Design of Pavements Using Deflection Equations From AASHO Road Test Results

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•THE OBJECT of this study is to investigate the structural performance of some satellite pavements in the Piedmont region of Virginia and to evaluate the thickness equivalency values (i.e., the ratio of the strength of material to the strength of asphaltic concrete) of the materials used on the basis of AASHO Road Test results. It is also proposed to recommend a tentative design procedure based on pavement rigidity.

The Benkelman beam rebound deflection method is a reliable and simple means of evaluating the structural performance of pavements and has been adopted in this investigation. The AASHO Road Test Committee has suggested model equations for designing pavements on the basis of pavement performance. These equations involve the following variables:

1. Thickness of each layer in the pavement,
2. Thickness equivalency of the material in each layer,
3. Subgrade strength,
4. Traffic,
5. Age, and
6. Climatic conditions.

The values of these variables were determined in the AASHO Road Tests but these values could not be applied to Virginia because of differences in (a) construction techniques, (b) type of materials used, (c) subgrade properties, (d) environmental conditions, (e) type and duration of traffic, and (f) age of pavement.

Twenty projects with varying pavement structures, all in the Piedmont region of Virginia, were chosen for this satellite study. All these projects are on primary or interstate roads. One is an experimental section on Route 360 with four different structural designs; another was an experimental project on Route 58 with four different designs. This last project was resurfaced in 1962 and only the data collected on it prior to 1962 have been evaluated. Thus a detailed study of 27 different sections—without any resurfacing or heavy maintenance—was undertaken. In addition, 16 other projects were considered.

The main purpose of this investigation was to conduct a pilot study for evaluating the thickness equivalency values of the different materials in the pavement system and to correlate these values with the pavement performance along with other variables such as soil support, traffic, and age. Since the study was within a limited geographic area, the climatic and regional factors were considered constant, and hence the unweighted traffic—i.e., actual traffic not corrected for climatic conditions—was considered in the analysis.

In order to introduce the soil support value in the correlation, a very preliminary study was proposed. After the study was initiated, additional details, such as tolerable rebound deflections and tentative designs for Virginia, were also investigated.

VARIABLES AND THEIR DETERMINATIONS

The variables required for this analysis are divided into two categories—dependent and independent. Only one dependent or performance variable has been considered in this investigation; as already explained, it is the Benkelman beam rebound deflection, d.

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The independent variables are the thickness of each layer in the pavement, h; the thickness equivalency value of the material in each layer of the pavement; subgrade strength; traffic; and age. As mentioned, the climatic conditions have been treated as a constant.

Determination of these variables could be divided into two parts: collection of data from the field or from past records, and evaluation of the data so collected. Thickness equivalency values, unlike other variables, are not directly determinable from the data, and their evaluation is discussed later.

Collection of Data

The type of data and the method of collection are described in the following.

Deflection—Benkelman beam rebound deflections for slow-moving vehicles were taken under 18-kip axle loads during the spring and fall of 1966 on the 20 satellite projects. The spring deflections were taken three times at the same place at 20-day intervals. The method of taking the deflections is given in Appendix A.

Layered Thickness—The thickness of each layer in the pavements was determined from construction records.

Subgrade Support—The Virginia design CBR values for the subgrade soil used by the Materials Division were adopted for this investigation. In order to correlate the subgrade support with AASHO Road Test results, Virginia CBR tests were carried out on the subgrade, subbase and base materials used for the AASHO Road Tests.

Subgrade soil samples were taken at the edges of the pavements because the moisture content at these locations would give some idea of the moisture content of the soil underneath the pavement; 142 soil samples were taken and their moisture contents determined, and 41 were tested for plastic limits.

Traffic—The data on the type and amount of each type of vehicle for each satellite project were available. Load surveys for sites with similar traffic were utilized to determine the 18-kip equivalents.

Age—The age of the pavement was calculated from the date of construction.

Evaluation of Data

The data collected and described in the foregoing were evaluated according to the following.

Deflection—In the past, Benkelman beam rebound deflections were generally taken anytime during the spring, i.e., between April 1 and June 30. It was therefore questionable whether these deflections could validly be considered as having been taken during the spring thaw period.

In this investigation, three deflection readings were taken on all satellite projects at about 20-day intervals commencing April 1. These results showed that the variations in the readings at the same place were not necessarily due to climatic conditions but may have been due to the testing conditions. In fact most of the projects showed very little difference between the three readings. Standard deviations were determined for each set of readings for each project, and the mean values of deflections as well as standard deviations are given in Appendix A. The method of evaluating curvature and the cross-sectional area of the deflection basin is also described in Appendix A. On the basis of these results, deflection data previously obtained on these projects and available on 16 additional projects from the Piedmont area were studied.

The spring deflection data showed that a straight-line relationship existed between (a) maximum deflection and curvature, and (b) maximum deflection and longitudinal cross-sectional area of the deflected basin. Deflection data taken in the Piedmont area prior to this investigation also showed the same kinship between deflection and curvature. This relationship is shown in Figure 1. The correlation coefficient is 0.97.

Since stresses are a function of curvature and since curvature has been shown to correlate well with the magnitude of maximum deflection, the stresses in asphaltic concrete pavements can be considered as being proportional to the magnitude of deflections, and the evaluation of relationships between the independent and dependent variables on

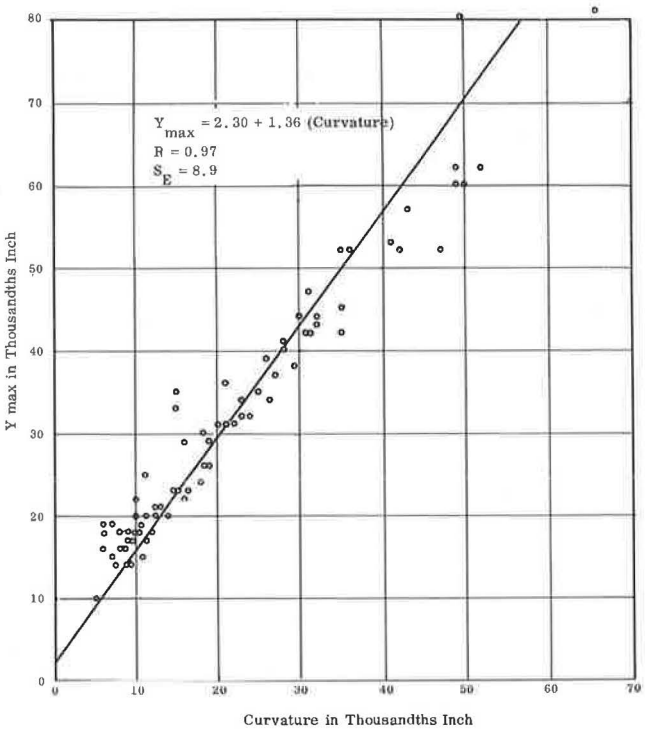

Figure 1. Correlation between maximum deflection and curvature.

the basis of maximum deflection will be as good as that made on the basis of curvature or the longitudinal cross-sectional area of the deflected basin.

Fall deflection data were also gathered and it was found that these data varied 60 to 80 percent from those obtained in the spring. Furthermore, there was no statistical relationship between the maximum deflection and the curvature using fall deflections. A curvilinear relationship did exist between the maximum deflection and the longitudinal cross-sectional area of the deflected basin.

Since a poor relationship existed between spring and fall deflections, and since spring deflections show higher deflections and hence the worst condition of pavements, the analysis of deflection in this investigation is based on spring deflection data only.

