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Contents

Analysis of Viscoelastic Flexible Pavements

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To develop a better understanding of flexible pavement behavior it is believed that components of the pavement structure should be considered to be viscoelastic rather than elastic materials. In this paper, a step in this direction is taken by considering the asphalt concrete surface course as a viscoelastic plate on an elastic foundation.

Assumptions underlying existing stress and deformation analyses of pavements are examined. Representations of the flexible pavement structure, impressed wheel loads, and mechanical properties of asphalt concrete and base materials are discussed. Recent results pointing toward the viscoelastic behavior of asphaltic mixtures are presented, including effects of strain rate and temperature.

Using a simplified viscoelastic model for the asphalt concrete surface course, solutions for typical loading problems are given.

● THE THEORY of viscoelasticity is concerned with the behavior of materials which exhibit time-dependent stress-strain characteristics. The principles of viscoelasticity have been successfully used to explain the mechanical behavior of high polymers and much basic work (1) has been developed in this area. In recent years the theory of viscoelasticity has been employed to explain the mechanical behavior of asphalts (2, 3, 4) and to a very limited degree the behavior of asphaltic mixtures (5, 6, 7) and soils (8). Because these materials have time-dependent stress-strain characteristics and because they comprise the flexible pavement section, it seems reasonable to analyze the flexible pavement structure using viscoelastic principles. This type of analysis should develop a better understanding of the behavior of flexible pavements, beyond that developed using elastic concepts.

Although this type of theoretical analysis will not result in a simple, universally applicable design formula, it can form the basis for a better understanding of flexible pavements subjected to loading conditions for which there is no precedent. For example, doubt exists among some designers that flexible pavements can accommodate heavy wheel loads and high tire pressures. If there existed a better understanding of the mechanics of behavior of the materials comprising the flexible pavement system, qualitative at least, then perhaps a definitive answer could be established.

It is hoped that this paper provides a step in this direction by illustrating some data pointing to the viscoelastic behavior of asphaltic mixtures and by illustrating the solution of typical loading problems.

BRIEF REVIEW OF FLEXIBLE PAVEMENT ANALYSIS

An analysis of flexible pavements with the intent of establishing a rational design procedure might include the following factors: (a) determination of mechanical properties of the components of the pavement system, (b) development of suitable methods of stress and deformation analysis, (c) identification of space and time characteristics of impressed loads and environmental conditions, and (d) establishment of appropriate failure criteria.

Although all the above factors are interrelated, factors (a) and (b) are perhaps most intimately connected. For example, the characteristic differences between the theories of elasticity, plasticity and viscoelasticity arise from differences in choice of a law relating stress and strain. To date most of the work related to the structural analysis of flexible pavements has been concerned with elastic theory of material behavior. This is not surprising because from a mathematical viewpoint this type of model of the mechanical properties of real materials is the simplest. As pointed out elsewhere in this paper, the inadequacy of the purely elastic model for asphalts and asphaltic mixtures has been demonstrated.

From the standpoint of structural analysis the concentrated normal force or pressure uniformly distributed over a circular area forms the simplest geometric idealization of an actual wheel load. Other space distributions (ellipse, tandem loads, etc.) can be included at the expense of computational effort. It appears, however, that little attention has been given to the effect of shear forces transmitted to the pavement surface, an effect which would seem to be particularly important during starting and stopping of vehicles. The effects of frequency and duration of load application as well as repetition of load have infrequently been considered. These factors are particularly significant for materials with frequency-dependent response mechanisms, such as asphaltic mixtures. Finally, it may be noted that the magnification effect on stress and deformation produced by moving loads does not seem to be significant at present, although substantial increases in landing speeds of aircraft coupled with increased wheel loads and pavement thicknesses could make this problem important. Accordingly, a quasi-static treatment of stress and deformation analysis is possible.

In formulating a specific boundary value problem for analytical solution, various levels of approach can be followed. The highest level (that is, fewest assumptions made) involves the treatment of the pavement system as a multi-layer continuous solid. This approach has been taken by Burmister (9), for example, in the case of an elastic material. The computational difficulties associated with this approach are quite severe, and it is difficult to obtain the effect of changes of various parameters of the system. A second level of approach, first applied to pavement analysis by Westergaard, treats the pavement as a plate on an elastic (set of independent springs) foundation. While simplifying the mathematics considerably, this method suffers from the disadvantage that stresses in the subgrade (elastic foundation) cannot be determined and further, that transverse normal and shearing stresses in the plate cannot be determined. Pickett (10) has extended Westergaard's work to permit determination of subgrade stresses. This is done by replacing the set of independent elastic springs (or equivalently, a dense liquid) by an elastic solid; however, the mathematics in this solution is also rather involved. A further approach (11) retains the simplification of the dense liquid subgrade but allows for the effect of transverse normal and shearing stresses. Results show that this effect is singificant in the range for which the ratio of radius of loaded area to pavement thickness unity.

It should be noted that each of the previously mentioned levels of approach has been applied only in the case of elastic material properties. (A recent paper by Hoskins and Lee (15) is an exception.) Extension of these results in part for application to viscoelastic materials is possible through the use of a correspondence principle due to Lee (12). Finally, it may be noted that the concept of the dense liquid subgrade has been generalized by Reissner (13) to allow for differential shear stiffness in the subgrade, a more realistic model than the independent spring idea. Some advantages of this approach for viscoelastic materials have been discussed by Pister and Williams (14).

While maximum stress or limiting strain (or deflection) theories of failure have frequently been applied as design criteria for flexible pavements, it is believed that these theories are inadequate for application to viscoelastic flexible pavements. A theory incorporating environmental effects on material properties as well as rate and repetition of loading and accumulation of deformation is needed. A preliminary step in this direction, applied to the related problem of failure of high polymer solid propellants, has been taken (16).

VISCOELASTIC BEHAVIOR OF ASPHALTIC MIXTURES

As stated previously, the theory of viscoelasticity is concerned with the relation between stress as a function of time and strain as a function of time. To illustrate this behavior conveniently, several mathematical models have been proposed. Depending on the imposed conditions of stress or strain and time, these models can be related to the actual behavior of a particular material. Various models are discussed in the following paragraphs. To illustrate their suitability to describe the behavior of asphaltic mixtures, typical results of tests on a particular asphaltic mixture are presented. Data for the components of this mixture have been described elsewhere (20) and are therefore not included in this paper.

Figure 1a shows a Maxwell element, incorporating in series a spring (representing elastic behavior with a modulus E_1) and a dashpot (representing viscous behavior with a viscosity η_1).

This model represents a material which when subjected to stress, undergoes an instantaneous elastic deformation together with deformation increasing with time. The model can also be used to represent a material exhibiting relaxation of stress with time when the material is held at constant deformation. This type of behavior is shown in Figure 1b. When the model is quickly deformed to a strain ϵ_1 and then constrained so that ϵ_1 remains constant, the stress will gradually relax with time. The differential equation relating stress and strain for the Maxwell element is:

$$\frac{d\epsilon}{dt} = \frac{1}{\eta_1}\sigma + \frac{1}{E_1}\frac{d\sigma}{dt} \tag{1}$$

in which σ is the applied axial stress. For the condition described above, $\frac{d\epsilon}{dt} = 0$ and the solution of the equation is

$$\sigma = \sigma_0 \exp(-\frac{E_1}{\eta_1} t) \tag{2}$$

in which σ_0 = initial stress for the material deformed to strain ϵ_1; and

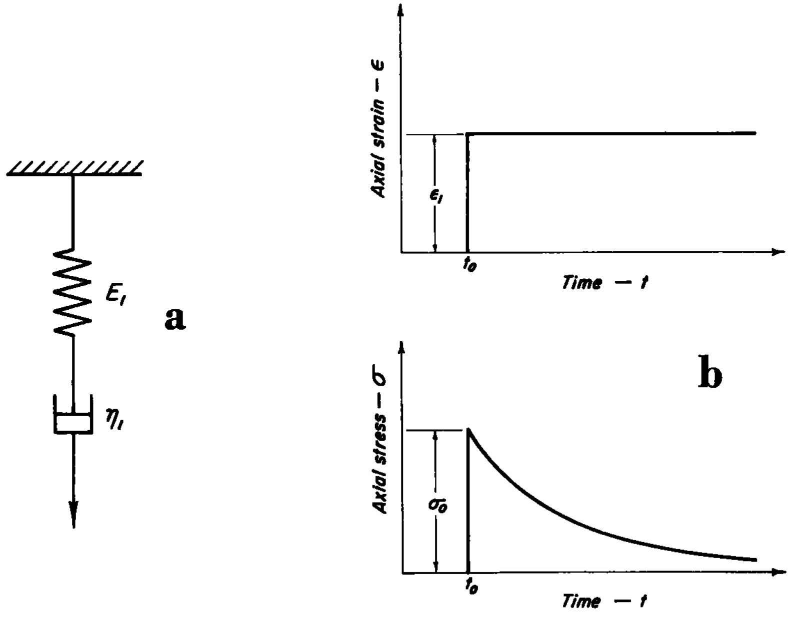

Figure 1. Representing (a) Maxwell model, and (b) relaxation of stress with time for simple Maxwell model.

σ = stress at any time, t, during relaxation of stress at constant strain. This equation indicates that the stress relaxes exponentially with time. The ratio $\frac{\eta_1}{E_1}$ has the dimensions of time and is called the relaxation time for the material, and designated by the symbol, τ. Thus the equation can be written:

$$\sigma = \sigma_0 \exp\left(-\frac{t}{\tau}\right) \quad \text{or} \quad \sigma = E_1 \epsilon_1 \exp\left(-\frac{t}{\tau}\right) \tag{2a}$$

It may be noted that τ represents the time required for the initial stress to decrease to the value $\frac{\sigma_0}{e}$. For asphaltic mixtures, there is evidence to show that a simple model is not sufficient to describe the behavior in stress relaxation. It is necessary to couple either a finite or an infinite number of Maxwell models in parallel. This type of model (shown schematically in Figure 2) is called a generalized Maxwell model. If the number of elements is allowed to approach infinity, it may be assumed that the material has a continuous distribution of relaxation time, that is, E is a continuous function of τ and the equation representing the relaxation of stress at constant strain becomes

$$\sigma = \epsilon \int_0^\infty E(\tau) \exp\left(-\frac{t}{\tau}\right) d\tau \tag{3}$$

Data from a stress relaxation test on an asphaltic mixture are shown in Figure 3. The general pattern of stress relaxation is an exponential decay with time. Thus, a Maxwell type model would probably be suitable for conditions pertaining to stress relaxation with time.

To illustrate behavior of an asphaltic mixture in creep and creep recovery, use can be made of a Burgers' model (Fig. 4a). The Burgers' model has the advantage that it displays under load, instantaneous elastic deformation, retarded elastic deformation and plastic or viscous deformation (Fig. 4b). Brown and Sparks (3) have found that this type of model defines the behavior of certain paving asphalts in creep and creep recovery experiments. In their work it was necessary to couple in series four Kelvin elements (spring and dashpot in parallel) rather than the one shown in Figure 4a. A similar type of model could be fitted to the data shown in Figure 5 for asphaltic mixtures in creep and creep recovery experiments.

The difficulty with application of the Burgers' model in this case, however, is that the instantaneous elastic recovery in the specimens is different from the instantaneous elastic deformation under load. Moreover, data for specimens subjected to various amounts of creep (not included in this paper) exhibited different amounts of elastic recovery. For the Burgers' model, the elastic recovery is the same regardless of

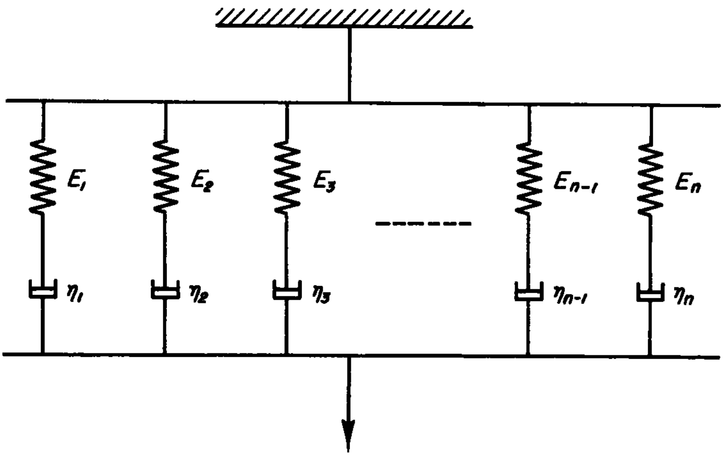

Figure 2. Generalized Maxwell model.

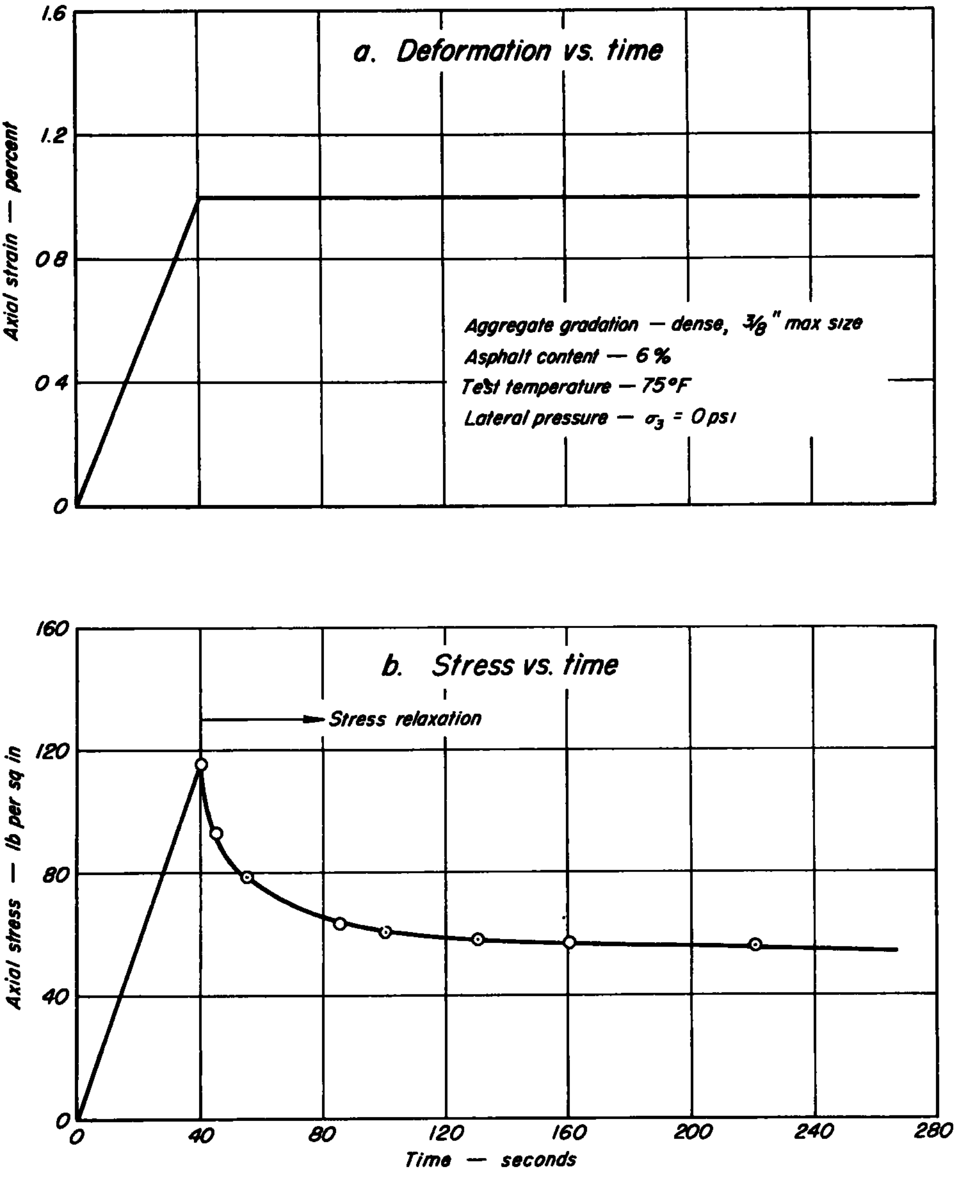

Figure 3. Stress relaxation with time—asphaltic mixture.

the time over which the specimen is allowed to deform under load. Hence it is believed that a model similar to the Burgers' model but incorporating this variability of elastic recovery with time would be more appropriate.

A model depicting such behavior is shown in Figure 6. It may be noted that this model was also suggested by Kühn and Rigden (4) for asphalts. The behavior of this model will be discussed in the section containing examples of stress analysis.

To illustrate that a generalized model can be used to represent the behavior of an asphaltic mixture under load, data from a series of triaxial compression tests at different rates of loading and temperatures of test on the same mixtures (Figs. 3 and 5) are shown in Figures 7 and 8. The technique for analysis and presentation is based on the treatment developed by Smith (17).

For a generalized Maxwell model (Fig. 2) subjected to a constant rate of strain, R, the relationship between stress, strain and time is

$$\sigma = R \int_0^\infty \tau \, E(\tau) \left[1 - \exp\left(-\frac{\epsilon}{R\tau}\right) \right] d\tau \tag{4}$$

Smith (17) has made use of the generalized Maxwell model in his studies of the viscoelastic behavior of a high polymer, polyisobutylene, under a constant rate of elongation in simple tension. In his presentation Smith has rewritten the foregoing equation into an equivalent form for more convenient analysis of the test data. The equivalent equation is

$$\frac{\sigma}{R} = \int_{-\infty}^{\infty} M(\tau) \, \tau \left[1 - \exp\left(-\frac{\epsilon}{R\tau}\right) \right] d \ln\tau \tag{5}$$

in which

$M(\tau)$ is a relaxation distribution function defined such that
$M(\tau) \, d \ln\tau$ is the contribution to the instantaneous modulus of those elastic mechanisms whose relaxation times lie between $\ln\tau$ and $\ln\tau + d \ln\tau$.

The advantage of this equation from an experimental standpoint is that σ/R is a function only of ϵ/R and that data obtained at different strain rates should superpose to give a single curve on a plot of log σ/R vs log ϵ/R. That this occurs for an asphaltic mix in triaxial compression is shown in Figure 7. Three curves representing three

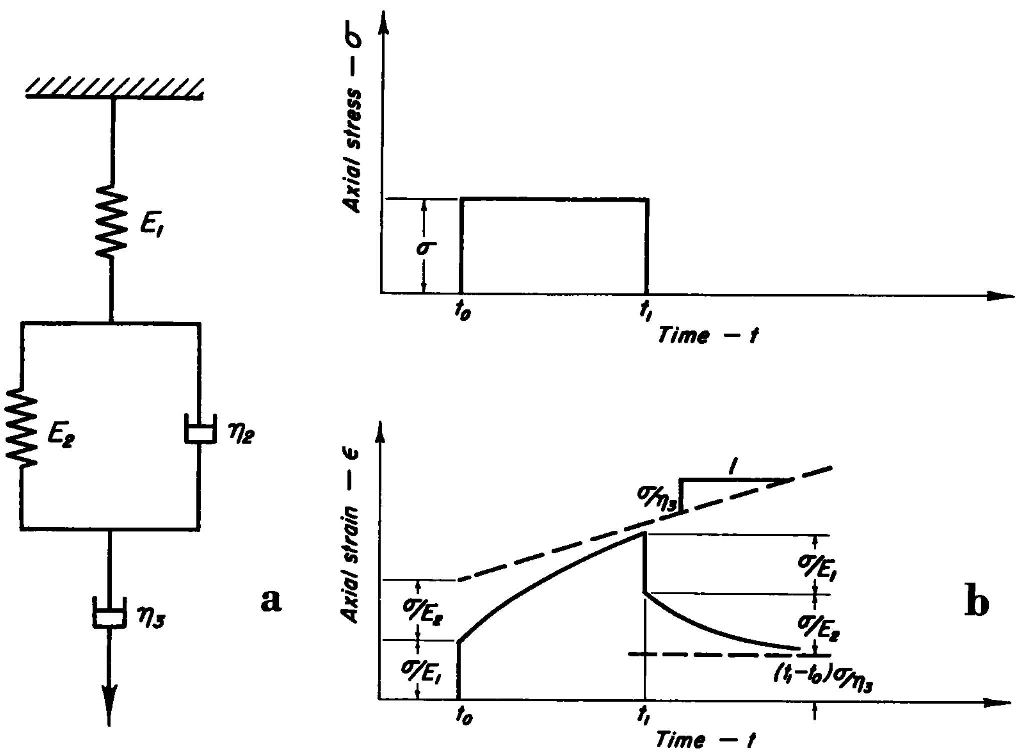

Figure 4. Schematic diagram of (a) Burgers' (four-element) model, and (b) axial stress and strain vs time relationships for Burgers' model subjected to stress in time interval t_0 to t_1.

different temperatures are illustrated. The stress-strain data shown in the figure are based on small deformations, up to approximately two percent strain.

To combine data obtained at various temperatures a reduced variable scheme proposed by Ferry (18) can be used. This analysis (17) is based on the assumption that all relaxation times have the same temperature dependence and that the modulus of each spring in the model is proportional to the absolute temperature. Introducing these assumptions Eq. 5 becomes

$$\frac{\sigma T_0}{RTa_T} = \int_{-\infty}^{\infty} M(\tau)\,\tau \left[1 - \exp\left(-\frac{\epsilon}{RTa_T}\right)\right] d\,\ln\tau \tag{6}$$

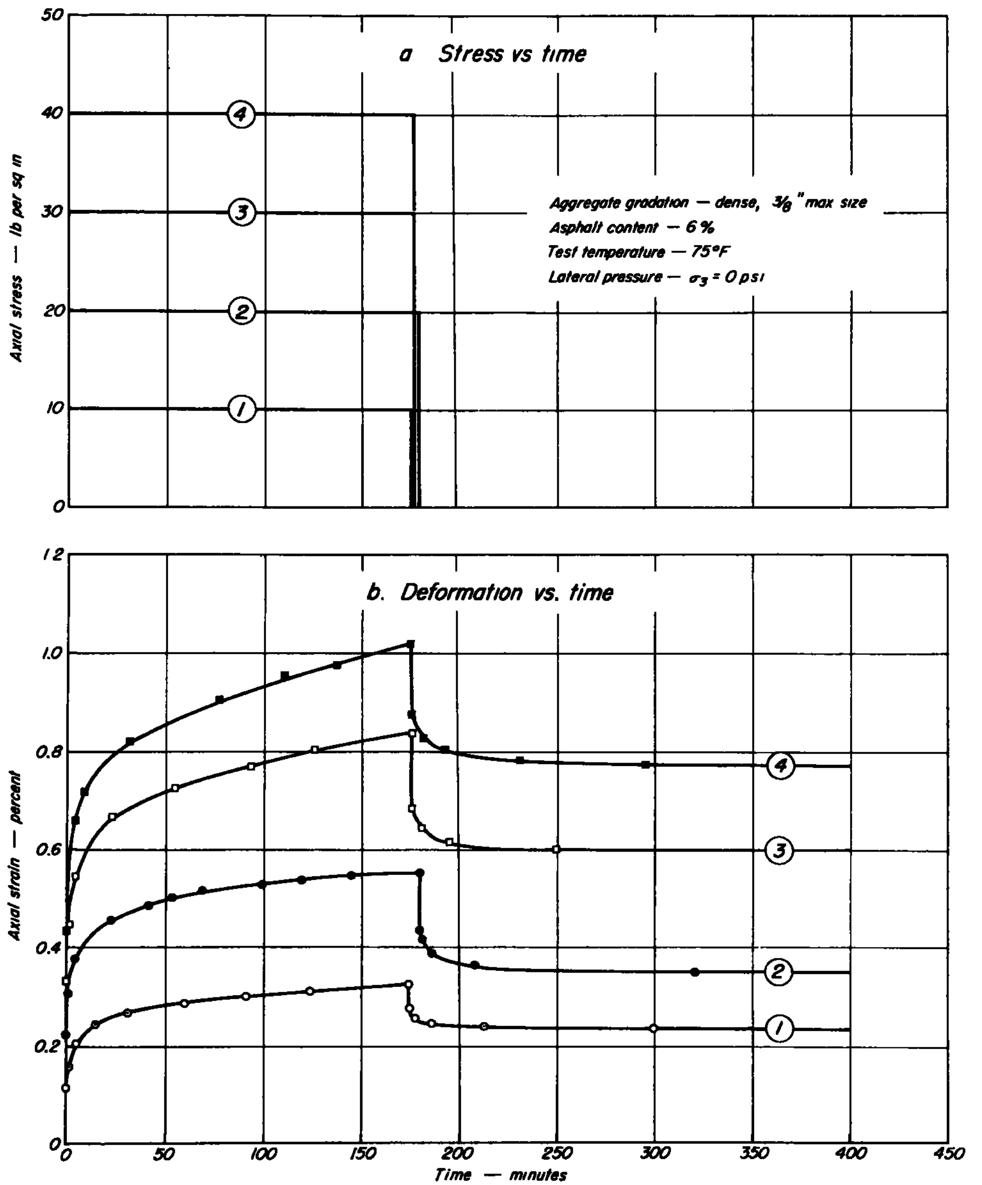

Figure 5. Creep and creep recovery for asphaltic mixtures subjected to different magnitudes of axial stress.

in which

T_0 = reference temperature (25°C or 298°K in this case);
T = temperature of test; and
a_T = ratio of relaxation time at temperature T to value at T_0.

This interpretation has been applied to the data in Figure 7 and the resultant plot of $\log \frac{\sigma T_0}{RTa_T}$ vs $\frac{\epsilon}{Ra_T}$ is shown in Figure 8.

Brodnyan (19) has used a similar technique to plot the dependence of shear modulus of asphalt as a function of frequency. The range in values of a_T vs temperature for a number of different asphalts is shown in Figure 9. Also in the figure are plotted the values of a_T required to obtain the curve shown in Figure 8 for the particular mixture under investigation. It can be noted that the values fall within the band, indicating that the temperature dependence of the viscoelastic characteristics of the mixture is related to that of the asphalt—a not unreasonable conclusion.

In general the data presented in this section indicate that asphaltic mixtures are viscoelastic (at least for small deformations) and that to depict the visco-

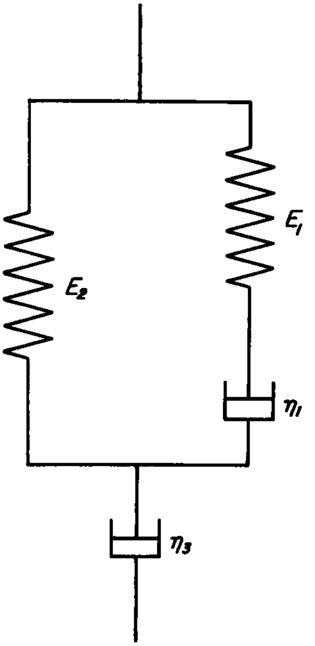

Figure 6. Model representing viscoelastic behavior of asphalt in extension.

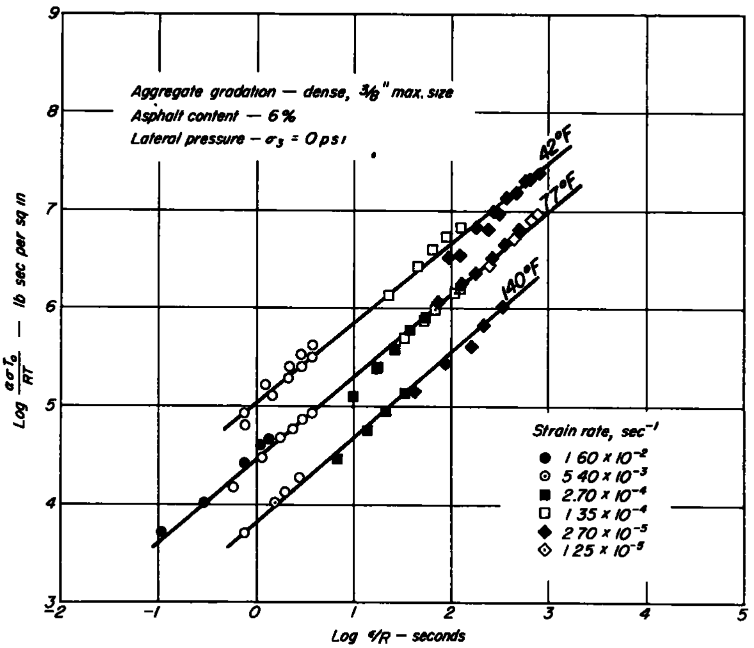

Figure 7. Stress vs strain data for asphaltic mixtures in triaxial compression reduced to unit strain rate at three temperatures.

