Engineering Characteristics of Some Nigerian Residual Soils for Highway Construction

JOSEPH B. ADEYERI

The engineering characteristics of residual soils from the three major geological zones of Nigeria are analyzed and discussed. The characteristics of the parent rocks seemed to have some influence on the observed textural differences among the three groups of soils. There was also a fair degree of correlation between weathering, as indicated by grain size, and their consistency and swelling characteristics. The group A soils, which are nonplastic and granitic in origin, compact well at relatively low optimum moisture contents. Their CBR values at natural moisture contents, which are lower than optimum, are much higher than those at their optimum moisture contents. The soils are considered suitable as base and subbase materials because they compact well and have high CBR values. The group B soils are sedimentary in origin and have a high plasticity and relatively low CBR values. Unlike the group A soils, their natural moisture contents are higher than their optimums. There appears to be some correlation between the natural moisture content and the maximum dry density. Their high plasticity may make it uneconomical to stabilize them with chemical additives. The group C soils are lacustrine in origin, have very low CBR values, and exhibit a high degree of swelling. Therefore they are not considered suitable as base and subgrade materials. However, appreciable improvements in the compaction characteristics and increases in CBR values were recorded when the soils in this group were blended with local sand.

Nigeria is bounded by the Atlantic Ocean on the south and by semidesert conditions along its northern fringe. The vegetation varies from dense equatorial forests in the south to desert scrubland in the north. The mean annual rainfall varies from more than 3,000 mm (120 in.) in the south to as little as 600 mm (24 in.) in the north.

Climatically the country can be divided into four regions: the subequatorial south, the tropical hinterland, the tropical continental north, and the high plateaus. The highest point is the Chappal Wade of the Gotel Mountains, which are close to the Cameroon border in the eastern part of the country, with an elevation of about 2,419 m (800 ft) above mean sea level. Generally the seasons group into a wet period from about mid-April to mid-October and a relatively dry period for the rest of the year.

The geographical spread of soils follows to a considerable degree the geology of the country, as can be seen on the simplified geological map of Nigeria (*1*) shown in Figure 1 and on the soil map shown in Figure 2. Residual soils constitute the largest proportion of engineering soils in the country. These soils derive from the intense weathering of the crystalline rocks of the basement complex, which underlies most of the country. The soils are predominantly laterites and/or lateritic soils.

FORMATION

Laterites and lateritic soils are fully described in the literature (*2–6*), as are the residual soils of Nigeria (*7–10*). Although there is not complete agreement on many details, it is generally agreed that laterites or lateritic soils are residual soils produced by in situ weathering and chemical decomposition of the parent rock under humid tropical conditions. Weathering starts with the physical breakdown of the parent rocks from differential expansion and contraction of the rock-forming minerals due to temperature variations. Rainwater, which contains dissolved gases from the atmosphere and organic acids from plant decay, penetrates the rock, causing chemical reactions between itself and the constituent minerals of the rock. Colloidal and soluble products of these reactions are leached to lower horizons or are removed altogether. Different parent rocks produce different residual soils, which have many common features and some important differences.

The less stable soluble rock minerals (e.g., biotite, felspars, and micas) are attacked in the weathering process, whereas the more stable minerals (e.g., quartz) are generally unaltered. The result is that the quartz content of some laterites may increase from about 35 percent by weight in the parent rock to more than 50 percent in some residual soils. As much as one-third of the original rock volume could disappear from the rock matrix by leaching accompanied by a corresponding reduction in density. This process is often cited as the cause of the characteris-

Ondo State Polytechnic, Owo, Nigeria. Current affiliation: Department of Civil Engineering, Howard University, Washington, D.C.

