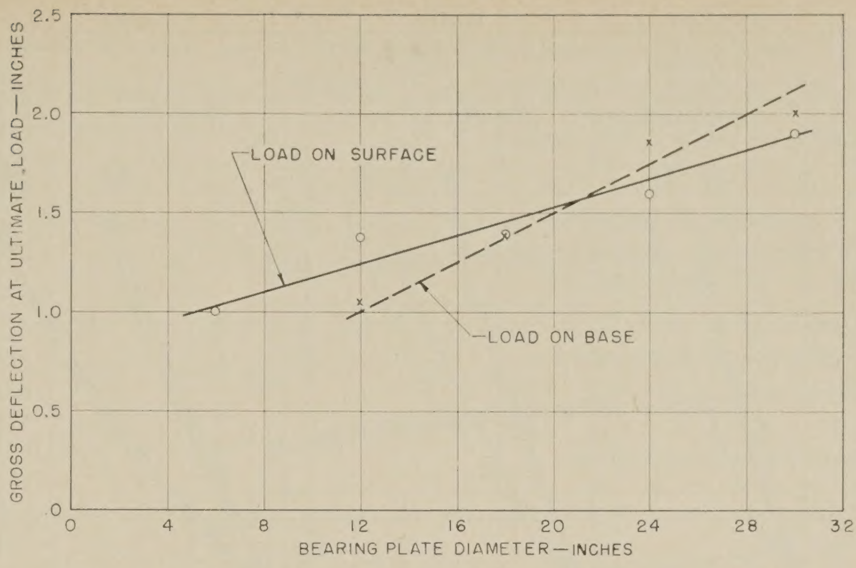


Figure 6.—Deflection-plate diameter curves at ultimate load.

The Incremental-Repetitional Test

The Incremental-Repetitional Test provided for the application of four loads of increasing magnitude, each of which was applied and released five times. Each load was maintained until the rate of movement of the surface under test slowed to 0.001 of an inch per minute. After release of the load, it was not reapplied until the rate of recovery had diminished to the same rate of movement. The loads were selected by estimation so as to cause gross deflections of about 0.125, 0.250, 0.375, and 0.500 of an inch prior to release of the fifth application of each load; as for the incremental part of the Accelerated Test, the gross deflections specified were intended to be considered as guides only.

Analysis of Accelerated Test Data

The data for the Accelerated Tests on the surface and base courses of test sections 1 to 12 inclusive are shown in table 4 and corresponding data for tests on the subgrade are shown in table 5 of reference (*3*). Figures 2 to 4 inclusive show relations between unit load and total pavement structure thickness for the various movement criteria for each of four bearing plate sizes. The data in these figures

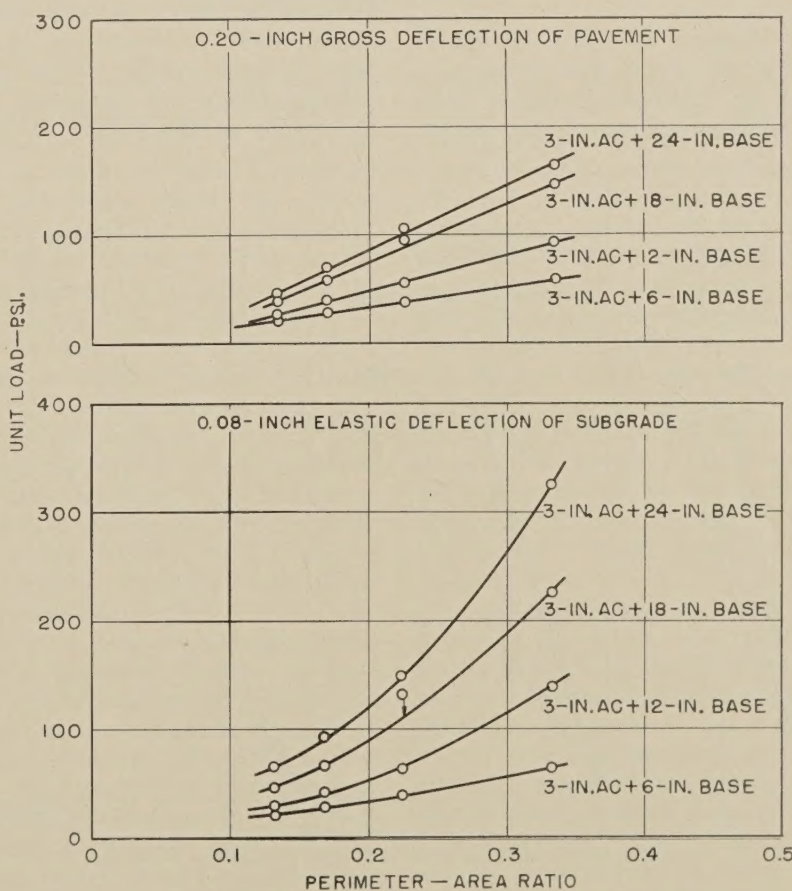


Figure 7.—Load-perimeter area ratio curves based on gross deflection of the pavement and on elastic deflection of the subgrade.

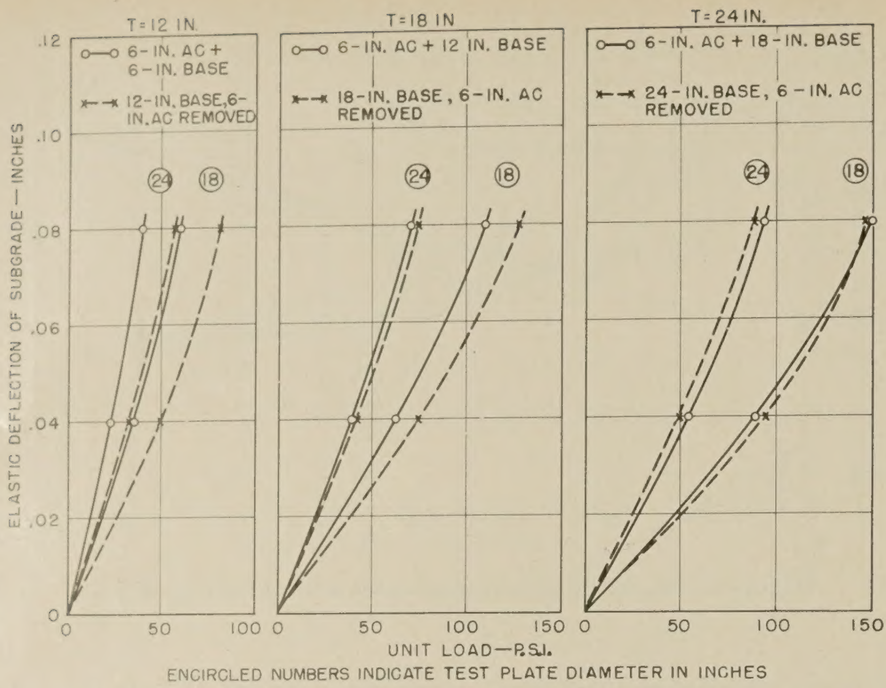


Figure 8.—Load-elastic deflection curves for equal thicknesses of pavement composed of either 6-inch AC surface plus base course or the base course alone.

were obtained from the incremental part of the test. The load data plotted in figure 2 are for gross deflections of 0.20 and 0.40 of an inch of the tested medium (pavement, base, or subgrade) and those in figures 3 and 4 are for elastic deflections of the subgrade of 0.04 and 0.08 of an inch. The gross deflections are given in column 4 in table 4, and in column 3 in table 5 of reference (*3*). The elastic deflections are tabulated in columns 9 and 7 in tables 4 and 5 respectively of the same publication.

In figures 2 through 4, load curves are shown for the granular base course alone acting as a structure, and for pavement structure composed of a bituminous concrete surface course plus the base. For test results shown in figures 2 and 3, the thickness of the surface course was 3 inches and for those in figure 4 it was 6 inches. The notation in the figures, "Load on base, 3-inch AC removed," refers to data for tests made on the base course after removal of the overlying surface course.

In each of the figures, the data plotted on the ordinate (zero thickness of pavement structure) represent those for tests made directly on top of the subgrade. The curves fitted to the plotted data for each of the test plates might be considered to originate at these subgrade points. These curves for the different plates and movement criteria show the extent of the ability of the pavement structure to support load as the thickness of the structure is increased. In general, the curves are parabolic in character and fit the data reasonably well.

With reference to test results depicted in figures 2 and 3, the following comments are made.

- The effect of thickness of pavement was orderly and consistent. The plotted points define lines that are slightly curved or parabolic and indicate that the unit load supported by a given plate varied as an exponential function of the thickness. Little difference exists in the shape of the curves for the 3-inch AC surface plus base and their shape for an equal thickness of base alone.
- Although the slopes of the curves for the smaller plates are steeper than those for the larger plates, little difference, generally, occurred with the increase in unit load on a percentage basis for any of the plates when computations were made for the same range in thickness.
- The data indicated that, for any given total thickness, the base course as a structure was somewhat more effective in supporting load than an equal thickness of combined surface and base course. This was true for all criteria employed to express load support. However, the difference in indicated effectiveness of the two combinations of structure to support load decreased appreciably as the size of the test plate was increased. In fact, for elastic deflections of the subgrade of 0.04 and 0.08 of an inch, as shown in figure 3, the relations of load to thickness for the plate 30 inches in diameter were practically the same, regardless of the makeup of the structure.

Curves similar to those shown in figures 2 and 3 can also be plotted for a given elastic deflection of the pavement structure. The comments in the three preceding paragraphs also applied when the elastic deflection of the pavement structure was used as a criterion.

The curves presented in figure 4 are similar to those shown in figure 3, except that they apply to the 6-inch AC surface plus base and to the base course after removal of the AC surface. The curves shown are only for the 18- and 24-inch plates. The indications of these data are much the same as those for the 3-inch AC surface plus base and the base alone; that is, the base course alone was more effective, generally, in supporting load than an equal thickness of surface and base course combined, but the difference was appreciably less pronounced. Similar curves showing the same trends can be developed for the pavement consisting of a 9-inch AC surface plus base.

The data for figure 5 were obtained by part *b* of the Accelerated Test procedure and are tabulated in column 7 of table 4, reference (*3*). The load was applied continuously at a comparatively rapid rate and the gross deflection at the end of the test was much greater than that obtained in the other test procedures. In figure 5, the ultimate unit load data for each of the sections consisting of 3 inches of AC surface plus base and for the base course under this surface have been plotted against the total thickness of the structure. Th

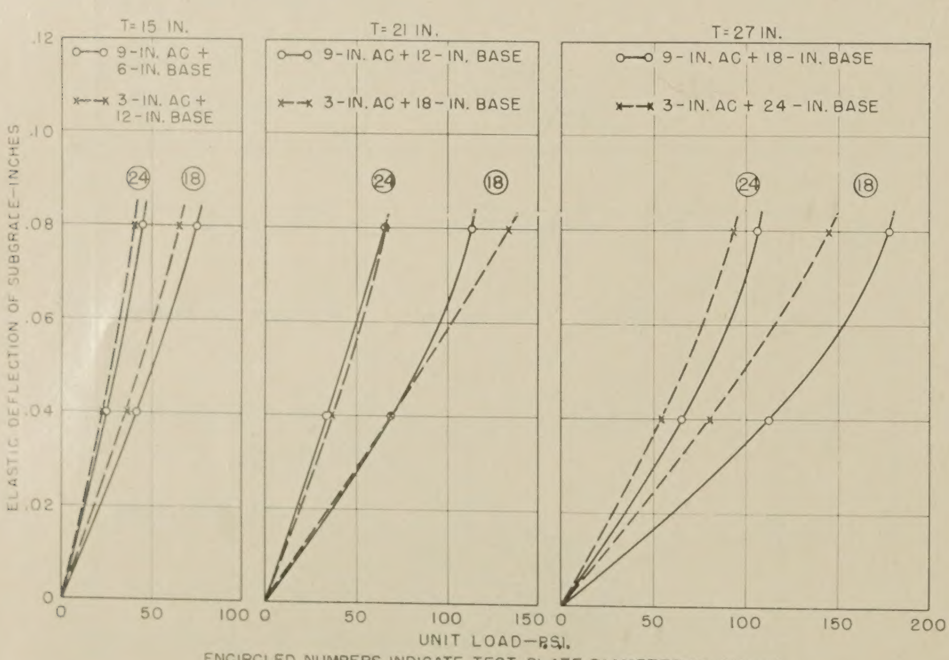


Figure 9.—Load-elastic deflection curves for equal thicknesses of pavement composed of either 9-inch AC surface plus base course or 3-inch AC surface plus base course.

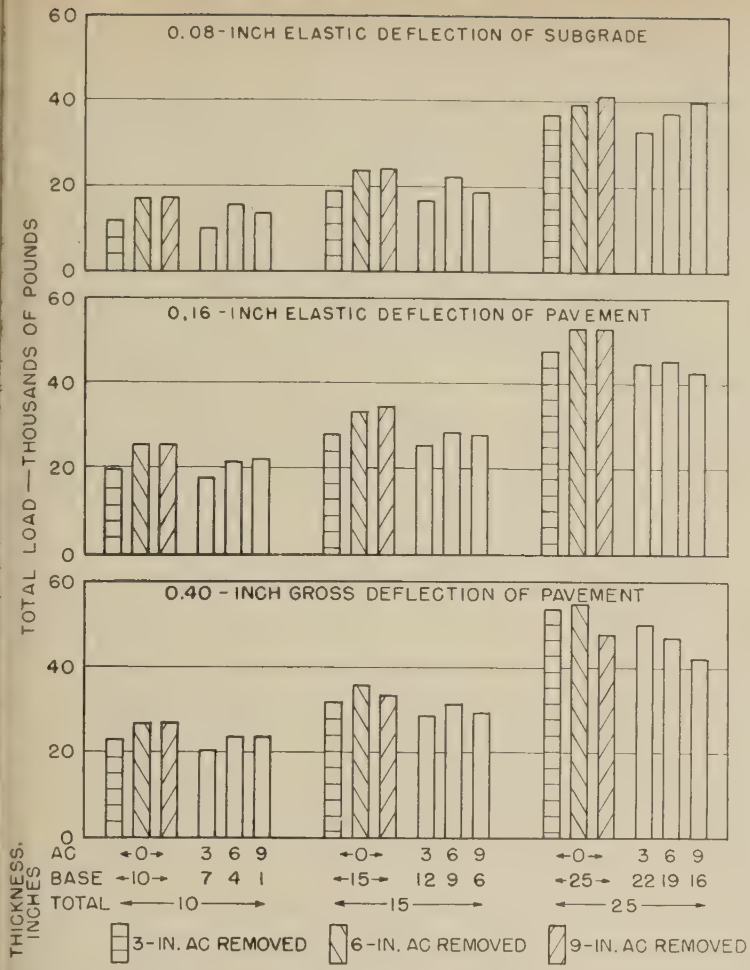


Figure 10.—Summary of Accelerated Test data obtained in tests made with the bearing plate 18 inches in diameter.

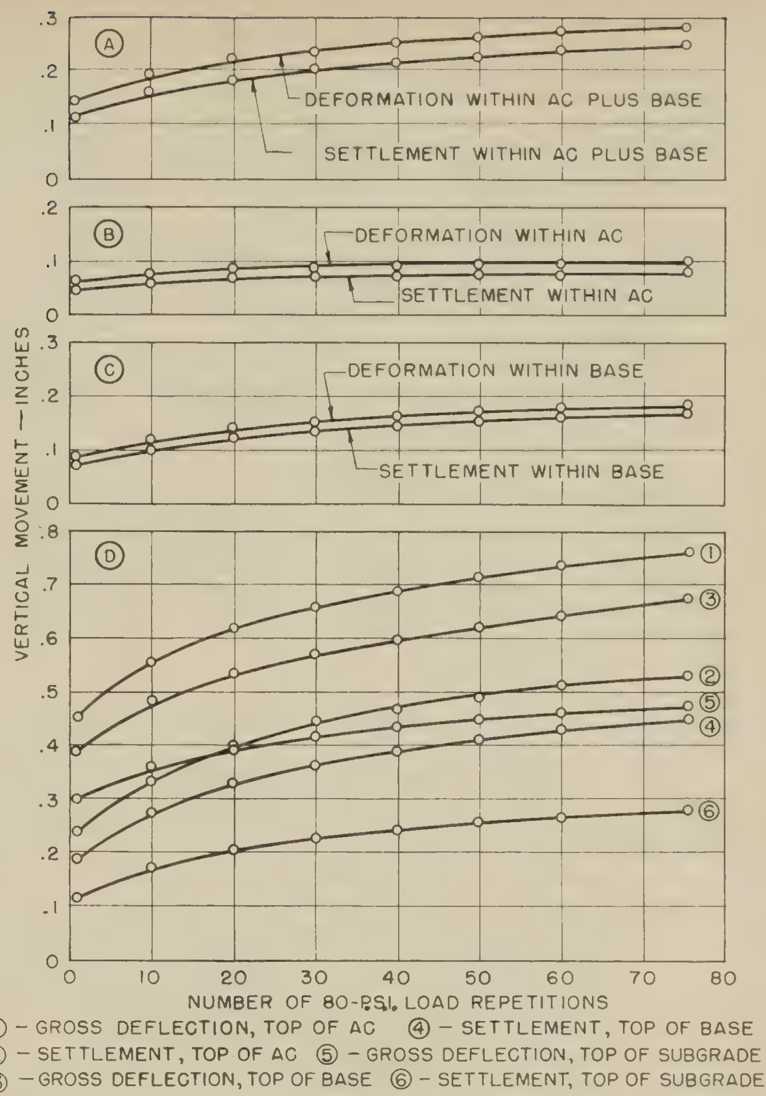


Figure 11.—Typical Repetitional Test data for tests in which progressive vertical movement of all pavement components and of the subgrade occurred.

extrapolated data were computed for either the condition in which the available reaction load or the range of the micrometer dials (2.0 inches) was exceeded. The general characteristics of the relations shown in figure 5 are much the same as the curves in figures 2, 3, and 4. However, for total pavement thicknesses of up to about 22 inches, the combination of surface plus base was more effective than the granular base course alone in resisting failure or in postponing the breakdown in resistance.

Column 7 of table 4 in reference (*3*) lists the gross deflections of the pavement structures (surface, base, and subgrade) at the ultimate (maximum) loads. These deflections were recorded when the surface failed in shear around the perimeter of the bearing plate or when the resistance of the material was overcome. These deflections have been plotted in figure 6 as a function of the diameter of the bearing plate. They represent the averages respectively for all the tests made with the 12-, 18-, 24-, and 30-inch diameter plates on all of the pavement sections. Some special tests were made on the surface of the pavement in which a plate 6 inches in diameter was used, and the average gross deflections obtained from these tests were also plotted. The linear curves fitted to the plotted points are reasonably well defined. They indicate that for the smaller plates the gross deflection at ultimate load for the AC surface plus base was greater than that for the base course alone; whereas, for the larger plates, the reverse was true. The range in gross deflections at ultimate load for loading either on the surface or base was from about 1 to 2 inches for the range in plate sizes used. Usually, the AC surface ruptured or failed in shear only when the 6- and 12-inch diameter plates were used in the test. Visual observations and a few radial deflection measurements showed that the curvature of the surface was much flatter when the test was made on the larger plates. This difference in curvature was probably responsible for the marked failure of the surface in the tests in which the smaller bearing plates were used.

Some of the data included in figures 2 and 3 was replotted in figure 7 in order to show more clearly the effect that the size of the bearing plate had on structural strength of the pavement. In figure 7, the unit load was plotted as a function of the perimeter-area ratio of the bearing plate, a ratio that increased with a decrease in diameter of the bearing plate. The upper part of the figure shows the curves for a gross pavement deflection of 0.20 of an inch, and the curves shown in the lower part of the figure are for an 0.08-inch elastic deflection of the subgrade. A consistent and well-defined effect of size of the bearing plate is shown by these test data. They indicated that the ability of the structure to support a given unit load diminished as the thickness of the pavement was decreased and the size of the bearing plate was increased. This effect has been demonstrated in other studies and is consistent with theories that are in use. The curves based on gross deflection were linear and those based on elastic deflection tended to be curvilinear in shape. The deviation from linearity became more pronounced as the overall thickness of pavement structure was increased and as the diameter of the bearing plate was decreased.

In figure 8, test data show the relations between unit load and elastic deflection of the subgrade for tests made with two sizes of plates on pavement structures that had overall thicknesses respectively of 12, 18, and 24 inches. For one series of tests the pavement structure consisted of a 6-inch AC surface plus base and for another the structure was the base alone after the 6-inch surface course directly beneath the test plate had been removed. In figure 9, similar relations are shown for tests made with two sizes of plates

on pavement structures having overall thic
nesses respectively of 15, 21, and 27 inch
For one series of tests the pavement structu
consisted of a 9-inch AC surface plus base a
for another, the structure consisted of a 3-in
AC surface plus base. The data shown
these figures are tabulated in column 9
table 4 of reference (3). The curves sho
in figure 8 are in agreement with those sho
in figures 2–4 in that the base course alo
tended to protect the subgrade from mo
ment more effectively than an equal thic
ness of 6-inch AC surface plus base. For te
on the 12-inch thickness of structure, and f
those made with the 18-inch plate on t
18-inch structure, the difference in unit lo
relations for tests on the AC surface pl
base and on the base course alone was a
preciable, otherwise, differences were i
significant.

Data for the 15- and 27-inch thicknesses
pavement, in figure 9, show that the co
bination of 9-inch AC surface plus base c
sistently supported more load for a giv
subgrade deflection than the 3-inch AC surfa
plus base. For the 21-inch thickness
pavement, very little difference was noted
the unit load supported by the two designs
pavement. No explanation was found f
this apparent inconsistency.

A summary of data from the Accelerat
Test made with the 18-inch bearing plate
presented in figure 10. Total loads are sho
for (1) elastic deflection of the subgrade
0.08 of an inch, (2) elastic deflection of t
pavement structure of 0.16 of an inch, a
(3) gross deflection of the structure of 0.40
an inch. The loads for each of three ar
trarily selected thicknesses of structure—
15, and 25 inches—were obtained direc
from load-thickness plots of the data. T
following comments pertain to these data:

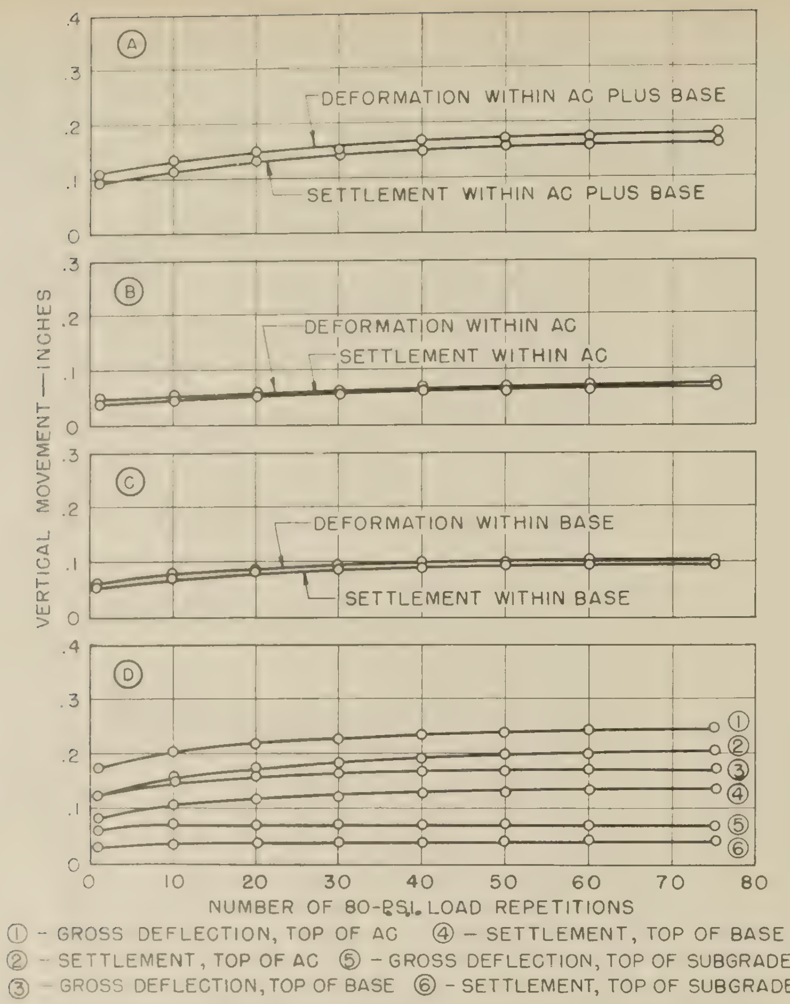


Figure 12.—Typical Repetitional Test data for tests in which the movement of the subgrade was nonprogressive.