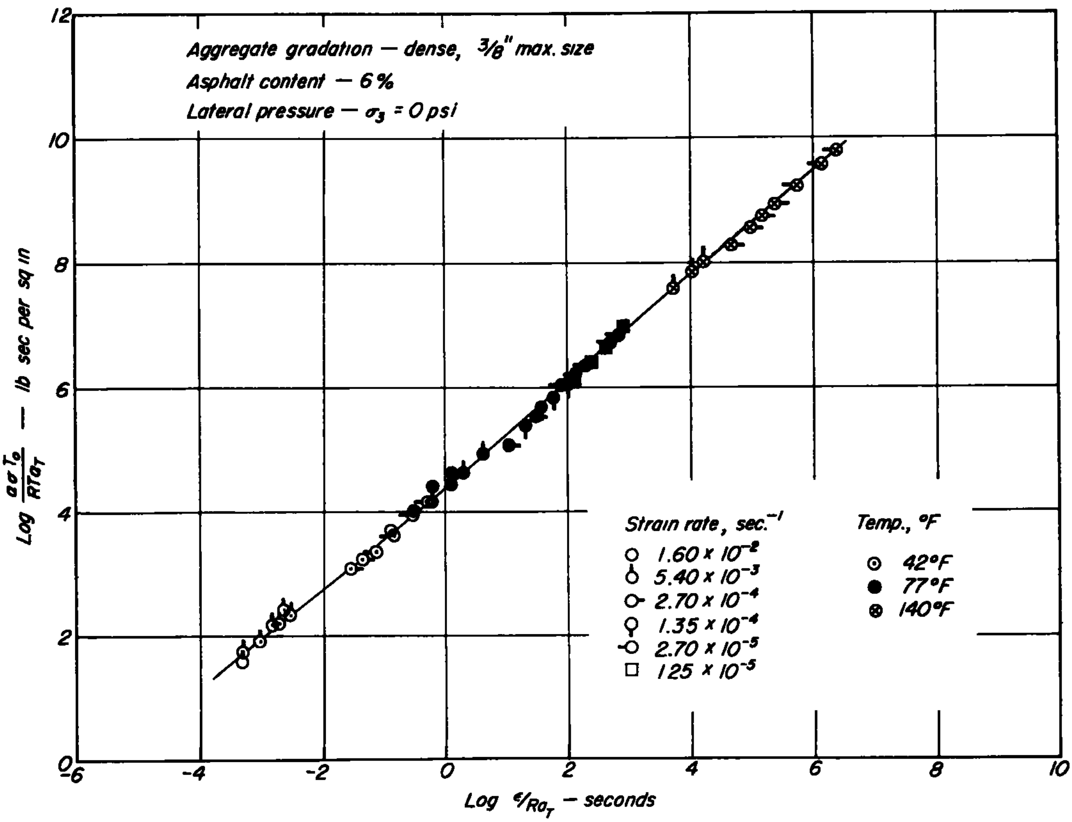

Figure 8. Stress vs strain data for asphaltic mixtures in triaxial compression reduced to unit strain rate and 77 F (298 K).

elastic behavior of an asphaltic mixture a complex type of model is probably required.

EXAMPLES OF STRUCTURAL ANALYSIS OF VISCOELASTIC PAVEMENTS

Two examples which provide qualitative insight into the behavior of flexible pavements are discussed. It must be recognized that the selection of the model for representing the viscoelastic mechanical properties of the surface course as well as the selection of the type of load has been dictated by the desire to present the results unencumbered by excessive mathematical argument. More realistic problems, for example, distributed pressure on a viscoelastic plate, including effects of transverse normal and shear stress, repeated loads, etc., and more realistic representations of material properties are currently under study.

Viscoelastic Beam on an Elastic Foundation

The problem of an infinite beam resting on an elastic foundation and loaded with a time-dependent load will serve to illustrate the effect of time and material properties on the deflection in a viscoelastic beam (Fig. 10). Considering a beam of unit width, subjected to a load q(x, t) per unit width, the Bernoulli-Euler equation for the elastic beam deflection is

$$\frac{E\,h^3}{12}\,\frac{\partial^4 w}{\partial x^4} + k\,w = q \tag{7}$$

in which E is the elastic modulus of the beam, h the beam depth, w the transverse deflection, q the load intensity on the beam and k the subgrade modulus. In the case of a viscoelastic beam, the modulus of elasticity must be replaced by the time-dependent relation between stress and strain. For the present example a 3-element model, Figure 10b (or Fig. 6 with $\eta_3 = 0$) exhibiting instantaneous elasticity, creep and recovery, has been selected. The stress-strain relation for this model can be written

$$\left[E_1 + \eta_1 \frac{\partial}{\partial t}\right]\sigma(t) = \left[E_1E_2 + \eta_1 (E_1 + E_2)\frac{\partial}{\partial t}\right]\epsilon(t) \tag{8}$$

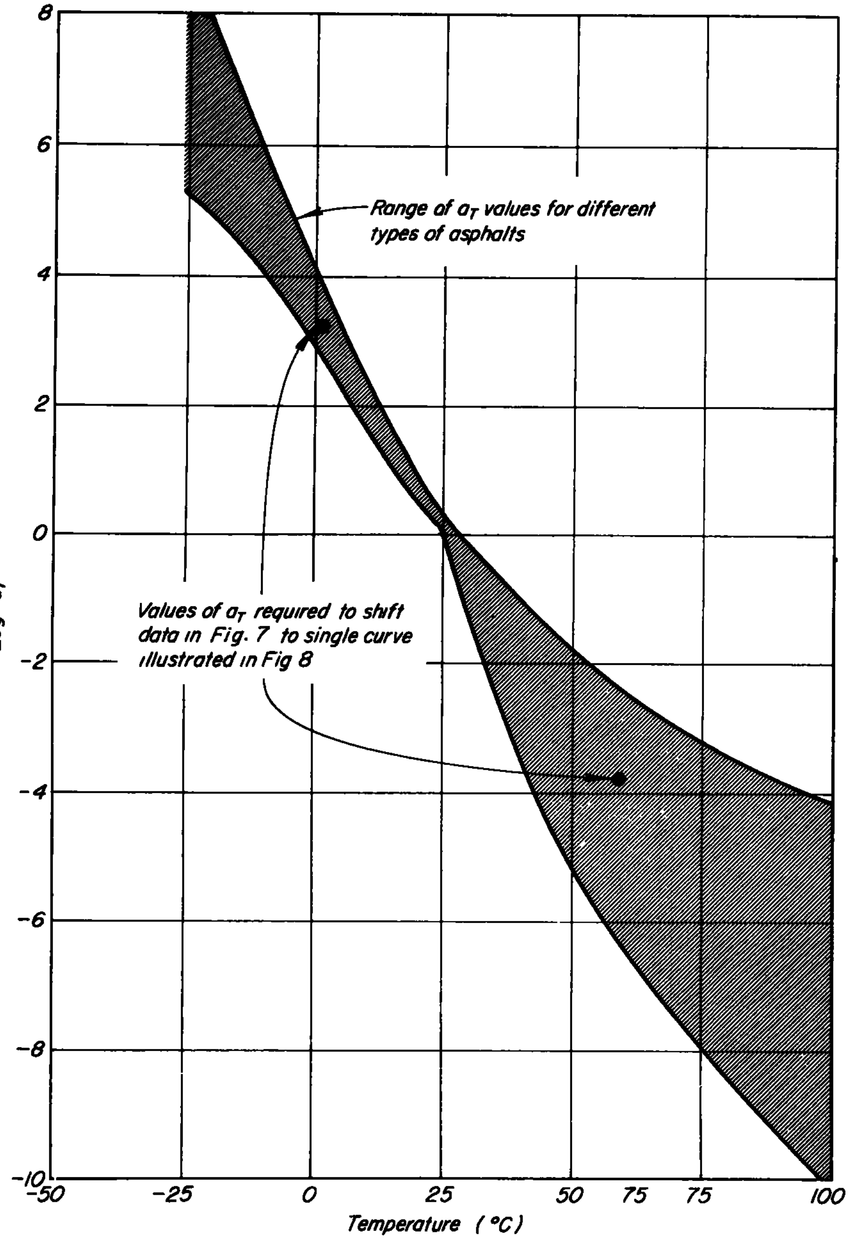

Figure 9. Temperature dependence of a_T values for various asphalts (after J.G. Brodnyan).

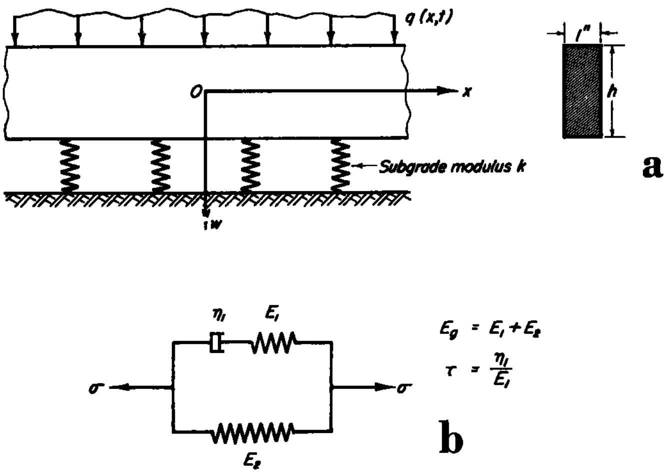

Figure 10. Schematic diagram of (a) viscoelastic beam on elastic foundation, and (b) mechanical model of material.

In terms of differential operators L_1 (t), L_2 (t) Eq. 8 can be written

$$L_1 \sigma = L_2 \epsilon \tag{8a}$$

By replacing the modulus of elasticity E in Eq. 7 by the ratio of the operators L_2/L_1, with time dependence of w and q implied, an equation for the viscoelastic behavior of the beam is obtained

$$\frac{h^3}{12}\ \frac{L_2}{L_1}\left\{\frac{\partial^4}{\partial x^4}\ \left[w(x,t)\ \right]\right\} + k\, w(x,t) = q(x,t) \tag{9}$$

To this equation are added the boundary conditions

$$w(\pm\infty,\ t) = \left.\frac{\partial w}{\partial x}\right|_{\pm\infty,\ t} = 0$$

Further, the beam is assumed to be at rest at t = 0. The solution of the differential equation subject to the prescribed boundary and initial conditions can be brought about through the use of repeated Laplace and Fourier Cosine Transforms. The details will not be recorded inasmuch as the procedure is not novel. However, it may be noted that in performing the inverse transformations to recover the time and space dependence of w, it is more convenient to perform the Laplace inversion before the Fourier inversion. The final result for the deflection due to a concentrated load applied at t = 0 and held constant is

$$\frac{\pi k}{P\left(\frac{k}{E_g I}\right)^{\frac{1}{4}}}\ w\left(x,\frac{t}{\tau}\right) = \int_0^\infty \left\{\left[\frac{1}{1+z^4} - \frac{1}{1+nz^4}\right] \exp\left[-\frac{(1+nz^4)}{1+z^4}\right]\frac{t}{\tau} + \frac{1}{1+nz^4}\right\} \cos\left[\left(\frac{k}{E_g I}\right)^{\frac{1}{4}} xz\right] dz \tag{10}$$

in which

$$n = \frac{E_2}{E_1 + E_2} = \frac{E_2}{E_g}$$

is the ratio of the long-time and short-time elastic moduli of the material in bending and $\tau = \frac{\eta_1}{E_1}$ is the relaxation time of the material under static loading.

The center deflection as a function of time for two values of the ratio of long-time and short-time elastic moduli, $n = \frac{1}{10}, \frac{1}{100}$ was obtained by numerical integration. The results are shown in Figure 11. It will be noted that elastic deflection of a beam with elastic modulus E_g is reached instantaneously. The beam then deforms viscoelastically attaining asymptotically the deflection of a beam with elastic modulus E_2.

Repeated Load on an Unconfined Compression Cylinder

As a final example of deformation analysis of viscoelastic materials, the repeated compressive loading of an unconfined cylinder (Fig. 12b) is discussed. The nature of the loading (Fig. 12a) is that of a series of load pulses. The axial stress and strain, using the model discussed in the previous example, are related by Eq. 8. It develops in this application that it is convenient to define the relaxation time on the basis of the combined elastic modulus of the two springs rather than for E_1 only as before. Accordingly, with

$$\tau^* = \eta_1 \left[\frac{E_1 + E_2}{E_1 E_2}\right]$$

where the bracketed expression is the compliance of the two springs in parallel, the above equation is

$$\left[\frac{E_1}{\eta_1} + \frac{\partial}{\partial t}\right]\sigma(t) = (E_1 + E_2)\left[\frac{1}{\tau^*} + \frac{\partial}{\partial t}\right]\epsilon(t) \tag{11}$$

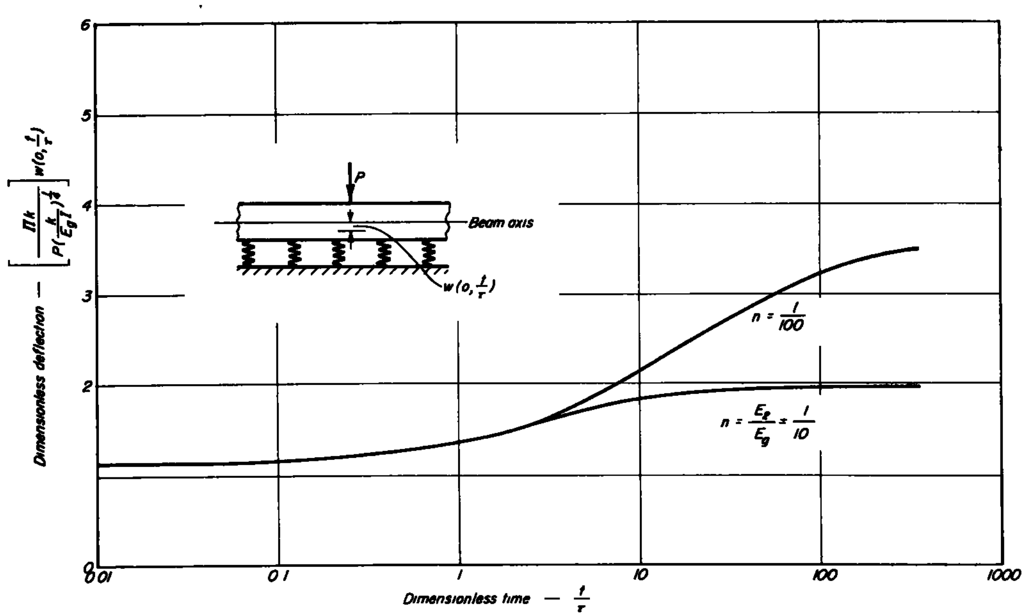

Figure 11. Center deflection vs time for viscoelastic beam on elastic foundation.

The solution of this differential equation for one cycle of loading and recovery (Figs. 12a and 12c) can be conveniently discussed in four steps as follows:

Referring to Figure 12c:

1. Segment a b, instantaneous elastic response at $t = 0$, with

$$\epsilon(0) = \frac{\sigma}{E_1 + E_2}$$

2. Segment b c, viscoelastic strain at constant stress, $0<t<t_0$, with

$$\epsilon(t) = \frac{\sigma}{E_1 + E_2}\left\{1 + \frac{E_1}{E_2}\left[1 - \exp\left(-\frac{t}{\tau^*}\right)\right]\right\}$$

3. Segment c d, instantaneous elastic response at $t = t_0$ on unloading, with change in strain $\Delta\epsilon(t_0)$ where

$$\Delta\epsilon(t_0) = \frac{\sigma}{E_1 + E_2} \exp\left(-\frac{t_0}{\tau^*}\right)$$

4. Segment d e, strain recovery at zero stress, $t_0<t<t_1$, with

$$\epsilon(t) = \frac{\sigma}{E_2}\left[1 - \exp\left(-\frac{t_0}{\tau^*}\right)\right]\exp\left(-\frac{t - t_0}{\tau^*}\right)$$

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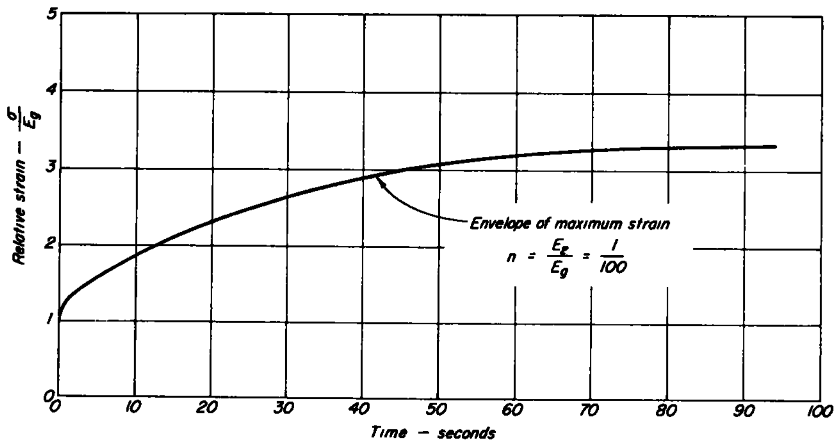

Figure 13. Axial strain vs time—repeated load on unconfined compression cylinder.

the possibility of accumulative buildup of strain occurs (Figs. 12c and 13). It may be noted that the addition of a dashpot in series with the present model (Fig. 6) would contribute a permanent deformation proportional to the total number of load repetitions. This model may possibly be suitable for determining the permanent deformation developed in flexible pavements subjected to repeated applications of load. Results of this type for triaxial compression repeated load tests are presented by Monismith and Secor (20).

Finally, it must be emphasized that more elaborate models must be used for analysis of the behavior of materials subjected to loads with differing frequency distributions. For example, to compare the effects of two different rates of load repetition, two sets of viscoelastic coefficients, each appropriate to one frequency must be employed.

SUMMARY

In this paper limitations imposed by purely elastic analysis of flexible pavements are reviewed. The importance of inclusion of time-dependent material properties and loading conditions in formulating a rational method of pavement design is emphasized. Some experimental data illustrating the viscoelastic behavior of asphaltic mixtures under various types of loading are presented. Representation of viscoelastic material properties by means of mechanical models is discussed and several typical models are shown. Two simple examples illustrating effects of time-dependent loading and material properties on the formulation and solution of viscoelastic boundary value problems are presented.

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A Tentative Flexible Pavement Design Method for Florida

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The current results of a study being made to develop a design procedure for determining the thickness of the layers of a flexible pavement structure to carry a specified predicted traffic load are described. The objective was to develop a correlation between bearing values of materials, layer thicknesses, traffic loads, and pavement performance.

The approach was to first select representative pavements in rural locations with 10-year performance records and observed condition ratings of poor to very good. Actual field conditions were determined at each test site, including moisture content, density and field bearing value of each layer, transverse and longitudinal profiles, and evlauation of surface cracking. Field densities and moisture contents were compared with laboratory tests for optimum moisture and density.

A rating system was developed which indicated by a single number the condition of the pavement in regard to deviation of longitudinal profile, depth of rutting, and degree of surface cracking. The resulting condition index was adjusted for traffic volume and converted to a service rating. A service rating of 60 was considered to be a realistic value for the dividing line between poor and good performance.

Thickness requirements were developed using service rating, equivalent wheel load data, and a modified California bearing test. Design curves that predict all unsatisfactory pavement performance with a minimum of over-design are presented.

IN 1955, the State Road Department of Florida initiated a study of its flexible pavement design method. The major purpose of the study was to evaluate the design criteria and the performance of flexible pavements constructed in Florida. At this time, the Department used an empirical method of design which generally resulted in a standard section design accompanied by minimum Florida bearing values. This method has been described in detail in a recent report (1).

In initiating the study, numerous design methods were reviewed and it was found that the California bearing ratio (CBR) design method or a modification of this method was the most widely used and accepted. In addition, extensive research had been performed in conjunction with this method by many agencies and principally the Corps of Engineers. Inasmuch as valuable information and experience was available on the CBR method, the state considered using this basic method if it was found that a new design method was warranted. Research was also planned to include work, at a later date, with plate tests and layered theory.

The first field and laboratory study was undertaken in 1957. The major findings and developments of this study were reported (1, 2) and are briefly as follows:

1. Test sites were selected using selective sampling techniques. The sites were limited to sections of rural highways constructed in 1947 or 1948, with "observed conditions" ratings of poor through very good. Seventy-six test sites were selected of which 23 sites were investigated in 1957. In 1959 28 additional sites were studied (Fig. 1).

2. A rating system was developed which indicated by a single number the condition of the road section relative to any other section; the rating system considered classes of cracking, deviation of longitudinal profile, and average depth of rutting. A "condition index" number resulted when the factors were weighted and expressed as a product (2, 3) (Fig. 2).

3. The condition index was adjusted for traffic according to a family of curves developed from the U. S. Bureau of Public Roads (4). An adjustment was derived for conditions existing in Florida, and the original Bureau of Public Roads' equation modified. The condition index when adjusted for traffic was denoted as service rating.

4. Field and laboratory strength tests performed on the base, subbase and subgrade were related to service rating. It was found that:

(a) The Florida bearing design methods did not correlate with performance.

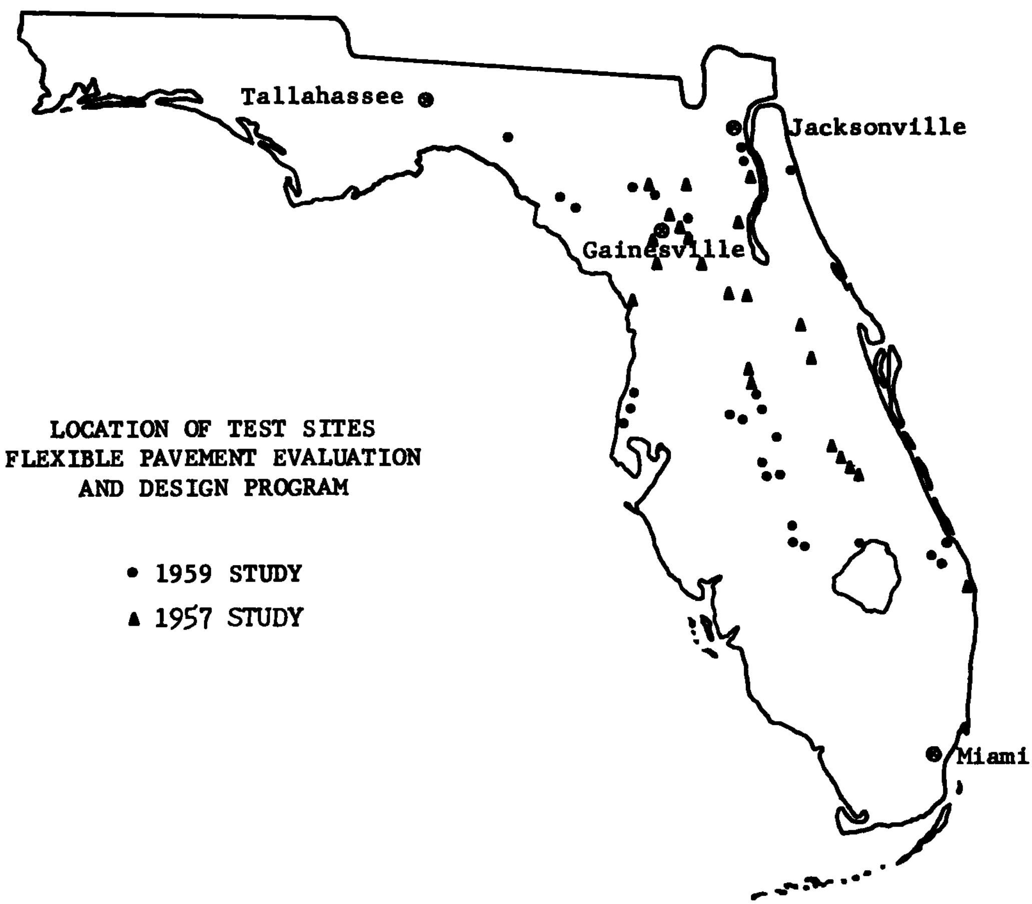


Figure 1.

(b) The California bearing ratio design method (5) did correlate with performance when a 12,000-lb wheel load design curve was used for all the test sections which had an average ADT of 1,850, and a service rating of 60 was used as the dividing line of good and poor performance.

The use of a service rating of 60 was considered to be realistic when compared to visual inspection and engineering judgment.

During 1958, field and laboratory studies were continued and additional data obtained to further evaluate performance and develop a design method for Florida. The investigation dealt primarily with flexible pavements having limerock as a base material. However, a very limited study was made of sand clay and other base materials.

FORMULA FOR DETERMINING CONDITION INDEXES

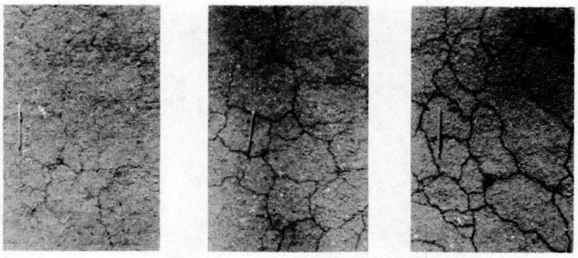
$$C = 100 \left[\frac{1 + \left(1 - \frac{b_1}{c_1}\right)\left(1 - \frac{b_2}{c_2}\right)\left(1 - \frac{b_3}{c_3}\right)\left(1 - \frac{b_4}{c_4}\right)}{2}\right]\left[1 - \frac{b_5}{c_5}\right]\left[1 - \frac{b_6}{c_6}\right]$$

b_1 = Area of Class 1(A) cracking divided by total area.	c_1 = 10
b_2 = Area of Class 2(B) cracking divided by total area.	c_2 = 4
b_3 = Area of Class 3(C) cracking divided by total area.	c_3 = 2
b_4 = Area of Class 4(D) cracking divided by total area.	c_4 = 1
b_5 = Standard deviation of longitudinal profile of original pavement, inches.	c_5 = 1 inch
b_6 = Average depth of rutting in wheel paths of original pavement, inches.	c_6 = 2.25 inches

Substituting the values listed above in Equation 1 with n = 6, the Condition Index for flexible pavement is computed by:

$$C = 100 \left[\frac{1 + \left(1 - \frac{b_1}{10}\right)\left(1 - \frac{b_2}{4}\right)\left(1 - \frac{b_3}{2}\right)\left(1 - b_4\right)}{2}\right]\left[1 - b_5\right]\left[1 - \frac{b_6}{2.25}\right]$$

Note: In patched or replaced areas the last recorded values of cracking, longitudinal profile, or rutting are to be used.



A B C D

(A) Class 1 cracking - fine cracks with no well defined pattern.
(B) Class 2 cracking - fine cracks with a grid-like pattern.
(C) Class 3 cracking - similar to Class 2 with widening of the cracks and some spalling along the edges.
(D) Class 4 cracking - progression of Class 3 cracking with pronounced widening of the cracks and separation of the resulting segments into individual loose pieces.

Figure 2.

All of the test sections in this study were 11 yr old and were of the same group selected for study in 1957. Selection of the test sites was discussed in detail in a previous report (2). Table 1 gives the number of test sites, class of traffic, and the range of average ADT for the 11-yr period.

Field data were obtained as in the 1957 study. However, additional field CBR tests were made. These were principally on the base and subgrade materials. The field testing was essentially as follows:

TABLE 1

TRAFFIC DATA 1959 STUDY

Traffic Class	11 Years Total Range of Traffic	Number of Test Sites	Range of Actual Average ADT (2 lane)
1	0 - 1 x 10^6	1	125
2	1 - 2 x 10^6	2	396 - 399
3	2 - 3 x 10^6	2	725 - 725
4	3 - 6 x 10^6	4	800 - 1,190
5	6 - 9 x 10^6	6	1,535 - 2,080
6	9 - 12 x 10^6	6	2,380 - 2,812
7	Over 12 x 10^6	7	3,220 - 4,568

Note: Average of System 2,070.

1. Profiles were established, both longitudinal and transverse (Fig. 3);
2. Pavement surfaces were evaluated for class of cracking;
3. Field CBR tests were made on all the layers encountered (Fig. 4);
4. Field density and field moisture tests were made on all layers, in and between wheel paths and on the shoulders; and
5. Samples were obtained for laboratory testing.

PRELIMINARY FINDINGS

As a result of the field and laboratory studies, preliminary correlations between density, moisture and rutting were made. The important findings and pertinent comments are as follows:

1. The average field density of the base material was about 95 percent of the maximum laboratory density determined by modified AASHO methods.
2. The average field density of the subbase material was generally equal to (100 percent) and sometimes greater than maximum laboratory density.
3. The field moisture content of the base material (limerock) was generally equal

Figure 3. Typical test site.

