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Recent Investigations Into the Use of Plastic Laterites as Bases for Bituminous-Surfaced Low-Volume Roads

H. GRACE AND D. G. TOLL

The development of specifications for bases for bituminous-surfaced roads in temperate zones is described in this paper. These specifications were developed in the 1920s and 1930s when compaction plant was light compared with that available today. These specifications have been adopted with limited modifications by developing countries in tropical zones for low-volume roads. The majority of lateritic gravels do not comply with the requirements of these specifications and bases of crushed stone or stabilized materials are normally used. A description is provided of full-scale, trial sections of road in Kenya and Malawi in which plastic lateritic gravel bases were used and the test results of site investigations gathered over a number of years. These trial sections have performed satisfactorily. A 3-year laboratory study of the Kenya laterite has been undertaken at Imperial College London to ascertain the reasons for their satisfactory performance. The construction procedures used and the relative densities found in the subgrade are also described. It is concluded that the satisfactory performance of the plastic laterite bases was a result of their high degree of compaction, the grading that resulted in low permeability and high stiffness, the construction procedures adopted, and the well-drained subgrade.

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The construction of an all-weather access road is a vital step in the evolution of a subsistence economy to a trading economy. This evolution is necessary if the standard of living is to be raised and would help solve the many problems associated with education, health, transportation, and famines.

A network of all-weather roads ensures that the communities in its vicinity can benefit from and contribute to the development process through all the seasons of the year. An all-weather road network normally requires the provision of a bituminous surface, at least on the steeper grades.

The currently accepted specifications for bases beneath a bituminous surface impose limitations on grading, plasticity, and bearing capacity. In the majority of cases, the requirements of these specifications eliminate the use of locally occurring "as dug" materials. Processed materials such as crushed and graded stone or stabilized materials are therefore required. Both materials are expensive and cost much more than locally occurring as-dug materials.

When an earth or gravel road is upgraded to bituminous standards, the provision of the base course with a bituminous surface constitutes the greatest proportion of the cost. The savings that can be realized by using as-dug, locally occurring gravels as base material instead of crushed stone are substantial and often amount to between 20 percent and 60 percent of the total cost.

Lateritic gravels are widespread in developing countries. These gravels do not generally comply with accepted base specifications but they have often been used for economic

reasons. In many cases they have performed satisfactorily. There are, however, instances in which the use of such materials has led to extensive failures that required complete reconstruction, but such occurrences have rarely been reported.

A discussion is provided in this paper of the development of existing base specifications and the results of recent investigations of full-scale trial sections of road in Kenya and Malawi in which base materials were used that do not comply with currently accepted specifications (referred to hereafter as nonstandard materials). The properties of these nonstandard materials and the construction procedures that have contributed to their successful performance are also described.

DEVELOPMENT OF CURRENT BASE SPECIFICATIONS

The majority of developing countries have received their existing specifications for base materials from the developed countries in the early, post-World War II period. Conditions in these developed countries, which are usually located in temperate zones, are often radically different from those in tropical developing countries.

Temperate zone specifications are designed for areas in which prolonged periods of wet, cold, or freezing conditions occur. They were developed during the 1920s and 1930s, when the principles of compaction were little understood and compaction plant was comparatively light and ineffective compared with that available today.

It is not surprising, therefore, that the developing countries have introduced modifications to the original, temperate zone specifications. However, funding agencies and government departments are naturally reluctant to change standards that have produced reliable results over many years. Modifications to original specifications have therefore been limited. Nevertheless, a few countries led others in relaxing specifications for low-volume roads. A summary of the main requirements of the specifications for countries in the tropics is provided in Table 1. Relevant sections of Transport and Road Research Road Note 31 and the French BCEOM and CEBTP specifications for laterite are given for the sake of comparison (BCEOM and CEBTP designate Bureau Central d'Etudes pour les Equipements d'Outre Mer and Centre Expérimental de Recherche et du Bâtiment et des Travaux Publics, respectively).

STUDIES OF LATERITIC GRAVELS

During the last 25 years, numerous studies have been made of the properties of lateritic gravels. Townsend, Krinitzsky, and Patrick have reviewed the work undertaken on a worldwide basis in a recent paper (*1*). The majority of this work has been devoted to laboratory and desk studies. Limited field studies of roads using nonstandard base materials have been reported by Lundgren (*2*). Test or trial sections of road have not generally been constructed because of the cost, the time required to obtain reliable results, and the fact that results from trial sections are valid only for the conditions that exist at the site of the trial. Extrapolation of these results to other conditions must be made within well-defined limits that account for all relevant environmental factors.