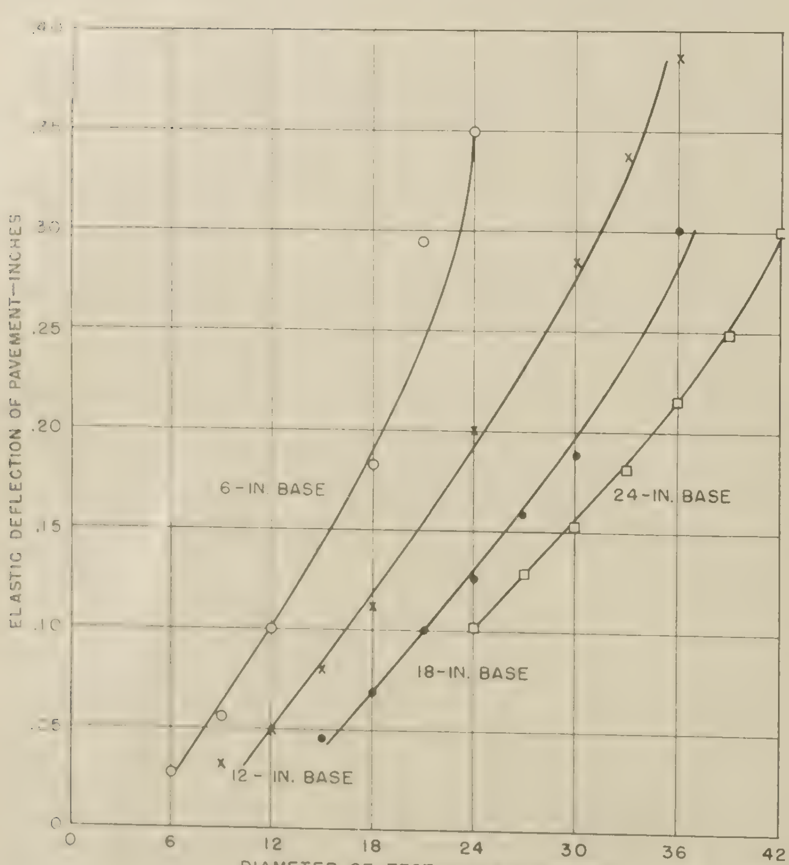


Figure 13.—Effect of size of bearing plate on elastic deflection of pavement—3-inch AC surface plus base.

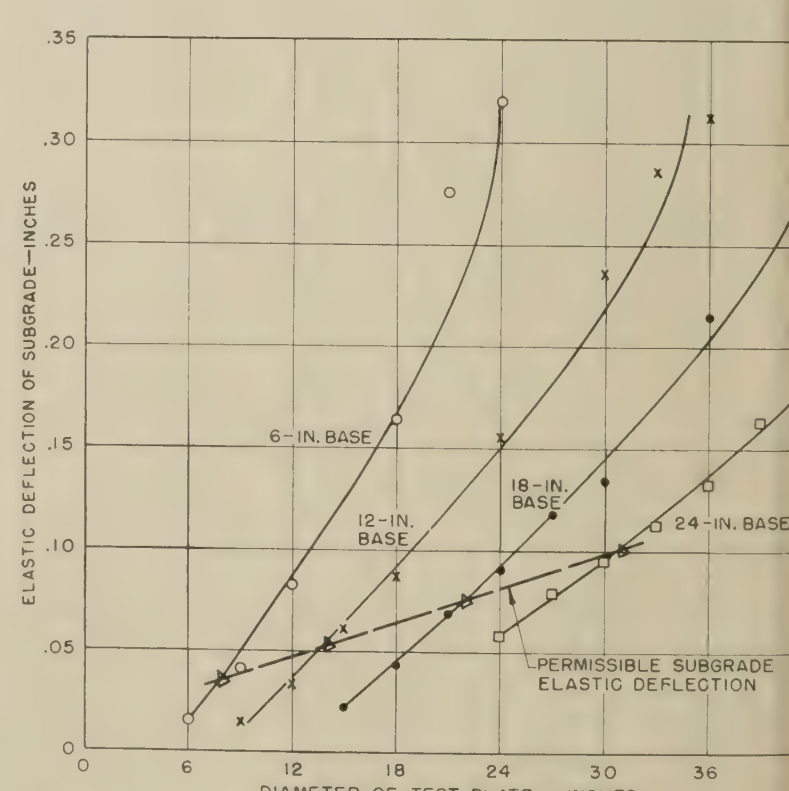


Figure 14.—Effect of size of bearing plate on elastic deflectio of subgrade—3-inch AC surface plus base.

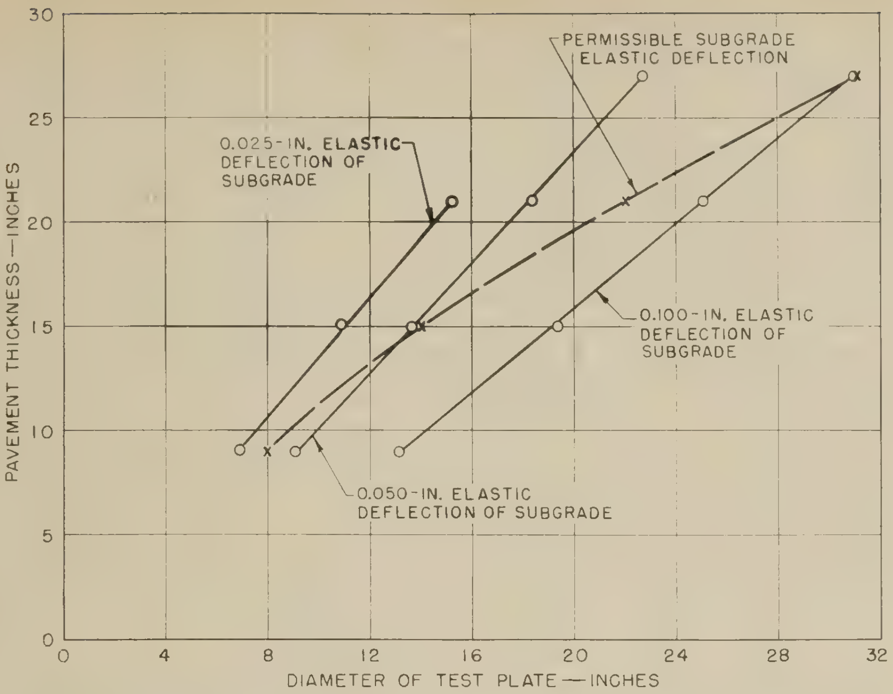


Figure 15.—Relation between size of bearing plate and pavement thickness based on elastic deflection of the subgrade—3-inch AC surface plus base.

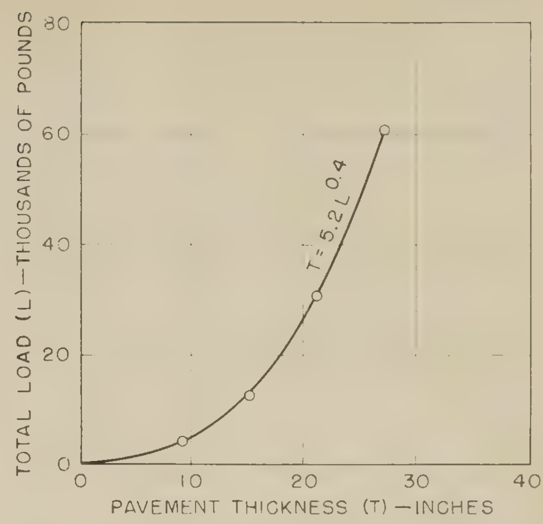


Figure 16.—Load-pavement thickness curve based on permissible elastic deflection of subgrade—3-inch AC surface plus base.

- In general, the indications are much the same regardless of the movement criterion used to express load support. For example, for all three criteria the difference between the loads supported by the base course and those supported by the AC surface plus base course combinations was of about the same order of magnitude. Also, the relative effect of overall thickness of pavement was about the same regardless of the movement criterion used.
- The total loads supported by a given thickness of base course alone increased some as the thickness of AC surface removed was increased from 3 to 6 inches. However, an increase from 6 to 9 inches in the thickness of AC surface removed had little effect on the total load supported by a given thickness of base course alone, except for pavements having overall thicknesses of 15 and 25 inches when the gross deflection of the pavement was 0.40 of an inch.
- Increasing the thickness of surface from 3 to 6 inches apparently was of more benefit than increasing it from 6 to 9 inches. This applied for all three thicknesses of pavement structure. The results of the Accelerated Test will be discussed further in connection with the analysis of the Repetitional Test data.

Analysis of Repetitional Test Data

The immediate objective in each of the Repetitional Tests was to determine the magnitude of the load under which the pavement or its supporting subgrade would act in an essentially elastic manner when a constant unit load was applied repetitiously on the pavement surface. It was reasoned that, if an elastic condition were developed a progressive permanent deformation of the subgrade would not occur and continued satisfactory structural performance of the pavement might be expected. In other words, the maximum load under which the desired degree of elastic action developed might constitute the maximum safe load for a particular pavement structure.

Some of the data developed from the Repetitional Tests, part *b*, are shown in figures 11 and 12. These data are typical of those obtained from each of these tests. In figure 11 the data are representative of those obtained when some progressive vertical movement of each component of the pavement structure and the subgrade occurred during the 75 applications of load. Figure 12 shows curves for a combination of load and pavement design conditions during which the subgrade had almost no progressive movement after about 10 applications of load. The data from these two tests, in a sense, represent the two extremes of the results obtained—in the former the movements of all components were progressive, but in the latter, the movement of the subgrade was essentially nonprogressive. In block A, the top block respectively in each of the figures, the deformation and settlement within the AC surface plus base are shown. The deformation was the total decrease in thichness of the two courses as a given load repetition was being applied. The settlement was the permanent decrease that remained when the load was released. The difference between the two amounts of decrease has been termed the elastic deflection within the pavement.

Elastic behavior

The deformation and settlement that occurred within the bituminous surface (AC) and within the granular base course are shown in blocks B and C, respectively (figs. 11 and 12). The gross deflections and settlements of the top of the AC surface, top of base, and top of subgrade occurring during these tests are shown in blocks D. Results from the Repetitional Tests showed that the elastic deflection (the difference between the gross deflection and settlement) was constant throughout the period of application and release of the 75 test loads for each of the Repetitional Tests. This relationship is illustrated by the curves in blocks D in figures 11 and 12. Note that the gross deflection and settlement relations for the combination of components—surface, base, and subgrade indicated by curves 1 and 2, the base and subgrade indicated by curves 3 and 4, and the subgrade alone indicated by curves 5 and 6—are essentially parallel. Little difference was noticed in this relationship regardless of whether the load applied through a particular plate had caused progressive, permanent deformations of the components.

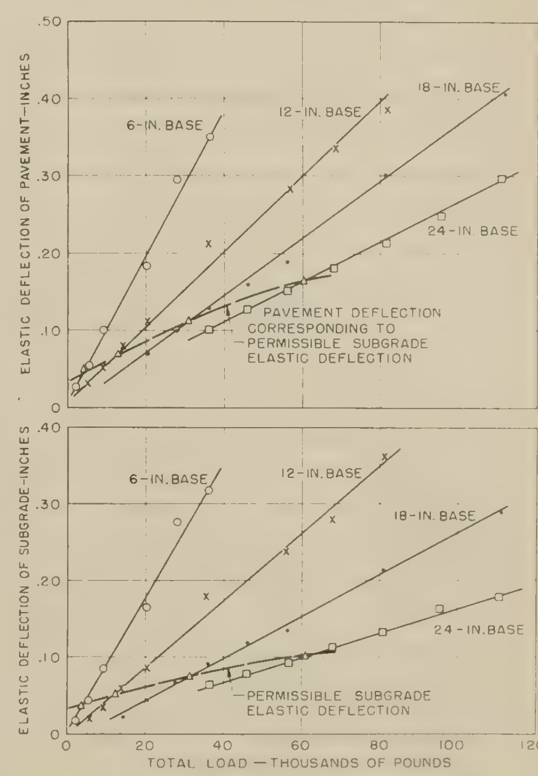


Figure 17.—Load-deflection curves for the 80-p.s.i. test load on the 3-inch AC surface plus base.

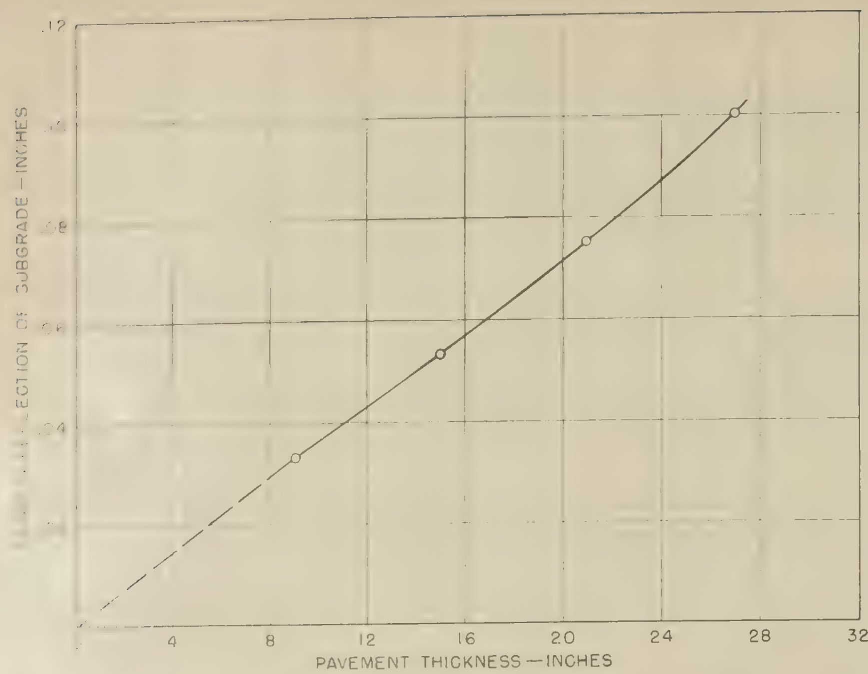


Figure 18.—Relation between pavement thickness and permissible elastic deflection of the subgrade for the 80-p.s.i. test load on 3-inch AC surface plus base.

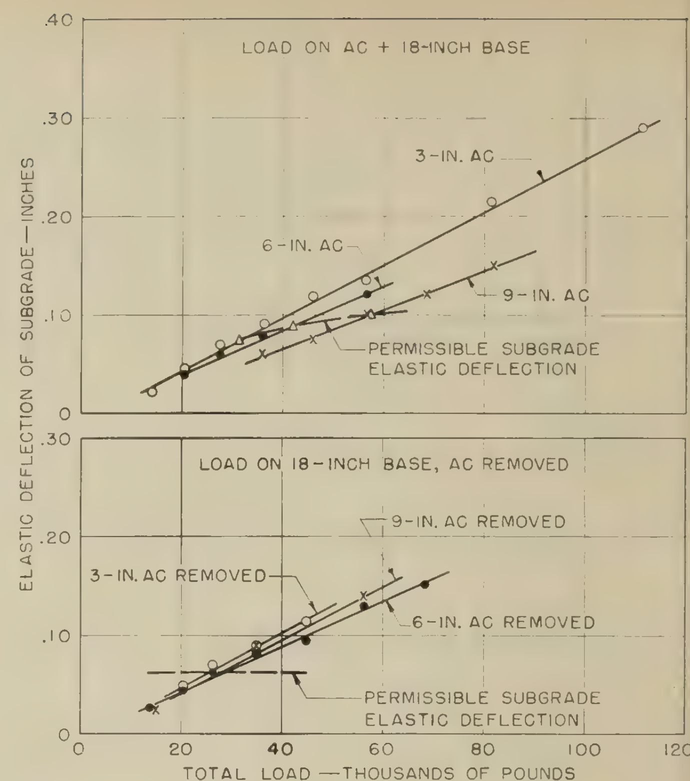


Figure 19.—Load-elastic deflection curves for surface courses plus 18-inch base pavements and for the bases with the surface removed—80-p.s.i. test load.

Early in the Repetitional Test program, it was noticed that the subgrade appeared to act in a completely elastic manner (fig. 12) when a given thickness of overlying pavement structure protected it from progressive, permanent deformations. Consequently, a reasonable and logical assumption seemed to be that such elastic action of the subgrade might serve as an index for determining the [illegible] load-carrying capacity of pavement. Therefore, the determination of the maximum load (80 p.s.i. times the area of bearing plate) for each test section under which the subgrade movement would cease to be progressive was adopted as another objective of this program.

In all of the Repetitional Tests, none of the surface and base courses themselves ceased to undergo some progressive deformation during the application and release of the 75 loads. The extent to which the pavement plus base was permanently deformed depended, of course, upon the load-pavement design relationship. For example, according to the data shown in figure 11, the surface plus base was deformed permanently about 0.25 of an inch after 75 applications of the test load (see lower curve in block A). The corresponding permanent deformation shown in figure 12 was 0.17 of an inch. Whether the permanent deformations that developed in the surface and base were the result of consolidation or distortion, or a combination of the two, is not definitely known. Initially, it was believed that such deformations in the granular base were the result of consolidation. However, little or no evidence in support of this belief was obtained from studies of the density of the base course before and after its being loaded.

Bearing plate relation to elastic deflection

Influence of the size of bearing plate on the elastic deflection produced by the 80-p.s.i. test load applied on the pavement surface is shown in figures 13 and 14. Data shown in these figures were collected from part *b* of the Repetitional Test for the different pavement sections consisting of 3-inch AC surfaces plus base and have been tabulated in column 7 of table 8a of reference (*3*). Figure 13 illustrates findings for the pavement structure and figure 14 shows data for the subgrade. The elastic deflections plotted in each of these figures are the averages of two or more test results. Also, for each test the deflections are averages of those for the 10th, 40th, and 75th load repetitions. Each plotted point therefore represents the average of a relatively large number of measurements. Consequently, the relations established are better than would have been shown if the elastic deflection at some arbitrarily selected repetition of the load had been utilized. The elastic deflections tended to be somewhat erratic during about the first 10 applications of the test load, after which the deflections remained practically constant.

For elastic deflections of the pavement and of the subgrade (figs. 13 and 14) of less than about 0.20 and 0.15 of an inch, respectively, the relations were well-defined and orderly. If larger elastic deflections were ignored, straightline curves could logically be drawn. Thus, within reasonable limits of total load applied to the surface of the pavement, the elastic deflections of the entire structure or that of the subgrade alone varied as a near-linear function of the diameter of the bearing plate. But it was evident that the total applied load (80 p.s.i. times area of the plate) caused progressive increases in elastic deflections of both the pavement and subgrade when larger bearing plates were used for tests on a pavement structure of a given thickness. This is shown in figures 13 and 14 by the progressive increase in curvature caused by increased load (larger plates).

Figures 11 and 12 represent the two extremes of data obtained from the Repetitional Tests. Figure 11 shows that some residual movement of the material occurred each time the load was applied and released. And figure 12 shows that the subgrade soil, after about the 10th application of load, acted in an elastic manner; that is, it deflected and recovered completely as each load was applied on the surface of the pavement and then released. From similar plots prepared for each of the tests made on each of the pavement sections, it was possible by interpolation to determine the approximate maximum total load and corresponding plate size that did not produce progressive permanent movement of the subgrade. These determinations and the resultant elastic deflection of the subgrade are listed in table 3.

Table 3.—Permissible subgrade elastic deflection and corresponding total loads for pavement sections of 3-inch AC plus base

Pavement structure, thickness	Plate diameter	Total load, 80-p.s.i unit load	Permissible subgrade elastic deflection
Inches	*Inches*	*Pounds*	*Inches*
9	8	4,020	0.034
15	14	12,320	.054
21	22	30,400	.076
27	31	60,300	.101

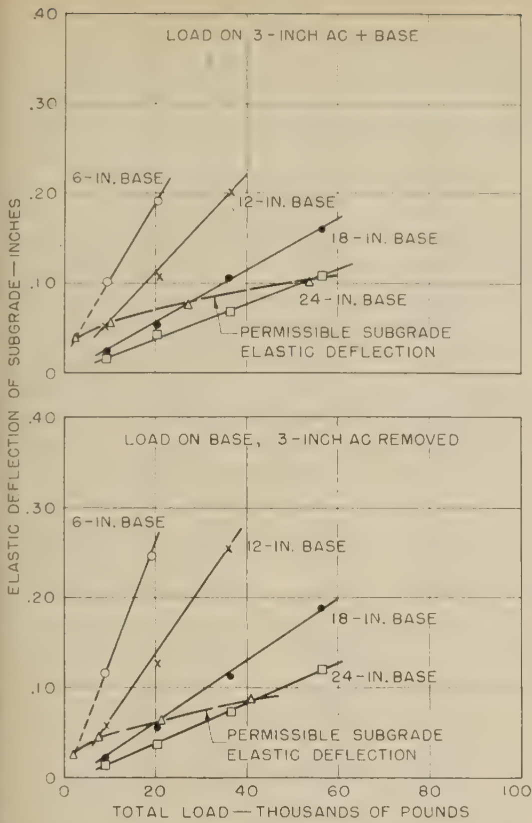


Figure 20.—Load-elastic deflection curves for 3-inch AC surface plus base pavements and for same bases with surface removed—80-p.s.i. test load—Accelerated Test.

As the subgrade elastic deflections shown in table 3 were caused by the maximum allowable load for the selected criterion, nonprogressive movement of the subgrade, they are referred to as the maximum permissible subgrade elastic deflections. The maximum permissible subgrade elastic deflections for each of the pavement sections consisting of 3-inch AC surface plus base have been plotted in figure 14. The resultant plot is a straight line. Note that as the plate size and AC-surface-plus-base thickness was increased, the permissible subgrade elastic deflection also increased. For the range in pavement structures tested (from 3-inch AC surface plus 6-inch base to 3-inch AC surface plus 24-inch base) the permissible subgrade elastic deflection increased threefold from 0.034 to 0.101 of an inch.

Load-pavement thickness relations

The relations between size of test plate and pavement thickness shown in figure 15 were developed directly from data illustrated in figure 14. The important influence of subgrade deflection on the load-thickness relation is shown clearly in figure 15. For elastic deflections of 0.025, 0.050, and 0.100 of an inch, the relations of size of bearing plate (load) to required thickness of pavement were linear. However, for the permissible subgrade deflections indicated by the intersection points of the dashed line in figure 14, the relation departed somewhat from a straight line. The load and related pavement thicknesses have been plotted in figure 16. The resultant curve shows the total load-pavement thickness relation developed by the method described; that is, it was based on nonprogressive movement of the subgrade between about the 10th and the 75th repetition of an 80-p.s.i. unit load applied on the surface of the 3-inch AC-surface-plus-base pavement structure. This curve primarily illustrates the type of information that can be developed from data obtained by the Repetitional Test. The equation of the curve is

$$T = 5.2\ L^{0.4}$$

where T equals pavement structure thickness in inches, and L represents the total load in kips. The curve of load and pavement thickness is parabolic and indicates that the thickness varied as the 0.4 power of the total load.