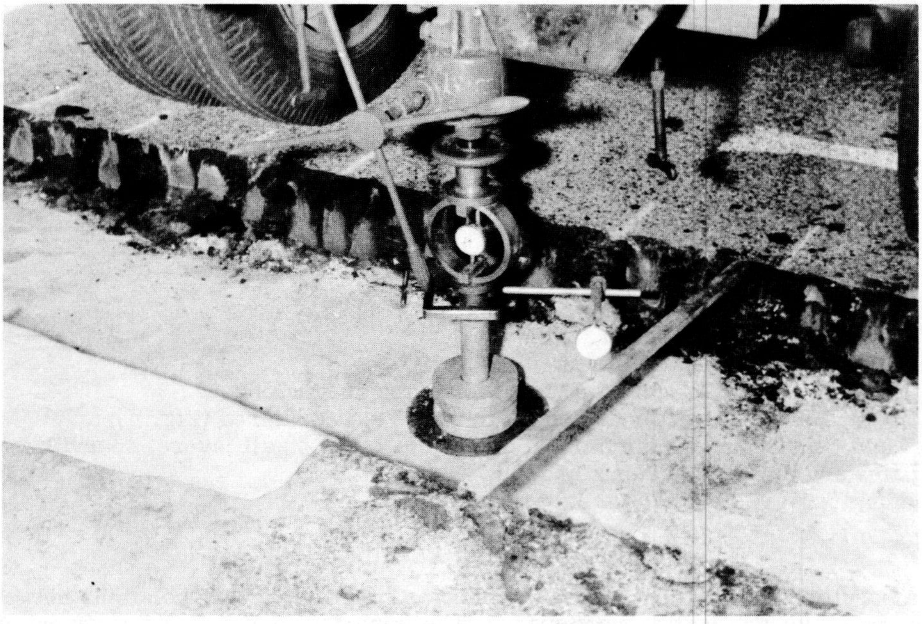


Figure 4. Field CBR test and density tests.

to the optimum laboratory moisture content as determined by modified AASHO methods. This same condition existed in the 1957 study, indicating that during the period of field testing, the base material did not reach a moisture content higher than optimum and was less than the moisture contents obtained when testing two-day capillary soaked CBR test specimens. Future field moisture content studies will be made during the wettest months to determine whether or not the field moisture content increases.

4. The field moisture content of the subbase material was generally equal to or less than the optimum laboratory moisture content. The field moisture was typically about 60 percent of laboratory optimum. This relationship was found to exist in previous studies and again indicates the need for future field moisture content studies to determine the most realistic moisture content for testing CBR specimens.

5. The field density beneath the outer wheel path was generally equal to the field density beneath the inner wheel path. This held true for both base and subbase.

6. The field moisture content beneath the outer wheel path was generally equal to the field moisture content beneath the inner wheel path. This was true again for both base and subbase.

7. The rutting in the outer wheel path was greater (50 to 150 percent) than the rutting in the inner wheel path. This was expected to occur on this system with roads constructed with stabilized soil shoulders only. Because the density and moisture showed no appreciable variation in the inner or outer wheel paths, it appears that the increased rutting in the outer wheel path may be attributed to shear-strain displacement. The shear resistance of the shoulder material and underlying native soil is undoubtedly less than the base and subbase and thus would lead to greater shear-strain displacement. Some evidence of displacement adjacent to the pavement section was noticed when the test sites were trenched. Extending the base course and surface would reduce outer-wheel-path rutting and essentially make the outer wheel path similar to the inner wheel path. The rutting beneath the inner wheel path was undoubtedly due to the compression of the base and subbase and to shear-strain displacement ("Special Shoulder Treatment to Alleviate Wheel Path Rutting," Research Bulletin 21, State Road Department of Florida).

CBR METHOD OF DESIGN

Major emphasis of the study was placed on investigating the CBR method of design or a necessary modification of this method. It was important to determine again if performance, service rating, and condition index could be related to an established design method or if modification of the method or design criteria would be necessary.

The CBR method studied was than originally proposed by O. J. Porter (5). The method has since been adopted in the original and revised forms by numerous states and the Corps of Engineers has done extensive work in reviewing and developing this method.

The CBR method basically consists of evaluating the bearing strength of a compacted laboratory specimen or field specimen, by penetrating the sample with a circular piston of 3 sq in. The stress obtained at 0.10-in. penetration is compared to a standard stress (1,000 psi). Prior to testing, the sample is normally subjected to soaking. The Corps of Engineers requires four days of soaking in order to simulate the worst possible field conditions. Other agencies require the same or less drastic soaking conditions. The State of Florida has found that a two-day capillary soak period would approximate most of the worst field conditions but that no soaking appears to be more typical of the average field conditions (1, 6). Both conditions of testing were used and studied by Florida and a complete testing procedure is in preparation. The desirability and advisability of soaking test specimens is discussed later.

EVALUATION OF PAVEMENT SECTION DESIGN

The flexible pavement sections investigated were typical of those constructed in 1947 and since. The over-all thickness of wearing surface, base, and subbase was generally greater than 17 in. and in most sections had a thickness of 19 in. The wearing surface varied generally from 0.5 to 2 in. in thickness, the base generally from 6 to 8 in., and the subbase from 10 to 12 in. The service ratings varied from 28 to 75. This variation included very poor through good performance. Excellent performance was not encountered on any of the test sections.

As mentioned, previous studies indicated that the original CBR curve for medium heavy traffic would generally define the thickness requirements and the strength index of flexible pavements constructed in Florida which have been subjected to 11 yr of traffic. A service rating of 60 divided good and poor performance, based on a limited number of test sections, the average ADT of the sections being 1,850 v.p.d.

The first analysis of the most recent survey dealt with the development of design curves based on CBR and service rating, service rating being the condition index adjusted for traffic volume. As a result of this analysis, a family of design curves was developed for thickness requirements as related to volume.

It was suspected that volume and equivalent wheel loads (EWL) were directly related in the State of Florida. The analysis on volume alone would be fairly reliable if volume and EWL were related.

The second analysis considered equivalent wheel loads (EWL) and condition index. Wheel load data were obtained (7) in July 1959, which permitted the analysis to be made.

Preliminary examination of the CBR data, the general thickness requirements, and the actual constructed thickness indicated that most failures could be attributed to either a weak base material, when wet, or an inadequate thickness of base for a weak (low CBR) subbase. Combined thicknesses of base and subbase appeared sufficient in all but one case to prevent failure of the subgrade. Analysis of the flexible pavement sections presented was made of the wearing surface, base material, subbase material, and subgrade.

Wearing Surface

The wearing surfaces encountered were of three major types: (1) surface treatment, (2) plant mix retread, and (3) asphaltic concrete. No major study was made of the wearing surface, however, it was possible to draw some general conclusions related to wearing surface.

Both surface treatments and asphaltic concrete surfacing gave satisfactory service although a somewhat higher percentage of surface-treated sections failed.

Surface treatments if properly maintained should be satisfactory. Lack of maintenance would undoubtedly lead to surface moisture penetration and weakening of the base. For thin surfacing, shear stress beneath the loaded area would definitely penetrate into the base material. This would warrant excellent base material of high shear resistance. The use of borderline, low-CBR base material should be avoided when the surface is of the surface-treatment type.

TABLE 2

WEARING SURFACE, MINIMUM REQUIREMENTS[1]

2 Lane ADT	Type of Surface
300 - 1,000	Double surface treatment
1,000 - 3,000	Triple surface treatment
3,000 - 4,000	Asphaltic concrete, 2½ in.
4,000 - 6,000	Asphaltic concrete, 3 in.
Over 6,000	Asphaltic concrete, 3½ in.

[1] See Ref. (1).

Asphaltic concrete surface sections gave good service and generally were from 1.5 to 2.0 in. thick. Available information indicates the use of 2 to 3 in. as being desirable. The use of asphaltic concrete surfacing for roads with a predicted ADT of 3,000 v.p.d. or greater seems advisable and was found to exist on the sections studied. Table 2 gives the minimum wearing-surface requirements.

Base Course

The major base-course material studied was limerock. This is the most common material used in the state, although sand clay, clay, and shell mixtures are used as base materials. The main purpose of the base course is to distribute the wheel load to the underlying layer. It should not compress excessively, should withstand the shear stress imposed by the wheel load, and should be stable under all degrees of field moisture. Hard limerock, compacted to high density, meets the requirements of good to excellent base material.

A minimum laboratory four-day-soak CBR of 80 and in some cases as low as 60, at 95 percent maximum density, is required by many agencies. Complete submergence of the specimen and a four-day soak period is a severe test. The soak test performed by the State of Florida is not complete submergence, but a two-day capillary soaking with a head of water equal to the height of the specimen. This may also be considered somewhat severe inasmuch as the resulting moisture content of the test specimen is generally slightly greater than the field moisture encountered to date. It is expected, however, that future field moisture data will show that the two-day capillary soak period is quite realistic for Florida soils and climate.

The minimum field CBR value normally required for base materials is 80. Again moisture content is extremely important when obtaining the minimum bearing value in the field. The field tests performed in this study were run between 8 and 12 percent field moisture and may not be the maximum values; however, it is certain that they are fairly realistic because the base study performed earlier (6) gave similar results. The general range of CBR values obtained is given in Table 3 (a). Table 3 (b) presents the range of CBR values obtained as a result of earlier base course material studies (6).

The data in Table 3 show that both the CBR at optimum and the field CBR met the requirements for good base material. Some soak CBR values are low; however, these samples were tested at moisture contents slightly greater than those occurring in the field. For analysis it was assumed that the base material was sufficiently strong to prevent base failure where the soak CBR was greater than 40.

The fact that some limerocks lose strength rapidly with soaking cannot be overlooked. Poor performance did occur in some test sites where the CBR (soak) was less than 40. A combination of poor surface treatment and a soak CBR of less than 40 was accompanied by a low service rating in two test locations and it is probable that poor base material did contribute to the failure of the test section. For the same test locations, however, the subbase strength or the base thickness was inadequate, which would also lead to failure. It is, therefore, difficult to attribute all of the poor performance to poor base material. No test locations were encountered where adequate thickness of poor

TABLE 3

RANGE OF CBR VALUES AT 0.10-IN. PENETRATION

CBR Laboratory (Opt. Moist.)[1]	CBR Laboratory (2 Day Soak)	CBR (Field)
	(a) 1959 Study	
Limerock		
60 to 150	30 to 130	90 to 200
	(b) 1958 Study	
Ocala limerock		
31 to 175	27 to 140	60 to 204
Miami oolite		
45 to 228	30 to 145	95 to 290
Sand-clay		
10 to 75	15 to 100[2]	23 to 192
Shell		
55 to 65	38 to 58	35 to 124
Shell-sand		
15 to 90	15 to 90	34 to 130

[1] CBR Laboratory: optimum moisture or optimum as used in this report pertains to CBR test samples compacted at optimum moisture and maximum density obtained by compacting five equal layers at 55 blows per layer with a 10-lb hammer and 18-in. drop in a CBR mold having a volume of about 0.10 cu ft.

[2] Indicates drying of surface of sample. Corrected in later tests (9).

base material existed; nevertheless, it appears that all base materials should have a definite minimum soak CBR of 40 and a desirable lower limit of 60. It also is suggested that where poor base material, soak CBR 40 to 60, must be used, additional thickness of wearing surface should be used. Specific values of CBR for different classes of traffic are noted later.

Subbase Course

Preliminary examination indicated that the low service ratings could be attributed to low strength of the subbase or inadequate thickness of base. Both of these terms may be used interchangeably because when designing the flexible pavement, base material may replace subbase. However, at any given depth beneath the surface, the subbase, if used, must have a minimum bearing value.

The technique used to correlate service rating and design curve was to plot actual thickness versus required thickness for a CBR value and note for each test site the service rating. Sites where the actual thickness is greater than the required thickness should have high service ratings. The opposite should result in low service ratings. The line of equality (actual thickness equals required thickness) should divide the good performance sites from the poor.

Actual thickness was obtained from field measurements, required thickness from the medium heavy traffic design curve. Having plotted the data it was possible to adjust the required thickness so that a better correlation resulted. This was done for all studies where desirable.

Another method of evaluating the data is to plot actual thickness against CBR, on a semi-logarithmic plot. When service rating is plotted a curve may be drawn which best fits the data and this then results in the design curve. Either method would obtain essentially the same end result.

The use of service rating numbers permits a fairly accurate analysis of the data. The original analysis included actual service-rating numbers. A later report grouped service rating as follows:

Grouping of Service Rating (SR)

A. Test sections—Service rating 30 to 59—Definitely poor performance
B. Test sections—Service rating 60 to 65—Questionable performance
C. Test sections—Service rating 66 to 75—Definitely good performance

Analyses were made of the field CBR, laboratory CBR at optimum moisture, and laboratory CBR after two-day capillary soaking. The subbase materials tested were principally A-3 and A-2-4 soils. The following conclusions resulted: Considering the field CBR, the use of the original California curve would predict about 38 percent of the failures. Adjusting the design curve by adding 0.5 in. to the required thickness would predict 50 percent of the failures. Adding about 4 in. would predict all failures.

Considering the laboratory CBR at optimum the use of the original California curve would predict about 56 percent of the failures. Adjusting the design curve by adding 0.1 in. would predict 67 percent of the failures. The use of 0.1 in. is not intended to illustrate accuracy of a design method but merely to show the small magnitude of adjustment required. Adjusting the design curve by adding about 2 in. would predict all failures.

Considering the laboratory CBR when a soaked specimen was tested, the use of the original design curve would predict 89 percent of the failures. Adding about 0.5 in. would predict all failures. The use of the soak CBR and the adjusted design curve would predict all failures but would result in slight overdesign for 8 out of 28 test sites.

As a result of the subbase study it was found that all of the failures could be attributed to inadequate thickness of base or insufficient bearing strength of the subbase. This conclusion could be drawn inasmuch as the actual total thickness of surface, base and subbase exceeded the required thickness, as obtained from design curves for wheel loads as high as 20,000 lb. Actual findings are noted in the discussion of the subgrade.

The CBR of a soaked sample would certainly be the best criterion for predicting failures but a maximum number of overdesigned sections would result. However, it appears that the use of the soak CBR test is desirable as a control test until a field moisture content study is completed which will cover all sections of the state and includes the various wet seasons. The use of the soak CBR for design will give general assurance of satisfactory performance and should be used until additional moisture data are obtained.

Subgrade

Flexible pavements distribute wheel loads from the surface through the underlying layers to the subgrade. The normal stress and maximum shear stress are at or near the surface and diminish with depth. Inasmuch as this type of stress distribution exists, it is logical to develop a pavement section having layers which decrease in strength with depth. This could be simply stated in terms of CBR—the CBR of the base should be greater than the CBR of the subbase which in turn should be greater than the CBR of the subgrade. This may be termed a normal flexible pavement section. The review of the subgrade was concerned principally with normal pavement sections. The subgrade materials were principally A-3 soils although a few A-2-4 soils were encountered.

Because the failures could be attributed to poor subbase or inadequate thickness of base, analysis of the subgrade or combined thickness of surface, base, and subbase should result in all sections having adequate required thickness. This was found to be true when actual total thickness was compared to required thickness.

Analysis based on the field CBR and service rating indicated that all sections had adequate thickness of material above the subgrade. A reduction of the actual design thickness of about 3 in. would give a better correlation.

Analysis based on the CBR at optimum moisture also indicated adequate thickness above the subgrade. A reduction of the actual design thickness of about 1 in. would improve the correlation.

Analysis based on the CBR when a soak specimen was tested indicated adequate thickness and very little reduction of design thickness was indicated.

As a result of the subgrade study it was found that sufficient total thickness of surface, base, and subbase existed in all test sites. The use of the original CBR design curve, and the two-day capillary soak test, showed the best correlation.

Design Curve

Figure 5 shows the corrections resulting from the various studies, the general limits of the study, and the resulting design curves. Additions and reductions noted previously are shown, as well as the CBR ranges investigated.

The design curve for the average of all test sites based on service rating is shown in Figure 6. This curve is for an ADT of 2,070 and a service rating of 60. The two-day capillary soak CBR, which was found to be the most realistic for predicting performance, is used. All flexible pavement sections designed on this basis could be expected to have a service rating of 60 or more for an ADT of 2,070 after 11 yr of traffic. The condition index would also be equal to 60 for this traffic volume.

Design Curves Related to Volume

Service rating was related to condition index and volume by

$$R = c + \frac{100\,c - c^2\,(\log \frac{T}{T_s})}{K} \tag{1}$$

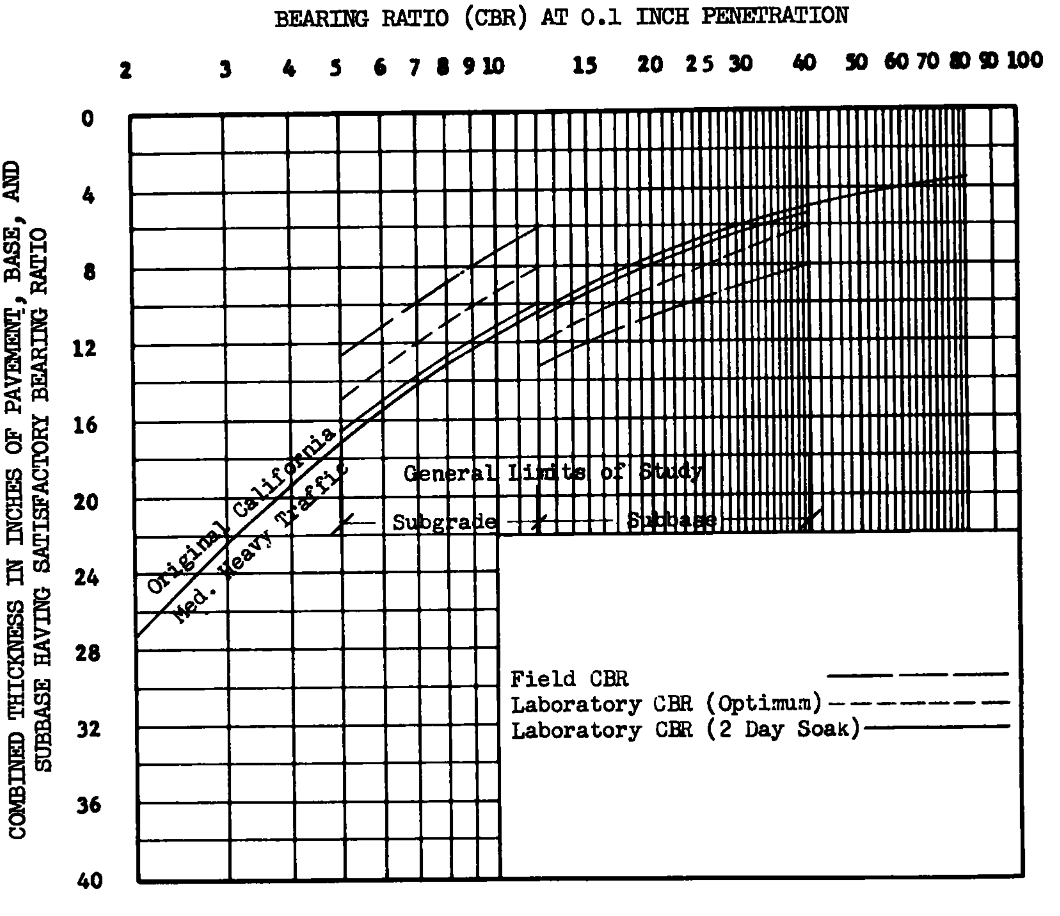


Figure 5. Adjusted design curves based on service rating for the average volume of the system.

and

$$K = \frac{100\,c - c^2}{50\,(\log T_s - \log T_m)} \tag{2}$$

in which

R = service rating;
c = condition index;
T = ADT on section;
T_s = ADT on system; and
T_m = minimum ADT = 50.

For a condition index of 60 and an average ADT of 2,070, K = 29.7 and

$$R = c + \frac{100\,c - c^2\,(\log \frac{T}{T_s})}{29.7} \tag{3}$$

Examination of service-rating and thickness data indicated that a definite trend resulted. When the actual thickness was greater than the required thickness the service rating increased. When the actual thickness was less than the required thickness the service rating decreased. Inspection of these trends led to the developments of service rating limits (Fig. 7a). A service-rating value of 60 is the dividing line between good and poor performance and has an ADT of 2,070. The condition index for this volume is also 60. Using Eq. 3 volumes were computed for the service-rating limits and a condition index of 60. Having calculated the volume, a family of design curves resulted.

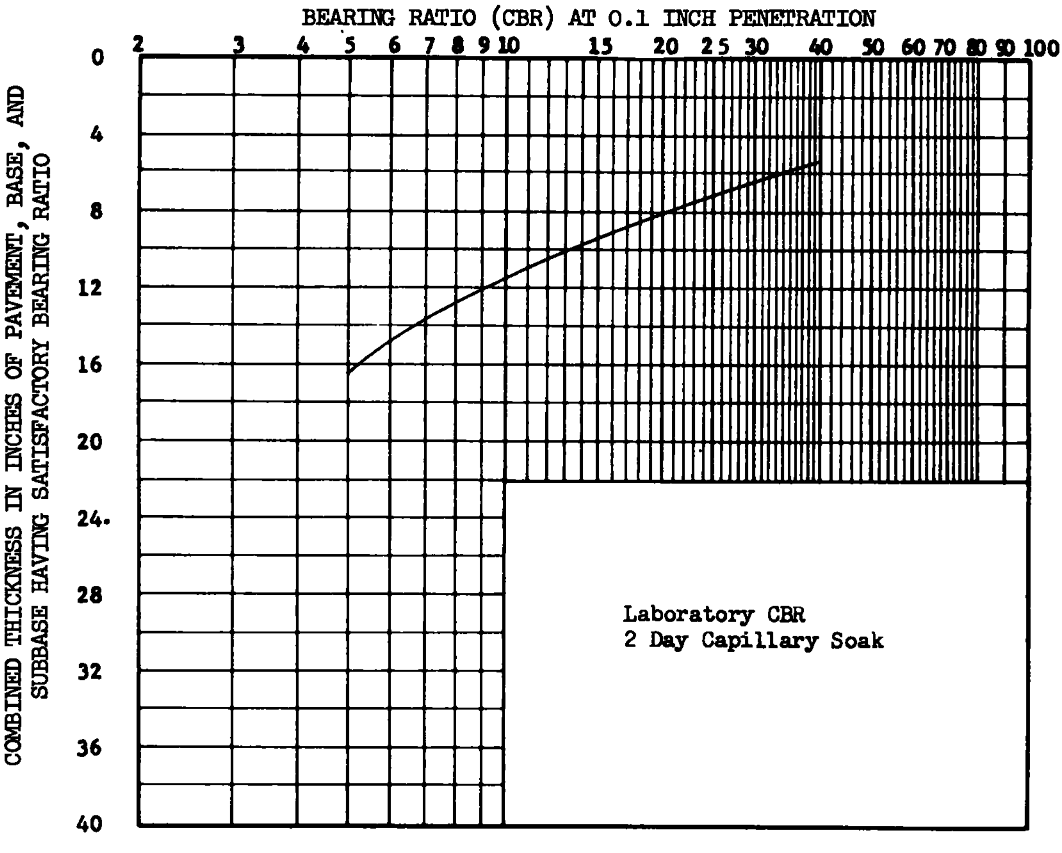


Figure 6. Design curve based on service rating for the average volume of the system (Volume 2,070).

The design curve for a volume of 2,070 and a condition index of 60 would require a service rating of 60 and would be the original curve developed. A design curve for a volume of 6,600 and a condition index of 60 would require a service rating of 75. The service-rating limit of 75 developed from the data would require an additional thickness of 4 in. A design curve for a volume of 440 and a condition index of 60 would require a service rating of 40. This would result in a reduction of the thickness by 3 in. Figure 7b is a graph of required thickness and corresponding volume for a condition index of 60.

Using the information in Figures 7a and 7b, a family of design curves was prepared (Fig. 8).

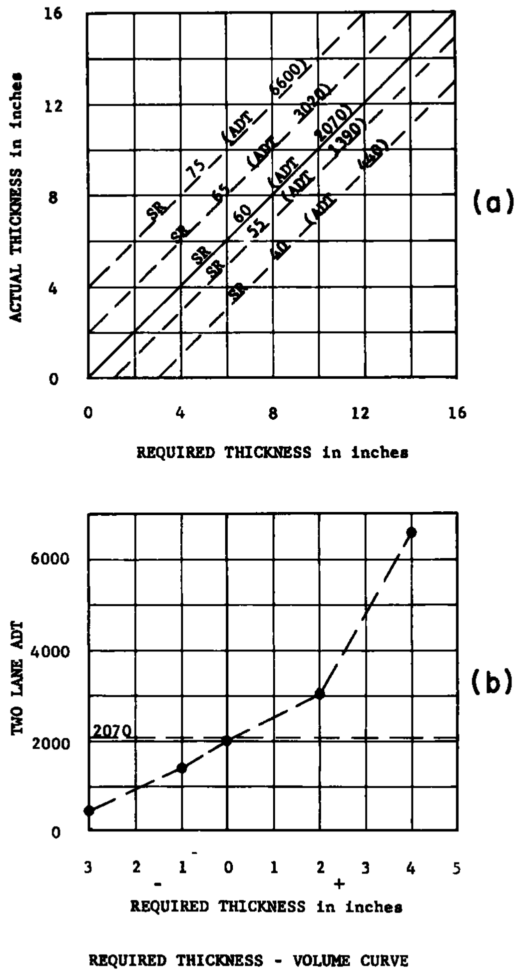


Figure 7. Service rating—volume curves for a condition index of 60.

Design Curves Related to Equivalent Wheel Load

Preliminary work in 1957 indicated that equivalent wheel loads may be directly related to volume. Since then the Traffic and Planning Division has performed an analysis relating volume, a lane ADT, and equivalent wheel loads, and the results have been used for the following correlation. Table 4 gives the important data used in the development of the design curves based on EWL and includes such information as test site, class of traffic, average ADT, total EWL, condition index, and service rating. The data indicate that EWL is almost directly related to volume and that class of traffic generally defines the range of EWL.

For the analysis of equivalent wheel loads and the development of design curves three ranges of EWL were studied. These were as follows:

1. EWL range 5 to 15 x 10^6; total for 11 yr, 2 lanes.
2. EWL range 19 to 33 x 10^6; total for 11 yr, 2 lanes.
3. EWL range 36 to 60 x 10^6; total for 11 yr, 2 lanes.

The 19 to 33 x 10^6 range of wheel loads included the maximum number of test sites—thirteen—and had an average ADT of 2,321 v.p.d. The average volume of the system was 2,070 v.p.d.

The technique of analysis was to plot actual thickness versus CBR on a semi-logarithmic plot and note the condition index. EWL includes the effect of volume, therefore service rating was not used for correlation. Using this presentation, a design curve which best fitted the data could be drawn directly. No adjustment would be necessary as when analyzing actual and required thickness. A similar analysis was used by Kentucky (8).

For each of the three EWL ranges, design curves have been developed for the field CBR, the laboratory CBR at optimum, and the laboratory CBR when tested after soaking.

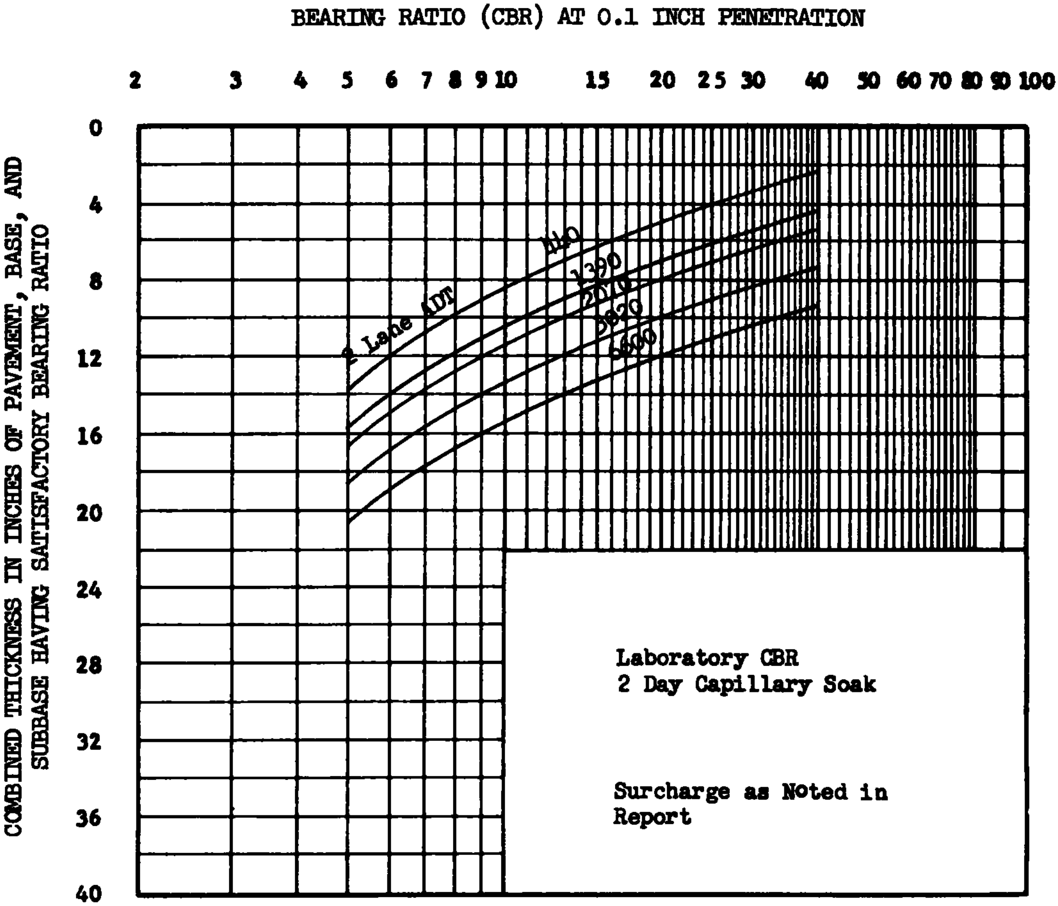


Figure 8. Design curves developed from service rating zones for a condition index of 60.