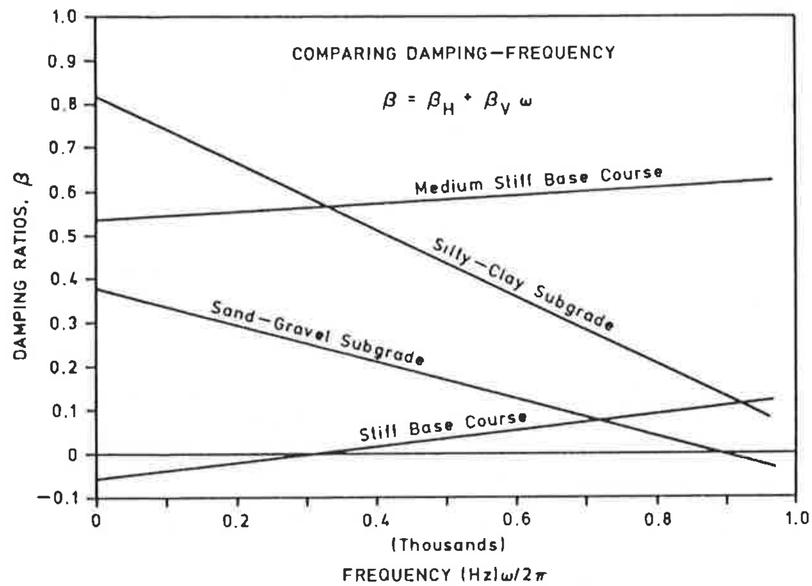


FIGURE 15 Damping ratios determined from spectral analysis for Section R10.

Figures 13 and 14, β_V and β_H can be found. Figure 15 shows the best-fit lines of the damping ratios for the four materials tested. Assuming that this model of viscous damping is correct, it appears that the hysteretic damping is proportional to the stiffness of the materials tested. The overall damping for the granular base course materials seems to increase with the frequency whereas the converse happens for the sand-gravel and the silty clay subgrade. The values of the hysteretic and the viscous damping ratios predicted for the various materials are shown in Table 1.

More likely than either of the foregoing simplified models of damping behavior in the base course and subgrade materials is the idea that the damping depends on the magnitude of the amplitude of movement at a given frequency as well as on the frequency itself. All of this suggests that the traditional methods of representing the damping properties of soils may need to be revised, and other implications of nonlinear vibrations need to be investigated. This makes the response of these materials nonlinear which can, in itself, explain the negative damping ratios that are known to occur with materials with softening stress-strain characteristics (*13*, pp. 439–452; *14*, pp. 130–154).

TABLE 1 DAMPING RATIOS OBTAINED FROM PAVEMENT DYNAMIC CONE PENETROMETER

Material Type	Damping Ratio β_H	β_V	Remarks
Equation 9: $k^* = k(1 + 2i\beta_H)$			
Granular base	0.04	—	Test R1XF
	0.16	—	Test R10K
Sand–gravel	0.21	—	Test R10P
Silty clay	0.33	—	Test R1XL
Equation 8: $k^* = k(1 + 2i\beta_H + 2i\beta_V\,\omega)$			
Granular base	−0.06	0.000029	Test R1XF
	0.53	0.000014	Test R10K
Sand–gravel	0.38	−0.00006	Test R10P
Silty clay	0.82	−0.00012	Test R1XL

CONCLUSION

This paper showed typical acceleration, velocity, displacement, and force histories of the PDCP in operation. The data were recorded using an accelerometer mounted on the extreme top of the PDCP. FFT techniques were used to transform the information recorded in real time to the frequency domain. Spectral analyses were performed and damping values were obtained for the base course and the subgrade material in two pavement sections. It was shown that it is possible to determine the hysteretic and the viscous damping ratios using the method of dynamic analysis described here. The averaged damping ratios were in accord with published values.