Figure 17 shows a series of total load-deflection curves for 80-p.s.i. unit loads applied to the surface of the 3-inch AC surface plus-base sections. For data tabulations see columns 5 and 7 of table 8a in reference (*3*). The curves in the upper section of figure 17 apply to the elastic deflection of the pavement as a whole, and those in the lower section apply to that of the subgrade alone. The deflections shown are the average of those obtained from the 10th, 40th, and 75th applications of the load. These curves demonstrate that, in this type of test, the magnitude of the elastic deflection of either the entire pavement structure or of only the subgrade varied as a linear function of the total applied load. The elastic deflections of the pavement corresponding to the permissible subgrade elastic deflections are represented by the triangular symbol in the upper part of figure 17. These deflections were 0.052, 0.074, 0.112, and 0.167 of an inch for the 6-, 12-, 18-, and 24-inch bases, respectively. In the lower part of figure 17, the corresponding permissible subgrade elastic

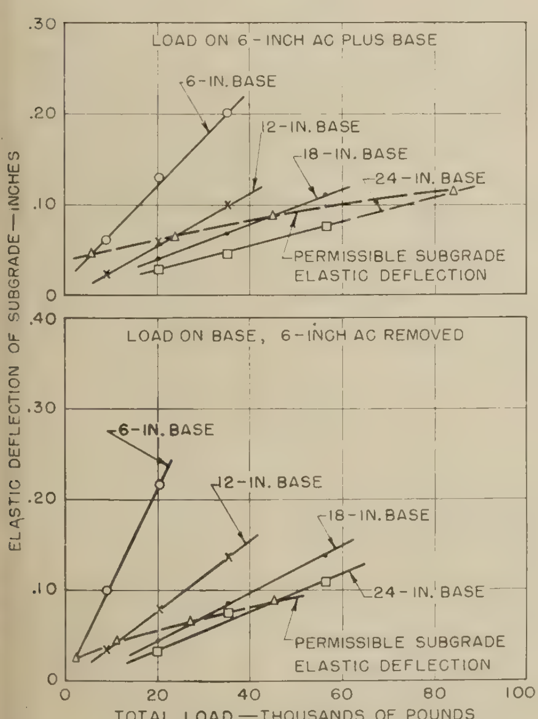


Figure 21.—Load-elastic deflection curves for 6-inch AC surface plus base pavements and for same bases with surface removed—80-p.s.i. test load—Accelerated Test.

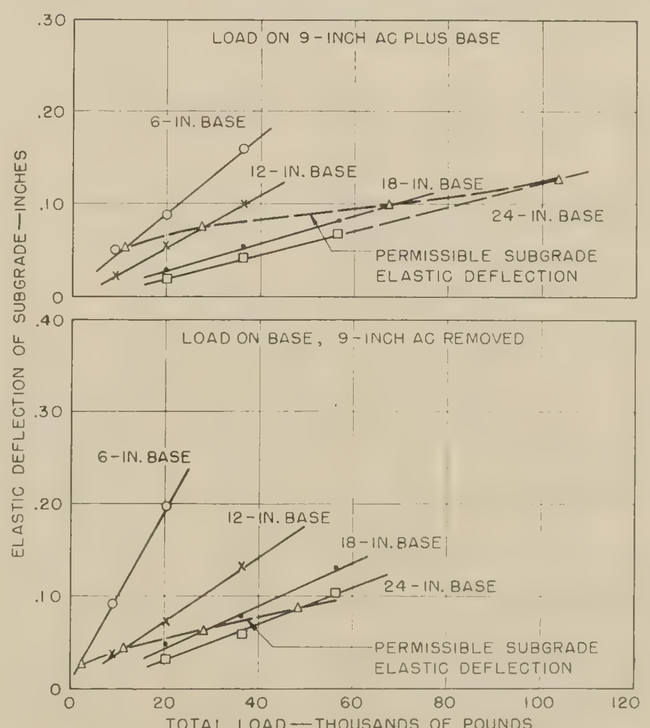


Figure 22.—Load-elastic deflection curves for 9-inch AC surface plus base pavements and for same bases with surface removed—80-p.s.i. test load—Accelerated Test.

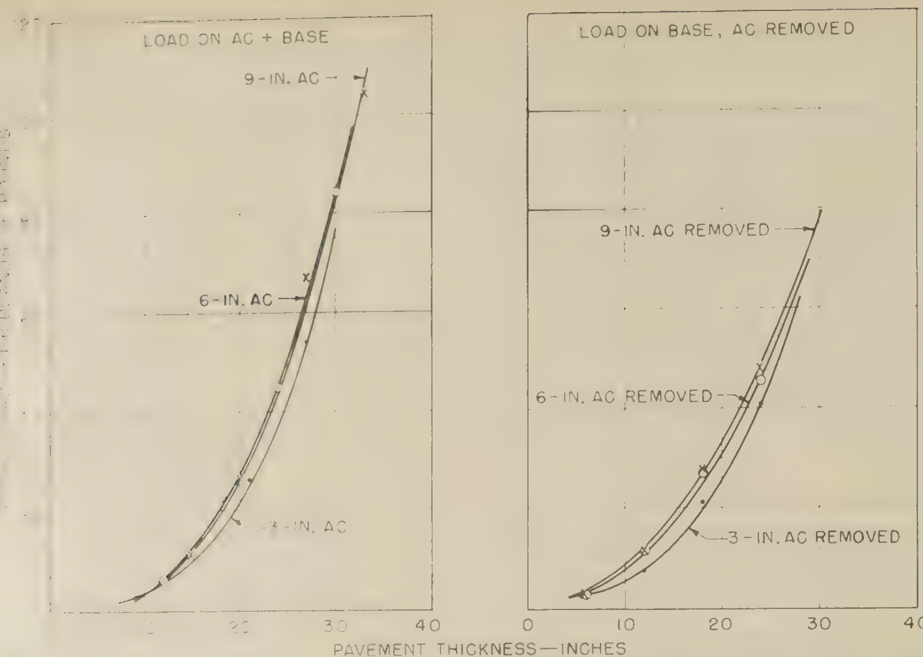


Figure 23.—Load-pavement thickness curves for surface course plus base pavements and for same bases with surface removed—80-p.s.i. test load—Accelerated Test.

...tions, 0.034, 0.054, 0.076, and 0.101 of ... also are indicated by the triangular ... The difference between each pair of ...lections for a given load represents the ...ement that occurred within the 3-inch AC surface plus base. The lines connecting the data represented by triangular symbols in each part of the figure are somewhat ...

In figure 18, the permissible elastic deflection of the subgrade for the 3-inch AC surface-plus-base pavement have been plotted against the overall thicknesses of pavement. ...permissible elastic deflection of the subgrade for any overall thickness of pavement, from 9 to 27 inches inclusive, for a unit load of 80 p.s.i. can be obtained from this plot. Although the line drawn through the plotted points has a slight curve, for practical purposes the data could be represented by a straight line drawn through the origin. This straight line could than be used for extrapolating to thicknesses of pavement less than 9 inches or more than 27 inches.

In figure 19 relations of total load to elastic subgrade deflection are shown for the three thicknesses of surfaces—3-, 6-, and 9-inch AC—on the 18-inch base course. Also shown are similar curve relations developed from tests made directly upon the 18-inch base course beneath each of the three thicknesses of surface. Again, the data were derived from part *b* of the Repetitional Test, and the tabulations are in reference (*3*).

Because of the limited number of tests made on the 6-inch and 9-inch AC surfaces and directly on the 18-inch base course, it was impossible to arrive at the probable safe load for these pavement sections on a basis of the actual performance of the subgrade as was done for the 3-inch AC pavement sections. However, an estimate of the load-carrying capacity of these sections can be made on the assumption that the relation between pavement thickness and permissible elastic deflection of the subgrade for the 3-inch AC surface (fig. 18) applies also to the total pavement thicknesses for the curves shown by solid lines in figure 19. The dashed line in each block of figure 19 was drawn through the points established on this assumption. The indicated loads at the points of intersection with the load-deflection curves were 32,000, 42,000, and 58,000 pounds for the pavements consisting of 3-inch AC surface plus base, 6-inch AC surface plus base, and the 9-inch AC surface plus base, respectively. And these indicated loads were 25,000, 28,000, and 26,500, pounds for the 18-inch base beneath the 3-, 6-, and 9-inch AC surfaces, respectively. These loads are discussed more fully later in this article.

In the analysis of the results of the Accelerated Tests, the different designs of pavement were compared on a basis of the pavement's ability to support load at some arbitrarily selected deflection—usually the gross or elastic—of the pavement and of the subgrade. The development of the permissible subgrade elastic deflection from the Repetitional Tests makes it possible to reexamine the Accelerated Test data by utilizing these permissible elastic deflections as a criterion of load support. To this end, the curves shown in figures 20, 21, and 22 were developed. Those in figure 20 were developed from tests made on the 3-inch AC section of pavement, both upon the surface and upon the base after removal of the surface. Those in figures 21 and 22 were developed from tests made on the sections of pavement having surface thicknesses of 6 and 9 inches respectively. The elastic deflections of the subgrade for a unit load of 80 p.s.i. used in the development of the curves shown in figures 20 and 21 were obtained directly from column 8 in table 4 of reference (*3*). The total loads are the product of the area of the bearing plate and the unit load. The dashed lines (permissible subgrade elastic deflections) intersect the individual curves at deflections corresponding to those shown by the curve in figure 18 for the pertinent, pavement structure thickness. The points of intersection of the dashed lines and the load-deflection curves indicate those loads that the sections might safely have supported under repetitive loading on rigid plates.

In figure 23, these loads have been plotted as a function of the overall thickness of pavement. The resultant curves are of the same character

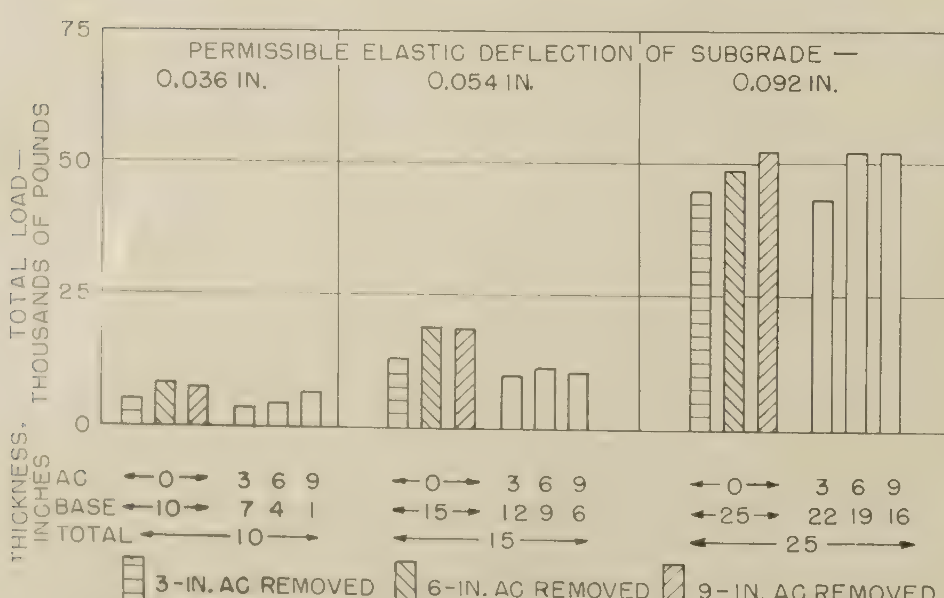


Figure 24.—Comparison of total loads supported by pavements of different thicknesses and composition—80-p.s.i. test load—Accelerated Test.

as those developed from the series of Repetitional Tests made on the 3-inch AC surface plus-base pavement (fig. 16). In fact, if the curves for these particular pavement sections were superimposed over those shown in figure 16, a remarkably close agreement would be found between them. As for the data from the Repetitional Tests, the data in figure 23 indicate that the required thickness of pavement varies as an exponential function of the applied load—approximately as its square root. This seemed to be true both where the pavement structure consisted of a range in thicknesses of surface plus base and where the pavement structure consisted of base course alone. It is emphasized that the load-pavement relations illustrated in figure 23 were based upon the assumption that the permissible elastic deflections of the subgrade corresponded to those developed from the comprehensive series of Repetitional Tests on the 3-inch AC surface plus-base pavement.

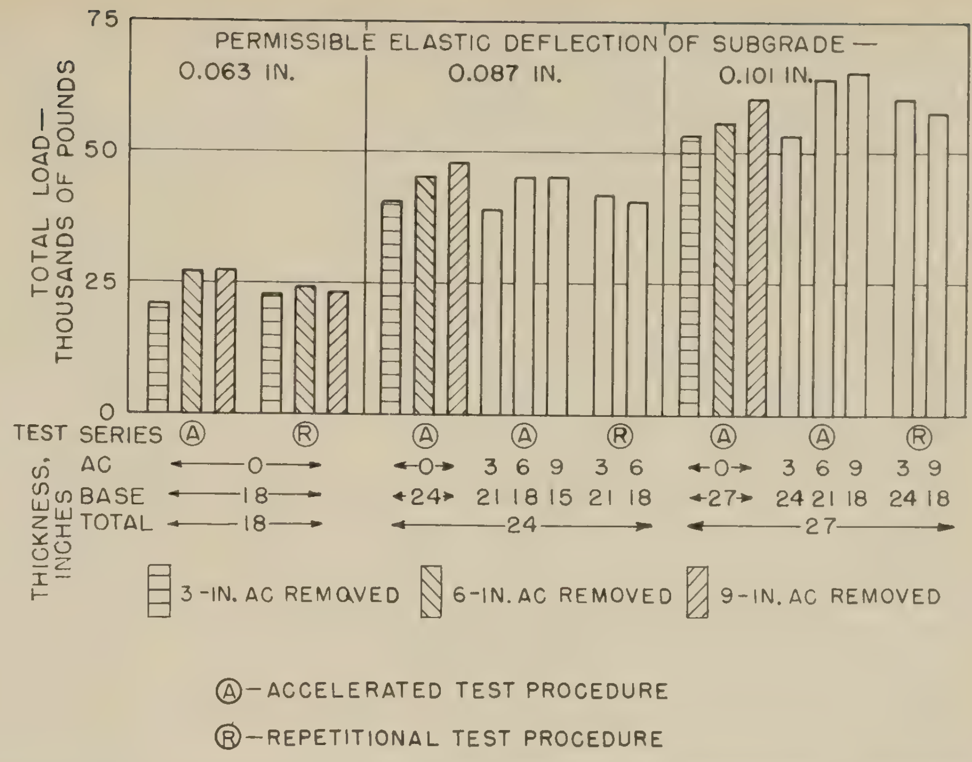


Figure 25.—Comparison of total loads supported by pavements of different thicknesses and composition—80-p.s.i. test load—Accelerated and Repetitional Tests.

Comparison of Accelerated and Repetitional Test Data

The results of the Accelerated and the Repetitional series of load tests have been summarized in figures 24 and 25. In figure 24 the comparisons shown are of the same general nature as those shown for the Accelerated Test data in figure 10. However, the total loads shown in figure 24 were obtained from data in figure 23 for 10-, 15-, and 25-inch thicknesses of pavement structure and are for a unit load of 80 p.s.i. The permissible subgrade elastic deflections are also shown. The deflections were obtained from the relations of pavement thickness and permissible elastic deflection shown in figure 18.

The following comments pertain to the data shown in figure 24. Except for the 6- and 9-inch AC surfaces for the 25-inch total pavement thicknesses, the granular base considered as a complete structure was somewhat more effective in supporting a given load without having progressive permanent movement of the subgrade than was an equal thickness of pavement that consisted of either 3, 6, or 9 inches of surface on a granular base. The differences were consistent for the data shown in figure 24, as were the comparable differences shown in figure 10.

Comparisons

Comparisons for as many as possible of the results of the Accelerated and Repetitional tests have been shown in figure 25. To avoid undue extrapolation of the data, these comparisons were limited to pavement thicknesses of 18, 24, and 27 inches. The data showed good agreement between the results of the two series of tests when consideration is given to the fact that these tests were made a year apart and the testing procedures differed appreciably. These data indicated that the relative effectiveness of the different pavement sections to protect the subgrade was similar to that shown by data in figure 24.

Elastic deflection-load relations

The elastic deflections of the different pavement components have been discussed previously. It was pointed out that the magnitude of the elastic deflection of the entire pavement structure, or that of the base plus subgrade, or that of the subgrade alone was practically constant throughout the period of application and release of the 80-p.s.i. loads. Substantiating evidence for this conclusion has been presented in figures 26 and 27 from the results of part *b* of each of the Repetitional Tests made on the 3-inch AC surface plus-base pavement. The data are tabulated in column 6 of table 8a, reference (*3*). The elastic deflections of the pavement and subgrade have been shown in figures 26 and 27, respectively, as a function of the number of load repetitions. Generally, for the smaller plates and smaller loads, the deflections were practically constant but for the larger plates and greater loads there was a tendency for the magnitude of the elastic deflection to increase somewhat during

Table 4.—Comparison of vertical movements of pavement components and subgrade

	Test conditions—							
Pavement structure	3-inch AC plus 6-inch base.		3-inch AC plus 12-inch base.		3-inch AC plus 18-inch base.		3-inch AC plus 24-inch base.	
Plate diameter [1] ....inches	8		14		22		31	
Total load, at 80-p.s.i. unit load ......kips	4.0		12.3		30.4		60.4	
ELASTIC DEFLECTION (Averages of 10th, 40th, and 75th load applications)								
Component	*Inches*	*Percent*	*Inches*	*Percent*	*Inches*	*Percent*	*Inches*	*Percent*
AC surface	0.008	16	0.008	11	0.016	14	0.039	23
Base	.010	19	.012	16	.020	18	.027	16
Subgrade	.034	65	.054	73	.076	68	.101	61
Entire pavement structure	.052	100	.074	100	.112	100	.167	100
GROSS DEFLECTION (at 75th load application)								
AC surface	0.070	22	0.099	31	0.098	28	0.112	27
Base	.152	48	.130	41	.132	37	.170	40
Subgrade	.094	30	.090	28	.122	35	.140	33
Entire pavement structure	.316	100	.319	100	.352	100	.422	100
RATIO OF ELASTIC TO GROSS DEFLECTION								
AC surface	11		8		16		35	
Base	7		9		15		16	
Subgrade	36		60		62		72	
Entire pavement structure	16		23		32		39	

[1] Plate size, obtained by interpolation, that caused nonprogressive movement of the subgrade.

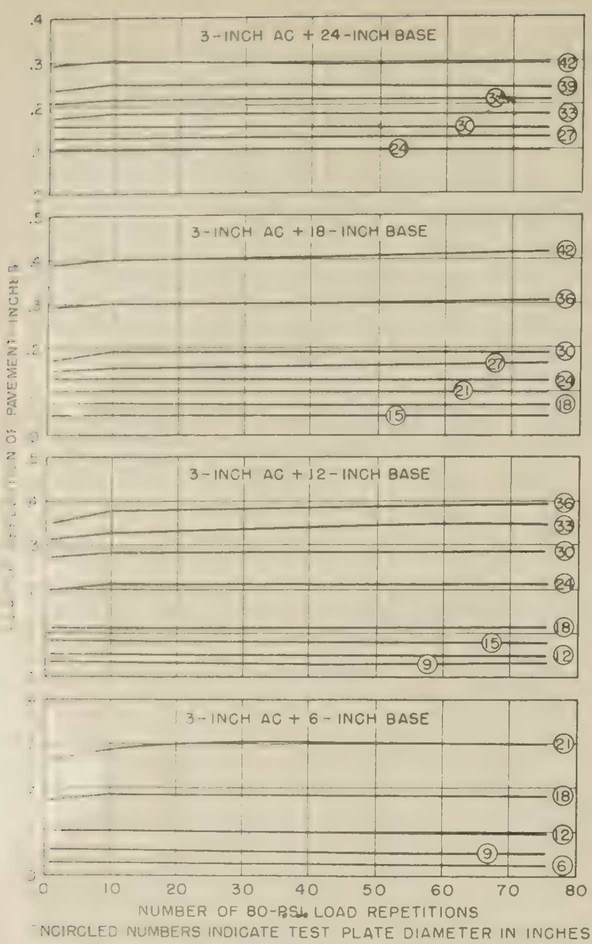


Figure 26.—Effect of load repetitions on elastic deflection of pavement—80-p.s.i. test load.

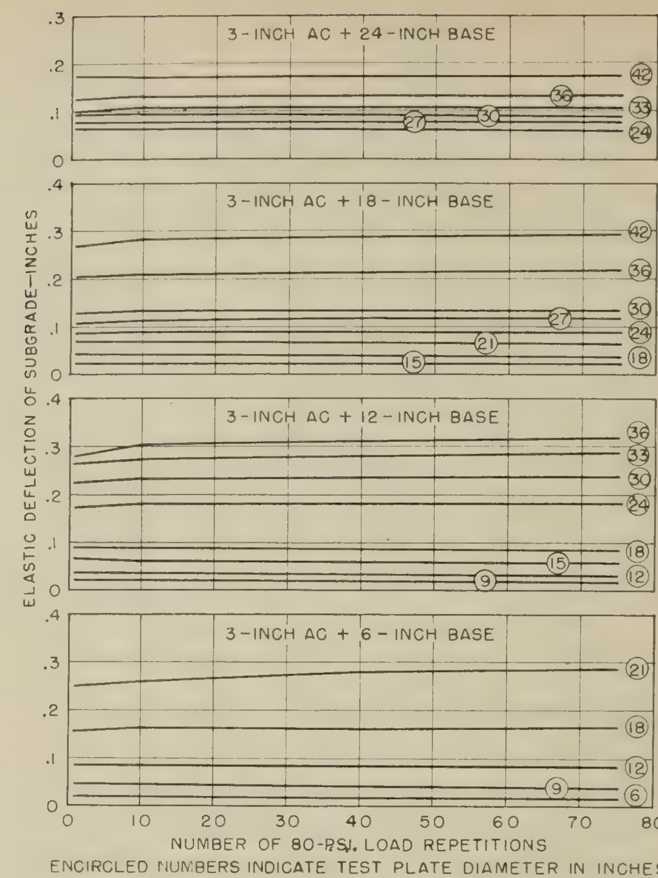


Figure 27.—Effect of load repetitions on elastic deflection of subgrade—80-p.s.i. test load.

the first 10 repetitions of the load. With very few exceptions the deflections remained essentially constant during succeeding repetitions of load. The differences between the deflections presented for the pavement in figure 26 and for the subgrade in figure 27 represent the magnitude of the elastic movement that occurred within the surface plus base.

Vertical movement

The method developed for measuring the vertical movements occurring at different levels in the pavement structure provided the means for many ways of studying the elastic and gross deflections of the separate components. For example, in figures 28 and 29, respectively, the elastic and gross deflection measurements have been plotted against the diameters of the test plate for the entire pavement, the base plus the subgrade, and the subgrade alone. The differences between the deflections shown in each of the two upper curves in each block of the figures represent the deflections that occurred within the 3-inch AC surface. The difference between the two lower curves in each block represents the deflection that occurred within the granular base. The elastic deflections of the supporting subgrade itself are shown directly by the lowest curve.

