The deteriorating condition of paved-road networks, the need to upgrade gravel roads to bituminous standards, and the limited resources available have led highway administrators and managers in developing countries to face a stark dilemma. Should they continue to adopt restrictive imported specifications that result in higher construction and rehabilitation costs or accept a more widespread use of lower-cost local materials and risk the possibility of increased future maintenance costs?

The observed behavior and performance of pavements with a base built with as-dug laterites indicate that this material can be used under a wide range of environmental and traffic conditions to build and rehabilitate roads. A minimum total road transportation cost resulting from this practice would make it easier for developing countries to establish an efficient and integrated transport system. Laterite bases can perform as well as crushed stone or stabilized laterites, and marginally better in some cases, under a wide range of circumstances. Their cost is about 20 to 30 percent of these more expensive materials, and their use will not incur extra road maintenance or vehicle operating costs.

Laterites are soils with a vesicular structure. Their colors range from yellow to red, sometimes with dark shades, frequently resembling a slag. Lateritic soils clay fraction (i.e., fraction passing 2 micra) shows a molecular silica/sesquioxide ratio (or SiO₂/R₂O₃) of less than 2 and low expansibility. Laterites occur in tropical and subtropical areas of Australia, South and Central America, Africa, and Asia and have long been used in the construction of roads, airports, and buildings.

Laterites are tropical soils that have been produced by advanced weathering accompanied by a relative enrichment in iron and aluminum sesquioxides (Fe₂O₃ and Al₂O₃, respectively) because of the decomposition of primary minerals and the removal of bases and silica, as described by Netterberg (1). Autret (2) has made a distinction between (a) lateritic fine soils or laterites; (b) lateritic gravels or granular laterites, used in road technology for base and subbase construction of paved roads, surfacing of unpaved roads, and in some cases as aggregate for surface treatments or asphalt concrete surfacings; and (c) cap rock (cuirasse lateritique), which are indurated concretions resembling slags, broken down by bulldozer blade or ripper. The laterites considered in this paper approach the second classification (b) above and consist mainly of gravel-sized concretions.

In general, laterites are among the more difficult materials to locate, and local prospecting may be required in many cases for precise location. However, the use of aerial photography in the search for laterites has been well documented (3,4). In the field, useful vegetation indicators can be developed on a regional basis, because laterites tend to be infertile. Charman (5) provides a good summary of guidelines for finding suitable sources of lateritic concretionary materials.

**NATURAL VERSUS PROCESSED MATERIALS**

The use of untreated local materials, such as laterites, has a considerable potential for cost savings in pavement construction and rehabilitation. However, when these materials do not satisfy the normal requirements for untreated road pavement materials, the alternative of improving locally occurring materials with stabilizing agents such as cement, lime, and bitumen has frequently been adopted (6–9). One other alternative is to improve the inadequate material through mechanical stabilization with sand or crushed rock. Two successful examples of the latter are the Tahoua-Arlit road in Niger, where some sections had the base course built with a 50–50
percent mixture of substandard laterite and crushed limestone \cite{10,11}, and the 40-km section of the Ouagadougou-Koudougou road in Burkina Faso \cite{12}.

Notwithstanding well-documented successes of the alternatives mentioned above, the additional cost (including relatively high proportions of foreign exchange) of providing the stabilizing agent, processing mixtures, or crushing and hauling rock has to be met. Moreover, despite the extra costs incurred to stabilize the otherwise substandard material, in many field examples the performance of the resulting pavement has not been demonstrably better than pavements built with the as-dug materials. The following are a few examples.

1. The Belem-Brasilia road in Brazil was paved from 1974 to 1976. The laterite-cement pavement showed premature failures, whereas as-dug laterite bases performed well. Although the reason for this unexpected behavior is not clear, several potential problems related to cement or lime stabilization of laterites have been cited by Netterberg \cite{13}: alkali-silica, carbonate, and alumina reactions; organic matter inhibition of stabilization; and unusual reactions with cement and lime (probably because of extremely high or low pH values). Carbonation has been shown to inhibit the formation of cementitious products in soil-lime and soil-cement reactions, thus adversely affecting pavement performance \cite{14}. Methods of predicting some of these problems have been described \cite{14,15}.

2. The South Bank Road in The Gambia was constructed from 1964 to 1968. The Brikama–Ba-Kalagi section was constructed with as-dug laterite. The Serekunda-Brikama and Kalagi-Soma sections were constructed with cement-stabilized laterite. In 1981, 13 years after construction, a condition survey showed about the same level of distress in both types of pavement. Twenty years after construction, roughness measurements, in terms of the International Roughness Index \cite{16}, showed an average 4.6 m/km for the as-dug laterite pavement and 4.2 m/km for the cement-stabilized pavement \cite{17}. At these levels of IRI, the minor difference is not significant in terms of vehicle operating costs, thus indicating similar performance of the two base types.

3. A 12-km section of the Mataara-Gatura road in Kenya was built in 1973 and 1974, 6 km with a crushed-stone base and the remaining 6 km with an as-dug laterite base \cite{18}. The main characteristics of the laterite were as follows. The mean laboratory-soaked California Bearing Ratio (CBR) at 95 percent of maximum dry density (MDD), British Heavy Compaction Test, was 54 (21 to 96 range); the mean plasticity index (PI) was 18 (7 to 21 range); the mean percentage passing a 0.063-mm sieve was 28 (15 to 37 range); and the mean in situ CBR at the surface of the base beneath bituminous surfacing was 89 (47 to 209 range). The local climate is characterized by a Torthwaite’s moisture index \cite{19} of 19, which defines a perhumid climate. Field investigations carried out in 1982 and 1985 indicated that the laterite performed marginally better than the crushed-stone base \cite{20}.