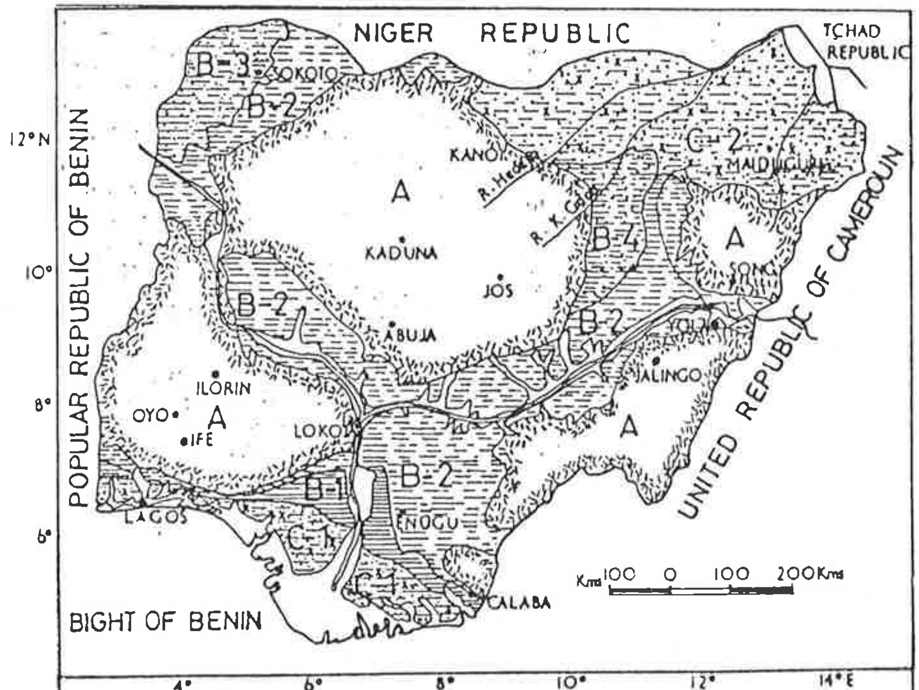
The study also shows that it is possible to determine damping properties of pavement materials in each layer in a relatively nondestructive manner in situ. The study also shows that it is possible to measure damping values of coarse granular materials. Results indicate that the frequency response is not a single value entity as is often assumed.

As a further refinement of the method presented here, it is suggested that a second accelerometer be mounted on the sliding hammer to record the force history. Also, a more efficient data acquisition unit could be used to improve the sampling rate and thus improve the resolution, in order to reduce the frequency interval between digitized points.

REFERENCES

1. E. Kleyn, J.H. Maree, and P.F. Savage. The Application of the Pavement DCP to Determine the In Situ Bearing Properties of Road Pavement Layers and Subgrades in South Africa. *Proc., 2nd European Symposium on Penetrometer Testing*, Amsterdam, May 1982.
2. J.A. Harison. *Correlation of CBR and Dynamic Cone Penetrometer Strength Measurement of Soils*. Technical Note 2, Australian Road Research Board, Vol. 4, 1986.
3. M. Livneh and I. Ishai. Pavement and Material Evaluation by a Dynamic Cone Penetrometer. *Proc., 6th International Conference on Structural Design of Asphalt Pavements*, University of Michigan, Ann Arbor, July 1987.
4. K.M. Chua. *Determination of CBR and Elastic Modulus of Soils Using a Portable Dynamic Cone Penetrometer*. Paper No. 065. First International Symposium on Penetration Testing, Orlando, Fla., March 20–24, 1988.
5. D.Z. Yankelevsky and M.A. Adin. A Simplified Analytical Method of Soil Penetration Analysis. *International Journal for Numerical and Analytical Methods in Geomechanics*, Vol. 4, 1980, pp. 233–254.
6. J.W. Cooley, and J.W. Tukey. An Algorithm for the Machine Calculation of Complex Fourier Series. *Mathematics of Computation*, Vol. 19, No. 90, 1965, pp. 298–301.
7. T.H. Wu. *Soil Dynamics*. Allyn and Bacon, Inc., Boston, 1971.
8. W.C. Hurty and M.F. Rubinstein. *Dynamics of Structures*. Prentice-Hall, Inc., Englewood Cliffs, N.J., 1964.
9. J.M. Roesset, R.V. Whitman, and R. Dobry. Modal Analysis for Structures with Foundation Interaction. *Journal of the Structural Division*, ASCE, Vol. 99, No. ST3, March 1973.
10. J.T. Christian, J.M. Roesset, and C.S. Desai. Two- and Three-Dimensional Dynamic Analyses. In *Numerical Methods in Geotechnical Engineering* (C.S. Desai and J.T. Christian, eds.), McGraw-Hill Book Co., New York, 1977.
11. P. Mario. *Structural Dynamics: Theory and Computation*. 2nd ed., Van Nostrand Reinhold Co., New York, 1985.
12. H.B. Seed and I.M. Idris. *Soil Moduli and Damping Factors for Dynamic Response Analysis*. Report No. EERC 70-10. Earthquake Engineering Research Center, University of California, Berkeley, 1970.
13. E. Volterra and E.C. Zachmanoglou. *Dynamics of Vibrations*. Charles E. Merrill Books, Inc., Columbus, Ohio, 1965.
14. W.T. Thomson. *Vibration Theory and Applications*. Prentice-Hall Inc., Englewood Cliffs, N.J.