Figure 9 shows the design curve for the three EWL ranges and the CBR test conditions.

The design curves (as noted previously) resulted from the study of three EWL ranges with a limited amount of data. It follows that the curves are, at best, good estimates of the thickness requirements. The curve for an EWL range of 5 to 15 x 10^6 was developed from eight test sites where three of the eight sites failed. The design curve based on the EWL range of 19 to 33 x 10^6 was developed from thirteen test sites. Although a scattering of points resulted, the design curves should be fairly realistic. The design curves resulting from a study of the EWL range of 36 to 60 x 10^6 were developed from six failed sections. Inasmuch as a number of the sections had condition index numbers of 60 to 65 the design curves were placed immediately beneath the lowest points.

Because complete traffic data were available for the test sites, it was possible to calculate the average two-lane ADT from the EWL groups. The volumes are given in Table 5.

Comparison of Design Curves

Figure 10 shows the design curves which resulted when the service-rating curves were adjusted to volumes corresponding to the actual average volumes obtained from the equivalent wheel-load study (Table 6). The three adjusted curves were superimposed on the results of the EWL study.

TABLE 4

EQUIVALENT WHEEL LOADS AND RELATIVE DATA

Traffic Class	Test Site Number	Total Equivalent 5,000-lb Wheel Loads 1948 - 1958	Average 2 Lane Volume—ADT	Condition Index	Service Rating T_S=1,850
1	51	904,006	125	75	47
2	55	4,357,709	396	79	65
2	84	7,210,139	400	56	35
3	70	7,493,889	725	76	67
3	71	7,493,889	725	68	57
4	86	13,359,667	800	68	58
4	82	6,004,561	824	82	75
4	83	15,566,607	870	38	28
4	78	13,847,791	1,190	48	42
5	68	19,343,560	1,535	68	66
5	87	19,343,560	1,535	71	69
5	72	27,878,509	1,892	76	76
5	73	27,878,509	1,892	70	70
5	65	21,234,307	1,948	59	60
5	66	24,329,003	2,088	48	50
6	67	28,491,134	2,378	40	43
6	62	33,173,391	2,685	41	46
6	63	33,173,391	2,685	71	75
6	81	30,667,717	2,743	70	75
6	85	29,385,712	2,743	79	83
6	56	26,542,419	2,812	64	69
7	58	31,378,346	3,220	60	67
7	64	41,365,061	3,221	60	67
7	54	42,845,530	3,248	65	72
7	79	38,950,933	3,420	57	65
7	80	41,094,096	3,616	66	74
7	57	36,670,335	3,648	54	63
7	77	60,644,152	4,568	57	69

The design curves are for the laboratory CBR of the two-day capillary soak specimens. Although the curves are not identical, close agreement did exist. It can also be seen that the shape of the original CBR design curve was generally duplicated.

The design curves (Fig. 11) have been adjusted to take into consideration the results of both the service rating and EWL studies. The curves have been extended to cover the range of CBR values normally encountered in Florida.

Current Studies

Additional work is currently being done on the development of a semi-empirical method of design. This method will consider in addition to the CBR, tests with larger plates, subgrade modulus, and the Burmister layered theory. Tests are in progress to determine seasonal variation in moisture contents of the pavement section, plate bearing tests of the layers and deflections in the pavements under load as measured with the Benkelman beam.

PROPOSED DESIGN CURVES AND SECTION DESIGN

The flexible pavement design curves proposed for the State of Florida are shown in Figure 11. The curves are a result of the service rating and equivalent 5,000-lb wheel-load design studies completed to date. The design curves are related to volume

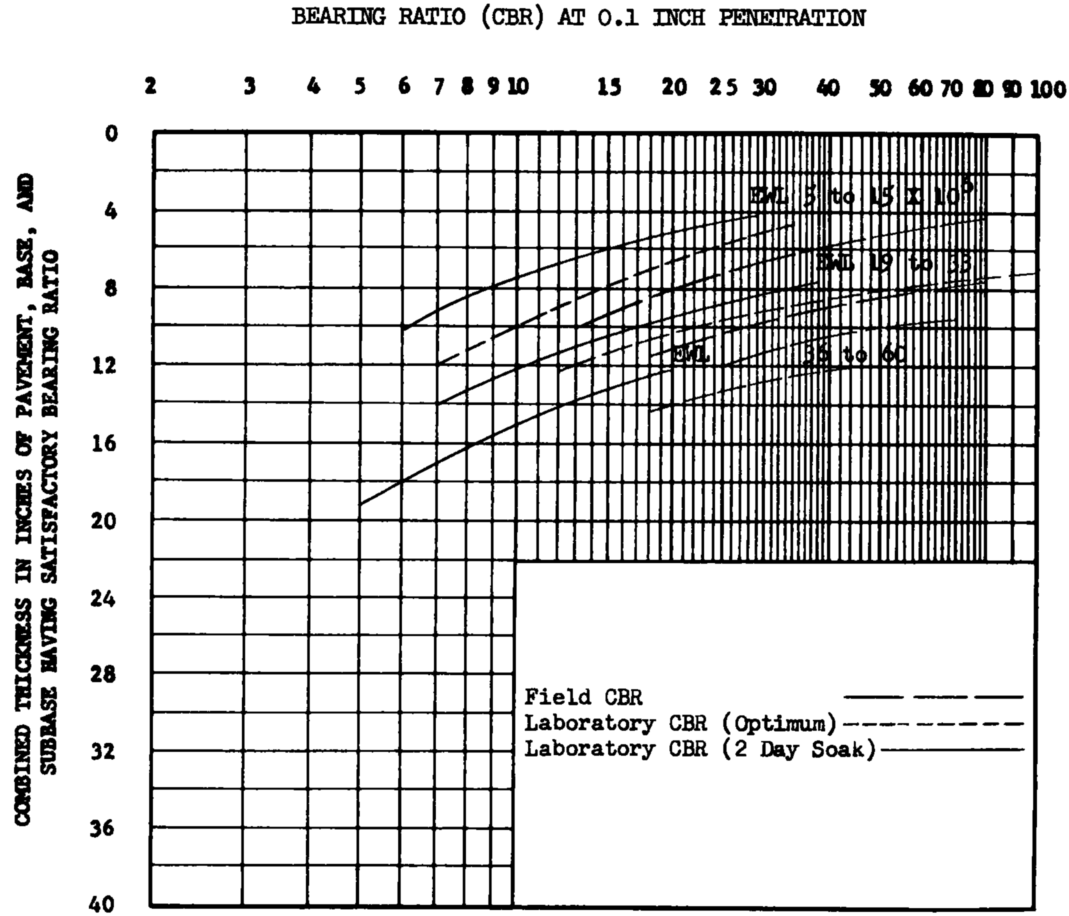


Figure 9. CBR design curves developed from EWL groups.

TABLE 5

AVERAGE ADT, 2 LANE VOLUME, FOR EACH EWL RANGE

EWL Range	Average ADT
5 to 15 x 10^6	742
19 to 33 x 10^6	2,321
36 to 60 x 10^6	3,622

(10-yr, 2-lane ADT) and equivalent wheel load (10-yr, 5,000-lb wheel loads for 2 lanes). Average daily traffic (ADT) and EWL data were determined by the Traffic and Planning Division (10) for the flexible pavement design study and can be estimated by that division for any proposed highway. The design curves include four major traffic groups. Three of these groups are the direct result of the analysis of performance data, the fourth curve has been extrapolated from the resulting original curves. The design curves are for the estimated 10-yr traffic conditions given in Table 6.

The proper design curve for any given traffic condition, considering both volume and EWL would be the curve indicating the maximum thickness. As an example, if the estimated volume were 2,000 v.p.d. and the estimated EWL was 38 x 10^6, this would result in the use of design curve C for heavy traffic even though the volume indicated medium traffic. After selection of the proper design curve, the design of the section would follow normal flexible-pavement design principles.

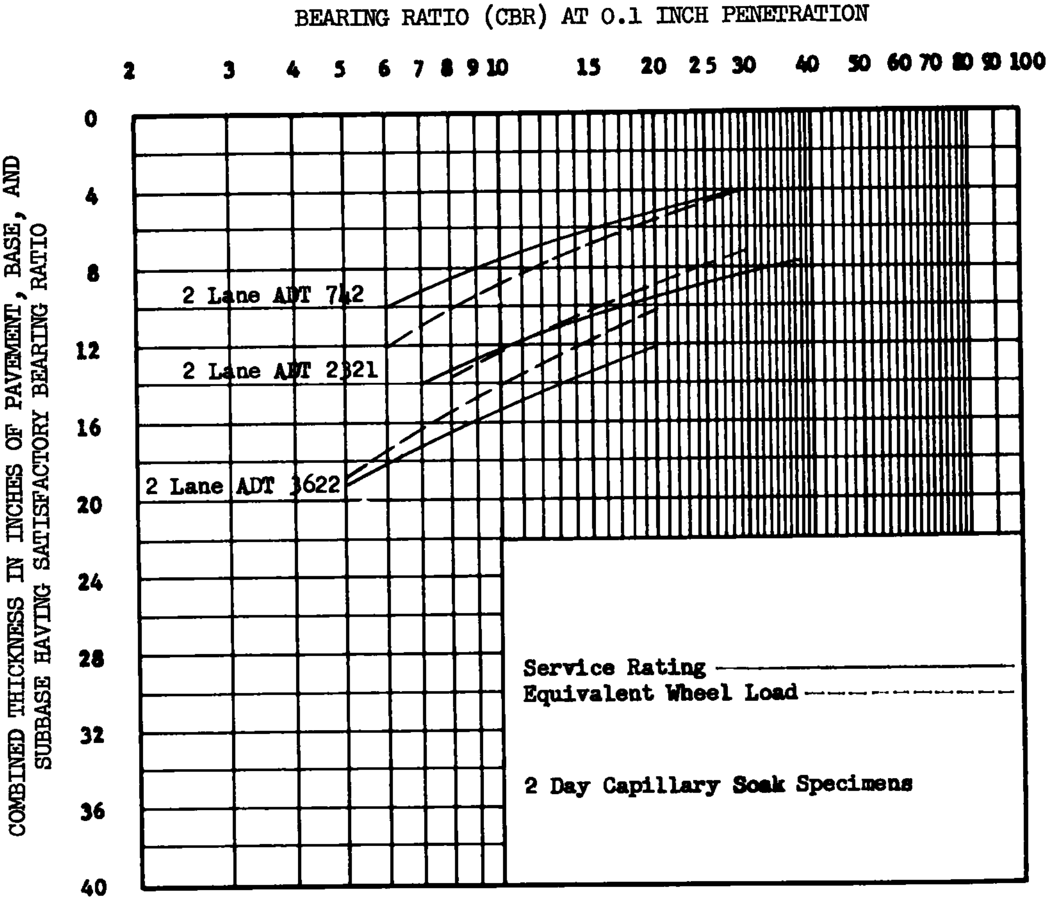


Figure 10. Comparison of design curves developed from service rating and EWL.

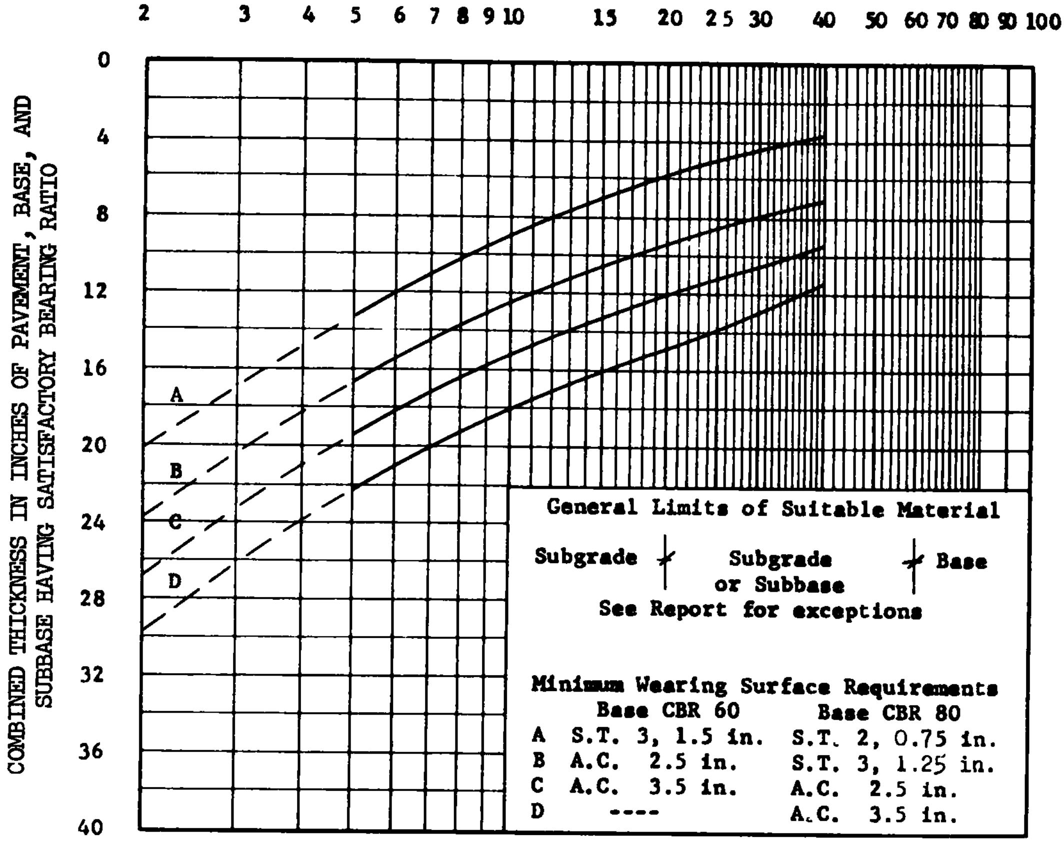
Surface Requirements

The wearing surface requirements are noted again in Table 7. The requirements are discussed briefly in this report and no further comments are added.

Base-Course Material Requirements

The requirements of the base material are given in Table 8. These are the minimum CBR requirements based on the flexible pavement study and base studies performed by Florida. Other state requirements such as chemical analysis, gradation, plasticity, and compaction should be adhered to.

The aforementioned requirements are for all materials used as base-course materials. The major flexible-pavement design study reported herein dealt with limerock base-course pavements; however, preliminary work was done with sand clay and other commonly used base-course materials (6). The CBR requirements were found to be generally the same. The use of a sand-clay base-course material having a two-day soak of 40 for light traffic should be avoided. Sand clay will continue to gain in moisture content beyond the two-day period with a resulting significant loss in bearing. Sand clay base materials should have a minimum bearing value of 60 for light, medium, and heavy traffic.



CLASSIFICATION OF TRAFFIC

CLASS	2 LANE ADT Estimated 10 Year	EWL GROUP(5000#) Estimated 10 Year Total
A. LIGHT	500 to 1500	5 to 15 X 10^6
B. MEDIUM	1500 to 3000	16 to 35 X 10^6
C. HEAVY	3000 to 4000	36 to 50 X 10^6
D. VERY HEAVY	4000 to 6000	51 to 80 X 10^6

Figure 11. Flexible pavement design curves—CBR in percent at 0.1-in. penetration, 2-day capillary soak specimens; compacted at optimum moisture and maximum density (modified AASHO surcharge as noted in report).

Subbase Requirements

The minimum required subbase CBR should be at least 20. Since additional thickness of surface and acceptable base course material can be substituted for inadequate bearing value subbase material, the lower limit cannot be established accurately; however, the lower limit of 20 should be used. Desirable subbase material should have a CBR of about 30. All subbase material should be compacted at optimum moisture and maximum density and tested after a two-day capillary soak period. A surcharge of 15 lb should be used. All other state specifications pertaining to grain size and plasticity requirements should be adhered to.

TABLE 6

DESIGN CURVE AND TRAFFIC DATA

Design Curves	Class of Traffic	Volume[1] 2 Lane ADT (11-yr tot.)	EWL 5,000 lb[1] 2 Lane (11-yr tot.)	Design Volume 2 Lane ADT (10-yr tot.)	Design EWL 5,000 lb 2 Lane (10-yr tot.)
A	Light	742	5 to 15 x 10^6	500 to 1,500 750	5 to 15 x 10^6
B	Medium	2,321	19 to 33 x 10^6	1,500 to 3,000 2,000	16 to 35 x 10^6
C	Heavy	3,622	36 to 60 x 10^6	3,000 to 4,000 3,500	36 to 50 x 10^6
D	Very heavy			4,000 to 6,000 5,000	51 to 80 x 10^6

[1] Actual data from flexible pavement design study.

TABLE 7

MINIMUM RECOMMENDED WEARING SURFACE TYPES AND APPROXIMATE THICKNESS

Class of Traffic	Type of Wearing Surface	Thickness (in.)
Light	Double surface treatment (S.T. 2)	0.75
Medium	Triple surface treatment (S.T. 3)	1.25
Heavy	Asphaltic concrete (binder + surface)	2.5 to 3.5
Very heavy	Asphaltic concrete (binder + surface)	3.5 to 4.0

TABLE 8

BASE MATERIAL REQUIREMENT (CBR)

Class of Traffic	CBR of Base Material	Thickness
Light	(40)[1] 60	As shown on design curves
Medium	(60)[2] 80	
Heavy	(60)[2] 80	
Very heavy	80	

[1] When base material with a CBR of 40 must be used, additional thickness of wearing surface should be provided. A CBR of 40 is permissible only for light traffic.

[2] Where base material with a CBR of 60 must be used, additional thickness of wearing surface should be provided.

Note: CBR determined by testing a sample compacted at optimum moisture and maximum density (Modified AASHO) and tested without surcharge after two days of capillary soaking.

Subgrade Requirements

No minimum subgrade CBR is required. The laboratory CBR should be established by generally duplicating field compaction of the subgrade. The maximum subgrade CBR should be established by compacting specimens at optimum moisture and maximum density and testing after a two-day capillary soak period. A surcharge of 20 lb should be used. All other state requirements pertaining to suitability and compaction should be adhered to.

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Flexible Pavement Research in Virginia

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This paper reviews developments in flexible pavement design in Virginia since the report by Woodson at the 1954 meeting of the Highway Research Board. The trend away from macadam toward dense-graded aggregate and finally to "black base" in flexible base construction is traced. Several notable failures which led to the abandonment of selective grading and the substitution of select borrow foundation courses are described. Mention is made of experimental construction involving cement- and lime-treated subgrades; more recently this feature has been incorporated into a number of projects where suitable select borrow has not been available locally.

Because recent trends have in most cases tended toward drastic increases in construction costs, Virginia's Highway Research Council has persistently sought to develop means of minimizing these increases. The construction of experimental projects as part of the regular construction program in order to compare the performance of a variety of pavement designs under the same traffic conditions has been proposed, and one such project has been built. The principal purpose of this paper is to describe the program of experimental construction, the methods of evaluation used, and the application of these methods to the first experimental project.

●WOODSON (1) described in 1954 Virgina's newly adopted modified CBR method of designing flexible pavements. A year later Maner (2), who had succeeded Woodson as Assistant Engineer of Tests, carried the description a bit further in a prepared discussion of a paper by Hveem. The method is still used in essentially the same form (3), but only to provide a guide with regard to total pavement thickness required to prevent overstressing the subgrade. The composition of the pavement, the type and thickness of the various layers making up this total thickness, is not determined by test, although in theory the CBR method could be used to establish the greater portion of the design. For example, suppose that the basement soil for a given project is expected to have a CBR value of 6, from which the total structural thickness might be established as 19 in. This thickness could be made up of, first, a 6-in. layer of material with a minimum CBR value of 13, then a 5-in. layer with a minimum CBR value of 30, and, finally, 8 in. of some combination of high-type base and surfacing materials. The make-up of at least this top 8 in., and usually the entire 19 in. in actual practice, reflects the preference of the designer on the basis of his experience and judgment.

It is far from the purpose of this paper to question the judgment of the persons who make these decisions. Instead, its purpose is to describe certain research which has been undertaken to broaden their experience. But first, a brief review of the recent history of flexible pavement performance in Virginia and of the evolution of present design practices seems in order.

Ten years ago, most heavy-duty flexible pavement designs incorporated water-bound macadam as the base course, with bituminous treatments of the mixed in place or penetration types usually not totaling more than 3 in. in thickness. But this type of construction is slow and requires skilled workmen, and macadam seldom appears now in Virginia's bidding proposals.

Envisioned as the successor to macadam, the pug-mill mixed dense-graded aggregate base was introduced to Virginia in 1953. Three of the major aggregate producers promptly equipped themselves to made this material, and a number of pavement designs for projects in the vicinity of these plants included graded aggregate as the base course.

Figure 1 represents one such design for a project advertised for bids in 1955 and built the following year to carry traffic which at the latest published count included 875 trailer trucks and buses in a total daily volume of 4,294 vehicles.

The 12-in. graded aggregate base here had an asphaltic concrete surface approximately 2½ in. thick. The balance of the typical section consisted of select borrow whose only requirement was that it have a minimum laboratory soaked CBR value of 12. The practice of attempting to obtain this select material from within the limits of regular excavation by the method known as selective grading has been largely discontinued. The difficulty seemed to lie in distinguishing the select soils from those not so select, and more recently all types of select material have been obtained from sources outside the project.

Trench-type construction also has practically disappeared on projects of major importance, and some type of select material is used from ditch to ditch. For this reason, the system of computing pavement construction costs in terms of the square yard has been abandoned in favor of one which considers the cost per lineal foot. In this manner the cost of all materials above the top of the earthwork is included. In the case illustrated, the cost per lineal foot was $9.16.

Unfortunately, Virginia's luck with dense-graded aggregate pavements was not as good as that of North Carolina, from whom the idea for this type of construction was borrowed. Of the first 17 major projects built, all have developed at least minor distress, ten have cracked badly, and two required either partial or complete resurfacing within six months after completion. All told, seven projects required some resurfacing in less than three years.

Figure 2 shows the condition of one such pavement only a month after it was opened to traffic. This is the same pavement whose design was described in Figure 1. Patching of this badly cracked surface along with a complete plant mix resurfacing applied the first summer, added $2.12 per lineal foot to the investment here before the pavement was eight months old. An investigation by the Virginia Council of Highway Investigation and Research revealed that deflections on this pavement under an 18,000-lb axle load averaged 0.069 in., as measured with the Benkelman beam. Specific causes for these high deflections could not be determined.

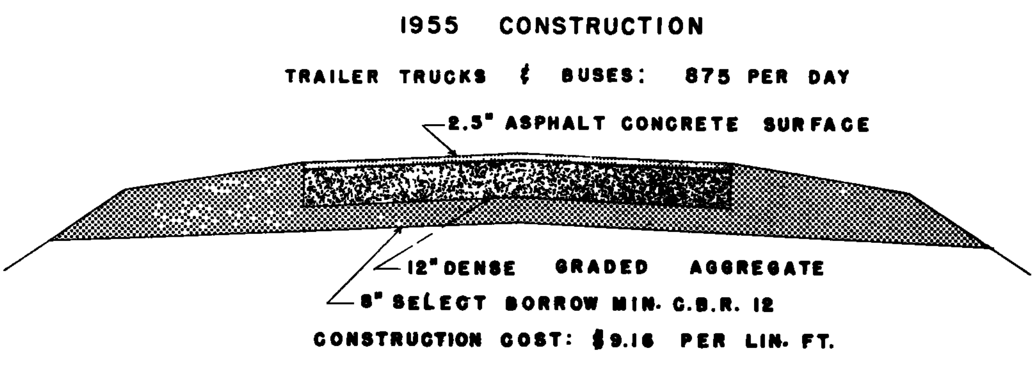


Figure 1. Typical pavement section for 1955 construction.

Figure 2. Cracking of pavement described in Figure 1.

It was plain that something had to be done to prevent repetitions of this failure. Because there was no definite evidence that construction had not met specifications, it was decided that future designs should be made stouter.

Figure 3 shows the extent to which the design of a pavement to carry traffic similar to that carried by the previously mentioned one was "beefed up" only two years later. (Actually this project does not carry as many heavy vehicles as the first one.) The CBR requirement for select borrow on this project is 20 and the total thickness of asphaltic concrete is 9½ in. instead of the corresponding 12 CBR and 2½-in. asphaltic concrete thickness for the earlier project. These and other minor changes brought the total cost to $17.33 per lineal foot, a jump of 89 percent over the cost of the lighter design.

The greatest single factor influencing this cost rise was the adoption of asphaltic concrete as the base material. Black base, to use the colloquial term, was introduced in Virginia about 10 years ago in an area where it could be produced relatively cheaply from local pit-run aggregates, an area also where crushed stone was non-existent.

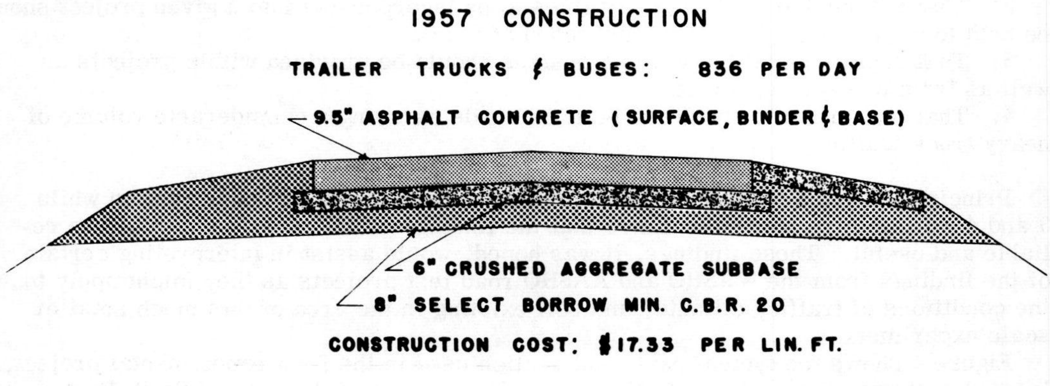


Figure 3. Typical pavement section for 1957 construction.

Its success was readily apparent, and soon black base found its way into designs for certain selected pavements in all parts of the state, though often at quite a considerable increase in cost.

The next most costly change was the requirement of increased CBR value along with certain gradation and plasticity limits for the select borrow. These requirements have largely ruled out roadside borrow pits, and on some still more recent projects have necessitated the use of commercially crushed aggregates. The unit cost of the CBR 20 select borrow in Figure 3 was almost four times that of the CBR 12 material in Figure 1.

The Virginia Council of Highway Investigation and Research when formed in 1948, had established as its first objective the carrying out of "research programs for the purpose of facilitating the economic design, construction, and maintenance of highways." In light of this stated objective, it became evident that special effort should be exerted to develop, through research, methods of minimizing such increases in construction costs as have just been described.

One of the first projects proposed toward this end involved studies of the benefits which might be realized through stabilization of subgrades and bases, as practiced in a number of western states. Soil-cement bases, with relatively high percentages of cement being added to soils occurring naturally in the roadway, had been built for many years in Virginia, but the idea of adding lower percentages of cement or even of hydrated lime to subgrade soils beneath heavy-duty pavements was relatively new. From the studies at the Research Council laboratories and from several successful experimental installations in the field (4), the incorporation of cement or lime treatments of subgrades into pavement designs is fast gaining favor. Cement and lime-flyash treatments of aggregate base materials have been quite successful also, and this type of construction is gradually winning acceptance.

But the most positive step has been the decision to include experimental projects as part of standard construction for the purpose of comparing the performance of a number of different pavement combinations built at the same time and subjected to identical conditions of weather and traffic. The intent of the balance of this paper is to show that projects of this type can be constructed at little if any added expense and can facilitate the gathering of a great deal of valuable data.

DESIGN AND CONSTRUCTION OF EXPERIMENTAL PAVEMENTS

Following the recommendation of T. E. Shelburne, Director of Research, the Virginia Department of Highways in 1957 created a four-man Research Advisory Subcommittee to assist the Research Council in outlining a program of experimental construction. At one of its earliest meetings, this subcommittee formulated certain basic principles to be considered in designing such experimental projects. These were:

1. That projects should be constructed to standard specifications with conventional equipment.
2. That the number of design variables to be incorporated into a given project should be held to a minimum (in the neighborhood of four).
3. That replication of the various designs should be provided within projects as well as from project to project.
4. That test projects should be built on roads carrying a considerable volume of heavy truck traffic.