4. Many road sections in Australia built with as-dug laterites have been performing well in comparison with sections with crushed rock or cement-stabilized materials. The performance of these road sections is evident in many areas where laterites are available, such as the Eyre Peninsular region of South Australia.

5. The 1,440-km-long Cuiaiba–Porto Velho road in northwest Brazil was paved in 1983 and 1984. This road has a section of about 130 km (Rio Marco Rondon–Igarape Grande) where as-dug laterite was used as base course; the remainder was built with crushed-stone base \cite{21}. A survey carried out in May 1990 indicated that the laterite section was in fair condition, whereas several other sections were in poor condition (e.g., Caceres–Corrego Dourado, about 360 km). None of these sections has been resurfaced since construction.

6. A 1-km road section (Luwawa Turnoff to Champhoya Trial Section) was built in 1984 and 1985 as part of a 51-km road contract on Route M12 in the Viphya highlands of the Northern Province of Malawi \cite{22}. The base course of the trial section was built with as-dug nodular laterite showing a mean PI of 17 and a 4-day soaked CBR of 31 (mean in-situ CBR 5 months after surface dressing was 80). The local climate is characterized by a Torthwaite’s moisture index \cite{19} of 82, that is, a humid climate. As observed by Grace \cite{22}, the trial length of pavement is performing as effectively as the adjacent lengths, which have more expensive crushed-stone bases (about four times the cost of the laterite), and Benkelman beam deflections measured in 1986 were marginally smaller on the laterite-based section. Laboratory studies of the base laterite, carried out by Toll \cite{23}, indicated that if the laterite was compacted satisfactorily (i.e., not less than 95 percent of MDD British heavy compaction), it would retain adequate strength, even when the moisture content was increased to the point of saturation.

In addition to these examples, many more roads built with as-dug laterite have performed well. Souza et al. \cite{21} give a list of road pavements built in Brazil with as-dug laterite bases that have shown good performance. These roads carry a wide range of traffic volumes, up to a maximum of about 2,400 veh/day/lane, with a 30- to 40-percent proportion of heavy commercial vehicles. The roads also represent a wide range of environmental conditions, varying from the semiarid Brazilian northeast to the Brazilian central plateau to the rain forest areas of the Amazonic region.

Although the use of as-dug laterites for pavement base course is most frequently recommended for the lower end of low-volume road, the Brazilian examples indicate that these naturally occurring materials led to good performance of roads carrying relatively high traffic volumes.

**LONG-TERM VARIATION OF LATERITE PAVEMENT DEFLECTION**

As part of the Brazil-UNDP road costs study [a major road research project conducted from 1975 to 1982 \cite{24}], Benkelman beam and Dynaflect pavement deflections were measured periodically on 116 paved road sections. Ten laterite sections within a radius of about 50 km from Brasilia were measured with Benkelman beams every 2 to 3 months from 1976 to 1980 \cite{25,26}. A summary of the test section characteristics is given in Table 1. All sections have base and subbase courses built with as-dug laterite. The PI value varied from 0 (nonplastic) to 20, and 19 to 32 percent of the base course material passed the 0.075-mm sieve (No. 200), which can be considered typical of laterite gravels.
The time series deflection measurements indicated a decrease of deflection over time. Results of linear regression run on data from each section, given in Table 2, indicated that deflection decreased on eight of the sections, increased slightly on Section 002, and remained unchanged on Section 009. On the average, the rate of deflection decrease was 0.02 mm/year on sections 008 and 011, 0.03 mm/year on Sections 001, 003, 004 and 007, and 0.04 mm/year on sections 006 and 010. The different trend of sections 002 and 009 could not be explained: the data available indicate that age, laterite plasticity, and surface type and thickness are not significant factors. However, the decrease in deflection over time can be interpreted as a phenomenon of self-stabilization, as discussed in the next section.

The deflection histories of the eight sections that showed a decrease in deflection were all similar in trend. The history of Section 001 is shown in Figure 1. Deflections were measured at 40 points in each wheelpath of each section; therefore, each point plotted in the figure represents the mean of 80 deflection points. Local climate is characterized by a Thorthwaite’s moisture index (19) of about 60 (humid climate). The deflection measurements obtained in the rainy season (average rainfall is about 1500 mm/year, mostly occurring from November through February) are indicated by a plus sign in Figure 1. Although the deflection trend is significant over time, no significant influence of the rainy season on deflection was observed.

The conclusion is that a seasonal correction factor for deflections in regions similar to the study area (i.e., the Brazilian central plateau) is not needed. Information from other sources indicates that this conclusion is valid in other tropical and subtropical areas, where seasonal variability is lower than that observed for typical flexible pavements in higher latitudes.