Full-scale trial sections, however, have the following advantages, especially when linked to appropriate laboratory studies:

- They provide a visible demonstration of what works and what does not work in the chosen environment.
- It can be assumed that similar nonstandard materials will perform satisfactorily in environmental conditions that are equal to or more favorable than those present at the site of the full-scale trial section.

TABLE 1 SPECIFICATIONS FOR BASES FROM VARIOUS COUNTRIES IN TROPICAL ZONES (*1*)

Country or Organization	Soaked CBR	Liquid Limit	Plasticity Index	Passing 63-μm Sieve	Remarks
Brazil	>60	<40	<12	5-20	For light traffic
Cameroons	>80	-	10-25	-	
Gabon	>60	-	20	-	
Gambia		20-37	13-22	13-28	
Côte d' Ivoire	>60	-	5-20	26-40	
Kenya	>50	-	15 or 20[a]	-	
Malawi	>85	<30	<6	5-15	
Mali	>50	-	6-16	-	
Niger	>80	-	<12	<25	
Nigeria	>80	<25	<10	5-15	
Senegal	>80	-	10-25	20-35	
Uganda		37-48	16-25	19-38	
Zambia	>120	<30	<6	-	
Road Note 31	>80	<25	6	5-15[b] 10-25[c]	
BCEOM-CEBTP	>60		<15	4-20	For traffic up to 300 vpd

Note: The CBR values have generally been determined from samples compacted to 95 percent of the maximum dry density obtained in the British Heavy Compaction (Mod AASHTO) Test. All tests are performed after soaking for 4 days.

[a] In wet areas less than 700,000 standard axles.
[b] Maximum size of gravel is 1 1/2 inches.
[c] Maximum size of gravel is 3/8 inches.

- They have a greater impact on the perspective of those responsible for allocating funds than theoretical or laboratory studies.

Further studies of the performance of roads made with nonstandard base materials have been made since the Lundgren report (*2*). In 1972 the UNDP commissioned a study to investigate why a number of roads in northern Nigeria in which nonstandard lateritic gravels were employed as bases had performed satisfactorily (*3*). In 1980 a paper by Grace and Erridge described how nonstandard materials had performed successfully for 8 years in a number of trial sections of road in the tea growing areas of Kenya, and the financial savings that resulted from this form of construction (*4*).

In 1982 the British Government and the Leverhulme Trust allocated funds for a detailed study of selected trial sections of road in Kenya. A 3-yr laboratory study was financed by the Science and Engineering Research Council at the Imperial College of London to determine the reasons for the successful performance of these nonstandard materials. The results of the Kenya study were included in a paper by Grace and Toll in 1985 (*5*).

The work in Kenya attracted the interest of the World Bank and in 1983 it was decided to include a 1-km trial section of road that used nonstandard laterite instead of crushed stone as part of a Malawi Government road contract on the Viphya plateau.

A description of the recent findings of a supplemental study in Kenya and the Malawi trial section follows.

KENYA AND MALAWI INVESTIGATIONS

Kenya Site Investigation

In 1982 investigations were started in Kenya to obtain information from a number of trial sections of bituminous-surfaced low-volume roads constructed in 1973 and 1974 using nonstandard laterite as base material.

The investigations continued from 1982 to 1986 and data were collected concerning the trial sections of roads, their performance, and the traffic they had carried. Samples were simultaneously taken from the trial sections and a 3-yr laboratory study was undertaken at the Imperial College of Science and Technology to determine why the nonstandard materials performed satisfactorily.

A summary of the site investigations shows which results are most likely to be of interest to those concerned with the design and construction of low-volume roads.

The results from the site in Kenya relate only to one of the trial sections, the Mataara-Gatura Road. This road was selected because it is the longest trial section studied; half of it has a stone base and half has a laterite base. The laterite is typical of that found in many areas of Kenya. The road has performed satisfactorily with little maintenance for a period of over 10 years.

The significant features of the road are listed in Table 2. The most important test results from the 1985 investigation of the Mataara-Gatura Road are summarized in Table 3. The estimated California bearing ratio (CBR) values were calculated by using a relationship derived from a large number of laboratory tests that were performed on lateritic gravels from Kenya and Malawi as part of these investigations. This relationship is similar to the relationship obtained by Clegg (*6*).