Principles 1 and 2 were designed to minimize complexities in construction while 3 and 4 were to give some assurance that the findings of the experiment would be reliable and useful. These findings, it was hoped, would assist in interpreting certain of the findings from the WASHO and AASHO road test projects as they might apply to the conditions of traffic, climate, and soil existing in the area of this much smaller scale experiment.

Figure 4 shows the typical pavement section used in the first experimental project, located in the westbound lane of US 58 immediately west of the town of South Boston. In this section of Virginia, United States Weather Bureau normal temperatures range

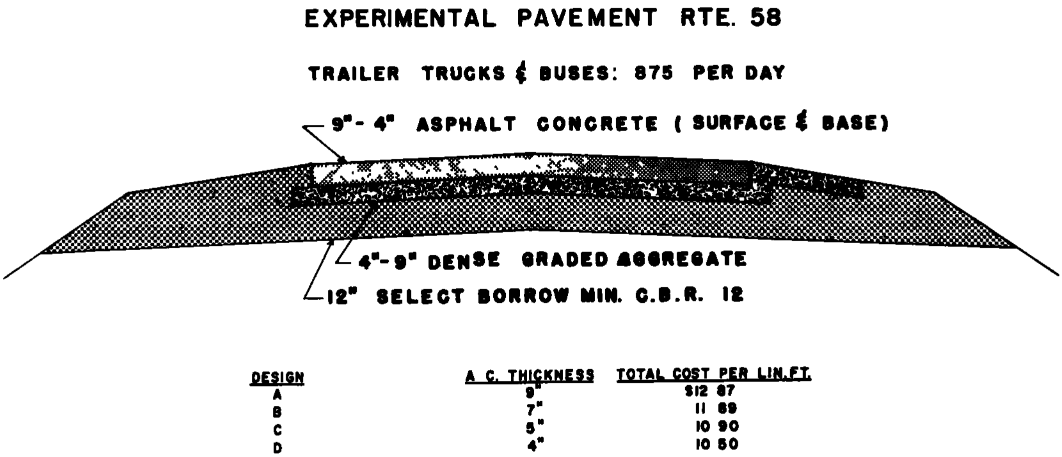


Figure 4. Typical pavement sections for experimental project on Route 58 west of South Boston.

from 80 F in July to 40 F in January; rainfall on the average totals approximately 44 in. per year and is rather uniformly distributed from month to month. These are long-term average figures, however, and extremes diverge markedly. In 1958, for example, monthly precipitation ranged from 1.66 in. in September to 6.48 in. in June, and extremes in temperature ranged from 98 deg to minus 2 deg. The variable investigated here was thickness of asphaltic concrete. The total structure above the native soil subgrade was 25 in. thick in each of four designs. The top 13 in. consisted of dense-graded aggregate, in thicknesses of 4, 6, 8, or 9 in., covered with asphaltic concrete in corresponding thicknesses of 9, 7, 5, or 4 in. Dense-graded aggregate was used also to surface the outer shoulder of this divided highway. The balance of the typical section was composed of select borrow, minimum specified CBR value of 12, obtained from a local pit near the east end of the project. (Although the borrow was only required to have a CBR value of 12, in actuality the CBR values on twelve samples tested averaged 33. The pit was in a deposit of disintegrated granite with a high percentage of coarse particles greater than 2 in. in size.)

The four design variables will be identified hereafter as Designs A, B, C, or D as shown in Figure 4. The costs per lineal foot for a single roadway for each of the four designs, based on contract unit prices, are given in Table 1.

TABLE 1

ROUTE 58 EXPERIMENT: SUMMARY OF CONSTRUCTION COST PER LINEAL FOOT FOR ONE ROADWAY (PAVEMENT AND SHOULDERS)[1]

Design A.C. Thickness	A 9 in.	B 7 in.	C 5 in.	D 4 in.
	($)	($)	($)	($)
Course:				
Asphaltic concrete	8.88	6.98	5.01	4.21
MC-O prime	0.27	0.27	0.27	0.27
Graded aggregate	2.05	2.98	3.96	4.36
Select borrow	1.24	1.23	1.23	1.23
Shoulder surface	0.43	0.43	0.43	0.43
Total	12.87	11.89	10.90	10.50

[1] Bid prices in usual units of tons, gallons, cubic yards, etc. By computation all prices converted to unit of one lineal foot.

The four guiding principles established for experimental construction were adhered to closely. Because standard specifications and conventional equipment were used and because the number of combinations was held to four, contract unit costs ran very close to the statewide averages for primary construction. The 4½-mi project was divided into eight subsections ranging from 1,830 to 3,550 ft in length, and replication was provided in that each design occurred twice. The ever increasing volume of trailer trucks and buses using this portion of US 58, totaling over 425 daily in each direction at the latest count, gives assurance that the heavy traffic requirement of principle 4 will be met.

Considerable detailed information on the classification, both engineering and agricultural, of the native soils, has been gathered by the Research Council. All available information regarding source, type, and quality of select borrow and other paving materials also has been catalogued for future reference. Such details have been generally omitted from this paper.

The test project was built in 1958 and opened to traffic in December of that year though not finally accepted from the contractor until January 15, 1959.

RESEARCH EVALUATION STUDIES

Realizing that conclusions based entirely on performance under traffic might not be available for some years, it was decided that certain observations and measurements should be made by the Research Council in an attempt to secure an earlier, though tentative, evaluation of the four pavement designs in this first experimental project.

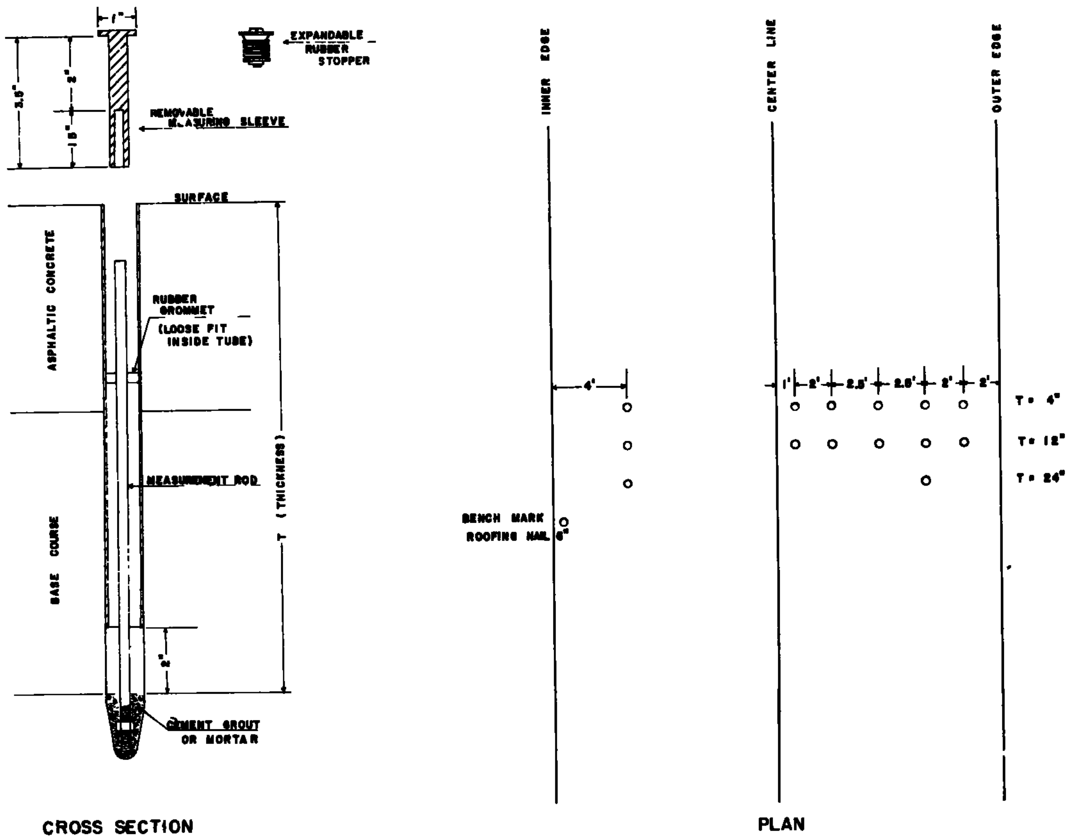
Council technicians were assigned to the test project during most of the paving operations to secure data on the compaction of the select borrow and of the dense-graded aggregate base course. Test sites were established in groups of five at regular intervals of 1,000 ft; in each group the individual sites were spaced 50 ft apart. At each site, with only two or three exceptions made necessary by the contractor's sequence of operations, measurements were made of the final construction density of the borrow and base courses just prior to the placing of the subsequent layer.

In place density values were determined by AASHO standard method T-147-54, except that test hole volumes were determined with a small water balloon volumeter. Research Council investigations have indicated that this instrument is superior in both accuracy and precision to the sandcone method in testing coarse-grained base materials. In place densities are expressed in terms of percent of laboratory standard density. Standard density is determined in Virginia by AASHO Standard T-99-57, method A, on the minus No. 4 material only. Correction for coarser aggregate assumes no voids associated with the plus No. 4 fraction and that minus No. 4 material remains at standard density. Research is under way aimed toward an improved method of establishing standard density in the case of aggregate materials.

Later, after the final surface had been laid, settlement rods were installed at 1,000-ft intervals to permit measurement of progressive changes in elevation of layers 4, 12, and 24 in. beneath the surface. The idea for these rods was borrowed from the WASHO Test Road (5). The rods were installed transversely across the pavement at one of the five sites in each test group (Fig. 5). Six are placed at a depth of 4 in., and two at 24 in. below the surface. The determination of the elevation of the top of each rod was facilitated by the use of a sliding scale graduated to the nearest 5 one-hundredths of an inch, attached to a standard level rod. As all sights were from less than 30 ft away, this scale could be read quite easily with a standard eye level. Since the rod could be adjusted to read zero at the bench mark each time, all changes in elevation became readily apparent by direct comparison with readings taken previously.

Additionally, soon after the pavement had been opened to traffic, extensive studies of pavement deflection were begun, measurements being made with the Benkelman beam and a test load of 18,000 lb on a single axle. The first series of tests was made in February and a second series in October 1959. Three separate techniques were used in these tests to obtain the following:

1. Maximum deflection at the surface, the conventional test with truck moving at



Installation: Hole drilled through asphaltic concrete pavement into base to exact outside diameter of protective metal tube. Hole continued to desired depth "T" by use of special driving rod. Proper quantity of cement mortar dropped to bottom of hole to cement rod to bottom. Protective tube driven into place and capped to prevent entry of water and foreign material. Measurements: Expandable rubber cap removed and special extension sleeve placed on top of rod. Level rod or Benkelman beam probe set in place on extension sleeve cap for determination of elevation or deflection of bottom of rod.

Figure 5. Settlement measuring rod installations, plan and cross-section.

creep speed past probe. Initial, maximum, and final dial readings recorded. Tests made at each site where base and borrow density had been determined, both in February and in October.

2. Longitudinal distribution of deflection. Truck stopped at 2-ft intervals both approaching and departing from the probe, dial reading recorded at each stop. Tests made in February at only one site in each group of five, in October at all sites.

3. Subsurface deflections. Test truck backed into position over probe, then pulled forward again. Initial, maximum, and final dial readings taken with probe on surface and on top of extensions of settlement rods at 4-, 12-, and 24-in. depths. Tests made at one site in each group of five in February only.

Table 2 summarizes the results of the density determinations made during construction and of the maximum surface deflection measurements made in both February and October. From this it may be seen that the magnitude of deflections determined

TABLE 2

ROUTE 58 EXPERIMENT RELATIONSHIP BETWEEN FINAL CONSTRUCTION DENSITY AND PAVEMENT DEFLECTION UNDER 18,000-LB SINGLE AXLE LOAD

Pavement Design	Average Density (% of Standard): Select Borrow	Average Density (% of Standard): Aggregate Base	Average Density (% of Standard): Borrow-Base Combined	Average Surface Deflections Thousandths of an Inch: Outer Wheel Path, Feb.	Outer Wheel Path, Oct.	Inner Wheel Path, Feb.	Inner Wheel Path, Oct.
B(7 in. A.C.)	98.8	96.6	97.7	33	35	30	32
D(4 in. A.C.)	96.6	97.7	97.2	38	37	37	36
C(5 in. A.C.)	97.6	95.9	96.8	45	46	44	43
A(9 in. A.C.)	95.3	94.4	94.9	47	55	46	50

in the conventional manner bore no relationship to the thickness of asphaltic concrete, but rather that high deflections seemed to be more the result of low density in the base and borrow materials. In other words, the highest deflections, on the average, occurred in the sections built to Design A, which had the greatest thickness of asphaltic concrete, 9 in., but had the lowest average construction density in both the base course and the select borrow. The fact that the combined average of base and borrow densities falls in perfect inverse order to the average magnitude of deflections (240 measurements in each wheel path at each of two seasons of the year) probably is the most important finding from the experiment to date.

The tests showing the longitudinal distribution or rate of occurrence of deflection give some indication of the relative abilities of the four pavement designs to act as a slab and distribute loads from the surface to underlying layers. This technique, described as number 2, was designed to ascertain the abruptness of the deflection, it being felt that a given deflection occurring gradually as the load approaches the point of measurement should not be as destructive as a similar deflection occurring suddenly. Analysis of the actual values, however, was complicated by two uncontrollable variables, (a) the wide range in maximum deflections from site to site and from design to design, as given in Table 2, and (b) a similarly wide range in residual or more or less permanent deformations, recorded as the differences between the initial and final dial readings. But these variables become less troublesome when all values are expressed as percentages of a maximum value (Table 3).

Study of Table 3 reveals that the pavement with the greatest thickness of asphaltic concrete, Design A, does exhibit some slab action, because deflections occur more gradually. In general, deflections are less gradual as plant mix thickness decreases. In October, the figures for Designs B and C were not noticeably different; the reason for this will become more apparent from a study of the section on Early Performance Observations.

In both series of tests, the most abrupt deflections occurred in Design D. Therefore it is assumed that were it possible to construct these four pavements on identical subgrades and compact all courses to identical densities, the destructive effect of pavement deflection would be in inverse proportion to the thickness of asphaltic concrete.

Analysis of the deflection values measured at the various subsurface levels (technique 3) also is complicated by the same variables that hindered analysis of the "rate of occurrence" figures. Again the percentage approach seems more understandable, and has been adopted in Table 4.

Here it seems that in Design A the lowest percentage of the total deflection originates in the pavement structure itself. Attempts to rationalize this in light of other evidence from density and settlement measurements have not been entirely successful. Therefore no apparent significance is evident from these figures, but it may be of more than passing interest to note that from 40 to 54 percent of the total deflection seems to originate in the subgrade below the 24-in. level.

TABLE 3

ROUTE 58 EXPERIMENT DISTRIBUTION OF DEFLECTION AHEAD OF AND BEHIND LOADED AXLE

Pavement Design	Deflection at Indicated Distance Ahead of Load % of Maximum		Maximum Deflection	Deflection Remaining at Indicated Distance Behind Load % of Recovery		
	4 ft	2 ft	Load Over Probe	2 ft	4 ft	6 ft
(a) February 1959 Series						
A(9 in. A.C.)	15	37	100	56	18	5
B(7 in. A.C.)	10	25	100	43	11	6
C(5 in. A.C.)	9	23	100	38	7	2
D(4 in. A.C.)	8	18	100	28	6	2
(b) October 1959 Series						
A(9 in. A.C.)	18	43	100	37	9	3
B(7 in. A.C.)	11	27	100	31	6	2
C(5 in. A.C.)	12	28	100	27	6	2
D(4 in. A.C.)	9	22	100	24	4	1

TABLE 4

DEFLECTIONS BENEATH PAVEMENT SURFACE

Pavement Design	Percentage of Total Surface Deflection Measured Within Layers Indicated		
	Top 4 in.	Top 12 in.	Top 24 in.
A(9 in. A.C.)	5	18	46
B(7 in. A.C.)	5	26	57
C(5 in. A.C.)	6	26	60
D(4 in. A.C.)	5	25	58

EARLY PERFORMANCE OBSERVATIONS

The Research Council makes periodic surveys, most of which are of the "quickie" type in which only the more obvious defects are logged. An odometer which measures distance in feet is attached to an ordinary automobile so that the location of such defects can be recorded on especially prepared log sheets which may be used over and over again. In each survey of a given project, a different colored pencil is used for recording; in this manner the progressive growth of distress can be noted. Two such surveys have been made of the US 58 project.

More detailed observations made on the US 58 test project have included measurement of changes in elevation of the settlement rods (described earlier under Research Evaluation Studies) and measurement of surface rutting.

After one year under traffic the most pronounced defect which has occurred was a rather serious one near the west end of the project in one of the two sections built to Design C. Since construction of the base and surface was from west to east, this failure occurred within the first 1,500 ft completed. It also occurred between two of the groups of test sites where the highest individual deflection and the lowest individual construction densities on the project had been recorded.

The pavement in this area started to crack in April 1959 and by June it had been heavily patched—in some places to the full depth of the base. After heavy summer showers, the patching crews noted an accumulation of free water in the loosely bonded aggregate base. Later in the summer a stretch several hundred feet long was completely resurfaced with about 1½ in. of asphaltic concrete. This, it is felt, accounts for the change in behavior of Design C with regard to rate of occurrence of deflections, as was noted in the October deflection readings (Table 3).

Aside from this one serious failure in Design C, the only other distress noted to date has consisted of minor alligator cracking at two isolated locations, one in Design A and one in Design D, and more or less general rutting throughout the project. It seems significant that both of the cracked areas also are near test sites where high deflections and low densities were recorded.

Rutting was measured downward from a string line stretched between the crown and the edge of the pavement. Measurements were made at each of the five test sites in each of the 24 groups. Results of the latest readings are as follows:

The most noticeable rutting on the project, at the location in Design A where slight cracking was also evident, was measured to be 1.188 in. deep, but is not included in the above averages because it did not occur at one of the regular test sites. It is noted that rutting on the average is now deepest in both wheel tracks in Design A. Average rutting in Design C was greatly reduced during the summer by the resurfacing of several hundred feet of the most distressed section.

TABLE 5

ROUTE 58 EXPERIMENT RUTTING IN INCHES, OCTOBER 1959

Pavement Design	Inner Wheel Path			Outer Wheel Path		
	Max.	Min.	Average	Max.	Min.	Average
A(9 in. A.C.)	0.750	0.063	0.263	0.875	0.125	0.332
B(7 in. A.C.)	0.313	0.063	0.143	0.375	0.125	0.263
C(5 in. A.C.)	0.563	0.000	0.204	0.438	0.000	0.215
D(4 in. A.C.)	0.250	0.063	0.170	0.438	0.125	0.306

In the attempt to determine the depths to which this rutting extended below the surface, elevations of the tops of all settlement rods (Fig. 5) have been measured. The results of all elevation determinations are shown in Figures 6, 7, 8 and 9. The zero line represents the original as built condition, and all deviations from this line represent elevation changes after six months and ten months of traffic.

It is interesting as well as puzzling to note how many of the so-called "settlement" rods appear to have risen rather than settled. This same condition apparently occurred in the WASHO test (6), and from unpublished correspondence it is understood to be happening again at the AASHO test in Illinois.

Nevertheless, it is felt that these settlement rod readings do present, in a qualitative sense at least, a picture of the movement of the various pavement layers relative to each other. The findings support those from the WASHO test report which states that "by far the greatest change in elevation in any section took place at the surface level (6)." Though settlement, like rutting, seems to be less pronounced in the stronger sections of the project (where compaction was better and deflections lower) some settlement of the surface did occur in the wheel tracks at every location. Inasmuch as the 4-in. level on the US 58 project is not below the asphaltic concrete in any of the designs, it seems obvious that much of this movement is the result of some sort of displacement within the asphaltic concrete. Further research along these lines, including observations of pavements in the field, would be desirable to discover what types of asphaltic concrete have the greatest resistance to this displacement.

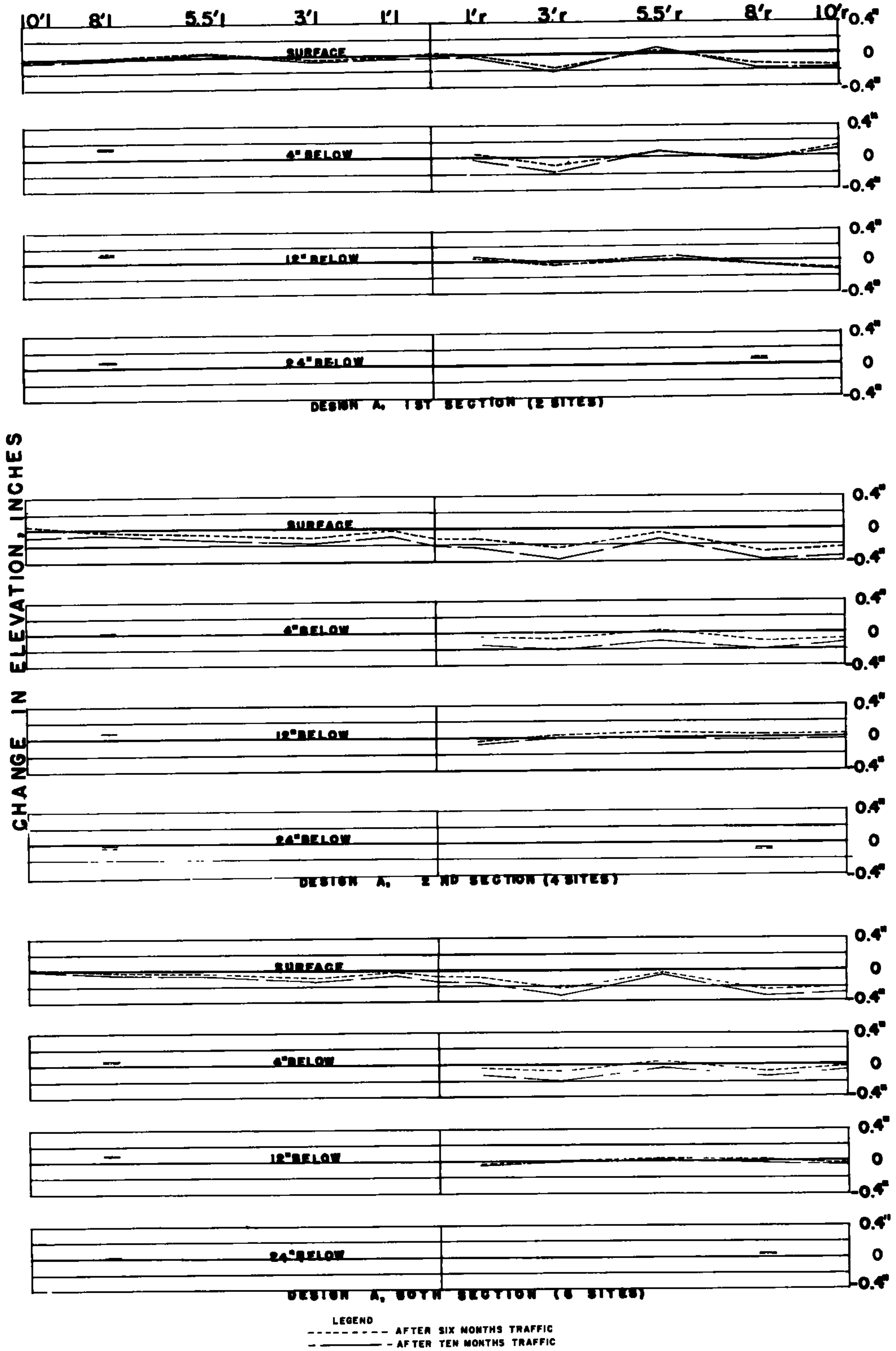


Figure 6. Changes in elevation of various layers in Design A.

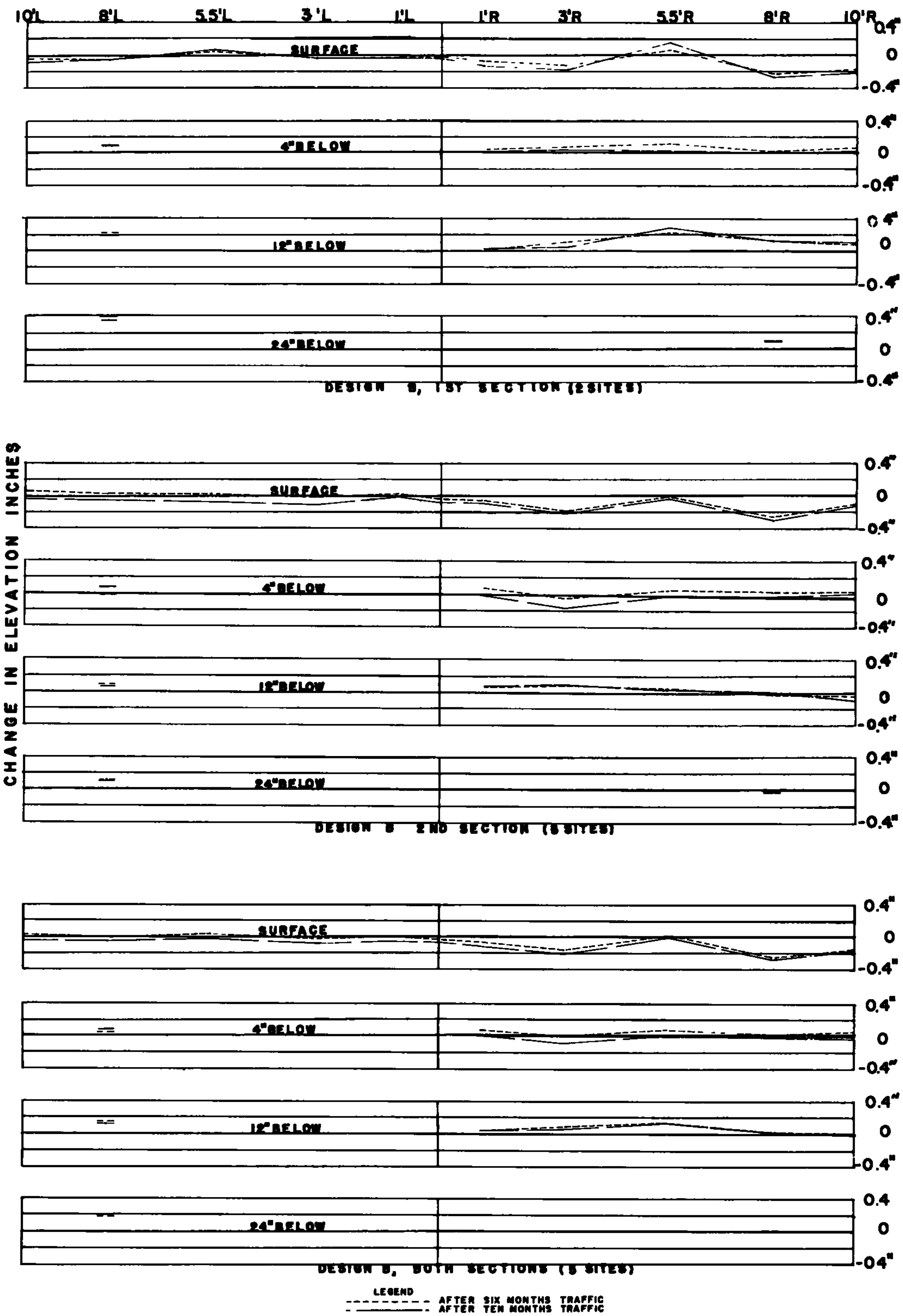


Figure 7. Changes in elevation of various layers in Design B.

