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Flexible Pavement Studies and Georgia Design Practice

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Contents

Load-Deflection Study of Selected High-Type Flexible Pavement in Maryland

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A cooperative program of load-deflection tests of several high-type flexible pavements in Maryland was inaugurated in the spring of 1955. A single test consisted of the application of a slowly moving 11,200-lb wheel load of a single-axle truck to an arbitrarily selected point on the pavement and the measurement of the resulting pavement deflection and rebound. Measurements were made at a point between the dual tires by means of the pavement-deflection indicator known as the Benkelman Beam. Tests were made in the spring and in the fall at approximately 1,000 marked locations over a distance of about 85 lane miles. The pavements tested range in age from two to eleven years and are in excellent condition.

This report contains a description of the pavements studied, the test procedure used, and the results of the tests conducted to date.

● A LIMITED PROGRAM of structural tests of several high-type flexible pavements in service was initiated in early 1955. The program is a cooperative effort of the Maryland State Roads Commission and the U. S. Bureau of Public Roads. Basically the purpose of this study is to gain further knowledge of the structural behavior of nonrigid pavements and to ascertain whether maximum deflection might be considered as a measure of their adequacy.

The study consists essentially of a series of load-deflection tests in the spring and fall of each year and an attempt to correlate the data obtained with some of the variables known to affect pavement behavior. A single load-deflection test generally consists of the application of a slowly moving 11,200-lb wheel load to the pavement and measurement of the resulting maximum deflection and rebound or recovery.

A brief account of this study and the highlights of the findings after the first year of observations are included as a part of a previous paper (1). In view of the fact that the investigation is a continuing one, this paper is simply a progress report of the work done to date. However, since there is little previously published information regarding tests of the type described herein, an account of the more important factors that were considered and a fairly detailed description of the manner in which the tests are being conducted will be included.

Three of the four pavements under study are located in Carroll, Frederick, Washington, and Montgomery Counties in the central part of the state; the fourth is in Queen Annes County on the Eastern Shore. All are high-type flexible pavements of modern design with 12-ft lanes. They were constructed on new location on important routes and all are in excellent condition.

To obtain an indication of the effect on pavement deflection of changes in the condition of the pavement structure and subgrade caused by seasonal changes, tests were conducted at two periods during the year. It was planned to select a period in the spring when conditions were at their worst, and in the fall when conditions were good. Selection of the most desirable test periods was based on visual observation of pavements in the area and on personal judgment. Thus far, four series of tests have been completed. These were made during the following periods:

1. Late March and early April 1955.
2. Mid-November 1955.
3. Early April 1956.
4. Mid-November 1956.

It has been found difficult to estimate in advance the period during which the most adverse conditions will exist and to recognize this period when it comes. As a result, it is believed that the spring test series of both years were made several weeks late.

The device used to measure the vertical movement of the pavement is the lever-type pavement deflection indicator known as the Benkelman Beam. Descriptions of this device and the details of the procedure for using it have been previously published (2). A general view of the test truck with the instruments in position just prior to the beginning of a test is shown in Figure 1. It is sufficient to say here that this is a simple and inexpensive device that measures the deflection at the pavement surface of a point between the rear dual tires of a truck moving at creep speed as well as the rebound or recovery of the pavement after the wheel has passed.

The final or recovery measurement was made after the load had passed well beyond the point where it might affect the instrument and after observation of the micrometer dial indicator showed no further movement of the pavement. The difference between the deflection and recovery measurements is referred to as the residual.

The organization of the field party for conducting tests of this nature is quite simple. Assuming that the test sites and points have been selected and marked in advance and that one truck and two deflection indicators will be used, the party might consist of a recorder, two instru-

Figure 1. Deflection indicators in position at beginning of test.

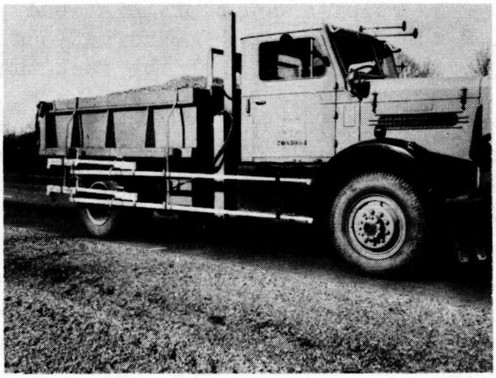


Figure 2. Deflection indicators in carrying position.

ment operators, two flagmen, and a truck driver. The recorder may act as party chief. All personnel can be trained within a day to perform their jobs and to work as a team. The most important characteristic required of the party chief and instrument operators is their sense of responsibility. They must realize that they are working with a precise instrument and that care must be exercised in order to obtain accurate results.

The vehicle used to apply the test load for the tests in the central part of the state was a two-axle, dual-tired dump truck. For the first series of tests (spring 1955) two such trucks were used, one having a 7,000-lb wheel load, the other having an 11,200-lb wheel load, the latter being the legal load limit in Maryland. The remaining three test series were made with the 11,200-lb load only. The desired wheel load was obtained by loading the truck with crusher-run stone and weighing on either platform or loadometer scales. Tire equipment consisted of 10.00 x 20 tires inflated to 80 psi. The truck used for the tests on the Eastern Shore was of the same general type as the others and was equipped with 11.00 x 20 tires.

It has been found that the best way to transport the deflection indicators is on the right side of the test truck. Specially designed, removable metal brackets fastened to the truck body were used for this purpose (Fig. 2).

Because for each test the probe arm of the deflection indicator was routinely placed between the two tires of a dual-tired wheel, the space available between the tires is of importance. The device must be placed in position so that as the vehicle moves forward the lever arm is not touched by either tire. The device is aligned by eye. The minimum space

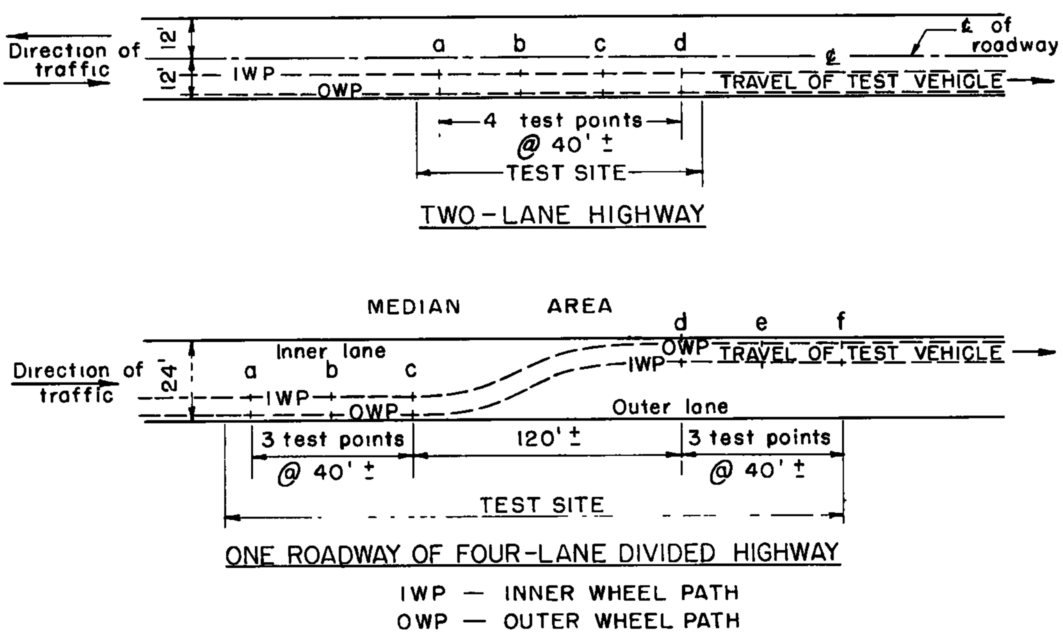


Figure 3. General layout of test site, showing testing procedure on the two highway types.

between tires on the trucks used for these tests was $1\frac{1}{2}$ in., whereas the minimum distance between contact areas was 4 1/8 in.

It has been found by experience that this is about the minimum space desirable, although studies to determine the effect of increasing the space between dual tires on deflection of the pavement have not been made. It has been noticed, however, that for certain pavements and under certain conditions there can be a tendency for the surface material to squeeze upward between the contact areas of the tires. For this reason, the distance between contact areas may have some significance and therefore should be kept to a reasonable maximum of about 5 in.

The scope and extent of the study was determined by the anticipated availability of the personnel and equipment required to make the field tests. Initially, portions of the three pavements in the central part of the state totaling 45 mi in length were selected. It was decided to allow a period of two weeks for completion of a test series. Previous experience with similar tests elsewhere made it possible to estimate the time required to complete a given number of tests over a known length of pavement.

Test sites were selected at random intervals averaging about $\frac{1}{4}$ mi along each of the pavements tested. The intervals range from a minimum of 0.1 mi to almost 1 mi, the chief consideration in the selection of the individual test sites being that of safety. It is essential that drivers of vehicles approaching the halted test truck have adequate sight distance and ample warning from the flagman. For this reason sites on or near horizontal or vertical curves were usually avoided. Other factors that should be taken into consideration are pavement condition, pavement design, type of subgrade soil, construction methods, grade line location, culverts and bridges, and drainage.

On the three pavements in the central part of the state, 171 test sites were selected. The layout of individual test points for the two highway types studied is shown in Figure 3. On two-lane highways four tests at approximately 40-ft intervals were made in one lane only; on four-lane divided highways, three tests at this interval were made in each of the two lanes of one roadway. All test points were marked with paint so that tests of all test series could be repeated at exactly the same spots. Two deflection indicators were used so that tests could be made in both the inner wheel path (IWP) and outer wheel path (OWP) simultaneously.

In the fall of 1955 initial tests were made on the fourth pavement included in the study. This added 10 mi of pavement and 38 test sites to the 45 mi and 171 sites previously mentioned, making a total of 55 mi and 209 sites. Because tests were made in both lanes of one roadway of the divided highways, it is apparent that approximately 85 lane-miles of pavement were studied. The average rate of progress for the entire study has been the measurement of 48 deflections at 24 points representing six test sites per hour.

Previous studies have shown that the location of the wheel load with respect to the pavement edge has a significant effect on the magnitude of the deflection. Also it was considered desirable to repeat each series of tests at the same points on the pavement, within practical limits. For these reasons it was decided to attempt to make all tests with the outside of the outside rear tire 3 to 6 in. from the pavement edge. This placed the centerline of the outer and inner rear wheels about 1.5 and 7.5 ft, respectively, from the edge of the pavement. This distance was selected because it places the outside wheel at or near the most critical location on the pavement.

The sections of pavement selected for this study are as follows:

1. Westbound lane of the 16.5-mi long section of US 40 west of Frederick between its intersection with US 40 (Alternate) and Hagerstown.

2. Eastbound lanes of US 40 east of Frederick, from the Monocacy River to the vicinity of Ridgeville, a distance of 12 mi.

3. Southbound lanes of the 16-mi long section of US 240 south of Frederick between the US 15 and State Route 118 interchanges.

4. Northbound lane of the portion of the Blue Star Memorial Highway, 10 mi in length, between State Routes 305 and 300 northeast of Centreville. For the sake of simplicity these four pavement sections are referred to as US 40W, US40E, US 240S, and Blue Star, respectively.

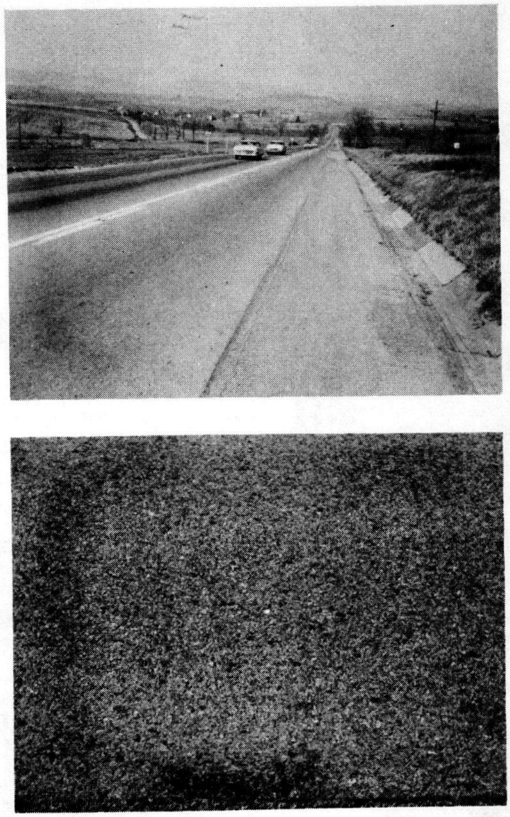


Figure 4. General view of roadway (upper), and detail of typical pavement surface (lower) - US 40W.

US 40W is a two-lane highway located in mountainous terrain with maximum grades of 8 percent. A general view and a close-up of the pavement surface are shown in Figure 4. It was constructed on a relocation of the old route, the grading for which had been completed a number of years earlier, and was opened to traffic about eleven years ago. A soil survey of the new grade was made just prior to construction of the flexible pavement structure. It was found that about 80 percent of the material at the subgrade level is A-5 soil consisting of silt, hard or soft shale, and decomposed rock, with solid rock at or near the surface at some locations. The remainder is composed, in general, of A-4 and A-7 soils. A cross-section of the pavement structure is shown in the upper part of Figure 5. This design, which is uniform throughout the 16.5-mi length, consists of a 2-in. stone screenings blanket or insulation course, an 8-in. waterbound macadam base course topped by a 4-in. penetration macadam course, and a 3-in. asphaltic concrete surface course, making a total thickness of 17 in.

US 40E is a four-lane divided highway in rolling terrain with a maximum grade of 6 percent, but with much of the pavement on grades of less than 4 percent. A general view of the eastbound roadway and close-up of the present pavement surface are shown in Figure 6. It was planned and built as a stage construction project, the asphaltic concrete surface course having been initially omitted. In its place a temporary surface course consisting of a double-surface treatment about 1 in. in thickness

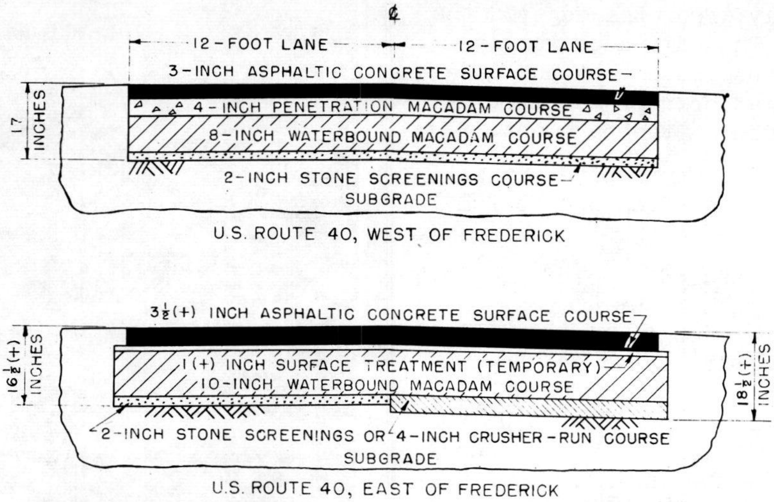
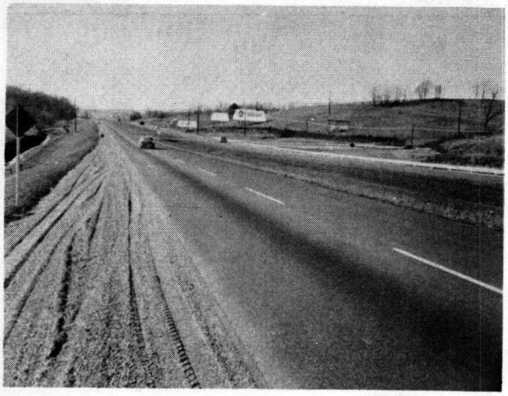


Figure 5. Transverse sections showing components of pavement structure.



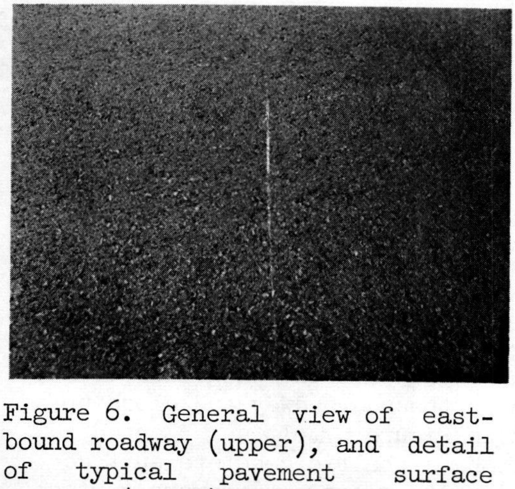


Figure 6. General view of eastbound roadway (upper), and detail of typical pavement surface (lower) - US 40E.

was laid. Early in the life of this uncompleted pavement structure extensive cracking of the surface began to develop and at the time of the first deflection test series (spring 1955) the road was undergoing considerable maintenance. Later in the year, but prior to the fall test series, the asphaltic concrete surface was placed. This surface was about one year old, in January 1957, whereas the remainder of the structure was about two years old. Since placement of the final surface, the performance of this pavement has been excellent. The predominating soils found on this route are an A-5 micaceous silt, decomposed shale and rock, and an A-4 silt. A cross-section of the pavement structure is shown in the lower half of Figure 5. On four of the five sections of this project the 4-in. crusher-run subbase shown to the right of the section centerline was constructed. On the fifth section a 2-in. stone screenings course was placed in lieu of this. The base course on four sections consists of 10 in. of waterbound macadam compacted in one layer by a vibratory method, whereas on the fifth section two 5-in. layers were compacted by rolling. This was first surfaced with a double surface treatment approximately 1 in. thick, and finally with a minimum of $3\frac{1}{2}$ in. of asphaltic concrete.

US 240S is a part of the Washington National Pike between Frederick and Rockville and is a four-lane divided highway. A view of the southbound roadway and a close-up of the pavement surface are shown in Figure 7. It was constructed on new location paralleling the old route and is in undulating topography with maximum grades of 4 percent. The first section was completed in 1952 and the last in 1954. The predominating soil is A-5 micaceous silt and decomposed rock, although a considerable amount of A-4 silt is also found on this route. A section showing the components of the pavement structure is given in the upper half of Figure 8. The part of the pavement located in Montgomery County has a 2-in. stone screenings course between the base course and subgrade, as indicated in the right half of the section. On the remainder of the project this course was omitted. The base course consists of an 8-in. thick-

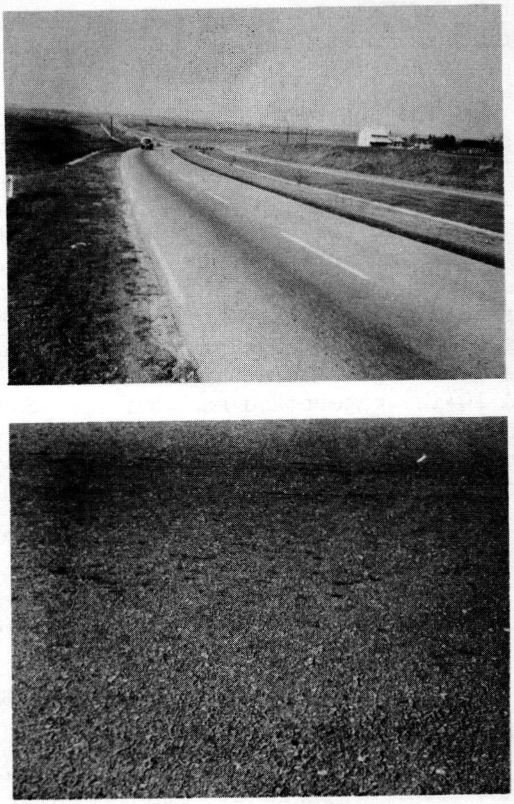


Figure 7. General view of southbound roadway (upper), and detail of typical pavement surface (lower) - US 240S.