A similar conclusion was obtained for the variation of Bellman beam deflections with pavement temperature. Plots of deflection versus temperature showed no significant trend, thus indicating that for relatively thin asphalt surfacings (up to 60-mm-thick layers), corrections for pavement temperature are not necessary. This conclusion is reflected in a model developed by Queiroz, Visser, and Moser (27), which only gives correction factors significantly different from one when the asphalt layer thickness is higher than about 60 mm.

### Table 1 Summary of Test Section Characteristics

<table>
<thead>
<tr>
<th>TEST SECTION</th>
<th>ROAD NUMBER</th>
<th>CONSTR. YEAR</th>
<th>AADT (vpd)</th>
<th>ESAL PER YEAR</th>
<th>SURFACE TYPE</th>
<th>FIRST CRACKING YEAR</th>
<th>BASE COURSE</th>
<th>CBR</th>
<th>PI</th>
<th>P75</th>
</tr>
</thead>
<tbody>
<tr>
<td>001</td>
<td>EPCT</td>
<td>1972</td>
<td>95</td>
<td>4,000</td>
<td>AC</td>
<td>1976</td>
<td>112</td>
<td>14</td>
<td>19</td>
<td></td>
</tr>
<tr>
<td>002</td>
<td>BR251</td>
<td>1970</td>
<td>280</td>
<td>25,000</td>
<td>DST</td>
<td>NC</td>
<td>77</td>
<td>13</td>
<td>32</td>
<td></td>
</tr>
<tr>
<td>003</td>
<td>BR020</td>
<td>1965</td>
<td>3,780</td>
<td>250,000</td>
<td>AC</td>
<td>BC</td>
<td>131</td>
<td>0</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>004</td>
<td>DF20</td>
<td>1976</td>
<td>110</td>
<td>8,000</td>
<td>DST</td>
<td>NC</td>
<td>134</td>
<td>12</td>
<td>24</td>
<td></td>
</tr>
<tr>
<td>005</td>
<td>BR040</td>
<td>1960</td>
<td>5,600</td>
<td>850,000</td>
<td>AC</td>
<td>BC</td>
<td>1977</td>
<td>102</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td>006</td>
<td>BR020</td>
<td>1966</td>
<td>1,150</td>
<td>75,000</td>
<td>DST</td>
<td>NC</td>
<td>76</td>
<td>18</td>
<td>26</td>
<td></td>
</tr>
<tr>
<td>007</td>
<td>BR060</td>
<td>1958</td>
<td>3,200</td>
<td>350,000</td>
<td>AC</td>
<td>BC</td>
<td>1976</td>
<td>83</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>008</td>
<td>DF08</td>
<td>1968</td>
<td>1,020</td>
<td>18,000</td>
<td>DST</td>
<td>NC</td>
<td>70</td>
<td>18</td>
<td>24</td>
<td></td>
</tr>
<tr>
<td>011</td>
<td>BR070</td>
<td>1971</td>
<td>1,090</td>
<td>140,000</td>
<td>DST</td>
<td>NC</td>
<td>71</td>
<td>18</td>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>

Notes: (1) AC: Asphalt concrete (2) DST: Double Bituminous Surface Treatment (3) Section 006 was overlaid in 1976 (4) Section 009 was overlaid in 1968 (5) ESAL: Equivalent single axle loads (6) NC: No cracking during observation period (7) BC: Cracking started before observation period (8) CBR: In situ California Bearing Ratio (9) PI: Plasticity Index (10) P75: Percent passing sieve no. 200 (11) AADT: Average annual daily traffic

### Table 2 Results of Linear Regression Run on Data from Each Section

<table>
<thead>
<tr>
<th>TEST SECTION</th>
<th>AVERAGE CHANGE (mm/year)</th>
<th>STANDARD ERROR (mm/year)</th>
<th>R SQUARED</th>
<th>NUMBER OF OBSERV.</th>
</tr>
</thead>
<tbody>
<tr>
<td>001</td>
<td>-0.034</td>
<td>0.005</td>
<td>0.62</td>
<td>26</td>
</tr>
<tr>
<td>002</td>
<td>0.016</td>
<td>0.008</td>
<td>0.13</td>
<td>29</td>
</tr>
<tr>
<td>003</td>
<td>-0.029</td>
<td>0.005</td>
<td>0.37</td>
<td>27</td>
</tr>
<tr>
<td>004</td>
<td>-0.026</td>
<td>0.005</td>
<td>0.30</td>
<td>27</td>
</tr>
<tr>
<td>005</td>
<td>-0.041</td>
<td>0.007</td>
<td>0.57</td>
<td>29</td>
</tr>
<tr>
<td>006</td>
<td>-0.026</td>
<td>0.007</td>
<td>0.34</td>
<td>27</td>
</tr>
<tr>
<td>007</td>
<td>-0.015</td>
<td>0.009</td>
<td>0.11</td>
<td>26</td>
</tr>
<tr>
<td>008</td>
<td>0.004</td>
<td>0.011</td>
<td>0.00</td>
<td>28</td>
</tr>
<tr>
<td>009</td>
<td>-0.040</td>
<td>0.009</td>
<td>0.45</td>
<td>27</td>
</tr>
<tr>
<td>011</td>
<td>-0.021</td>
<td>0.007</td>
<td>0.25</td>
<td>26</td>
</tr>
</tbody>
</table>

### Self-Stabilization of Laterite Pavements

Laterites, being a product of weathering, may actually be forming in the ground at the time of excavation. If this process continues in the road, it may give rise to the phenomenon of self-stabilization (28).
Self-stabilization (or self-hardening) can be defined as a natural improvement in the strength of a pavement layer, not caused by traffic compaction or the addition of stabilizing agents (29). Although hard evidence of self-stabilization is not well documented, a test for potentially self-stabilizing laterites (i.e., petrifaction degree) was developed by Nascimento in the 1960s (30,31).