The heavy vertical lines drawn through the three curves in each block of figures 28 and 29 designate the maximum size of plate that did not produce progressive permanent movement of the subgrade when loaded on the pavement surface. These plate sizes were obtained by interpolation or extrapolation (table 3). The relation between plate diameter and elastic and gross deflections shown by the curves in these figures indicates that, for loads applied on the pavement surface, the major portion of the elastic deflection occurred in the subgrade and that the

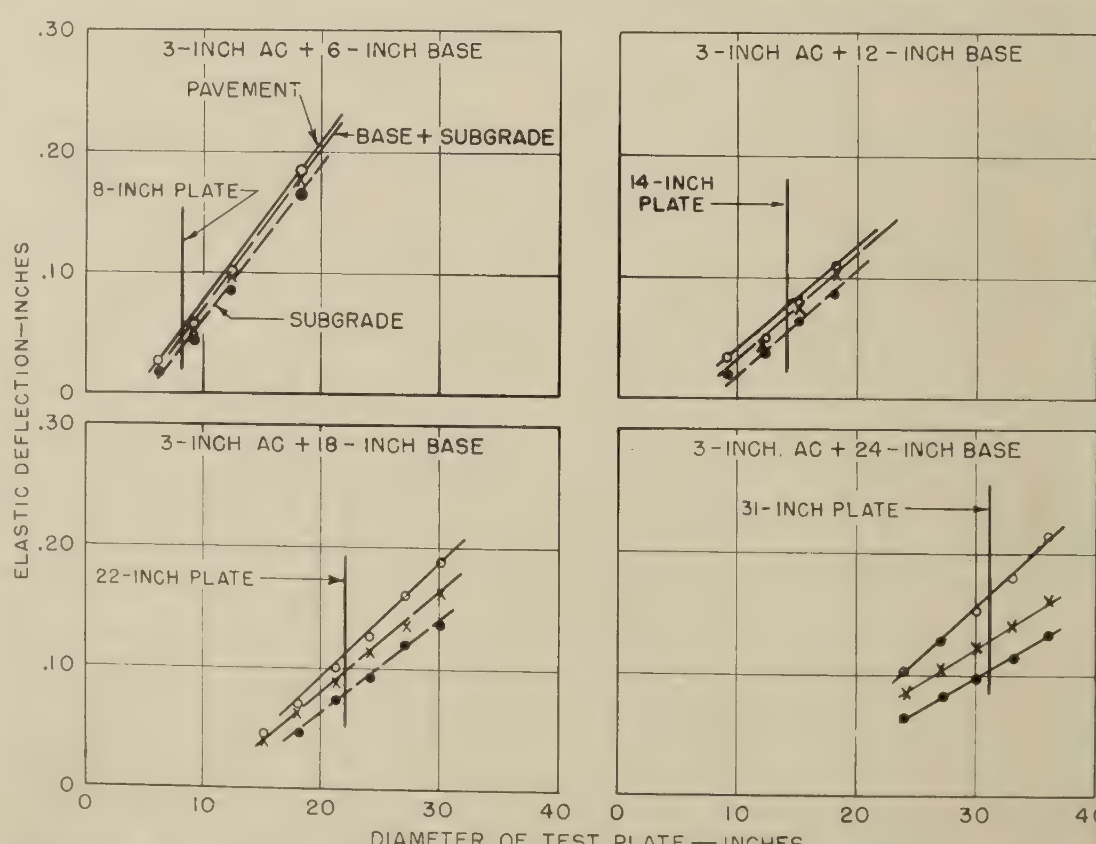


Figure 28.—Relation between size of bearing plate and elastic deflection of the pavement, the base plus subgrade, and the subgrade alone—80-p.s.i. test load.

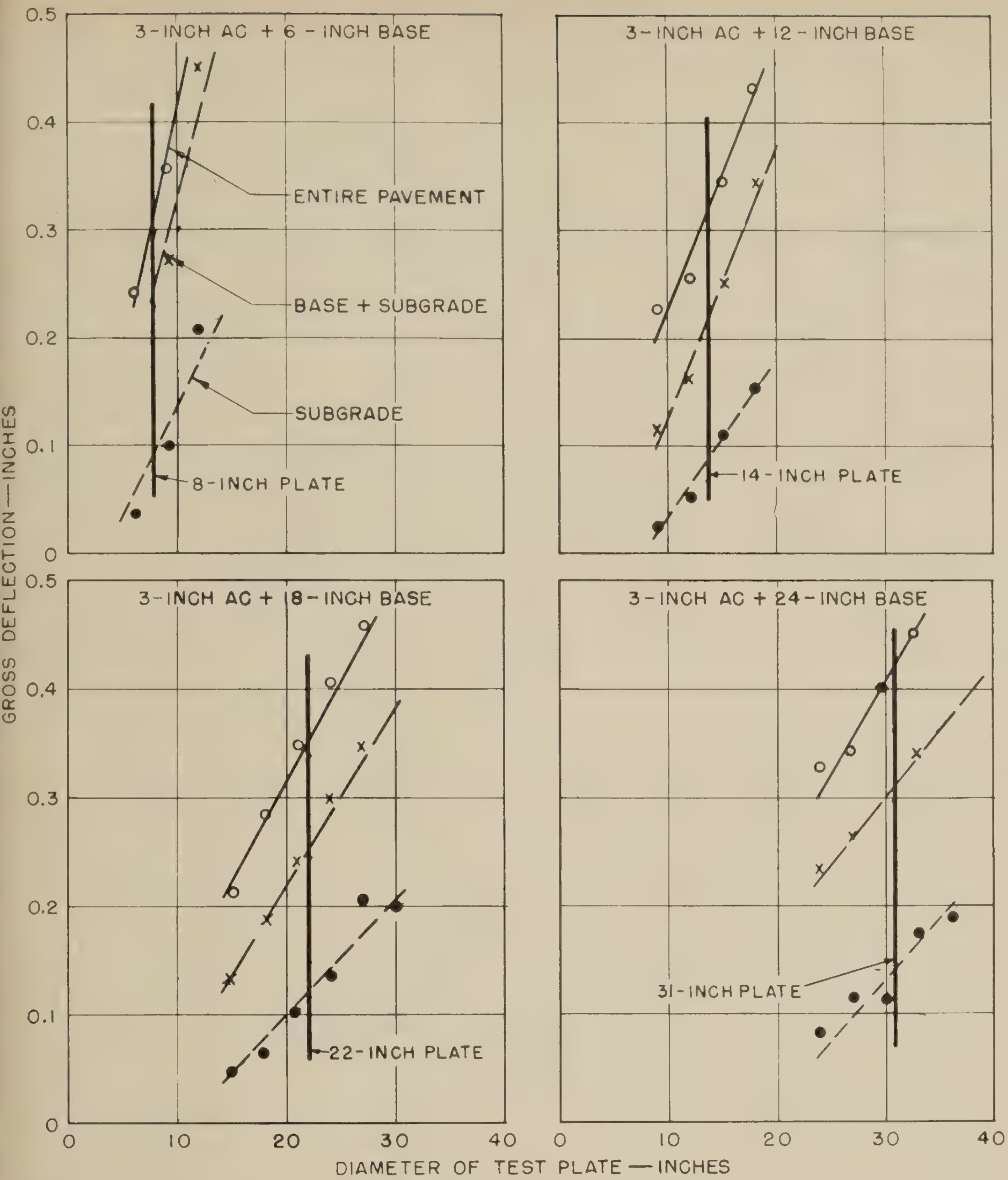


Figure 29.—Relation between size of bearing plate and gross deflection of the pavement, the base plus subgrade, and the subgrade alone—80-p.s.i. test load.

Table 5.—Effect of temperature [1] of the surface course on the load-bearing capacity of the test sections

Pavement section and time of test	Total load, thousands of pounds, for—					
	0.08-inch elastic deflection of subgrade and overall thickness of—			0.40-inch gross deflection of structure and overall thickness of—		
	10 inches	15 inches	25 inches	10 inches	15 inches	25 inches
Base course, beneath 3-inch AC:						
Spring	17.6	25.8	50.2	28.9	39.4	64.2
Summer	14.5	21.7	43.0	29.0	39.4	64.2
3-inch AC plus base:						
Spring	19.9	28.5	52.0	33.9	43.9	67.9
Summer	12.7	19.0	37.1	28.1	36.2	57.5
6-inch AC plus base:						
Spring	23.5	31.7	54.3	36.2	45.7	67.9
Summer	17.6	24.4	45.2	28.5	37.6	62.0
9-inch AC plus base:						
Spring	24.9	34.4	57.0	36.2	45.7	67.9
Summer	14.9	20.4	39.4	26.7	33.9	53.4

[1] Approximate temperature range of AC surface:
Spring, 40° to 50° F.
Summer, 75° to 95° F.

deflection of the different component combinations tended to vary as a linear function of the diameter of the bearing plate for the range in plate sizes shown. For the pavements with 6- and 12-inch bases (upper blocks in figs. 28 and 29) the curves, representing movement of the entire pavement structure and of the base plus subgrade, are practically parallel. This indicates that the movements within the AC surface remained constant as the plate size (load) was increased. However, the curves for the base plus subgrade and the subgrade alone tend to diverge as the plate size was increased, indicating that the greater the load the greater the movements within the granular base. For the 18- and 24-inch base courses all the curves tend to diverge as plate size was increased.

Data in table 4 show the elastic and gross deflections of each component of the pavement and of the entire pavement structure. The portion of the total movement that occurred in each component, expressed in percentages, is also shown. The data listed are for the four plates—8, 14, 22, and 31 inches in diameter—that were selected as representing the maximum size through which 80-p.s.i. loads could be applied to the pavement surface without causing progressive permanent movement in the subgrade. The ratios of the elastic deflection to the gross deflection, expressed in percentages, are also listed in table 4. These data were obtained from that shown by figures 28 and 29. The following comments apply to these data.

- Most of the elastic deflection occurred in the subgrade. About the same portion, 70 percent, occurred in the subgrade for each of the tests made with the four combinations of load and design for which results were tabulated. About half of the remaining percentage of deflections occurred in the AC surface and half in the base.
- The gross deflection occurring in each of the components was more evenly divided than the elastic deflection; about 30 percent occurred in both the AC surface and the subgrade, and about 40 percent in the base. These percentages were about the same for each of the load-design combinations.
- The ratios of elastic to gross deflection indicate that by far the major portion of the total vertical movement was permanent. For the 6-inch base section, only 11 percent of the movement occurring within the 3-inch AC surface was elastic and only 7 percent occurring within the base was elastic. For the 24-inch base section, the corresponding movement percentages were appreciably greater, 35 and 16 percent, respectively. The behavior of the subgrade was markedly more elastic than that of the AC surface and base, the range in ratios of elastic to gross deflection being 36 to 72 percent.

MISCELLANEOUS TESTS

Effect of Bearing Plate Size Under Constant Total Load

In connection with the program of Repetitional Tests a limited study was made of the effect that the size of bearing plate had on the

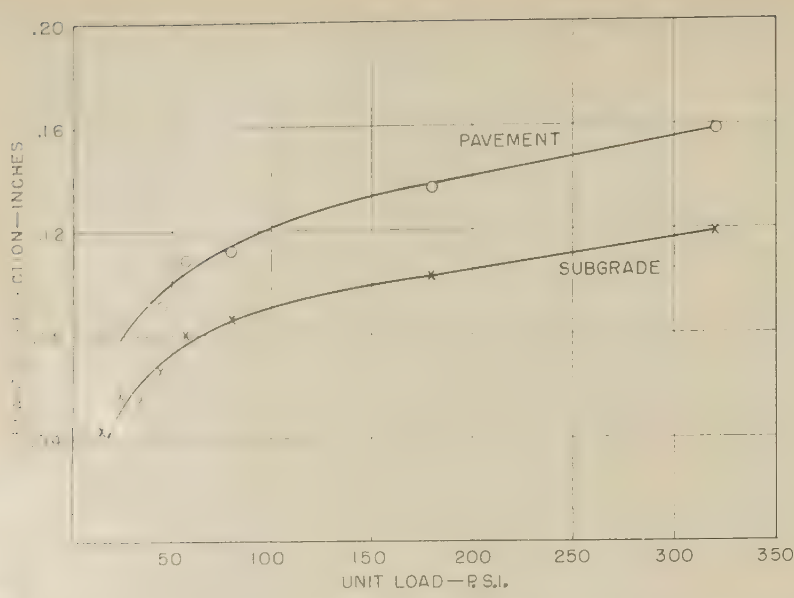


Figure 30.—Relation between unit load and elastic deflection for a total load of 20,320 pounds on bearing plates 9 to 42 inches in diameter—3-inch AC surface plus 12-inch base.

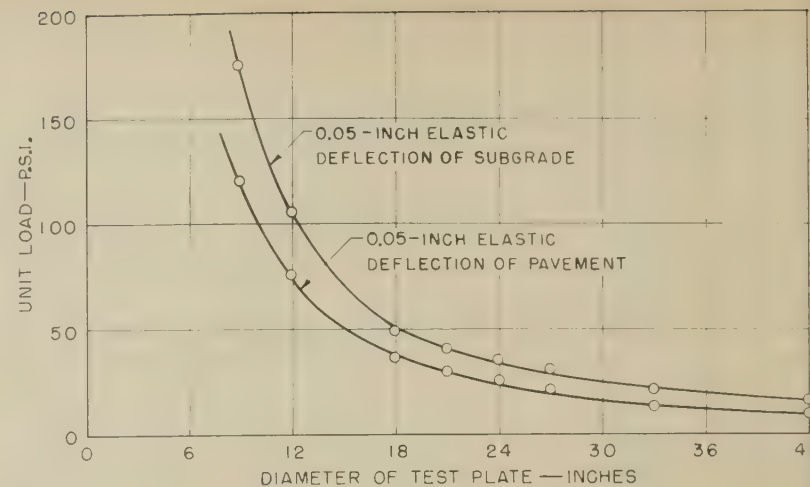


Figure 31.—Relation between unit load and size of bearing plate for 0.05-inch elastic deflection of the pavement and subgrade—3-inch AC surface plus 12-inch base.

structure when the repetitiously applied total load was held constant. The ... confined to the 3-inch AC surface ... 12-inch base section. As for the regular ... of Repetitional Tests, the plates ranged ... from 9 to 42 inches in diameter and the ... load was 80 p.s.i. The total load was 20,320 pounds, which is the equivalent of an 80-p.s.i. load applied to a plate 18 inches in diameter.

The data obtained from these tests are presented in columns 5 and 7 of table 8b of reference (3) and are shown here in figures 30 and 31. In figure 30 unit loads have been plotted against the elastic deflections (average of 10th, 40th and 75th load repetition) of the entire pavement structure, and of the subgrade alone. In figure 31, the unit loads have been plotted ... function of the diameter of the test plate ... 0.05 of an inch elastic deflection of the subgrade and 0.05 of an inch elastic deflection ... the pavement structure.

In figure 30, the test plate diameters and corresponding unit loads for the two extreme pairs of data (the points plotted at the ends of the curves) shown respectively are 42 inches and 15 p.s.i., and 9 inches and 320 p.s.i. The curves show that, for a 20,320-pound total load, an increase in unit load from 15 p.s.i. to 100 p.s.i. caused a more marked increase in elastic deflection than an increase from 100 to 320 p.s.i. The unit load-plate size curves in figure 31 show clearly the importance of the size of test plate, particularly the smaller plates. The unit loads decreased at a decreasing rate as the diameter of the plate was increased up to 42 inches.

Effect of Temperature of AC Surface

As previously stated, in studying the ability of pavements of the flexible type to support load applied through rigid plates, it was necessary to control the temperature of the bituminous surface, particularly when the different test sections were made up of several different thicknesses of the surfacing material. Information on the effects of temperature of the bituminous surface was obtained from several series of special tests. One study was made during the spring months. Using the 12- and 24-inch diameter plates, load tests were made on the 3-, 6-, and 9-inch AC surfaces of the 6-, 12-, and 24-inch base sections and also directly on the base course of these three sections beneath the 3-inch AC surface. The procedure used for loading was similar to that used in part *a* of the Accelerated Test; that is, application and release was made of three separate load increments, each one time only. The data from these tests are presented in table 6 of reference (*3*). Comparisons of the data obtained in the spring tests with those obtained from similar tests made during the summer, table 4, reference (*3*) have been presented in table 5 and in figure 32.

The results on the effect of temperatures shown in table 5 are for those obtained in tests made only with the 24-inch diameter plate. Total loads are shown for three overall thicknesses of pavement, for the base beneath the 3-inch AC surface, and for the 3-, 6-, and 9-inch AC surface plus base for the spring and summer testing periods. Comparisons of the data obtained during the two periods of testing are shown by figure 32. Unit loads for the two periods have been plotted for the 12- and 24-inch diameter plates. The deviation of the points from the line of equality shows the extent to which the loads differed. The following comments concern the data obtained in these tests.

- On the basis of a subgrade elastic deflection of 0.08 inch, the base course beneath the 3-inch AC surface was somewhat more effective in the spring than in the summer (fig. 32). The same result can be shown when the gross deflection of the pavement structure is used as the criterion. On the basis of both elastic deflection of the subgrade and gross deflection of the pavement, the effect of the presence of the AC surface on the base course was much more pronounced in the spring than in the summer.

Effect of Surcharge

Reference was made previously to the fact that the indicated resistance to load of the base course of the various pavement sections was somewhat less beneath the thin bituminous surface than beneath the thicker ones. Initially it was believed that this was due to the weight of the overlying material or surcharge effect of the surfacing material. A limited series of preliminary tests to study the effect of surcharge was made on the base course and on the subgrade of an auxiliary test pavement: the results failed to indicate any effect of surcharge.

The matter was studied further in a subsequent series of tests made on the oval track pavement. Curves are shown in figure 33 for tests on the base course and for tests on the subgrade. The data from these tests do not appear in reference (*3*). In both cases, the unit load was related to the elastic deflection of the subgrade. In the nonsurcharge tests the overlying components were removed from an area having a diameter three times that of the bearing plate. In the surcharge tests the normal procedure for testing the base course and the subgrade was employed, that is, the opening in the pavement was just sufficient to accommodate the plate.

In the tests on the 18-inch base in which the 18-inch diameter plate was used, the curves indicate some beneficial effect of the presence of the 9-inch AC surfacing, whereas the results from the tests made with the same plate on the 24-inch base indicated no effect whatever. Little influence of surcharge was indicated by the results of tests made on the 12-inch base. In the tests on the 6-inch base, the data actually indicated that the surcharge load had a detrimental effect.

The tests on the subgrade were made with the 12- and 24-inch diameter plates beneath the pavement sections consisting of the 3-inch AC surface plus the 18-inch base. Little influence of surcharge was evident in results of tests made with the larger of the two plates. However, some detrimental effect of surcharge was indicted in tests made with the 12-inch plate; these effects were similar to the effect of tests made with the 18-inch plate on the 6-inch base beneath the 9-inch AC surface.

It was noted previously that the base courses showed greater resistance when tested beneath the thick AC surfaces than they did when tested beneath the thin AC surfaces. The results of the limited series of nonsurcharge tests did not provide an explanation for this finding. In general, the data indicated no important effect of surcharge.

Asphaltic Concrete Surfaces Laid on Subgrade

In addition to the test sections listed in table 1 and shown in figure 1, a section 50 feet in length was constructed of 6 inches of asphaltic concrete laid directly upon the subgrade soil at one end of the north tangent. A few plate-bearing tests were made on this section during the same period of time the other tests were made on the regular test sections. The results of these few tests, although neither consistent nor conclusive, were considered of sufficient interest to justify the construction of three additional sections; each section was 12 by 150 feet in size. Asphaltic concrete 3, 6, and 12 inches thick, respectively, was placed directly on the subgrade soil in these test sections located on the south tangent.

Accelerated Tests were made on the three special sections of pavement. The bearing plates used were 12, 18, 24, and 30 inches in diameter. Replicate tests were made with each plate on the surface and with the 30-inch plate on the subgrade after removal of the surface. Data from these tests are presented in columns 4 and 8 of table 7 in reference (*3*) and are shown in figure 34. The averages of the unit loads for the replicate tests have been plotted as a function of the thickness of the AC surface. The three separate groups of curves apply to an elastic deflection of the subgrade of 0.08 of an inch, an elastic deflection of the pavement (AC plus subgrade) of 0.08 of an inch, and a gross pavement deflection (AC plus subgrade) of 0.40 of an inch. Also shown are the data from tests made with the 30-inch diameter plate directly on the subgrade (zero pavement thickness) beneath each of the three thicknesses of pavement. These data are tabulated in columns 1 and 5 of table 7 in reference (*3*). The following comments pertain to these data.

- Tests made with the 12- and 18-inch diameter plates indicated a definite increase in load-carrying capacity as the AC surface thickness was increased from 0 to 3, to 6, and to 12 inches.
- The curves for the 24- and 30-inch plates show that little or no increase occurred in the load supported by the thicker surfaces. In fact, sometimes a slight decrease was indicated in the unit load supported.
- The bearing capacity of the subgrade of the 12-inch AC section was about 10 p.s.i. greater than that for the 3-inch AC surface, and that of the 6-inch AC surface was between the other two.

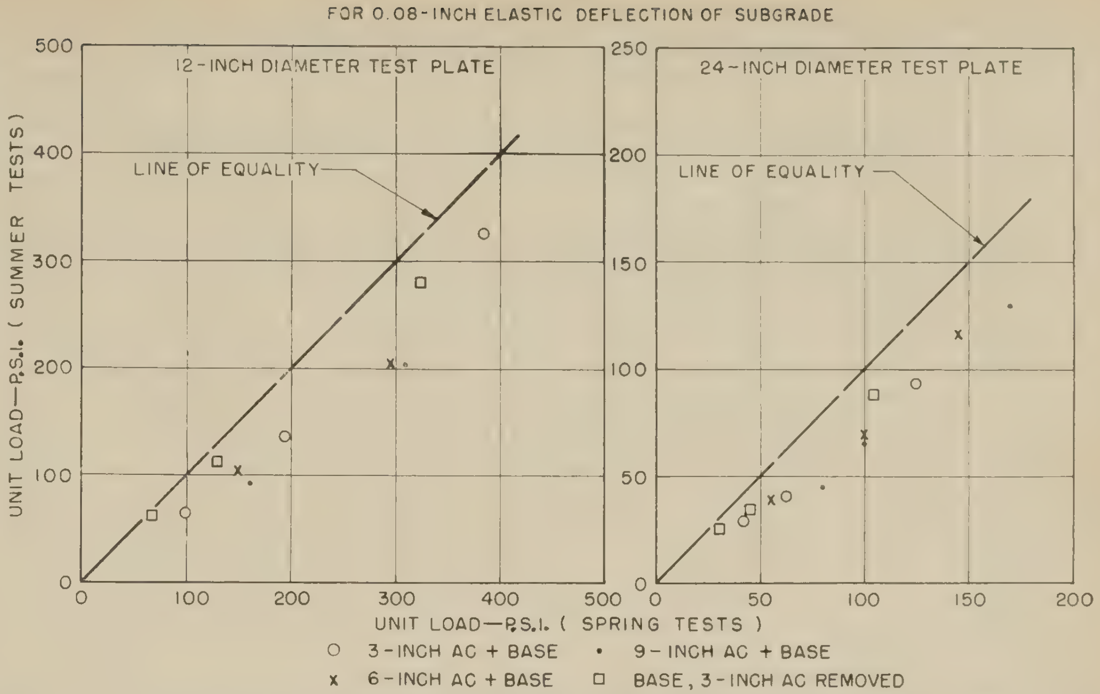


Figure 32.—Comparison of load support data obtained in the spring at 40° to 50° F. with data obtained in the summer at 75° to 95° F.

Plate Bearing Tests on Subgrade

Several series of plate bearing tests were conducted to obtain a measure of the bearing capacity of the subgrade soil of both the north and the south tangents. Two procedures were employed, namely, the Incremental-Repetitional Test and the Accelerated Test. The data from the first test procedure are presented in table 3 of reference (*3*). Tests were made directly on the subgrade, after the overlying courses had been removed, as described in the following paragraphs.

- Two series of tests were made in test section 1 (table 1) with plates 12, 18, 24, and 30 inches in diameter.
- One series of tests was made in test sections 1, 2, 3, and 4 with the plate 30 inches in diameter.