Subgrade Support—The subgrade support value depends on two important factors: (a) resistance to a single applied load, and (b) rebound due to soil resiliency. The value of the resistance due to a single applied load is determined by various methods—in Virginia the CBR method is used. Resiliency is the property that causes material to rebound after the load has been removed. Due to rebound, fatigue often results in failure of the pavement. The subgrade soils in the Piedmont area are silty, contain mica and are generally highly resilient. Resiliency is not presently considered quantitatively in the design of pavements; however, in such soils, soil-stabilized subgrades are usually provided to reduce deflections and hence the detrimental effect of resiliency combined with high deflections.

In this investigation, three methods of determining the subgrade support value were tried: (a) CBR test method, as adopted in Virginia, (b) design CBR-resiliency method, and (c) design CBR-physiographic method.

Subgrade Support Value Based on CBR Test—The AASHO Design Chart for flexible pavements (1) is based on subgrade support values. Many states are trying to correlate their CBR or other soil strength values (obtained by the method adopted by them) with these subgrade support values. Their correlation is based primarily on the soil strength tests carried out by them on AASHO Road Test materials.

The Virginia CBR values determined on AASHO Road Test materials were reported in 1961 by Shook and Fang (2). These values showed that the CBR of the subbase material was higher than that of the base material, the values being 95 and 53, respectively. To verify these illogical results, more AASHO Road Test materials were tested. This second test did indeed verify that the CBR of subbase material was higher than that of the base material; the values were about 140 and 40, respectively. No reason for the reverse order of these values is known. The highlights of the Virginia CBR method are given by Shook and Fang (2).

It was therefore not possible to correlate Virginia CBR values with the subgrade support values given in the AASHO Design Chart.

Subgrade Support Value Based on Design CBR-Resiliency—From prior knowledge, resiliency was thought to be a very significant factor in determining subgrade support. It was therefore thought that resiliency combined with the CBR value might give a better evaluation of the soil-subgrade support. During this preliminary stage of investigation, the property of resiliency was divided into three classifications—low, medium, and high —and these were assigned values of 1.5, 1.0, and 0.5, respectively. Thus a soil with low resiliency with a design CBR of 4 (i.e., CBR-resiliency value = 1.5 × 4 = 6) is considered to have the same subgrade support value as a soil with high resiliency with a design CBR of 12 (i.e., CBR-resiliency value = 0.5 × 12 = 6).

Subgrade Support Value Based on Design CBR-Physiographic Regional Factor—Virginia has been divided into 12 physiographic regions based on pavement performance (3). These regions have been classified according to the support properties of the subgrade materials found. In this investigation, these properties were divided into five categories, with 1.5 being the best quality soil and 0.5 the poorest. Subgrade soil support values based on this method were obtained by multiplying the design CBR by the physiographic regional factor in a manner similar to that mentioned earlier for CBR-resiliency.

A stepwise regression analysis was carried out to correlate the variables and evaluate the thickness equivalency values, as explained later. In this analysis the subgrade support values obtained by each of the two methods described were introduced separately to determine which of the values gave the better correlation. It was found that the subgrade support values obtained by either method gave better correlation between variables than without them. However, values based on design CBR-resiliency gave better correlation than the values based on design CBR-physiographic regional factor. The former values have, therefore, been used for further analysis.

Moisture Content of the Subgrade Soil—The study of the moisture content of the subgrade soils of the satellite projects showed that in most cases the moisture content was higher than that of the soaked CBR samples during the time of construction.

As mentioned, 41 of the soil samples were tested for plastic limits. About 40 percent of these soils were non-plastic and most of the rest had field moisture contents less than 20 percent of their plastic limits. In the case of pavements with cement-treated subgrade, when the subgrade moisture content was within 10 percent of the plastic limit (two cases only), no increase in pavement deflection from the usual pattern was noted. In the case of one pavement (no cement-treated subgrade) with the subgrade moisture content near the plastic limit, a slight increase in pavement deflection from the usual pattern was noted. From the above it seems possible that in cases of pavements with soil-stabilized subgrade, the increase in subgrade moisture content does not affect the deflection values, while with an increase in subgrade moisture content with non-stabilized subgrades there is a slight increase in deflections when the moisture content approaches the plastic limit of the soil.

EVALUATION OF THICKNESS EQUIVALENCY VALUES

In the previous paragraphs the methods and the determination of all the variables have been discussed with the exception of thickness equivalency values. In this section, the method and the determination of thickness equivalency values are presented. Further, a new conception of tolerable rebound deflection based on pavement rigidity is presented.

To determine the thickness equivalency values, it is essential to have a basic idea of the behavior of pavements, particularly with respect to the region for which they are to be designed. This is discussed in the following paragraphs.

Behavior of Pavements

In the past, pavements were designed against plastic and shear failure. An example of this is shown in Figure 2 for the experimental project on Route 58. This figure shows that the deflections increased during the third to the fifth year. This indicates that the pavements were poor in design, and that after failure the deflections increased until

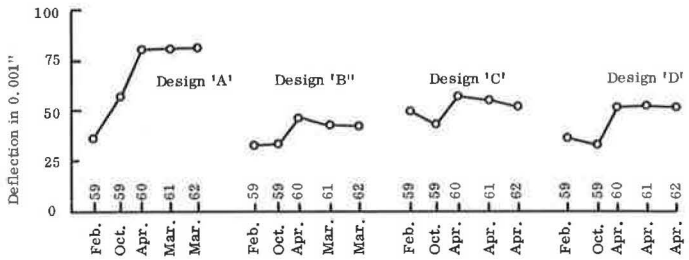

Figure 2. Examples of pavement deflections when failure is by shear (experimental project on Route 58—cement-treated subgrade).

the pavements achieved stability, and then the deflections again became uniform. During the period of increased deflections the pavement became cracked and rutted.

Study of the rebound deflection data of the satellite projects shows that for a certain period immediately after construction the rebound deflections decrease with an increase in the total traffic, indicating a consolidation phase. The duration of this consolidation phase varies with the pavement rigidity from 3 to 18 months; the more rigid the pavement, the shorter the duration of this phase. A typical example of this phase is shown in Figure 3 for four designs on Route 360. On this project, designs B and C are provided with soil-stabilized base and hence are more rigid than designs A and D. In designs A and D the consolidation phase is clearly indicated by the falling rate of deflection from December 1962 to March 1963, while in designs B and C the consolidation phase is absent.

A recent survey (4) of flexible pavements in Virginia has shown that the distress of pavements due to rutting is generally uncommon. This study showed that the deterioration is mostly due to other causes, such as extensive longitudinal and pattern cracking in the wheelpath. These cracks are usually seen after a period of about 5 to 8 years and it is felt that they are the result of fatigue failure of an elastic pavement.

Thus the pavements could be considered to pass through three different phases—the consolidation phase, the elastic phase, and failure due to fatigue. During the consolidation phase, the materials in the layered system of the pavements and the subgrade soil consolidate and become more dense and, as this process continues, the pavement deflections decrease. During the elastic phase the consolidation is almost negligible and the materials, including the subgrade, behave more or less elastically unless the axle weight increases, in which case further consolidation occurs. During this phase, the pavement does not seem to crack, and the deflections remain more or less constant. In Virginia this phase lasts for a period of about 5 years.

The elastic phase slowly creeps into the third phase, i.e., fatigue. Thin longitudinal cracks develop and later widen and are then followed by pattern cracks, all of which usually take place in the wheelpaths. The pavements are resurfaced after the third phase.

