

# New Tests for Measuring Elasticity of Micaceous Subgrades and Their Application to Flexible Pavement Design

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This report summarizes a study (1) of flexible pavement performance conducted by Clemson College for the South Carolina Highway Department and the U. S. Bureau of Public Roads. The research was initiated in July 1960 to determine the cause of poor flexible pavement performance on several primary highway sections and to recommend procedures for the improvement of performance in future construction.

Routine classification, moisture and density tests of the subgrade as well as tests of the pavement and base were performed on sites of widely differing performance ratings. In addition to the routine tests, other special tests were also performed. Test results and pavement performance were then correlated with Benkelman beam deflection measurements taken at all test sites at different seasons for a period of two years.

• TWO HIGH-TYPE flexible pavement sections located in the Piedmont area of South Carolina (US 29 near Spartanburg and US 123 near Clemson) were selected for the study. Each section was about five miles long and consisted of a 10-in. uniform macadam base course surfaced with high-type asphaltic concrete when the study began. Both highway sections carried heavy truck traffic through an area where elastic, micaceous subgrades are commonly found.

After visually rating the pavement performance along the entire length of both highway sections, eighteen test sites (summarized in Table 1) were chosen to include areas of poor, good, and excellent performance.

The net pavement deflections (maximum and residual) in Table 1 were measured with a standard 12-ft Benkelman beam. A semi-trailer tractor unit with the single rear axle loaded to 18,150 lb supplied the load for these measurements.

## SPECIAL TESTS

There was no indication of instability or poor quality in the pavement or base material. It became apparent early in the investigation, however, that the sites of poor performance and high deflection were associated with subgrade soils containing a high percentage of mica. The presence of mica in some of the resilient soils tested is revealed by high liquid limit, low plasticity index, and A-5 classification. However, the coarser grained soils of this type will have a liquid limit of less than 40 and a classification of A-4 or A-2-4, the same as the highly desirable top soils of the Piedmont. The test method and design procedure that came closest to indicating poor performance and subgrade resilience was the Georgia Highway Department method discussed by Abercrombie (2). In this method, higher density soils having low volume change and low clay content are considered superior, whereas lighter soils with high volume change are resilient soils of inferior performance.

TABLE 1  
SUMMARY OF TEST SITES

Test Site	Performance Rating	Station	Profile	Deflection (in.)	Base and Pavement Thickness (in.)	Pavement Thickness (in.)
US 29						
1	Excellent	25 + 80	15-in. Fill	0.021	12.0	3.6
3	Poor	34 + 30	5-ft Cut	0.062	12.6	3.6
5	Poor	58 + 69	9-ft Fill	0.067	12.2	4.8
7	Excellent	210 + 50	4-in. Fill	0.023	13.7	3.8
8	Excellent	21 + 73	10-in. Fill	0.010	14.2	3.8
9	Poor	48 + 68	8-ft Cut	0.034	12.8	3.8
10	Good	109 + 18	3-ft Cut	0.019	12.5	4.3
11	Poor	184 + 08	12-ft Cut	0.080	13.0	3.9
US 123						
21	Poor	291 + 40	30-in. Fill	0.065	11.0	3.2
22	Excellent	269 + 55	10-ft Cut	0.016	13.0	3.5
23	Good	254 + 20	3-ft Cut	0.049	13.0	3.5
24	Good	207 + 95	4-ft Cut	0.047	14.0	4.0
25	Poor	154 + 65	18-ft Cut	0.113	13.0	3.7
26	Excellent	150 + 10	1-ft Cut	0.020	11.6	3.5
27	Poor	154 + 80	26-ft Cut	0.077	12.0	3.2
28	Poor	177 + 10	10-ft Fill	0.071	13.0	3.2
29	Poor	299 + 10	18-ft Cut	0.070	11.5	3.5
30	Excellent	126 + 30	5-in. Fill	0.014	12.9	3.5

None of the test methods and design procedures, however, gave a very accurate evaluation of the coarser grained, micaceous subgrades that were associated with sites of poor performance. As the grading of the soil becomes coarser, response to conventional testing is lower, yet the performance of the coarse grained subgrades was as bad or worse than all other subgrades investigated.