The decrease of Benkelman beam deflections over a 5-year period on eight different laterite pavements in Brazil, as described in the previous section, provides field evidence of laterite self-stabilization. Gains in the strength of a laterite base 17 months after surfacing was 16 percent greater than the average of the in situ CBRs measured on the laterite base 17 months after surfacing (4). The average relative moisture content increased from 0.93, 6 months after surfacing, to 0.96, 18 months after surfacing. This increase in moisture content would normally lead to a decrease in strength. However, the average increase of in situ CBR during this period indicates that self-hardening of the laterite is taking place.

1. In Western Australia, six laterite trial sections built on the Great Northern Highway in 1982 showed increase in strength over a 2.5-year observation period, as measured by a Clegg impact testing device (ITD) (32). The ITD measures the deceleration of a 4.5-kg hammer dropped from a height of 460 mm. The deceleration is indicated in $g$s, and $10 g$ is one impact value (IV) (33). The IV has been correlated with in situ CBR (34), and the increase in IV of the laterite base with time would correspond with decrease in deflection, had deflection tests been carried out.

2. In Malawi, a 1-km road section (Luwawa Turnoff-Champhoyo Trial Section) built in 1984 and 1985 indicated that the average of the in situ CBRs measured on the laterite base 17 months after surfacing was 16 percent greater than the average of the CBRs measured at the same locations 5 months after surfacing (4). The average relative moisture content increased from 0.93, 6 months after surfacing, to 0.96, 18 months after surfacing. This increase in moisture content would normally lead to a decrease in strength. However, the average increase of in situ CBR during this period indicates that self-hardening of the laterite is taking place.

3. In The Gambia, Benkelman beam deflections were measured on the South Bank Road (Brikama-Ba-Kalagi section), where as-dug laterite was used as base course, in 1981 and 1987 (17). Mean deflections were 0.96 and 0.91 mm in 1981 and 1987, respectively, resulting in an average annual decrease in deflection of about 0.01 mm/year.

**ELASTIC BEHAVIOR OF LATERITE PAVEMENTS**

In the analysis of pavement structural response to applied loads, unbound materials normally show nonlinear elastic behavior (i.e., their elastic moduli depend on the induced stress or strain), as pointed out by Haas and Hudson (35) and Yoder and Witzczak (36). Medina and Motta (37) reported on several laboratory tests that also indicated nonlinear behavior of compacted tropical soils in Brazil.

To test in the field the nature of laterite pavement structural response to loads, an experiment was carried out on Sections 001, 004, and 006, which represent a wide range of traffic and age and two surface types (basic characteristics of these sections given in Table 1). Deflection measurements were taken on these sections with axle loads varying from about 3 to 12 tons (27).

Results of the deflection versus load analysis are presented in Table 3. The relationship obtained for section 006 is shown graphically in Figure 2. Deflections were measured at 10 points in each wheelpath of each section, for each axle load; therefore, each point plotted in Figure 2 represents the mean of 20 deflection points.

The results indicated a proportionality between deflection and axle loads, for a range of frequently occurring traffic loadings, thus indicating that the pavement systems (i.e., pavement layers plus subgrade soil) tested exhibit a linear elastic behavior. This behavior suggests the applicability of

### TABLE 3 RESULTS OF DEFLECTION VERSUS LOAD ANALYSIS

<table>
<thead>
<tr>
<th>Sec. 001 Regression Output:</th>
<th>Sec. 004 Regression Output:</th>
<th>Sec. 006 Regression Output:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Std Err of Y Est</td>
<td>1.267</td>
<td>1.839</td>
</tr>
<tr>
<td>R Squared</td>
<td>0.995</td>
<td>0.988</td>
</tr>
<tr>
<td>No. of Observations</td>
<td>10</td>
<td>8</td>
</tr>
<tr>
<td>Degrees of Freedom</td>
<td>9</td>
<td>7</td>
</tr>
<tr>
<td>X Coefficient(s)</td>
<td>0.572</td>
<td>0.427</td>
</tr>
<tr>
<td>Std Err of Coef.</td>
<td>0.005</td>
<td>0.007</td>
</tr>
<tr>
<td>Std Err of Y Est</td>
<td>5.229</td>
<td>5.229</td>
</tr>
<tr>
<td>R Squared</td>
<td>0.950</td>
<td>0.950</td>
</tr>
<tr>
<td>No. of Observations</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>Degrees of Freedom</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>X Coefficient(s)</td>
<td>0.876</td>
<td>0.876</td>
</tr>
<tr>
<td>Std Err of Coef.</td>
<td>0.022</td>
<td>0.022</td>
</tr>
</tbody>
</table>
layer elastic theory to laterite pavements and indicates that correction for nonlinear behavior is not necessary. Although unbound pavement materials tend to show significant stress-dependent properties when tested in the laboratory, the results previously discussed indicate that they exhibit linear behavior as part of pavement structures.