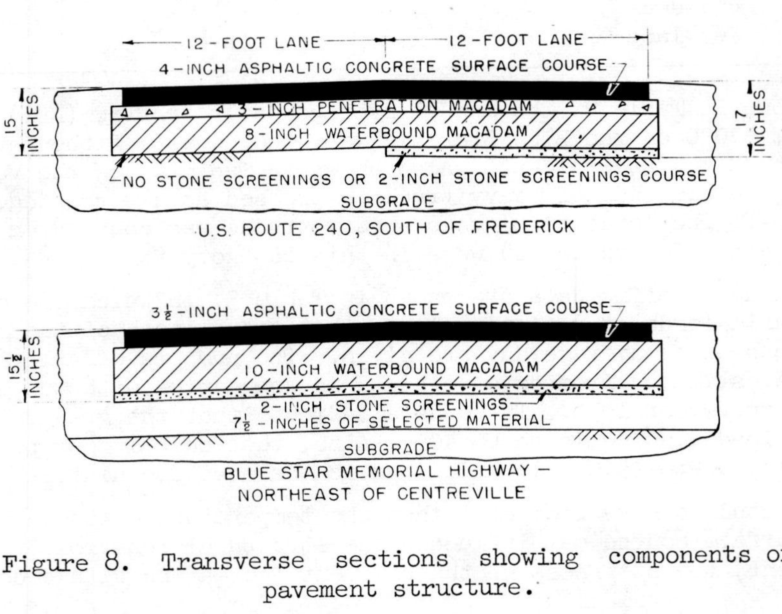


Figure 8. Transverse sections showing components of pavement structure.

ness of waterbound macadam topped with 3 in. of penetration macadam. The surface course is 4 in. of asphaltic concrete, making a total thickness of pavement structure of 15 to 17 in.

The pavement section designated Blue Star is a portion of a new north-south route that connects the Chesapeake Bay Bridge to major routes in Delaware. It was designed as a four-lane divided highway, but only two lanes were constructed at this time. The section under study has been open to traffic for only a year. It is located on comparatively flat terrain, the maximum grade being 1.5 percent. The predominant soil type is an A-4 silt, which exists at about two-thirds of the test sites. At most of the remaining sites the soils are A-2 or A-3 sands and gravel. A typical cross-section of the pavement structure is shown in the lower part of Figure 8. A 2-in. stone screenings course was placed on a 7½-in. layer of select soil. A 10-in. waterbound macadam base course was then constructed and finally surfaced with 3½ in. of asphaltic concrete. The base course material on one-half the project is crushed slag; on the remainder, crushed stone. The structure thickness, excluding the layer of select material, is 15½ in.

For convenience and for the sake of clarity the foregoing general information is summarized in Table 1.

TABLE 1

SUMMARY OF DESCRIPTIVE INFORMATION REGARDING THE FOUR PAVEMENT SECTIONS STUDIED

Pavement Designation	Highway Type	Length, mi	Pavement Thickness, in	Predominant Soil Type	Age, yr	Maximum Grade, %
US 40W	Two-lane	16.5	17	A-5	11	8
US 40E	Four-lane divided	12	16½+ or 18½+	A-5	2	6
US 240S	Four-lane divided	16	15 or 17	A-5	2 to 4	4
Blue Star	Two-lane	10	15½	A-4	1	1.5

As shown in Table 2, the annual average daily traffic (ADT) in 1955 ranged from 3,000 on the Blue Star Highway, the newest of the pavements, to 7,700 on US 40W, the oldest. Also shown in Table 2 are the volumes of commercial vehicles and of heavy trucks expressed as a percentage of the total volume. The largest volume of heavy trucks was counted on US 40E, where 9 percent (585 vehicles) were in this category.

Additional traffic data are shown in Table 3, in which the estimated number of axle loads of various weights that travel each of the four pavement sections on an average day are given. The numbers of axle loads tabulated were estimated from data obtained in connection with the statewide loadometer survey of 1956. These data indicate that the greatest frequency of heavily loaded axles is on US 40E. Also, that on all four pavement sections the 14,000-to-15,999-lb axle load is the most frequent.

Other studies have indicated that the temperature of the asphaltic concrete surface course may influence the structural behavior of a flexible pavement, the stiffness of the pavement increasing with a decrease

TABLE 2

SUMMARY OF TRAFFIC DATA, 1955

Pavement Designation	Annual Average Daily Traffic[a]	Percentage of Total Traffic	
		Commercial Vehicles	Heavy Trucks[b]
US 40W	7,700	15	5
US 40E	6,500	19	9
US 240S	4,900	12	4
Blue Star	3,000	20	3

a All lanes, both directions.
b 5 tons or more.

TABLE 3

ESTIMATED NUMBER OF AXLE LOADS OF VARIOUS RANGES IN MAGNITUDE FOR AN AVERAGE DAY IN 1956

Range in Axle Load, lb	Axle Loads			
	US 40W	US 40E	US 240S	Blue Star
12,000 - 13,999	136	240	81	47
14,000 - 15,999	176	300	103	63
16,000 - 17,999	167	293	99	58
18,000 - 19,999	128	227	77	44
20,000 - 21,999	64	116	45	28
22,000 and over	41	74	30	18

in temperature. However, the relation between the deflection of a pavement of this type and the temperature of its surface course for a given condition of loading has not been established. In the present studies, as a matter of general interest, air and pavement temperature were measured in the morning, at noon, and in the evening of each test day. Pavement temperatures were measured with a mercury thermometer inserted in an oil-filled hole in the pavement surface. The hole, approximately $\frac{1}{4}$ in. in diameter and $1\frac{1}{2}$ in. deep, was located about $1\frac{1}{2}$ ft from the pavement edge.

A summary of the pavement temperature measurements obtained during the four test periods is given in Table 4. These data indicate that:

TABLE 4

SUMMARY OF PAVEMENT TEMPERATURE RANGES FOR EACH TEST SERIES

Pavement Designation	Pavement Temperature Range, F			
	1955		1956	
	Spring	Fall	Spring	Fall
US 40W	58-90	38-67	58-96	38-71
US 40E	33-67	44-62	61-89	39-59
US 240S	40-92	45-76	55-91	45-78
Blue Star	-	36-54	48-78	40-59

1. For any one pavement section and test series there was a temperature variation from about 20 to as much as 50 F.

2. Temperatures were generally about 10 to 20 F less in the spring of 1955 than in the spring of 1956, whereas those of the two fall test series were about the same.

3. Temperatures during the fall tests were generally 10 to 20 F less than those of the spring tests.

Plots of typical deflection data obtained on the four pavement sections are shown in Figures 9 through 14, the maximum deflection values in the upper and the residual values in the lower part of each figure. Each plotted value is the average of the three or four individual measurements at each test site. The test sites are plotted at equal intervals along the abscissa, although actually they are located at irregular intervals along the pavements as explained earlier. The lines connecting the plotted points have no particular significance, but were drawn merely to emphasize the relative values of the points and the trends that might be indicated. All figures show deflections for an 11,200-lb wheel load and for all test series completed to date, with the exception of Figure 11, which is for the 7,000-lb wheel load used in the first test series only.

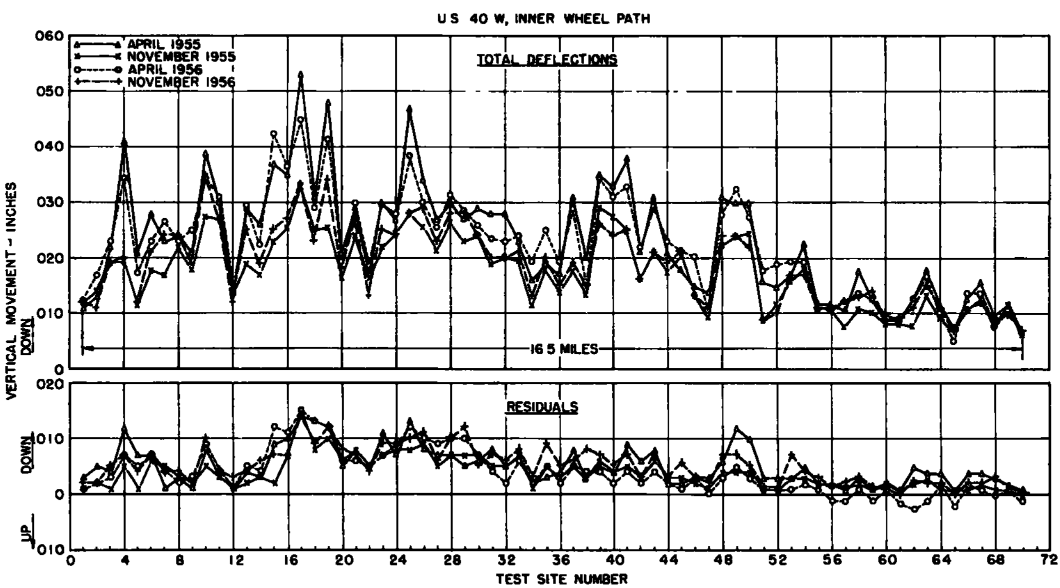


Figure 9. Average deflection and residual values at each test site for the various test periods - 11,200-lb wheel load.

Figures 9, 10 and 11 show all of the data obtained on US 40W in both wheel paths. Figures 9 and 10 are for an 11,200-lb and Figure 11 for a 7,000-lb wheel load. However, because the trends discussed are the same for either lane and either wheel path, the detailed data of the remaining three pavements are presented in Figures 12, 13 and 14 for the outer wheel path of the outer lane only. This is done for the sake of brevity and to avoid repetition.

Several general observations of the data contained in Figures 9 through 14 that are characteristic of the results obtained on all four pavement sections may be made, as follows:

1. For any one test series the deflection and residual values vary markedly from site to site.

2. Where the deflection value is relatively large or relatively small with respect to those of other sites for one test series, it generally remains so for the other test series. This is true also of the residual values, although the data for these are more erratic.

3. Values of deflection obtained in November are appreciably less than those measured in spring.

4. The residual values tend to vary directly as the deflection values.

5. Residual values are larger than might be expected.

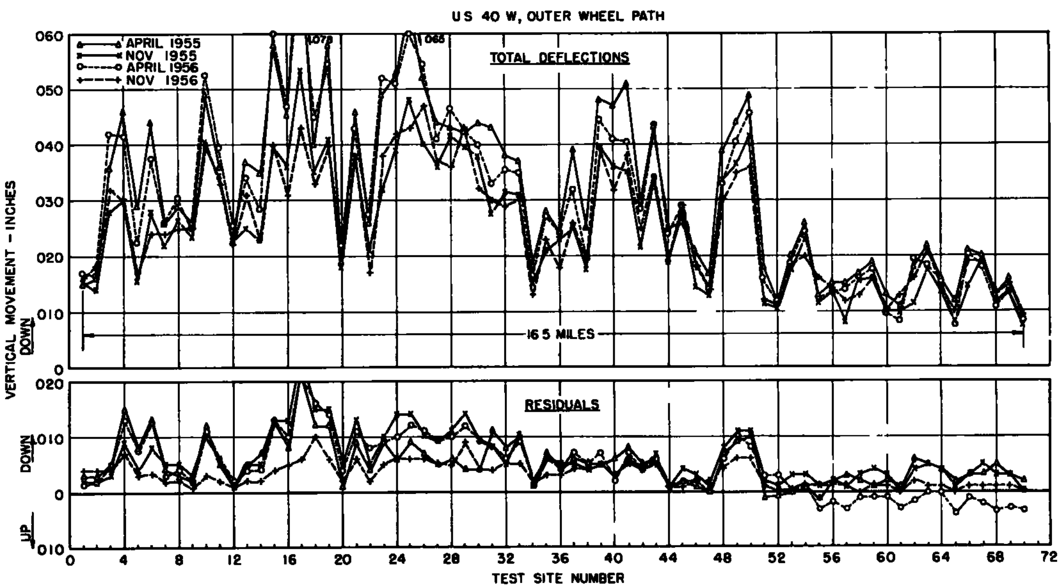


Figure 10. Average deflection and residual values at each test site for the various test periods - 11,200-lb wheel load.

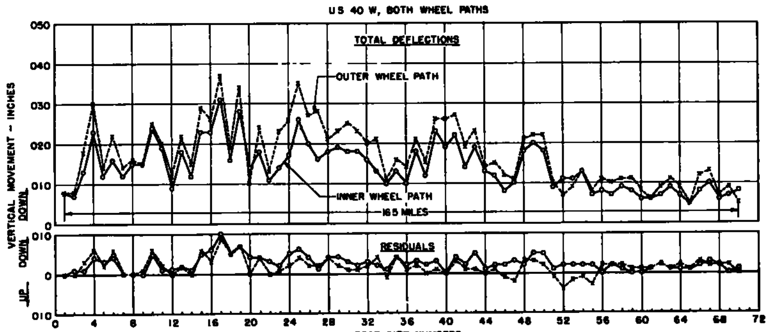


Figure 11. Average deflection and residual values at each test site in the spring of 1955 - 7,000-lb wheel load.

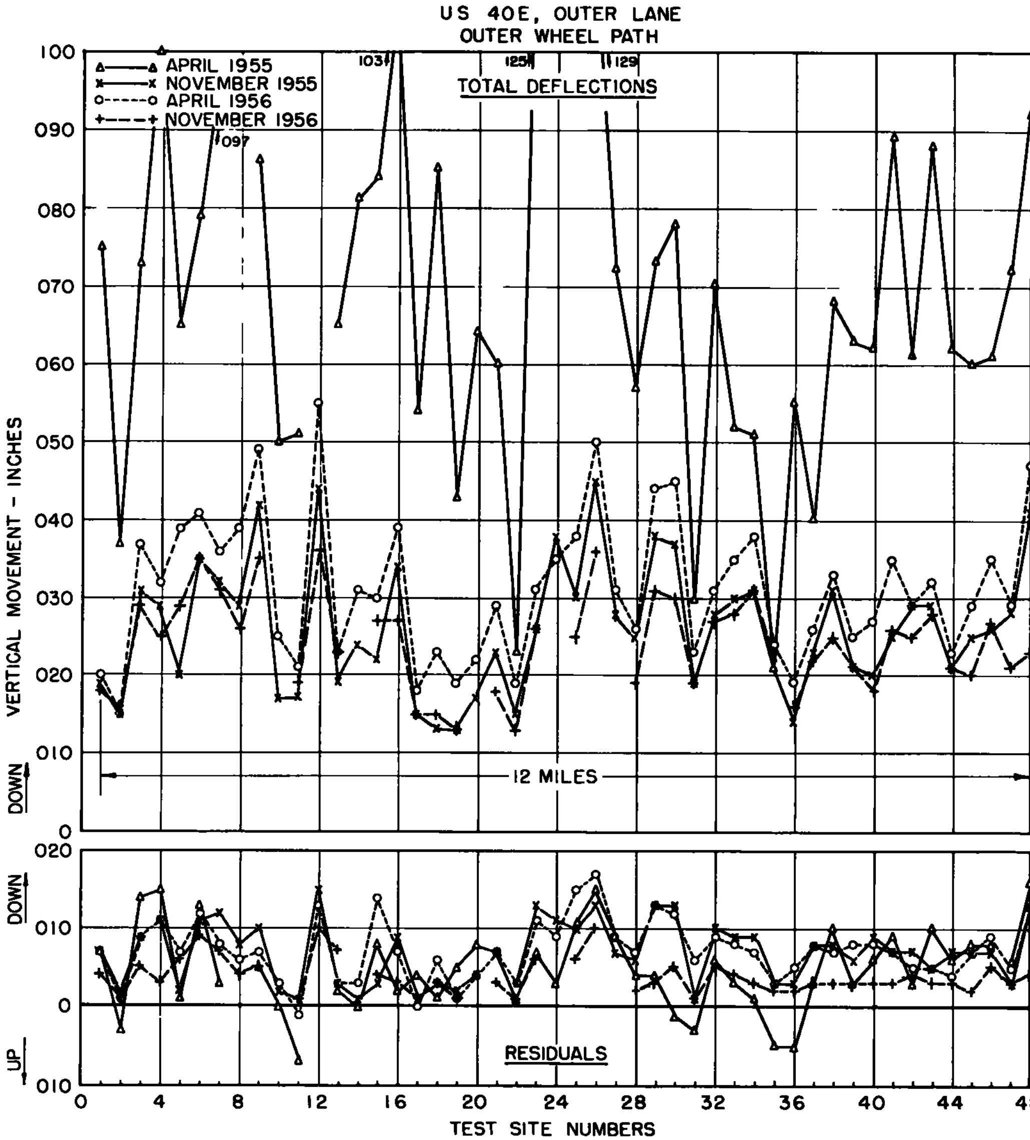


Figure 12. Average deflection and residual values at each test site for the various test periods - 11,200-lb wheel load.

The reason for the large variations in deflection and residual values from one location to another has not been determined thus far in the study. Attempts to correlate pavement deflection with the type and nature of the subgrade soil, as determined by surveys conducted prior to construction, have not been successful. Also, careful visual inspection of the pavement surface at locations where deflection values are comparatively large has resulted in no explanation for this behavior, except in the case of the spring tests on US 40E, where surface cracking was evident. However, it is known that large variations of this sort are typical of data previously obtained on other flexible pavement by the same methods (3). This has also been found on similar tests conducted in several other states, the results of which have not been published.

Except for the first test series on US 40E, the average residual values in the various wheel paths for all test series on the four projects range from 7 to 29 percent of the corresponding average deflection values. However, in most cases these percentage values lie between 15 and 25.

A comparison of the data in Figures 9 and 10 shows that although the deflection and residual values are appreciably larger in the outer wheel path, the variations from site to site are generally the same. Also, a comparison of the values of either wheel path for the four test series discloses that those of the two fall test series are of about equal magnitude. This is not as true of the two spring series, where those of 1955 are, in general, somewhat the larger. In the inner wheel path the relations between the residuals for the various test series are quite erratic. However, in the outer wheel path those of the first test series (April 1955) are the largest and those of the fourth series are the smallest.

The data for both wheel paths of US 40W obtained with the 7,000-lb wheel loading are shown in Figure 11. Comparison of Figure 11 with Figure

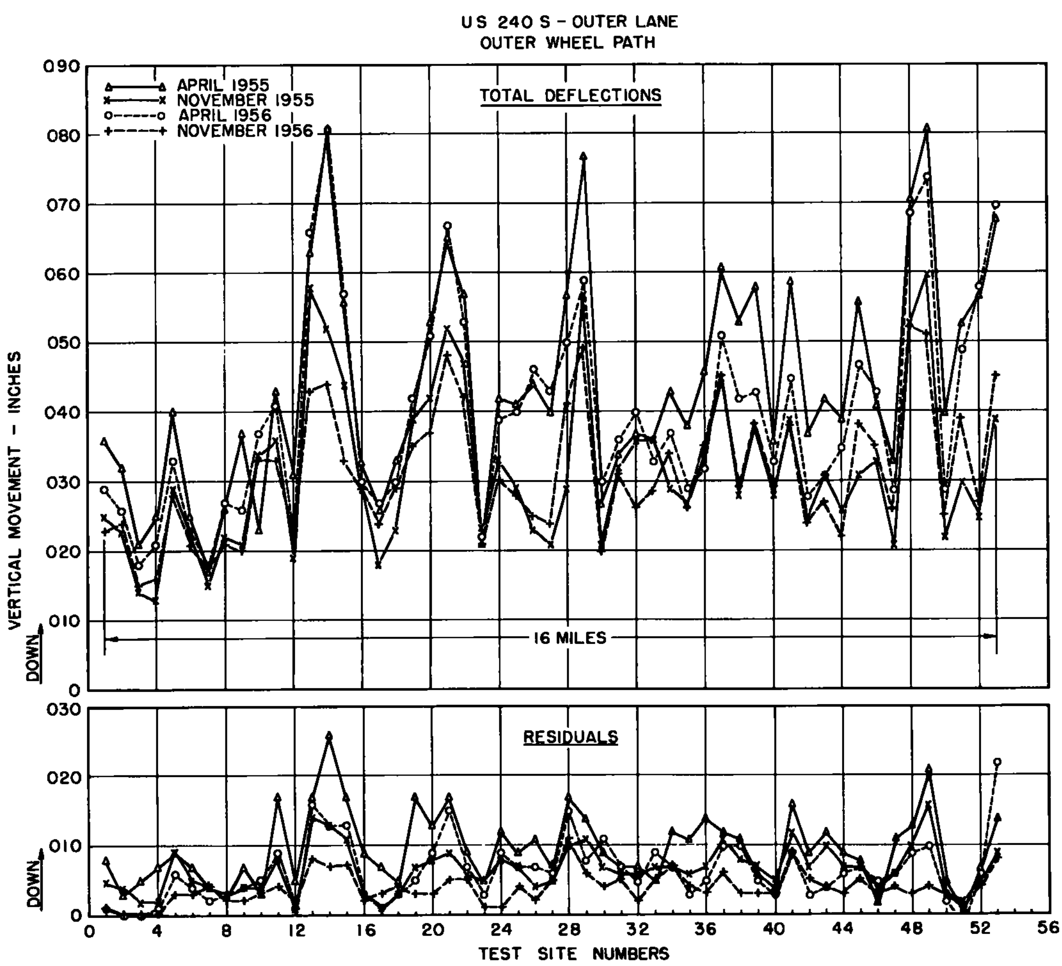


Figure 13. Average deflection and residual values at each test site for the various test periods - 11,200-lb wheel load.