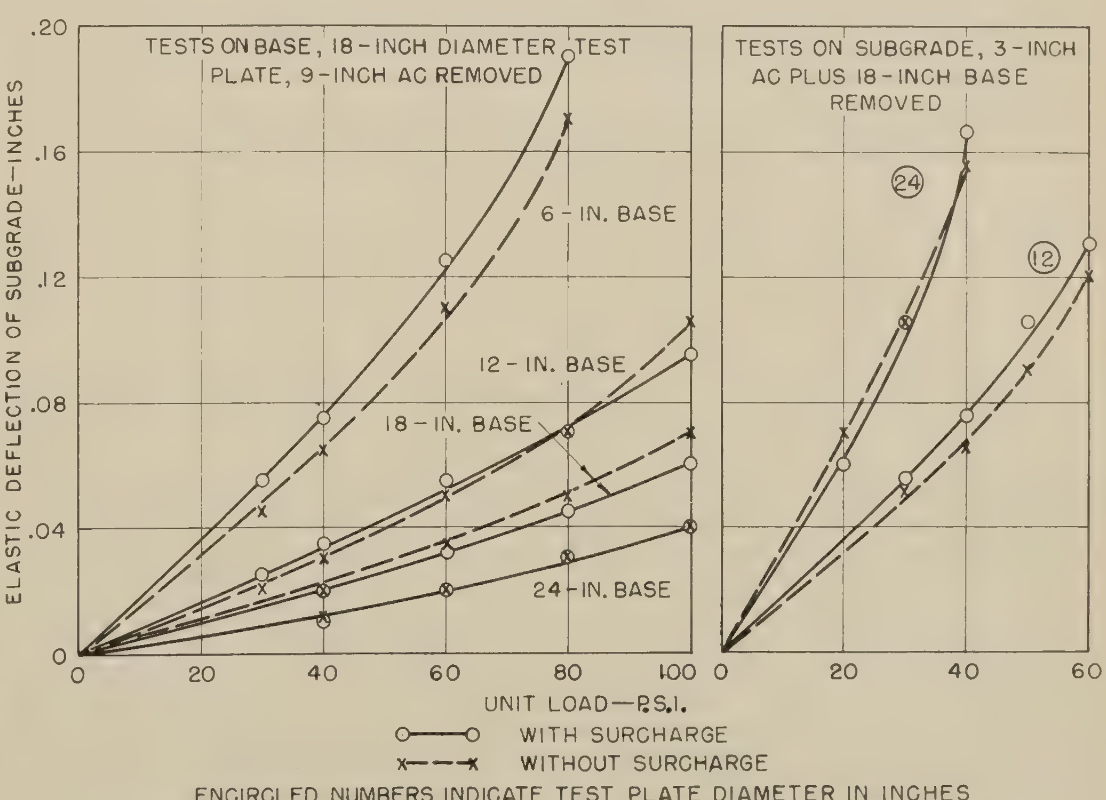


Figure 33.—Effect of surcharge based on elastic deflection of the subgrade.

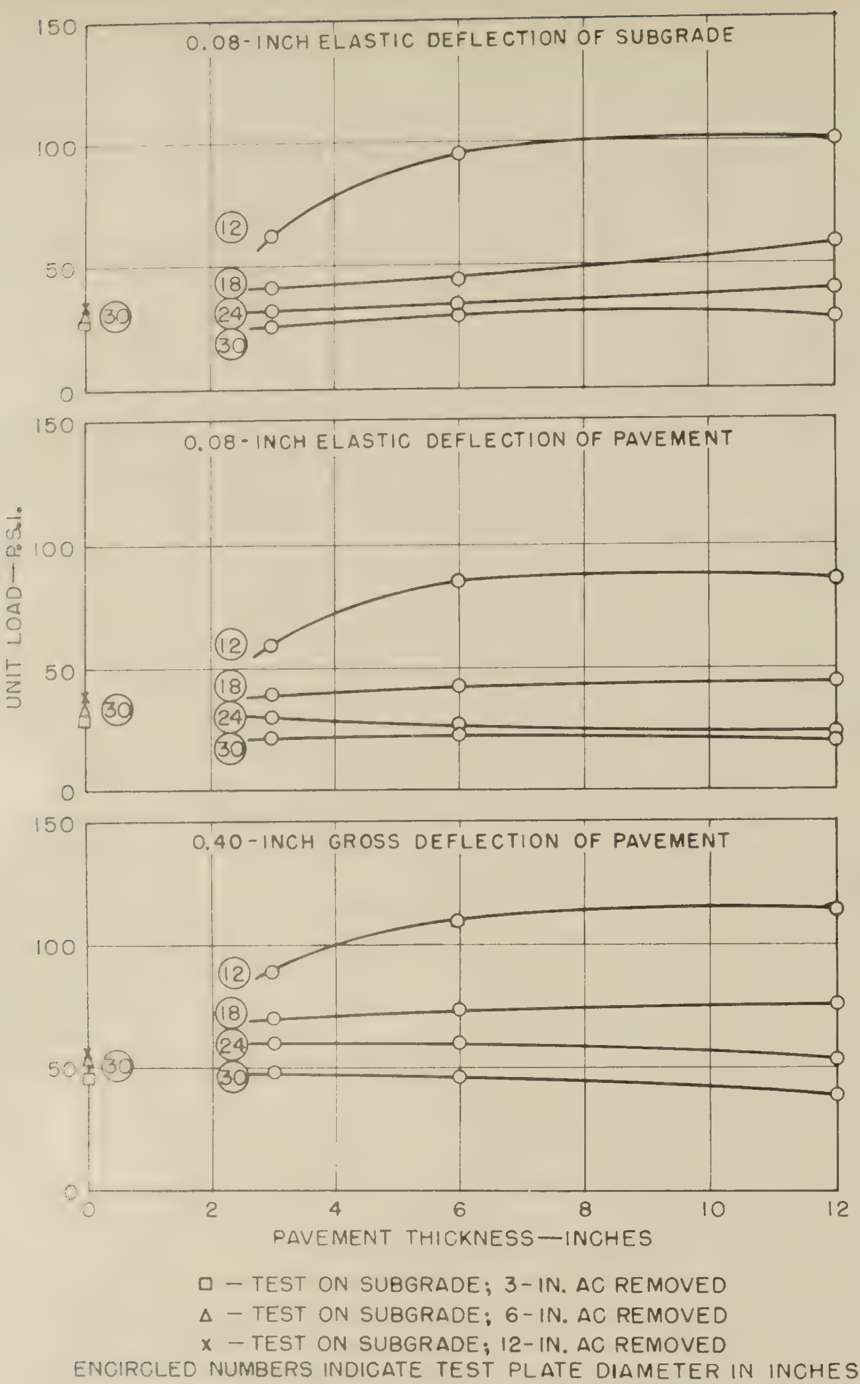


Figure 34.—Load-pavement thickness curves for sections of AC surface laid directly on the subgrade soil of the south tangent.

• One series of tests was made in test section 16 on the south tangent. Seven bearing plates whose diameters ranged from 12 to 84 inches were used. The three larger bearing plates in the group—42, 60, and 84 inches in diameter—were precast, circular concrete blocks that were 12 inches thick. The other four plates used were steel plates having diameters of 12, 18, 24, and 30 inches. The 8 inches of bank-run gravel surfacing that had previously been placed on the soil was removed before these tests were made.

The deflection data from the Accelerated Test presented in table 5 of reference (*3*) include information on one series of tests made in test section 3 with the 12-, 18-, 24-, and 30-inch plates; and for one series of tests made in test sections 1, 2, 3, and 4, with the 30-inch plate.

Curves in figures 35 and 36 were based on data from the plate load tests made on the subgrade soil beneath the pavement test sections on the north tangent. See tables 3 and 5 of reference (*3*) for tabulations. The unit load data have been plotted as a function of the diameter of the bearing plates in figure 35 and as a function of the perimeter-area ratio of the plates in figure 36. The data were plotted for both an elastic deflection of the material of 0.08 of an inch and for a gross deflection of 0.40 of an inch. From the Incremental-Repetitional Tests, the data shown for results of tests made with the 30-inch diameter plate are the average of results of 20 tests and for the other plates the average of results of three tests. For the results obtained from part *a* of the Accelerated Test, the data shown for the 30-inch plate are the average of results from 16 tests, and those shown for the other plates are averages of results from two tests.

Size of test plate

The curves in figures 35 and 36 indicate that the size of the test plate had a consistent and orderly effect on the bearing capacity of the soil. Curves in figure 35 tend to be curvilinear, and those in figure 36 are linear. The difference between the relations shown by these curves and obtained by the two test procedures cannot be definitely explained. Part *a* of the Accelerated Test provided for the application and release of the three load increments one time only, and in the Incremental-Repetitional Test each of four increments was applied and released five times. This difference in procedure might have accounted for some difference in the results although perhaps not to the extent indicated by the curves. Also, the difference might partially have been caused by the fact that the Accelerated Tests were made on the subgrade beneath the 18-inch base course section, and the results of all tests made on the subgrade, as well as on the pavement of this section, provided somewhat higher loads than results of tests on the 6-, 12-, and 24-inch base course sections. Moisture and density data also failed to account for these findings.

The data from the tests made on the subgrade soil of the south tangent are shown by curves in figures 37 and 38. The nature of the curves is the same as that of curves representing results of the tests made on the subgrade of the north tangent. Based on elastic deflection, the load results were slightly higher than those obtained from tests made on the north tangent. When based on gross deflection, the load results for tests made on the two tangents were practically equal. The curves in figures 35 and 37 indicate that the ability of the soil to support load decreased at an ever diminishing rate as the size of the test plate was increased up to the maximum size used in these tests (84 inches in diameter). Linear curves in figure 38 were fitted to the load data plotted as a function of the perimeter-area ratio of the plates. Although some of the points deviated somewhat from the curves as drawn, undoubtedly a linear load-deflection relation existed.

Values based on elastic deflection were computed for the soil constant, *k*, from the load figures for the 30-inch diameter plate plotted in the upper part of figure 35. Soil constant values of 200 and 183 pounds per cubic inch, respectively, were computed from results of the Accelerated and Incremental-Repetitional Tests. The corresponding soil constant value for the subgrade soil of the south tangent was 208 pounds per cubic inch. Thus it can be said that the subgrade soil of the test track, when existing with a moisture content of about 26 percent and a dry density of about 100 pounds per cubic foot, had a *k* value of about

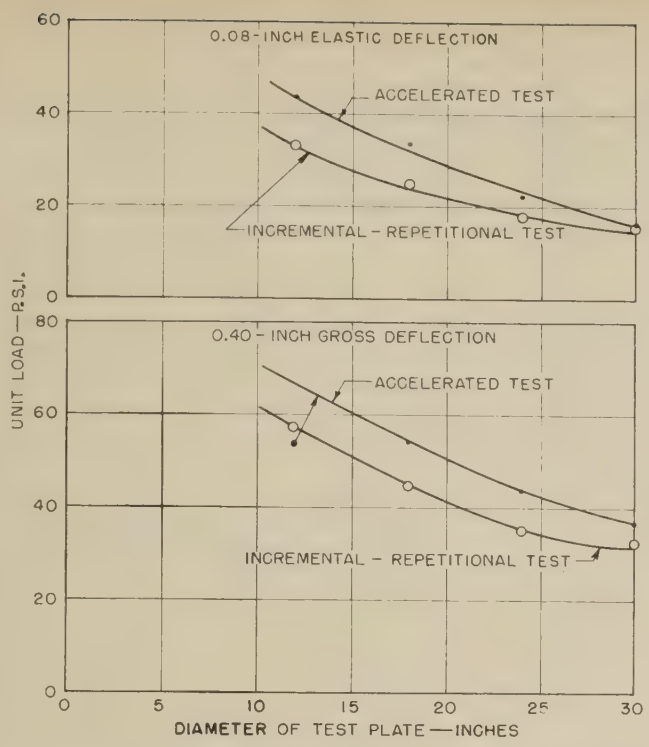


Figure 35.—Load-size of test plate curves for Incremental-Repetitional and Accelerated Tests on the north tangent subgrade soil.

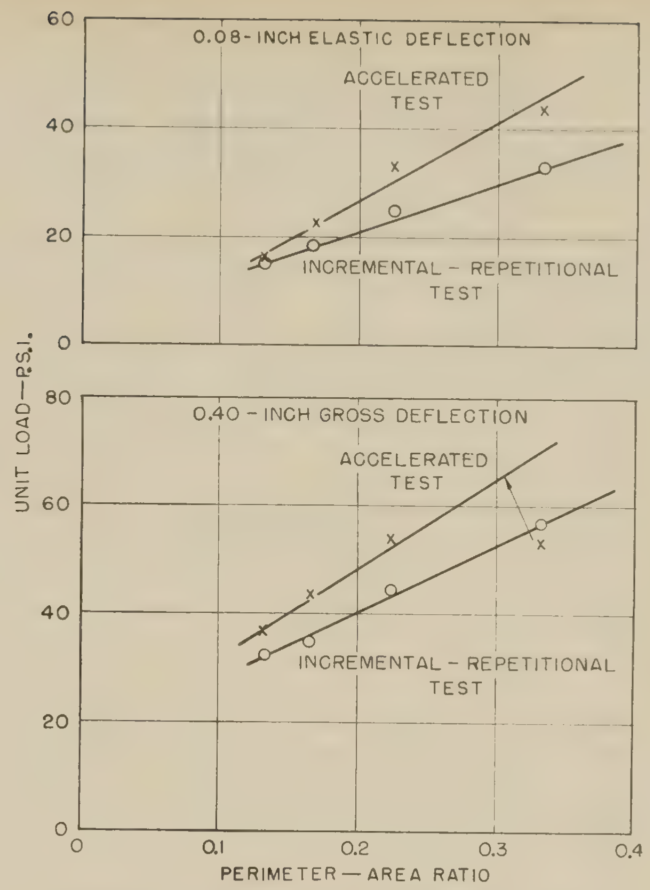


Figure 36.—Load-perimeter area ratio curves for Incremental-Repetitional and Accelerated Tests on the north tangent subgrade soil.

200 pounds per cubic inch. The laboratory CBR value of the subgrade soil was reported to be about 2 in reference (*3*).

The scheduled program of Repetitional Tests was limited to tests on the surface and base course of the pavement sections. However, enough tests of this type were made on the subgrade to determine whether, under direct loading with comparatively small unit loads, the soil would act elastically in the same manner as when the load was transmitted to it through the pavement structure. Loads varying in intensity from 5 to 20 p.s.i. were applied repetitiously through plates having diameters of 18, 30, and 42 inches, respectively.

In these Repetitional Tests on the subgrade, the elastic deflections were considerably less than the permissible elastic deflections that occurred in tests on the pavement. Nevertheless, in no instance did the subgrade soil attain a condition of completely elastic behavior during these tests. Each application of the load produced some additional permanent movement. The same type of difference in the behavior of a fine-grained soil that had been loaded directly through rigid plates and indirectly through a pavement structure has been observed previously (*7*). Differences in the elastic behavior of the soil are believed to have been caused by the manner in which the load was distributed to it. When the load was applied directly to the subgrade through a rigid plate, the highest contact pressure occurred around the peripheral area of the plate. However, when the load was applied to the subgrade through the pavement structure, the contact pressure tended to diminish outward from a maximum on the axis of loading.

Application of the Test Data

The results of the Repetitional Tests indicated that a limit exists as to the amount of repeated deflection that fine-grained subgrade soils can sustain without the pavement suffering progressive movement or deformation. Therefore, application of test data analyzed in this article to the design of new pavements would necessitate the determination of the permissible elastic deflection of the soils that would compose the subgrade. During the Hybla Valley investigation, the soil loaded directly through rigid plates never developed the same degree of elasticity that the test indicated occurred when the load was distributed through an overlying pavement structure. The design of pavements to be constructed on other types of soils would therefore require construction and testing of trial sections to determine the elastic deflection of the subgrade soil. The necessity for construction of such test sections would be obviated and data presented here would be useful if some type of load bearing test were to be developed by which the same reaction could be obtained when the subgrade soil was tested directly as when it was tested by having the load distributed through the pavement structure.

Also, if the data presented here were utilized in pavement designing, it would be necessary to ascertain whether asphalt pavements in service can tolerate without undue distress the magnitude of elastic deflections listed in table 4.

ACKNOWLEDGMENTS

The scope of the investigation at Hybla Valley required the assistance of many individuals. Contributions of engineers of the cooperating agencies and of Public Roads employees, who were intimately associated with the project, are acknowledged by the authors. W. N. Carey, Jr., represented the Highway Research Board and J. E Driscoll the Asphalt Institute. With the exception of K. E. Young, other employees of Public Roads who made major contributions to the investigation have retired, are deceased, or are employed elsewhere. These employees were: L. W. Teller who, with the assistance of E. C. Sutherland, directed the investigation. Field testing was supervised by R. J. Lancaster and K. E. Young was an engineering aid; F. R. Olmstead directed the sampling and testing of soils and aggregates.

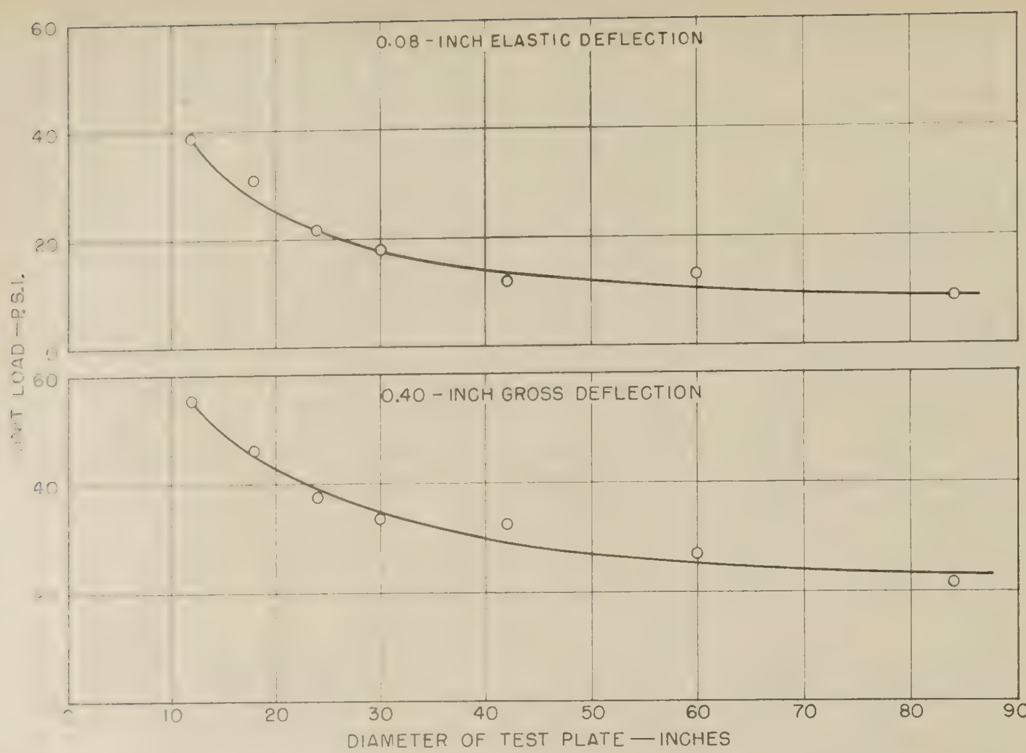


Figure 37.—Load-size of test plate curves for Incremental-Repetitional Tests on the south tangent subgrade soil.

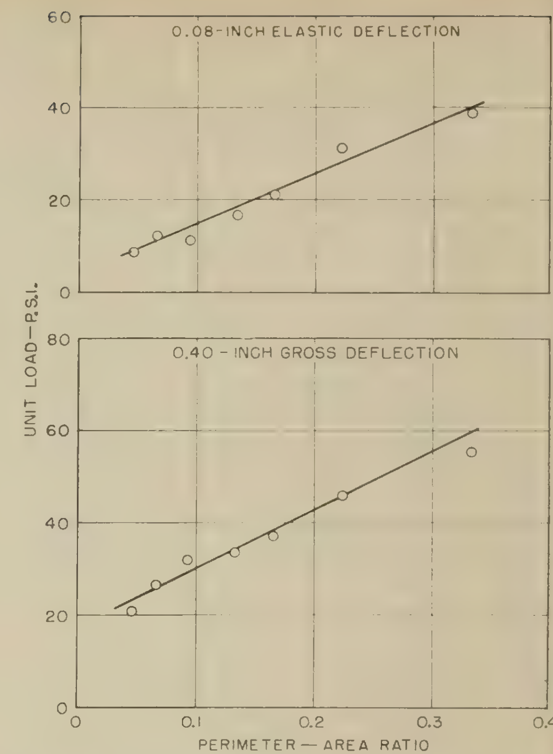


Figure 38—Load-perimeter area ratio curves for Incremental-Repetitional Tests on the south tangent subgrade soil.

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Shear Loads on Pavements

BY THE PHYSICAL RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

By[1] EDWARD S. BARBER,
Highway Research Engineer[2]

Stresses caused by shear loads should be considered when pavements are being designed, according to information presented in this article. Often, only the stresses caused by normal loads are considered when a stress analysis is being prepared for a pavement design. Information presented in this paper shows that stresses induced when vehicles stop may be more critical than had heretofore been realized, and that under static vehicles the effective vertical stress is increased by inward acting shear stresses caused by pneumatic tires.

Introduction

STRESS analysis of pavements is often based on normal loads alone; however, certain shear loads should not be ignored. In this article, two sources of applied shear stresses are considered—those produced by a pneumatic tire under a vertical load, and those caused by a component of the total load parallel to the surface of contact, which generally is considered to be horizontal.

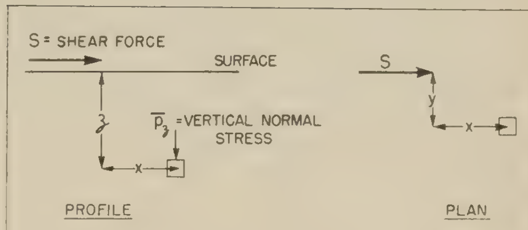
Conclusions

From the information presented in this article, it is concluded that stresses caused by static loads and strong horizontal loading should be considered when a flexible pavement is being designed. An exact analysis of anticipated stresses must include consideration of these two factors, either of which causes stresses having a more critical effect on the bearing capacity of a pavement than stresses caused by a vehicle moving at constant velocity along a horizontal tangent. Under the static load exerted by pneumatic tires, the shear stresses often cause the development of a critical creep of the pavement. Critical stresses in a pavement also are greatly increased and the bearing capacity greatly decreased when acceleration or deceleration of a vehicle produces strong horizontal loading components.

Inward Acting Shear Stresses

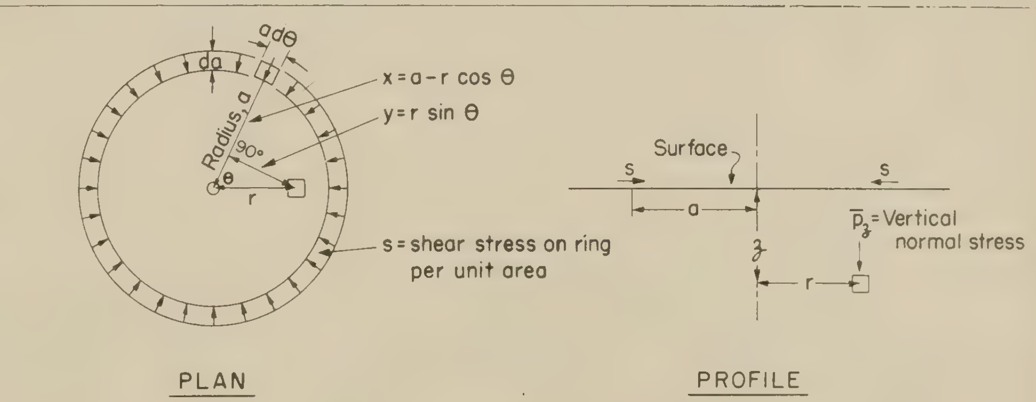
When a vertical load is applied to a pneumatic tire on a horizontal surface, the tensile prestress in the tire carcass caused by the inflation pressure is reduced, and shear stresses acting toward the center of the loaded area are produced on the surface of contact. These shear stresses are in the opposite direction from those produced under a solid rubber tire and their magnitude may be as great as the vertical pressure; this was shown by Markwick and Starks (*1*)[3] and by Bonse and Kuhn (*2*). A shear force applied to the surface produces interior stresses in different directions, including vertical; this is shown in figure 1. The formula presented by Westergaard (*3*) is for a homogeneous isotropic elastic material.