Different axle loads have been used by various agencies to measure pavement deflection. The finding of proportionality between axle loads and pavement deflection indicates that a simple proportion can be used to convert between Benkelman deflections obtained with two different axle loads, making it simpler to compare and interpret deflection data from different sources.

**RELATION BETWEEN BENKELMAN BEAM AND DYNAFLECT ON LATERITE PAVEMENTS**

Pavement structural capacity can be evaluated through laboratory testing of the materials or directly, by in-place tests in the field. Field tests commonly used in a number of countries include Benkelman beam and Dynaflect. The Benkelman beam has had long and widespread use and is probably more familiar to pavement designers and engineers than any other deflection measuring device (35). The Dynaflect is an electromechanical device that consists of a dynamic cyclic-force (1,000-lb, 8-Hz) generator mounted on a two-wheel trailer, a control unit, and a sensor assembly (38).

Several equations relating Benkelman beam and Dynaflect deflections have been published, as reviewed by Paterson (39). Deflection data collected on Brazilian and Nigerian laterite pavements, using both Benkelman beam and Dynaflect devices in (quasi-) simultaneous measurements, are used here to investigate the relationship between these two types of deflection for laterite pavements.

The data from Brazil was collected as part of the Brazil-UNDP study (24) and included pavements in the Brazilian central plateau built with as-dug laterite bases and asphalt concrete surfacings (26,40). The Dynaflect parameter considered is the maximum deflection, that is, measured at the center of the applied load (or Geophone 1 deflection). Figure 3 shows the data points and the relationship obtained by regression analysis:

\[ B = 23.4D \]  

where \( B \) and \( D \) are Benkelman and Dynaflect deflections, respectively, in 0.01 mm. The \( r^2 \) value is 0.21, standard error of the \( B \) estimate is 16, the number of degrees of freedom is 85, and the \( t \)-statistic of the coefficient is 33. Adjusting a quadratic equation to the same data would yield a somewhat higher coefficient of determination \( (r^2) \) with still significant coefficients (at the 5 percent significance level):

\[ B = 35.3D - 4.3D^2 \]  

The \( r^2 \) value is 0.41, the standard error of the \( B \) estimate is 14, and the \( t \)-statistic values of the coefficients are 15 and 5, respectively.

The data from Nigeria were obtained from pavements built with as-dug laterite bases and asphalt concrete surfacings in Kaduna and Niger States (41). Figure 4 shows the scatter diagram and the linear regression equation derived from the data:

\[ B = 35.2D \]  

The \( r^2 \) value is 0.41, the standard error is 12, the number of degrees of freedom is 296, and the \( t \)-statistic of the coefficient is 64.

The relationships in Figures 3 and 4 show significant scatter and indicate that the two deflection measuring devices give different rankings of pavement strength in some cases. As Paterson (39) concludes, deflection measurements by Benkelman beam and Dynaflect are not directly interchangeable. The Dynaflect probably applies to the pavement a much lower treatment and asphalt concrete surfacings (26,40).
load (with a geometry that is different from real vehicle axles) than commercial vehicles do, resulting in stress levels and depth of influence that are not representative of pavement reaction under heavy loads.

Independent analyses of Brazilian road deterioration data by Paterson (39) and Queiroz (42) showed that Dynaflect deflection parameters (e.g., maximum deflection, surface curvature index, base curvature index, and spreadability) were very poor explanatory variables (rarely significant) for predicting pavement deterioration, whereas Benkelman beam deflections had much stronger explanatory power. Benkelman beam and Dynaflect were the only deflection measuring devices used in the Brazil-UNDP study.

Input variables to the Highway Design and Maintenance Standards Model, HDM-III, (43) include Benkelman beam deflections. The practical implication of the previously discussed results for users of HDM-III (or similar road investment analysis models) is that efforts should be made to measure deflections using a device similar to that used to develop the model. Otherwise, accuracy of the analyses will be limited by the degree of interchangeability between the base and the adopted deflection measuring device.

RELATION BETWEEN SOAKED AND UNSOAKED CBRS

One of the main selection criteria for unbound base materials has been the soaked CBR (SCBR). However, this criterion has been questioned by several investigators, including Millard (44). In well-drained pavements, the field moisture content of laterite bases is normally well below saturation, even in wet climates, but more so in arid or semiarid areas. (Pavement engineers in general consider it axiomatic that poorly drained pavements behave poorly, irrespective of the quality of the base material.)

Autret (2) has shown that SCBR can be much lower than as-molded CBR, and that letting the compacted soil specimen dry for 4 days in warm air leads to even higher CBR. Analysis of CBR results from laterite bases obtained in 10 different asphalt pavements in Central Brazil (25) yielded the following relationship between SCBR and unsoaked CBR (UCBR):

\[
\text{SCBR} = 0.78 \times \text{UCBR}
\]

The \( r^2 \) value is 0.36, the standard error of SCBR estimate is 23, the number of degrees of freedom is 18, and the \( t \)-statistic of the coefficient is 14. The scatter diagram and the derived relationship are shown in Figure 5.