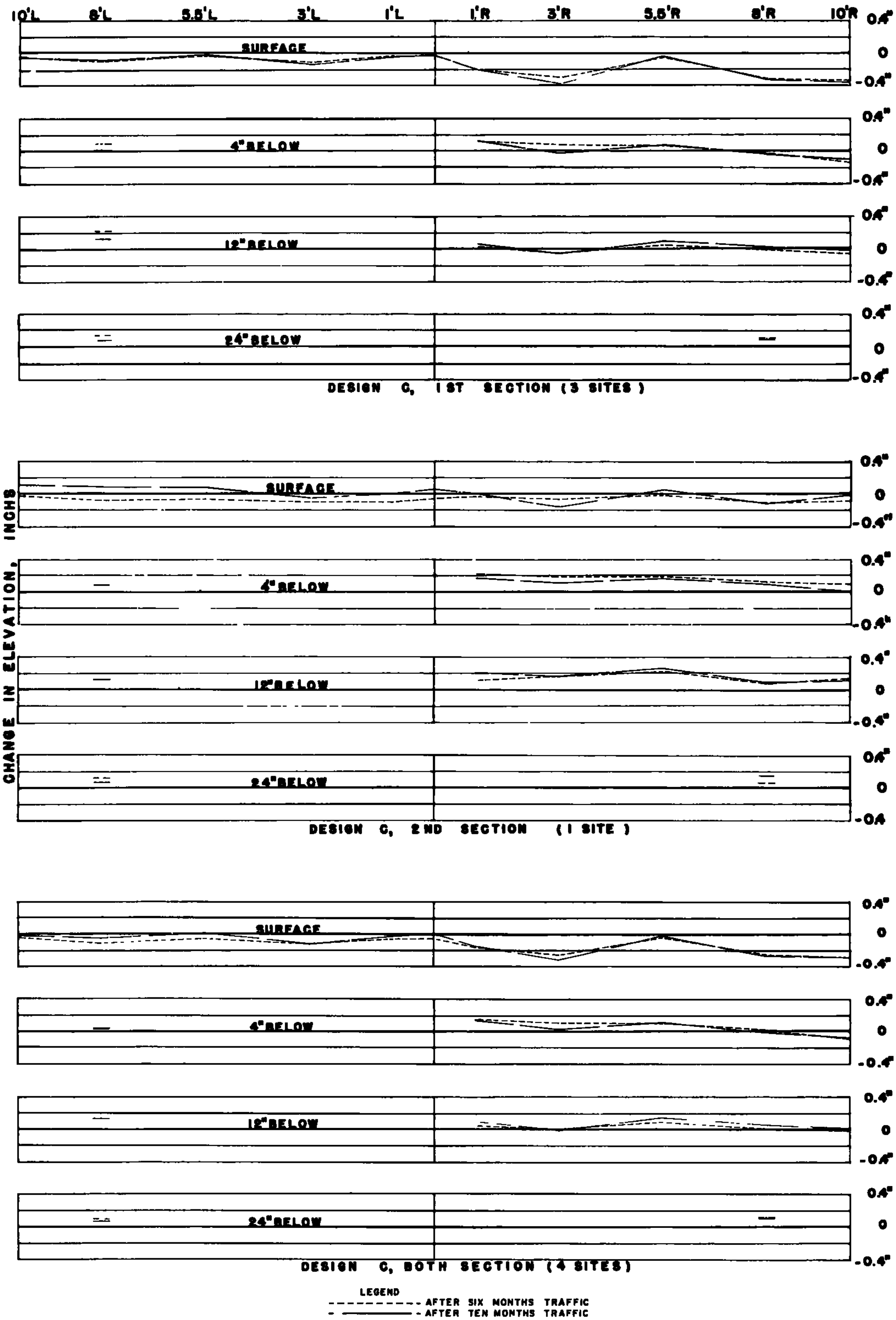


Figure 8. Changes in elevation of various layers in Design C.

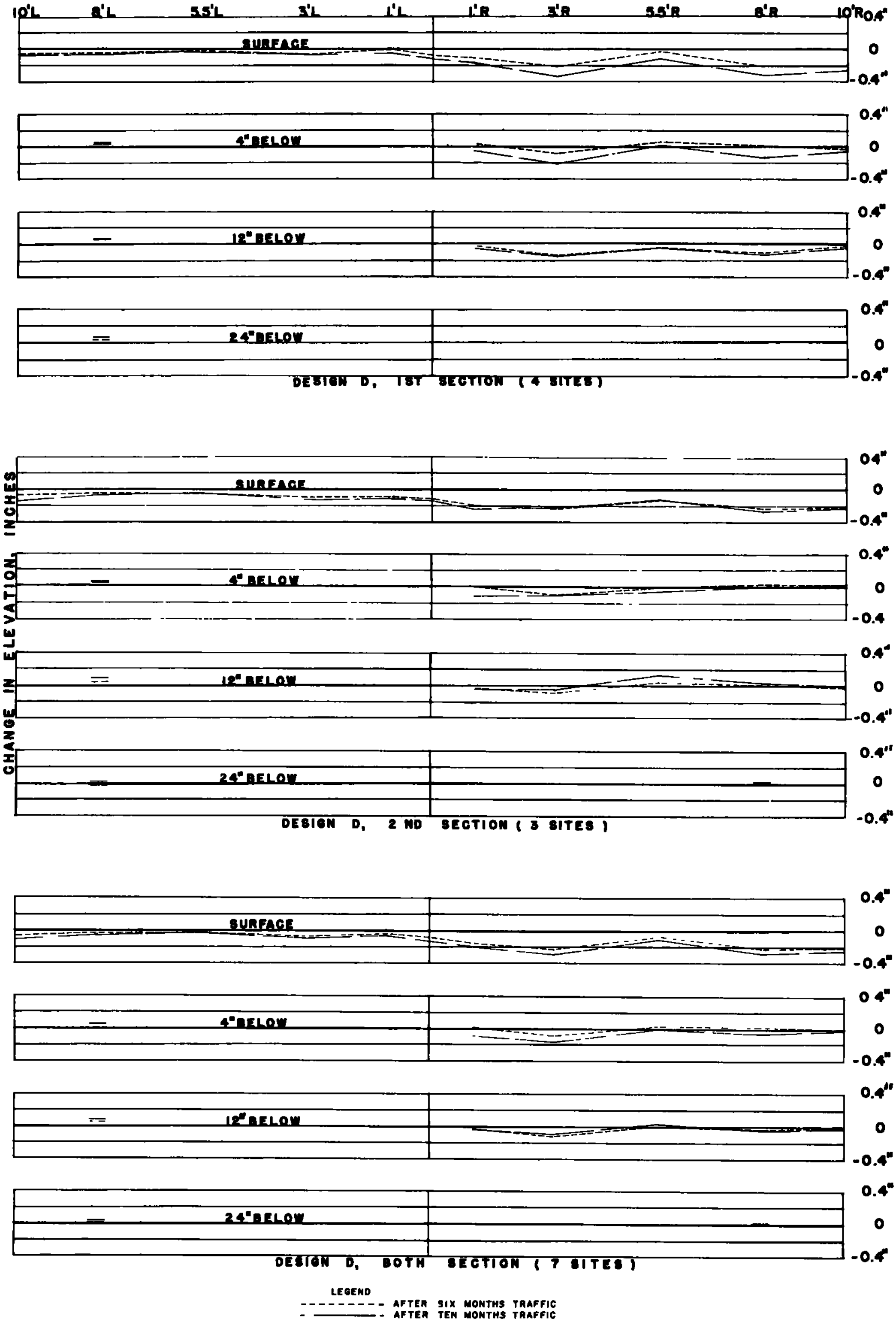


Figure 9. Changes in elevation of various layers in Design D.

SUMMARY AND CONCLUSIONS

To recapitulate briefly, the development of current pavement design practices in Virginia has been traced. Inasmuch as the CBR design method still is used only as a guide in establishing total pavement thickness (3), it was felt that the construction of experimental flexible pavements was in order so that the performance of different pavement designs built at the same time on the same project and subjected to the same traffic might be compared.

The first such experimental pavement, with four different pavement designs (Fig. 4), was completed and opened to traffic in December 1958. Its purpose was to compare thicknesses of asphaltic concrete of 4, 5, 7, and 9 in. in designs of the same total thickness. One major failure which occurred in April 1959, in a section with 5 in. of asphaltic concrete, has been described. Mention has also been made of the incidence of minor cracking at two other locations and of general surface rutting throughout the project. The Research Council's method of securing an earlier evaluation of the relative merits of the four pavement designs have been outlined in some detail.

The principal findings from the first experimental project are that:

1. Deflections and performance seem more closely allied with compaction than with pavement design characteristics. High deflections rather consistently occurred where construction densities were found to be low. Also, the one major failure did not occur in Design D, which had only 4 in. of asphaltic concrete, but in Design C, which had 5 in. It occurred near the point where the highest deflection as well as the lowest base course density on the project had been measured. Minor distress also is beginning in Design A, which had 9 in. of asphaltic concrete, and in Design D, in both cases again at points of high deflection and low density.
2. Deflections are somewhat less abrupt in the sections with greater thickness of asphaltic concrete.
3. Rutting in the wheel tracks has occurred to some extent throughout the project, regardless of pavement design. The deepest rutting on the project has occurred where the asphaltic concrete is the thickest.

It is too early to base definite conclusions on findings from this project. Also, conclusions based on a single experiment should never be considered final. Therefore, a second experiment has been designed and is scheduled for construction in 1960. Two of the designs from the US 58 project, Design B (7 in. of asphaltic concrete) and Design D (4 in. of asphaltic concrete), with minor revisions, will be repeated on the new project. Two other designs will include cement treatment of the crushed aggregate base material. All four designs will be built on a cement-treated subgrade. Special efforts will be exerted toward securing more complete data on compaction of all pavement components in the hope that an even more definite relationship between deflections and pavement densities can be established. Advice is being solicited from a University of Virginia faculty member, who serves as a Research Economist on a part-time basis with the Research Council, to assist in a more proper statistical analysis of the data.

But on the basis of early observations of the US 58 project, the following tentative conclusions have been advanced:

1. That greater slab action, with somewhat wider distribution of pressures to underlying layers, is afforded by increased thicknesses of asphaltic concrete.
2. That total deflection and the resultant poor performance can be minimized more effectively by increased emphasis on compaction of all components of the pavement than by "beefing up" the pavement design.
3. That rutting in the wheel tracks results primarily from displacements which occur within the top 4 in. of the pavement structure, and that some rutting may be expected in most asphaltic concrete regardless of the support offered from underlying layers.

Future research in Virginia will place great emphasis on compaction. Experiments are continuing for the purpose of evaluating and comparing the accuracy and precision of various methods of measuring field density. Better laboratory methods of determining realistic standard densities of all sorts of aggregate materials as well as asphaltic concrete are being sought also. Such research is aimed toward greatly improved control of compaction during construction. However, the evidence so far points strongly toward the necessity that the control of compaction be assigned to trained specialists and not relegated to junior inspectors as a collateral duty.

It is hoped that this report may encourage other agencies to set up experimental projects along similar lines to the one just described, incorporating variables which seem most important in the specific locality, and to report their findings at future meetings of the Highway Research Board.

ACKNOWLEDGMENTS

The planning and construction of an experimental project of this nature could not have been accomplished without the interest and cooperation of a great many people representing the Virginia Department of Highways, the Bureau of Public Roads, and the Thompson-Arthur Paving Company. While it is difficult to single out individuals, it is felt that the following persons deserve mention for their efforts: H.H. Harris, A.B. Cornthwaite, J.E. Johnson and K.E. Ellison, of the Central Highway Office, for their assistance in establishing the pavement designs; E.L. Alsop (now deceased), S.T. Barker, and R. Worthington, Resident Engineers under whom construction was accomplished and who furnished valuable assistance in the performance of research evaluation tests; R.V. Fielding, District Materials Engineer, for furnishing much information on the materials used; and particularly R.W. Gunn and E.C. Snell, Technicians with the Research Council, for performing all the deflection and density tests and most of the computations, drafting and related chores necessary to make this report possible.

Finally, the author wishes to express his appreciation for the encouragement and wise counsel furnished by Tilton E. Shelburne, Director of the Virginia Council of Highway Investigation and Research.

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Discussion

W.H. CAMPEN, Omaha Testing Laboratories—The writer has studied Mr. Nichols' paper on "Flexible Pavement Research in Virginia" and unless the writer has misinterpreted the preliminary performance behavior of the 1958 experimental road, the final results will be disappointing.

Having had many years experience with the design, control and performance of flexible pavements, the writer has come to the following conclusions in regard to the project:

1. The subbase and base were not compacted to proper density.
2. The subbase and base may not possess other desirable qualities even though they may have the desired CBR values.
3. The asphaltic concrete was not compacted to the proper density.

4. The asphaltic concrete may not have sufficient stability or other desirable qualities.

Certainly the subgrade must not be the cause of the distress because its CBR would have to be less than one, which would be too weak to carry the construction equipment for subbase construction.

Experience has shown that flexible pavement subbases and bases must be constructed with good aggregates and compacted to at least 100 percent of maximum laboratory density. The asphaltic concrete must possess high stability and must be compacted to at least 95 percent of maximum theoretical density, and if more than 2 in. thick, the lower layers should contain high percentages of large aggregate (3/4 in. or larger). In addition, at least, the upper 6 in. of the subgrade should be compacted to 100 percent of standard laboratory density.

FRANK P. NICHOLS, JR., Closure—Mr. Campen's many years of experience seem to have led him to some of the same conclusions with regard to this project that the author has attempted to put across in the paper. However, some of Mr. Campen's conclusions must have been reached intuitively, inasmuch as nothing was said in the paper regarding "other desirable qualities" of the base and subbase or the density of the asphaltic concrete. Most of the attributes prescribed by Mr. Campen for a successful flexible pavement are also prescribed in Virginia's Road and Bridge Specifications. It is believed that the quality and gradation of all aggregates and the percentages of asphalt used would have met with Mr. Campen's approval. The fact that in certain sections compaction was not adequate and that this has already affected performance was stressed in the paper.

It should be emphasized that the Research Council did not attempt to control the quality of the construction on this project, but preferred to leave that to the inspectors assigned for the purpose. It was desired that the construction be representative in quality to that of conventional projects, and it is believed that this desire was realized. While in some respects the results may be disappointing, it should be of more than passing interest that the least expensive design with superior construction may outlast the most expensive design with mediocre construction.

Finally, Mr. Campen's CBR design chart obviously is different from the one now in use in Virginia (3). For the volume of traffic now using this road, the 25-in. total structural thickness would be adequate for a subgrade CBR value of 4, but a CBR value of 1 would require a total thickness of 52 in. Fortunately, actual subgrade CBR values generally ran well above 4 on this project, but in certain areas the value was around 4. Actually, the major failure near the west end of the project occurred in one of these areas of low subgrade CBR value. This leads to speculation that weak subgrades contribute to poor pavement performance in two ways: they offer poor support to the finished pavement under traffic loads and they also offer poor support to the rollers attempting to compact subsequent layers.

Mr. Campen's comments are appreciated in that they called attention to these omissions in the original text.

Deflection Tests on Texas Highways

WAYNE A. DUNLAP and L. E. STARK, respectively, Assistant Research Engineer, and Associate Research Engineer, Texas Transportation Institute, Texas A and M College System, College Station

This paper presents some of the experiences and results obtained from an investigation of pavement deflections in Texas. The data discussed were taken from measurements made by use of the Benkelman beam during annual deflection tests on 117 mi of flexible pavements. Results of laboratory and field tests are presented showing the reliability of the Benkelman beam. Factors which may affect the deflection data are discussed, including temperature of the pavement surface, length of the deflection "trough," friction in the Benkelman beam and Helmer recorder, and field techniques in the operation of the Benkelman beam. Methods are also given for correcting the deflection data for these factors.

●DEFLECTIONS in flexible pavements have received considerable attention from highway engineers in recent years. From a design standpoint, several methods have been advanced which are based on the limiting deformation or deflection of a pavement system under load. For the most part, these methods rely wholly on theoretical analysis or partly on theory and partly on experience. As a group, they suffer from the assumptions that the materials in the pavement system behave elastically at all times and that the applied loads are uniformly distributed. Considerable research must be undertaken in an effort to determine the effect of these assumptions on stresses and deflections in actual soil materials before the validity of the design methods can be accepted.

On the other hand, the use of deflections for evaluation purposes has shown more promise. Engineers visualize the application of deflections to the determination of useful pavement life, the selection of allowable wheel loads during both ordinary and critical climatic periods, the evaluation of assumptions made in pavement designs and other more specialized uses. The most obvious advantages of obtaining deflections directly on existing pavements are the speed of the determinations and the release of certain theoretical assumptions regarding the interaction of layers in pavement systems. Deflections have been successfully used in large-scale evaluations (1, 2, 3), but the deflection results have been supported by other information not normally available to the engineer who must evaluate several miles of pavement with a 1-mi budget.

Means for measuring pavement deflections include electronic methods (1), photogrammetric techniques (4), rigid beams equipped with a series of extensometers (3), and lever-type beams such as the Benkelman beam (1). At present, the Benkelman beam appears to be the most popular deflection measuring device.

In Texas, impetus in deflection measurements was provided in 1955 by a preliminary investigation of the Benkelman beam by the Texas Highway Department. In 1956, the Texas Highway Department authorized the Texas Transportation Institute to conduct an evaluation of selected flexible pavements using deflections obtained by the Benkelman beam. Within six months after the initiation of the test program, deflections had been obtained on nearly 500 mi of pavement. The ease of obtaining deflections with the Benkelman beam temporarily overshadowed studies concerning reliability of the deflection values. Not until several hundred test sites were analyzed was it realized that the accuracy of the deflection values was highly questionable.

Subsequent research into the equipment and field measuring technique disclosed methods of collecting reliable data. Of prime importance was the determination of various corrections to be applied to the Benkelman beam measurements. This report is concerned primarily with these corrections and the field techniques. At a later date it is anticipated that a more complete pavement evaluation process based on deflections can be reported.

DESCRIPTION OF EQUIPMENT AND METHODS

The primary equipment used in this study consisted of a load vehicle and two Benkelman beams. Equipment of secondary importance included a thermometer for measuring pavement temperature and a tire pressure gauge.

The Benkelman beam (Fig. 1) has been discussed adequately elsewhere (1, 5) and only a brief description is required here. The beam rests on one rear reference support and two front reference supports. The probe arm has an effective length of 12 ft and is suspended by a single bearing bracketed exactly 4 ft forward of the extensometer contact point. The forwardmost part of the probe arm, the toe, rests on the pavement exactly 8 ft from the bearing axis of rotation. Vertical movement of the pavement surface at the toe will indicate one-half of this movement on the extensometer.

To produce the desired load, a standard dump-truck was loaded with steel grader blades. The load was arranged such that the dual wheel loads were accurate to within 15 lb. Tire pressures on the load vehicle were maintained at 90 psi. The dual tires

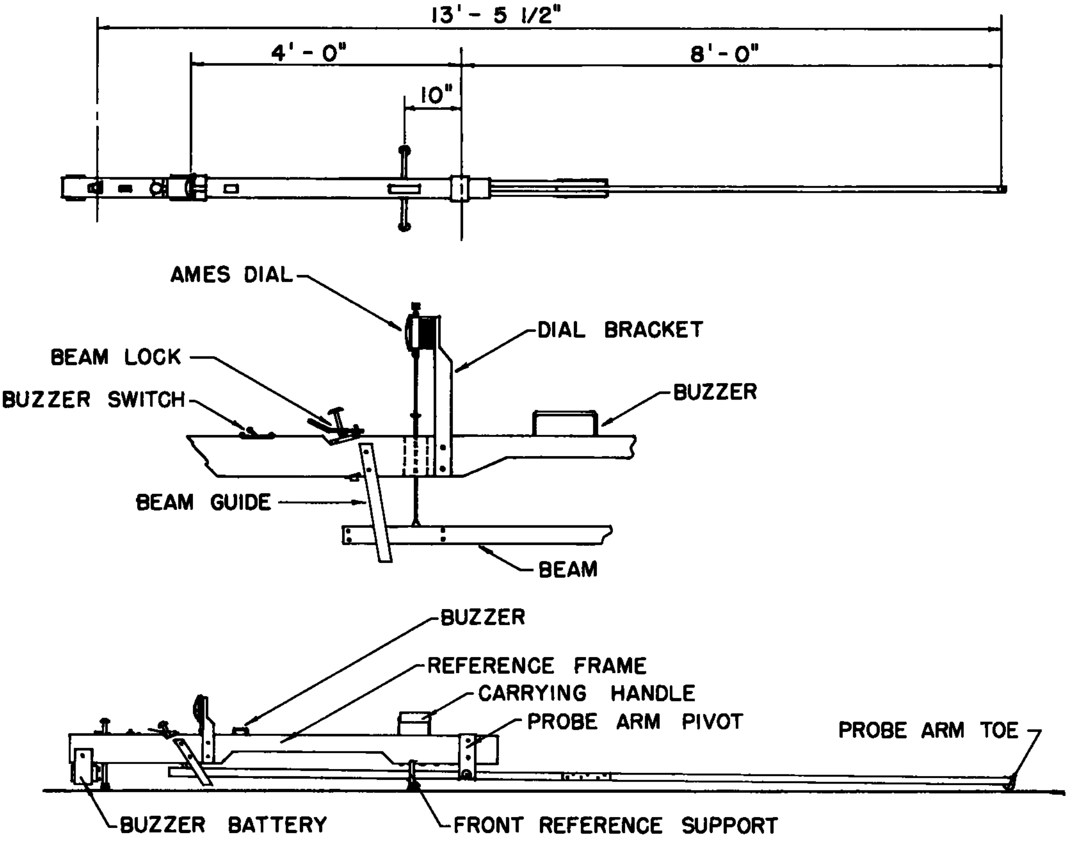

Figure 1. The Benkelman beam.

were spread apart an additional ½ in. with steel spacers to reduce the possibility of rub between the beam toes and the tires.

Deflection tests were performed by inserting the probe arms of the beams between the dual tires until the tires were equidistant from the front reference supports and the beam toe. With the load vehicle in this position an initial extensometer reading was recorded. The load vehicle then moved forward at creep speed (0.3 mph) to approximately 15 ft past the beam toe. Both the maximum and final readings of the extensometer were recorded during this movement.

In accordance with the accepted procedure, the difference between the initial and maximum extensometer readings was doubled and was termed the deflection. The difference between the initial and final readings was doubled and termed the residual (or permanent) deformation. Early in the testing program it was noted that the residual values were sometimes negative. These negative values resulted from pavement extrusion between the dual tires of the load vehicle and reveals that one of the inherent disadvantages of the Benkelman beam is that it measures deflection between and not underneath the tires. It was found, however, that extrusion occurred primarily on pavements of very high asphalt cement content or on pavements with very low load-carrying capacity.

Pavement temperatures were measured by inserting a thermometer into an oil-filled hole approximately 1 in. deep and ½ in. in diameter formed by driving a pointed steel rod into the surface. There is some indication that temperatures measured in this manner are about 10 F less than temperatures determined from thermocouples inserted in the interior of the pavement surface (6).

When the deflection testing program was initiated, one of the first correlations attempted was wheel loads versus deflections. Wheel loads of 7,000, 9,000 and 12,000 lb were selected for this study. Surprisingly, little correlation was found and in most instances deflections for the 12,000-lb wheel load were smaller than for the 9,000-lb wheel load. Examples of the results obtained on a typical test section are shown in Figure 2.

The residuals followed the expected pattern more closely than the deflections. For the 7,000-lb wheel load, residuals were almost non-existent, registering greater than 0.002 in. only 10 percent of the time; for the heavier loads they increased significantly as the load increased. Figure 3 shows the residual values corresponding to the deflections shown in Figure 2. The lack of correlation between wheel loads and deflections

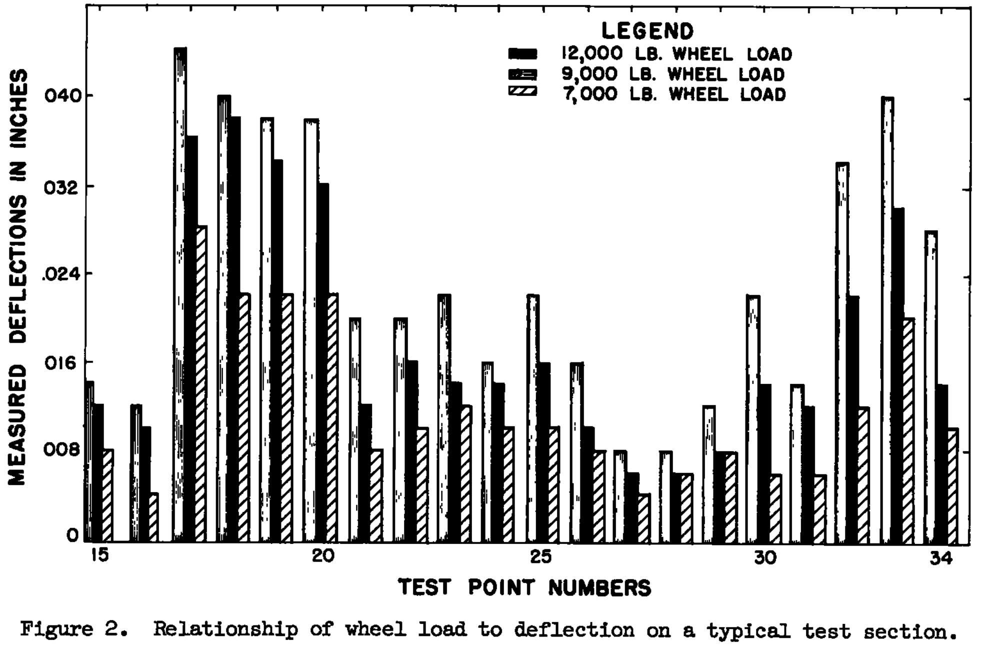

Figure 2. Relationship of wheel load to deflection on a typical test section.

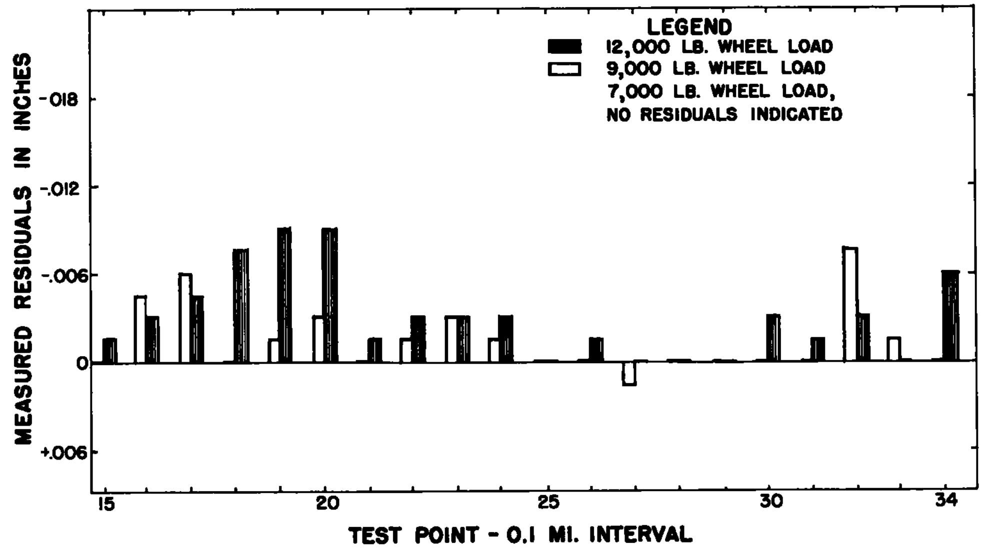

Figure 3. Relationship of wheel load to residual deformation for test section shown in Figure 2.

was very disappointing when considered in the light of the excellent correlations obtained in other investigations (1, 2).

After the original testing program was completed and the results tabulated, it was found that the data were too erratic to indicate any trends. Also, the data showed many factors which were extremely confusing. Several of the pavements which were rated as good showed higher residual values than those rated as poor. In addition, many researchers familiar with flexible pavement deflections felt the beams were indicating values much smaller than the actual road deflections.

HELMER GRAPHICAL RECORDER

In an effort to increase the accuracy of the results, Helmer graphical recorders were added to the Benkelman beams in 1957. The recorder, developed by R. A. Helmer of the Oklahoma Highway Department, operates from a lever actuated by the Benkelman beam probe arm. A pen mounted on the lever draws a graph of the beam toe deflection as the test vehicle is driven up to and beyond the beam toe. The vertical movement of the beam toe is magnified ten times on the graph. A friction drive motivated by a cord attached to the load vehicle provides horizontal movement of the graph paper at the

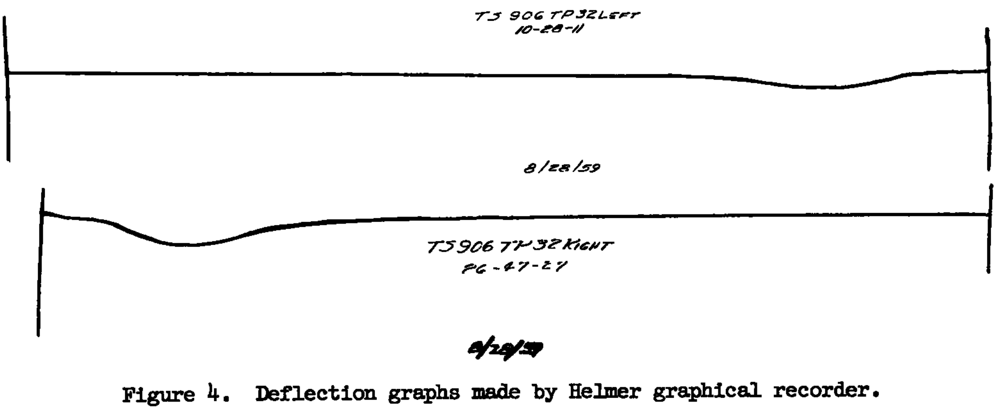

Figure 4. Deflection graphs made by Helmer graphical recorder.

rate of 1 in. = 1 ft. Examples of the graphs drawn by the recorder are shown in Figure 4.