TABLE 2 SIGNIFICANT FEATURES OF MATAARA-GATURA ROAD

Item	Description
Location	
Geographical	Eastern slopes of the Abedares
Site topography	60 km north of Nairobi Ridge or shallow side cut
Trial lengths	
Laterite base	6.2 km
Stone base	5.7 km
Carriageway width	5 m
Thickness of base	150 mm
Construction period	
Base course	1973-4
Bituminous seal	Mid-1975
Subgrade	A red residual clay containing oxides of iron and alumina locally known as red coffee soil
Rainfall (mean yearly)	1,980 mm
Mean monthly temperature	15°-20°C
Depth to water table	Over 10 m
Source of laterite	Kiunyu Quarry 5 km northwest of Thika

Detailed visual inspections were made of the condition of the Mataara-Gatura Road in 1982 and 1985 and potholes, areas of crazing, edge failure, and patching were observed.

A comparison of the defects noted in 1982 and 1985 is made in Table 4. Negligible maintenance in regard to patching and surface drainage has been performed since the road was constructed in 1974 and 1975. It can be seen that the laterite base has performed marginally better than the stone base.

Although there was a greater number of potholes in the laterite base than in the stone base in 1985, the area of potholes in the laterite base was less than in the stone base because the laterite was less permeable and more cohesive. This restricted the tendency for the potholes in the laterite base to enlarge. It should be noted that the deterioration that took place in the 3-yr period from 1982 to 1985 was greater than the deterioration in the 8-yr period from the time of construction to 1982, which emphasizes the importance of performing maintenance at the appropriate time.

In March of 1983, 6-day, 12-hr traffic counts and a 3-day axle load survey were made on the Mataara-Gatura Road. An analysis of the results showed that the average equivalent standard axle (ESA) per medium and heavy vehicle was in excess of 0.64. An estimate has been prepared on this basis of the ESAs carried by the road since its construction until December 1986 that assumes traffic has increased by 5 percent per year since 1983. This and other traffic data are as follows:

ESAs since construction 12 yrs ago	100,000
Average daily traffic (ADT) (12 hrs)	190
Medium and heavy lorries (percent of ADT)	19

Arrangements are being made to perform more traffic and axle load surveys to provide accurate, up-to-date information.

Imperial College Laboratory Study of Lateritic Gravel from Kenya

A detailed study was made at Imperial College of the lateritic gravel used in the construction of the Mataara-Gatura Road.

TABLE 3 SUMMARY OF TEST RESULTS OF 1985 INVESTIGATIONS OF THE MATAARA-GATURA ROAD

Description of Test	No. of Tests	Test Results			Remarks
		Maximum	Minimum	Average	
CBR at 95 percent of maximum dry density, British Heavy Compaction Test after 4 days of soaking	19	96	21	54	
In situ CBR at surface of base beneath bituminous surfacing	40	209	47	89	CBR values estimated from Clegg Impact Hammer Tests
In situ relative dry densities (%)	48	113	97	105	
In situ relative moisture content	48	1.08	0.72	0.9	
Plasticity index	24	21	7	17.6	
Percentage passing 63-μm sieve	48	37	15	28	Clay fraction is 8 percent to 16 percent of total sample

Note: In order to approximately convert the relative dry densities to relative dry densities related to the maximum dry density obtained in the British Heavy Compaction Test (modified AASHO) it is necessary to divide the figures given by 1.05. This adjustment should be made before a comparison of the similar results in Table 6. The in situ relative dry density was computed by dividing the in situ dry density by the maximum dry density of the BS compaction test and multiplying the result by 100. The in situ relative moisture content was calculated by dividing the in situ moisture content by the optimum moisture content of the BS compaction test.

TABLE 4 COMPARISON OF DEFECTS IN STONE AND LATERITE BASES IN 1982 AND 1985, MATAARA-GATURA ROAD

Item	Stone Base		Laterite Base	
	1982	1985	1982	1985
Number of potholes per km	11.26	32.63	11.60	69.24
Area of potholes (percent of total area)	0.18	1.04	0.06	0.68
Area of crazing (percent of total area)	0.54	5.62	None	0.39
Total area of potholes and crazing (percent of total area)	0.72	6.66	0.06	1.07
Area of edge failures (percent of total area)	0.57	1.92	0.17	1.44

Some of the salient points of the investigation are outlined below. For full details of the study, refer to Toll (*7*).