Comments by Yoder (3), Dehlen (4), and Ritter and Paquette (5) emphasize the lack of some truly rational method of flexible pavement design. It is thought that the poor soils of the Piedmont area are essentially elastic and that some new rational design procedure to limit flexure should be applied to these soils. It is felt that design methods that utilize ultimate shear strength will, when properly applied, prevent lateral displacement and permanent rutting of plastic subgrades, but this is not the answer to the problem of elastic deflection that fatigues and ultimately cracks the pavement structure. Thus, the search for new, easily performed testing procedures to measure soil elasticity was started.

#### IGNITION LOSS TEST

The crystalline and chemical structure of a number of common soil minerals including clays and micas have been determined (6). The formula for muscovite mica,  $KAl_2(AlSi_3O_{10})(OH)_2$ , is relatively fixed; for other micas, the formula is more variable permitting the substitution of several possible metallic ions. Muscovite decomposes between 600 C and 700 C into orthoclase, corundum, and  $H_2O$ . The loss in weight of  $H_2O$  from the decomposition of a local sample of muscovite averaged 4.5 percent in laboratory tests and agrees quite closely with the 4.52 percent computed from the atomic weights of the formula. Biotite averages 3.9 percent crystal water while Paragonite is 4.71 percent  $H_2O$ .

Mechanical and chemical weathering of micas result in alteration products that have much higher amounts of combined water. The degradation of mica is thought to proceed through a hydrous mica, in which some of the alkali ion has been replaced by water, to a moderately plastic clay mineral. There is no sharp line of demarcation between hydrous mica, illite, and the final clay product as mica disintegrates.

Thus, two forms of combined water may be present: (a) hydrate water ( $H_2O$ ) or water of crystallization found in the hydrous micas and lost at moderate temperatures (200 to 300 C), and (b) water of constitution or the hydroxyl ion part of the crystal lattice lost at higher temperatures (600 to 1,000 C). Pure hydrous micas often contain as much as 10 percent total combined water.

With the exception of mica and clay none of the common Piedmont soil minerals contain appreciable crystal water. Therefore, if the clay is first removed from the

sample, the water of crystallization present in the remaining portion of the sample will be an indication of the amount of mica responsible for the elastic behavior of the soil. The following test procedure was developed to measure the weight of crystal water in the +200 fraction as a percentage of the total sample.

The sample to be tested for mica is washed through a No. 200 sieve following wet mechanical analysis. The +200 portion of the sample is then oven-dried at 105 C to remove all free water and hygroscopic moisture. It is then heated in a small crucible to approximately 1,000 C until constant weight is reached. The crystal water (percent ignition loss) as a percentage of the total sample is computed from the weight loss and mechanical analysis as percent ignition loss = (wt loss) (100 - percent passing the No. 200 sieve) ÷ (wt of the +200 material tested).

The sand fraction samples of this investigation tested for ignition loss were usually about 20 gm in size and were usually heated about thirty minutes in a small electric furnace capable of continuous operation at 1,000 C.

### REPEATED-LOAD CONSOLIDATION TEST

Another new test procedure to measure the elasticity of the soil directly was also developed as part of this investigation. In this test, the consolidation apparatus (Soil-test Model C-280) was used to measure the confined modulus of elasticity of the subgrade soil. Although the consolidation apparatus is generally not used in the study of highway subgrades, it was felt that useful information that could be correlated with ignition loss and pavement deflection could be obtained. The basic principle of the test is not unlike that of the resiliometer developed by Hveem (>) to measure the resilience of soils.

In the repeated-load consolidation test, the vertical stress increment for the repeated loads was based on expected stress conditions in the subgrade about 15 in. below the surface of the pavement, assuming a 9,000 lb dual-wheel load and no bridging action on the part of the pavement. Stresses from which the modulus of elasticity was computed were an initial stress of 1.2 psi (due to overburden) and a final stress of 15.25 psi (due to overburden plus wheel load).