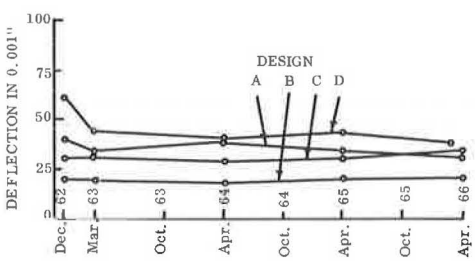

Figure 3. Examples of pavement deflection during elastic phase (experimental project on Route 360—cement-treated subgrade).

Method Recommended for Design

In view of the general behavior of pavements, the pavements could be designed on the basis of elastic theory. The AASHO Committee (5) has recommended a model equation of the form, $\log d = a_1h_1 + a_2h_2 + a_3h_3 + \ldots$ where a_1, a_2, a_3, etc., are the strength coefficients of the materials in the layers having thickness equal to h_1, h_2, h_3, etc., respectively.

Burmister's elastic theory (6) shows that the deflection of a pavement is not merely a summation of the strength of the individual

layers, but a ratio of the strengths of adjoining layers. Thus deflection is not merely a function of a_1h_1 (as indicated in the model equation given by AASHO) but also of

$$\frac{a_1h_1}{a_2h_2 + a_3h_3 + \ldots}$$

Thus, for elastic design, if the ratio of the strengths of adjoining layers remains more or less constant, the AASHO Committee's model equation would be applicable. It was, therefore, necessary to determine whether the pavements in Virginia satisfy this criterion. The layered system and the range of values for the pavements studied are given below.

The asphalt concrete mat is about 5 to 10 in. thick, consisting of about 3 to 7½ in. of asphalt base (Marshall stability less than 300 lb), overlaid by about 2 to 2½ in. of bituminous binder and surface course (Marshall stability of about 700 to 900 lb). This asphalt mat is underlaid by a stone base of about 6 to 9 in., which may or may not be cement-treated. A cement-treated subgrade or select material, if provided, is sandwiched between stone base and subgrade soil. The thickness of the cement-treated subgrade is about 6 to 8 in.

The description of the pavement structure in Virginia shows that the ratios of the strengths of the adjoining layers, though variable, remain more or less constant to satisfy Burmister's elastic theory requirements. It is therefore believed that the AASHO Committee's method of design might hold good. The thickness equivalency values determined from the study of such pavements will be applicable only to pavements with layered systems and with materials of the type used and placed in the order described.

Method Adopted for Determining Thickness Equivalency Values

As already mentioned, the thickness equivalency of a material is the ratio of the strength coefficient of the material to the strength coefficient of asphaltic concrete. The strength coefficients of the materials in each layer have been taken as follows:

- a_1 = strength coefficient of asphalt mat; the asphalt mat may consist of surface, binder, and base layers of asphaltic concrete.
- a_2 = strength coefficient of stone base.
- a_{21} = coefficient for additional strength due to cement stabilization of stone base; thus, cement-stabilized stone base has a strength coefficient equal to $(a_2 + a_{21})$.
- a_{22} = coefficient for additional strength due to bitumen-stabilized stone base; thus, bitumen-stabilized stone base has a strength coefficient equal to $(a_2 + a_{22})$.
- a_3 = strength coefficient of select material.
- a_4 = strength coefficient of soil-stabilized layer in the subgrade.
- a_r = strength coefficient of subgrade when graded according to design CBR-resiliency factor.

A stepwise regression analysis was carried out with a computer to determine the thickness equivalency values of the materials by means of the equation

$$\log d = a_0 + a_1\left(h_1 + \frac{a_2}{a_1}h_2 + \frac{a_3}{a_1}h_3 + \ldots + \frac{a_S}{a_1}G_S\right)$$

where $\frac{a_S}{a_1}$ = thickness equivalency value of the subgrade and G_S the equivalency grading of the subgrade soil support. The object of this analysis was to determine the thickness equivalency values of the materials and the effect of each variable on these values.

About 70 combinations of different variables have been tried; the different groups used are given in Appendix B.

TABLE 1
THICKNESS EQUIVALENCY VALUES OF MATERIALS

Material	Thickness Equivalency As Determined	Thickness Equivalency As Given by AASHO
Asphalt	1.0	1.0
Stone base	0.35	0.31
Cement-treated stone in base	1.1	0.52
Asphalt-treated stone in base (lean-mix)	0.75	0.77
Select material in subbase	0.0	0.25
Cement-stabilized subgrade	0.5	not given

The same equation was tried with log curvature and log $(d + 2\sigma)$ instead of log d on the left-hand side. The results showed almost the same trend as with deflections and hence are not reproduced here.

The stepwise regression analysis clearly pointed out two broad aspects of design:

1. Within each group, a change in the number of variables did not greatly affect the thickness equivalency values. However, with a change in groups, the thickness equivalency values of any given layer changed, sometimes to a great extent. This, therefore, shows that with the change in the arrangement and thickness of the layers, resulting in a change in ratio of the strengths of adjoining layers, these values will change.
2. The increase in the number of independent variables within each group gave higher values of the correlation coefficients and lower values of the standard error of estimate. This, therefore, shows that with an increase in the number of independent variables the analysis improves.

The thickness equivalency values obtained by regression analysis of the satellite projects are given in Table 1. The values given by the AASHO Committee are also given in this table for comparison. These obtained values apply only to those designs which are similar to those of the satellite pavements, and should be considered tentative. These values are discussed in the following paragraphs.

Thickness Equivalency Value of Asphaltic Concrete, $\frac{a_1}{a_1}$ —The thickness equivalency value of this material is usually higher than that of any other material, and is taken as 1.

Thickness Equivalency Value for Stone Base, $\frac{a_2}{a_1}$ —The value of the thickness equivalency of this material varies from 0.35 to 0.40 for all satellite projects. The Asphalt Institute (7) and the CGRA (8) have recommended a value of 0.31. On the basis of this investigation a thickness equivalency value of 0.35 appears appropriate.

Thickness Equivalency Value of Cement-Treated Stone Base, $\frac{a_2 + a_{21}}{a_1}$ —The additional thickness equivalency value of stone base due to stabilization with cement was found to vary from 0.5 to 1.25. If we assume the thickness equivalency of stone base as equal to 0.35, the thickness equivalency of cement-stabilized base would vary from 0.85 to 1.55. A tentative value of 1.10 is assumed. It may be mentioned that only two of the 27 projects had cement-treated bases.

Thickness Equivalency Value of Asphalt-Treated Stone Base, $\frac{a_2 + a_{22}}{a_1}$ —None of the 27 projects in the satellite study had asphalt-treated (lean mix) stone base. Of the 43 projects in the Piedmont area only two had this base. Analysis of these projects gave additional thickness equivalency values varying from 0.25 to 0.50. If we assume the thickness equivalency of stone base as 0.35, the thickness equivalency of asphalt-stabilized stone base would vary from 0.60 to 0.85. A tentative value of 0.75 is recommended.

Thickness Equivalency Value for Select Material, $\frac{a_3}{a_1}$ —Analysis of all groups of projects, except the group with no stabilized subgrade, showed that the value of a_3 was negative and tended almost toward zero. The negative sign was probably due to an interaction in the regression analysis. In the case of the group with no stabilized subgrade the value of a_3, though positive, was very small. It could therefore be concluded that the select materials do not contribute appreciably to reducing deflections and the thickness equivalency value of this material could therefore be taken as zero.

This is no reason for not providing a subbase. It is felt that a subbase may be necessary for improving drainage and preventing the penetration of subgrade material into the base course, etc.; however for better structural performance of the pavements the subbase material, if provided, should be nonresilient.