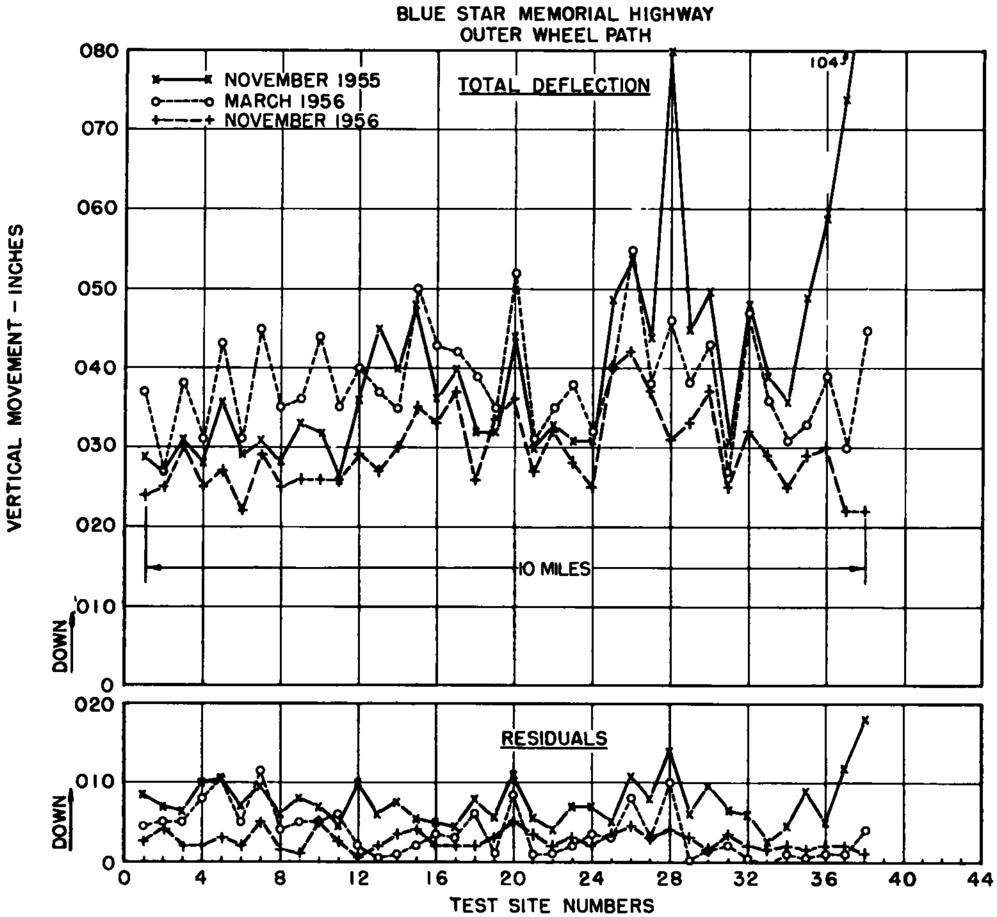


Figure 14. Average deflection and residual values at each test site for the various test periods - 11,200-lb wheel load.

9 or Figure 10 shows that although the magnitude of the deflection and residual values is proportionately less, the variations from site to site are about the same. A direct comparison of the deflection and residual values for the two wheel paths is afforded in Figure 11. At nearly all sites the deflection in the outer wheel path is larger than that of the inner. However, the residual values show no definite trend in this regard.

The detailed data for the outer wheel path of the outer lane of US 40E are given in Figure 12. It should be recalled before studying this figure that at the time of the initial series of tests (April 1955) the final surface had not been placed on the structure and signs of surface distress were prevalent. The deflection values of the April 1955 test series are extremely large compared to those of the later test series. During the late summer and fall of 1955 the final asphaltic concrete surface course was constructed. Following this, the second series of tests was made. As stated previously, the values of this and subsequent series were considerably smaller. Residual values of the first test series are the most erratic, some indicating permanent upward movement. Those of the most recent test series are the least erratic and also the least in magnitude.

The average deflection and residual values at each site for US 240S are shown in Figure 13. The relations between the deflections of the four test series are about the same as those of US 40W; that is, those of the first and fourth test series are the largest and smallest, respectively.

The deflection and residual data for the Blue Star pavement are given in Figure 14. This project was completed in November 1955, and the initial test series was made late that month. This series corresponds to the second test series of the other three pavements. The deflection data obtained at the first 24 test sites generally indicate that the March and November 1956 values are the largest and smallest, respectively, and the others are in between. However, for the remaining test sites the November 1955 val-

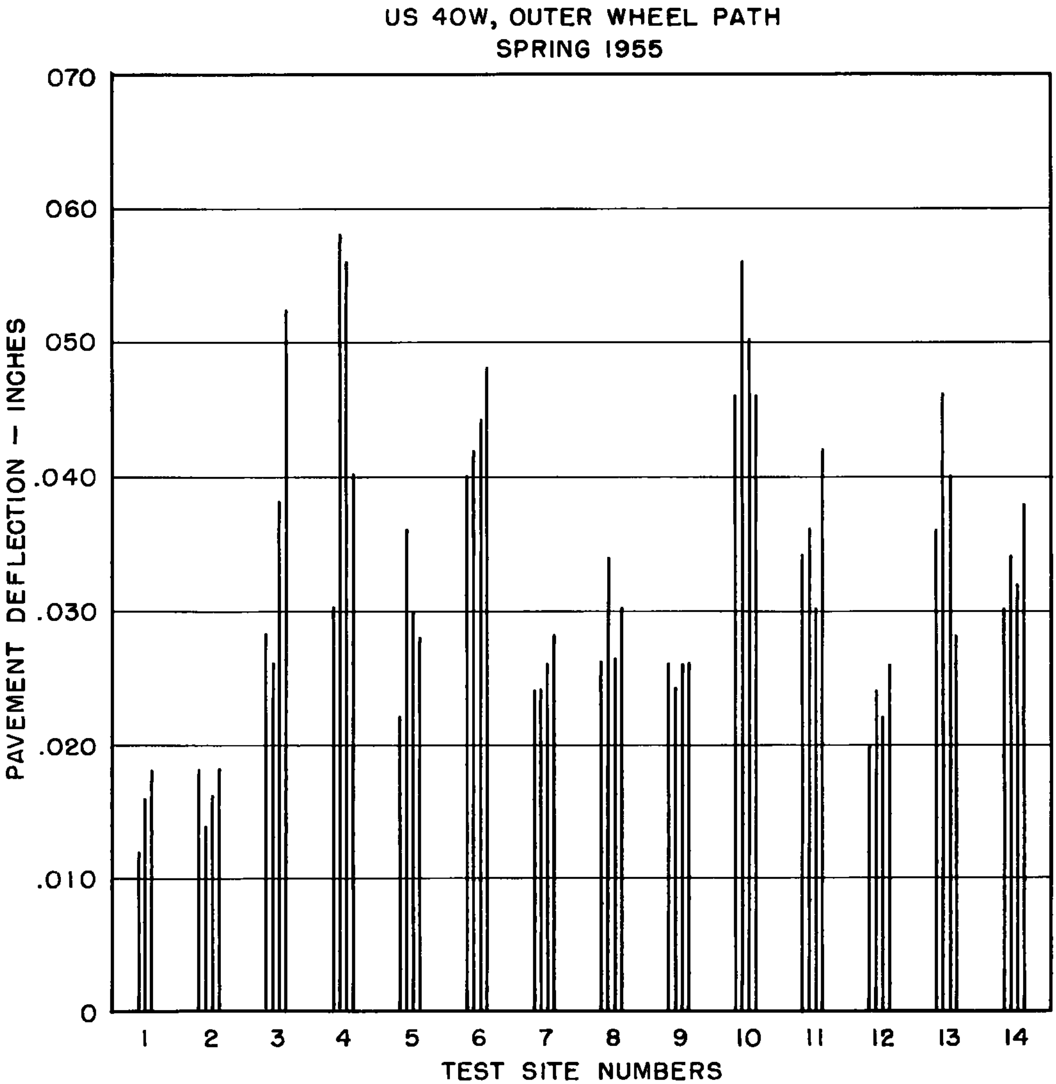


Figure 15. Typical deflection measurements at a number of test sites showing the variations in the individual values obtained - 11,200-lb wheel load.

ues are the largest. In fact, at sites 28, 37, and 38 the deflection values of this test series are exceptionally large as compared to those at the other sites.

When these measurements were obtained, it was noted that there was a tendency for the pavement to "roll" ahead of the truck tire. However, in spite of this unusual type of movement observed immediately after construction, subsequent measurements have indicated only normal movements at these locations. The residual values for the first test series were appreciably larger than those of the other test series. The values at sites 28, 37, and 38 were particularly large at that time.

The data previously presented show the rather large variations in deflection values which may be found from site to site of a given pavement. In addition, it was found that the three or four individual deflection measurements obtained under constant test conditions at a single site may vary considerably. This is illustrated by the data shown in Figure 15, obtained at a series of typical although arbitrarily selected test

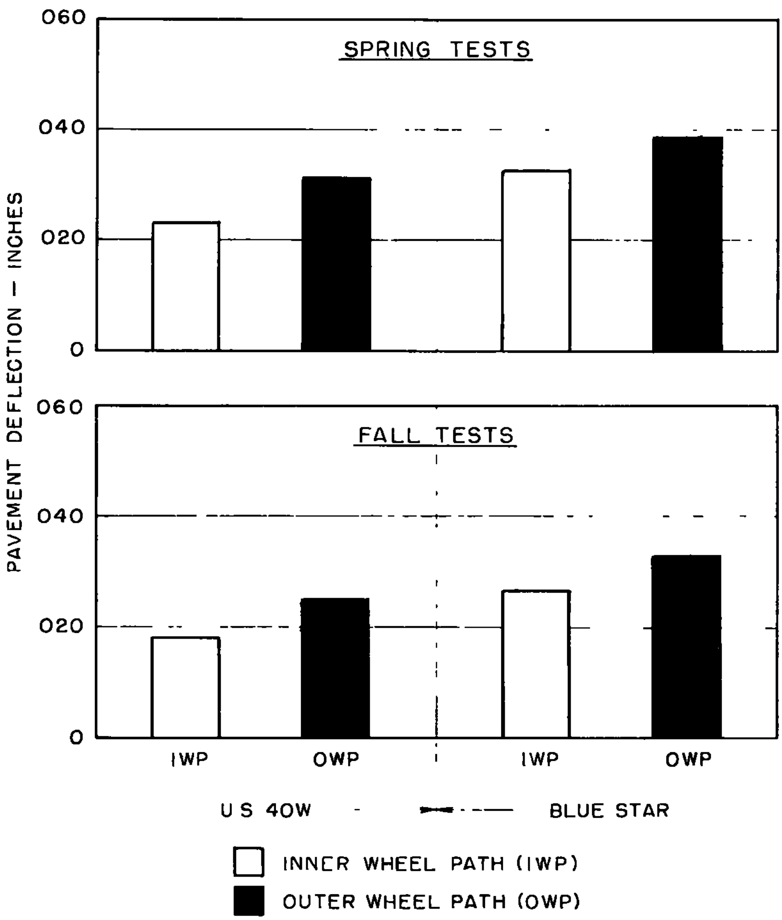


Figure 16. Comparison of average deflections in the inner and outer wheel paths of the two-lane highways - 11,200-lb wheel load.

sites. The variations in values range from a minimum of 0.002 in. at site 9 to a maximum of 0.028 in. at site 4.

A comparison of the average deflection values of the inner and outer wheel paths of the two-lane highways (US 40W and Blue Star) is made in Figure 16. Each value shown for US 40W is the average of about 550 measurements. The spring test values for Blue Star are the average of about 150 and the fall test values of about 300 measurements. Figure 16 indicates that for both pavements and both seasons the values in the outer wheel path are considerably larger than in the inner wheel path.

Similar graphs of the average deflection in the four wheel paths of the four-lane divided highways (US 40E and US 240S) are shown in Figures 17 and 18, respectively. Each value for the spring tests of US 40E is the average of about 140 measurements, the first series of readings being excluded, whereas the values for the fall tests include 280 measurements. The values for US 240S are the average of 300 measurements.

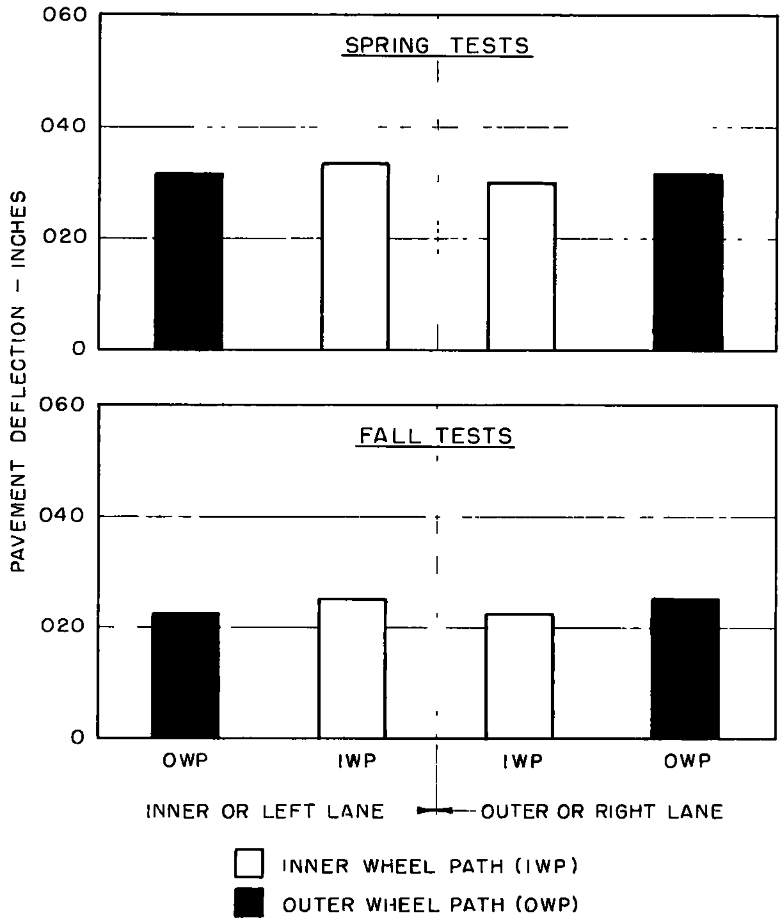


Figure 17. Comparison of average deflections in the four wheel paths of the eastbound lanes - US 40E, 11,200-lb wheel load.

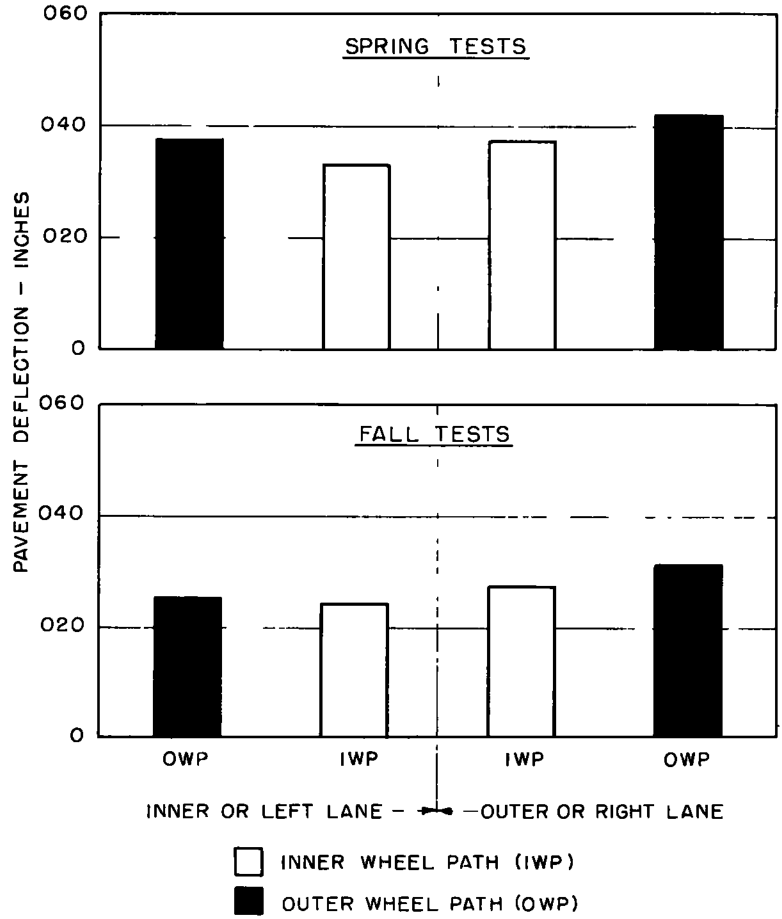


Figure 18. Comparison of average deflections in the four wheel paths of the southbound lanes - US 240S, 11,200-lb wheel load.

The relations for US 40E indicate that in the outer lane the deflection in the outer wheel path is slightly the larger, whereas in the inner lane, the reverse is true. The relations for US 240S show a somewhat more marked difference between the values in the two wheel paths of both lanes. However, the differences are much less pronounced than are those of the two-lane highways. Also, the inner or left lane deflection values are somewhat less in magnitude than those of the outer or right lane.

A comparison of the grand average deflection values for all four test series on the four pavements studied is shown in Figure 19. The values plotted are the averages of measurements made in all lanes and wheel paths. The following comments may be made concerning these data:

1. With the exception of the spring 1955 value for US 40E, all deflection values are between 0.022 and 0.038 in.
2. The deflections of US 40W are generally the smallest and, with the exception of the first test series of US 40E, those of US 240S the largest.
3. The values obtained in the spring are in all cases the largest,

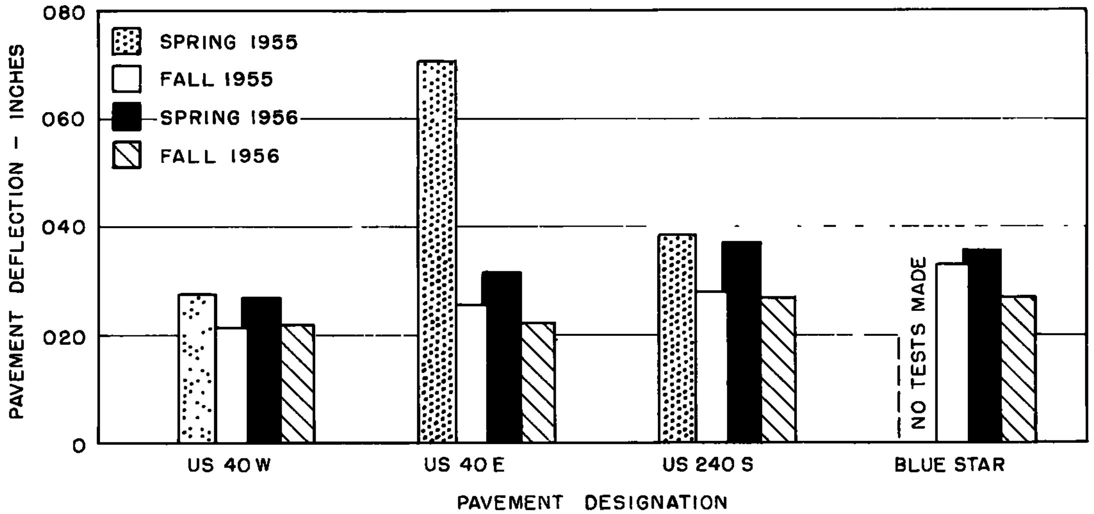


Figure 19. Comparison of average deflections of the four pavement sections for each test series - 11,200-lb wheel load.

that of the initial test series of US 40E being exceptionally large (about 0.070 in.).

4. For the two older pavements (US 40W, 11 years, and US 240S, 2 to 4 years) the seasonal changes in deflection have been of appreciable magnitude, but quite constant. On the other hand, the seasonal variation between the spring and fall of 1956 on the two newer pavements is somewhat greater than that between the fall of 1955 and spring of 1956.

SUMMARY

As mentioned earlier, this is a progress report on a study of the structural performance of certain high-type flexible pavements in service by means of load-deflection tests. Fairly detailed descriptions of the test procedure used, the pavements studied, and the more interesting findings obtained, are included.

Studies of this type were made possible by the development in 1953 of the Benkelman Beam pavement deflection indicator. The test procedure was developed to make maximum use of this instrument in the time available. The four pavement sections studied are of modern design, range in age from 1 to 11 years, are on moderately traveled routes, and are in excellent condition.