The shear stresses on the contact area under a static pneumatic tire can be considered to be made up of rings of shear stresses. When a ring of uniform, inward acting shear stresses is considered as being made up of shear loads on infinitesimally small points, the formula in figure 1 can be integrated to give the formula shown in figure 2. Figures 3 and 4 show graphs of vertical normal stresses at several points from a ring of shear stress calculated from this formula. Because the applied shear stress has no vertical component, the total vertical normal stress at any depth caused by shear stress is zero. The total vertical normal stress is the volume between the horizontal axis and the surface that is produced by rotation of any one of the curves shown in figure 3 about the vertical axis. By use of a numerical combination of concentric rings, the vertical normal stresses produced by a uniform distribution of inward



$$\bar{p}_z = \frac{3Sxz^2}{2\pi(x^2+y^2+z^2)^{5/2}}$$

Figure 1.—Vertical normal stress from applied shear force.



From formula for single force, vertical stress transmitted from ring is $\bar{p}_z = \frac{3sa\,da\,2z^2}{2\pi}\int_0^{\pi}\frac{(a-r\cos\theta)\,d\theta}{(a^2-2ar\cos\theta+r^2\cos^2\theta+r^2\sin^2\theta+z^2)^{5/2}}$

Letting $a/z=A$, $r/z=R$, $A/R=H$ & $\frac{1+A^2+R^2}{2AR}=J$, $\bar{p}_z = \frac{3s\,RA\,dA}{\pi(2AR)^{5/2}}\int_0^{\pi}\frac{(H-\cos\theta)\,d\theta}{(J-\cos\theta)^{5/2}}$

Integrating, substituting $\Psi = 90 - \theta/2$ and rearranging, $\bar{p}_z = \frac{s\,dA}{2\pi\,(AR)^{3/2}(J^2-1)^2\sqrt{2/(1+J)}}\left[(J-1)(J-H)K+(4JH-J^2-3)E\right]$

where K and E are respectively complete elliptic integrals of the first and second kinds with modulus (k) equal $\sqrt{2/(1+J)}$.

Figure 2.—Vertical normal stress from ring of inward acting shear stress.

[1] Presented at the International Conference on Structural Design of Asphalt Pavement, University of Michigan, Ann Arbor, Mich., August 1962.

[2] Mr. Barber is also Associate Professor of Civil Engineering at the University of Maryland, College Park, Md.

[3] The references indicated by italic numbers in parentheses are listed on page 144.

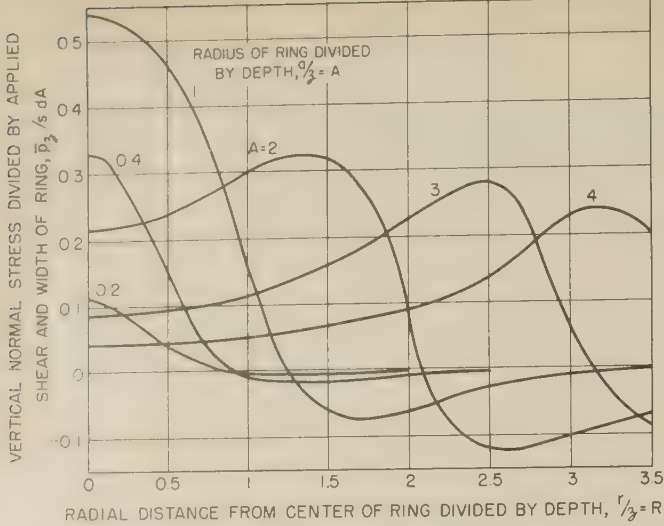


Figure 3.—Vertical normal stress at different radial distances from shear on ring.

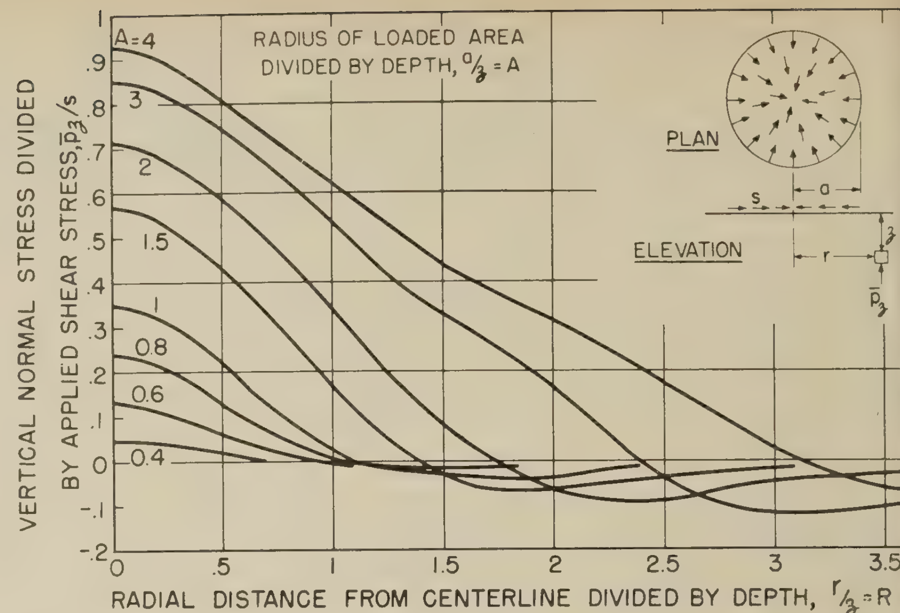


Figure 5.—Vertical normal stress from uniform inward shear over circular area.

...ing shear stresses over a circular area have been calculated; these calculated vertical normal stresses are shown in figure 5.

Figure 6 shows the vertical normal stress under the center of a circular area to which several different distributions of inward acting shear stresses were applied. The parabolic distribution approximated the measured applied stresses but, for simplicity, a uniform distribution was used for this study. Stresses, measured by McMahon and Yoder (*4*) and by Foster and Fergus (*5*), under vertically loaded plates approximated those that were obtained by the application of the theory of elasticity in which only vertically applied loads were considered. The measured stresses caused by vertically loaded pneumatic tires generally were somewhat greater than those calculated for the stresses from the vertical load alone. Spangler (*6*) measured vertical pressures under a pavement that were greater than those applied at the surface. The increase in vertical stress obtained in this study when the inward acting shear stresses were included in the calculation is shown in figure 7. Therefore, the increase from inward acting shear stresses should be considered when studies are made of the effect that pavement thickness has on transmitted stresses. This concentration of vertical deflection caused by shear stresses from different sources has been reported by Barber and Sawyer (*7*) and is illustrated in figure 8.

Shear Stresses in One Direction

Shear stresses in one direction between tire and pavement surface may be produced in several ways. Tangential forces up to about 10 percent of vertical loads may be produced by longitudinal grades or superelevation. The ratio of tangential to normal loads is limited by the coefficient of friction between tire and surface; this coefficient often reaches 0.8. This ratio occasionally is reached when wind acts on a

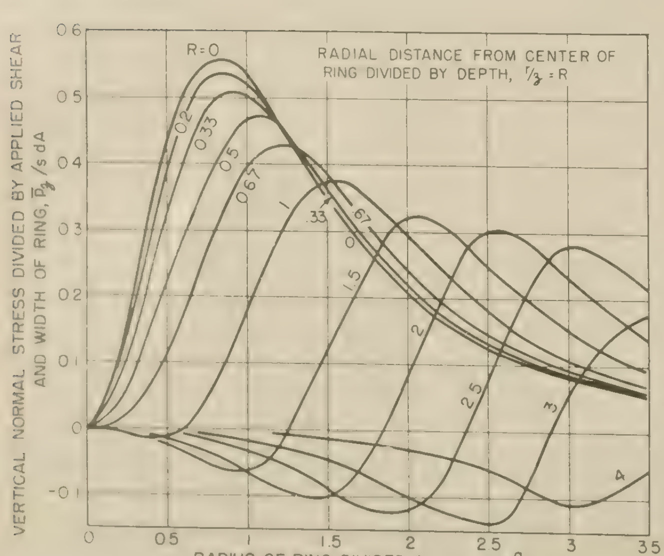


Figure 4.—Vertical normal stress from shear on rings of different radii.

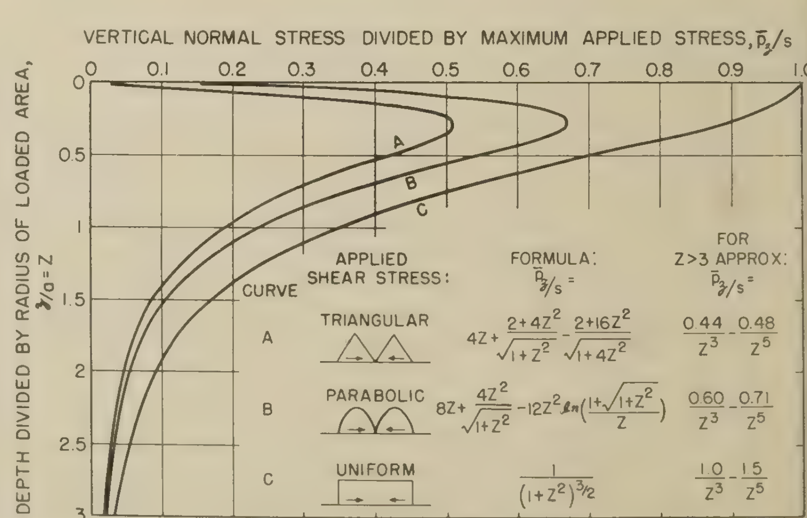


Figure 6.—Vertical normal stress under center of different inward shear stresses on circular area.

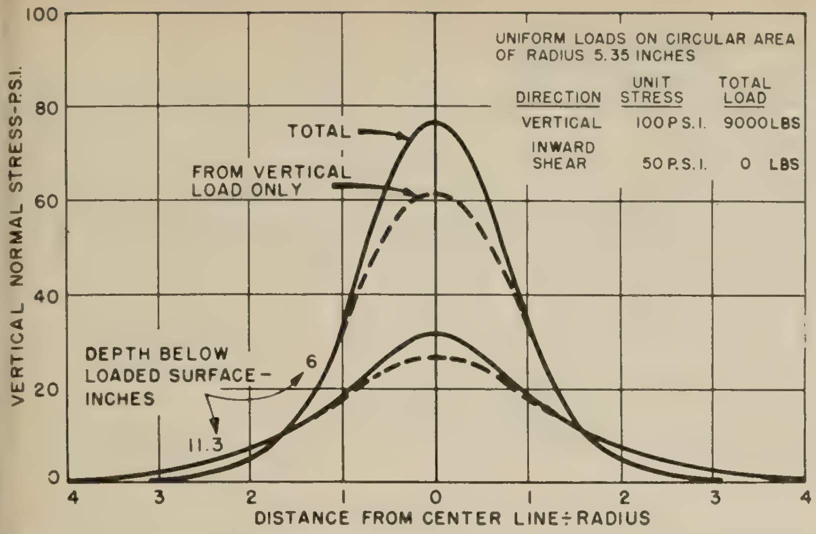


Figure 7.—Combined vertical stress from vertical and inward shear loads.

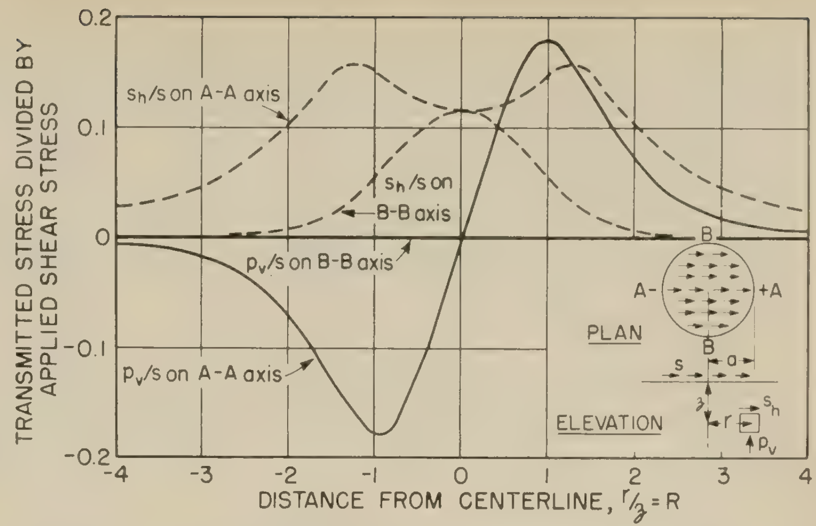


Figure 9.—Lateral distribution of stresses from shear stress in one direction at surface, $A/Z=1$

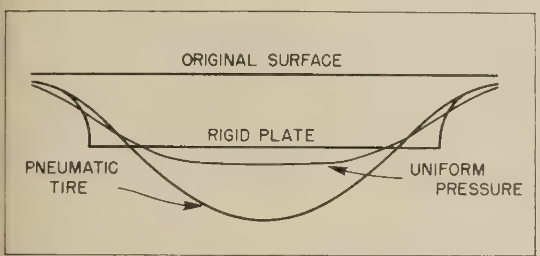


Figure 8.—Measured surface displacements of soil under different types of equal vertical loads.

vehicle, sometimes is reached when a vehicle is stopped, and may be reached when velocity of a vehicle is constant on a horizontal curve. On a curve having a 75-foot radius and no superelevation, a vehicle traveling at a speed of 30 miles per hour develops a horizontal force that is 0.8 of the vertical force. In addition to the foregoing factors and conditions, the distribution of vertical loads is affected by the vehicle's spring oscillation. Figure 9 shows interior vertical normal and horizontal shear stresses caused by application to a circular area of a uniform shear stress in one direction, these stresses are in addition to the stresses caused by the applied normal loads.

The stresses shown in figure 9 were derived from a table of stresses produced by a uniform vertical stress on a circular area (*8*) by the relationships given in the following statements.

p_v/s=the component of horizontal shear stress in the direction of s, divided by the applied vertical stress.

s_h/s=the horizontal normal stress (for Poisson's ratio=0.5) in the direction of s, divided by the applied vertical stress.

The differences in the maximum stresses on a horizontal plane at different depths are shown in figure 10. The horizontal shear

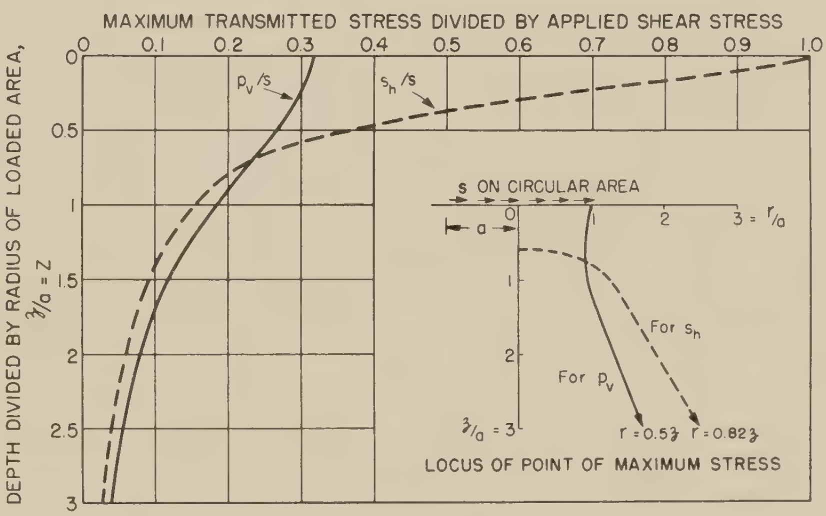


Figure 10.—Maximum stress at different depths from shear in one direction at surface.

$$q = \left(1 - \frac{i}{1.6}\right)^2 cN_c + \left(1 - \frac{i}{f}\right)^2 waN_a$$

Where, q = Bearing capacity, p.s.i.
i = Ratio of horizontal to vertical load
w = Unit weight, p.c.i.
a = Radius of loaded area, inches
f = Coefficient of internal friction
N_c & N_a = Functions of f, see table below

f	N_c	N_a
0	7.4	0.0
0.1	9	0.1
0.2	14	0.4
0.3	19	1.2
0.4	27	3.0
0.5	36	6.6
0.6	53	13
0.7	75	26
0.8	106	51
0.9	156	102
1.0	224	192

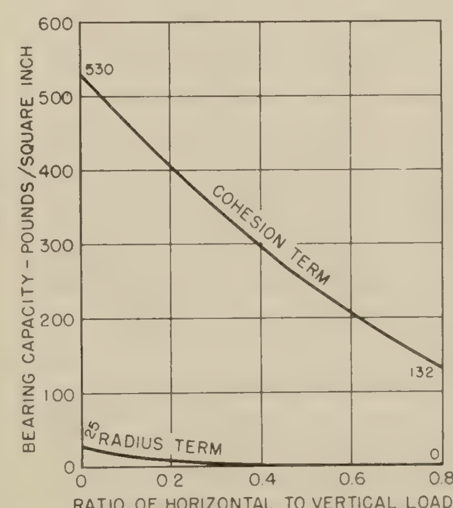


Figure 11.—Effect of horizontal force on bearing capacity.

stress is an important factor that affects the stability of layered systems, and its effect should be considered carefully for construction planned at locations where the bond between layers may be critical.

The information presented in the foregoing paragraphs should be useful when overstress of a few points must be considered. But when the overall failure of a pavement is to be considered, the theory of plasticity should be used to calculate the bearing capacity.

The effect of horizontal load components on bearing capacity developed by Meyerhof (*9*) is shown by the formula in figure 11; this illustration has been included here because the values shown for the coefficient of friction and its function might apply to the bearing capacity of bituminous pavement material. A bearing capacity of $530+25=555$ p.s.i. under a vertical load will be reduced to $132+0=132$ p.s.i. at impending skidding of a vehicle. The 132 p.s.i. is the resultant effect of vertical and horizontal applied stresses. When horizontal and vertical stresses are applied together, their combined effect must be considered in any analysis of bearing capacity of a pavement.

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Relation of Absolute Viscosity of Asphalt Binders to Stability of Asphalt Mixtures

(*Continued from page 152*)

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(*10*) *Stability Experiments on Asphaltic Paving Mixtures*, by W. J. Emmons, PUBLIC ROADS, vol. 14, No. 11, January 1934, pp. 197–211, 218.

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Relation of Absolute Viscosity of Asphalt Binders to Stability of Asphalt Mixtures

*BY THE PHYSICAL RESEARCH DIVISION
BUREAU OF PUBLIC ROADS*

By[1] J. YORK WELBORN, Chief, Bituminous and Chemical Branch, WOODROW J. HALSTEAD, Supervisory Chemist, and ROBERT E. OLSEN, Research Engineer

This article presents the results of laboratory compressive strength tests made to show the relationship of asphalt viscosity to the stability of pavement mixtures. A summary of the more important conclusions drawn by other authors in previously reported research is included in appendix I.

The results reported here illustrate the significant effect of the type and gradation of aggregate on compressive strengths of asphalt mixtures and the variation of strength that can occur at different temperatures for mixtures made with asphalts of the same penetration grade but with different viscosity-temperature susceptibilities. When comparisons were made at the same temperature, mixtures of a particular asphalt and the crushed stone aggregate used in these studies had significantly higher strengths than mixtures of the same asphalt and the gravel aggregate, which in turn had higher strengths than the mixtures made with sand. Such differences in strengths of the crushed stone, gravel, and sand mixtures were indicated to be more significant at temperatures around 140° F. than at lower temperatures.

When comparisons of compressive strengths were made on the basis of equal absolute viscosities, the differences caused by the three types of aggregates were still significant, but the strength of mixtures made with the same type and grading of aggregate and the same percentage of asphalt were equal for equal viscosities regardless of the source and grade of the asphalt.

Introduction

THE MAJOR goal of the asphalt paving technologist is to design asphalt paving mixtures that will have sufficient stability to resist displacement of the asphalt pavement in the form of shoving or rutting under traffic and also have the proper amount of flexibility, skid resistance, and durability. Many research studies have been conducted to evaluate such factors as the type, quality, and gradation of aggregates, and the character and amount of bituminous binder and construction variables that affect the quality and performance of the pavement.

The consistency of the asphalt binder in paving mixtures is one of the most important variables that affects performance. Consequently, the penetration test and other empirical consistency measurements were developed. Since then it has been common practice to specify certain penetration grades of asphalt for paving mixtures for use under specific conditions of traffic and climate. Initially, the grade of asphalt was selected on a trial and error basis; the lower penetration materials were used in warm areas and for heavy traffic conditions, and the higher penetration asphalts were used in cold areas and for lighter traffic conditions.

During the 1930's, the great amount of cracking in pavements precipitated a trend toward the use of softer asphalts (higher penetration) in sheet asphalt and asphalt concrete construction in many areas of the country. Since that time, laboratory stability tests have come into general use for designing paving mixtures. Using these tests, the researcher and engineer have developed a much better knowledge and understanding of the properties of aggregates, mineral fillers, and bituminous binders that influence the stability of various types of paving mixtures. Several research studies that show the effect of asphalt consistency on stability in laboratory tests have been reported.

A review of the literature reveals that these studies generally agree with respect to the qualitative relationships involved. However, the quantitative effects of type and gradation and consistency of the asphalt binder on stability have not been well defined. Consistency, measured by both penetration and viscosity, has been used in the various studies. Some authors have indicated that for a given mixture no difference in stability should occur when asphalts of the same consistency are used. However, a number of others have reported that other properties of the binder, as well as consistency, have a significant effect on stability. No specific information has been found that showed the relative effect of asphalts of different viscosity-temperature or shear-susceptibility characteristics in mixtures made with different aggregates where internal friction differed widely. The study reported in this article was undertaken to obtain such information. The relation shown is the effect of viscosity on the stability of pavement mixtures measured by laboratory test. No attempt was made to correlate the test data with characteristics of the pavement in place, but it is believed that the relative effects indicated will serve as valuable criteria for further studies on the design of mixtures in the laboratory and for predicting the performance of asphalt pavements in service.

This article also includes summaries of some of the more important research reports, which were reviewed to provide the background information for this study. These summaries are given in appendix I and are arranged in approximate chronological order to provide a general picture of the development of knowledge concerning the basic relationships involved.

[1] Presented at the 65th annual meeting of the American Society for Testing and Materials, New York, N.Y., June 1962.

Table 1.—Source, method of refining, and test characteristics of asphalts

Asphalt sample	Source of crude	Method of refining	Penetration, 100 g., 5 sec. at 77° F.	Ductility, 5 cm./min. at 77° F.	Softening point	Furol viscosity at 275° F.	Specific gravity at 77° F.	Thin-film oven test, ⅛-in. film at 325° F., 5 hrs.: Loss [1]	Tests on residue: Softening point	Tests on residue: Ductility, 5 cm./min.	Tests on residue: Percent of original penetration
				Cm.	*°F.*	*Sec.*		*Percent*	*°F.*	*Cm.*	*Percent*
3	Mexico	Atmospheric steam distillation.	87	248	121	318	1.037	0.55	134	138	60
13	Venezuela	Unknown	89	175	120	255	1.033	.18	132	175	62
38	Midcontinent	Vacuum distillation to 225–275 penetration, blown at 480° F.	90	170	120	189	1.000	.00	129	120	62
56	Kansas	Vacuum distillation.	86	166	121	180	1.004	+.03	139	13	62
69	Texas	Steam-vacuum distillation.	87	112	116	120	1.029	+.02	125	178	61
100	California	Steam distilation.	88	245	120	228	1.031	1.03	136	125	49
121	Mexico	Steam distilation.	57	>250	128	431	1.039	.29	140	100	65
154	Texas	Steam-vacuum distillation.	63	212	122	144	1.033	+.04	127	176	62
196	Mexico	Steam distilation.	135	184	113	250	1.034	.93	129	235	50
226	Texas	Steam-vacuum distillation.	126	118	109	86	1.023	.01	118	120	57

[1] + sign indicates gain in weight.