After analyzing many deflections taken with the Helmer recorder, the data were still found to be erratic and inconclusive. Only in isolated instances were there any measurable correlations between pavement performance and deflections. Attempts were also made to correlate other data obtained from the Helmer recordings—such as slope of the deflection curves and minimum radius of curvature—with pavement performance, but these efforts proved fruitless.

INVESTIGATIONS OF ACCURACY OF DEFLECTION VALUES

Subsequent investigations into the causes of the erratic data led to a closer examination of the residual values. It was not at all uncommon for pavements rated as very good to have residual values as high as 0.010 in. On most of these good pavements the average of the ten heaviest daily wheel loads exceeded the test vehicle wheel load by 1,000 to 3,000 lb. Because these pavements had received thousands of the heavier load repetitions it was reasoned that either the residual values were in error or else they recovered after some unknown period of time. Otherwise many of the older pavements would have settled several feet under the action of the traffic.

Tests were then conducted to determine the extent to which the residuals actually existed. This was accomplished by varying the manner in which the load vehicle traveled over the beam toe. Three test cycles, designated as A, B and C in Figure 5, were used at several different test sites. With Test Cycle A, the usual manner of obtaining deflections, a real difference was found between the initial and final extensometer readings, thereby indicating a residual deformation. Test Cycle B was accomplished by driving the load vehicle forward in the normal manner to a distance of 15 ft past the beam toe and then returning the load vehicle to the original starting position. Normal values obtained on the forward pass indicated a residual deformation, but when the load vehicle returned to the starting position it was found that the original extensometer reading was unchanged even though two repetitions of the wheel load had passed over the beam toe. Several repetitions of the test cycle at each test point showed this relationship to be true in all cases except for those pavements classified as very poor. In these pavements, definite accumulative residual deformations were evident.

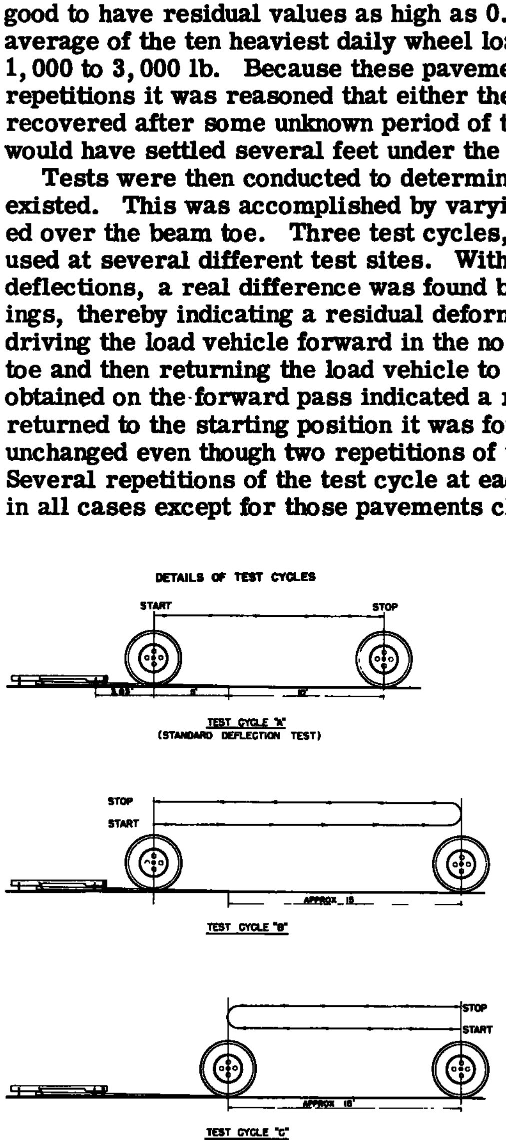

Figure 5. Plan of wheel movement for Test Cycles A, B, and C.

To check the trends noted in Test Cycle B another procedure—known as Test Cycle C—was developed. In this cycle, the load vehicle was started approximately 15 ft away from the beam toe, backed to a point slightly beyond the toe, and then returned to the starting position. Again residual deformation of the pavement surface was not evident even though 6 to 8 load cycles passed over the test points.

The only obvious reason for this discrepancy in residual deformations was that the usual placement of the load vehicle at the start of the test (Test Cycle A) caused sufficient downward movement of the front support and toe of the beam to bias the initial reading. It will be recalled that the Benkelman beam was designed on the basis that the deflected area of the pavement would not extend more than 4.5 ft from the center of the load wheel.

In the light of the results obtained from Test Cycles B and C, further tests were conducted to determine the length of pavement influenced by the load. In these tests the load vehicle was started approximately 20 ft away from the beam toe and backed slowly toward the beam. The position of the load wheel was marked at the first discernible movement of the beam extensometer and also when the extensometer indicated a movement of 0.002 in. at the beam toe (a movement of one division on the extensometer dial). Results for several test sites on four different test sections (Fig. 6) indicate much larger areas of influence than previously reported in literature (1, 2).

Although Test Cycles B and C eliminated most of the false residual values resulting from the long depressed areas, it was a difficult and time-consuming task to back the load vehicle over the probe arms of the beam. In addition, if a graphical recording was desired, a rather complicated pulley system would have been necessary to actuate the Helmer recorder on the back-up passes of Test Cycles B and C.

DATA CORRECTIONS

In order to use Test Cycle A and still use the results of the graphical recordings, a method was developed to correct the recordings based on measurements obtained from the recordings themselves. This method requires several assumptions which are discussed as follows:

1. The graph drawn by the recorder shows the deflection at the beam toe produced by a moving wheel load. If conditions of homogeneity can be assumed within the area influenced by the wheel load, the curve can also be considered as the deflection of the area produced by the wheel load at the instant it passes over the beam toe. From a

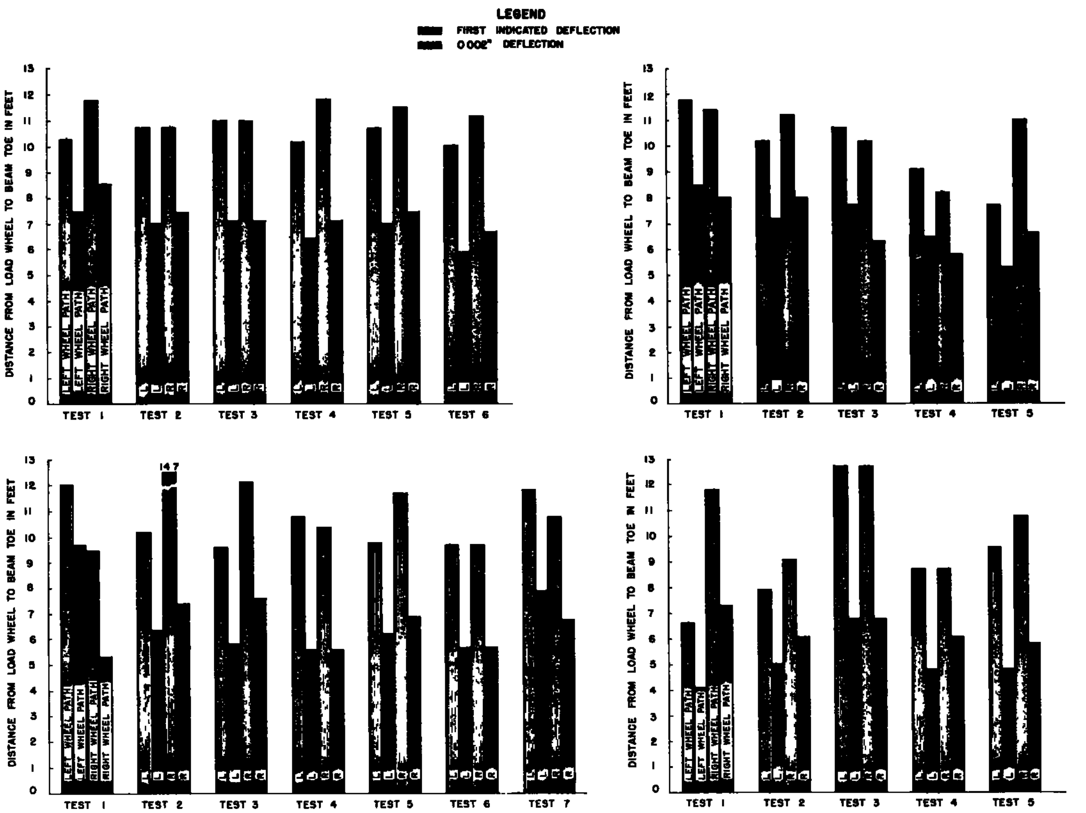

Figure 6. Average distances of load wheel from toe of beams at extensometer readings indicated.

practical standpoint the assumption of homogeneity appears reasonable for the small area in question.

2. Another assumption is that all points on the pavement deflect to their full value immediately upon application of the load and rebound almost immediately when the load is removed. Proof of this assumption has been determined as follows. At several points on the pavement during the performance of the standard deflection test (Test Cycle A) the load vehicle has been stopped and allowed to stand as long as 3 min. Each time the load vehicle came to a halt, the extensometer hand immediately stopped moving and remained constant until the load vehicle moved again. It has also been noted that the "stop" end of the recordings shows a level, straight line several inches long. This could not have occurred if the beam toe continued vertical movement as the load vehicle reached the end of the cycle.

3. A further assumption, somewhat related to that listed previously, is that no residual deformation occurs under the action of a single pass of the load vehicle. On the basis of the results obtained from Test Cycles B and C, it appears that this assumption is valid for most pavements. For those pavements that show a real and definite residual deformation, this method cannot be used.

4. The final assumption is that the deflection curve produced by a slowly moving wheel load is symmetrical on both sides of the wheel. It would at first appear from the deflection curves shown in Figure 4 that this assumption is incorrect, however these curves actually are asymmetrical because of the bias caused by downward movement of the front beam support. In other words, at the same time the load wheel is moving closer to the beam toe and causing it to deflect, the front beam support is rising. This interaction causes the graphical recording to be incorrect as long as the front support is within the deflected area. A further check on this assumption was made by limited tests using the Helmer recorder in Test Cycle C. A comparison of the back-up and forward graphs showed that regardless of the direction of travel of the test vehicle the graphs become symmetrical at the point where the front support was no longer in the deflected area.

On the basis of the foregoing assumptions, the deflections were corrected by measurements taken from the recordings made with Test Cycle A. The first step was construction of a base line from the "stop" end of the recording parallel to the top edge of the paper. The scaled distance from the "trough" of the deflection curve to the point where the deflection curve becomes level (or intersects the base line) represents one-half of the length of the deflected area. If this distance was less than the distance from the load wheel to the front supports or beam toe at the beginning of the test then neither the front supports nor the toe were in the deflected area at the beginning of the test. In these cases there are no residual deformations (except in the case of very poor pavements) and it is not necessary to correct the Helmer recordings.

If one-half of the deflected area was found to be greater than 3.83 ft but less than 8.9 ft then both the toe and front supports of the beam were in the deflected area at the beginning of the test. In this case the initial portion of the curve from the "start" point to the deflection trough was incorrect. It is possible to geometrically determine a correction for the initial reading in this case, but its use is not warranted inasmuch as the curve is correct at least from the "trough" to the "stop" end.

If one-half of the deflected area was greater than 8.9 ft, then not only was the initial reading in error, but the maximum reading also was in error. In this case it was necessary to correct the maximum Helmer recording value by a geometrical process. The first step in this process was to determine from the recording graph the distance that the front support was depressed at the time of the maximum reading. This was done by measuring 8.9 in. from the "trough" of the deflection curve along the base line. At this point (which corresponds to a distance of 8.9 ft away from the beam toe on the pavement) the measured vertical distance from the base line to the deflection curve represents the front support movement when the load wheels are at the beam toe.

The influence of downward front support movement on the maximum deflection was determined by rather simple geometrical means and the results are shown in Figure 7.

Front support movements of 0.013 in. have been recorded on Texas highways. This means that corrections as high as 0.038 in. have been required.

As an aid in extracting the dimensions from the recordings, the lucite template shown in Figure 8 was made. On the template, scaled dimensions of the Benkelman beam are etched on a horizontal base line. The vertical scales allow direct measurements of deflections and front support movements.

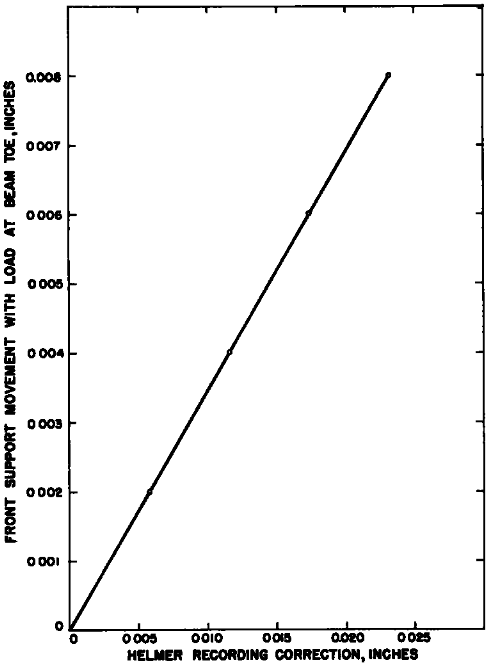

Figure 7. Correction to be added to deflections obtained from the Helmer graphical recorder when the front support is depressed at time of maximum deflection.

The use of the Helmer recorder with the Benkelman beams prompted another investigation to determine the reliability of recorders. Previous tests had been performed on the beams by placing gage blocks of known thicknesses under the beam toes. To check the recorders it was felt that the movement of the beam toes should duplicate that occurring during actual testing. This was satisfied by placing the beam toes on the lower platen head of a Universal testing machine (Fig. 9). As the recorder was actuated to reel out the graphing paper, the beam toe was lowered and raised again to the initial position. The testing machine operator was quickly able to reproduce curves like those obtained in the field. A comparison between the deflections measured from the recordings and the actual movements of the beam toes is shown in Figure 10 for both the right (outer wheel path) and left (inner wheel path) beams.

Because of wear, looseness of parts in the recorder or faulty adjustments, these corrections change often and it is necessary to periodically check the beams. This has been accomplished satisfactorily in the field by the use of the apparatus shown in Figure 11. The beam toe is placed on the hinged lever which may be moved up or down with a fine thread bolt. As the Helmer recorder is placed in

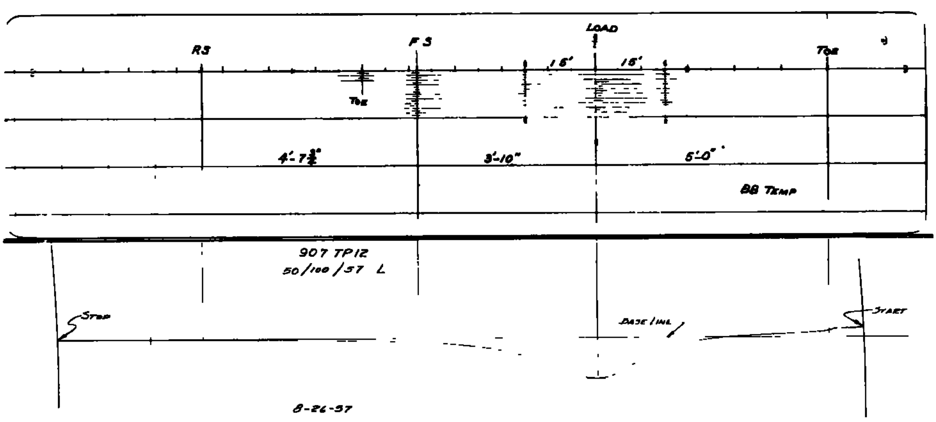

Figure 8. Template used to simplify measurements from graphical recordings.

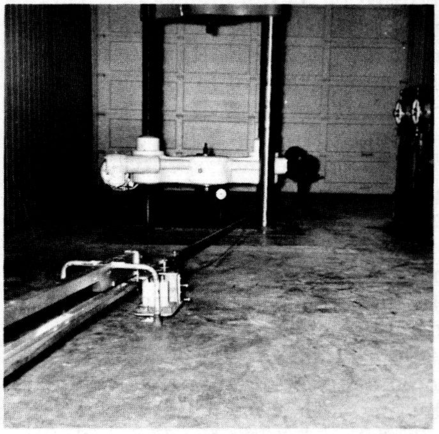

Figure 9. Laboratory set-up used to calibrate Benkelman beams and Helmer recorder.

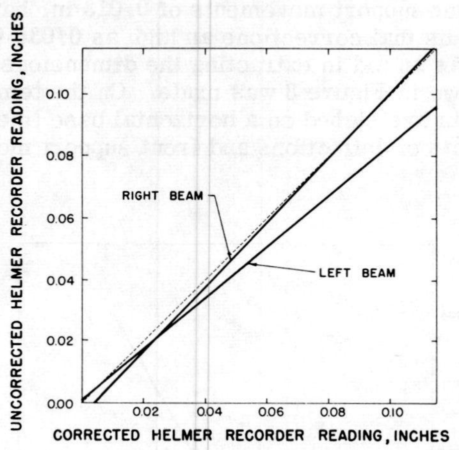

Figure 10. Corrections to be applied to Helmer recordings as a result of laboratory calibrations.

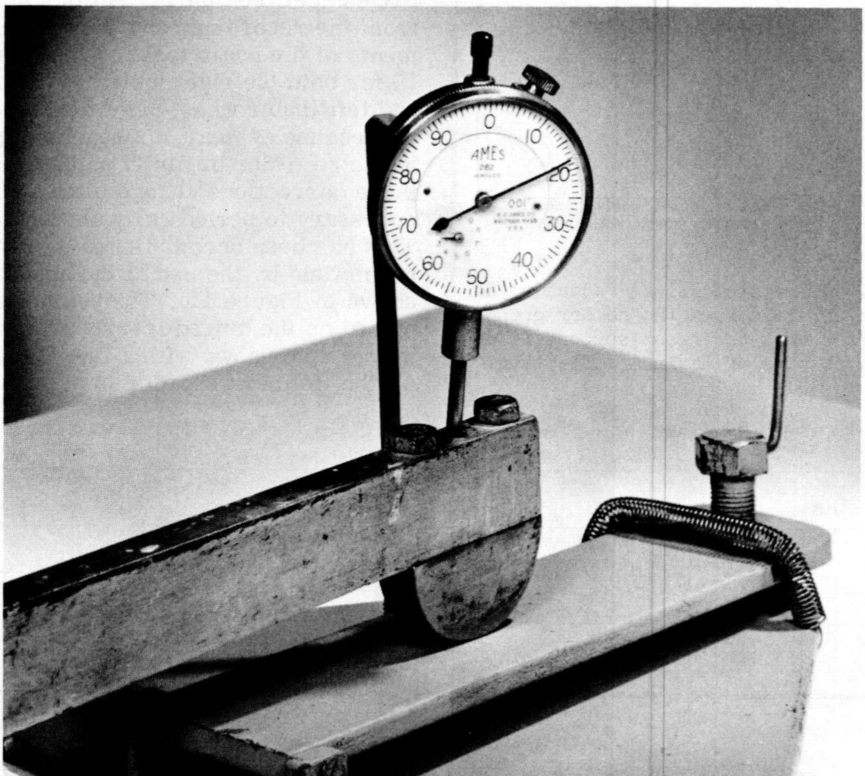

Figure 11. Apparatus for checking Benkelman beam calibration in the field.

motion, the lever is lowered and then raised to the initial position. The movement of the beam toe, as measured by the extensometer mounted on the stationary base plate, is compared to the deflection measured from the graphical recording. When this comparison shows a significant variation from the correction charts, the beams and recorders are examined for looseness of parts, etc. If necessary the beams are returned to the shop for cleaning and adjustment, which is then followed by another laboratory calibration. Experience has shown that the recorders easily get out of adjustment and field checks should be performed several times daily.

Another factor to be considered, although not directly associated with the accuracy of the Benkelman beam, is the effect of temperature on flexible pavement deflections. This could be particularly important when comparing deflections at a certain point during various times of the year or even at various times of the day. To check this effect, two pavement sections were selected. One section was surfaced with a single asphalt surface treatment and the other was surfaced with 4 in. of hot-mix asphaltic concrete. At several test points on each pavement at least 4 separate deflection and temperature tests were obtained between dawn and dusk. Temperatures varied between 75 F and 119 F which corresponded nicely to the over-all temperature variation of 70 F to 140 F noted during previous deflection tests. It was suspected that if any temperature effect was present it would be more noticeable in the asphaltic concrete section than in the surface treatment. The results given in Table 1 indicate a general but erratic tendency on both sections for the deflections to increase as the temperatures increase. The correlation is rather poor for this limited data and it appears that other unknown variables are affecting this data. As a matter of interest it is pointed out that pavement deflections at the WASHO Road Test were not significantly affected by temperatures above 70 F (5). This relationship may not hold true for other pavements, and it appears that more research must be undertaken on the temperature-deflection relationship for a wide range of pavement surfaces and thicknesses.

TEST RESULTS

After the deflection testing program was completed on the original 500 mi of highway, it became apparent that it would be extremely difficult and expensive to collect the information on all of the significant variables involved. As a result, several of the test sections were eliminated. The remaining sections (totaling approximately 117 mi) were selected to have a maximum variation in age, traffic characteristics, pavement cross-sections, and construction materials. Determination of the effect of each of these variables on surface deflections will still involve a major statistical analysis. Although it is too early at the present time to predict general trends or values, there are a few specific points considered worthy of mention.

Table 2 shows the results of deflected length measurements for 1,114 tests taken during the summer of 1959. The values were obtained for a 9,000-lb wheel load. It is seen that the deflected length was longer than 16 ft for about 50 percent of the test points. Also, about 82 percent of the test points had deflected lengths greater than 10 ft. In these instances the initial Benkelman beam readings were biased to the extent that erroneous residual deformations were indicated. Approximately 40 percent of the test points had deflected areas greater than 18 ft thereby indicating that the maximum deflection also required correction.

Attempts to correlate the deflected length with pavement behavior have been unsuccessful. This is probably a result of the many factors affecting this length. One such factor is given in Table 3 which presents the deflected lengths for 5 adjacent test sections. These 5 sections, each 250 ft long, were constructed for a soil stabilization study. The sections are alike in every manner except that the top 6 in. of the subgrade in each section is stabilized with a different type or amount of stabilizer.

Shortly after the test section was opened to traffic it was used as a haul road for a nearby construction project. Several hundred heavily-loaded gravel trucks traveled on the west-bound lane and returned empty on the east-bound lane. The high wheel loads caused a significant increase in the deflected length of the west-bound lane. This would indicate that an increase in density also increases the deflected length of the pavement.

TABLE 1

PAVEMENT TEMPERATURES AND CORRECTED DEFLECTIONS[1] FOR A SERIES OF FOUR TEST REPETITIONS

Test Point	Wheel Path							
	Inner	Outer	Inner	Outer	Inner	Outer	Inner	Outer
	(a) Test Section 903[2]							
27	77	77	88	88	102	102	108	108
	(11)	(14)	(16)	(14)	(15)	(11)	(14)	(15)
28	75	75	90	90	104	104	110	110
	(10)	(13)	(17)	(11)	(15)	(9)	(19)	(38)
29	76	76	87	87	105	105	113	113
	(22)	(10)	(22)	(18)	(22)	(14)	(23)	(35)
30	77	77	91	91	108	108	114	114
	(14)	(21)	(20)	(19)	(17)	(16)	(20)	(40)
32	76	76	90	90	109	109	114	114
	(19)	(13)	(24)	(14)	(19)	(8)	(21)	(23)
33	75	75	93	93	109	109	112	112
	(17)	(16)	(38)	(30)	(38)	(23)	(37)	(32)
34	76	76	95	95	111	111	111	111
	(12)	(12)	(24)	(7)	(20)	(10)	(18)	(18)
35	77	77	94	94	108	108	111	111
	(18)	(10)	(21)	(10)	(15)	(12)	(18)	(15)
38	77	77	96	96	113	113		
	(12)	(10)	(22)	(11)	(20)	(12)		
39	77	77	93	93	111	111	113	113
	(7)	(8)	(18)	(7)	(27)	(14)	(30)	(18)
40	79	79	96	96	119	119	116	116
	(25)	(18)	(31)	(8)	(25)	(35)	(27)	(10)
41	81	81	101	101	112	112	113	113
	(22)	(19)	(25)	(16)	(25)	(15)	(29)	(30)
42	81	81	98	98	107	107	112	112
	(16)	(8)	(17)	(9)	(23)	(20)	(22)	(26)
43	81	81	98	98	102	102	106	106
	(22)	(20)	(25)	(16)	(21)	(14)	(46)	(36)
44	80	80	97	97	106	106	107	107
	(24)	(28)	(33)	(27)	(35)	(27)	(34)	(32)
46	81	81	99	99	106	106		
	(24)	(16)	(24)	(14)	(29)	(14)	(26)	(32)
47	81	81	95	95	105	105	111	111
	(14)	(16)	(11)	(9)	(18)	(12)	(13)	(30)
	(b) Test Section 905[3]							
148	79	79	83	83			97	97
	(22)	(26)	(24)	(29)			(28)	(36)
149	79	79	85	85	100	100	97	97
	(30)	(42)	(32)	(36)	(35)	(45)	(39)	(52)
150	79	79	84	84	101	101	100	100
	(37)	(44)	(29)	(40)	(27)	(41)	(41)	(54)
151	78	78	86	86	98	98	101	101
	(32)	(32)	(22)	(19)	(53)	(43)	(41)	(40)
152	78	78	87	87	99	99	101	101
	(29)	(35)	(26)	(39)	(31)	(34)	(40)	(33)
153	78	78	86	86	102	102	102	102
	(27)	(30)	(16)	(32)	(28)	(34)	(27)	(29)
154	78	78	87	87	103	103	102	102
	(29)	(35)	(38)	(36)	(28)	(35)	(38)	(29)
155	78	78	87	87	102	102	99	99
	(24)	(18)	(22)	(26)	(30)	(24)	(26)	(31)
156	79	79	89	89	102	102	102	102
	(31)	(29)	(21)	(62)	(34)	(30)	(35)	(38)
157	79	79	89	89	98	98	107	107
	(34)	(41)	(25)	(40)	(33)	(44)	(43)	(49)
158	78	78	89	89	99	99	107	107
	(29)	(25)	(30)	(53)	(43)	(42)	(36)	(35)
159	79	79	93	93	97	97	103	103
	(26)	(17)	(28)	(39)	(29)	(30)	(15)	(19)
160	79	79	93	93	99	99	106	106
	(29)	(41)	(30)	(49)	(31)	(49)	(22)	(33)
161	79	79	93	93	99	99		
	(31)	(26)	(33)	(33)	(38)	(29)	(37)	(27)
162	79	79	95	95	98	98		
	(26)	(41)	(27)	(57)	(29)	(47)	(38)	(50)

[1] Values in parentheses are corrected deflections, in 10^{-3} in.
[2] Four-inch hot-mix asphaltic concrete surface.
[3] Single asphalt surface treatment.