The average deflection in April 1955 for the stage-constructed pavement (US 40E) that showed signs of distress before receiving the final surface course, was about 0.070 in. However, many individual measurements exceeded this value and a few exceeded 0.100 in. Because of the excellent performance of this pavement since the addition of the final surface course, and the similar performance of the other three pavements to date, no other correlation could be made between their structural adequacy and total deflection. With the exception of the value previously mentioned, the grand average deflections for all seasons on all pavements ranged from 0.022 to 0.038 in.

It was found that the magnitude of the deflections of a certain pave-

ment of constant design may vary to a great degree from site to site, and even at different points at the same site. Also, that in spite of the comparatively large deflections at some test sites for the spring test series there is no evidence of structural distress at these sites. An attempt to correlate the magnitude of the pavement deflection with the results of the subgrade soil surveys made prior to construction has not shown any definite trends.

The residual values generally range from 0 to 0.015 in., but comparatively few values are larger than 0.010 in. They tend to vary directly as the deflection and, in general, range from about 15 to about 25 percent of the deflection values. Although the residual values seem quite large, only one of the four pavements studied has a measurable amount of permanent settlement or consolidation in the wheel paths. It is believed that most or all of the residual movement is eventually recovered by slow elastic action and the "ironing out effect" of traffic.

The magnitude of the deflection of the outer wheel path of the two-lane highways is considerably larger than that of the inner wheel path—about 40 and 25 percent for the US 40W and Blue Star pavements, respectively. On the other hand, the differences are not as marked in the case of the four-lane highways. In fact, for the US 40E pavement the deflection of the inner wheel path of the inner lane is larger than that of the outer wheel path.

The effect of changes in climatic conditions on pavement deflection is quite marked, the spring values being the greater. The seasonal variations between spring and fall have been generally quite constant. It was found difficult to schedule the tests for the most adverse period in the spring, there being indications that both spring test series followed the spring breakup period by several weeks.

It is expected that periodic observations of these pavements will be continued in the future in an effort to establish a correlation between pavement performance and deflection under a single-wheel load.

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Discussion

W. H. CAMPEN, Manager, Omaha Testing Laboratories, Omaha, Nebraska—Can it be concluded from this study that flexible pavements can tolerate about 0.05 in. total deflection without cracking and, consequently, failing the bituminous mats? This question is prompted by the fact that Hveem in 1955 (HRB Bulletin 114) indicated that a much lower deflection would cause failure.

STUART WILLIAMS and ALLAN LEE, Closure—A conclusion such as suggested cannot be drawn from the results of this study. Because the pavements observed are now generally in excellent condition, the data are not of sufficient scope to justify broad conclusions. Perhaps as data accumulate it may become possible to correlate structural adequacy and total deflection.

Flexible Pavement Design as Currently Practiced in Georgia

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Flexible pavement designs as currently practiced in Georgia are primarily based on the study of the foundations occurring along the projects by means of surveys, analysis of samples representing the various horizons of the foundations, and treatment of the foundations according to their strength and the condition under which they serve, all combined with a uniform depth base and surfacing, utilizing the available materials either existing in the vicinity of or economically accessible to a project.

This paper describes the basic factors considered and the manner in which they are incorporated into flexible pavement design.

●BECAUSE THERE ARE so many variables and factors which have not been evaluated but which influence the design and construction of a satisfactory flexible pavement, no rational formula is used for design of such facilities in Georgia. Instead, the several factors and variables are studied individually and then combined to give what are considered to be adequate pavements. The factors studied include the results of surveys of the foundation materials, in both cuts and fills; surveys of existing deposits of materials, both in the vicinity of and commercial materials economically available to the project; the location of the project; and the present and future anticipated traffic.

Although several types of flexible pavements are built, they all have the same basic design in that provisions are made to strengthen the weaker sections of the foundations by placing over these sections stronger and more stable materials in varying depths so as to result in a reasonably uniform strength subgrade throughout the length of a project. Over this subgrade is placed a base of uniform thickness commensurate with the basic factors previously given and the thickness of the surface course chosen.

FOUNDATION MATERIAL CLASSIFICATION

For general construction purposes, foundation materials are divided into six classes, which from description and field observation can usually be recognized and treated accordingly. The general descriptions and uses of the classes are given in Table 1.

The six classes nicely cover the whole range of natural materials encountered in roadbed construction and are found to be invaluable in obtaining a satisfactory foundation. As previously stated, they can normally be differentiated by field observation and examination; but for judging their value for flexible pavement design, more definite methods of examination are necessary for the first three classifications (see Table 2).

TABLE 1

GENERAL CLASSIFICATION OF FOUNDATION MATERIALS

Group Class	General Description	Usual Types of Significant Constituent Materials	Characteristics and Uses
I	Friable soils	Sands, low clay content sand-clays, sand-gravels, talus materials, cherty materials, and marls	Under proper compaction, generally can be used to full height of embankment, generally do not need subgrade treatment.
II	Plastic soils with low volume change	High clay content sand-clays, sand-clay gravels, cherts, and silty materials	Require careful compaction and moisture control, generally need a topping layer or special treatment to obtain satisfactory subgrade
III	Highly plastic and highly expansive soils	Clays, fine-grained dis-integrated granitic rock, and clay marls	High volume changes or detrimental expansive properties, normally wasted or placed either in the outer areas or bottom of embankments where the unstable features will have little or no effect on the service value of the embankment. If occurring in excavation, they are undercut and drained so that as nearly constant moisture content as possible is maintained Require careful compaction and moisture control, when used as foundation material, need topping layers or special treatment to obtain a satisfactory subgrade
IV	Peat, muck, and organic soils	Highly-organic soils, peat, muck, and other unsatisfactory materials usually found in marshy or swamp areas	Not satisfactory for embankment purposes, usually removed and wasted If used, require special consolidation and topping layers to obtain a satisfactory foundation and subgrade.
V	Laminated materials	Shale, slate partially disinte-grated schists	Detrimental weathering properties and tend to disintegrate Use in embankment requires careful compaction and moisture control, large particles must be broken down until at least 40% passes No. 4 sieve. Generally need some form of treatment for a satisfactory subgrade
VI	Rock	Ledge rock, boulders	Cannot be readily incorporated into an embankment by layer construction, requires careful distribution of the particles to avoid pockets and voids. In excavation, undercutting and a topping layer is needed to obtain a satisfactory subgrade, with the rock surface so finished as to furnish proper drainage for the subgrade

TABLE 2

LIMITATIONS OF THE VARIOUS CLASSIFICATIONS OF FOUNDATION SOILS

Foundation Material Group Classification	Foundation Soil Classification	Dry Density, lb/cu ft*	Total Volume Change, %
I	I	100 to 165	0 to 15
II	II-A_1	100 to 165	15 to 25
	II-A_2	90 to 100	0 to 25
III	III-A	90 to 165	25 to 50
	III-B	80 to 90	0+
	III-C	80-	0+

* At 2.65 specific gravity.

Soil surveys are made on each project, with representative samples of the various horizons of the materials encountered being submitted to the laboratory for analyses.

The soil survey consists of drillings made along the centerline of the project, with occasional supplemental soundings to the right and left of the centerline. On the four-lane and wider projects, a three-line survey is made; two of the lines being at some equal distance each side of the centerline with random additional borings being made to secure definite limits of the soil horizons in the cross-section. As the borings are made, the materials in each horizon are given a field identification or description, together with its thickness and extent along the project. All this information is placed on the identification card accompanying the sample. This field description, together with the tests made in the laboratory, enables the engineer to make the correct identification for classification. The tests conducted in the laboratory include gradation, Proctor density, and total volume change from dryness to saturation.

From the results of these tests and the field description, the classification of the material is made for the purpose of designing the subgrade for flexible pavements. Development of this classification, resulting from the study of foundations and subgrades under many hundreds of miles of flexible pavements, has been previously discussed (1).

In general, Class I corresponds to A-1, A-2, and A-3 of the Bureau of Public Roads Soil Classification System; Class II to A-4 and A-5; and Class III to A-6 and A-7.

Each of these classes of materials carries its own depth of subgrade treatment. Class I material conforming to the required subgrade specification requires no treatment; otherwise, a 6-in. treatment. Class II-A_1 carries 6 in.; II-A_2 9 in.; III-A 12 in.; III-B 18 in.; and III-C 24 in. of treatment.

During grading operation the foundation is undercut to the depth required in accordance with these schedules, and backfilled with selected subgrade treatment materials. This furnishes a reasonably uniform subgrade of at least 6 in. in depth throughout the projects. Figure 1 shows the depth of undercut generally made. These subgrades all conform to uniform

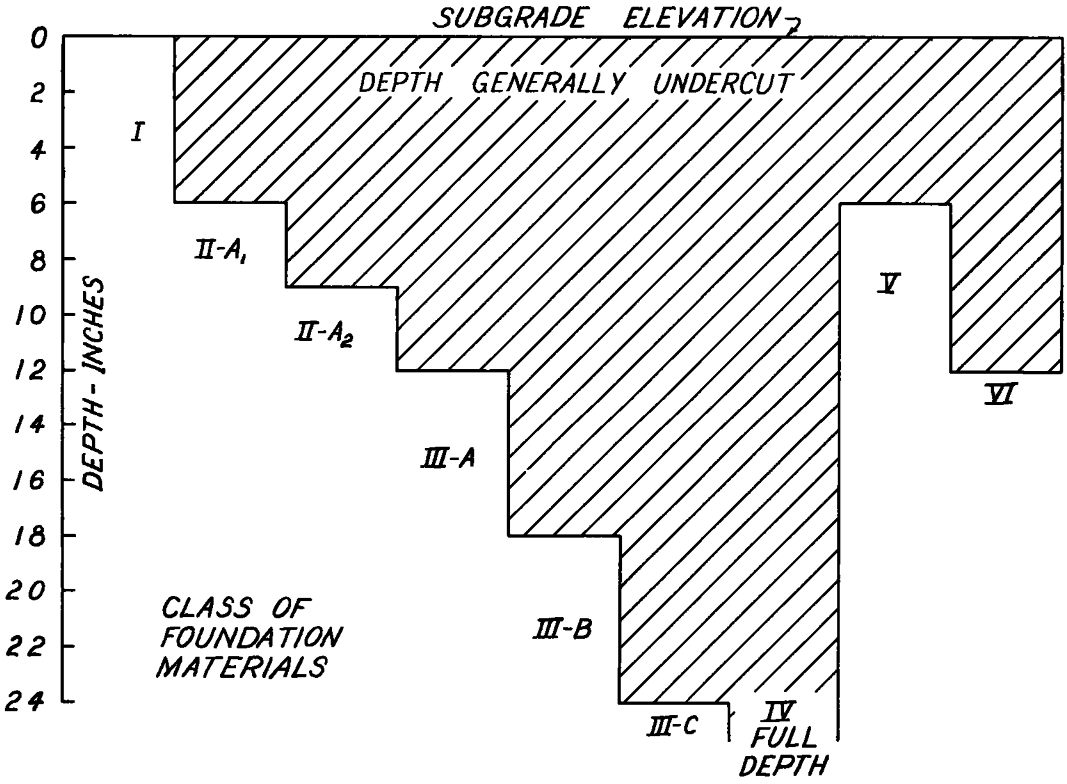

Figure 1. Depth of undercut.

specifications and are required on all projects, regardless of the type and volume of traffic to be carried.

SUBGRADE MATERIALS

The subgrade materials usually consist of the best selected soil materials existing in the vicinity of a project. These soils have varying gradation characteristics, depending on the section of the state in which the projects are located. In the Coastal Region they are usually sands or sand-clays; in the Piedmont Region or the central part of the state they are usually sand-clays, topsoils, or sand-clay gravels; and in the northern region or mountainous part they consist of cherts, talus, or river terrace deposits of sand-clays and sand-clay gravels.

In all cases, wherever possible from an economical point of view, where the undercutting required is 12 to 24 in. selected borrow of at least a Class II material is taken from the cut excavation and placed on the foundation material so that not more than 6 to 9 in. of the more stable subgrade treatment material is required.

In all instances the top 6 in. of the subgrade or subgrade treatment is chosen so as to have a volume change not in excess of 12 percent, but satisfying the other requirements of density and gradation, so that a highly stable material is obtained in the top 6 in. of the subgrade. For comparison purposes this usually conforms to about an A-I material in the Bureau of Public Roads classification.

In many instances materials conforming to the subgrade specifications are not economically available in the vicinity of the project, particularly in the southern sandy regions. In such cases, fines (such as fine silts or friable clay silts) are artifically mixed into the top 6 in. of the sandy material. In other cases the Class II materials, which are the clays, are stabilized with coarse granular materials (sands, stone screenings, etc.), using whichever type may be most economically available.

BASES

From the finished subgrade the bases and surfaces are designed according to the classification of the road, anticipated traffic, type of materials available for use, and other local features, such as water tables, anticipated rainfalls, drainage conditions, and type of traffic expected to use the road. Bases are normally 8 in. in depth, with occasional bases 6 in. or 10 in. deep.

Studies are made in the vicinity of the projects to determine the type of local material suitable for use in base construction.

In the sandy Coastal Region, bases on farm-to-market roads are usually 6 in. sand-bituminous or sand-limerock. The sands are chosen to have a 50-lb stability by the Florida bearing test and the roadway is required to have a minimum of 12 in. of compacted sand conforming to these specifications. This gives the top 6 in. of subgrade high stability, and the 6-in. base is then stabilized with either the bituminous material or crusher-run limerock.

Through the central part of the state the bases are constructed with local sand clays and topsoils, which normally have a weight per cubic foot of not less than 120 lb and a volume change of not more than 6 percent. If the project is anticipated to take heavier traffic than the farm-to-market volume, the top 4 in. of the base is stabilized by scarifying in 1 cu ft of crushed aggregate ranging in size from about 1 in. to $\frac{1}{4}$ in.

In the northern or mountainous section of the state the bases are built of either crushed chert or crushed talus containing a minimum of 60 percent of coarse aggregate retained on the 10-mesh screen and 100 percent passing the $1\frac{1}{2}$-in. screen.

On all primary projects the bases are usually constructed of soil-bound macadam, which is a stone base having the voids filled with a local binder either occurring naturally or artifically produced so as to have a required gradation and a volume change of not more than 6 percent.

All bases, regardless of the type of road, are primed with approximately 0.2 gal per sq yd of a bituminous material after they have been consolidated to 100 percent Proctor density. They are then maintained under rolling for a period of several days to allow curing to take place. After the primes have cured sufficiently, various surfaces are placed thereon.

SURFACES

The bituminous surface consists of either the so-called double surface treatment or hot plant mixtures. A double surface treatment is constructed of an application of bituminous material on the prime, covered by a spreading of crushed aggregate of nominal 1-in. to $\frac{1}{2}$-in. size. This is "choked" with a small amount of crushed aggregate (size $\frac{1}{2}$ in. to No. 4) prior to a second application of bituminous material and a surface covering of material ranging from $\frac{1}{2}$ in. to No. 8. This type of surfacing is used on all farm-to-market, secondary, and medium-traffic roads.

The heavier-traffic roads carry about $1\frac{1}{2}$ in. of hot plant mix binder and a 1-in. asphaltic concrete surface course. To accommodate increased anticipated traffic volume and loads, these surfaces are thickened to as much as 3 in. of asphaltic concrete base course, $2\frac{1}{2}$ in. of binder, and $1\frac{1}{2}$ in. of asphaltic concrete surface.

Most hot plant mix surface course mixture designs are based on minimums of about 2,000-lb stability (Hubbard-Field), but sometimes the stability minimums are increased for more heavily traveled roads.

SHOULDERS

So far, the shoulders usually have been turfed. With a few excepceptions, the shoulder material is a selected local pervious topsoil or other material with sufficient stability to withstand traffic without excessive rutting, but also suitable for growing grass. The exception to the use of the pervious material is that because of the high rainfall (60 to 100 in. per year) impervious material is usually placed on the high side of the steeper curves to keep as much water as possible from seeping into the base and subgrade and consequently softening them.

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Evaluation of Pavement Systems of the WASHO Road Test by Layered System Methods

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This paper evaluates the pavement system for the WASHO Road Test by layered system methods. This involved analysis of the available data on the load bearing tests made on the prepared subgrade, on the compacted subbase, and on the top of the completed pavement. The evaluation of the prepared subgrade disclosed the uniformity and the rather high degree of improvement achieved by compaction of the 2-ft subgrade layer, and yielded values of elastic moduli of the natural subgrade and of the compacted subgrade. The evaluation of the compacted subbase, varying in thickness from 0 to 22 in. by steps of 4 in., was then made possible by layered system methods and yielded somewhat scattered values of the elastic modulus. The values considered to be the more representative were relatively high. Finally, an evaluation of the pavement system as a whole yielded values of the elastic modulus for the pavement layer of two different types (2-in. AC and 4-in. base, and 4-in. AC and 2-in. base), which were quite high. This evaluation indicates the relative excellence achieved in the construction of the subbase and pavement layers of the pavement system.

However, the results of the evaluation, although clearly indicating the potentialities and possibilities of the layered system method of analysis, were not as conclusive as they might have been, because the data were not entirely consistent and comparable, as they should be for such evaluations. The basic problem of, and the essential need for obtaining fully consistent and comparable data for such evaluations are discussed. The implications and the potentialities of this evaluation are discussed with regard to the responses and performances of the pavement system in the WASHO Road Test under the action of the repetitive traffic loadings.

●THE UNPRECEDENTED expansion of major expressway and turnpike systems in the United States within the past few years and the proposed multi-million dollar Government-sponsored road program make imperative the development of adequate scientific methods for evaluation, design, and construction of highway pavements. A clear understanding of the fundamental nature of layered pavement system phenomena and correct conceptions regarding the mechanics, load-spreading capacity, and stress-deflection responses are essential, as a first prerequisite.

Every important advance in science and engineering has stemmed from theoretical working hypotheses, which have brought phenomena into the realm of greater certainty and have served as a guide to experimentation and investigation. It should be realized, however, that no theory or statement of physical laws in any science or engineering is complete in its present form. It cannot fully and adequately include, explain, and take into account present apparent "exceptions to the rule," outside its realm of validity of limiting boundary conditions. Yet such theories and statements of physical laws have provided the essential stimulus and guide to major scientific advances, and have established the nature and basic form of the physical laws governing phenomena.

Two- and three-layer system problems presented in 1943 (1) and 1945 (2) provide working hypotheses regarding the basic functional form of the physical laws which govern the mechanics and stress-displacement responses of layered systems. This physical form of the layered system equations is mathematically and dimensionally correct, as a first requirement. It is necessary, however, to learn how to make valid interpretations and applications of the elastic theory in dealing with the problems involving the stress-displacement responses of an imperfectly elastic material such as soil.

In contrast either to unquestioning acceptance or to completely skeptical rejection, the major problems now are those: (a) of thoroughly and competently testing a working hypothesis against observed phenomena, both in the field and in carefully controlled experiments; (b) of comprehending and establishing the nature and importance of the inherent limitations and the reasonable realm of validity; and (c) and most important of all, of evaluating and reliably establishing the regions in and the degree to which real soils and real conditions may be expected to agree with and/or to depart from the idealized conditions of the working hypothesis, particularly with regard to the stress-displacement responses of layered systems.

It has been argued that not enough is yet known about the stress-strain responses of soils in homogeneous deposits to justify any investigations and applications of theory to layered systems. This attitude is unrealistic and has led to an increased and undue emphasis on the empirical-correlation approach, which has definite limitations and shortcomings.

On the contrary, a working hypothesis, imperfect though it may be, can increase understanding and knowledge of phenomena, and can tell the investigator better what and how to observe significantly through fundamental parametric relations. Further, it can enable the investigator more completely and correctly to interpret and evaluate the results of observations and measurements. With more clearly defined objectives experimental investigations can work to greater advantage, and can be made to yield results having maximum usefulness and reliability.

Thus, the hypothesis of stresses and displacements in layered systems can become a powerful and competent scientific tool for interpreting, analyzing, and evaluating layered pavement system phenomena and responses for purposes of pavement design and construction control. But it must be realized, from the very nature of layered system phenomena, that the use and applications of such a working hypothesis can never become merely a routine unimaginative matter, but must always be a matter of basic knowledge and understanding, of training and competence, of engineering imagination and feeling regarding the nature of phenomena, and of common sense and judgment.

LAYERED SYSTEM ACTION AND RESPONSES

The principal problems involved in design and construction of satisfactory and economical pavement systems are those of limiting accumulated permanent surface settlements to non-objectionable values; of insuring the permanence, integrity, and stability of the pavement system against failure under repeated wheel loadings; and of increasing the expected life of pavements. In design and construction of pavement systems, the engineer is always dealing with multi-layered systems, consisting of an asphalt or concrete pavement layer, a compacted base course of selected crushed stone or gravel, sometimes a subbase of compacted gravel or selected material, and a natural or processed and compacted subgrade soil layer.