For each subgrade sample approximately 120 gm, representative of the material passing the No. 10 sieve, was pulverized and tested for elasticity in the following manner. A known weight of pulverized material passing the No. 10 sieve, sufficient to nearly fill the apparatus, was placed in the apparatus dry. A small 0.75 lb weight producing 1.2 psi on the sample is placed on the weight hanger and left there throughout the entire test. The dry sample is then compacted with 100 quick applications of three additional 8.8 lb weights raising the pressure on the sample to 43.4 psi with each application. The sample is then saturated with water added through the porous sandstone disk at the base of the sample while confined with the small weight only. Again 100 applications of 43.4 psi are applied to further compact the sample—allowing a few seconds for each application cycle. In a similar manner, applications of one 8.8 lb weight (15.25 psi) are repeated at a rate of 1/cpm until dial readings at the end of each 30-second period indicate that equilibrium has been reached. Both high and low dial readings are recorded to the nearest 0.0001 in. The deflection for the load increment attributed to the give in the apparatus is determined by calibration with a 1-in. steel blank. The final net rebound for the test-load increment is obtained by subtracting the apparatus deflection from the difference between the two observed dial readings recorded for the test on the soil sample.

The confined modulus of elasticity,  $E_s$ , is computed from the net rebound,  $\delta$ , and the height of the sample,  $h$ , for the subgrade sample using the following relation:

$$E_s = \frac{14.05 h}{\delta} \quad (1)$$

The modulus of elasticity, as measured by the repeated-load consolidation test previously described, was obtained while the sample was confined and lateral displacement restricted. A lower modulus of elasticity would have been obtained had a triaxial or unconfined compression test been used. However, it was felt that the results obtained

by the test method used were still abnormally low, particularly for those samples containing a high percentage of clay and silt because of the relatively long time period allowed for deformations to occur in the test. Therefore, a correction based on the percent retained on the No. 200 sieve was applied in the following manner to obtain the corrected modulus of elasticity for the samples in the saturated condition: Corrected  $E_S = \text{Measured } E_S / 1 - 0.01 (\text{percent passing the No. 200 sieve})$ .

#### MODULUS OF ELASTICITY VS IGNITION LOSS

For a number of samples, the corrected modulus of elasticity was compared with results of the ignition-loss test. It was found that an equivalent modulus of elasticity yielding good correlation could be computed from the ignition loss using the following inverse relationship: Equivalent  $E_S = (8,000/I_g)$  psi. The equivalent modulus of elasticity, computed from the ignition loss, was compared with the corrected modulus of elasticity of the same subgrade sample, and the comparison of these two values for 182 samples from the US 29 and 123 highway sections yielded a coefficient of correlation of 0.75.

The next step in the evaluation of the two new tests was to compare actual pavement deflections, as measured with the Benkelman beam, with deflections computed from the test results using deflection analysis.

#### DEFLECTION ANALYSIS

If the Boussinesq equation for vertical stress is multiplied by "dz" and divided by the modulus of elasticity,  $E_S$ , of the semi-infinite continuum and then integrated between the limits of "d" and infinity, the result will be the total elastic deflection,  $\delta$ , of a point at radial distance, r, and depth, d, from the point of application of load, P, indicated as follows:

$$\delta = \int_d^{\infty} \frac{3Pz^3 dz}{2\pi E_S (r^2 + z^2)^{5/2}} \quad (2)$$

The above equation was integrated and various values of P and r were introduced to accommodate the geometry and dimensions of the Benkelman beam and to approximate an area load produced by dual tires loaded to 9,000 lb. To predict the net deflection of the flexible pavement as measured by the Benkelman beam, the following equation is used:

$$\delta = \frac{4,300}{E_S} \left[ \frac{d^2}{6(5.4^2 + d^2)^{3/2}} + \frac{1}{3(5.4^2 + d^2)^{1/2}} + \frac{d^2}{6(8.7^2 + d^2)^{3/2}} + \frac{1}{3(8.7^2 + d^2)^{1/2}} - \frac{0.97d^2}{(108^2 + d^2)^{3/2}} - \frac{1.93}{(108^2 + d^2)^{1/2}} + \frac{0.63d^2}{(165^2 + d^2)^{3/2}} + \frac{1.27}{(165^2 + d^2)^{1/2}} \right] \quad (3)$$

If the eight terms of the deflection equation are multiplied by 4,300 and the product designated as the coefficient of deflection,  $C\delta$ , the equation may be written in the following simplified form:

$$\delta = \frac{C\delta}{E_S} \quad (4)$$

Figure 1 shows a plot of the coefficient of deflection for various depths below the pavement surface.