The material from Kiunyu, laterite cuirrasse (or hardpan), broke down on excavation to a gravel. The percentage of gravel that passed the 63-μm sieve was between 15 and 37 percent (see Table 3) and the gravel had a clay fraction of 8 percent to 16 percent. The clay present was a kaolinitic mineral that had a low surface activity. Calculated values of activity ranged from 0.65 to 0.86 (*8*). The soil did not exhibit excessive shrinkage or swelling, which was expected from the small proportion and low activity of the clay.

Although the gravel contained only a small amount of clay, studies of the fabric of samples taken from the Mataara-Gatura Road show the clay to be widely distributed (*9*). This indicated that the compacted gravel had low permeability, particularly at high densities. The permeability values for samples at or near full saturation are shown in Figure 1. At a density of 94 percent (the maximum dry density (MDD) obtained in the BS compaction test), the permeability is 3.7×10^{-7} cm/sec but permeability is reduced significantly as the density is increased. The permeability is reduced to 4.5×10^{-8} cm/sec at 106 percent of MDD. Before surfacing in 1975, moisture contents in the base of the Mataara-Gatura Road had dropped to about 0.5 of British Standard (BS) optimum moisture content (OMC). Laboratory determinations of suction-moisture content relationships indicated that the in situ suctions at this moisture content would be high, exceeding 500 kPa. In this condition, the strength and stiffness of the material would be more than adequate to carry the traffic loadings. Measurements of a nearby road at Gatanga in a similar condition suggest CBR values would be of the order of 200 percent.

The critical period for a road base is when the moisture content increases, either because of direct infiltration during a wet season or as a result of moisture equalization with time. The low permeabilities noted earlier would have restricted infiltration. However, the investigations of the Mataara-Gatura Road show that moisture contents have increased from 0.5 of the OMC BS compaction in 1975 to just over 0.8 in 1982, and have reached about 0.9 of the OMC by 1985 (see Table 3). Even so, in situ CBR values estimated from Clegg Impact Hammer Tests averaged 89 percent, which is roughly twice that of the 4-day soaked CBR values (see Table 3).

Because infiltration tests and suction water content determinations show that large reductions in suction can result from only small changes in moisture content, the road must be able to operate at low suctions. For this reason only tests on saturated samples were considered, even though a large part of the study has examined the behavior of the material in its unsaturated state. Because of its low permeability, a transient wheel load can be considered undrained loading. Results of undrained tests

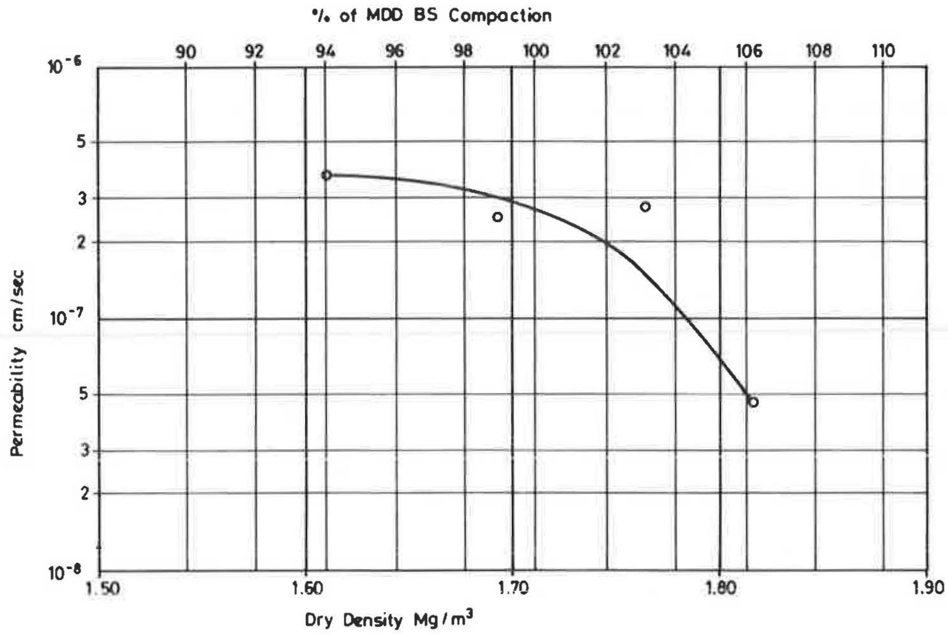

FIGURE 1 Permeability—dry density relationship, lateritic gravel, Mataara-Gatura Road.