Thickness Equivalency Value of Soil-Stabilized Subgrade, $\frac{a_4}{a_1}$ —The value of the thickness equivalency of this material varies from 0.42 to 0.49 for the satellite projects. This value varied greatly from group to group. For satellite and other projects, but including the experimental project on Route 360, this value was found to be above 1.0. The reason probably is the variation in strength of the cement-stabilized material. A value of 0.5 is considered suitable for design. The AASHO Road Test has not recommended any value for stabilized soil subgrade.

Thickness Equivalency Value of Subgrade Strength, $\frac{a_S}{a_1}$ —As already explained, the soil support value was evaluated in terms of design CBR-resiliency factor and design CBR-physiographic factor. As mentioned previously, regression analysis showed that the correlation was better with the variable using the CBR-resiliency value than with the CBR-physiographic value. Thus it shows that the design based on CBR-resiliency method would give better results than design based on CBR-physiographic method. In the 70 combinations of different variables tried the value of $\frac{a_S}{a_1} G_S$ was found to be 0.13.

Deflection as a Function of Thickness Index

The thickness equivalency values discussed in the preceding paragraphs were obtained from the equation

$$\log d = a_0 + a_1 \left(h_1 + \frac{a_2}{a_1} h_2 + \frac{a_3}{a_1} h_3 + \ldots + \frac{a_S}{a_1} G_S \right)$$

The expression in parentheses on the right-hand side of this equation without considering $\frac{a_S}{a_1} G_S$ —the subgrade support value—when multiplied by a_1 is known as the thickness index, D, and represents the strength of the pavement without considering the support value of the subgrade. With the thickness equivalency values tentatively recommended above, the correlation given in Table 2 was obtained for different groups of projects between thickness index, D, and log d, where d is deflection in thousandths of an inch.

In Table 2, the group of 19 satellite projects shows a better correlation. These projects have shown consistently good performance. Their ages vary from 5 to 12 years and none of them have been resurfaced. A design based on the results of these projects may, therefore, be acceptable for pavements similar to those studied. In this investigation, a new concept of design, as explained in the following section, is proposed.

THE CONCEPT OF TOLERABLE DEFLECTION AS A FUNCTION OF PAVEMENT RIGIDITY

Rigidity of Layers

When a load is applied, stresses are created in a pavement. These stresses will cause only bending in a perfectly rigid layer and only compression in a perfectly flexible layer. A semiflexible layer will partly bend and partly compress under the applied load. Further, the amount of bending will decrease as the rigidity (or modulus of elasticity) and thickness of the layer increase. Compression will increase as the rigidity (i.e., modulus of elasticity) decreases and thickness increases. In a pavement these properties of the layers could be evaluated by determining the deflections at the top and the bottom of each layer under a given applied load. If the deflections at the top and bottom of any layer are almost the same, the layers could be considered as rigid. A good example of this is a concrete slab.

TABLE 2

CORRELATION BETWEEN THICKNESS INDEX AND log d

Group of Projects	Standard Error Of Estimate = S_E	Correlation Coefficient = R	Equation
1. Satellite study projects, excluding the two experimental projects (number of projects = 19)	.087	-0.86	log d = 2.06 - 0.068 D
2. Satellite and other projects including the two experimental projects (number of projects = 43)	.110	-0.70	log d = 2.81 - 0.130 D

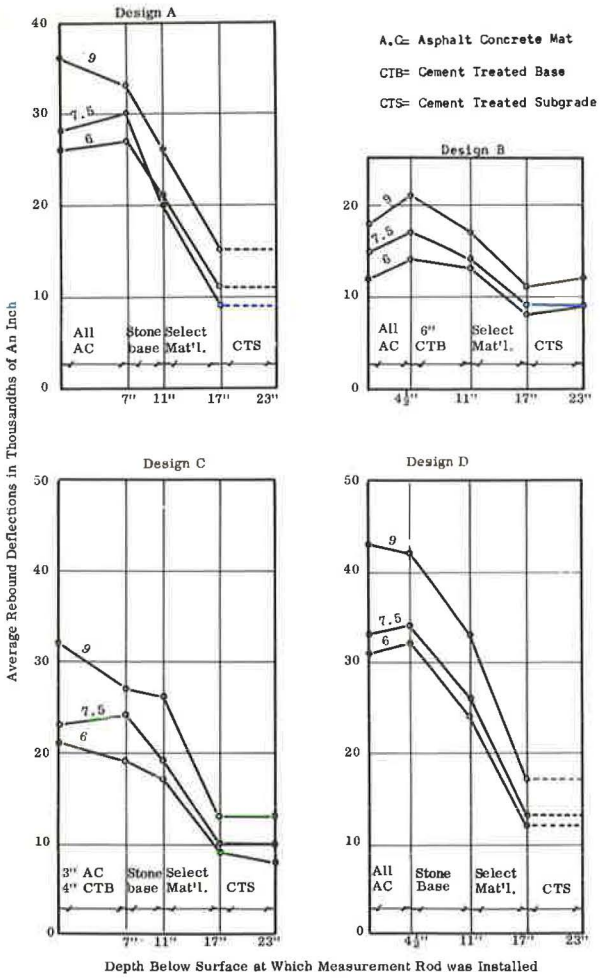

Figure 4. Average rebound deflections at surface and at various depths below surface corresponding to layer interfaces produced by indicated single wheel loads.

In the case of a perfectly flexible layer, the deflection at the bottom of a layer is zero with a certain amount of deflection at its top. A good example of this is the subgrade soil, which because of its infinite thickness absorbs all the deflection by compression.

A graphical interpretation of this hypothesis is shown in Figure 4, which gives the measured values of deflection of different layers of pavement on the Route 360 experimental project. In order to get some specific results from these four figures we have ignored the shortcomings of the deflection measuring tool; e.g., in some cases the deflection is more beneath the asphaltic concrete layer than on top of it.

These figures show that the moduli of elasticity and thicknesses of the asphaltic-concrete layer and cement-treated subgrade are such as to cause bending and very little compression in these layers. This is because the deflection of the top of the layer is almost equal to that at the bottom of the layer. It could therefore be assumed that layers with such materials are rigid, and hence the deflection is due to bending only. These figures also show that the stone base and select material layers have higher values of deflection at the top than at the bottom, indicating they both are under bending and compression. Thus, in design A, under a 9-kip load the deflection of the stone base is 0.033 in. at the top and 0.026 in. at the bottom, indicating that the deflection of 0.026 is due to bending and (0.033 - 0.026) = 0.007 is due to compression.

Since the pavements are being considered as elastic, the failure of the pavement would be by fatigue due to repetition of bending only. Thus the best approach to pavement design would also be on the basis of bending only, a function of rigidity. However, this is impracticable because it is not possible to separate the increase (or change) in deflection due to the presence of the flexible (compressible) layer from that due to bending. Hence in this preliminary investigation, the total deflection has been used for analysis.

Design Based on Tolerable Deflection vs Rigidity

Many investigators have recommended certain values of tolerable deflections for flexible pavements with and without soil-stabilized subgrades. Some recommended tolerable deflections are as follows: WASHO Committee (9)—0.030 in.; Nichols (10)—0.036 in.; Ruiz (11)—0.035 in.; Hveem (12) has qualitatively recommended lower maximum values of deflection with increases in the thickness of asphaltic-concrete or cement-stabilized base.