Thus, the expected deflection resulting from the deformation of any soil layer can easily be calculated if the depth range and modulus of elasticity of the soil layer is known. The expected deflection for the layer,  $\Delta \delta$ , is obtained by dividing the difference between the coefficient of deflection at the top and the bottom of the layer by the modulus of elasticity as follows:

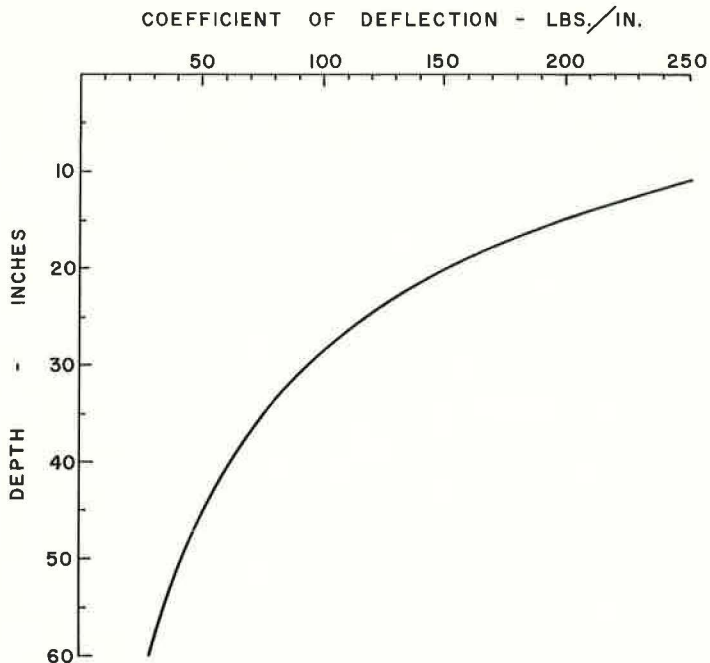


Figure 1. Coefficient of deflection plot for various depths below pavement surface.

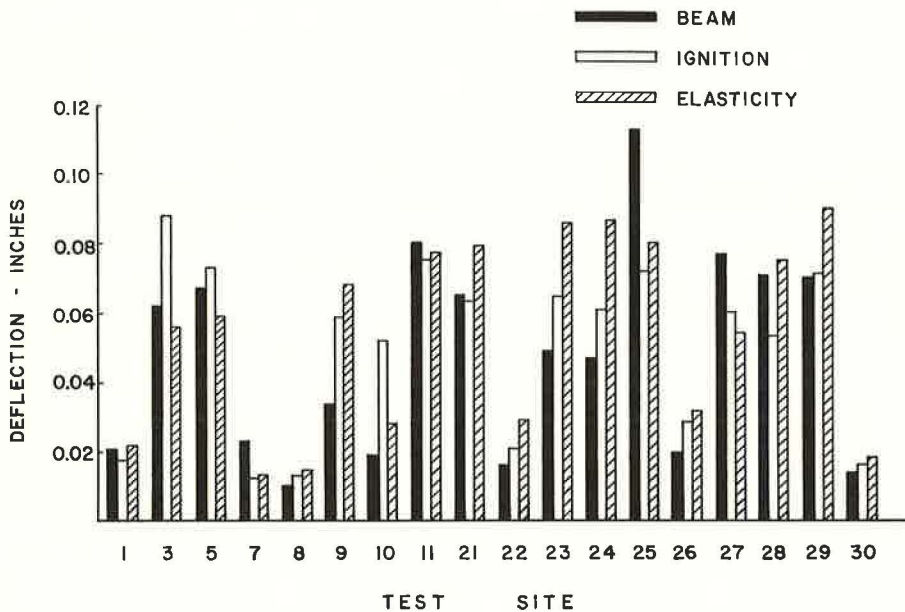


Figure 2. Observed pavement deflections.

$$\Delta \delta = \frac{\Delta C \delta}{E_s} \tag{5}$$

The total expected deflection will be the sum of the expected deflection for all subgrade layers to a depth of about 10 ft and the deformation of the pavement and base course.