Publication of this paper sponsored by Committee on Soil and Rock Properties.



GROUP	ORIGIN	RELATED GEOLOGIC FORMATION	
A	Granitic	Basement Complex	
B	Sedimentary	Predominantly Limestone	B-1
		Shale, Coal, Limestone, Sandstone	B-2
		Predominantly Sandstone - Gwandu	B-3
		Predominantly Sandstone - Kerri - Kerri	B-4
C	Transported.	Coastal Plains Sands	C-1
		Chad Formation	C-2

FIGURE 2 Distribution of residual soils, based on geological formation.

clude materials reflecting different stages in the process of decomposition. Soil handling causes further breakdown, which can also affect the engineering properties of the soil. Many writers have discussed the difficulty of testing these types of soil without changing their in situ properties. Rao and Ouoleye (*17*) have presented data showing the effect of different treatments on the Atterberg limits of a Nigerian residual silty clay of sedimentary origin. Their study showed an irreversible reduction of plasticity on air-drying or oven-drying the soil, which they attributed to aggregation of the particles (Table 1). The liquid limit of the soil was found to vary from 32 to 94 percent, while there was a corresponding change in the plastic limit from 18 to 38 percent, depending on the type of treatment. These results confirm a similar earlier observation by Terzaghi (*18*) on the tests by R.H.S. Robertson of a Hong Kong laterite. Winterkorn and Chandrasekharan (*19*) similarly reported that the liquid limit of Matanzas clay from Cuba increased from 46 to 56 percent by remolding, although the plastic limit was unaffected.

TABLE 1 EFFECT OF HANDLING ON ATTERBERG LIMITS AND STRENGTH OF A NIGERIAN RESIDUAL SILTY CLAY

Sample Treatment	Atterberg Limits LL	PL	Linear Shrinkage	Strength Cu/P	0'
Natural state	75.0	38.0			
Air dried	94.0	31.0	20.6	0.41	22.5
Oven dried	81.0	34.0	17.5	0.45	28.0
With NaCl[a]	82.0	23.0	15.0	0.49	25.5
With $Na(PO_6)_3$[b]	32.0	18.0	5.0	0.56	34.5

[a]NaCl: Sodium chloride solution.
[b]$Na(PO_6)_3$: Sodium hexametaphosphate solution.

For the Nigerian residual silty clay, it was also found (Table 1) that handling has a pronounced effect on the strength parameters (*c*, 0). The usual classification tests for evaluating the textural and plasticity characteristics of temperate-zone soils may not be adequate, particularly for lateritic residual soils. Grading analyses are often used for classifying normal residual soils from the same parent materials. The tests are very useful once their limitations are well understood and should be interpreted taking into consideration the origin (parent rock), mode of formation, and degree of weathering.