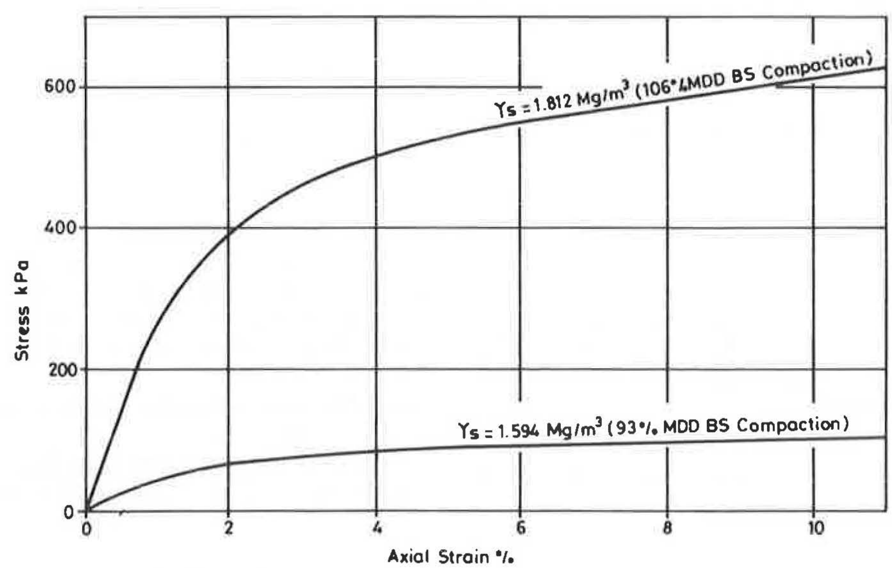

FIGURE 2 Stress-strain curves for lateritic gravel, Mataara-Gatura Road.

will therefore give an indication of the behavior exhibited in the field.

Undrained tests on saturated samples of compacted lateritic gravel that started from low initial effective stresses showed that failure was reached at an early stage. However, because the material was graded, it exhibited strongly dilatent behavior under shear. Under undrained conditions the dilatency is suppressed, which causes a decrease in pore water pressure. The development of negative pore water pressures allows the sample to carry greater stress. The stress-strain curves for two such tests on samples of differing density are shown in Figure 2. Both samples were consolidated under the same isotropic stress of 10 kPa.

Although both tests show that the load carrying capacity increased up to very high strains, it was only in a denser sample (106 percent of MDD BS compaction) that the response was sufficiently stiff to achieve reasonable stress levels at acceptable strains. The effect of an increase in stiffness with density can be seen in Figure 3. In order to compare tests at different confining pressures, the undrained stiffness E_u was normalized by $P^{\prime}_{0}$, which was the initial mean effective stress. Because of the nonlinearity of the stress-strain behavior, E_u was calculated as secant stiffness at three different strain levels. The stiffness values at 1 percent strain may have been applicable in the immediate vicinity of the wheel load, but lower strain levels should apply in less heavily stressed areas of the road base.

