CONCLUSIONS

The conclusions drawn from this research are as follows:

1. All design thickness, from 4.5 to 6 in (11.4-15.2 cm), provided quite adequate service for a low-volume secondary roadway with minimal maintenance for 30 years.

2. The 4.5-in (11.4-cm) thick pavement has resulted in a slight reduction of aggregate use when compared with requirements for an unpaved gravel roadway, but the 6-in (15.2-cm) pavement results in a substantial increase in aggregate use.

3. The 4.5-in thick mesh-reinforced section has provided the best overall performance.

4. Slab lengths of the nonreinforced sections without contraction joints vary directly with thickness and are all just less than the current design length of 20 ft (6.1 m).

5. The amount of longitudinal cracking varies inversely with the thickness of pavement.

6. The cost of maintaining a paved road is less than that for a gravel roadway. If only basic maintenance is provided, the cost for a paved road may be less than half that for a gravel-surfaced roadway.

ACKNOWLEDGMENT

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Pavement Evaluation and Upgrading of Low-Cost Roads

JACOB GREENSTEIN

Design, construction, and performance experience on nonsurfaced gravel low-cost roads show that, even when proper design and construction procedures are followed, unexpected early failure often occurs. These modes of failure generally occur after the wet season and are normally attributed to the development of unexpected, very high moisture content in the subgrade. This moisture buildup in localized subgrade areas cannot be predicted and is frequently attributed to changes in topography, drainage, and environmental factors that take place during and after construction. This paper presents a new methodology to minimize unexpected pavement failure and to upgrade the underdesigned road sections. The methodology is based on the subgrade strain criterion. The strain characteristics developed on top of the subgrade are calculated for a given dynamic loading and therefore permit the determination of the location of these high-moisture and low-strength sections of the pavement. The proposed methodology also permits the calculation of the amount of roadway reinforcement required to achieve uniform roadway strain characteristics. The uniform strain should obviously result in a uniform life expectancy of the roadway. The automated instrumentation for making this strain inventory is now being used in Thailand, several countries in Latin America, and elsewhere. It has proven to be simple, economical, and practical.

Many unsurfaced roads constructed in accordance with the best design technology have failed well before their predicted life expectancy. Such an unexpected failure is shown in Figure 1. Normally, these failures are attributed to the development of very high moisture content in the subgrade, moisture substantially greater than the in situ moisture content found in the natural subgrade prior to construction. This moisture buildup in localized subgrade areas cannot be predicted; it is frequently attributed to changes in topography, drainage, and environmental factors that develop during and after construction. It is rarely assumed that all of the natural subgrade would become saturated because this would lead to conservative, low-strength values for the natural subgrade that would generate thick and costly pavement.

In conventional design methods for unsurfaced roads, the stability of the pavement is ensured through material quality requirements for the subbase or the base materials. By contrast, only the subgrade strain criterion is represented qualitatively in the determination of pavement thickness. In other words, under given traffic and environmental loadings, the subgrade strain developed at the top of the subgrade must be identical along the road. The magnitude of the subgrade strain is affected strongly by environmental factors, which correspond to the conclusions of numerous field studies: The principal failure cause, particularly in areas of high rainfall, is due to underground springs that pump moisture from a high-water table, which causes saturation of the subgrade due to flooding of side ditches during the rainy season.

The equipment and methodology have now become available for determining the strain characteris-
where \( E \) denotes the modulus of elasticity.

**DESIGN CONSIDERATIONS**

The basic objective of the design procedure is to provide a structure made of local materials that will be suitable to carry the anticipated traffic loadings under a given environmental condition. To achieve this goal, the following five phases must be completed.

1. Design criteria—limited strain, limited stress, and smooth ride;
2. Data collection—traffic and environmental loadings and subgrade and material properties;
3. Calculation and evaluation of the pavement system—subgrade and material properties, thickness, and stresses and strains;
4. Design conclusion—structural section, lifetime, and material specifications; and
5. Early performance study—immediately after the construction stage (generally 0.5-2 years after completion of construction).

Most design methods of low-volume roads (1-6) include only the first four phases; that is, the structural section and lifetime performance curves are analyzed before the construction stage. Thus, unexpected environmental conditions and material performance after the completion of the project may cause early and unexpected failure of the pavement (see Figure 1).