The grading analyses of residual soils (decomposed granite and rhyolite) from Hong Kong (*20*) and from some South American countries (*2*) confirm the wide range and variability of residual soils, showing that they can vary from red clay to cemented gravel. On the basis of field and laboratory experience with these soils, many investigators (*2, 12*) have texturally classified them into three main groups:

- Laterite rocks and boulders, which are cemented laterite rock and rock pieces that are used extensively in many African countries as concrete and road surfacing aggregates.
- Laterite and laterite-quartz gravels, which form the most important naturally occurring gravels and are extensively used as base and subbase materials in road construction.
- Laterite fine-grained soils, which have been given little attention in the past, but recent highway subgrade and shallow foundation failures have emphasized the importance of studying and understanding them. The magnitude of the clay fraction affects such properties as consistency, colloidal activity, compaction, specific gravity, and linear shrinkage.

The above textural groups reflect definite stages in the process of laterization.

CURRENT METHODS OF PAVEMENT DESIGN

Pavement design methods in Nigeria have been derived largely from the United Kingdom. The 1973 Nigerian Federal Highway Manual and the pavement design curves (*21*) have been adopted from the Transport and Road Research Laboratory, England (*22*). The design procedure requires estimating the daily traffic in terms of vehicles heavier than 29.8 kN (3 tons) loaded weight and determining the base and subgrade CBR. The base and subgrade materials must also meet required specifications regarding gradation and consistency.

The selection of the pavement structure is then made from CBR-versus-pavement-thickness charts (Figure 3), taking into consideration the expected loading of the pavement. The recommended minimum asphalt pavement thicknesses for roads in the country are as follows (vehicles