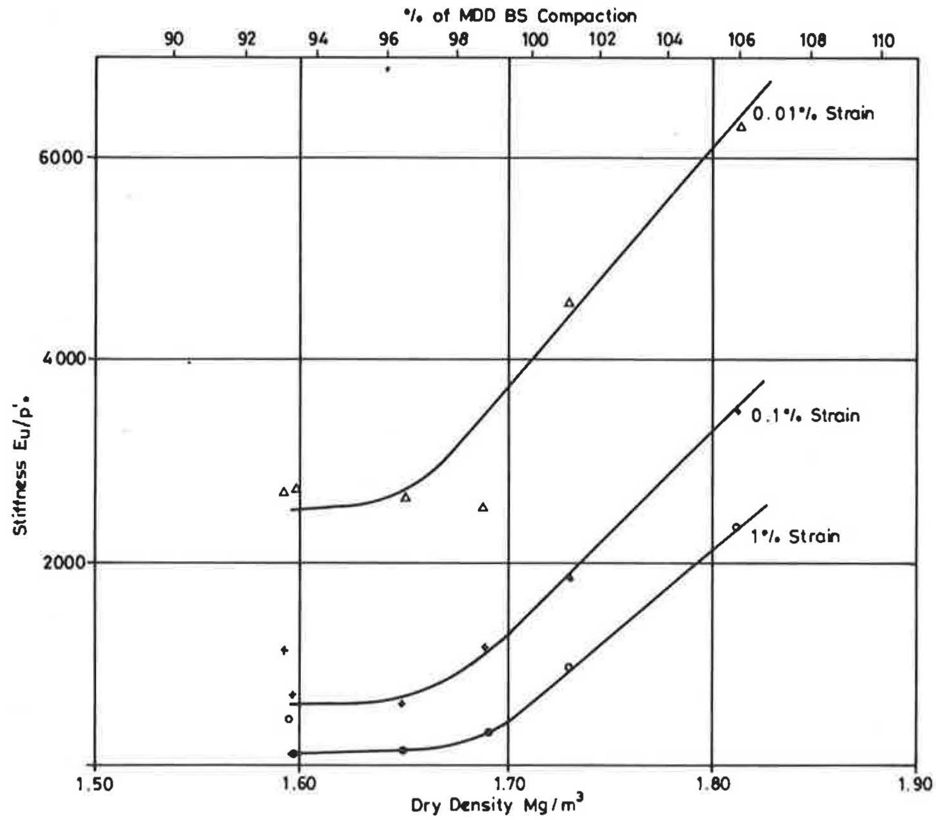

FIGURE 3 Stiffness-dry density relationship, lateritic gravel, Mataara-Gatura Road.

It can be seen that the stiffness of the material was comparatively low at densities below 98 percent of MDD for BS compaction. However, at higher densities the stiffness increased significantly. In the denser samples, the required stresses could therefore be carried without developing unacceptable deformations.

The densities achieved at construction of the Mataara-Gatura Road base averaged 102 percent of MDD BS compaction, and had increased with time to an average of 105 percent. At such a degree of compaction, the lateritic gravel has low permeability and high stiffness. It is this combination that explains how the road has performed satisfactorily, even under low suction conditions.

Malawi Site Investigation

In 1983 the Malawi Government authorized the construction of a 1-km trial section of road as part of a 50-km road contract. The trial section was identical to the remainder of the road with the exception of the base, in which as-dug lateritic gravel was used instead of crushed and graded stone. This resulted in a reduction in cost of $30,000 for 1 km of road.

The significant features of the road are listed in Table 5 and the most important test results are summarized in Table 6. The in situ CBRs given in Table 6 were estimated from Clegg Impact Hammer tests on the surface of the base at the 150-m sections at the north and south ends of the trial section, which were surfaced in January 1985 approximately 2 months after construction (see Construction period, Table 5). These tests were performed 18 months after the sections were surfaced and had been in existence for two wet seasons.

TABLE 5 SIGNIFICANT FEATURES OF LUWAWA-CHAMPHOYO ROAD AND TRIAL SECTION

Item	Description
Location	
Geographical	Southern edge of Viphya Plateau in northern Malawi Chainage 4+500 - 5+500
Site topography	Ridge or shallow side cut
Trial length	
Laterite base	1 km
Stone base	At either end for remainder of road
Carriageway width	6.7 m
Thickness of base, stone, or laterite	150 mm
Thickness of laterite sub-base	100 mm
Construction period	Nov/Dec 1984
Laterite base	Jan 1985 Chainages 4+500 - 4+650 5+350 - 5+500
Bitumen seal	Aug 1985 Chainage 4+650 - 5+350
Subgrade	Residual sand clay derived from biotite gneiss parent rock
Rainfall (mean yearly)	1300 mm rainy season November to April
Temperature (mean monthly)	16° - 18°C
Depth to water table	Over 10 m
Source of laterite	Quarry on line of road at Chainage 4+200

It was anticipated that the equilibrium moisture content would not be attained for some years after the sections were surfaced and the moisture content would probably increase, which would in turn result in a decrease in the in situ CBRs. The

TABLE 6 SUMMARY OF TEST RESULTS, 1986 INVESTIGATION OF THE MALAWI TRIAL SECTION

Description of Test	No. of Tests	Test Results			Remarks
		Maximum	Minimum	Average	
CBRs at 95 percent of maximum dry density, British Heavy Compaction Test after 4 days of soaking	23	45	12	31.7	
In situ CBR at surface of base beneath bituminous surfacing	24	120	58	87	Results confined to sections that were surfaced in January 1985. See Construction Period, Table 5
In situ relative dry density (%)	84	99.8	90.1	95.4	
In situ relative moisture content	84	1.14	0.70	0.93	
Plasticity index	23	18	14	16.3	
Percentage passing 63-μm sieve	23	49	26	37.6	

Note: These relative dry densities and moisture contents are not directly comparable with the results shown in Table 3, which need to be adjusted in accordance with the note of Table 3.

changes in both CBR and moisture content at the end of the 5-yr period after the road was surfaced are intended to be monitored.