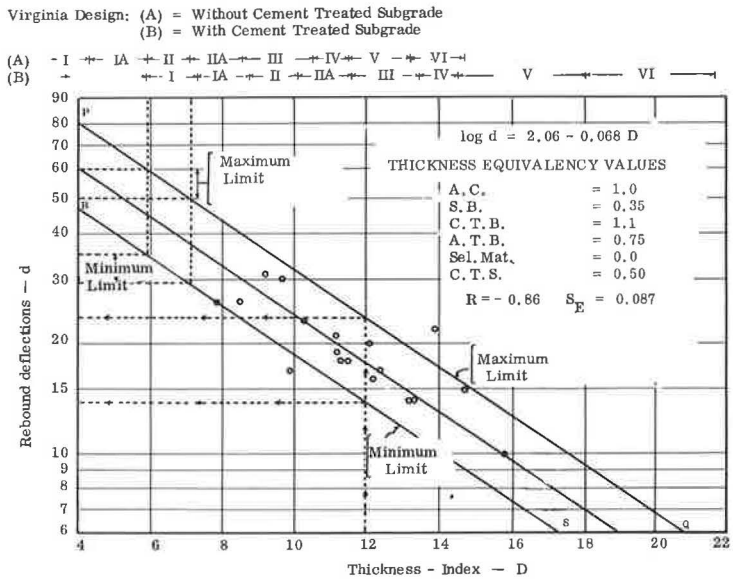

Figure 5. Minimum permissible and tolerable deflection limits.

Previously it has been shown that the pavement deflection is a straight-line function of the thickness index and in the case of 19 projects it could be represented by

$$\log d = 2.06 - 0.068\ D = 2.06 - 0.068\ a_1 \left(h_1 + \frac{a_2}{a_1} h_2 + \frac{a_3}{a_1} + \ldots \right)$$

having a correlation coefficient R = 0.86 and a standard error of estimate S_E = 0.087 on the log scale. This equation is based on the values of thickness equivalency of asphaltic concrete, stone base, cement-treated base, asphalt-treated base, select material, and soil-stabilized subgrade of 1.0, 0.35, 1.1, 0.75, 0.00, 0.50 respectively.

This relationship is shown in Figure 5. In this figure line PQ has been drawn for log d + 1.5 S_E (which includes 43 percent of the area under the normal curve) to indicate the maximum permissible limits—comparable to a tolerable deflection—of deflection for a given value of thickness index. If the deflection of a pavement exceeds this limit, the pavement is considered to be under-designed. The line RS in the figure has been drawn for log d - 1.5 S_E (which includes an additional 43 percent of the area under the normal curve) to indicate the minimum permissible limit of deflection for design. Thus a pavement is considered to have been over-designed if the deflection is much less than this limit.

To make this relationship adaptable for the pavement design categories based on traffic in Virginia, an additional scale based on traffic categories adopted in Virginia is also shown in Figure 5, in parts A and B. The design categories and the thickness indices with and without stabilized subgrade are given in Appendix C.

To illustrate the design based on these limits, let us assume a traffic category of II and a nonresilient subgrade soil with a high design CBR value that does not need a cement treatment. Figure 5 shows that under these conditions the maximum limit of deflection would be 0.050 to 0.060 in. and the minimum about 0.030 to 0.035 in. A design based on the average of these two values, i.e., 0.040 to 0.047 in., would be appropriate. Figure 5 shows that for these design deflections the pavement should have a thickness index of between 5.9 and 7.1. This is also shown in Appendix C. The pavement should therefore be designed for an average thickness index of 6.5 so that the

deflections should lie between the maximum and minimum permissible limits. Various choices in the structural cross section could be made:

	Choice A	Choice B	Choice C
Asphalt mat	5 in.	4.5 in.	4 in.
Stone base	4 in.	6 in.	8 in.
D =	6.4	6.5	6.8

The flexible pavement design chart (Appendix C) recommends 4 to 8 in. of subbase, 3 in. of asphalt base (Marshall stability less than 300 lb, say), and 1½ in. of surface course (Marshall stability of about 700 to 900 lb). Thus we find that design of new pavements on the basis of rigidity—with the help of Figure 5 or Appendix C—may help in choosing a more economical structural cross section. It is likely that in providing the structural section as in choice A above, the deflections would be less, i.e., be nearer to line RS, than would be the case with choice C. In case of choice C the deflections would be nearer to line PQ. The reason for this would be that asphalt mat is more rigid and less compressible while the stone base is not. Hence with an increase in thickness of the asphalt mat and a reduction in the thickness of stone base, the deflections for the same thickness index would decrease. A proper selection of design on the basis of rigidity of layers is therefore essential.

Since in Virginia the pavements have been considered to behave elastically after a short consolidation phase, increased rigidity of a layer would result in decreased deflections. Increased rigidity is also due to increased values of the thickness index caused by increased rigidity or thickness of rigid layers such as asphaltic-concrete or cement-stabilized base. This shows that permissible or tolerable deflection of the pavement is a misnomer, unless the rigidity of the pavement is known. Thus a pavement with high rigidity will completely fail at a value of deflection which may be well within the permissible value of deflection for a pavement with low rigidity.

In support of these conclusions, reference could be made to the work by Nijboer and Van de Poel (13), who have shown that deflection and stiffness could be correlated as follows:

$$\log d = a_0 - \log(\text{stiffness})$$

or

$$= a_0 - \text{rigidity}$$

or

$$= 2.06 - 0.068\ D \text{ in this investigation.}$$

Deflection is therefore a function of the thickness index of the pavement or a function of thickness equivalency values and vice versa. Since rigidity indicates resistance to deflection only, the thickness equivalency values would therefore represent the same.

Thus, given two layers of different materials but of the same thickness, the one with the higher value of thickness equivalency or rigidity will have a lower tolerable deflection under bending than the one with the lower value of thickness equivalency or rigidity. Furthermore, with two layers of different thickness but of the same materials, the one with the higher thickness will have a lower tolerable deflection under bending than the one with the lower thickness.

Further, when designing flexible pavements with flexible layers sandwiched between rigid layers, the increased deflection due to the flexible layers should be kept within permissible limits so as not to overstress the overlying rigid layers. This was clearly evident on the experimental projects on Route 360, which have higher deflections as compared to other projects with cement-treated subgrade and no select material. The reason for this is the introduction of a select material layer as shown in Figure 4. This figure shows that if select material was not provided the total deflection might have been less.

TENTATIVE DESIGN METHOD FOR VIRGINIA

The design recommended below is based entirely on the present design method used in Virginia. This method has resulted from experience and research in the state. It is based on the principle that when the subgrade support value—based on the CBR value—is low, soil stabilization is to be provided. In the method recommended for design, the soil support value is obtained from the CBR-resiliency method as discussed earlier.

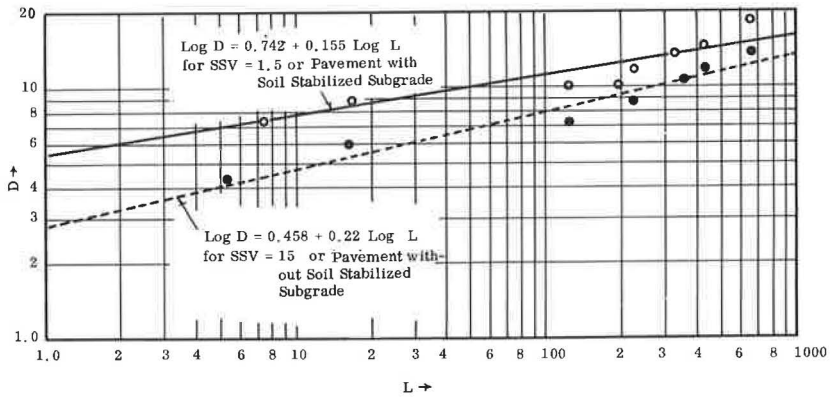

Figure 6. Correlation between thickness index and total traffic for pavements with and without soil-stabilized subgrades.