**DESIGN CRITERIA**

Use of Subgrade Strain Criteria in Conjunction with California Bearing Ratio Methodology

Most design methods are variations of the California bearing ratio (CBR) method. The basic concept is an empirical application of the Boussinesq theory of homogeneous media to prevent shear failure in the subgrade (7-10). CBR methodology is widely used, and its values are simple to measure in the field or laboratory. Subgrade strain is difficult to measure.

Since subgrade strain is used in this paper as the principal criterion of design, the ratio between CBR and vertical strain must be shown to be constant for any material (e.g., for a given modulus \( E \)). The fundamental CBR formula stipulates that the ratio between the allowed maximum shear stress \( (\tau_{\text{max}}) \) in the subgrade and its CBR should be constant:

\[
\frac{\tau_{\text{max}}}{\text{CBR}} = D
\]

(1)

Results of studies that use this methodology indicate that the relation between flexible pavement thickness (\( H \)) to wheel load (\( P \)) and the contact pressure (\( p \)) is as indicated in Equation 2 (10).

\[
H = \left[ P \left( 0.75 / \pi D \right) (1 / \text{CBR}) - (1 / \text{pm}) \right]^{0.5}
\]

(3)

The factor 0.75/\( \pi D \) is determined empirically as 1/8.1 for most materials that have a CBR less than 12. Therefore, the constant \( D \) in Equation 1 is equal to 1.93.

In an elastic linear material that has Poisson’s ratio of 0.5 and for a state of axi-symmetric loading there exists a simple relation between the vertical strain \( (\epsilon_z) \) and the maximum shear stress \( (\tau_{\text{max}}) \) (10):

\[
\frac{\tau_{\text{max}}}{\epsilon_z} = E / 2
\]

(3)

where \( E \) denotes the modulus of elasticity.

Substitution of Equation 3 into Equation 1 gives:

\[
\frac{\text{CBR}}{E} = \frac{1}{2D}
\]

Equation 4 indicates that for a given media (e.g., \( E = \text{constant} \)) the ratio between the CBR and the vertical strain is constant.

Subgrade Strain Criterion

Most design methods that incorporate the results from the American Association of State Highway Officials (AASHO) road test of 1962 (11) use rutting as a failure criterion of flexible pavement. Rutting is considered to occur only on the subgrade and is controlled by limiting the value of the vertical compressive strain of the top subgrade. This implies that the pavement layers above the subgrade will be structurally adequate so that only negligible plastic deformation will occur with each layer. The Shell method (12) is associated with ultimate rut depths on the order of 0.75 in (19 mm).

The subgrade strain criterion was verified by the Waterway Experiment Station (WES) after analysis of pavement performance of test sections (13,14) and the range of the WES criterion is given in Figure 2, which presents a comparison of other strain criteria (12,15–18). Figure 2 shows that the Shell strain criterion (12) is in the range of the WES criterion. On the other hand, the criterion developed by Brabston and others (19) shows lower strain values for the same number of repetitions than recommended by the WES. The differences in the recommended criteria are mainly due to the following reasons.

1. The dynamic modulus of the subgrade and the granular materials is determined by different approaches, as shown in Figure 3, which presents the relation between the dynamic modulus and the subgrade CBR according to WES (19), Shell (20), and Wiseman and others (21). For example, a subgrade CBR of 5 percent, the dynamic modulus \( (10^{5} \text{ lb/in}^2) \) is 17.0 (19), 7.8 (20), and 7.0-10.0 (21).

2. Local environmental conditions affect the design parameters differently.

**METHODOLOGY OF PAVEMENT EVALUATION**

To a certain extent, the structural evaluation of a pavement system is an inverted design process. If the pavement cross section and properties of the paving materials and subgrade soil are known, it is possible to compute pavement responses (stresses, strains, and deflections) under a given load at any point within the structure. In the evaluation process, the response of the pavement is observed and material properties are back-calculated. Of the different responses of the pavement to load, the only practical measurements are deflections. For structural evaluation, the real pavement-subgrade system is replaced by a mathematical model, and the measured surface deflections are used as input to back-calculate the model’s parameters. Two mathematical models are used in order to evaluate the elastic modulus of the subgrade and the pavement.

**First Mathematical Model**

The first mathematical model is a thin plate (to represent the pavement) supported on an elastic foundation (the subgrade). By using this theory (21,22), the pavement and its supports are expressed in terms of the following basic pavement parameters.

**Flexibility**

The pavement flexibility (\( F \)) is the center deflec-
Figure 2. Comparison of subgrade strain criteria \( (E_s) \).