In situ base course CBR values for the 10 previously discussed laterite pavements are also available (25). The field penetration test was performed similarly to the laboratory test by adapting a CBR press to a reaction truck, and the laterite base was kept at field moisture content. Analysis of the data showed erratic correlations between laboratory CBR values (soaked and unsoaked) and field CBR values, as expected for granular materials (36). Although no significant regression equation could be generated, the average relationships observed are given by the following equations:

\[
\text{ICBR} = 1.02 \times \text{UCBR}
\]

and

\[
\text{ICBR} = 1.27 \times \text{SCBR}
\]

where ICBR is the in situ CBR value of the laterite base.

On the trial sections at Mataara Gatura in Kenya, ICBR values carried out on the surface of the laterite base 10 years after surfacing showed a mean value of 92, whereas the mean 4-day SCBR value of the same material was 53. Similar tests carried out at Luwawa Champhoyo in Malawi showed a mean ICBR value of 80 and a 4-day SCBR value of 31 (22).
LONG-TERM VARIATION OF PAVEMENT MOISTURE CONTENT

The possible reduction in pavement material strength caused by increased moisture content and the normal assumption that pavement moisture contents in flexible pavements will, at some time, reach saturated conditions, leads to the rejection of many naturally occurring materials (e.g., laterites) whose SCBR values are below performance requirements. However, three test sections of roads built in 1967 and 1969 in Australia indicate that the long-term pavement moisture content of pavements are more related to as-constructed, as-sealed moisture content than to environmental or climatic conditions. Beavis' evidence (45) indicated that pavement moisture content remained at, or very close to, as-sealed levels (Table 4).

Consequently, if pavements are allowed to dry out after construction and before sealing, and if the in situ moisture contents should remain at or near those levels as demonstrated in Australia, the acceptance criterion for pavement materials could more appropriately be the CBR value at the projected in-situ moisture content than the SCBR value.

That same series of tests indicated that the moisture contents in the middle 4.5 to 5.0 m of a 6.5- to 7.5-m sealed pavement remained substantially stable irrespective of climatic conditions; variations in moisture contents were limited to the 0.5 to 1.0 m of pavement nearest to the outer sealed edges. Beavis (45) concluded that the adverse effects of this increase in moisture content and subsequent weakening of the materials (especially the subgrades) under the edges of paved roads warranted “partial bituminous surfacing of shoulders . . . for a width of approximately 1 m outside edge lining.”

Results from the Brazil-UNDP study (24) were consistent with those from Australia. In 18 laterite-base pavements in central Brazil, the moisture content of the laterite base was usually very close to optimum moisture content and was only 0.2 percentage points higher on average. Those field moisture contents ranged from 6.5 to 12.8 percent, and on a larger set of 25 pavements, up to 19.2 percent, in a humid climate of 30 to 100 on the Thorthwaite moisture index. Analysis of the materials showed that the field moisture content was related to the percentage of fines and the liquid limit of the material, as follows:

\[
EMC = 3.75 + 0.167P075 + 0.136LL
\]  

where EMC is the field equilibrium moisture content (percent) and LL is the liquid limit (percent). The \( r^2 \) value is 0.49, the standard error of EMC estimate is 2.59, and the \( t \)-statistics are 1.7 and 2.5, respectively. PI and climate moisture index had no significant influence within the range observed.

PERFORMANCE

Intensive analysis of performance for a wide range of flexible pavement types, including natural gravel, crushed stone, and cement-stabilized bases, and wide ranges of pavement strength (0.2- to 2.0-mm Benkelman deflection under 80-kN axle load) and traffic loading [from 300 to 2 million equivalent standard 80-kN axle (ESA) loads per lane per year] have been reported by Paterson (39). This analysis was based largely on the Brazil-UNDP road costs study but also incorporated extensive validation from other countries and climates. When the performance of the lateritic base pavements from the Brazil-UNDP study were compared with other pavement types in these global models, the performance of lateritic flexible pavements proved to be similar to that of other pavement types of equivalent strength, and in some cases superior.

The time before cracking first appears is a function of both the time of exposure to weathering and the traffic loading relative to the pavement strength. The cracking behavior of 23 lateritic pavements observed in the Brazil-UNDP (24) study compared well with predictions from the following models (39):

Asphalt concrete:

\[
TY = 4.5 \exp (-0.14SNC - 0.2Y/E/SNC^2) 
\]  

Surface treatment:

\[
TY = 13.2 \exp (-1.7YE/SNC^2) 
\]

where

TY = predicted surfacing age (years) at first cracking,  
YE = annual traffic loading (million ESA per lane per year), and  
SNC = modified structural number of pavement strength.

---

**TABLE 4 MOISTURE OF PAVEMENT MATERIALS WITHIN MIDDLE 4.5 m IN FOUR TEST SECTIONS IN AUSTRALIA**

<table>
<thead>
<tr>
<th></th>
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<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Merredin</td>
<td>Laterite Gravel</td>
<td></td>
<td>89</td>
<td>-</td>
<td>89</td>
<td>91</td>
</tr>
<tr>
<td>Loxton</td>
<td>Crushed Limestone/Limestone Rubble</td>
<td></td>
<td>-</td>
<td>62</td>
<td>31</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>Limestone Rubble</td>
<td></td>
<td>-</td>
<td>46</td>
<td>30</td>
<td>32</td>
</tr>
<tr>
<td>Lamerro</td>
<td>Sand Clay</td>
<td></td>
<td>-</td>
<td>40</td>
<td>39</td>
<td>51</td>
</tr>
</tbody>
</table>

Source: Beavis (45)
Asphalt concrete surfacings on lateritic bases tended to crack 27 percent earlier than expected when compared with all crushed-stone or natural gravel base types; these observed lives also tended to be rather short, but that was partly a function of the brittleness and susceptibility of those asphalt materials to weathering. Double-surface-treatment surfacings, however, which are more flexible than asphalt concrete, survived at least as long as expected on lateritic bases when compared with other base types, and the observed lives were much longer, ranging from 11 to 19 years. Once cracked, cracking progression rates were not significantly different from those for nonlateritic base types for both types of surfacing. Lateritic bases, therefore, tend to be less suitable for asphalt concrete surfacings than high-quality crushed-stone bases, and the fatigue strength of the asphalt material or the pavement strength must be used to compensate.