Materials Used

The asphalts used for the tests reported here were selected from the group included in the studies of the properties of highway asphalts produced in the United States reported by the Bureau of Public Roads in 1959 and 1960 (*1*, *2*). Table 1 provides information, which was given in earlier reports, on the sources of the asphalt, methods of refining, and some of the more important physical characteristics of the asphalts. The same identification number for each sample has been used in this article to permit convenient cross-reference. Table 2 shows the composition of the selected 85–100 penetration asphalts and also the changes in composition during aging tests conducted by Rostler and White, which they reported at the 1962 meeting of the Association of Asphalt Paving Technologists (*3*). The composition shown was based on the Rostler-Sternberg method in which the asphaltenes are first precipitated with normal pentane and the dissolved fractions are treated successively with sulfuric acid of specified strength. It is believed that the asphalts selected for this study, and described in the following paragraphs, had sufficiently different characteristics so that their behavior would provide a general cross-section of all types of commercial-grade asphalt materials.

The series of Texas asphalts, samples 154, 69, and 226 were respectively 60–70, 85–100, and 120–150 penetration grades from the same producer, and they represented the most temperature-susceptible materials found in the previous studies, where three grades were available from the same source. Some 85–100 penetration grade asphalts from California sources have lower furol viscosities at 275° F. than sample 69, the 85–100 material in this series, but the difference is not great.

The series of Mexican asphalts, samples 121, 3, and 196, were 60–70, 85–100, and 120–150 penetration grades and represented the materials with the lowest viscosity-temperature susceptibility tested in the earlier study.

The other four asphalts selected for this study were of the 85–100 grade: (1) Sample 13, a Venezuelan material, was typical of much of the asphalt used on the east coast. (2) Sample 38 was a midcontinent material that was highly resistant to hardening in both the microfilm and thin-film tests reported to ASTM in June 1961 (*4*), and this material had a relatively high shear susceptibility. (3) Sample 56 material also had a high shear susceptibility, and the ductility of its thin-film residue was very low (*4*). Although Rostler and White (*3*) reported that their test showed this material had excellent abrasion resistance, performance of asphalt having similar characteristics and believed to have been obtained from the same source was very poor. (4) Sample 100 was a California asphalt, which had a relatively high loss and a low percentage of retained penetration in the thin-film test.

The aggregates used in this study were selected to give test mixtures of three distinct types having widely different stability characteristics. These are designated in this article as sand, gravel, and stone mixtures. The aggregate for the sand mixture was a blend of concrete and sheet asphalt sands and limestone dust. The aggregate for the gravel mixture was composed of uncrushed gravel, natural sand, and limestone dust. The aggregate for the stone mixture was entirely crushed stone, including the material passing the number 200 sieve. The compositions of the test mixtures, including the type and gradations of the aggregates are shown in table 3. Although the gradations chosen for each type of aggregate were generally within limits that are used for actual pavement construction, it should not be assumed that these gradations represent the best combinations for each type. The primary basis for selection was to obtain aggregate systems having low, medium, and high levels of internal friction.

Table 2.—Composition of 85–100 penetration asphalt [1]

Asphalt sample [2]	Composition [3]: A	N	A_1	A_2	P	$\frac{N+A_1}{P+A_2}$	Durability rating
Original	30.7	16.0	20.6	23.7	9.0	1.12	2
Mixed	34.3	16.9	18.1	22.2	8.9	1.14	
Aged	36.0	18.3	16.2	21.5	8.0	1.17	
13:							
Original	28.8	22.5	23.0	18.9	6.8	1.77	5
Mixed	32.0	23.5	18.5	19.1	6.9	1.61	
Aged	36.0	24.8	17.1	15.5	6.6	1.90	
38:							
Original	20.3	18.4	23.8	24.8	12.7	1.13	2
Mixed	24.6	16.2	21.9	26.7	10.6	1.02	
Aged	30.2	16.5	19.7	23.8	9.8	1.08	
56:							
Original	20.3	12.1	21.0	30.8	15.8	0.71	1
Mixed	22.7	12.4	21.3	29.7	13.9	0.77	
Aged	25.0	11.9	14.8	34.7	13.6	0.55	
69:							
Original	21.0	14.6	23.9	31.0	9.5	0.96	7
Mixed	22.7	20.4	23.0	24.4	9.5	1.28	
Aged	25.7	20.7	27.5	26.8	9.5	1.05	
100:							
Original	33.4	21.7	23.1	13.2	8.6	2.06	7
Mixed	35.6	21.7	19.7	16.3	6.7	1.80	
Aged	41.8	20.7	16.5	14.1	6.9	1.87	

[1] Taken from paper by Rostler and White presented at meeting of Association of Asphalt Paving Technologists, New Orleans, La., January 1962.
[2] Asphalt:
Original, as received.
Mixed, after mixing with Ottawa sand 6 minutes at 325° F.
Aged, after aging 7 days at 140° F.
[3] Composition determined by Rostler-Sternberg method:
A = Asphaltenes.
N = Nitrogen base resins.
A_1 = First acidaffins.
A_2 = Second acidaffins.
P = Paraffins.

Table 3.—Composition and aggregate grading of test mixtures

	Sand mixture	Gravel mixture	Stone mixture
Asphalt, aggregate basis	*Percent* 8.5	*Percent* 6.0	*Percent* 6.0
Aggregate type:			
Red Hill granite	----	----	100
Massoponax gravel	----	58	----
Potomac River sand	----	36	----
White Marsh concrete sand	60	----	----
Potomac River sheet asphalt sand	38.5	----	----
Limestone dust	1.5	6	----
Aggregate passing sieve:			
½ inch	----	100	----
⅜ inch	----	90	100
No. 4	100	62	82
No. 10	90	42	36
No. 20	72	32	----
No. 40	52	19	13
No. 80	17	9	9
No. 200	4	6	4

Variation of Compressive Strength with Temperature

Results of the compressive strength test, made at various temperatures on specimens 3 inches in diameter and 3 inches high in which each of the aggregates were used with each asphalt, are shown in table 4. The details of mixing, molding, and testing these specimens are given in appendix II. The compressive strength result reported for each asphalt at each test condition is the average for four individual specimens. The bulk specific gravity and the voids content of the mixtures given in the table also are averages for the group of four specimens tested. Figure 1 shows the effect of temperature on compressive strength of the three types of mixtures containing asphalts 3 and 69—the 85–100 penetration grade asphalts representing the extremes of temperature-susceptibility of the asphalts used in this study.

As expected, a large difference was noted in the compressive strength of the three types of mixtures, especially at the higher temperatures. For example, at 120° F. the sand mixture for asphalt 69 had a strength of 8 p.s.i.; the gravel mixture had a strength of 32 p.s.i. (interpolated); and the crushed stone mixture had a strength of 102 p.s.i. At lower temperatures the differences were less pronounced. The strength at 60° F. for asphalt 69 was: 327 p.s.i. for the sand mixture; 440 p.s.i. (interpolated) for the gravel mixture; and 486 p.s.i. for the stone mixture. Differences in the compressive strength of mixtures containing asphalt 3 were of about the same order of magnitude. The flatter slope of the plotted line for the crushed stone mixtures reflects the greater interlocking or internal friction present in the crushed stone aggregate.

The same type of aggregate mixtures made with asphalts of the same penetration grade but from different sources also had significantly different compressive strengths. The differences in strength caused by differences in asphalt source were greatest with the sand

Table 4.—Compressive strengths of mixtures made with different asphalts and aggregates

Asphalt sample	Type of aggregate [1]	Bulk sp. gr.[2]	Air voids,[2] percent	Compressive strength, p.s.i., at—[2]							
				40° F.	59° F.	60° F.	82° F.	95° F.	100° F.	120° F.	140° F.
154	Gravel	2.255	7.4	1,228	585	----	205	----	90	----	22
69	Sand	1.939	18.2	708	----	327	----	33	----	8	----
	Gravel	2.256	7.3	1,084	469	[3] 440	166	[3] 90	71	[3] 32	16
	Stone	2.365	8.1	1,383	----	486	----	169	----	102	----
226	Gravel	2.251	7.7	774	327	----	130	----	48	----	11
121	Gravel	2.275	6.7	999	547	----	252	----	136	----	50
3	Sand	1.953	17.7	558	----	273	----	55	----	21	----
	Gravel	2.272	6.7	785	396	[3] 390	192	[3] 125	106	[3] 59	38
	Stone	2.380	7.4	959	----	411	----	193	----	133	----
196	Gravel	2.270	6.9	676	320	----	146	----	75	----	23
13	Sand	1.955	17.4	----	----	298	----	48	----	14	----
	Gravel	2.273	6.2	----	----	428	----	122	----	54	----
	Stone	2.383	7.3	----	----	448	----	177	----	123	----
38	Sand	1.944	17.3	----	----	256	----	38	----	12	----
	Gravel	2.263	6.2	----	----	362	----	110	----	46	----
	Stone	2.355	7.6	----	----	382	----	188	----	122	----
56	Sand	1.941	17.2	----	----	249	----	72	----	29	----
	Gravel	2.267	6.4	----	----	366	----	139	----	78	----
	Stone	2.372	7.2	----	----	381	----	198	----	140	----
100	Sand	1.958	17.2	----	----	324	----	56	----	18	----
	Gravel	2.272	6.4	----	----	451	----	132	----	58	----
	Stone	2.383	7.3	----	----	554	----	197	----	130	----

[1] See table 3 for grading and composition of mixtures.
[2] Each value is an average of results for 4 test specimens.
[3] Interpolated strengths.

Table 5.—Viscosities of original and recovered asphalts, 85–100 grade, at various temperatures

Asphalt sample and test temperature	Viscosity [1]			
	Original asphalt	Recovered asphalt from—		
		Sand mixture	Gravel mixture	Stone mixture
° F.	*Poises*	*Poises*	*Poises*	*Poises*
3:				
40	1.2×10^{8}	----	1.9×10^{8}	----
59	8.7×10^{6}	----	2.0×10^{7}	----
60	8.3×10^{6}	2.0×10^{7}	----	2.0×10^{7}
77	1.1×10^{6}	3.4×10^{6}	3.0×10^{6}	2.8×10^{6}
82	5.2×10^{5}	----	1.8×10^{6}	----
95	1.3×10^{5}	4.7×10^{5}	----	3.9×10^{5}
100	8.5×10^{4}	----	3.4×10^{5}	----
120	1.5×10^{4}	5.5×10^{4}	----	3.9×10^{4}
140	3.3×10^{3}	----	1.8×10^{4}	----
13:				
60	1.1×10^{7}	1.7×10^{7}	1.5×10^{7}	1.7×10^{7}
77	1.2×10^{6}	2.4×10^{6}	2.1×10^{6}	2.2×10^{6}
95	2.3×10^{5}	3.0×10^{5}	2.4×10^{5}	2.7×10^{6}
120	1.8×10^{5}	3.0×10^{4}	2.4×10^{4}	2.4×10^{4}
38:				
60	1.0×10^{7}	1.5×10^{7}	1.5×10^{7}	1.3×10^{7}
77	1.2×10^{6}	2.9×10^{6}	2.1×10^{6}	1.9×10^{6}
95	1.3×10^{5}	3.3×10^{5}	2.0×10^{5}	2.3×10^{5}
120	1.3×10^{4}	4.0×10^{4}	2.0×10^{4}	1.6×10^{4}
56:				
60	8.0×10^{6}	1.3×10^{7}	1.9×10^{7}	1.6×10^{7}
77	1.7×10^{6}	3.8×10^{6}	3.7×10^{6}	3.7×10^{6}
95	2.1×10^{5}	6.5×10^{5}	4.6×10^{5}	6.3×10^{5}
120	3.4×10^{4}	7.0×10^{4}	6.8×10^{4}	9.3×10^{4}
69:				
40	5.2×10^{8}	----	1.2×10^{9}	----
59	1.8×10^{7}	----	3.2×10^{7}	----
60	1.6×10^{7}	3.0×10^{7}	----	2.6×10^{7}
77	1.0×10^{6}	2.9×10^{6}	2.1×10^{6}	2.6×10^{6}
82	5.7×10^{5}	----	1.1×10^{6}	----
95	1.1×10^{5}	2.6×10^{5}	----	2.7×10^{5}
100	6.6×10^{4}	----	1.0×10^{5}	----
120	8.3×10^{3}	1.8×10^{4}	----	2.0×10^{4}
140	1.5×10^{3}	----	2.1×10^{3}	----
100:				
60	1.1×10^{7}	2.4×10^{7}	2.3×10^{7}	2.2×10^{7}
77	1.2×10^{6}	3.1×10^{6}	2.4×10^{6}	3.3×10^{6}
95	1.9×10^{5}	3.8×10^{5}	3.5×10^{5}	3.9×10^{5}
120	1.9×10^{4}	3.3×10^{4}	3.6×10^{4}	3.5×10^{4}

[1] Shear rate 0.05 sec.$^{-1}$

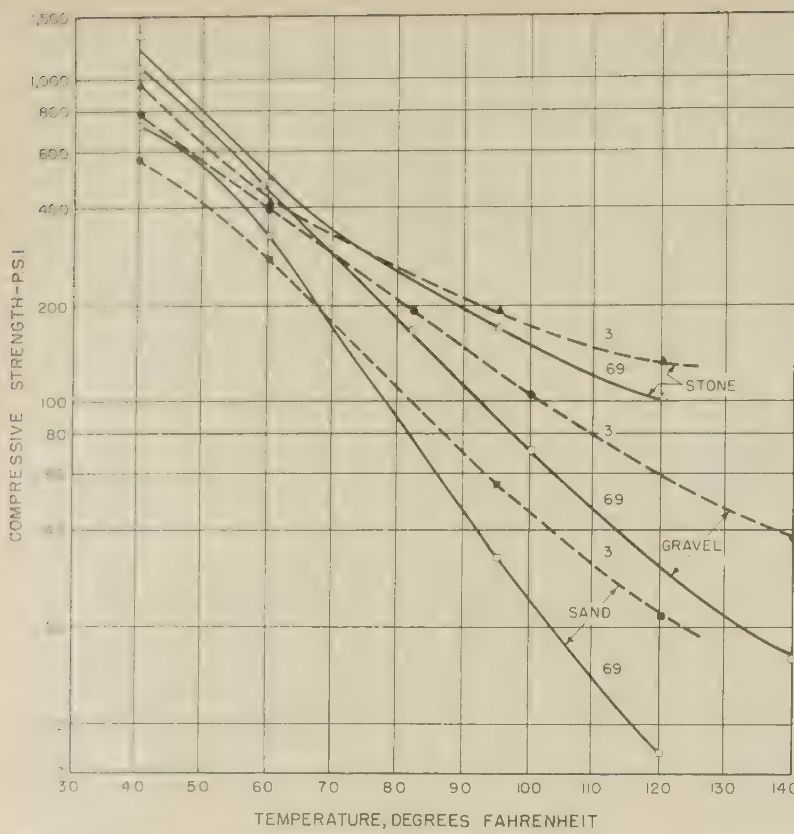

Figure 1.—Relation between log strength and temperature—different aggregates and asphalts.

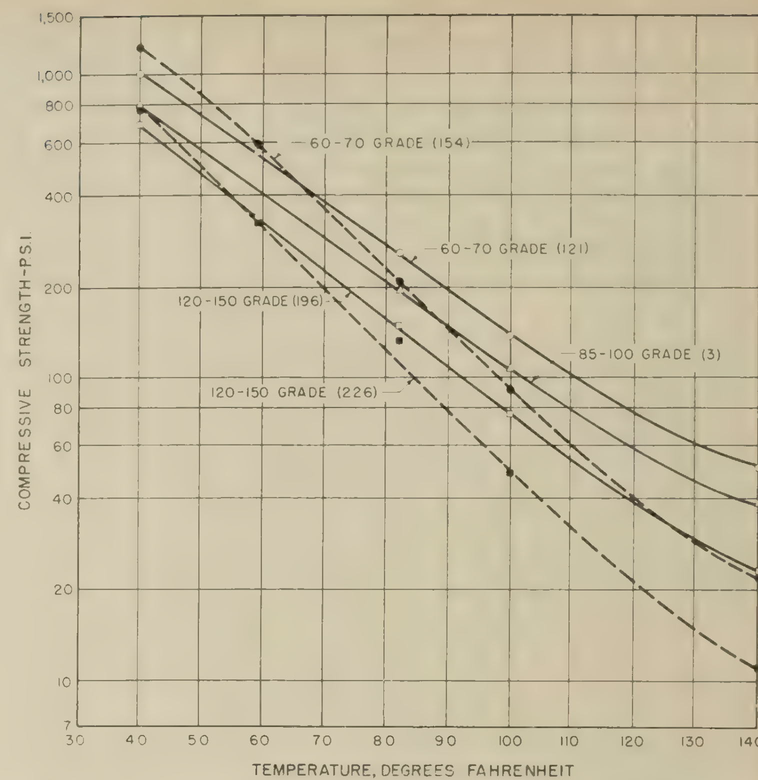

Figure 2.—Relation between log strength and temperature—asphalts of different grades.

mixture and least with the stone mixture. For example, the strength of the sand mixture at 120° F. containing asphalt sample 3 was 2.6 times the strength of a similar mixture containing asphalt 69. At this temperature the ratios of the strengths of the gravel and stone mixtures were 1.8 and 1.3 respectively for the same two asphalts.

The variation in compressive strength with temperature for gravel aggregate mixtures made with asphalts from the same source but of different penetration grades is illustrated in figure 2. The data plotted are for the mixtures containing 60–70 and 120–150 penetration grades of the Mexican and Texas asphalts, representing the extremes of temperature-susceptibility. The curve for mixtures made with the 85–100 Mexican asphalt is also included for comparison. As shown, the curves for different grades from a given source are substantially parallel but differ significantly in slope from the curves for mixtures made with corresponding grades of asphalt from a different source.

Table 6.—Viscosities of original and recovered asphalts, 60–70 and 120–150 grades, at different temperatures

Test temperature, ° F.	Viscosities [1] for 60–70 grade asphalts— Original	Viscosities [1] for 60–70 grade asphalts— Recovered [2]	Viscosities [1] for 120–150 grade asphalts— Original	Viscosities [1] for 120–150 grade asphalts— Recovered [2]
	Asphalt 121		Asphalt 196	
40	3.4×10^{8}	3.5×10^{8}	6.1×10^{7}	3.3×10^{9}
59	2.9×10^{7}	3.6×10^{7}	4.6×10^{5}	1.1×10^{7}
77	3.1×10^{6}	5.6×10^{6}	5.8×10^{6}	1.3×10^{6}
82	1.3×10^{6}	3.6×10^{6}	2.3×10^{5}	6.9×10^{5}
100	2.0×10^{5}	6.8×10^{5}	4.4×10^{4}	1.1×10^{5}
140	6.3×10^{3}	3.6×10^{4}	1.7×10^{3}	4.2×10^{3}
	Asphalt 154		Asphalt 226	
40	1.2×10^{9}	2.4×10^{9}	1.6×10^{8}	5.8×10^{8}
59	3.9×10^{7}	6.9×10^{7}	6.7×10^{6}	1.4×10^{7}
77	2.3×10^{6}	4.8×10^{6}	3.5×10^{5}	8.8×10^{5}
82	1.1×10^{6}	2.2×10^{6}	2.4×10^{5}	4.6×10^{5}
100	1.3×10^{5}	2.1×10^{5}	3.1×10^{4}	4.2×10^{4}
140	2.5×10^{3}	4.4×10^{3}	7.8×10^{2}	8.5×10^{2}

[1] For shear rate of 0.05 $sec.^{-1}$
[2] Asphalt recovered from gravel mixture.

The relative position of the curves for the mixtures containing 60–70 Texas asphalt sample 154, and those containing the 120–150 Mexican asphalt sample 196, are of interest. These mixtures had approximately the same compressive strengths at 140° F., but at 40° F. the strength of the mixture containing sample 154 was almost double that of sample 196. However, the mixtures containing asphalt samples 3 and 226 had the same strengths at 40° F. but widely different strengths at 140° F. (Sample 3 was the 85–100 Mexican asphalt and sample 226 the 120–150 Texas asphalt.) These comparisons illustrate the effect of the temperature-susceptibility of asphalts on mixture strengths.

The general relations shown in figures 1 and 2 are well known from the qualitative standpoint and will be exhibited by all types of mixtures, but the amount of change in strength will differ and depends on both the asphalt characteristics and the differences in aggregate such as grading, particle shape, etc. The degree of compaction of the laboratory specimen and the percentage of asphalt used also will influence the strength relationships obtained with each mixture composition. Because the effect of asphalt viscosity was the primary consideration in this study, no evaluation was made of the quantitative effect of other factors.

Variations of Compressive Strength with Viscosity

To show the extent to which the differences in stability of bituminous mixtures determined in the laboratory by the compressive strength test can be attributed to differences in viscosity of the contained asphalt, the asphalts were recovered from test specimens representing each type of mixture and their viscosities

Table 7.—Comparison of penetration of asphalts recovered from test specimens with penetration of original asphalts

Asphalt grade and sample number	Penetration, 100 g. 5 sec., 77° F.			
	Original asphalt	Asphalt recovered from—		
		Sand mixture	Gravel mixture	Stone mixture
85–100:				
3	92	61	63	63
13	88	62	66	65
38	89	63	68	70
56	78	54	61	57
69	88	53	61	56
100	90	54	59	58
60–70:				
121	66	--------	47	-------
154	61	--------	43	-------
120–150:				
196	127	--------	91	-------
226	145	--------	89	-------

were determined at several temperatures. The viscosities of the original asphalts also were determined over the same temperature range to measure the hardening that had occurred during the preparation and testing of the specimens.

All viscosities were determined with the sliding plate microviscometer. This instrument has been adequately described in a number of published reports (*14*, *15*). Determinations for each material were made at four temperatures covering the normal working range of the instrument. Although there were some exceptions, most of the determinations were made at 60° F., 77° F., 95° F., and 110° F. All viscosities were computed on the basis of a shear rate of 0.05 $sec.^{-1}$ These data were plotted on a viscosity temperature chart using the ASTM coordinates. The basis for this chart is log log viscosity in centistokes, which is essentially equivalent to centipoises for these materials, plotted against log of absolute viscosity in degrees Rankine (° F.+459.7). The viscosities reported in tables 5 and 6 were extrapolated or interpolated from the plotted lines based on the best straight line through the determined data points. Such extrapolated viscosities may not be exact for the extrapolations at the upper and lower temperatures (120° F. or 140° F. and 40° F.) but they are believed to be sufficiently precise for the purposes of this study.

The data in table 5 for the 85–100 penetration asphalts include viscosities of the original asphalts and of the asphalts recovered from mixtures made with each of the aggregates. All of the recovered asphalts had higher viscosities than the original asphalt; and, generally, the viscosity increase occurring with the use of each type of aggregate was not significantly different. The greatest difference occurred for asphalt 38. At 120° F. the viscosity of the material recovered from the sand mixture made with this asphalt was approximately double the viscosity of the material recovered from other aggregate mixtures; but at 60° F. the viscosities for all three recovered asphalts were essentially the same. Table 6 shows the data for both the original and recovered asphalts for the 60–70 and 120–150 grades. Asphalts of these grades were recovered from only the gravel mixtures.