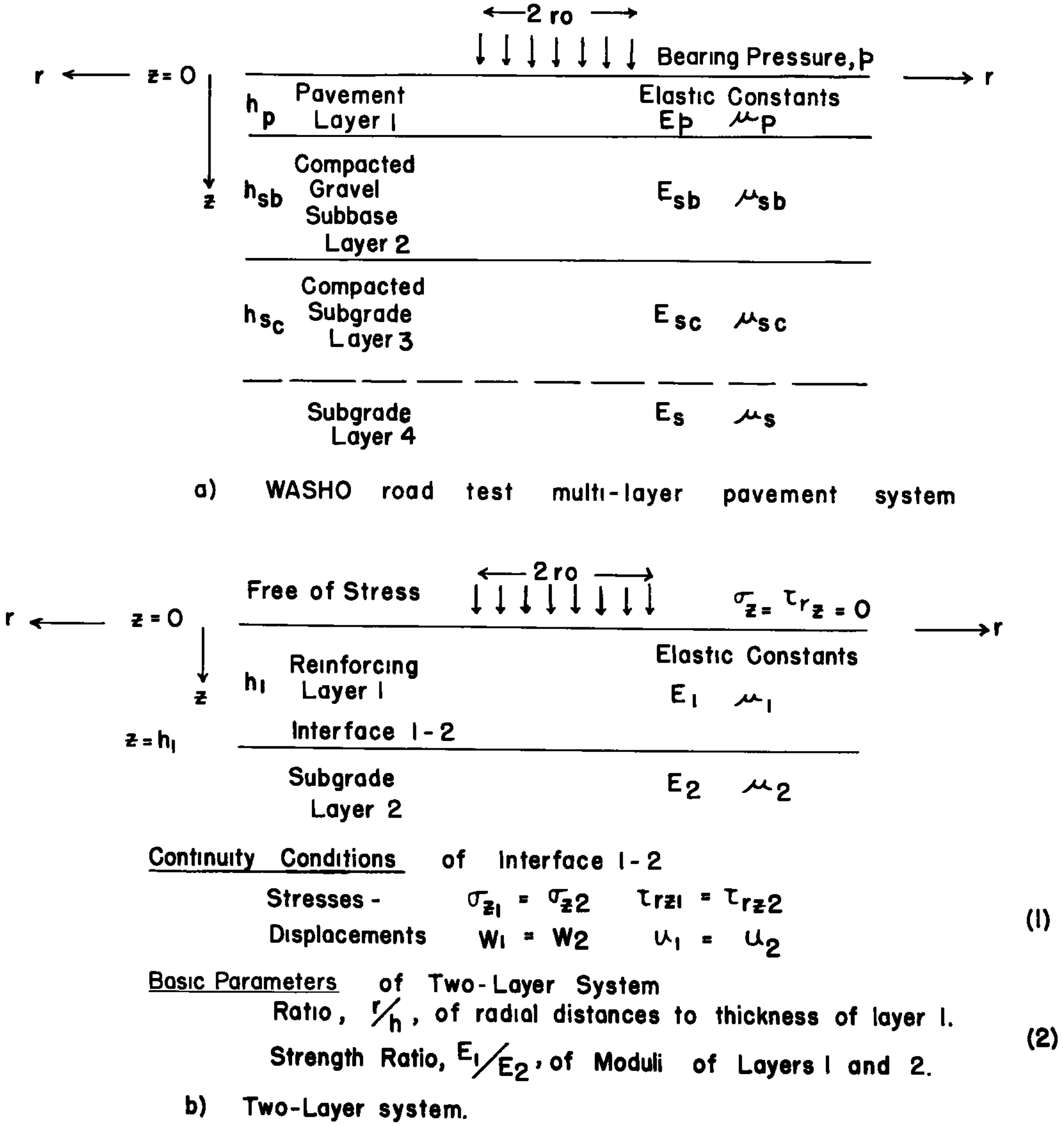


Figure 1. Layered pavement system notations.

The subgrade soil must ultimately support the wheel loadings imposed by traffic on the pavement system. The combined pavement, base course, and subbase layers have an important reinforcing and load-spreading capacity in protecting the supporting soils of the subgrade layer, which are very favorable aspects in reducing the stressing and deforming of the subgrade soils. These important stress distribution and deformation characteristics of a layered pavement system should be fully comprehended and appreciated, and adequately investigated and taken into account in pavement investigations.

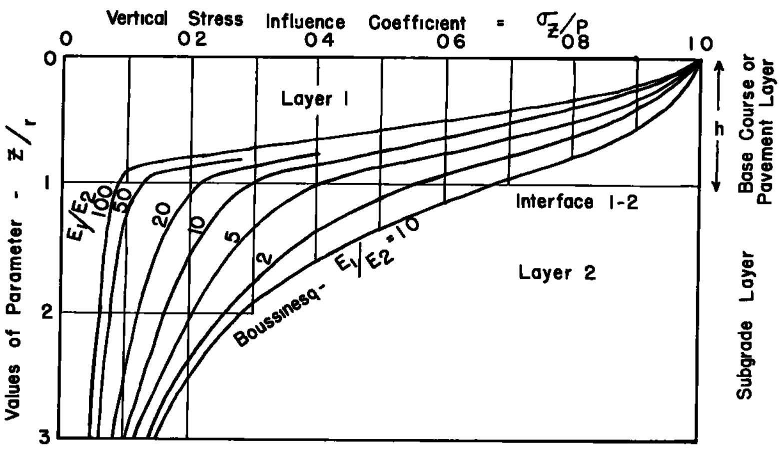
The two- and three-layer system problems (1,2) represent closer agreement to actual conditions encountered in design and construction of layered pavement systems. The basic boundary and continuity conditions and the fundamental parameters of the two-layer system are given in Figure 1 and Eqs. 1 and 2. The stresses and displacements of principal interest and concern are: (a) the distribution of vertical stresses, σ_z throughout the layered system; (b) the distribution of shearing stresses, τ_{rz} on the horizontal planes of the layer interfaces; and (c) the settlements or deflections, w, at the surface and at each layer interface beneath the loaded area. The magnitude and distribution of these stresses and displacements discloses the essential nature of the mechanics and effectiveness of layered system action.

Vertical Stresses

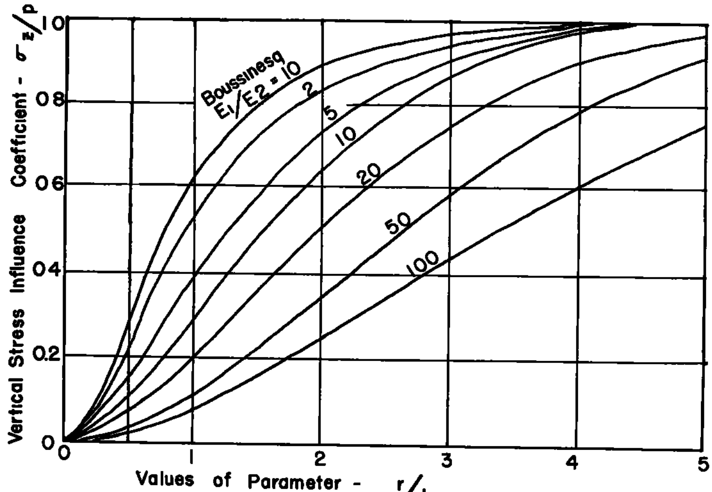
Figure 2a shows the distribution of vertical stresses in a two-layer system which are imposed beneath the center of a load uniformly distributed over a circular area for r/h = 1.0. This figure discloses significantly the effectiveness of the load-spreading and reinforcing capacity of a strong layer 1 of a layered system in protecting and in reducing stresses imposed on the subgrade layer, in comparison with the Boussinesq stress distribution ($E_1/E_2 = 1$) for a homogeneous deposit. Two important aspects are to be noted especially: (a) the marked and favorable increase in effectiveness of the reduction of vertical stresses imposed in the subgrade layer 2 with increase in the layer strength ratio, E_1/E_2, as a measure of the reinforcing and load-spreading capacity of a layered system; and (b) the marked increase in the vertical stress gradient within the lower portion of layer 1 toward the layer interface. The only possible mechanism by which such a negative vertical stress gradient and stress reduction can exist and be maintained in a layered system is by an equal positive shearing stress gradient and stress build-up in accordance with the stress equilibrium condition of the theory of elasticity:

$$\frac{\partial \sigma_z}{\partial z} + \frac{\tau_{rz}}{\partial r} + \frac{\tau_{rz}}{r} = 0 \qquad (3)$$

In addition, the effectiveness of the subgrade stress reducing capacity of a layered system is markedly influenced by the size of the loaded area in relation to the thickness of the reinforcing layer 1 through the basic parameter, r/h, as illustrated in Figure 2b. It is to be noted that the effectiveness of a layered system is greatly reduced by increase in the value of r/h (increase in r for a constant h), but at a less rapid rate for the larger strength ratios, E_1/E_2. This becomes an important and even controlling consideration, because the effectiveness of the reinforcing, load-spreading, and subgrade stress reducing capacity of a layered pavement system decreases unfavorably for a constant thickness of layer 1



a). Basic Pattern of Two-Layer Vertical Stress Influence Curves (σ_z/p vs z/r) for $r/h = 1.0$ and $\mu_1 = \mu_2 = 0.5$



b) Basic pattern of two-layer vertical stress curves (σ_z/p vs r/h) at the interface - $z = h$ $\mu_1 = \mu_2 = 0.5$

Figure 2. Effectiveness of two-layer systems in reducing vertical stresses imposed on the subgrade layer; Burmister problem; Fox stress influence coefficients (Ref. 3).

with increase in size (tire imprints) of the loaded area by approaching the Boussinesq stress conditions.

Shearing Stresses

The foregoing vertical stress gradient relations have a controlling and significant influence on the magnitude and distribution of horizontal shearing stresses induced in the interface region between two layers, as disclosed in Figure 3. The layered system shearing stresses at the interface become much more important and critical than in the case of the Boussinesq shearing stresses imposed at the same depth in a homogeneous deposit. Furthermore, they become more important with increase in the strength ratio, E_1/E_2, due to the marked increase in the vertical stress gradient toward the layer interface.

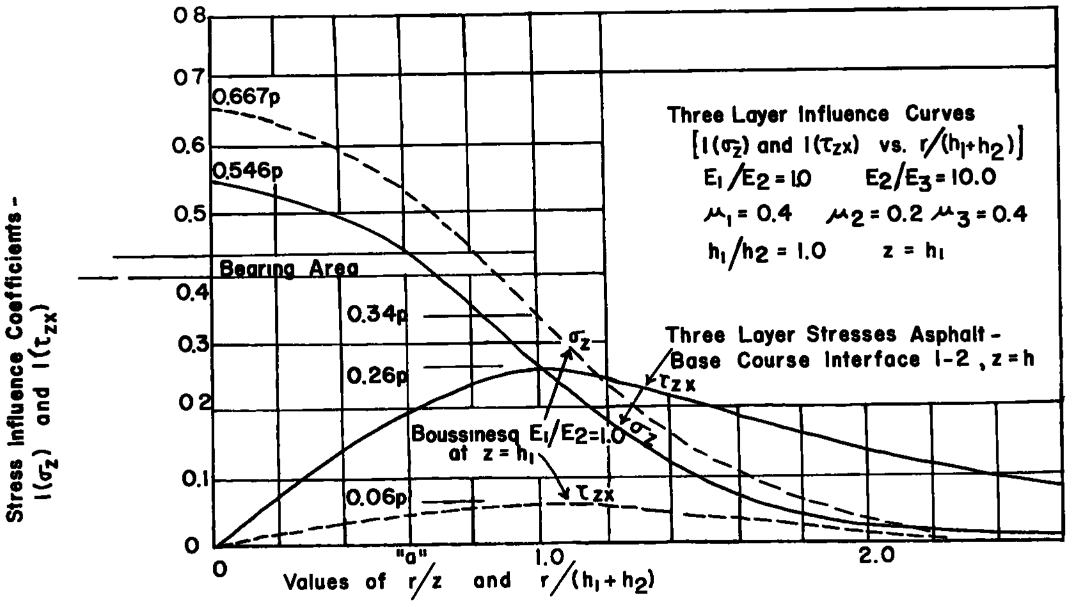


Figure 3. Evaluation of vertical stress and shearing stress conditions at the asphalt-base course interface for a three-layer system with regard to relative magnitudes and distribution on the interface, and to the potential critical conditions in comparison with the Boussinesq stresses for a homogeneous system; Burmister problem (Ref. 4).

These shearing stresses in the interface have most important implications with regard to their control of the effectiveness and permanence of the original continuity conditions incorporated into a layered system during its construction. The higher the shearing stresses which can be permanently and competently sustained at an interface, the more effective is the continuity between layers and the greater is the reinforcing capacity and stiffness of the layered system. But it also must become apparent that with increase in shearing stresses at the interface the interface responses become increasingly more critical with increase in the strength ratio and the accompanying increase in the reinforcing and load-spreading capacity of a layered system. A critical instability condition could develop where the imposed shearing stresses in the interface approach the maximum shearing strength which can be mobilized on the weaker side of the interface by deflections of the layered system under an imposed loading.

The critical nature of the shearing stress distribution at the first interface of a three-layer system (3) is disclosed in Figure 3, where the layered system shearing stresses are more than four times those given by the Boussinesq stress pattern. Furthermore, it is to be noted that the maximum shearing stress occurs beneath the edge of the loaded area. Here the layered system shearing stress becomes equal in magnitude to the vertical stress for the case noted. This is in marked contrast to the Boussinesq conditions in a homogeneous deposit at the same depth with shearing stresses only 17.5 percent of the vertical stress at that point. Thus, the τ_{rz}/σ_z condition can become much more critical in a layered system.

It is almost certain, therefore, that the breakdown in a layered pavement system would be initiated on the weaker side of an interface by excessive shearing straining and by accumulation of permanent inelastic shearing displacements at an interface under repeated loadings that induce shearing stresses approaching the shearing strength of the weaker soil. Such action would tend gradually to reduce the reinforcing and load-spreading capacity of a layered system by destroying the interface continuity, and correspondingly to increase layered system deflections to an ultimate breakdown and failure of a pavement system.

Deflections

These layered system shearing stress relations have important and significant influences on the deflections of a layered system at the surface and at each interface. The only possible mechanism by which horizontal shearing stresses at an interface can be mobilized and maintained is by relative horizontal straining of the upper and lower faces of the interface induced by deflections. The deflection mechanics of layered system action are disclosed in Figure 4 by the implicit dependence of the settle-

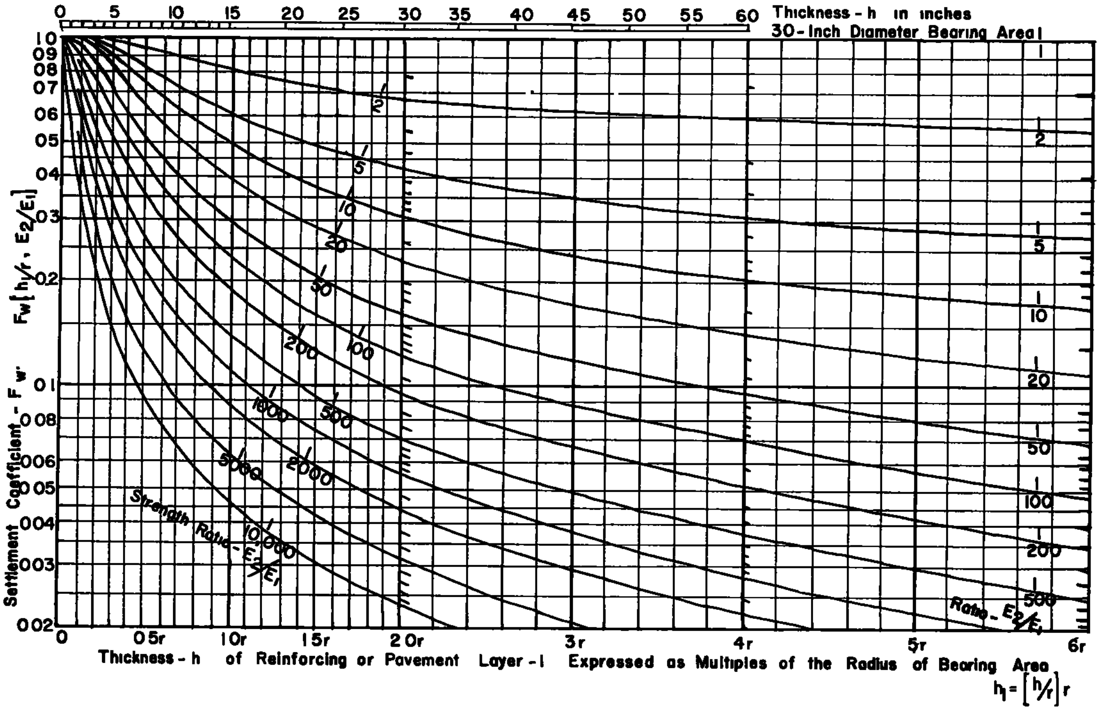


Figure 4. Influence curves of the settlement coefficient, F_w, for the two-layer system; basic load-settlement relation: $w_c = 1.5 \frac{p\ r}{E_2} \cdot F_w$

$$F_w = \frac{w\ E_2}{1.5\ p\ r} \qquad \mu_1 = \mu_2 = 0.5$$

ment coefficient, F_w, for surface deflections on the basic layered system parameter, h/r, and the strength ratio, E_1/E_2. For a constant h/r the load-deflection responses of a layered system are markedly improved with increase in the strength ratio, E_1/E_2, as indicated by a markedly decreasing value of F_w.

This decrease in surface deflections is evidence of the increase in stiffness and effectiveness of layered system action. The reinforcing and load-spreading capacity of a layered system are a consequence of this stiffness, which in turn is predetermined and maintained by the continuity conditions incorporated into the layered system during construction and by the maximum shearing stresses that can be permanently and competently sustained at the interface. The pattern of the curves for the two-layer settlement coefficient, F_w, shows that for a constant radius of bearing area the greatest improvement of layered system action can be achieved in the region where the curves are steepest by increasing the thickness of layer 1 from some minimum value toward a value corresponding to h/r = 1.0. Thereafter the improvement with increase in thickness becomes much less pronounced and effective, where the curves tend to flatten out. It should be noted that more marked and effective improvement can be achieved by selection and use of materials of higher quality and strength properties, and by actual constructional excellence in the field to attain the full potential strength of these materials.

Nevertheless, it is also significant to note that for a constant thickness of reinforcing layer 1 the effectiveness of layered system action decreases markedly with increase in the radius of bearing area or decrease in the value of h/r. The load-deflection responses are adversely affected. For thin thicknesses of layer 1, the layered system can become ineffective, with the value of F_w approaching unity, or the Boussinesq condition for a homogeneous deposit.

In view of these intimately interrelated aspects of layered pavement system action and responses, particularly with regard to the fundamental role of deflections in inducing shearing stresses at the interface, the load-deflection responses are a satisfactory measure of the effectiveness of the stiffness, reinforcing action, and load-spreading capacity of layered systems. The basic problem, therefore, appears to be that of limiting deflections to such magnitudes under design wheel loadings, so that the shearing stresses induced in the interfaces are well below critical breakdown values, and so that accumulated settlements under repeated wheel loadings will not reach objectionable values within the expected life of a pavement system.

Prestress and Continuity

Of greatest significance, however, is the fact that a layered pavement system is in reality a "prestressed" system with an essential continuity incorporated and maintained between the component layers of the system by high resistance to shearing action, so that the component layers act effectively together in the pavement system, as a continuum. During construction a high state of prestress and strength has been incorporated into each layer by heavy compaction. Furthermore, the construction of each layer increases the state of prestress and strength of the layer below, effectively confines that layer, and establishes an important shearing strength continuity between the component layers of the layered system. These qualities and conditions are determinative in the effectiveness of layered system action, the load-spreading and reinforcing capacity, and the stiffness and load-deflection responses.

These qualities and conditions cannot be determined and evaluated by laboratory tests on materials of the component layers. They depend on and are principally predetermined by the constructional excellence actually attained in the field. They can be disclosed and evaluated only by field load tests and by performance of the pavement system under traffic conditions in service. In view of these considerations, much greater importance must be attached to achieving the highest possible degree of constructional excellence in the field, in order to attain consistently and uniformly the full potential of these qualities and conditions of prestress and continuity, and to the selection and use of superior high-quality materials in construction.