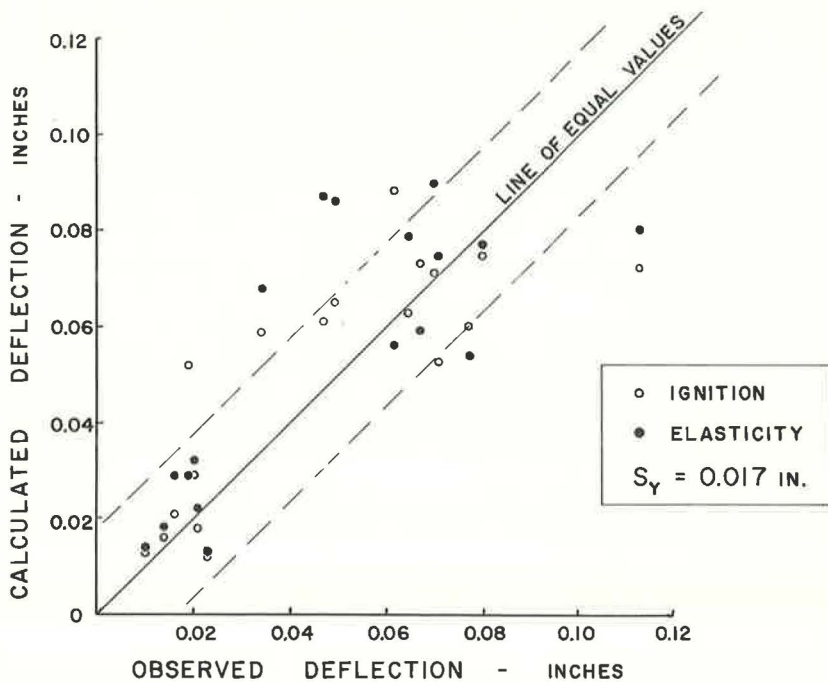


Figure 3. Observed and calculated pavement deflections.

A deformation of 0.004 in. was estimated for the pavement and base studied in this investigation.

The accuracy of the previously mentioned analysis depends on the truth in the assumption that flexible pavements and untreated bases have little or no beam strength and that the vertical stresses at various depths are as predicted by the Boussinesq equation. Also, it must be assumed that deflection is a result of elastic compression rather than lateral displacement of the soil layer. Experimental studies by Sowers and Vesic (8) confirm the first assumption. A study of these deflection curves indicates that vertical rather than lateral displacement predominates.

To check the validity of the theory, the deflection of each of the eighteen sites was computed using the deflection equation. The modulus of elasticity was determined by both ignition loss and elasticity, and the deflection from the deformation of each soil layer was computed in the manner explained previously. Calculated and observed deflections are graphically compared in Figures 2 and 3.

The coefficient of correlation between calculated and measured deflections was nearly the same for both methods of calculation. In the case of the ignition-loss method, the coefficient of correlation was 0.70. For the elasticity method, which is based on the repeated-load consolidation test, the coefficient of correlation was 0.68. There is, therefore, no strong evidence favoring one method of calculation over the other.

#### DESIGN PROCEDURE

The calculation of elastic deflection by the ignition-loss method and the modulus of elasticity method proved to be reasonably accurate in this study; either test procedure in combination with conventional classification procedures is considered suitable for evaluating the performance of upper South Carolina micaceous subgrade soils.

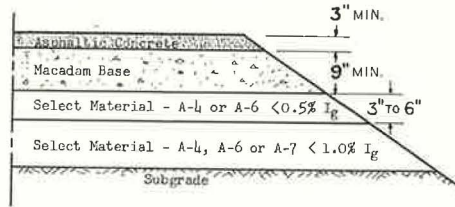
The typical cross-section shown in Figure 4 is designed to limit the total deflection as measured with the Benkelman beam to about 0.030 in. This is accomplished by providing compacted select-material subbases of sufficient rigidity and thickness so

that the total deformation of these materials will be no more than 0.030 in. less the expected deflection of the subgrade on which the subbase is placed. The design curve, subgrade ignition loss vs combined thickness of pavement, base, and subbase (Fig. 5), was developed to satisfy the previously mentioned criteria and can be used to determine the thickness of select material required for the given Piedmont subgrade.