The trial section was opened to construction traffic on January 26, 1985. At this time, the 700-m central section was still unsurfaced and was left in this condition during the rainy season until July 1985. Kerb stones were placed at appropriate intervals on the unsurfaced section of road and moved laterally about once a week. The full width of the road was thus subjected to the compacting action of traffic.

On a fine day over 200 vehicles used the road, 40 percent of which were 7-ton construction trucks, and half of those were being fully loaded. On wet days traffic reduced to about 50 vehicles. During the period between January 26 and July 11 of 1985, 500 mm of rain fell and 17,000 construction vehicles (approximately 100 vehicles per day) passed over the 1-km trial section. The unsurfaced section did not rut or corrugate but approximately 10 mm of lateritic base was eroded during this period by weather or traffic. The unsurfaced section was finally surfaced in early August of 1985, and on August 20, 1985, the complete trial section was opened to public traffic, which amounted to about 150 vehicles per day. An automatic traffic counter was installed and classified, and 7-day traffic counts and axle load surveys are being arranged.

No defects in the laterite trial section or the stone sections at each end were observed since the completion of construction. No difference in the riding qualities of the stone and laterite sections was detected.

Benkelman beam tests performed in June of 1986 showed that the maximum and the residual deflections measured were marginally less on the laterite trial section than on the stone control sections.

The condition of the trial section will be monitored at the end of each rainy season in May or June of each year. At the end of 5 years, an economic report will be prepared that will take into account construction, maintenance, and vehicle operating costs.

OBSERVATIONS

Construction Procedure

The Imperial College Laboratory study indicated that a high degree of compaction is necessary to ensure that the laterite retains its strength even when it is fully saturated. Adequate compaction is therefore a priority. During construction and the period before the road is surfaced, the following procedures should be followed:

- The surface of the base course should be accurately graded to ensure that all water on the surface will be shed. This is a normal practice of good engineering. If water is allowed to stand on the surface, potholes and ruts will form.
- The final grading and compaction should also result in a smooth, firm, closely knit, and dense surface. The surface will dry out and cracks will form, but the cracks will fill with dust and mud or they will be closed by the kneading action of traffic. It is essential that shrinkage cracking occur before surfacing.
- Traffic and the weather will gradually erode the fine material on the compacted surface until a rough surface is exposed that consists of hard laterite nodules, firmly locked into position by a dense and dried-out plastic soil mortar.

It was initially believed that the laterite base should be left unsurfaced for a complete rainy season. The base has performed in an unsurfaced condition for a complete rainy season without rutting or corrugating in the trial sections in Kenya and Malawi, where plastic laterites have been used and traffic has been less than 200 vehicles per day. Ruts and corrugations are likely to form relatively fast in cases in which the laterite is of lower plasticity and the traffic is heavier.

It is necessary that there be a period when the base is left unsurfaced to ensure that the base material dries out, any cracks that form are filled, and a rough, compact surface is developed

by the erosion of the fine surface material. The length of time required depends on the plasticity of the laterite, the intensity of traffic, and the weather, and can only be determined by experience. Recent experience in Malawi showed that sections that were exposed to traffic and the weather for 1 to 2 months have performed satisfactorily.

Densification of Subgrades

It has been known for many years that subgrades of roads, particularly those of cohesionless soils, undergo densification by traffic over an extended period. Studies have been made of the density required at various depths beneath motorway pavements, but similar information relating to low-volume roads has not been located (*10*). Current specifications call for the compaction of the subgrade to a given density. This is usually interpreted to mean that the top 150 mm of the subgrade must be compacted to the specified densities.

Density profiles beneath the Mataara-Gatura and Gatanga roads and comparable profiles of undisturbed soil in adjacent areas that were unaffected by construction or normal traffic are shown in Figure 4. It can be seen that considerable densification has been achieved; it extends to a considerable depth below the subgrade level, and it is still measurable at a depth of 1 m below road level.