Figure 3. Relationships between dynamic modulus and CBR.

Function per unit force and is reported in micrometers per kiloNewton (\( \mu m/kN \)).

\[
F = \frac{\Delta_0}{P} \tag{5}
\]

where \( \Delta_0 \) is the center deflection (\( \mu m \)) and \( P \) is the total load (kN), which equals peak to peak force when a dynamic device is used.

Characteristic Length

This characteristic length \( (l) \) presents the shape of the deflection basin and is determined according to Figure 4, which shows the relation between \( l \), the offset distance \( r \), and the deflection ratio \( (\Delta_0/\Delta_0) \).

Subgrade Modulus

The subgrade modulus \( (E_s) \) can be computed by using the measured pavement flexibility \( (F) \) and the characteristic length \( (l) \) by the following equation (in megaNewtons per square meter):

\[
E_s = 10^6/4.14 PR(S/S_0) \tag{6}
\]

where

\[
F = \text{measured flexibility (} \mu m/kN \text{)},
\]

\[
l = \text{characteristic length (mm)},
\]

\[
(S/S_0) = \text{ratio of the measured pavement stiffness (S) to the theoretical point load stiffness (S_0), which is a function of } l \text{ and geometry of load application, stiffness being defined as the required load to cause unit deflection.}
\]

For the 18-in (46-cm) diameter plate used in this study, \( S/S_0 \) is determined by the following equation (22):

\[
S/S_0 = 68 (0.18 + 5)^{2.06}/k^2 \tag{7}
\]

Substitution of Equation 7 and the results of Figure 4 for \( r = 50 \) cm in Equation 6 results in the relation among \( E_s/P, F, \) and \( \Delta_0/\Delta_0 \) (p denotes the contact pressure). This relation is presented in Figure 5a. According to this model, the relation between the characteristic length per pavement thickness \( (l/H) \) and \( Bo/Es \) is determined by the following equation (21,22)

\[
S/H = [E_p 1.5/E_s 12 (1 - \mu^2)]^{1/3} \tag{8}
\]

Equation 8 and Figure 4 are applied in this study to determine the relation among the deflection ratio \( \Delta_0/\Delta_0 \), \( E_p/E_s \), and \( H/a \). This relation is presented in Figure 6a. \( E_p \) and \( E_s \) denote the elastic modulus of the pavement and subgrade, respectively. \( H \) is the pavement thickness, and \( a \) is the radius of the circular area.

Second Mathematical Model

The second mathematical model used in this study is based on a two-layer linear elastic model by Burmister (23). This model is used to determine the vertical strain \( (e_v) \) on top of the subgrade. The results are presented in Figure 6b in the form of a nondimensional relation between the subgrade strain factor \( (e_v E_v/p) \), the modulus ratio \( (E_p/E_s) \), and \( H/a \). Thus, by combining Figures 5a and 6b, it is possible to determine the relation among \( \Delta_0/\Delta_0 \), \( H/a \), and \( e_v E_v/p \). In a similar way, the combining of Figures 5a and b enables the determination of the relation among the deflection ratio \( \Delta_0/\Delta_0 \), the flexibility \( (F) \), and the vertical strain on top of the subgrade \( (e_v) \).

The following example is presented to demonstrate how to determine the subgrade strain by means of Figures 5 and 6:
F = 12 \, \mu m/kN and \Delta s_f/\Delta_0 = 0.41 were determined under an 18-in (46-cm) plate loaded with a dynamic contact pressure of p = 67.77 kN/m². The total thickness of the pavement is 32 cm. By using Figure 6 (see example 1) for \Delta s_f/\Delta_0 = 0.41 and H = 32, the following two nondimensional parameters can be determined: \( E_p/E_o = 2.15 \) and \( \epsilon_z E_p/p = 0.29 \). The second step is to determine the subgrade modulus and the vertical strain on the top of the subgrade by means of Figure 5. In example 1, the result is

\[ E_p/p = 1.07 \times 10^3 \quad (\text{or} \quad E_p = 1.07 \times 10^3 \times 67.7 = 72.5 \times 10^3 \, \text{kN/m}^2) \quad \text{and} \quad \epsilon_z = 2.7 \times 10^{-4}. \]

Figure 4. Deflection bowl, Hogg model—18-in diameter plate.

Figure 5. \( \epsilon_z \) versus \( E_p/p, F, \Delta s_f/\Delta_0 \).