If a given subgrade cannot meet the minimum requirements of the top layer of select material shown in Figure 4, three to 6 in. of A-4, A-6, or some granular select material having an ignition loss of less than 0.5 percent shall be provided as a subbase to support the macadam base. If the required total combined thickness exceeds 18 in., a lower layer of select material must be provided having an AASHO classification of A-7 or better and less than 1.0 percent ignition loss. A-5 soils of the Piedmont area usually have high mica content and high ignition loss and, therefore, are not suitable as a subbase material.

Good subbase materials meeting the previously mentioned requirements are generally found in the top 3 ft of the normal Piedmont soil profile. If this material is stockpiled from the cuts within the right-of-way, enough subbase material should be available to meet the needs of the construction.

It is obvious that failure of flexible pavements is primarily related to the thickness of the asphaltic concrete surface and the sharpness of bending rather than the magnitude of total vertical deflection. Therefore, the permissible deflection from which the design curve was calculated was increased slightly as the required depth of combined thickness increased. It was assumed that for a 3.5-in. asphaltic concrete pavement and a combined thickness of 15 in., the pavement performance would be satisfactory if 0.025-in.



FLEXIBLE PAVEMENT DESIGN CHART FOR MICACEOUS SUBGRADES

Figure 4. Typical pavement and subbase cross-section.

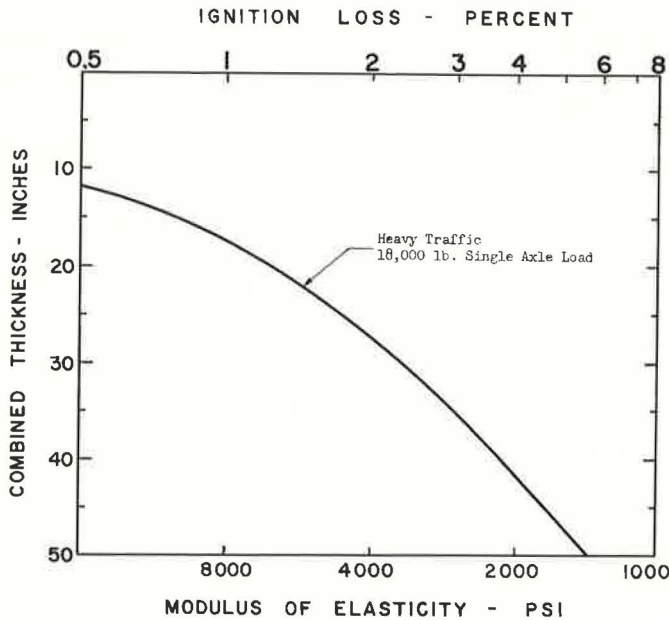


Figure 5. Subgrade ignition loss vs combined thickness of pavement, base, and subbase.

deflection is not exceeded. A 0.035-in. deflection would not be excessive if the combined thickness of the roadway structure is greater than 36 in.

Even though there is good correlation between the results of the ignition-loss test and pavement performance, it should be pointed out that the study of the US 29 and 123 sections was limited to a few square miles of area located about five miles west of Spartanburg and five miles west of Clemson, South Carolina. Further study of micaceous soils from other areas of the piedmont will be necessary to establish the relationship between elasticity and ignition loss for a greater variety of soils.

#### CONCLUSIONS

1. Benkelman beam deflection measurements were correlated with pavement performance of the test sites studied.
2. The ignition-loss test is simple and economical. The results of this test were an indication of the mica content and elasticity of the subgrades tested.
3. Direct measurement of elasticity by means of the repeated-load consolidation test is also relatively easy to perform; test results were inversely related to the ignition loss of the subgrade soils tested.
4. The calculation of elastic deflection of a flexible pavement system by the ignition loss (or repeated-load test) method was reasonably accurate in this study and is the basis for a recommended design procedure.

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