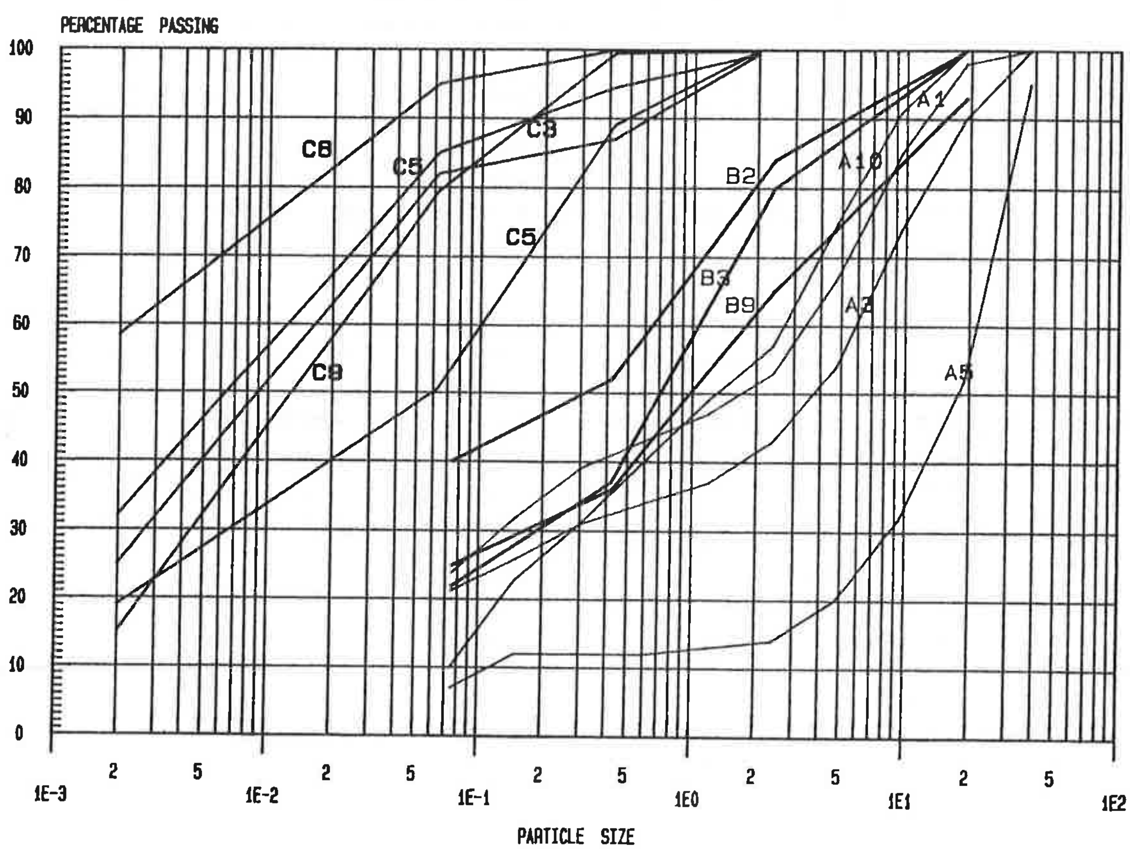


FIGURE 3 Grain size distribution.

are those exceeding 3,000 kg or 1,400 lb laden weight):

Traffic	*Vehicles per Day*	*Thickness (mm)*
Light	0–45	50
Medium	45–450	75
Heavy	450–4500	100

ENGINEERING PROPERTIES

For road construction purposes, the soil properties of primary interest are strength, consistency, gradation, permeability, compressibility, and swelling, from which secondary properties can be derived, e.g., bearing capacity and consolidation characteristics. Some of these properties will be considered for three soil types taken from three locations in Nigeria.

The three locations and the associated soil types (A, B, and C) reflect the three major geologic formations of Nigeria. Group A soils, from Kano State in northern Nigeria, are the weathering products of the basement complex and are granitic in origin. Group B soils, from Cross River State along the Ikom-Obudw road, are sedimentary in origin and are the products of weathering of limestone, shale, and sandstone rock, the decomposition products of which are finer than those of granite and the basement rock complex. Group C soils, from Borno state, lacustrine (quarternary lacustrine sediments consisting mostly of shales, clays, and sandy sediments) products of the Lake Chad formation. The soils from groups A and B are sometimes referred to as laterites and/or lateritic soils, while the group C soils are the local black cotton soils of northern Nigeria.

The data on soils A and B were obtained from the laboratory test results of borrow pits used in road construction in the two parts of the country. Some of these data have been considered by Aggarual and Jafri (*23*) and Sadiku (*24*). Some of the data on soils from group C were taken from published data by Madedor and Lal (*10*) in their work on the classification of Nigerian black cotton soils.

GRADING

The grading curves of representative residual soils from the three different parts of the country are shown in Figure 4. The median diameters (Md) range from 1 mm to 20 mm for the group A soils and from 0.3 mm to 1 mm for the group B soils, and the Md is less than 6 microns for the black cotton soils from Borno State (group C soils). It is not surprising that the black cotton soils are appreciably finer than the other soils. These are fine-grained and high-activity soils of the Lake Chad formation. The difference between the median sizes of the groups A and B soils can