METHODS OF EVALUATING LAYERED PAVEMENT SYSTEMS

The analysis of field load bearing tests is intended to disclose and to evaluate the effectiveness of layered system action. In the empirical approach various methods of plotting the observed data have been used, which yield only relative measures of effectiveness. There is an urgent need for a significant numerical evaluation of effectiveness, which is adequate for design and construction purposes. The essential role of the two- and three-layer system theory is a reliable evaluation of (a) the strength properties of the component layers "in place"; (b) the load-spreading and subgrade stress-reducing capacity; (c) the stiffness and load-deflection responses; (d) the critical regions and magnitudes of shearing stresses, particularly at the layer interfaces; and (e) the degree of potential constructional excellence and uniformity attainable. All these form essential bases for design and construction control of layered pavement systems.

To test the validity and competence of the layered system theory, it is necessary to show that it can explain layered system action and can bring into a unified consistent pattern all of the various methods of plotting load-deflection data commonly used in evaluation studies. This testing of the theory is made in Figures 5 and 6, where the different common plotting arguments are used in the different figures. The basic two-layer settlement equation of Figure 4 is used to describe the load-settlement responses in each figure. This equation is dimensionally correct and complete in its functional form. It differs from the Boussinesq equation principally in the dimensionless argument, or settlement coefficient, F_w, which describes the load-deflection responses of a two-layer system. The numerical value of F_w can always be computed directly from load test data, as noted (1). For each figure the basic load-deflection equation is rewritten in a form, which separates the plotting arguments of the figures. For all these figures the equation is solved for the y-axis argument of applied plate bearing pressure, p, or the applied total load, P. The pressure, p, and the total load, P, are made deflection-dependent for some selected constant deflection, w, which is considered to be significant or critical in the load-deflection responses of the layered pavement system being evaluated. This selected deflection must now appear specifically in the functional quantity in the brackets. The first term on the right-hand side of the solved equation is the independent plotting argument for the x-axis; the second term is the independent plotting argument for the family of curves; and the quantity in the brackets is the dependent functional relation consisting essentially of $\left[\frac{w\ E_s}{h\ F_w}\right]$, which fixes the positions,

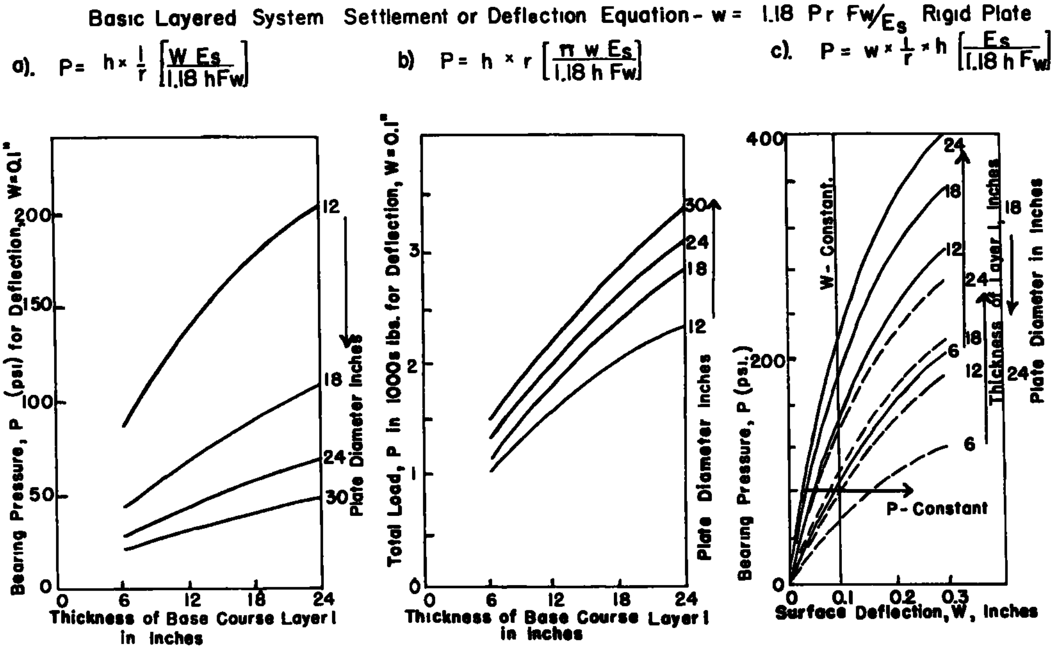


Figure 5. Relation between basic layered system parameters and curve patterns for different methods of plotting; testing of validity and competence of two-layer theory; E_1 = 30,000 psi, E_2 = 3,000 psi, E_1/E_2 = 10, $\mu_1 = \mu_2 = 0.5$.

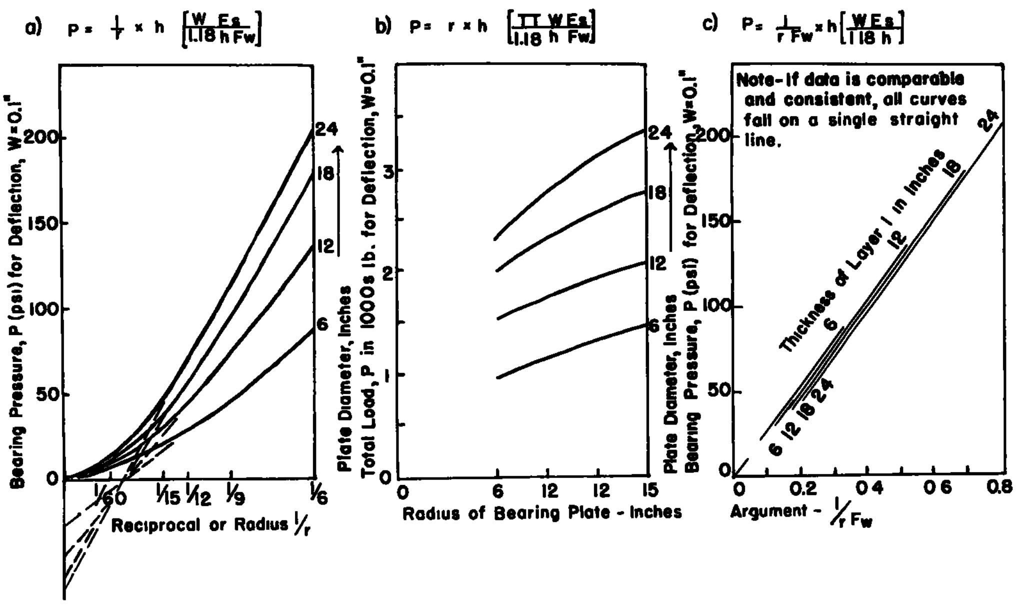


Figure 6. Relation between basic layered system parameters and curve patterns for different methods of plotting; testing of validity and competence of two-layer theory; E_1 = 30,000 psi, E_2 = 3,000 psi, E_1/E_2 = 10, $\mu_1 = \mu_2 = 0.5$.

slopes, and curvatures of the family of curves in each figure. For each figure the $\frac{w\,E_s}{h}$ portion of the quantity in the brackets is a constant. The deflection coefficient, F_w, however, varies for each curve and along each curve in accordance with the plotting arguments of the basic influence curves of Figure 4 of $[E_1/E_2,\ h/r]$, which define and describe its functional form.

It is to be noted that the order of increasing argument of the family of curves in each figure, as indicated by the arrow, is explicitly defined by the nature of that argument in the load-deflection equation for that figure. In Figure 5a the decreasing argument of plate size and the spacing of the curves is directly proportional to the value of the reciprocal, $1/r$, when pressure, p, is used as the plotting argument on the y-axis, whereas in Figure 5b the increasing argument of plate size and the spacing of the curves is directly proportional to the value of r when total load, P, is used as the plotting argument on the y-axis. The positions slopes, and curvatures of these curves for a constant value of $(E_2\ w/h)$ are determined by the values of $1/F_w$ from Figure 4. In Figure 5c both arguments of plate size and layer thickness describe the increasing or decreasing order of the respective families of curves, as in Figures 5a and 5b. This figure is of special interest, because it illustrates how comparable data may be selected for analysis and evaluation. Because of the general similarity of subgrade soil character and moisture-density conditions, of the quality of materials for the other component layers of a pavement system, and of construction conditions for a given pavement system being evaluated, the load-deflection curves are generally of similar form and character, as indicated. Comparable data may be selected on the basis either of deflections for some selected constant pressure, or of pressures for some selected constant deflection, provided all points of the selected data fall on similar homologous portions of the load-deflection curves. The portions of the load-deflection curves having only relatively slight curvatures before any break or considerable increase in curvature occurs, may be considered to be similar and homologous in form. If both pressures and deflections vary, greater care should be exercised to insure comparable data.

In using such selected data, it is the "secant modulus" through the pressure-deflection point which is evaluated by the analysis. For pressures or deflections lower than the selected values, the actual deflections will be less than estimated values, the smaller the discrepancy between the actual curve and the secant modulus, the more reliable is the evaluation. Extrapolations beyond the range of the selected values are not justified, because of the increasing importance of curvature of the load-deflection curve making estimates now increasingly too small. The selection of deflection or pressure values is important in order to cover a working range of pressure conditions for each component layer, and at the same time to keep to a minimum the discrepancies between the values indicated by the actual curve and the secant modulus. If these conditions can be reasonably satisfied, the influences of the imperfectly elastic behavior of soils can be kept to a minimum and the validity of the evaluations increased.

The two-layer theory is also competent to describe and adequate to explain the nature of two other common methods of plotting load-deflection data, as illustrated by Figures 6a and 6b. The fundamental theoretical argument for the x-axis is seen from the load-deflection equation for this figure to be the reciprocal of the radius, $1/r$, of the bearing area. Use of the "perimeter-area ratio" is merely an expedient to express this reciprocal, and it does not have any real physical or functional significance here. The phenomenon illustrated in Figure 6a is in marked contrast to that observed from data obtained from a series of load bearing tests on homogeneous soil deposits, using different sizes of bearing plates. Such data yield a straight line having a positive intercept on the y-axis.

It is essential to consider two facts in interpreting the curves of Figure 6a—first, a two-layer system with a strong reinforcing layer 1, which has a capacity to spread the load on the underlying weaker subgrade layer; and second, with increasing radius of bearing plate, the two-layer system becomes increasingly less effective in its reinforcing action and load-deflection responses, as disclosed in Figures 2 and 4. This decrease in effectiveness of the layered system with increase in bearing area is clearly evident in Figure 6a. From the functional form of the equation with increasing radius the curves must pass through the origin. Because the upper portions of these curves for a radius of bearing area smaller than about 15 in. is almost linear, it is common to plot them as straight lines in accordance with conditions encountered in homogeneous soil deposits. Straight lines averaged through the plotted points now appear to have an intercept on the x-axis and a negative intercept on the y-axis, which is of course seen to be an impossibility, in view of the functional form of the load-deflection equation. Where the curves begin markedly to curve toward the origin, the effectiveness of the layered system decreases considerably, so that the subgrade layer finally controls pronouncedly the deflection responses of the layered system.

In Figure 6b the plotting arguments of total load, P, and radius of bearing area, r, on the y- and x-axis for the same data produce quite different results, which are disclosed by the load-deflection equation for this figure. The family of curves, if approximated by a series of averaged straight lines, would now show a positive intercept on the y-axis. But actually these curves must pass through the origin with decreasing radius of bearing area. The change in character and form of the two sets of curves is a consequence in the second case of multiplying both sides of Eq. 6a by the area of the bearing plate, πr^2. In both Figures 6a and 6b the positions, slopes, and curvatures of the family of curves are fixed by the value of the quantity in the brackets, particularly by the quantity $1/F_w$ as defined by the influence curves of Figure 4 for the deflection coefficient.

The most interesting and illuminating new way of plotting the data is given in Figure 6c, where the argument for the x-axis is the quantity $1/(r\ F_w)$. The family of curves of Figure 6a now reduces to a single straight line through the origin, with all curves superimposed in their respective regions of layer thickness, as indicated. The slope and position of the single line is now determined, (a) by the selected value of the deflection, w, and (b) by the particular value of the subgrade modulus of layered pavement being tested and evaluated.

Figures 5 and 6 illustrate the validity of the basic load-deflection equation and its competence to interpret and to explain the nature of the observed phenomena. However, these methods of plotting observed data cannot by themselves disclose the strength properties and effectiveness of layered system action on a factual numerical basis. The quantities contained in the basic bracket, $\left[\frac{w\ E_s}{1.18\ F_w}\right]$, appear as a single number, and cannot be separated and evaluated numerically from such methods of plotting. This is the principal reason for the limitations and shortcomings of the empirical approach; hence, only relative measures of performance and effectiveness can be obtained from the empirical approach alone. No specific information on the strength properties of the component layers of a layered system can be determined. The influences of differences in the strength properties and of the basic parameter, r/h, on the effectiveness of layered system action and performance cannot be factually evaluated.

The layered system deflection coefficient, F_w, can always be determined numerically, as a first measure of the load-deflection responses of a layered system. An evaluation of the layered system through the family of curves of Figure 4 for the particular values of the coefficient, F_w, and of the parameter h/r disclose the fundamental strength ratio, E_1/E_2, of that layered system on a factual basis. The important influences of layer thicknesses and of materials of higher quality and strength ratios in relation to the range of tire imprint sizes expected can be studied, evaluated properly, and rated with regard to the effectiveness of layered system action, responses, and performances. The methods of evaluation of the strength properties of layered systems have been discussed and illustrated elsewhere (1).

EVALUATION OF THE WASHO ROAD TEST PAVEMENT SYSTEMS

An evaluation of the pavement systems of the WASHO Road Test was made on the basis of analyses of plate bearing tests by layered system methods. The information for this study was obtained from reports of the Highway Research Board (4,5,6). The analyses and evaluations of these data on the WASHO Road Test pavement systems is treated under the respective headings of the component subgrade, subbase, and pavement layers.

Subgrade Layer

A prepared subgrade layer 24 in. thick was constructed by processing and compacting soil in 6-in. lifts to specified densities under field control of moisture content. The identification (7) of the compacted subgrade soils with regard to character and range of soil material were made on the basis of data from Tables 2-1 and G-1 of Special Report 18 (4), and are given in Table 1 together with other significant soil identification characteristics.

The layout of the test road is given in Figure 7 for reference, showing the test loops A-B and C-D. The branches of the loops are designated with their respective pavement construction. The sections of each branch of the loops having 0-, 4-, 8-, 12-, and 16-in. thicknesses (nominal) of compacted gravel subbase are noted by letter designation and number.

Data on 14 plate bearing tests (6) made on 18-in. diameter plates in different sections of the WASHO Road Test loops were analyzed by layered system methods to evaluate the magnitude and uniformity of strength properties attained by compaction of the 24-in. compacted subgrade soil layer. An equivalent combined subgrade modulus for an 18-in. plate was determined for each test by solving for $E_s^{\prime}$ (= E_s/F_w) for the basic two-layer settlement equation:

$$w = 1.18\ p\ r\ F_w/E_s \qquad (4)$$

TABLE 1

IDENTIFICATION OF COMPACTED SUBGRADE SOILS OF THE WASHO ROAD TEST WITH REGARD TO CHARACTER AND RANGE OF SOIL MATERIAL AND OF CHARACTERISTICS

Identification Range- "SILT & CLAY-, trace- to little - medium to fine+ Sand"

to "CLAY- & SILT, trace- to little- medium to fine+ Sand"

Low- to Medium- Plasticity

				Single-Axle Loop		Tandem-Axle Loop	
Characteristic	Low	Ave.	High	AC 4-in.	AC 2-in.	AC 4-in.	AC 2-in.
Liquid limit	31.9	35.6	38.9	36.3	35.5	35.4	35.0
Plasticity index	6.1	9.7	12.9	10.1	9.5	10.1	9.4
Plasticity	Low-	Low+	Medium-	Medium-	Low+	Medium-	Low+
Percentage passing No. 200 sieve	80	87	95	87	86	90	85

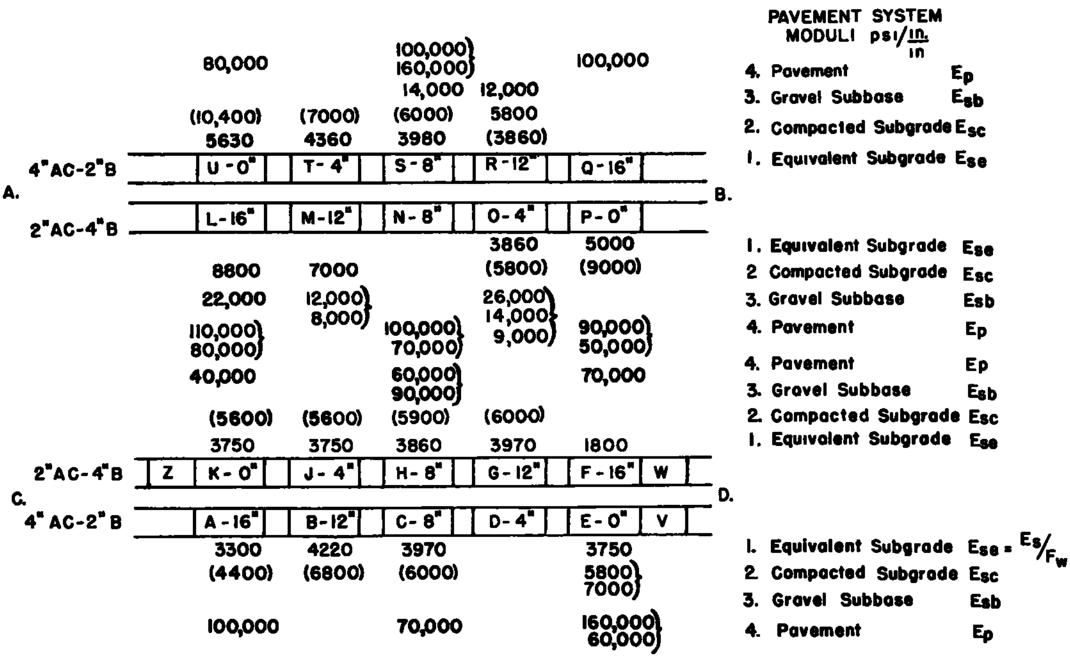


Figure 7. Layout of test loops of WASHO Road Test; summary of strength properties or moduli, E, of component layers of pavement systems. Evaluated by layered system methods on the basis of a subgrade modulus, E_S, of 2,000 psi. Test sections designated by letter and subbase thickness in inches. Moduli in brackets evaluated by layered system methods on the basis of a subgrade modulus, E_S, of 2,000 psi.

The plate bearing data for each test consisted of five repetitions each of four increasing applied pressures of 12.7, 32.3, 51.9, and 71.5 psi. A consideration of first importance in making these evaluations was to select pressure-deflection data, which are representative of and comparable to the load-deflection responses and layered system action of a pavement system to be expected under service conditions. The first cycle of stressing of a component layer of a layered system yields a deflection, which is generally erratic and relatively too large, and is not representative of performance responses. The net deflection obtained from the second reloading cycle of the first pressure loading of 12.7 psi was used for this evaluation. It was considered to be more representative of the initial deflection responses of the compacted subgrade, as a component part of a layered pavement system, and more comparable to that induced under stresses imposed at the subgrade level through a layered pavement system in accordance with its reinforcing and subgrade stress-reducing action, as illustrated in Figure 2a.

The values of the equivalent subgrade modulus, E_g', estimated on this basis are given in Lines 1 of Figure 7. Discounting the exceptionally low value (1,800) in Section F and the exceptionally high values in Sections P (5,000) and U (5,630), the low, average, and high values for the equivalent subgrade modulus are 3,300, 3,870, and 4,360, respectively. The average (3,870) of the equivalent subgrade modulus is considered to represent a good compacted subgrade. The range from 3,300 to 4,360 is considered to represent a good degree of uniformity of processing and compaction in the field, with a spread from about 85 to 113 percent of the average.

The strength properties of the compacted subgrade system were further analyzed to evaluate in some degree the influences of repeated cycles of the same pressure and of repeated cycles of increasing pressures on the deflection responses and strength properties of the compacted subgrade layer, and also to evaluate what pressures may be considered to be most representative of and comparable to probable actual performance conditions. The results of this evaluation are given in Table 2.

Study of Table 2 discloses certain significant facts regarding the deflection responses of this compacted subgrade layer. In the majority of instances (10 exceptions in 56 instances) there is a slight to significant increase in the equivalent subgrade modulus in the last cycle of repetition (5, 10, 15, and 20) for each applied pressure loading. This indicates that there is generally some increase in strength of the system by permanent consolidation under repeated applications of a given pressure. This is in accordance with the well-known favorable influences of traffic consolidation of a pavement system in the early stages of performance in service. The first application of a given pressure, starting with a low value of 12.7 psi, "stress conditions" a system by a relatively too large deflection and makes it stronger by permanent consolidation influences. Therefore, the second repetitive cycle for each new increased pressure is used for evaluations.