Table 7 contains data that compares the penetrations of the original and recovered asphalts at 77° F. The penetration data provide a more familiar evaluation of the hardening occurring during the mixing and curing of the various specimens and show that the increase in consistency, measured by the penetration at 77° F., is equivalent to approximately a one-grade change in penetration. For the 85–100 grade materials, the original penetration at 77° F. varied from 78 to 92. Asphalt sample 56, which had a penetration of 78, was out of grade. The penetration of the recovered asphalts ranged from a low of 53 for the asphalt recovered from the sand mixture containing asphalt sample 69 to a high of 70 for the asphalt recovered from the stone mixture containing asphalt sample 38. Generally, the hardening during the mixing and curing of the sand mixtures, measured by penetration at 77° F., was greater than that for the gravel and stone mixtures.

In figure 3, the logarithm of the compressive strength at each temperature is plotted against the logarithm of the viscosity of the recovered asphalt at the same temperature. Regardless of the source or penetration grade of the asphalt used, the data points fall on three distinct curves, one for each type of aggregate mixture. The influence of the aggregate system in the mixtures is again illustrated by the different levels and slopes of the curves. Although there is some curvature in the plotted lines, small segments can be considered to be straight and the slopes of such segments can be used to indicate the ratio of change in strength to the change in viscosity. Examination of these slopes in the range of viscosity from 10^4 to 10^5 poises, which is approximately equivalent to viscosities at summer temperatures, shows that a 10-fold increase in viscosity produced approximately a 3 2-fold increase in strength for the sand mixture, a 2.4-fold increase for the gravel mixture, and a 1.5-fold increase for the stone mixture. Corresponding strengths for viscosities such as would exist for winter temperatures in the range of from 10^8 to 10^9 poises could not be estimated precisely. However, it is probable that further increase in viscosity produced by decrease in temperature would not cause as large an increase in strength. This tendency is indicated by the curvature of the plotted line for the sand mixture and, to a lesser extent, by the curvature of the plotted line for the gravel mixture. The data for the stone mixture was less definite, but it is believed that strengths at higher viscosities than obtained in this study would show similar behavior of the mixture.

Other researchers have studied the relation of asphalt viscosity to laboratory determined stability and some have indicated that a straight line is the result when log strength is plotted against log log viscosity. Summaries of three studies using this relation are included in appendix I; these studies were by Weetman and Hurlburt (*7*), Neppe (*8*), and Wood and Goetz (*9*). To examine this relationship, the data obtained in this study (tables 4, 5, and 6) were plotted as shown in figure 4. It should be noted that, although the horizontal axis of the plot shows viscosity in poises, the basis for locating the points was the log log viscosity in centipoises. The data for the gravel mixtures with all asphalts fall on a straight line over the entire viscosity range explored. The plotted data for the sand and stone mixtures made with all asphalts closely approximate straight lines for viscosities less than 10^7 poises, but for greater viscosities the lines show some curvature.

Some authors have indicated that factors other than viscosity have a significant effect on strength measured by laboratory stability tests. It is obvious from figures 3 and 4 that much of such an effect, if it existed, would be masked by aggregate systems having good inherent stability. Special attention therefore was directed to the results obtained with certain asphalts in the sand and gravel mixtures. Figure 5 shows viscosity and strength data for three asphalts replotted on the same basis used in figure 4. These data are for asphalts 3 and 69, which represented the extremes of temperature-susceptibility, and asphalt 56, which had the greatest degree of complex flow. Although tests were made at only three temperatures for the sand mixture prepared with each asphalt, there were some indications that the data points would form a slightly different line for each asphalt. Data were available at five temperatures for the gravel-mixtures for asphalt samples 3 and 69 and, as indicated, no difference in the plotted line could be detected. There was some indication that the three data points for the gravel-mixture curve for asphalt sample 56 formed a slightly different line than was obtained from data points for asphalt samples 3 and 69, but this difference might have been the result of experimental error.

When evaluating the significance of this difference, the possible effect of the shear rate used to determine the viscosity of the asphalts compared to the shear rate of the asphaltic film during the strength test should be considered. As previously stated, all of the viscosities reported in this study were based on a shear rate of 0.05 $sec.^{-1}$ This basis was selected because this shear rate is the one most commonly used with the sliding plate microviscometer. Also, this shear rate could be determined with relatively good precision for the tests made on these asphalts as it could be bracketed by actual test results. This rate of shear, however, did not correspond to the rate of shear in the mixture specimen during the strength test. Based on the average film thickness of the asphalt in the mixture and the speed of the test, the rate of shear in the compressive strength specimen at failure most likely was in the range of 1 $sec.^{-1}$ to 2 $sec.^{-1}$

Attempts were made to extrapolate the viscosity test data so that the apparent viscosity could be calculated on the basis of

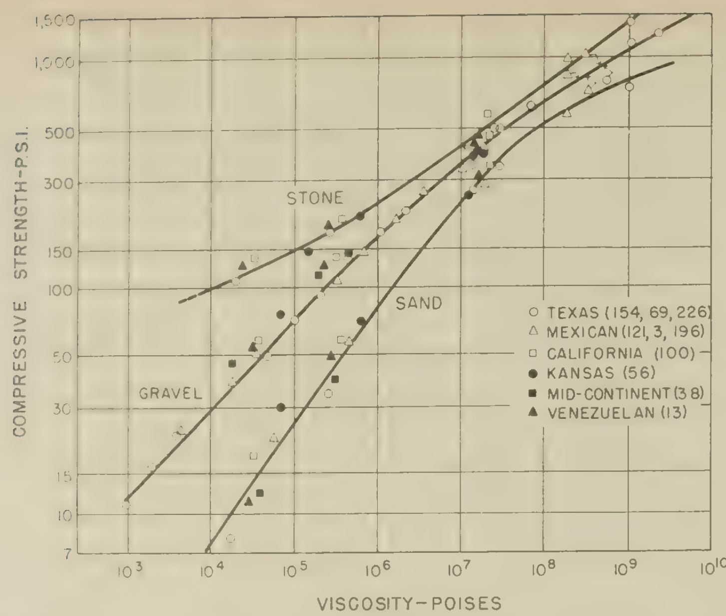

Figure 3.—Relation between log strength and log viscosity of asphalt.

... estimated shear rate existing during the ... but the extent of the extrapolation so ...ly reduced the precision of the estimation ... the results proved to be of no value. ... entirely possible that more precise ...asurements of strength would show greater ...ifferences in strength for the same viscosity than were indicated in this study, but it is believed that more study is needed to determine whether such differences are significant with respect to pavement behavior.

Discussion of Results

...sentially all of the relationships shown b... ...ne tests conducted in this study have been ...ited in previous research reports; some ... reports are summarized in appendix I. However, such previous studies were limited in scope as to tests over a wide range of temperatures on mixtures made with different distinctive types of aggregate systems and asphalts from different sources. The data reported in this article are believed to be of considerable value as they provide a quantitative estimate of the effect of the preceding factors. The variations in strengths of mixtures produced by differences in viscosity-temperature susceptibility of the asphaltic binder were much greater for the sand mixtures than for the gravel or crushed stone mixtures selected for this study. These differences were most pronounced at higher temperatures; but, as the temperatures decreased the differences between aggregates became less important.

When comparisons were made on the basis of the same viscosity, regardless of source or penetration grade, the relative differences in the strengths of the sand, gravel, and stone mixtures were greatest at the maximum service temperature (140° F.) at which the viscosities were lowest. The relative differences decreased with a decrease in temperature or an increase in viscosity. For viscosities of about 10^9 poises or higher, the effects of aggregate or possible differences in asphalts were not clearly evaluated in this study. There were some indications that the curve for the sand mixture leveled off; that is, it approached a maximum strength, but the curves for the gravel and stone mixtures did not show this tendency. It is believed that at such high viscosities, asphalts may cease to behave as viscous liquids and act more nearly as solids. Thus, tests to measure properties such as brittleness, stiffness, or fatigue resistance may provide a better evaluation of mixture properties at temperatures near 40° F. or lower than compressive strength tests such as were used in this study.

In certain aspects, the data from this study support the suggestion made by some asphalt technologists that asphalt cements should be graded on the basis of their viscosities at 140° F. rather than on penetrations at 77° F. Differences in viscosities of asphalts at 140° F. have been shown to be much more critical, insofar as compressive strength is concerned, than viscosity differences at lower temperatures.

Tests in this study showed no significant differences in compressive strengths for mixtures made with asphalts of the same viscosity from different sources. There was some indication that different asphalts having the same viscosity may produce slightly different strengths for weak aggregate systems at high temperatures, but differences of the order indicated would have no practical significance. Even these small differences were minimized at low temperatures.

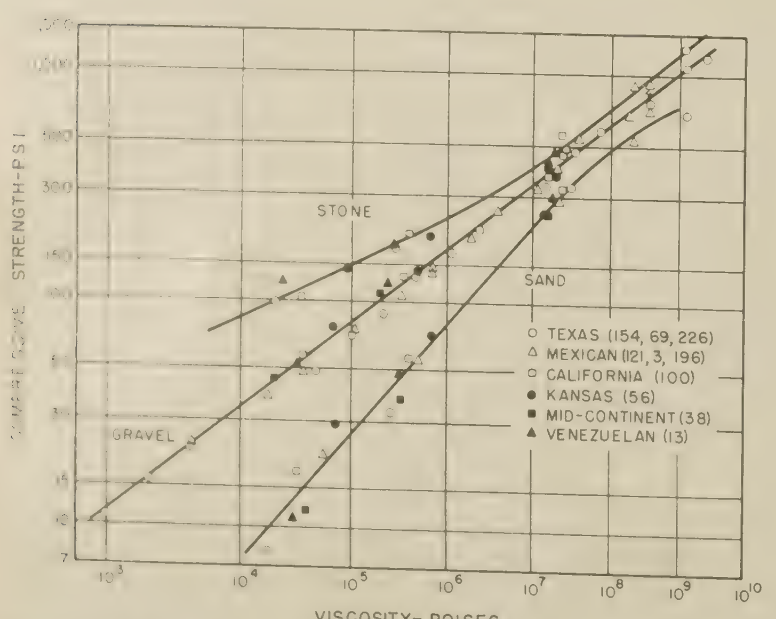

Figure 4.—Relation between log strength and log log viscosity of asphalt.

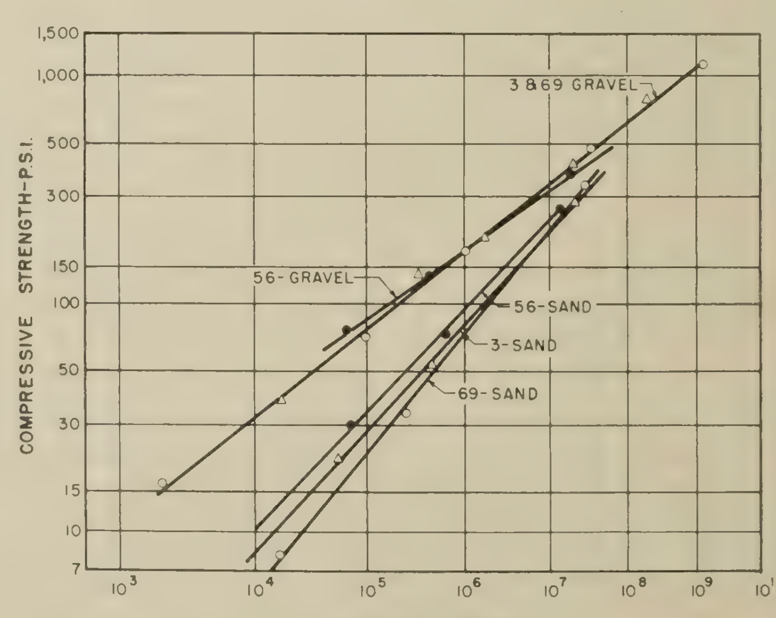

Figure 5.—Relation between log strength and log log viscosity of selected asphalts.

Summary of Previous Research

To provide background information for the work reported in this article, a literature survey was made. The following listed papers have been summarized to illustrate the general development of information and the present state of knowledge concerning the various relationships of viscosities and stabilities of asphaltic mixtures. Only a brief summary and major conclusions from each paper have been included. For a more detailed description of the research, the complete article should be referred to.

Stability Experiments on Asphaltic Paving Mixtures

In 1934, Emmons (*10*) reported a study conducted by the Bureau of Public Roads to establish performance data for the development of laboratory strength tests for bituminous pavement mixtures. Experimental pavement sections were constructed of many different compositions on a circular roadway 180 feet in mean diameter. The performance of these sections was observed under controlled-traffic conditions. Asphalts having different penetrations were used as binders in sheet asphalt and asphaltic concrete paving mixtures. Observations on the performance of the test sections showed that stable mixtures for sheet asphalt had been obtained from the use of asphalts having penetrations of 35 to 55. Use of softer, steam-refined asphalts having penetrations of 63 and 72 caused somewhat more plastic mixtures; however, only one section of test pavement had been laid that contained each of these asphalts. The consistency of the asphalt cement had no apparent influence on the service characteristic of the coarse-graded asphaltic concrete. Stable mixtures were laid with asphalts having penetrations ranging from 45 to 75.

Effect of Consistency and Type of Asphalt on Hubbard-Field Stability of Sheet Asphalt Mixtures

In 1940, Hillman (*11*) reported the effect of consistency and type of asphalt on the Hubbard-Field stability of sheet asphalt mixtures. Mixtures were made with 10 different grades of Mexican asphalt having penetrations that ranged from 23 to 182 and that were obtained from the same producer, and with two grades of asphalt, 50–60 and 85–100, that were obtained from each of five different sources. The mixtures prepared with the different asphalts were tested for Hubbard-Field stability at temperatures of 104°, 122°, and 140° F.

The findings of this study are summarized, as follows: (1) For asphalts from the same source, the stability of sheet asphalt mixtures depends on the consistency of the asphalt at the temperature of the stability test. (2) There are greater differences in the stability of mixtures made with the same penetration grade of asphalts from different sources than in the stability of mixtures made with 50 and 100 penetration asphalts from the same source. (3) There are some characteristics of asphalt that caused mixtures made with asphalts from different sources to vary in stability even when the consistency of the asphalts, measured by penetration, was the same.

Effect of Characteristics of Asphalts on Physical Properties of Bituminous Mixtures

In 1948, Lewis and Welborn (*12*) reported on the effects of the characteristics of asphalts on the physical properties of bituminous mixtures. This article included results of laboratory tests conducted on a particular bituminous mixture and a review of other investigations. It was concluded that: (1) The strength of a given mixture is dependent upon the consistency of the contained asphalt at the test temperature. (2) The test data from other investigations indicate that there is some characteristic of asphalts that causes comparable mixtures prepared with different asphalts to vary in strength even when contained asphalt has the same consistency.

Flow Properties of Bituminous Binders

In a paper published in 1950, Lee and Warren (*13*) commented on the fact that the flow properties of asphalt or other types of bituminous binders are recognized as being of primary importance in road construction. These authors stated that most of the specification tests, such as penetration, softening point, ductility, and furol viscosity, had no relation to the fundamental nature of flow and that up to that time little reliable information had been established to correlate road behavior with fundamental characteristics of binders.

To develop such fundamental information the Road Research Laboratory (England) obtained viscosity-temperature relations over a temperature range of about 15° C. (59° F.) to 180° C. (355° F.) on a number of asphalts of approximately 65 penetration, which had been obtained from several sources. Rotating-cylinder, Ostwald, and falling-cylinder viscometers were used to cover the temperature range. On the basis of the results of their tests on specimens of road surfacing mixtures made with the different asphalts, the authors concluded that the temperature susceptibility of the road mixture is determined by that of the binder. However, their tests also indicated that differences in the plastic and elastic properties of the asphalts were of more significance in determining the mechanical properties of mixtures than differences in viscosity-temperature coefficients.

Effect of Asphalt Viscosity on Stability of Asphalt Paving Mixtures

Weetman and Hurlburt (*7*), in 1947, investigated the effect of temperature and asphalt viscosity on the punching shear stability of a sheet asphalt mixture. Three asphalts of different penetration grades, from each of two sources, that differed widely in temperature susceptibility were used. Stability of a sheet asphalt mixture in which each asphalt had been used was determined over a temperature range of 40° F. to 160° F. Absolute viscosity of the asphalts recovered from the stability specimens was determined at several temperatures within this range by a coaxial-cylinder viscometer. The more important conclusions drawn from this study are:

- The punching shear stability of sheet asphalt mixtures at temperatures in the range of 40° F. to 160° F. are linear semilog functions of temperature.
- For asphalts of the same penetration at 77° F., the high viscosity-temperature susceptible asphalts had higher stabilities at lower temperatures and lower stabilities at high temperatures than asphalts of lower viscosity-temperature susceptibility.
- Comparable stabilities at 140° F. can be obtained by using a low-susceptible asphalt having an appreciably higher penetration at 77° F. than a highly susceptible asphalt. For the asphalt from the two sources reported, this difference in penetration was approximately 50.
- Stabilities of mixtures made with asphalts from the same source but having widely different penetrations at 77° F. are directly related to the asphalt viscosity at the shearing temperatures. A linear relation between log log viscosity and log stability was found, but the curves for the series of asphalts were not coincident. This indicated that some factor other than viscosity, such as absorption, affected the stability of the sheet asphalt mixture.

Viscosity Effects on the Marshall Stability Test

In 1951, Fink and Lettier (*14*) reported on a study concerned primarily with the effects of the absolute viscosity of asphalt on Marshall stability. Nine asphalts obtained from several sources and of widely different methods of manufacture and temperature-susceptibility were selected for this study.

Viscosities of the asphalts were determined, using the vacuum capillary viscometer, in the range of 120°–205° F. and the Saybolt furol viscometer at higher temperatures. The furol viscosities were converted to absolute viscosities. Marshall stability specimens were made by using one aggregate composition and each of the asphalts. Specimens for mixtures containing each asphalt were molded at temperatures at which the asphalt had a viscosity of 6 poises, and the specimens were tested at temperatures at which the viscosity was 1,400 poises. This study emphasized the fact that the stability was strongly influenced by the viscous resistance of the binder.

Influence of Rheological Characteristics of Binder on Mechanical Properties of Bitumen-Aggregate Mixtures

In 1953, Neppe (*8*) reported the results of compressive strength tests on mixtures made with aggregate of one type and grading and with 40 asphalt cements having different penetration grades and rheological properties. The author calculated stabilities from the times required for the specimens of the mixture to deform by predetermined amounts during the application of a compressive load that was being increased at a uniform rate. The tests were made at 77°, 104°, 122°, and 140° F. for each asphalt mixture. The viscosities of the asphalts were determined at several temperatures.

The author's primary purpose in this article was to show the usefulness of his "Softening Point Number" as a characterization factor for determining the influence of rheological characteristics of the binder on mechanical properties of mixtures. However, he also reported the effect of the viscosity of the asphalt on compressive strengths of the mixtures. A summary of the study included the following information:

- The results for compressive strengths at 77° F. showed no marked differentiation between mixtures prepared either from bitumens of low or high Softening Point Number.
- The mechanical stability of a bituminous mixture at any temperature is approximately a direct function of the log log viscosity of the contained binder at that temperature and is independent of the source, nature, and proportion of the latter constituent.
- The fluidity characterization factor, which is the numerical difference of log log viscosities at 77° F. and 275° F. multiplied by 100 was shown to be directly proportional to the Softening Point Number. The authors concluded that the Softening Point Numbers must also give a direct measure of the temperature-susceptibility of bitumens.

Relationship Between Unconfined Compressive Strength of a Bituminous Mixture and Viscosity of the Binder

In 1958 Wood and Goetz (*9*) used a compressive strength test to study the effect of asphalt viscosity on mixture stability. One of the two objectives of this research was to determine the relation between the viscosity of the binder and compressive strength of a mixture at several temperatures and rates of shear. The parallel-plate microviscometer was used to determine the viscosities at 40° F., 100° F., and 140° F. over a range of shear rates for four asphalts from different sources. Sheet asphalt mixtures composed of 74 percent sand, 17 percent filler (material passing the No. 200 sieve) and 9 percent asphalt were prepared with each of the four asphalts. Specimens 2 inches in diameter and 4 inches in height were molded by the double-plunger method. Compressive strength tests were made at 40° F., 100° F., and 140° F. at three rates of deformations. Relationships between viscosity and maximum compressive strength at different shear rates were determined. Conclusions drawn from the study were:

- The shear rate has a very marked effect on viscosity of some asphalts and little effect on others. The effect of shear rate on viscosity also varies with temperature.
- A plot of log compressive strength versus temperature gives a simple means of evaluating the temperature-susceptibility of mixtures.
- No direct relationship was found between compressive strength of sheet asphalt mixtures and the viscosity of the contained asphalt even though the effects of shear rate were taken into consideration. This indicated that factors other than viscosity of the binder have an effect on mixture strength.

Effects of Viscosity in Bituminous Construction

In a paper presented at the 1961 annual meeting of ASTM, Verdi Adam (*15*) showed the effect of asphalt viscosity on the Marshall stability of an asphalt concrete mixture from tests in which several different asphalts had been used. The four asphalt cements used in his tests varied from 60–70 to 120–150 penetration at 77° F. The furol viscosity of these asphalts at 140° F. varied from 3.5×10^4 to 3.6×10^5 (determined by conversion from absolute viscosity at 140° F.). The author showed that a good relationship existed between furol viscosity and Marshall stability, both at 140° F., for the different asphalts.

APPENDIX II

Compression Test Procedure

Mixture Preparation

The aggregates for the gravel and stone mixtures for the compression test were separated on various coarse and fine sieve sizes, including the 200 mesh. Portions of each size were then recombined to meet predetermined gradations in batches of sufficient amounts to mold two specimens. Similar test batches for the sand mixture were prepared by combining concrete sand, sheet asphalt sand, and limestone dust in definite proportions. All batches of aggregates were heated overnight in a forced draft oven at 325° F. The asphalt for each batch was prepared by putting a slight excess of the amount required per batch in a sealed, 8-ounce tin that then was placed in a 300° F. oven 45 minutes prior to the asphalt's being blended with the heated aggregate. The required amount of asphalt was added to the aggregate in a modified Hobart mechanical mixer and the materials were mixed for 2 minutes. Each batch contained slightly more mixture than that required for two, 3-inch by 3-inch cylindrical specimens.

Molding Procedure

After being mixed the amount of material required to mold one specimen was transferred to a heated mold. The bowl with the remaining mixture was placed in the 325° F. oven for approximately three minutes, after which the second specimen was molded. For each specimen, the mixture in the heated mold was spaded once around the inside of the mold, then subjected to a 3,000 p.s.i. compaction load that was held for two minutes by use of the double-plunger technique.

Test Procedure

After removal from the molds the specimens were allowed to cool to room temperature and then were placed in a forced-draft oven at 140° F. for 24 hours. The specimens were then cooled overnight at room temperature and the bulk specific gravity of each was determined by use of the water displacement method. The air voids were calculated on the basis of effective specific gravity of the aggregate determined by the Rice vacuum saturation method. The specimens were placed in air baths controlled at the various test temperatures indicated in table 4. In general, the time in the air bath was one-half hour more than that required for a dummy specimen to reach the desired test temperature, which was measured with a thermocouple inserted in the center of the specimen. The bearing blocks of the testing machine were also placed in the air baths to bring them to the test temperature. The compressive strengths of the specimens were determined at a loading rate of 0.15 inches per minute. Each of the compressive strengths and specific gravities given in table 4 is the average result for four duplicate test specimens.

(*References for this article are listed on page 144.*)

U.S. GOVERNMENT PRINTING OFFICE: 1963

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The 1961 Interstate System Cost Estimate, House Document No. 49 (1961). 20 cents.

U.S. HIGHWAY MAP

Map of U.S. showing routes of National System of Interstate and Defense Highways, Federal-aid Primary Highway System, and U.S. Numbered Highway System. Scale 1 inch equals 80 miles. 25 cents.

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