Roughness in flexible pavements has been shown to develop as a function of three types of components: (a) structural deformation, (b) surface defects, and (c) nontraffic or environmentally related deformation (34). When the roughness progression of the 25 laterite pavements was compared with the predictions of the general model, the predictions represented the observed behavior well.

**BASE MATERIAL SPECIFICATIONS**

Road-building practices, in general, have been heavily influenced by the prevailing conditions and pavement performance and behavior in temperate and cold climates. In those areas, pavement designers take great care to avoid the use of any materials in the pavement layers that are susceptible to the weakening effects of water and frost. Crushed rock and river-washed gravels are the predominant natural materials used for building roads in the higher latitudes. Because of the limited empirical or scientific research data from other parts of the world, the normally accepted material specifications for pavement bases in those areas are based on the pavement materials and their performance and behavior in the higher latitude countries. As pointed out by Toole and Newill (46), these specifications often result in acceptance of unsuitable materials and rejection of satisfactory materials. The consequences are that unanticipated financial penalties can be incurred or potential savings lost.

Some agencies have tried to disseminate more realistic material specifications for tropical regions. For example, 30 years ago the Portuguese National Civil Engineering Laboratory recommended increasing to 15 the maximum acceptable PI value for laterites to be used as base courses, provided that swelling does not exceed 1.0 percent (47).

Experience gained over the last four decades on the behavior and performance of laterite pavements indicate that as-dug laterites, which are outside the range of normally accepted specifications, have provided satisfactory performance as base courses without incurring increased road maintenance and vehicle operating costs. But examples of premature road failures attributed to particular materials and their deficiencies also exist. These failures are normally confined to a 1-m-wide strip of the pavement adjoining the edge of the surfaced pavement.

Previous sections presented some of the recorded field evidence from several countries, including Australia, Brazil, The Gambia, Niger, Nigeria, Kenya, and Malawi. On the basis of this evidence, more realistic specifications are proposed for the use of laterite bases, which would result in more widespread use of locally available natural materials in pavements. As long as special care is taken during construction to ensure adequate compaction and uniformity along the road, and if the bituminous surface is extended at least 1 m beyond the normal pavement width, the following specifications for using as-dug laterites in base courses could be adopted:

- Mean CBR value at 95 percent MDD British 4.5 kg compaction after 4-day soaking: not less than 40;
- Minimum CBR value at 95 percent MDD British 4.5 kg compaction after 4-day soaking: not less than 20;
- PI: not greater than 20;
- Percent passing 0.076-mm sieve: not more than 40; and
- Los Angeles abrasion of course grains: not greater than 60.

These specifications will allow the use of as-dug laterite from the majority of naturally occurring laterite gravel deposits. The main basis for relaxing existing specifications is the evidence of good performance and behavior of laterites that comply with the new specifications. However, additional factors also support the adoption of the relaxed specifications: (a) in situ material properties are the ones influencing performance, and ICBR values of laterite pavement layers tend to be higher than laboratory-soaked CBR values; and (b) laterite pavement strength tends to increase over time and under traffic.

The substitution of as-dug laterite for crushed stone or materials stabilized with cement or lime would result in substantial savings in construction cost without incurring additional road maintenance or vehicle operating costs. In a typical case, the savings in construction costs amounted to over $40,000/km in a 50-km contract. On the average, a crushed-stone base costs three to six times more than a laterite base.

**CONSTRUCTION PROCEDURES**

The inherent physical and chemical properties of laterites and the moisture behavior of laterite pavements under sealed surfaces are the factors that led to the recommendations for relaxing specifications for laterites used in pavements. However, the specifications, controls, and management of the construction process should not be relaxed. It is critically important that construction procedures be carefully defined and closely controlled and supervised. This control is particularly critical in quarry definition and working; laboratory testing and control; and site procedures for handling, spreading, compacting, grading, and making preparations for applying the seal coat.

In the quarry or pit, the depths of overburden and usable laterite, the quality of the materials, and the methods of working the pit must be carefully defined and specified. A grid of test pits and laboratory tests are essential to define the usable materials and the way in which the pit should be worked to prevent contamination of the usable material with unsuitable overburden, unacceptable laterite, and poor material in the
floor of the pit. Normally, deposits with less than 10 000 m$^3$ of usable laterite, in layers less than 0.5 m thick would be difficult to use in road works other than routine maintenance. To avoid contamination, average quarry yields would rarely exceed 50 percent of the total identified suitable material. In calculating material quantities, pavement compacted densities will be 20 percent higher than in situ quarry densities.