With subsequent repetitions of the same pressure, however, the system tends generally to become somewhat stronger by additional permanent consolidation, as noted by the increase in the average equivalent subgrade modulus from an initial value of 3,870 at the second cycle to 4,150 at the fifth cycle. With increase in pressure to 32.3 psi in the sixth cycle, the system is further "stress conditioned" by a large deflection and consolidation, so that in the second cycle of this pressure (7th of the series

TABLE 2

INFLUENCES OF REPETITIVE LOADINGS AND OF MAGNITUDE OF APPLIED PRESSURE ON THE VALUES OF THE EQUIVALENT SUBGRADE MODULUS

Section		22-4	18-2	14-4	6-4	22-2	18-2	14-2	10-2	6-2	10-2	6-2	14-4	10-4	6-2
Axle		S	S	S	S	S	S	S	S	S	T	T	T	T	T
		A	B	C	E	F	G	H	J	K	O	P	S	T	U
Test No.		9	8	7	3	4	5	6	10	11	13	12	16	15	14
Pressure, psi	Cycles	Values of E_s'													
		-	+	+	-	--	+	ave.	-	-	ave.	++	+	+	++
12.7	2	3300	4220	3970	3750	1800	3970	3860	3750	3750	3860	5000	3980	4360	5630
	5	3380	4480	4480	3850	1960	4320	3960	5400	3750	3860	4660	3970	4650	5400
32.2	7	3620	4650	4770	4280	2560	4570	4340	4340	4140	4230	5220	4000	4040	5450
	10	4130	4770	4920	4130	2730	4770	4450	4520	4130	4180	5260	4140	4140	5620
51.9	12	3700	4550	4710	4150	2920	4670	4180	4320	3860	3820	4960	3880	3940	5420
	15	3730	4630	4650	4170	3070	4870	4320	4340	3990	4140	4970	3940	3940	5200
71.5	17	3550	4250	4580	4130	3110	4980	4230	4070	3160	3170	4440	3660	3570	5020
	20	3650	4350	4580	4280	3390	4920	4170	4220	3320	4020	4600	3590	3710	5100
Moisture Content, w %*		22.2	22.9	23.9	21 9	24.2	22.9	22.0	22.2	22 0	21.4	20.6	21.1	22.3	20.2
Subgrade Densities, pcf, construction 1952**															
18	0-6"	85.6	91.2	87.7	87.9	87.6	88.1	85.6	87.3	86 0	88.3	85.7	93.8	93.8	93.1
32	6-12"	91.2	92.7	91.8	92.0	90.8	92.2	89.7	90.2	84.7	88.2	86.4	89.6	89.0	86.9
22	0-6"	90.1	82.3	85.7	87 8	89.6	91.0	85.1	87.7	88.0	86.5	89.2	89.8	93.0	86.9
40	6-12"	91.0	89.1	92.2	93.9	92.1	92.5	89.4	92.1	84.5	89 8	86.7	94.2	84.4	85.7
Max. density***			94.3				93.7				97.5		92.2		
Ave., pcf, w (opt.) %			24.6				24.7				22.8		22.9		

Range and averages of the equivalent subgrade modulus, E_s'

Pressure	Cycle	Low	Ave.	High
12.7 psi	2	3300	3870	4360
	5	3380	4150	4650
32.2 psi	7	3620	4270	4770
	10	4130	4380	4920

* Special Report 18, Tables F-1 to F-16, pp. 95-106.
** Special Report 22, Table 4-f-7, p. 120.
*** Special Report 18, Table 2-2, p. 11.

the modulus has increased to 4,270. At the fifth cycle of this pressure (10th of the series) there is a further increase in the modulus to 4,380.

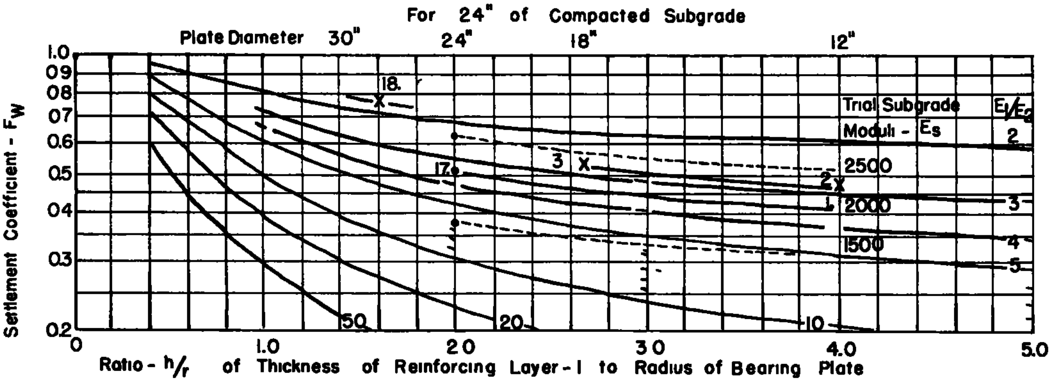
This phenomenon may continue for a number of applied pressure increases, but there is always a definite limit for each type of soil, degree of compaction, and other controlling conditions. For the subgrade soils of the WASHO Road Test, the limit was reached at a rather moderate pressure, the 51.9-psi pressure exceeding this limit. It is to be noted that for 12 out of 14 plate bearing tests (underlined) there is a significant drop in the equivalent subgrade modulus from the last cycle (10th of the series) of the 32.3-psi pressure to the second cycle (12th of the series) of the 51.9-psi pressure of 240 pressure average with a range from 200 to 430. There is also generally a much smaller increase in strength thereafter by consolidation under the fifth repetition of this pressure. The rather low and unusual pressure-deflection responses may have had important implications with regard to the performances of the subgrade soils in the pavement system, especially for the thinner thicknesses of subbase.

This phenomenon does give point, however, to careful consideration of the pressure range, which should be considered representative of and comparable to the probable actual imposed stresses at the subgrade level through the pavement system. Higher pressures tend to penalize the observed deflection responses and performances of a system. The equivalent subgrade modulus and the accompanying deflection responses of the second cycle through the fifth cycle of repetitions of the 12.7-psi pressure are considered to be more representative and significant of the performances of the compacted subgrade layer, as an essential component of the layered pavement system in all sections, excepting Sections E, K, P, and U, where no subbase was used. In these four sections the second applied pressure of 32.3 psi, or somewhat smaller, may be more representative of the stressing of the compacted subgrade layer. In view of these findings and of Figure 2, it is believed that 25 repetitions of the 12.7-psi pressure in all sections of 4-in. or greater subbase, and 25 repetitions of the 32.3-psi pressure or smaller in the four sections with zero subbase thickness, would have been representative and comparable to the probable actual conditions of stressing of the compacted subgrade layer. The increased number of repetitions might have disclosed some significant information on the influences of repetitions on the deflection responses and on a stable equivalent subgrade modulus, after some initial consolidation influences.

In addition, the departures of the equivalent subgrade modulus above and below the average, as noted in Table 2 by the plus and minus signs, were studied with regard to the influences of moisture content and soil density. The data were taken from tables of Special Reports 18 and 22 for the exact location and date, where given, or as closely as could be approximated from the data. There is some slight evidence in plate bearing tests 4, 12, and 14 (extreme low and high values of modulus) that moisture content of the subgrade was significant. In the average density data the underlining indicates the lane where the plate bearing test was made. It is probable that the upper 6 in. of compacted subgrade soil may have had greater significance on deflection responses, but this is indeterminate on the basis of average data for a section. Specific data at the location of plate bearing tests might have yielded some significant dependence of the modulus on density.

In Section E, where plate bearing tests were made on 12-, 18-, 24-, and 30-in. diameter plates, it was possible to evaluate approximately both the natural subgrade modulus, E_s, and the modulus of the compacted subgrade layer, E_{sc}, by layered system methods. This evaluation involved a graphical trial solution, equivalent to solving simultaneous equations. Such a solution requires data either on two or more sizes of bearing plates, or on different thicknesses of layers. The data must be comparable and reasonably consistent with regard to the strength properties incorporated into the respective layers at the several locations. This solution is illustrated in Figure 8, in which are plotted a significant portion of the pattern influence curves of the settlement coefficient, F_w, of Figure 4. The plate bearing test data are given in Cols. 1 to 6. The parameter, h/r, of the 24-in. compacted subgrade layer to the radius of the respective plates is given in Col. 7. Trial subgrade moduli, E_s, were selected and used in Cols. 8, 9, and 10, for which the corresponding settlement coefficients, F_w, were computed by the equation noted.

From the plotting of the values of F_w versus h/r it became evident that the data as a whole were not comparable, but rather a range of conditions was indicated. In analyzing the data for tests Nos. 1 and 17 it is to be noted that the curve (dotted) for a trial modulus of 1,500 crosses the pattern curves with increasing values of h/r from the bottom upward, and that the curve (dotted) for a modulus of 2,500 cross the pattern curves with increase in the value of h/4 from the top downward. The curve of best fit in conformity with the pattern curves is for a subgrade modulus, E_s = 2,000, as indicated by the heavy curve, which defined a ratio, E_1/E_2 = 3.5. This yields a compacted subgrade modulus of 7,000. This value is, of



1 Section	2 Test No	3 Station	4 Plate	5 p (psi)	6 w (in) Cycle No 2	7 h/r	8 Trial 1,500	9 Subgrade 2,000	10 Moduli, E_s 2,500	Corresponding Values of F_w	11 E_1/E_s	12 E_{sc}
E	1	160 + 58	12 in	28 6	0 042	4 0	0 312	0 416	0 520			
	2	159 + 50	12 in	28 6	0 047	4 0	0 349	0 466	0 582	← E_{sc} = 2,000 x	3 5	= 7,000
	3	160 + 79	18 in	12 7	0 036	2 67	0 400	0 534	0 667	2,000 x	2 9	= 5,800
	17	160 + 98	24 in	7 2	0 026	2 0	0 382	0 510	0 637			
	18	161 + 27	30 in	4 6	0 032	1 6	0 590	0 787	0 985	2,000 x	1 9	= 3,800

The evaluation yields a Subgrade Modulus, E_s = 2,000, and a compacted Subgrade Modulus, E_{sc} of 5,800 to 7,000

Average and Range of Equivalent Subgrade Modulus
E(eq) = 1 18 pr/w for 11 Load Tests

Range	Low	Average	High	p, psi
Cycle No. 2	3,300	3,880	4,350	12 7
Cycle No 10	4,130	4,380	4,920	32 3

Corresponding Compacted Subgrade Modulus, E_{sc}
for a Subgrade Modulus, E_s = 2,000

Range	Low	Average	High
Cycle No 2	4,400	5,800	7,000
Cycle No 10	6,000	7,200	9,000

Figure 8. Evaluation of strength properties of compacted subgrade system: 24-in. compacted. Influence curves of settlement coefficient, F_W of two-layer system; Burmister problem. Basic load-settlement relation:

$$w = 1.18\ p\ r\ F_W/E_S \qquad\qquad F_W = \frac{w\ E_S}{1.18\ p\ r}$$

course, considerably higher than the equivalent subgrade modulus, E_S', for the E section of 3,750 psi. In a similar manner the curve for tests Nos. 2 and 3 for a subgrade modulus, E_S = 2,000, of best fit lies above that for tests Nos. 1 and 17, and yields a ratio, E_1/E_2= 2.9, and a compacted subgrade modulus of 5,800 psi. The data for test No. 18 are completely out-of-step. Although the data as a whole are not comparable or consistent, it is important to note that the layered system method of analysis is competent to select data that may be considered comparable and consistent, thus defining a possible range of conditions within Section E, due to differences in construction and soil conditions.

To increase the validity and reliability of an evaluation, there should be a check by having at least three test values fall on the same conforming evaluation curve. Where only two values form the evaluation curve, there may be some doubt with regard to the adequacy of the evaluation. As a comparative basis, a subgrade modulus, E_S, of 2,000 was used for estimating the approximate compacted subgrade modulus, E_{sc}, at the bottom of Figure 8 for the range and average of the equivalent compacted subgrade modulus, E_S', to cover a useful spread of values from the second cycle of the 12.7-psi pressure to the last cycle (10th) of the 32.3-psi pressure. The appropriate value of the parameter, F_W, for each value of E_S' was obtained from the relation, $F_W = 2{,}000/E_S'$. Corresponding values of the ratio E_1/E_2 were then obtained for an 18-in. plate with an h/r value of 24/18 (= 1.33) from the pattern curves of Figure 8. This analysis then yields the value of E_{sc} = 2,000 E_1/E_2 for the average and range of the compacted subgrade modulus, E_{sc}, at the bottom of Figure 8. Likewise, values of the compacted subgrade moduli for the sections in Figure 7 are given in Line 2 of Figure 7, based on the values of the equivalent compacted subgrade modulus in Line 1. These values are considered to represent a very good degree of compaction and improvement of the strength properties of the subgrade soil.

This set of data illustrates the need for planning such load tests to insure representative and comparable data, which is the basic requirement. The 18-in. bearing plate location may be selected in a random manner in order to obtain coverage in accordance with sampling concepts and principles, and to indicate the relative uniformity achieved in the processing and compaction of the subgrade layer. But when a sequence of tests is planned for the express purpose of evaluating the load-deflection responses and strength properties of the component layers of a pavement system and of the system as a whole, random site selection is no longer permissible for the remaining plate bearing test locations in the sequence. The first priority here is to obtain data that are comparable and interpretable, and that are consistent within the group. This means that for each test of the sequence the soil conditions, moisture content, degree of compactness, and the incorporated strength properties must be of the same order and comparable for at least three out of a series of such tests, in order to obtain fully valid data and checks on the evaluations.

An agreement of two tests by an evaluation curve of best fit does not provide such a check. To insure such comparable conditions three plate bearing tests for the compacted subgrade should be made within 10 ft or less of each other in a regular pattern around the initial random selected location.

These requirements were not met, as indicated by the station numbers of the five tests in Figure 8; hence, erratic results may be expected. The uniformity of conditions for reliable and adequate evaluations has to be considerably greater than would be considered acceptable and satisfactory for the project as a whole. The spread either of the equivalent subgrade modulus, E_s', or of the compacted subgrade modulus, E_{sc}, is considered to be entirely satisfactory from the constructional excellence point of view, but it is too great from an adequate evaluation point of view.

The evaluation of the strength properties and of the effectiveness of the layered system is intended to disclose and to provide specific reliable values for one comprehensively tested location, whereas the random tests are intended to disclose the probable range of conditions and the degree of constructional excellence and uniformity achieved. On such a basis it may be possible to develop a rational and reliable approach and method of evaluation of layered pavement systems, which will serve as an adequate and reliable basis for design and construction control of layered pavement systems.

Subbase

The subbase was constructed to specified densities in 4-in. lifts. The planned thickness of the subbase varied systematically (0, 4, 8, 12, and 16 in.) in the sections of each loop, as shown in Figure 7. The actual thicknesses in some cases were somewhat less than these values. The compacted subbase was composed of selected well-graded gravel having a maximum size of 2 in. The identification (7) of the subbase gravel with regard to character and range of soil material, and a summary of average in-place densities in the different sections for each lift were obtained from data in Special Report 18 (4), as follows.

TABLE 3

IDENTIFICATION AND SUMMARY OF FIELD DENSITIES OF THE COMPACTED SUBBASE MATERIAL OF THE WASHO ROAD TEST

Identification Range-* "Coarse medium+ to fine GRAVEL, Little- to some- coarse to fine Sand, trace to little - Silt" Maximum size- 2-in.

Composition-	Range of percentages passing			Range of Moisture
Sieve-	3/4"	No. 10	No. 200	
%	66 to 87	18 to 33	2 to 11	2.8 to 6.1%

Summary of Field Densities by Sections and Lifts-**

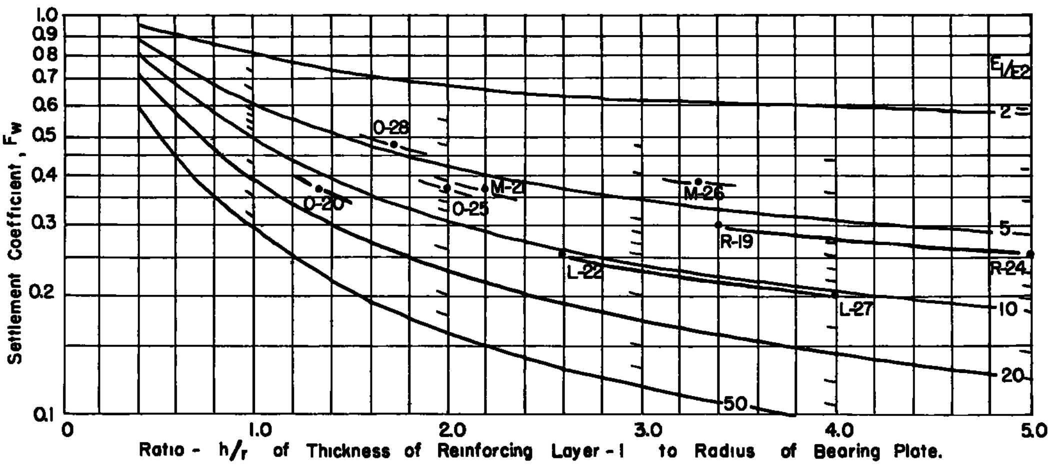
Single-Axle Loop 4 in. AC					Tandem-Axle Loop 2 in. AC				
U	T	S	R	Q	K	J	H	G	F
	135.4	140.0	129.0	128.8		133.0	139.7	124.4	133.0
		128.2	124.3	136.6			126.1	128.5	128.1
			127.7	131.2				129.5	132.8
				137.5					123.5

2 in. AC					4 in. AC				
L	M	N	O	P	A	B	C	D	E
128.7	128.2		129.1		129.6	133.2	133.9		
128.9	128.2	128.9			131.2	134.1	127.3		
125.9	127.2				132.2	128.9			
125.1									

* Fig. 2-20, Special Report 18 (4).
** Figs. F2-14 to F2-17, Special Report 18 (4).

Data from eleven plate bearing tests (6) were analyzed to evaluate the deflection responses and strength properties of the subbase-subgrade system. In these plate bearing tests the pressure was increased in four steps of one cycle each to the constant repetitive pressure sequence for the test of 62.0 and 70.4 psi on 12- and 18-in. plates, respectively. The evaluation was made on the basis of a subgrade modulus, E_s, of 2,000, which was derived from the evaluation of the subgrade plate bearing tests in Figure 8. The fifth loading cycle (2nd cycle) pressure of the constant repetitive application, was selected as being representative of stable initial deflection responses. The essential plate bearing test data are given in Cols. 1 to 7 of Figure 9. However, in view of Figure 2, pressures of 42.7 and 51.9 psi on 12- and 18-in. plates might be considered to be more representative of the stressing of the subbase as a component part of the completed pavement system.

The site locations were rather far apart for comparable conditions of soil character, compacted densities, and moisture content, and possibly other significant conditions that control, as seen from the station num-



1 Section	2 Test	3 Station	4 Plate	5 p (psi)	6 w (in) Cycle 5	7 h-Subbase	8 Total-h	9 Equivalent-h'	10 E_s	11 E_{sc}	12 E_{sb} (Subbase)
L	22	105 + 26	18 in	70 4	0 096	16	24 + 16 = 40	24 in	2,000	8,800	22,000
	27	105 + 06	12 in	62 0	0 043						22,000
M	21	109 + 30	18 in	70 4	0 139	12	36	20 in	2,000	7,000	12,000
	26	109 + 05	12 in	62 0	0 089						(8,000)
O	20	117 + 52	18 in	70 4	0 136	4	28	12 in	2,000	5,800	26,000
	25	117 + 20	12 in	62 0	0 082						14,000
	28	117 + 33	14 in	118 0	0 234						(9,000)
R	19	117 + 95	18 in	70 4	0 112	10	34	18 in	2,000	5,800	12,000
	24	116 +	12 in	62 0	0 057						12,000
			Range	Low	Average	High					
			Cycle 5	12,000	17,100	26,000				Aver	17,100

Figure 9. Evaluation of strength properties of compacted gravel subbase: equivalent thickness. Influence curves of settlement coefficient, F_W of two-layer system; Burmister problem. Basic load-settlement relation:

$$w = 1.18\ p\ r\ F_W/E_s \qquad F_W = \frac{w\ E_s}{1.18\ p\ r}$$

bers. Due to the fact, as noted previously, that adequate checks could not be made on evaluations on the basis of two comparable load tests per section, more approximate methods had to be resorted to in making these evaluations. The data on two load tests each in Sections L and R yielded by the approximate methods an evaluation line conforming to the pattern influence curves of Figure 9.

The evaluations in these cases involved finding and establishing by trial methods for a range of selected trial strength ratios, E_1/E_2, a total combined equivalent thickness, h', of 24-in. compacted subgrade plus compacted subbase, which has the strength properties of the subbase layer, such that the F_W and h/r values fall on and define an evaluation curve conforming to the pattern influence curves of Figure 9 in that region.