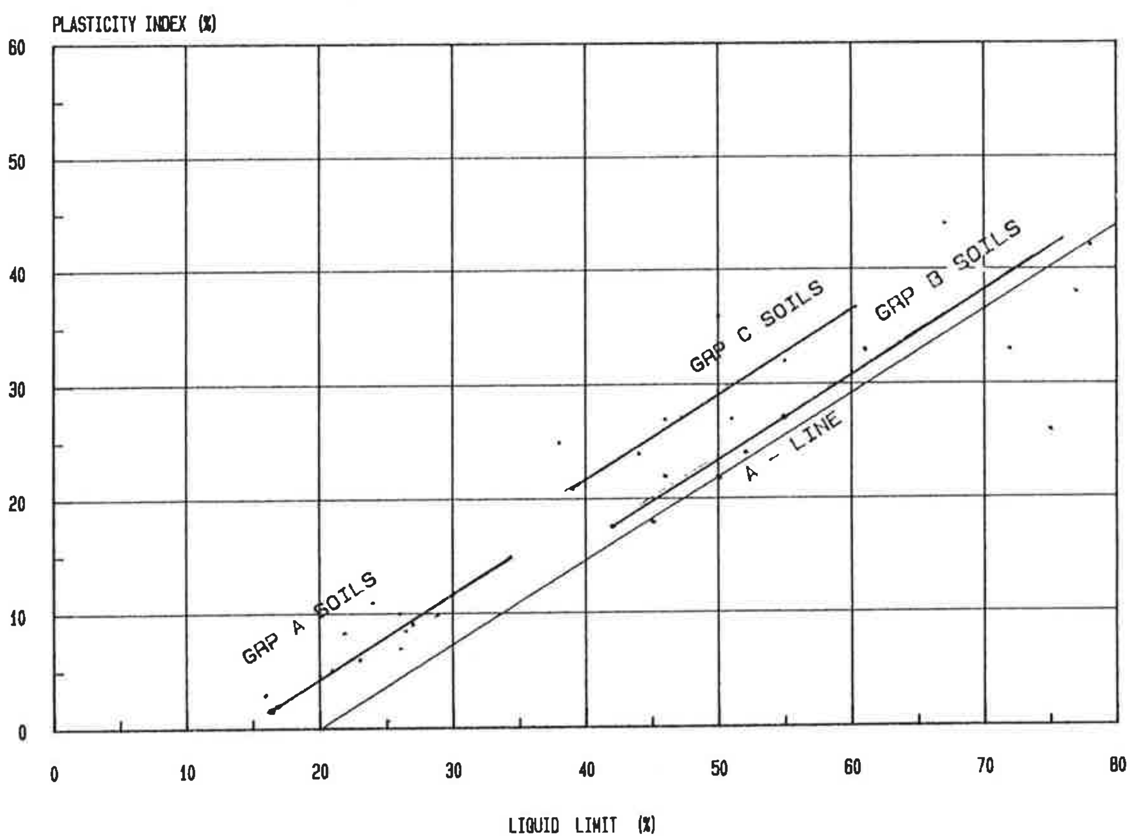


FIGURE 4 Casagrande plasticity chart.

be attributed to the parent materials. The group B soils were found to be predominantly of the A-7 group with a few in the A-2–7 group using the Public Roads Classification system, while group A soils fall under A-2. Although the texture of these three groups of soils is influenced to some extent by the degree of weathering, the petrographic characteristics of parent rocks seem to account more for the textual differences in the soils.

PLASTICITY AND ACTIVITY

The results of investigations into the plasticity characteristics of these soils are shown in Figure 5. The index properties of the soils were obtained on air-dried samples. The tests were performed in accordance with BS 1377, 1975 *(25)*. Also free swell tests were performed on group C soils in accordance with the USBR classification method. All the soils in groups A and C fall above the A-line on the Casagrande Plasticity Chart. They are all inorganic soils of low-to-high plasticity. The group A soils are generally of low plasticity. Most of the soils in group B are above the A-line, although a substantial number fall below the A-line, possibly due to the presence of mica in the soils *(26)*. The Atterberg limits of group B soils are often relatively high, with liquid limits ranging from 45 to 77 and plastic limits from 18 to 38. The percentage passing BS sieve No. 200 is relatively high for the soils of this group, but their relatively low swell in the CBR mold indicates that the clay fraction is apparently not active.