Note that the principles of the methodology presented in this section might be applied to different nondestructive equipment, such as the road rater, Benkelman beam, Dynaflect, and falling weight deflectometer. Nevertheless, in this paper, only the application of the road rater is presented (see Figure 7).

**EVALUATION AND UPGRADE OF UNSURFACED ROADS**

The design lifetime of unsurfaced pavements in developing countries such as Ecuador, Bolivia, and Thailand varies between 5 and 8 years. The lower limit is used for laterite roads and the upper limit for granular-based roads. In order to eliminate unexpected early failure performance, analysis is conducted in these countries during the early service period to locate the underdesigned road sections.

**Performance Analysis and Upgrading of Laterite Roads**

In this case, 18 cm of laterite were designated (2) to carry 25 000 standard axles/lane over a silty subgrade that has a design CBR of 6.5 percent in Thailand. About 8 months after the completion of the project and after the major rainy season, a dynamic deflection, which used a pavement profiler with 46-cm diameter plate, was conducted along this unsurfaced road. The following representative deflection parameters were determined for two sections of the road:

1. Section A (about 75 percent of the length of the road)—Representative value of flexibility \( F \) = 13.2 \, \mu m/kN; deflection ratio \( \Delta s_f/\Delta_0 = 0.185 \); and pavement thickness \( H = 18 \) cm.

2. Section B (about 25 percent of the length of the road)—Representative value of flexibility \( F \) = 9.2 \, \mu m/kN; deflection ratio \( \Delta s_f/\Delta_0 = 0.225 \); and pavement thickness \( H = 20 \) cm.
Figure 6. \(c_j E_p/p\) versus \(E_d/E_s\), \(\Delta_{50}/\Delta_0\).

1. Use Figure 5 to determine the subgrade strain factor \((\varepsilon_2 E_d/p)\). When \(E_p/p = 0.81\) and \(\varepsilon_2\) is reduced from \(5.9 \times 10^{-3}\) to \(3.8 \times 10^{-3}\), the subgrade strain factor varies from 0.48 to 0.31.

2. Use Figure 6 to determine the required pavement thickness \((H)\). When \(E_d/E_s = 2.75\), and the strain factor varies from 0.48 to 0.31, \(H\) varies from 18 to 29 cm. Therefore, the additional thickness is 29 cm - 18 cm = 11 cm.

Performance Analysis and Upgrading of Granular Base Roads

A 32-cm section of granular base that has CBR > 80 percent was designated to carry 100 000 standard axles/lane over silty clay subgrade (3). During the first rainy season, a short section (about 500 m in length) started to fail; consequently, a dynamic deflection survey was conducted on the failed and unfailed sections of the road. The following representative deflection parameters were described:

1. Failed section (about 5 percent of the road)—Flexibility \((F) = 12.0 \ \mu m/kN\); deflection ratio \((\Delta_{50}/\Delta_0) = 0.41\); and pavement thickness \((H) = 32\) cm. From Figures 5 and 6, \(\varepsilon_2 = 2.7 \times 10^{-3}\), \(E_d/p = 1.07 \times 10^3\), and \(E_p/E_s = 2.15\).
Figure 7. Pavement evaluation of unsurfaced roads.

![Image of a table]

**Table 1. Analysis of deflection survey of laterite road.**

<table>
<thead>
<tr>
<th>Road Section</th>
<th>After Figure 6</th>
<th>After Figure 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>ε₁E₁/p = 0.5</td>
<td>ε₁ = 0.000 38</td>
</tr>
<tr>
<td></td>
<td>E₂/E₁ = 2.70</td>
<td>E₂ = 89 500 kN/m²</td>
</tr>
<tr>
<td></td>
<td>ε₂ = 0.000 59</td>
<td>E₂/p = 810</td>
</tr>
<tr>
<td></td>
<td>E₁ = 54 900 kN/m²</td>
<td></td>
</tr>
</tbody>
</table>

Notes: E₁ = vertical strain, E₂ = elastic modulus of the subgrade, p = contact pressure, and Eₚ = elastic modulus of the pavement.

2. Unfailed section—Flexibility (F) = 8.5 μm/kN; deflection ratio (d₃g/d₄g) = 0.52; and pavement thickness (H) = 32 cm. From Figures 5 and 6, εₑ = 1.7 x 10⁻², Eₚ/Eₛ = 4.9.