Although the densification of the top 150 mm of the subgrade will be achieved by normal construction compaction, it has been found that additional compaction at greater depths has been achieved by traffic over an extended period of time.

If deformations of the pavement of a bituminous-surfaced road are to be kept within acceptable limits, the densities of the subgrade to a depth of at least 1 m must exceed a specified density profile with depth. The most reliable way of establishing the specified density profile is to determine the densities at various levels in the subgrade of a road that has the proven ability to carry the design traffic. The subgrade soil of this road should have similar properties to the subgrade of the road being designed. It is often found, especially with cohesionless soils, that the density requirements greatly exceed the densities necessary to produce the specified CBRs. The density profile is therefore often more demanding than the CBR requirements.

CONCLUSIONS

The successful performance of nonstandard materials as road bases can be attributed to the following:

- An adequate state of compaction,
- The characteristics of the material, particularly the grading

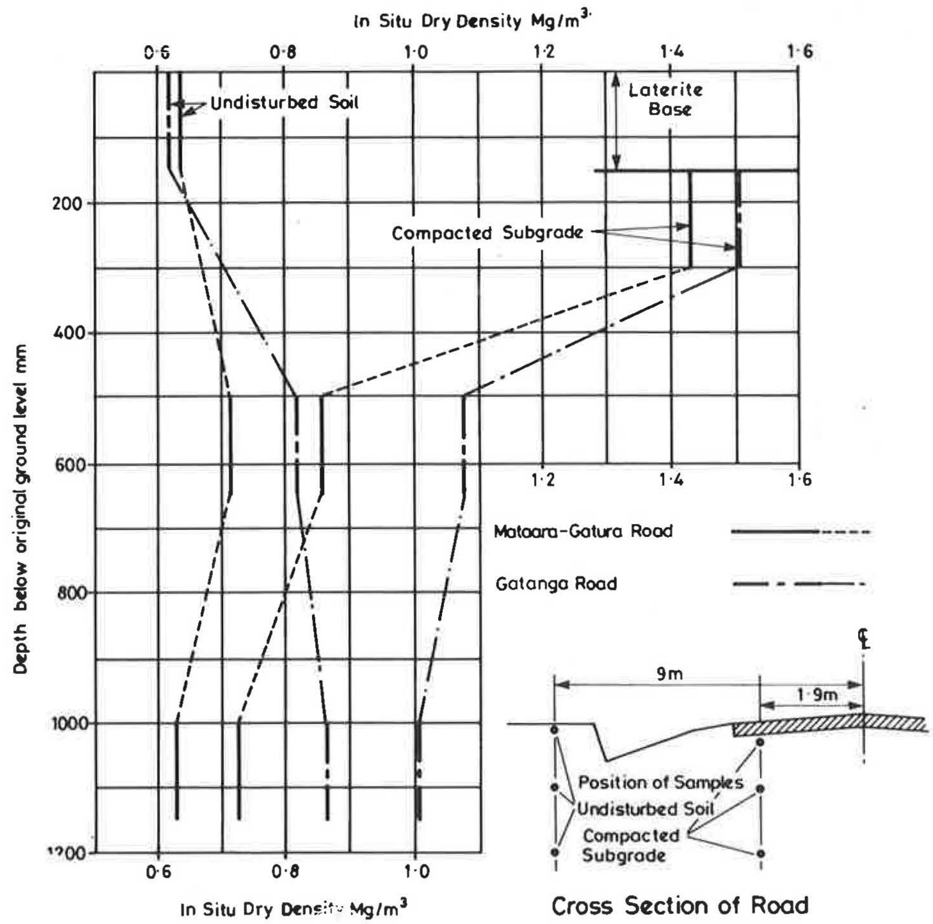

FIGURE 4 Comparison of in situ dry densities in undisturbed soil and in the subgrade of the same soil after compaction by low-volume traffic over many years.

and clay fraction, which result in low permeability and high stiffness at the densities achieved,

- The construction procedures, in which the base was exposed to weather and traffic for a period of time before it was surfaced, and
- Well-drained subgrade conditions.

Although these conclusions are based on tests that were performed on lateritic gravel, the same conclusions are likely to apply to other materials that have similar gradings, clay mineralogy, and distributions.

The laboratory tests suggest that, at adequate states of compaction, the lateritic gravels retain high stiffness even when they are saturated. However, no field observations support this conclusion to date.

Attention should also be drawn to the need to adequately compact the subgrade to a depth of at least 1 m beneath the finished road level.

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