Thickness Index of Different Design Categories

Pavement design in Virginia is divided into eight categories, which are given in Appendix C. The thickness indices for each category based on the tentative thickness equivalency values determined in this investigation are also given in Appendix C. These thickness index values have been calculated for two classes of pavement, i.e., with and without soil-stabilized subgrades. For soils having low subgrade support values (i.e., say CBR-resiliency = 1.5) it is assumed that the soil-stabilized subgrade will be provided and hence the thickness index value for the design with a stabilized subgrade will be suitable. For soils with a high subgrade support value (i.e., CBR-resiliency = 15) it is assumed that a soil-stabilized subgrade will not be necessary and hence the thickness index value for the design without stabilized subgrade will be suitable.

A regression analysis between daily 18-kip equivalent loads and thickness index was carried out and the relationships obtained are as follows (L = equivalent daily 18-kip single axle loads):

1. For SSV = 1.5, or pavements with soil-stabilized subgrades,

$$\log D = 0.742 + 0.155 \log L$$

having $R = 0.95$ and $S_E = 0.12$.

2. For SSV = 15, or pavements without soil-stabilized subgrades,

$$\log D = 0.458 + 0.22 \log L$$

having $R = 0.98$ and $S_E = 0.047$.

The graph for these equations is shown in Figure 6. A nomograph based on these values is shown in Figure 7. Thus, in Figure 7 the range of soil support values varies from 0 to 15, with an approximate value of 1.5 for highly resilient soils in the Piedmont area of Virginia.

The SSV scale on this nomogram has been divided into three categories from the point of view of soil treatment. The author feels that the soils with an SSV value of less than 6 should be stabilized and the soils with SSV values more than 12 do not require stabilization. The soils with SSV values between 6 and 12 are left to the option of the designer. However, as a guide it is felt that in cases of highly resilient soils with design CBR-resiliency values = 0.5 the soil should be stabilized; in cases of a medium-resilient soil with design CBR-resiliency values = 1.0 the soil should be modified; and with a CBR-resiliency value of 1.5 no soil treatment should be provided.

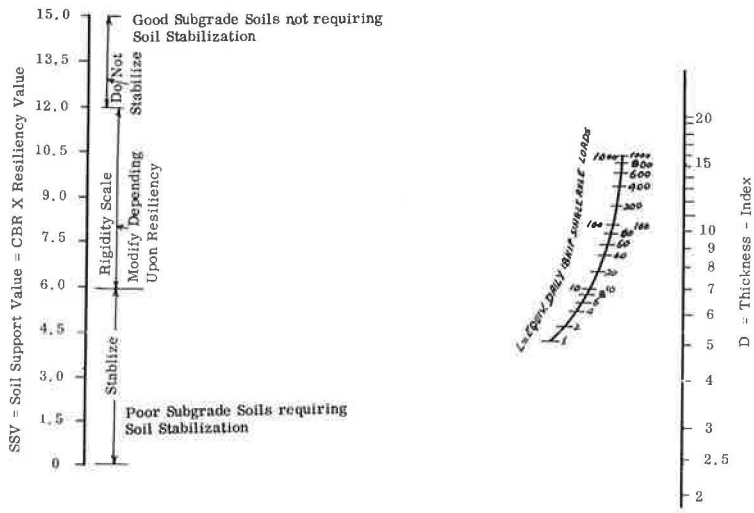

Figure 7. Nomograph for determining thickness index.

This nomograph can be utilized for determining the thickness index required for the design of new pavements, based on the subgrade soil properties and traffic in terms of daily 18-kip equivalents, or can be used for maintenance by evaluating the modified thickness for the revised traffic in terms of daily 18-kip equivalents. For example, for projects on Route 220 and Route 360, the load survey study in 1963 showed that the design daily 18-kip equivalents (average of 20 years) on these projects are 326 and 622 respectively. The pavement on Route 220 has a stabilized-soil subgrade and the pavement on Route 360 has no stabilized-soil subgrade. Assuming their SSV to be 1.5 and 15 respectively, the thickness indices (from Fig. 7) required are 14.5 and 13 respectively. Thickness indices provided are 14.1 and 11.1. Thus it is seen that the project on Route 220, built in 1962, does not need additional strength on the basis of 1963 traffic data while the project on Route 360, built in 1959, needs 2 inches of asphaltic concrete on the basis of 1963 data.

If a new pavement was proposed on Route 360, with the same amount of traffic (18-kip equivalent = 622) but with SSV = 1.5, the thickness index required with a soil-stabilized subgrade would be 17.2. The choices of the structural cross section could be very many; for example:

	Choice A	Choice B	Choice C	Choice D
Asphalt mat	7 in.	8 in.	9 in.	10 in.
Stone base	6 in.	6 in.	5 in.	8 in.
Cement-treated base	5 in.	5 in.	4 in.	—
Cement-treated subgrade	6 in.	4 in.	4 in.	8 in.
D =	17.6	17.6	17.2	16.8

Thus, with the help of this nomograph, pavement design can be simplified to facilitate the most economical design compatible with engineering judgment. While designing on the basis of this chart, it is recommended that, in addition to the basic fundamental requirements of better service and durability, the following requirements must be met. These requirements are recommended on the basis of the author's engineering judgment.

1. Flexibility or rigidity is a function of strength and the thickness of material in each layer. Too much flexibility might overstress the asphaltic concrete.
2. A minimum thickness of 1½ in. of asphaltic concrete is necessary for low-traffic roads such as designs I and IA. With an increase in traffic the thickness must also be increased to keep the deflection category low. Thus, for medium traffic it should not

be less than 4 to 6 in. and for daily 18-kip equivalents of 660, the thickness of asphaltic concrete might be 10 in.

3. The stone aggregate base course thickness may vary from 4 to 8 in. Since pavement flexibility increases with the increase in thickness of the base course, an approximate rule of one inch of stone base for every inch of asphaltic concrete may be a good one.

4. Select material has a thickness equivalency value of zero. It may therefore be provided for improving drainage only.

CONCLUSIONS

The following conclusions drawn from this investigation are applicable mostly to the Piedmont area of Virginia from which the satellite study projects were chosen. Some are tentative pending further investigation.

1. The structural performance of the pavements can be evaluated from rebound deflection or curvature, or longitudinal cross-sectional area of the deflected basin, obtained from the Benkelman beam data.

2. Subgrade soil strength, when determined by the Virginia CBR method, cannot be correlated with subgrade soil support values given by the AASHO Committee.

3. Assuming the thickness equivalency value of asphaltic concrete as equal to 1.0, the following thickness equivalency values could be considered for design in Virginia with the layers placed in the order described. In the base course, either a cement-treated or asphalt-treated base layer is to be provided if desired. Below the base, either select material or cement-treated subgrade is to be provided, if desired. Any layer, excluding the asphaltic-concrete layer, could be omitted but not interchanged.

a. Material in surface course	= Asphaltic concrete = 1.0
b. Materials in base course	= Stone base = 0.35, cement-treated base = 1.1, asphalt-treated base = 0.75
c. Materials in subbase for non-stabilized subgrade	= Select material = 0.0
d. Subgrade	= Cement-treated soil = 0.50

4. The tolerable deflection of a pavement is a function of its rigidity; the higher the rigidity the lower the tolerable deflection.

5. The method presently used in Virginia is suitable for design but could be made more flexible by using a system as shown in Figure 7 and a design based on thickness equivalency values.

ACKNOWLEDGMENTS

It is a pleasure to acknowledge the help received from C. S. Hughes and the staff of the Pavement Section. Mr. Hughes was at all times available for free and open discussions and guided the study so that it would serve the best interests of the Virginia Department of Highways. He also offered helpful criticism of the paper.

The author is also grateful for the encouragement and support given by Dr. Tilton E. Shelburne, State Highway Research Engineer.