In order to upgrade the failed section, additional pavement thickness is required to reduce the subgrade strain from 2.7 x 10⁻² to 1.7 x 10⁻². This can be achieved in the following way:

1. Use Figure 5 to determine the subgrade strain factor εₑEₑ/p. When Eₑ/p = 1.07 x 10⁻² and εₑ is reduced from 2.7 x 10⁻² to 1.7 x 10⁻², εₑEₑ/p varies from 0.29 to 0.20.

2. Use Figure 6 to determine the required pavement thickness (H). When Eₚ/Eₛ = 2.15 and the subgrade strain factor varies from 0.29 to 0.20, H varies from 32 to 46 cm. Therefore, the additional required thickness is 14 cm of granular-base course.

**CONCLUSIONS**

Early performance study of low-cost roads has proved to be an economical and practical means to avoid unexpected early failure. A pavement evaluation process is used to determine the response of the pavement and subgrade system to dynamic loading.

The two-layered pavement-subgrade system is replaced by a simple mathematical model. The deflection basin is determined by nondestructive means and the results are used as input to back-calculate the model's pavement.

The equipment and methodology have now become available for determining the strain characteristics. This methodology permits a precise determination of where the high-moisture, high-deflection, or high-strain areas are located.

The subgrade strain criterion is used to determine the minimum overlay thickness required to achieve uniform roadway strain characteristics. This technique is being applied in Thailand, Nepal, and Latin America to laterite and gravel roads that have a pavement design CBR of 30 to 80 percent.

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Pavement Design for a 3.5-Million-Pound Vehicle

WILBUR CHARLES GREER, JR.

The investigative and analysis procedures used to evaluate roadways for the movement of two nuclear reactor vessels from Knoxville, Tennessee, to the Tennessee Valley Authority's (TVA) Phipps Bend power generation plant are presented. The route included 74 km (46 miles) of state, county, and private roadways, both paved and unpaved. The total weight of each reactor pressure vessel and its transport trailers was approximately 13 345 kN (3 million lb). The five prime movers brought the total weight for each move to approximately 15 569 kN (3.5 million lb). The transport trailer was supported on 24 axle lines with 16 wheels/axle line. The methods of compiling existing construction data, field testing, and analysis procedures are discussed. The application of proofrolling to verify the results of the analyses is also presented. Deflection and density testing, both before and after the moves, were performed by the Tennessee Department of Transportation. An analysis of these data indicates no serious effects due to the two moves that occurred in 1980 and 1981. A comparison between the results of the U.S. Army Corps of Engineers procedures originally used to evaluate pavement thickness requirements and the results of layered elastic computer program analyses after the second move is presented. The results of the two methods compare favorably.

The construction of large industrial manufacturing plants and power-generation facilities may require that very heavy mechanical items be brought to the site in one piece. Items such as generators and nuclear reactor pressure vessels may weigh a few thousand kilonewtons (several hundred thousand pounds) to in excess of 9000 kN (2 million lbs). Most projects may have only one or two such pieces of equipment to move. Where possible, these items can be brought to the site by water or rail transportation; however, the equipment has to be transported across paved roads to many sites. The vehicles used to move the equipment often have many axles and numerous tires per axle in order to spread the load and reduce individual tire loads to acceptable levels. The high axle loads, numerous tires, and very low number of applications complicate the analysis of the pavement structures.

The Tennessee Valley Authority's (TVA) Phipps Bend nuclear power generation plant is located in northeast Tennessee. The plant, currently under construction, will have two nuclear units when completed. The steel reactor pressure vessel (RPV) for each unit was fabricated in Memphis nearly 692 km (430 miles) away. Each RPV weighs 9190 kN (1033 tons) and thus just cannot be loaded on a normal truck and hauled to the plant site. Plans for the transport of the RPVs to the plant site called for each RPV to be barged to Knoxville and then hauled over 74 km (46 miles) of state, county, and private roadways to the plant site. These are believed to be some of the heaviest, if not the heaviest, loads ever moved over U.S. roads.

The VBL Corporation, the transport contractor, authorized an investigation and evaluation of the 74 km (46 miles) of roadways to be traversed relative to each pavement's ability to withstand the loads imposed through the transport trailers. The objectives of the investigation were to investigate and develop recommendations with regard to pavements and to the geotechnical aspects of the travel route, of planned detours, and of planned road widenings. The basic criteria for a successful move were defined as the following:

1. Prevent, as economically as possible, the transport trailers from becoming stuck; stoppage could result in enormous delay costs; and
2. Minimize visible and measurable damage to the roadways that could result in significant damage charges against the contractor.