Procedures on the road must ensure that the material is worked in compacted layers between 75 and 150 mm deep. The issue of pavement layers is critical. and under no circumstances should thin make-up layers be added to compacted surfaces. Also, surface ponding must be avoided by ensuring a well-drained surface profile. Following compaction, as the surface dries out, cracks of about 1 to 2 mm, and occasionally up to 4 mm, may occur. These cracks will gradually disappear with continued watering, grading, and rolling. The use of traffic during this period could accelerate the process and more rapidly identify weak spots in the pavement that may have been caused by contamination of the laterites. These weak spots could then be repaired before sealing. After final compaction, the surface sweeping and cleaning preparation should expose the mosaic of laterite nodules to which the seal coat will be bonded.

ENVIRONMENTAL IMPACT

In addition to its economic and technical benefits, the use of as-dug laterites to build pavement bases may bring about positive environmental impacts, including (a) reducing air pollution because of the improved fuel efficiency of vehicles on the longer lengths of roads that can be upgraded or kept in good condition with the construction cost savings; (b) avoiding the use of energy that would be required for crushing and hauling rock; and (c) avoiding the importation of chemical products into the country, and the use of energy that would be required to process and haul materials to build chemically stabilized bases. Moreover, the increased use of materials such as laterites would enable developing countries to rely more on local resources and capabilities, thus reducing the requirements for foreign exchange. When opening the Seventh Conference of African Ministers of Transport. Adedeji (48) stated that every penny that can be saved, must be saved, and wastage of resources must be completely eliminated. The better use of local materials fits this policy statement well.

ADDITIONAL AND CONTINUING STUDIES

The proposed specifications can be adopted immediately for road construction and rehabilitation projects not exceeding 500 vehicles per day per lane, including up to 50 percent commercial vehicles, in areas at an elevation of not more than 2000 m, where the annual rainfall is not more than 2000 mm, providing the permanent water table is well below the road level. However, final conclusions on the precise relationship between laterite characteristics, environmental factors, traffic loadings, and pavement performance can not yet be drawn. Efforts are under way in several countries to further investigate the performance and behavior of laterites in pavement construction and rehabilitation. Trial sections are being built near Faraffenni in The Gambia, and others are planned for construction on the Gabu-Buruntuma road in Guinea-Bissau. Additional work is being planned in Malawi, where a 100-km section of road is being upgraded to bituminous standards using naturally occurring lateritic base materials and the knowledge of construction procedures obtained from the Malawi Trial Section. The results of these trial sections will not be available for some years. However, many road sections have been built in the past with as-dug laterites and have not yet been systematically assessed and recorded. These existing sections could give an excellent sample from which actual performance and behavior could be analyzed. A systematic survey and study would provide a valuable laboratory for data of use to many developing countries.

Research authorities in several countries have recognized the useful properties of lateritic materials for road base construction, and studies and investigations have been undertaken in the United States, Australia, Kenya, Malawi, Gambia, Nigeria, Niger, Cote d'Ivoire, Brazil, England, France, Portugal, South Africa, and elsewhere. These investigations have shown beyond all reasonable doubt that naturally occurring lateritic materials can be successfully used as base materials for bituminous-surfaced roads. So far, however, very few highway authorities have amended their specifications to permit the use of these materials.

It is understandable that highway authorities, consulting engineers, and funding agencies are reluctant to relax specifications that have proven satisfactory over an extended period. However, these standard specifications, which have been developed in the temperate zones of North America and Europe, are overconservative or simply not applicable for conditions in tropical and semitropical regions. It is also understandable that any international authority would hesitate before assuming responsibility for such a research project. All research projects are by their nature open-ended. In view of the extensive work that has been carried out during the past 20 years, this research project could be finalized in a relatively short period of about 3 years. Few research projects would be likely to produce more benefits in less time for less cost.

The primary objective of further studies on this subject should be to provide hard evidence, in the form both of field and laboratory data, to substantiate the case for the use of suitable naturally occurring lateritic materials in base courses of bitumen-surfaced roads.

The next step would be for some international authority, such as the Permanent International Association of Road Congresses or the International Road Federation, to collect and examine the various studies and investigations that have been completed. This examination should suggest what additional work, if any, is needed and how and where it should be carried out. Finally, an authoritative document should be produced specifying the range of lateritic materials that can be used and the construction procedures that should be employed to ensure their satisfactory performance.

CONCLUSIONS

Data on the behavior and performance of bituminous-surfaced roads with base courses built from as-dug laterites has been
obtained from several countries. Although the use of these materials frequently does not comply with normal, internationally accepted specifications, many have shown excellent behavior and, in several well-documented cases, they have behaved marginally better than more costly alternatives, such as crushed-stone and cement-treated bases.

On the basis of the behavior of laterite pavements, new specifications for the selection of lateritic materials for base courses are advocated. These specifications would represent significant relaxation of previous internationally accepted limits. First, however, various completed studies and investigations need to be collected and examined. Additional work, if any, needs to be identified. Finally, an authoritative document should be produced, specifying the range of lateritic materials that can be used and the construction procedures that should be employed to ensure their satisfactory performance under a wide range of traffic loadings and environment. This is a role that some international agency should undertake.

REFERENCES


47. As Laterites do Ultramar Portugues. Memory No. 41. Portuguese National Civil Engineering Laboratory, Lisbon, Portugal, 1959.