This investigation was conducted in cooperation with the Bureau of Public Roads.

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Appendix A

DETERMINATION OF DEFLECTION, CURVATURE AND LONGITUDINAL-SECTIONAL AREA OF BASIN

Method of Measuring Deflections

In Virginia, Benkelman beam deflections are determined under a very slowly moving tandem wheel. The tandem wheel has a tire pressure of 60 to 80 psi and a load of 9,000 lb. The procedure used is to begin the test with the truck wheels 2 ft behind the tip of the beam. An initial reading at -2 ft is taken while the truck is stationary at this position. The truck then is moved forward and the maximum dial reading is recorded as the 0-ft reading. The truck is then stopped briefly at a point 2 ft in front of the tip of the beam, and this reading recorded as +2 ft. Simiarly, the readings at +4, +9 and +50 ft are recorded. The deflection at 0-ft reading is therefore equal to 2 (0-ft reading -50-ft reading).

In case the 9-ft dial reading differs from the 50-ft dial reading, the front leg of the beam is assumed to have been in the deflection basin. The Canadian Good Roads Association (14) recommends a formula for correcting this situation. This formula is based on the difference between the 9-ft and 50-ft readings. The standard deviation of the repetitive Benkelman readings in this investigation was found to be 0.0019 in. compared to a value 0.0016 in. which had been obtained previously. As indicated by the standard deviation of 0.0019, the standard deviation of the dial reading (which is one-half the deflection value) would be approximately 0.001 in. Therefore, because of the imprecision of the dial reading, no correction has been made in this investigation for a difference of 0.001 in. This procedure agrees with the procedure used in the Canadian Good Roads Association method, in which no corrections are made unless the 9- and 50-ft readings differ by more than 0.001 in. A difference larger than 0.001 in. was very seldom found in this investigation and hence no correction was applied.

The average deflection readings $\overline{X}$, their standard deviations σ, and the values of $\overline{X} + 2\sigma$ are given in Table A-1.

Method of Measuring Curvature

The curvature was measured by deducting the average of the initial reading at -2 ft and the 2-ft reading from the maximum dial reading recorded at the 0-ft reading.

TABLE A-1

BENKELMAN BEAM DEFLECTION DATA
(Average Standard Deviations and Average Plus 2 σ)

Serial Nos.	Project No.	Project	Average Deflection Readings at 20 Day Intervals			Standard Deviations of Col. 4, 5, and 6			$\bar{X}$ = Mean of Col. 4, 5 and 6	σ = Mean of Col. 7, 8 and 9	$\bar{X}$ x 2σ = Col. 10 + 2 Col. 11
			I	II	III	I	II	III			
1	2	3	4	5	6	7	8	9	10	11	12
1	5	Exp. Proj. Rte. 360 (Design A) (6 Groups)	32	29	35	4.18	4.28	4.27	32.0	4.24	40.48
2	6	Exp. Proj. Rte. 360 (Design B) (6 Groups)	22	18	21	4.69	5.03	6.28	20.0	5.33	30.66
3	7	Exp. Proj. Rte. 360 (Design C) (6 Groups)	35	31	36	18.96	11.27	14.82	34.0	13.35	60.70
4	8	Exp. Proj. Rte. 360 (Design D) (6 Groups)	40	35	39	5.56	4.19	2.19	38.0	3.98	45.96
5	10	7220-033-032	20	20	-	3.59	3.13	-	20.0	3.36	26.72
6	11	220-044-030 (10 Groups)	20	21	25	6.66	3.74	4.74	22.0	5.05	32.10
7	12	220-044-019-C501	20	16	18	3.79	6.45	5.04	18.0	5.09	28.18
8	13	058-071-020	23	22	25	4.27	3.80	5.23	23.33	4.43	32.19
9	14	501-041-102-C501	26	24	27	8.67	8.46	13.47	25.67	10.20	46.07
10	15	304-041-002-C501	16	14	17	1.23	1.52	2.51	15.67	1.75	19.17
11	16	015-019-101, C-2 (Light Design) (8 Groups)	15	17	20	4.83	4.94	6.44	17.33	5.40	28.13
12	17	015-019-101, C-2, (Heavy Design) (6 Groups)	18	15	17	5.04	4.26	6.83	16.67	5.38	27.43
13	18	017-030-008-C	16	13	14	2.66	2.73	3.37	14.33	2.92	20.17
14	19	066-030-001-PI	9	7	13	1.79	2.04	6.01	9.67	3.28	16.23
15	20	066-076-101-PI	15	14	16	2.81	3.21	3.17	15.0	3.06	21.12
16	21	0236-029-007-008	16	12	14	3.89	3.28	3.19	14.0	3.45	20.90
17	33	Exp. Proj. Rte. 58 (Design A) 1962	81 in 1962				24.6		81.0	24.6	130.2
18	34	Exp. Proj. Rte. 58 (Design B) 1962	42 in 1962				3.5		42.0	3.5	49.0
19	35	Exp. Proj. Rte. 58 (Design C) 1962	52 in 1962				12.9		52.0	12.9	77.8
20	36	Exp. Proj. Rte. 58 (Design D) 1962	52 in 1962				16.4		52.0	16.4	84.8
21	37	460-009-019	33	29	29	9.33	8.71	9.28	30.33	9.11	48.55
22	38	460-009-017	27	36	30	12.73	9.38	14.6	31.0	12.23	55.46
23	39	501-041-102-C502	36	37	39	10.16	9.71	11.66	37.33	10.51	58.35
24	40	015-014-101, C-502	25	24	29	5.80	6.58	7.27	26.0	6.55	39.10
25	41	0360-020-031-GI	21	17	16	1.84	1.88	1.94	18.00	1.89	21.78
26	42	0360-020-032-028	19	20	19	6.64	6.38	6.36	19.33	6.46	32.25
27	43	0360-020-019-027	19	22	23	5.41	7.06	6.89	21.33	6.45	34.23

Method of Measuring Cross-Sectional Area of Basin

The longitudinal area of the deflected basin was calculated by assuming the shape of the basins either as a sin curve of the equation $Y = a \sin\left(\frac{\pi}{2} - c\right)$ or a triangular deflection of the equation $Y = b - a\,x$ or a sin curve within a 2-ft radius of applied load, followed by a straight-line slope. In these equations, Y = maximum deflection, a = a constant, and c = $\pi \div$ horizontal distance of the deflected basin.

Appendix B

GROUPS OF THE COMBINATIONS FOR REGRESSION ANALYSIS ON B5500 COMPUTER

1. Satellite projects without stabilized subgrades (7 projects).
2. Satellite projects with stabilized subgrades only (12 projects).
3. All satellite and other projects but excluding experimental project on Route 360 (39 projects).
4. Satellite projects without stabilized subgrades, experimental project on Route 58 and other projects without stabilized subgrades (22 projects).
5. Satellite projects with stabilized subgrades and experimental project on Route 360 (16 projects). The Route 360 project has select material between the soil-stabilized layer and the stone base.
6. Satellite projects with stabilized subgrades, experimental project on Route 360 and five other stabilized subgrade projects in the Piedmont area (21 projects).
7. All satellite projects excluding experimental projects (19 projects).
8. All satellite projects including the two experimental projects and other projects in Piedmont area (43 projects).