The data for Sections L and R were found to be sufficiently comparable and yielded solutions by this approximate evaluation method, as noted by the conforming evaluation curves in Figure 9 for the strength ratios, E_1/E_2, of 22,000 and 12,000, respectively. These two evaluations for 16- and 10-in. subbase thicknesses established the order of magnitude of the equivalent thickness, h' (= $h_s' - h_{sb}$), for very approximate evaluations of the remaining non-comparable test data, which were treated accordingly as single-point evaluations, as noted in Figure 9. The solution for Tests L-22 and L-27 will illustrate this approximate method of solution and evaluation in Table 4.

TABLE 4

APPROXIMATE EQUIVALENT THICKNESS METHOD OF LAYERED SYSTEM EVALUATION

24-in. Compacted Subgrade;	Subgrade, E_s = 2,000;		Compacted Subgrade, E_{sc} = 8,800		
Subbase, h_{sb} = 16 in.	Test 27, 12-in. plate		Test 22, 18-in. plate		
Pressure, p, psi	62.0		70.4		
Deflection, w, 5th cycle, in.	0.043		0.096		
$F_w - \dfrac{E_s\ w}{1.18\ p\ r}$	$\dfrac{2{,}000 \times 0.043}{1.18 \times 62.0 \times 6}$ = 0.197		$\dfrac{2{,}000 \times 0.096}{1.18 \times 70.4 \times 9}$ = 0.257		
Trial values of E1/E2	Values of h/r and equivalent h' corresponding to values of F_w				
	h/r	h	h/r	h'	
8	6 x 6	36	3.3 x 9	29.7	
10	4.4	26.4	2.8	25.2	
11	4.0	24.0	2.7	24.3	Solution
12	3.5	21.0	2.5	22.5	

Then, E_1/E_2 = 11 and E_{sb} = 2,000 x 11 = 22,000

Effective equivalent thickness of compacted subgrade having same strength properties as the compacted subbase is equal to 24 - 16 = 8 in., or 1/3 of 24 in. of compacted subgrade. This value is smaller than the ratio (8,800/22,000 = 1/2.5) due to less effective layered system action under an 18-in. plate as compared with the 12-in. plate.

The compacted subbase moduli, E_{sb}, for the different sections were evaluated in a similar manner and are given in Col. 12 of Figure 9, and also in Lines 3 of Figure 7. It is to be noted that there are quite definite indications that a higher degree of compaction, reinforcing action —hence strength properties—corresponding to E_{sb} = 22,000 can be attained on the thicker 16-in. subbase layer of Section L, where it was compacted on a stronger less yielding compacted subgrade layer having a higher subgrade modulus (E_{sc} = 8,800), whereas a thinner compacted subbase layer 10 in. thick constructed on a weaker more yielding compacted subgrade layer in Section R having a lower subgrade modulus, E_{sc}, of 5,800 yields a correspondingly lower compacted subbase modulus (E_{sb} = 12,000).

From the standpoint of constructional excellence the strength properties of the compacted subbase layer of Section R were about one-half the potentially attainable value of Section L. These evaluations tend to confirm this important fact tentatively established by other pavement evaluation studies. It may also be inferred and reasoned that in general the first 4 and 8 in. of the compacted subbase may not be as highly compacted as the upper 4-in. lifts, where constructed. There is some indication to this effect in Table 3.

The average and range of the subbase modulus, E_{sb}, for Sections L, M, O, and R, discounting the very low and very high values, which appear to be out of step, are as follows:

Cycle	Low	Average	High
5 (2)	12,000	17,000	26,000
18 (15)	14,000	19,000	26,000
	Thinner subbase Weaker subgrade		Thicker subbase Stronger subgrade

The degree of protection of the compacted subgrade provided by the compacted subbase layer and the effectiveness of the layered system action are given in Table 5 by comparison of the deflections at the top of the compacted subgrade, as a percentage of the deflection at the top of the compacted subbase.

TABLE 5

EFFECTIVENESS OF LAYERED SYSTEM ACTION ON DEFLECTION RESPONSES; INFLUENCES OF STRENGTH PROPERTIES OF THE COMPONENT LAYERS, AND SUBBASE THICKNESS; DEFLECTIONS AT INTERFACE AS A PERCENTAGE OF SURFACE DEFLECTIONS; FOURTH PRESSURE APPLICATION

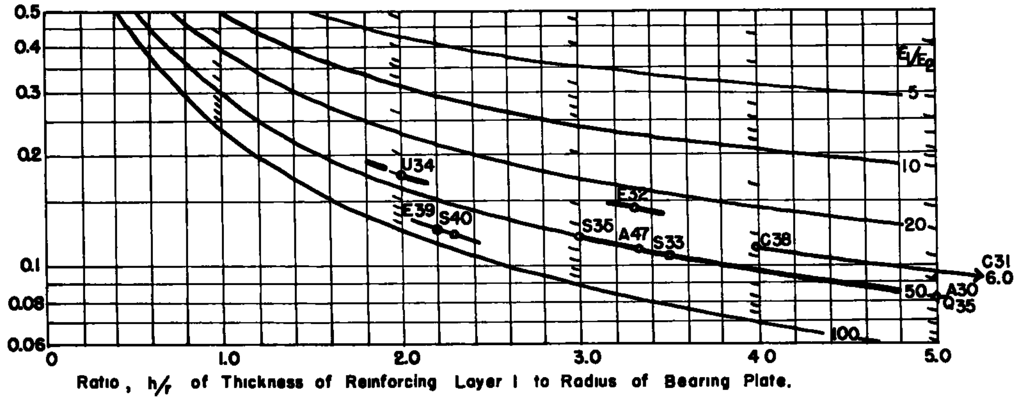
Item	p, psi	Plate Dia., in.	Sect. L	Sect. M	Sect. R	Sect. S	Sect. O
Thickness, h_{sb}	-	-	16	12	12	8	4
	-	12	15	$11\frac{1}{2}$	$9\frac{1}{2}$	8	4
	-	18	16	$11\frac{1}{2}$	10	8	4
(a) 5th Cycle							
Modulus, E_{sb}	-	12	22,000	12,000	12,000	-	26,000
	-	14	-	-	-	-	14,000
	-	18	22,000	12,000	12,000	20,000	9,000
E_{sc}	-	18	8,800	7,000	5,800	6,000	5,800
w_{1-2}/w_o, %	70.4	12	42	75	33	-	92
	62.0	18	64	78	75	87	84
(b) 18th Cycle							
Modulus, E_{sb}	-	12	24,000	15,000	14,000	-	29,000
	-	14	-	-	-	-	16,000
	-	18	24,000	15,000	14,000	22,000	11,000
w_{1-2}/w_o, %	70.4	12	29	78	22	-	91
	62.0	18	69	80	93	91	88

It is evident that the effectiveness of layered system action and the protection of the compacted subgrade layer provided by the compacted subbase layer, as indicated by these percentages, are influenced by the strength properties of the respective layers of the system, by the thickness of the subbase, and also markedly by the plate size, being less effective for the larger plate size. There is, in general, some increase in effectiveness of layered system action by consolidation influences from the 5th to the 18th cycle, accompanied by some strengthening of the subbase, as a component part of the layered system; but there are also some exceptions, which are significant of weakening influences.

Asphaltic Concrete Layer

A pavement layer composed of 4-in. asphaltic concrete and 2-in. gravel base, and of 2-in. asphaltic concrete and 4-in. gravel base, was constructed in 2-in. lifts on the compacted subbase layer to specified quality and density, as shown in Figure 7, for the loops of the test road. The crushed gravel for the base and asphaltic concrete was a well-graded material with a maximum size of 3/4 in. The bituminous surfacing consisted of hot plant-mixed asphaltic concrete of 120 to 150 penetration asphalt cement. The total combined pavement subbase thicknesses in the different sections of each loop varied systematically from 6, 10, 14, 18, to 22 in. The actual thicknesses in some cases were somewhat less than these values.

Eleven plate bearing tests were evaluated (Fig. 10) for the 4-in.-AC-2-in.-B pavement in Sections A, C, E, S, Q, and U. Eleven plate bearing tests also were evaluated (Fig. 11) for the 2-in.-AC-4-in.-B pavement in Sections F, H, K, L, N, and P. The approximate equivalent thickness method of evaluation was used, as developed for the subbase evaluations. In these plate bearing tests the pressure was increased in four steps of 5 cycles each. The 12th loading cycle (2nd cycle of the third pressure application) was used in these evaluations, as being approximately representative of the probable actual stressing in service of the pavement layer. In order to be comparable for 12-in. and 18-in. plates the third pressure loading had to be used (116.6 and 51.9 psi, respectively), the first being some-

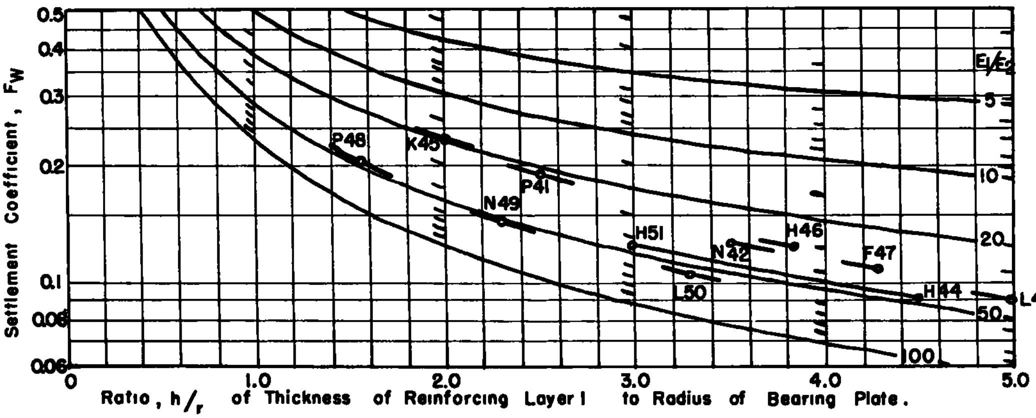


1 Section	2 Test	3 Station	4 Plate Dia	5 Pressure p psi	6 w (in) Cycle 12	7 Subbase h_{ss} in	8 Equiv h' in.	9 E_p Pave psi	1 Section	2 Test	3 Station	4 Plate Dia	5 Pressure p psi	6 w (in) Cycle 12	7 Subbase h_{ss} in	8 Equiv h' in	9 E_p Pave psi
C	31	153 + 40	12	116 6	0 032	8	36	70,000	A	30	145 + 25	12	116 6	0.032	16	30	100,000
	38	153 + 30	18	51 9	0 031	8	36	70,000		37	145 + 30	18	51 9	0 032	16	30	100,000
S	33	111 + 71	12	116.6	0 043	8	21	100,000	E	32	158 + 70	12	116 6	0 059	0	20	60,000
	35	111 + 57	14	85 6	0 042	8	21	100,000		39	158 + 60	18	51 9	0 034	0	20	160,000
	40	111 + 64	18	51 9	0 033	8	21	160,000	U	34	106 + 30	14	85 6	0 061	0	14	80,000
Q	36	119 + 70	14	85 6	0 028	16	30	100,000									

Range	Low	Average	High
Cycle 12	60,000	100,000	160,000

Figure 10. Evaluation of strength properties of 4-in.-AC + 2-in.-B pavement system; equivalent thickness. Influence curves of settlement coefficient, F_W of two-layer system; Burmister problem. Basic load-settlement equation:

$$w = 1.18\ p\ r\ F_W/E_S \qquad\qquad F_W = \frac{w\ E_S}{1.18\ p\ r}$$



1 Section	2 Test	3 Station	4 Plate Dia	5 Pressure p psi	6 w (in.) Cycle 12	7 Subbase h_{ss} in	8 Equiv h' in	9 E_p Pave psi	1 Section	2 Test	3 Station	4 Plate Dia	5 Pressure p psi	6 w (in.) Cycle 12	7 Subbase h_{ss} in	8 Equiv h' in	9 E_p Pave psi
H	44	150 + 71	12	116 6	0 037	8	27	90,000	P	41	119 + 60	12	116 6	0 080	0	14	50,000
	46	150 + 57	14	85 6	0 045	8	27	60,000		48	119 + 70	18	51 9	0 057	0	14	90,000
	51	150 + 64	18	51 9	0 035	8	27	90,000	L	43	106 + 20	12	116 6	0 037	16	30	80,000
N	42	114 + 30	12	116 6	0 053	8	21	70,000		50	106 + 40	18	51 9	0 029	16	30	110,000
	49	114 + 40	18	51 9	0 039	8	21	100,000	F	47	158 + 60	14	85 6	0 038	16	30	70,000
K	45	145 + 40	14	85.6	0 084	0	14	40,000									

Range	Low	Average	High
Cycle 12	40,000	78,000	110,000

Figure 11. Evaluation of strength properties of 2-in.-AC + 4-in.-B pavement system; equivalent thickness. Influence curves of settlement coefficient, F_W of two-layer system; Burmister problem. Basic load-settlement equation:

$$w = 1.18\ p\ r\ F_W/E_s \qquad F_W = \frac{w\ E_s}{1.18\ p\ r} \qquad E_s = 2{,}000 \text{ psi}$$

what too high and the second somewhat too low. The essential data in each case are given in the tabulations of Figures 10 and 11.

The site locations for two or more tests in a section were fairly close together, with an average of 9 ft, a low of 5 ft, and a high of 14 to 20 ft apart, as indicated in Figures 10 and 11. The data for Sections A, C, and S for the 4-in.-AC-2-in.-B pavement were found to be comparable, as shown by the conformity of the evaluation curves with the pattern influence curves. The average and range of the pavement modulus, E_p, for Sections A, C, E, S, Q, and U are as follows:

Cycle	Low	Average	High
12th	60,000	100,000	160,000

The average and upper limit are considered to be exceptionally good values, but the spread between the low and the high values is quite large.

There is not evident a significant casual relation of the low and high values with regard to the subgrade modulus, E_s, the subbase modulus, E_{sb}, and the thickness of the subbase. This is believed to be the result of the more predominating influences of the non-comparable pressures used in the third pressure application (namely, the 116.6-psi high pressure on the 12-in. plate and the 51.9-psi low pressure on the 18-in. plate), which tended to mask the foregoing more usual predominating influences of moduli and subbase thickness. It is probable, however, that the compacting of the pavement in the case of the high values of E_p was more effective in stress conditioning and strengthening the pavement layer and the pavement system as a whole; whereas there seems to be evident a deficiency of compaction and of stress conditioning and strengthening of the pavement system in the case of the below-average values. This poorer performance should be evident in the pavement condition under the traffic testing.

With only two comparable tests for each section and conforming evaluation curve, it was not possible by layered system trial methods to solve for the effective compacted subgrade modulus. Evaluations of layered system action and effectiveness from other data have practically always indicated a substantial increase in the subgrade modulus after construction of a base course and a pavement layer, due principally to the confining influences of these surmounting layers, and also to some additional consolidation and stress conditioning of the pavement system.

The data for the 2-in.-AC-4-in.-B pavement were found to be more erratic, due to influences of the non-comparable pressures on the 12- and 18-in. plates. Only data from Section H yielded a conformable evaluation curve. The remaining evaluations were made as point evaluations, on the basis of probably similar equivalent thickness conditions as the best approximation. The average and range of pavement modulus, E_p, for Sections F, H, K, L, N, and P are as follows:

Cycle	Low	Average	High
12th	40,000	78,000	110,000

These values are appreciably lower as a whole, indicating more inferior deflection responses and effectiveness of this lighter pavement system. The spread between the low and high values is about the same. The influences of the non-comparable pressures on the 12- and 18-in. plates seem to be more important, and may account for some of the erratic responses.

The effectiveness of the layered system action on the deflection responses for the different pavement systems can be judged and evaluated on the basis of the deflection at the first interface (1-2) between the pavement layer and the subbase layer, and at the second interface (2-3), between the subbase layer and the compacted subgrade layer, as a percentage of the deflection at the surface of the pavement system. This information is given in Table 6 for comparative purposes in judging effectiveness.

Table 6 shows that there is a consistent relation between effectiveness, as measured by the percentage deflection at the subgrade level. The pavement system with a 16-in. subbase thickness exhibits subgrade deflections of the order of only 30 percent of the surface deflections, thus providing good protection to the subgrade with effective layered system action for both types of asphalt pavements. If the pavement performance for the 2-in.-AC-4-in.-B pavement was poorer, it could be due to a breakdown in the asphalt pavement layer itself, which is thinner. The order of magnitude of the subgrade deflections, which are given in parentheses, is only 0.01 in. There is a significant increase in the percentage deflections at the subgrade level for the 8-in. subbase pavement systems, which indicates its less effective layered system action, which should be reflected in performance. The sections where there was no thickness of subbase exhibit very high percentages and large subgrade deflections.

TABLE 6

EFFECTIVENESS OF LAYERED SYSTEM ACTION ON DEFLECTION RESPONSES. INFLUENCES OF STRENGTH PROPERTIES OF THE COMPONENT LAYERS, SUBBASE THICKNESS, AND PAVEMENT THICKNESSES. DEFLECTIONS AT THE FIRST AND SECOND INTERFACES, AS A PERCENTAGE OF SURFACE DEFLECTIONS. THIRD PRESSURE APPLICATION. 12TH LOADING CYCLE.

4-in.-AC-2-in.-B

Section		A	Q	C	S	E	U
Thickness, h_{sb}		16	16	8	8	0	0
Modulus	Plate						
E_p	12	100,000		70,000	100,000	60,000	
	14		100,000		100,000		90,000
	18	100,000		70,000	160,000	160,000	
E_{sc}		4,400		6,000	6,000	5,800	10,400
w_{1-2}/w_o %							
p,psi	Plate						
116.6	12	81 %		97	70		
85.6	14		58		93		96
51.9	18	72		64	64		
w_{2-3}/w_o %							
116.6	12	32(.010)*		79(.025)	49(.021)	78(.046)	
85.6	14				45(.019)		77(.047)
51.9	18	32(.010)		35(.011)	45(.015)	62(.021)	

2-in.-AC-4-in.-B

Section		F	L	H	N	K	P
Thickness, h_{sb}		16	16	8	8	0	0
Modulus	Plate						
E_p	12		80,000	90,000	70,000		50,000
	14	70,000		60,000		40,000	
	18		110,000	90,000	100,000		70,000
E_{sb}	12		22,000				
	14						
	18		22,000				
E_{sc}			8,800	5,900		5,900	9,000
w_{1-2}/w_o %							
p,psi	Plate						
116.6	12		65	76	75		-
85.6	14	55 %		64		-	
51.9	18		69	55	74		-
w_{2-3}/w_o %							
p,psi	Plate						
116.6	12		27(.010)	29(.014)	58(.032)		84(.067)
85.6	14	32(.011)		49(.022)		88(.074)	
51.9	18		31(.009)	52(.032)	64(.024)		81(.047)

* Note: The figures in parentheses are the deflections in inches at the top of the compacted subgrade.

It is evident from Table 6 and from Figures 10 and 11 that the strengt properties of the 4-in.-AC-2-in.-B pavement systems are more consistently higher, with more comparable data and less erratic evaluations, than in the case of the 2-in.-AC-4-in.-B pavement systems. This should be significantly reflected in the deflection responses and performances under traffic testing and service conditions. In addition, these evaluations have indicated that a higher standard of constructional excellence was more consistently attained for the 4-in.-AC-2-in.-B pavement systems.

CONCLUSIONS

1. These evaluations show that the essential strength properties of the component layers can be significantly evaluated, as measures of the effectiveness and constructional excellence of pavement systems.

2. Evaluations at the subgrade level, the subbase level, and the finished pavement level, of the strength properties and deflection responses during construction of pavement systems should provide significant and essential criteria for effective control of construction in order to attain the desired degree of constructional excellence.

3. Having the strength properties and deflection responses of different pavement systems over the country, where climatic and soil conditions are significantly different, it should be possible to develop rational and adequate bases for evaluating the effectiveness of layered pavement systems and for design and construction control.

4. The evaluations in this paper show the necessity for more careful planning and execution, not only of investigations for layered pavement systems, but also of construction control in order to insure the attainment of constructional excellence.

5. Furthermore, these evaluations give point to the uses of superior quality materials, as well as to adjustments of component layer thicknesses for increasing the effectiveness, performance, and life of layered pavement systems.

6. Finally, the mechanics of layered system action and the effectiveness of its reinforcing and load-spreading capacity on the subgrade layer, as a structural unit, are intimately dependent on the interface continuity conditions and the mobilizable shearing strength incorporated into the layered system by compaction and stress conditioning influences, particularly on the weaker side of an interface. These aspects should be thoroughly investigated and evaluated.

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