TABLE 2

PERCENTAGE OF TESTS FALLING WITHIN VARIOUS INTERVALS OF DEFLECTED LENGTH OF PAVEMENTS

Test Section No.	Percentage of Tests Falling Within Deflected Lengths Shown — Deflected Length, ft															Total Test Points
	2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	22-24	24-26	26-28	28-30	30-32	
201	0	0	0	11.5	11.5	19.3	19.3	3.8	3.8	15.4	3.8	7.8	3.8	0	0	26
204	0	5.0	5.7	9.3	14.3	14.3	12.8	10.0	11.4	9.3	3.6	2.9	1.4	0	0	140
205	0	3.0	9.0	13.4	13.4	11.9	10.4	4.5	6.0	7.5	11.9	7.5	1.5	0	0	67
207R	0	1.2	8.1	11.6	14.0	23.1	7.0	14.0	8.1	4.7	3.5	1.2	3.5	0	0	86
207L	0	3.8	11.5	7.8	3.8	15.4	19.4	3.8	7.8	11.5	3.8	11.5	0	0	0	26
901	1.6	0.8	0.8	3.2	7.2	9.6	16.0	17.6	12.8	11.2	10.4	7.2	1.6	0	0	125
902	0	0	1.1	1.1	4.4	5.5	7.7	12.1	9.9	19.7	8.8	16.5	11.0	1.1	1.1	91
903	14.1	9.1	8.1	16.2	16.2	10.1	6.1	2.0	4.0	3.0	5.1	4.0	1.0	0	1.0	99
905	0.6	7.8	9.1	14.5	13.0	12.3	5.8	5.2	2.6	5.8	7.8	5.8	3.9	5.2	0.6	154
906	0	2.4	3.6	5.4	4.8	5.4	9.0	15.3	13.9	12.0	7.8	7.2	6.0	5.4	1.8	166
907	0	0	0	4.1	2.7	4.1	4.1	12.2	13.2	12.2	16.2	12.2	12.2	6.8	0	74
908	1.7	0	5.0	3.3	13.3	13.3	8.3	16.7	11.7	5.0	10.0	5.0	1.7	5.0	0	60
All Sections	1.6	3.3	5.1	8.4	10.2	11.1	9.5	10.7	9.2	9.4	7.8	6.8	4.1	2.3	0.5	
Acc. Percent	1.6	4.9	10.0	18.4	28.6	39.7	49.2	59.9	69.1	78.5	86.3	93.1	97.2	99.5	100	

TABLE 3

DEFLECTED LENGTHS FOR BOTH LANES OR SPECIAL TEST SECTIONS

Test Section	Test Point	Deflected Length, ft — East-Bound Lane O.W.P.	East-Bound Lane I.W.P.	West-Bound Lane I.W.P.	West-Bound Lane O.W.P.
I	1	17.4	15.4	-	16.6
	2	15.5	16.0	15.4	16.4
	3	17.0	16.2	18.4	24.2
	4	15.5	17.5	23.8	17.6
	Avg	16.4	16.3	19.2	18.7
II	5	19.8	18.0	18.2	17.2
	6	16.2	16.0	26.2	22.4
	7	23.6	20.2	16.2	16.0
	8	19.4	14.1	26.6	17.6
	Avg	19.8	17.1	21.8	18.3
III	9	12.8	13.0	18.8	-
	10	12.4	13.6	Inc	23.4
	11	11.0	13.0	14.4	19.0
	12	14.5	12.4	24.4	19.8
	Avg	12.7	13.0	19.2	20.7
IV	13	17.0	-	27.0	18.8
	14	20.3	17.2	24.6	20.6
	15	-	18.8	22.0	17.4
	16	14.7	18.4	26.0	-
	Avg	17.3	18.1	24.9	18.9
V	17	20.0	18.0	29.2	23.8
	18	16.2	14.4	19.4	-
	19	17.0	18.8	31.4	17.2
	20	18.5	17.6	25.0	17.5
	Avg	17.9	17.2	26.3	19.5

TABLE 4

DEFLECTION DATA FOR TEST SECTION—9,000-LB WHEEL LOAD

Test Point		Corrected Deflection, 10^{-3} in.	½ Deflected Length, ft	Deflection ÷ ½ Deflected Length, $\frac{in.}{ft}$	Pedological Soil Type
1		19.0	11.1	0.0017	Irving clay
	Avg.	19.0	11.1	0.0017	Irving clay
2		31.0	7.2	0.0043	Catalpa clay
	Avg.	31.0	7.2	0.0043	Catalpa clay
3		45.0	12.8	0.0035	Houston clay
4		50.0	11.4	0.0044	Houston clay
	Avg.	47.5	12.1	0.0039	Houston clay
5		29.5	6.3	0.0047	Wilson clay
6		17.0	7.5	0.0023	Wilson clay
7		36.0	12.4	0.0029	Wilson clay
8		18.0	7.2	0.0025	Wilson clay
9		23.0	9.9	0.0023	Wilson clay
10		32.0	8.1	0.0040	Wilson clay
11		30.0	8.3	0.0036	Wilson clay
12		32.0	-	-	Wilson clay
13		26.0	8.5	0.0031	Wilson clay
14		26.0	10.8	0.0024	Wilson clay
15		20.0	10.0	0.0020	Wilson clay
16		33.0	9.0	0.0037	Wilson clay
17		25.0	13.8	0.0018	Wilson clay
	Avg.	26.7	9.3	0.0029	Wilson clay
18		31.0	12.7	0.0024	Irving clay
19		15.0	6.1	0.0025	Irving clay
20		34.0	10.4	0.0033	Irving clay
21		25.0	9.8	0.0026	Irving clay
22		29.0	12.3	0.0024	Irving clay
23		36.0	-	-	Irving clay
24		44.0	8.8	0.0050	Irving clay
25		28.0	10.3	0.0027	Irving clay
26		11.0	5.9	0.0019	Irving clay
27		31.0	12.5	0.0025	Irving clay
28		21.0	6.7	0.0031	Irving clay
	Avg.	27.7	9.6	0.0028	Irving clay
29		29.0	12.1	0.0024	Bell clay
	Avg.	29.0	12.1	0.0024	Bell clay
30		27.0	10.3	0.0026	Irving clay
31		32.0	10.6	0.0030	Irving clay
32		24.0	8.4	0.0029	Irving clay
	Avg.	27.7	9.8	0.0028	Irving clay
33		21.0	6.2	0.0034	Lewisville clay
	Avg.	21.0	6.2	0.0034	
34		53.0	11.0	0.0048	Houston clay
35		69.0	12.0	0.0058	Houston clay
36		53.5	-	-	Houston clay
37		39.0	10.3	0.0038	Houston clay
38		34.0	11.4	0.0030	Houston clay
	Avg.	49.7	11.2	0.0044	Houston clay
39		39.0	12.9	0.0030	Wilson clay
40		30.0	11.7	0.0026	Wilson clay
41		25.0	7.1	0.0035	Wilson clay
42		30.0	9.0	0.0033	Wilson clay
43		30.0	10.4	0.0029	Wilson clay
	Avg.	30.8	10.2	0.0030	Wilson clay
44		40.0	9.3	0.0043	Houston black clay
45		36.0	5.4	0.0067	Houston black clay
46		40.0	9.6	0.0042	Houston black clay
47		33.0	11.9	0.0028	Houston black clay
	Avg.	37.3	9.1	0.0051	
Grand Avg.		31.5	9.8	0.0032	

The effect of subgrade soil type on both the deflected length and the corrected deflection is given in Table 4. The data were obtained from a pavement with a cross-section of 12 in. of crushed limestone base and a single asphalt surface treatment over the prepared native subgrade. It can be seen that there is a very close relationship between similar pedological soil types. This is best shown by a measure of the curvature of the surface similar to the "index ratio" used at the WASHO Road Test (1). In Table 4, curvature is expressed as a ratio of the deflection in inches to one-half of the deflected length in feet. For the Wilson clay between test points 5 and 17 this ratio is 0.0029 in. per foot and between test points 39 and 43 the ratio is 0.0030 in. per foot. Similar close relationships are noted for the Houston clay and Irving clay. Because of this relationship it should be possible to evaluate pavements on various subgrades by the use of the pedologic classification rather than the much more expensive method of sampling and testing the subgrade soil. This method will undoubtedly be limited to those areas of the pavement where the depth of fill is insignificant.

SUMMARY

The increased use of the Benkelman beam in pavement evaluations makes it necessary that the limitations and accuracy of the equipment be known. A significant and misleading error in deflection values occurs when the front beam supports are within the deflected length of the pavement during the test cycle. Test results are presented showing that the deflected lengths obtained on 117 mi of Texas highways were significant enough to require correction of the deflections for a large percentage of the tests. Means have been given for correcting the deflections when a recorder attachment is used on the Benkelman beam. Examples are also given which show the value of the deflected length measurements obtained from the recordings.

ACKNOWLEDGMENTS

The authors are deeply indebted to the many people in the Texas Highway Department who have contributed to this project. Thanks are also due to Tom J. Kelly, formerly Project Supervisor, Texas Transportation Institute, and to Kirby Meyer, Texas Transportation Institute, who was responsible for many of the evaluation tests reported.

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Analysis of Stresses in Flexible Pavements and Development of a Structural Design Procedure

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A procedure is developed for the analysis and design of flexible pavements, based on the premise that the lateral stress produced in the pavement structure by surface loads must not exceed the passive lateral resistance provided by the components of the pavement structure. Stress constants of cohesion, c, and angle of internal friction, ϕ, obtained from a strength envelope for the flexible paving material developed from triaxial or direct shear test data, are the predominant factors influencing the resistance of the paving material to deformation. The procedure is applicable to the stability analysis of bituminous mixtures or to the thickness design of the pavement. Examples are given to illustrate the application of this design procedure.

●THERE ARE many factors influencing the design of flexible pavements. Included among the design considerations should be (a) the shearing resistance of the components of the total pavement structure for both dynamic and static loading, (b) the stresses produced both at the surface and within the pavement by surface wheel loads, and (c) the resistance of the pavement structure to the effects of moisture, frost action, and abrasion from vehicle tires. The author feels that the first two of these could and should be based on an analysis of the stresses produced in the components of the pavement structure and of the resistance offered by the pavement to these stresses. It is realized that changes in moisture content will influence the strength properties of subgrade, subbase, and base materials in which a soil binder functions. It is therefore necessary that the strength properties of these materials be determined for the material in its critical service condition. The control of frost action in the pavement must come from a control on the placement of frost susceptible materials and a control of moisture, with consideration to both surface and subsurface water. This paper presents a method for flexible pavement design which gives primary consideration to the theoretical analysis of the strength properties of the pavement materials.

STRESSES IN PAVEMENT

Load Over Circular Area

The vertical intensity of stress along the axis below a circular loaded area has been determined by integration of unit loads from the Boussinesq point load equation over the surface area to be (1)

$$p_z = p\left[1 - \left(\frac{1}{1 + \left(\frac{a}{z}\right)^2}\right)^{3/2}\right] \tag{1}$$

in which

p = intensity of contact pressure,

a = radius of equivalent circular area,
z = vertical distance, and
p_z = vertical stress at depth z.

Eq. 1 gives the intensity of vertical stress below a circular load due to a surface contact pressure. The total vertical stress at some depth below the pavement surface would include the unit stress due to the weight of the overlying material, and is included in this analysis.

The Boussinesq equation, modified for loading over a circular area, may be employed to determine the maximum vertical stress intensity within a flexible pavement structure, as has been demonstrated by load tests (2).

Pavement Resistance

An element of soil triaxially loaded will provide resistance to failure in accordance with the expression (3)

$$p_z = p_x \tan^2\left(45^\circ + \frac{\phi}{2}\right) + 2c \tan\left(45^\circ + \frac{\phi}{2}\right) \qquad (2)$$

If it is assumed that p_x is equal to the passive lateral pressure provided by a wedge of soil subjected to a vertical load equal to the overlying weight of pavement (Fig. 1), then (3, p. 237)

$$p_x = \gamma z \tan^2\left(45^\circ + \frac{\phi}{2}\right) + 2c \tan\left(45^\circ + \frac{\phi}{2}\right) \qquad (3)$$

and, by substitution

$$p_z = \left[\gamma z \tan^2\left(45^\circ + \frac{\phi}{2}\right) + 2c \tan\left(45^\circ + \frac{\phi}{2}\right)\right] \tan^2\left(45^\circ + \frac{\phi}{2}\right) + 2c \tan\left(45^\circ + \frac{\phi}{2}\right)$$

Since $\tan^2\left(45^\circ + \frac{\phi}{2}\right) = \frac{1 + \sin\phi}{1 - \sin\phi}$, the above becomes

$$p_z = \gamma z\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^2 + 2c\left\{\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^{\frac{1}{2}} + \left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^{\frac{1}{2}}\right\}$$

$$= \gamma z\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^2 + 2c\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^{\frac{1}{2}}\left\{1 + \left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]\right\}$$

$$= \gamma z\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^2 + 2c\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^{\frac{1}{2}}\left\{\frac{1 - \sin\phi + 1 + \sin\phi}{1 - \sin\phi}\right\}$$

$$= \gamma z\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^2 + \frac{4c}{1 - \sin\phi}\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^{\frac{1}{2}} \qquad (4)$$

In the development of Eq. 4 confinement due to the wheel load itself has been neglected as such confinement would tend to reduce the effective passive lateral pressure of the pavement component. It is considered that the maximum lateral resistance to deformation which can be developed is the passive lateral pressure mobilized from the surcharge effect of the pavement layer only.

It will be noted that the resistance formula as developed here is for strip loading whereas the equation for vertical stress is for circular loading and possible circular failure. This would represent a conservative estimate of the resistance provided, except for moving loads where failure would approach that for strip loading, as has been observed from wheel tracking.

DESIGN PROCEDURE

The Basic Formula

Eq. 1, modified to include unit stress from overlying material, gives the intensity of vertical stress at a point beneath the center of a circular area acted on by a load "p" per unit area. Eq. 4 gives the resistance which must be developed in the pavement at that point if failure of the element is prevented. Failure is assumed to occur along a plane within the element at an angle of $(45^\circ + \frac{\phi}{2})$ with the pavement surface, and the wedge resisting this action slips along a plane making an angle of $(45^\circ - \frac{\phi}{2})$ with the pavement surface, as indicated in Figure 1. Equating Eq. 1, as modified, and Eq. 4 gives the basic structural design formula

$$\gamma z + p\left[1 - \left(\frac{1}{1 + \left(\frac{a}{z}\right)^2}\right)^{3/2}\right] = \gamma z\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^2 + \frac{4c}{1 - \sin\phi}\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^{\frac{1}{2}} \qquad (5)$$

in which

- p = surface contact pressure,
- a = radius of equivalent circular contact area,
- z = thickness of flexible pavement structure,
- γ = bulk density (unit weight) of surcharge material,
- ϕ = angle of internal friction of bearing material, and
- c = cohesion of bearing material.

Eq. 5 is applicable to any layer of the flexible pavement structure. For instance, if $z = 0$, the equation could be applied to asphaltic surfaces, and would be reduced to

$$p = \frac{4c}{1 - \sin\phi}\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^{\frac{1}{2}} \qquad (6)$$

Pavement Failure

Surface loads produce a squeezing action of the flexible pavement surface layer. The critical area at the tire contact with the pavement is at the perimeter of the surface contact, and failure is produced by forcing the resisting wedge to slip. Actually, failure is a relative thing, as applied to flexible pavements, and vertical stresses of intensity lower than that for massive slippage may cause some deformation within the structure. This may be observed in the triaxial test where considerable deformation of a specimen may occur prior to the actual development of a failure plane, or before a peak strength is reached. It is assumed in this analysis that the passive lateral resistance provided by the components of the flexible pavement structure will prevent element or wedge slippage. Consequently, the number of repetitions of loads of magnitude lower than that required for slippage may cause pavement deformation or distress.

Study by Paquette

It is interesting to note that Eq. 6 is identical to that developed by McLeod (4) from the geometry of the Mohr diagram for the stability design of bituminous

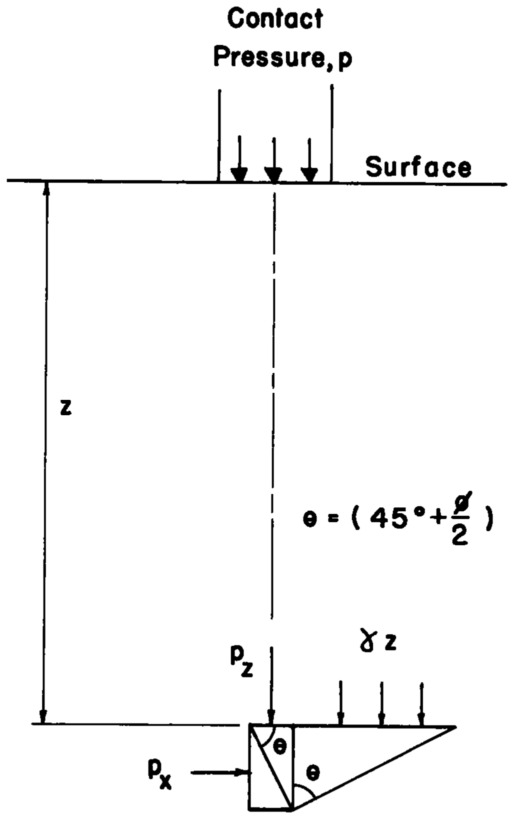

Figure 1. Stresses on an element of flexible pavement below a wheel load.

mixtures. This equation has been found by Paquette (5) to give a rather good correlation with results of the Marshall and Hveem stabilometer methods for contact pressures of 100 psi, which is near the maximum required for highway design. Paquette tested six samples at each of the four asphaltic contents for seven different gradations of aggregate, two for the Marshall Method, and four for the Hveem Method. Values of ϕ were determined from the Hveem stabilometer test data and c from the cohesiometer test data for the computation of "p" from formula 6. A summary of test results for one gradation of aggregate (the ideal gradation for New York State specification 1-A mixture) is extracted from Paquette's thesis and given in Table 1.

TABLE 1

EVALUATION OF IDEAL GRADATION MIXTURE FOR NEW YORK STATE SPECIFICATION 1-A MIX[1]

	Percent Asphalt Cement			
	4	5	6	7
Marshall:				
Voids 2 - 8%	8.85	7.17	4.93	1.27
Flow < 16	13	16	15	23
Stability > 1000	2414	1788	1906	2051
Preference	-	2	1	-
Hveem:				
Voids 3 - 8%	8.85	7.17	4.93	1.27
Cohesion > 250	244	361	368	429
Stability > 25	25.1	32.9	27.2	14.8
Preference	-	1	2	-
Stability Formula:				
Voids 2 - 8%	8.85	7.17	4.93	1.27
Stability 100	131	248	220	142
Preference	-	1	2	-

[1] From Paquette.

Table 2 presents data relating to angle of internal friction, cohesion, and stability which were used by Paquette to develop the summary in Table 1.

There are two aspects of the test procedure and computations which need clarification. First, all samples were compacted using the Marshall method of compaction. This procedure has been found to give lower stability values when using the Hveem stabilometer test procedure than when the kneading compactor is used for the preparation of samples. This fact is believed to be the reason for the relatively low values (as given in Table 1) for Hveem stability. Second, Paquette considered the extreme fiber stress in flexure to be a better measure of cohesion than the intercept of the Mohr diagram and used this in his computations for stability, employing Eq. 6. It will be noted from Table 2 that there is not a great difference between the values for cohesion by the two methods.

Comparison With Smith Triaxial Method

A chart for determining the suitability of bituminous mixtures tested by the Smith triaxial method has been developed (6). The author has superimposed curves on this chart for various contact pressures (Fig. 2) using Eq. 6 for the computations. Also, a curve for ultimate failure as determined by the log spiral (McLeod) analysis (7) for a contact pressure of 200 psi is presented. It should be pointed out that the Smith

TABLE 2

FRICTION, COHESION, AND STABILITY DATA[1]

Percent Asphalt Cement	Friction Angle, ϕ	Cohesion, C			Stability			
		Mohr Diagram	Cohesiometer, gm in.	Cohesiometer, psi	Marshall lb	Marshall Flow	Hveem, %	Equation, psi
4	-	-	-	-	2733	12	-	-
	-	-	-	-	2095	14	-	-
	41.4	6	232	5.5	-	-	29.8	144
	39.4	5	260	6.3	-	-	25.0	146
	34.1	6	232	5.5	-	-	24.4	94
	38.9	6	252	6.2	-	-	21.3	139
5	-	-	-	-	-	-	-	-
	43.0	6	397	9.4				
	42.8	6	350	8.3	-	-	37.4	238
	-	-	-	-	1625	17	-	-
	-	-	-	-	1950	15	-	-
	42.6	7	336	8.3	-	-	29.2	234
6	38.4	5	393	9.6	-	-	31.6	210
	-	-	-	-	2175	17	-	-
	32.3	8	382	8.9	-	-	24.8	134
	45.1	7	354	9.1	-	-	25.7	303
	-	-	-	-	1637	13	-	-
	41.0	9	345	8.7	-	-	26.4	232
7	25.1	9	498	12.3	-	-	13.1	134
	14.7	8	453	11.6	-	-	6.2	81
	-	-	-	-	2233	24	-	-
	-	-	-	-	1870	22	-	-
	29.7	8	464	11.1	-	-	20.1	152
	42.1	7	302	7.4	-	-	20.7	202

[1] From Paquette.

triaxial method specifies testing at 75 F, whereas the work of Paquette and others testing by the Marshall and Hveem procedures employ 140 F as a test temperature.

An interesting observation of the Smith triaxial evaluation chart is that all mixtures having an angle of internal friction below 25 are unsatisfactory for flexible pavement surfaces. Also, inasmuch as most asphaltic concrete mixtures have a value of cohesion between 5 and 15 psi, the chart indicates the need for a high angle of internal friction, approaching 45 degrees.

It is apparent from this study that many asphaltic mixtures which have performed well under existing service conditions on our highways may not have performed so satisfactorily under contact pressures of 200 or 300 psi, which is a requirement imposed on asphaltic surfaces with modern high pressure plane tires. It is also apparent that the answer to greater stability in asphaltic surfaces must come from an increase in the resistance to deformation by greater cohesion in the asphaltic mixture. Many of the asphaltic mixtures which meet present standards of design have an angle of internal friction approaching 45 degrees. A small increase in cohesion to such a mixture will produce a considerable increase in stability (Table 3).

Thickness Design

The basic thickness design formula (Eq. 5) may be expressed

$$pQ = \gamma z\ (R-1) + cS \qquad (7)$$

in which

$$Q = \left[1 - \left(\frac{1}{1 + \left(\frac{a}{z}\right)^2}\right)^{3/2}\right]$$

$$R = \left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^2$$

$$S = \frac{4}{1 - \sin\phi}\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^{\frac{1}{2}}$$

TABLE 3

INFLUENCE OF COHESION ON STABILITY (BY COMPUTATION FROM EQ. 6)

Cohesion, c, psi	Stability, p, psi	
	$\phi = 30^0$	$\phi = 45^0$
5	70	165
10	140	330
15	210	495
20	280	-

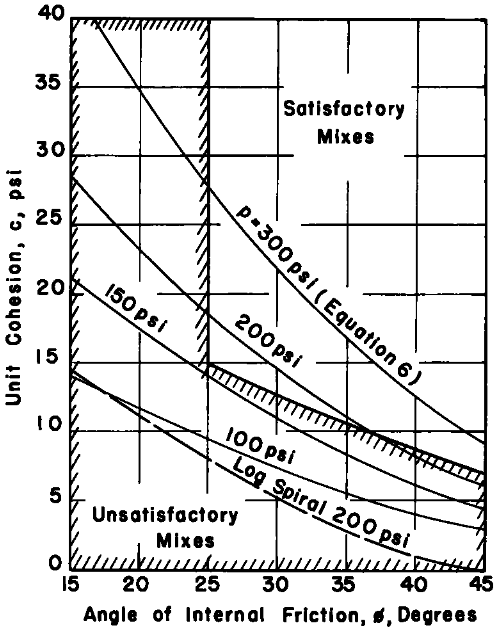

Figure 2. Smith closed-system triaxial compression test evaluation chart for asphaltic concrete, with curves superimposed for other stability equations.

Values of Q have been computed for a number of values of the ratio $\left(\frac{z}{a}\right)$ by Barber (8) and are presented in both graphical and tabular form in Figure 3. Values of R and S have been computed by the writer for a number of values of angle of internal friction (ϕ) and are shown in Figure 4 in both tabular and graphical form.

Eq. 7 may be solved by trial for the determination of thickness for a given design situation. A few examples follow.

Example 1

Given: P $= 12,000$ lb
p $= 90$ psi
$\phi = 10^0$
c $= 2$ psi

Determine thickness of cover required to protect given subgrade.

$$\frac{\pi d^2}{4} = \frac{12,000}{90}$$
$$d^2 = 170$$
$$d = 13$$
$$a = 6.5 \text{ in.}$$

From charts:

$$R = 2.0$$
$$S = 5.78$$

Assume density of surcharge material to be 130 lb/cu ft = 0.075 lb/cu in.

Trial 1.

$$z = 15 \text{ in.}; \frac{z}{a} = \frac{15}{6.5} = 2.3, \text{ and } Q = 0.23$$

then $90 \times 0.23 = 0.075 \times 15 \times 1.0 + 2 \times 5.78$
$20.7 = 1.12 + 11.56 = 12.68$ ----- z is too small.

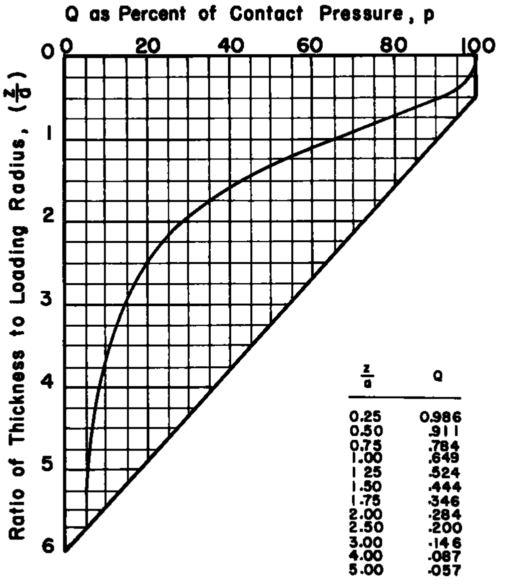

Figure 3. Vertical stress transmitted to a point on the axis in a semi-infinite mass from a surface load uniformly distributed over a circular area, expressed as a percent of surface load intensity.

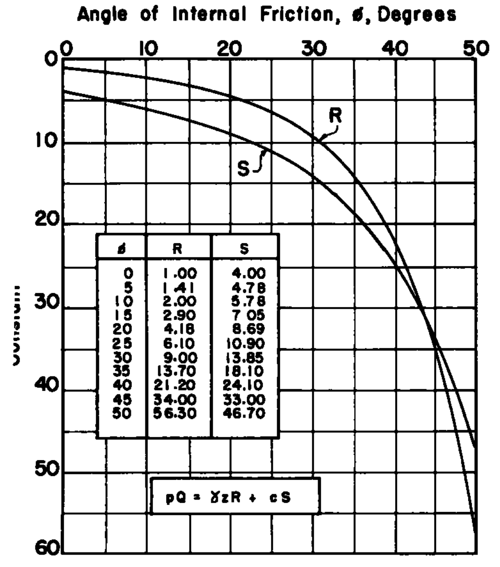

Figure 4. Values of "R" and "S" for use in flexible pavement design formula $pQ = \gamma zR + cS$, as a function of angle of internal friction, ϕ.

Trial 2.

$z = 20$ in.; $\frac{z}{z} = 3.08$, and $Q = 0.14$

then $90 \times 0.14 = 0.075 \times 20 \times 1.0 + 11.56$

$12.6 = 1.5 + 11.56 = 13.06$ ----- z is near correct value.

Trial 3.

$z = 19$ in.; $\frac{z}{a} = 2.92$, and $Q = 0.16$

then $90 \times 0.16 = 0.075 \times 19 \times 1.0 + 11.56$

$14.4 = 1.43 + 11.56 = 12.99$ ----- z is too small.

The design thickness of pavement structure over given subgrade would be 20 in., to the nearest inch.

	Examples				
	2	3	4	5	6
Load, P, lb	15,000	15,000	150,000	15,000	15,000
Pressure, p, psi	100	100	125	100	100
Radius, a, in.	6.9	6.9	19.6	6.9	6.9
Cohesion, c, psi	2.0	0	0	5.0	0
Friction angle, ϕ	0	30	10	40	40
Pavement thickness, z	28	22	42	0	5

The road materials in the foregoing examples might be generally classified as follows:

Example	Material Classification
1	Cohesive silt
2	Saturated soft clay
3	Cohesionless sand
4	Cohesionless silt
5	Asphalt concrete or crushed stone with binder
6	Crushed stone without binder

CONCLUSIONS

The basic design equation appears to give results in fairly close agreement with practice. It is evident, however, that in using this design equation a factor for abrasion from vehicle wheels is not included in the pavement resistance determination. On this basis, some granular materials with binder would satisfy the basic criteria for resistance to shear at the surface, as in example 5 for crushed stone; however, experience dictates that for roads of even moderate traffic the surface would not withstand the abrasion. Many soils may also be tested in a dry condition and show considerable shearing resistance but when subjected to the infiltration of water their shearing resistance would be reduced drastically.

There has been considerable discussion on the topic of stress distribution through various materials in a flexible pavement. Studies do show that in a layered system the degree of reduction of vertical stresses below the surface is a function of the relative strengths of the two materials. This would indicate that the vertical stress determined by the modified Boussinesq equation is not entirely correct. Without sufficient evidence to determine a modifying factor at this time, the author has eliminated this from the basic design equation. Should evidence in the future justify, the basic equation could be modified as follows:

$$pQD = \gamma z\ (R - 1) + cS \tag{8}$$

in which

D = a stress distribution factor.

Another factor deserving consideration in the design of the pavement would be the anticipated volume of heavy wheel loads and the degree of tracking to be expected on the completed structure. A few wheel loads moving along a fixed line may produce much more ultimate deformation in the pavement than many more load applications appropriately spaced over the pavement surface.

In the first case each application of load may add a small deformation to an already deformed section; whereas, in the second case each application of load may only knead the pavement such that it will retain a fairly smooth surface. As study is continued and experience is gained, the basic equation may be modified further as follows:

$$pQDT = \gamma z\ (R - 1) + cS \tag{9}$$

in which

T = a traffic factor.

Of particular importance in the design of bituminous mixtures is the rate of loading, both in testing operations and in field loading. It is possible that the shearing resistance for asphaltic materials as determined from Eq. 6 may be reduced for static loading and increased for dynamic loading, when the shear constants are determined in the laboratory by standard testing procedures. Eq. 6 may include a factor to adjust for the rate or type of loading, as follows:

$$p = \frac{4cL}{1 - \sin\phi}\left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^{\frac{1}{2}} \tag{10}$$

in which

L = a factor to adjust for rate of loading.

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