Appendix C

FLEXIBLE PAVEMENT DESIGN CHART — PRIMARY AND INTERSTATE SYSTEM — REVISED APRIL, 1967

Traffic Category	Daily Equivalent 18 kip Axle Loads	*Stabilized Subgrade	*Subbase	Base	Binder Course	Surface Course	Thickness Index	
							with sub-grade stabi-lization	with no sub-grade stabi-lization
1	2	3	4	5	6	7	8	9
I	0 - 7	6"	None	4" - 6" (a)	None	P & DS (b)	5.9 to 7.3	2.9 to 4.3
IA	8 - 16	6"	None	6" - 8" (a)	None	P & DS (c)	7.3 to 8.8	4.3 to 5.9
II	17-124	6"	4" - 8"	3" B-3 (345#)	None	165# S-4 or S-5	8.8 to 10.1	5.9 to 7.1
IIA	125-224	6"	4" - 8"	4" B-3 (460#)	None	165# S-4 or S-5	10.1 to 11.6	7.1 to 8.6
III	225-329	6"	4" - 8"	6" B-3 (690#)	None	165# S-5	11.6 to 13.5	8.6 to 10.5
IV	330-429	6"	6" - 10"	6" B-3 (690#)	165# I-2 (1.5")	100# S-5 (1.0")	13.5 to 14.5	10.5 to 11.5
V	430-659	6" - 12"	6" - 10"	8" B-3 (920#)	None	165# S-5 (1.5")	14.5 to 18.0	11.5 to 13.2
VI	660 and Over	6" - 12"	8" - 12"	8" B-3 (920#)	165# I-2 (1.5")	100# S-5 (1.0")	18.0 to 20.7	13.2 to 14.7

(a) Stone-base.
(b) Prime and Double Seal on Contract I. — 165# Plant Mix on Contract II when warranted.
(c) Prime and Double Seal on Contract I. — Up to 300# Plant Mix on Contract II.

*Minimum depth of subgrade stabilization and subbase will depend on soil conditions and design CBR value.

Discussion

W. H. CAMPEN and L. G. ERICKSON, Omaha Testing Laboratories, Inc.—We wish to ask two questions and make some comments.

Question and Comment No. 1: You classify the resilience of subgrades in terms of low, medium, and high, and assign values of 1.5, 1.0 and 0.5 respectively. However, these values appear to be arbitrary without coordination with some field test which can give a numerical range for the three categories. We are wondering what these ranges might be in terms of plate load tests or Benkelman beam measurements.

Question and Comment No. 2: We question the implication of your definition of a flexible layer. You say that a flexible layer shows compression on loading and rebound when the load is removed. This is not necessarily so. You, yourself, have come to the conclusion that asphaltic concrete layers are practically incompressible. It can be proven also that base and subbase layers for flexible pavements can be so designed and compacted as to render them incompressible. The elasticity or rebound in such cases is due entirely to the subgrade resilience.

Since we question your implication, it seems up to us to attempt to define a flexible pavement. We will admit that although we have used the term "flexible pavement" for 30 years, we have not read or heard a definition for it. However, engineers seem to have a common understanding concerning its composition, mechanical properties, and load-distributive characteristics.

As far as composition is concerned, a flexible pavement contains a bituminous paving mat as a wearing surface and layers of base and subbase. The bituminous layer may range from sheet asphalt to coarse asphaltic concrete. The base and subbase layers may consist of compacted soil-aggregate or crushed-aggregate mixtures, or mixtures of the two. These mixtures may be coated with bituminous material and designated as bituminous-treated bases. The layer components must be stable enough to resist displacement (plastic flow) under traffic. Generally, these mixtures possess very little cohesion.

As far as the principal mechanical property of each layer is concerned, it must be able to distribute load to the layer beneath. The layered system as a whole must be able to distribute load over the subgrade. For a given load and a given subgrade, the total thickness of the layered system must be such as to limit the elastic deformation or rebound at the top of the system to prevent cracking and eventual destruction.

This describes the composition and mechanical property of a flexible pavement. In giving a definition for such a pavement, reference must be made to a rigid pavement. The distinction between the two types is based on manner in which they distribute load. A rigid pavement layer distributes load by beam action, whereas a flexible pavement layer distributes load by some angle of distribution such as described by the Boussinesq equation.

N. K. VASWANI, Closure—In reply to Question No. 1 by Messrs Campen and Erickson, the resiliency of subgrade soils has been classified in terms of low, medium and high and arbitrarily assigned values of 1.5, 1.0, and 0.5. These resiliency values are based on the soil classification reports of the satellite projects in the Piedmont area of Virginia and the general knowledge of the types of the soils prevalent; e.g., micaceous silts could be considered to be highly resilient. These values are therefore not recommended for adoption in other soil areas.

In the Piedmont area of Virginia, the Benkelman beam rebound deflections of the pavement are high and the values mentioned were multiplied by the CBR values to provide the effect of resiliency in reducing the soil support value.

The answer to Question No. 2 has been divided into 3 parts: (a) the implication of the definition of a flexible layer; (b) rebound of a flexible layer; and (c) design of a pavement with incompressible layers of asphaltic concrete, base, and subbase.

Implication of the Definition of a Flexible Layer

The term "flexible pavement" was probably used when portland cement concrete rigid pavements came into use. The flexible pavement was considered to consist of materials whose ingredients were not well bonded together and hence had no bridging effect like the materials in the rigid or concrete pavements.

Sophisticated design methods and construction techniques, e.g., improved bituminous mix designs and better compaction techniques, provide a high modulus of elasticity of the materials in the pavement layers. The flexible pavements built presently are of a

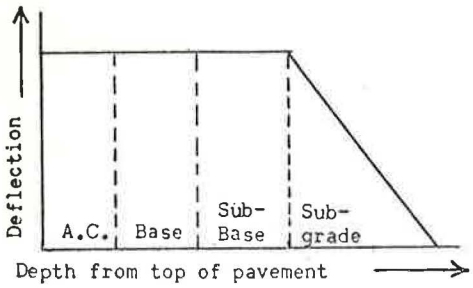

Figure 8.

more rigid type than those built in past years. The old understanding of the term "flexible layer" or "flexible pavement," therefore, needs modification. The modification is necessary to express the concept of present design techniques. This was the object of the definition provided here.

The discussants agree that a "rigid layer" or "rigid pavement" is one which exhibits the bridging effect, which, in terms of design concept, indicates a high modulus of elasticity in bending. Road materials which satisfy this requirement also have a high modulus of elasticity in compression, and since the modulus of elasticity = stress/strain, it follows that the compression produced in such a pavement or layer is very low for a given amount of stress or load.

Thus a perfectly rigid pavement has been defined in the paper as one which does not compress but only bends due to the applied load. A portland cement concrete pavement is, for all practical purposes, considered a perfectly rigid pavement and hence is designed on the basis of the theory of bending only.

The question now arises as to what the other extreme case would be that only compresses and does not bend due to the applied load. We should be able to supply a term for this extreme case. The discussants admit that they have not read or heard a definition of a "flexible pavement," though this word is commonly used. Why then hesitate to define this extreme case by the terms "flexible pavement" or "flexible layer" as the case may be? These terms would retain the same meaning as they have in the past and also would pinpoint the properties of this case.

In the paper these two terms have been defined under the heading "rigidity." The time is coming when the term flexible pavement will be forgotten and the pavements will be designed on the basis of the "degree of rigidity." Design on this basis has been given in the paper for the Piedmont area of Virginia.

Rebound of a Flexible Layer

In the paper, the author states only that a flexible layer shows compression on loading, and not that it is a flexible layer because it rebounds.

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If these layers are stable enough to resist displacement—due to plastic flow—under traffic, the deflection of the layers could be represented by the graph shown in Figure 8. In such case the pavement should preferably be designed on the basis of the theory of bending as given by Westergaard for design of portland cement concrete slabs, rather than on Boussinesq's equation for homogenous isotropic plastic materials.