The black cotton soils are very plastic and are expected to present problems if used in road construction. Madedor and Lal *(10)* indicate that residual soils of the black cotton type show a fair correlation between weathering, as indicated by grain size, and other properties of the soil (i.e., free swell and plasticity index). This finding is in line with the findings earlier by Lumb *(20)* for Hong Kong residual soils. On the basis of the analyses of several tests, Madedor and Lal *(10)* derived the following empirical relations for free swell of the soils studied in terms of their index properties:

- Relation between swelling potential and liquid limit (*LL*)

$$FS = 1.93LL - 17.12 \tag{1}$$

- Relation between swelling potential and plasticity index (*PI*)

$$FS = 2.38PI + 13.44 \tag{2}$$

- Relation between swelling potential and clay fraction (*CF*)

$$FS = 1.92CF + 30.43 \tag{3}$$

It should be mentioned that the above equations are based on regression analysis and that there is a degree of subjectivity involved in drawing an "average" line for a mass of scattered data points which form the basis of these

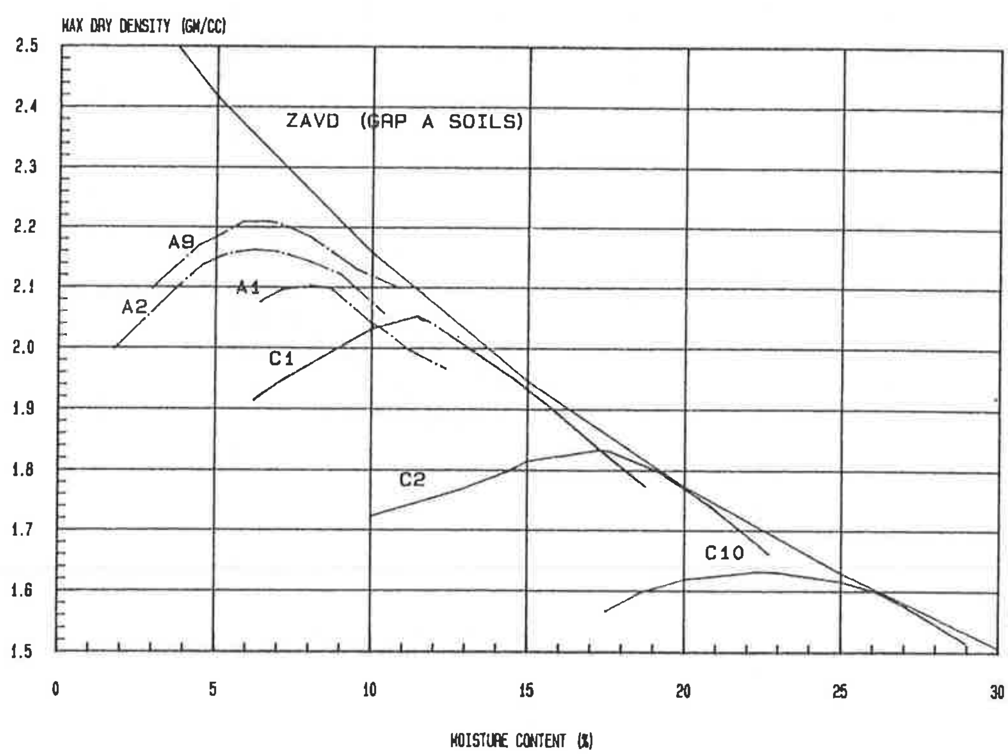


FIGURE 5 Moisture content versus density curves, Group A and C soils.

equations. However, of more importance is the summary of their conclusions:

- Low swell (i.e., $\leq 40\%$) when $LL < 30$; $PI < 15$ and $CF < 20\%$
- Medium swell occurs if $30 < LL < 50$; $CF > 20\%$
- High swell will take place for $LL > 50$, $CF > 20\%$, expected swell $> 80\%$

In other words, free swell of the soils under consideration (i.e., group C) increases with liquid limit, plasticity index, or clay fraction.

STRENGTH CHARACTERISTICS

At present, very little data exist on the strength characteristics of Nigerian soils, although a number of investigations are being conducted. However, Ola (*16*) has presented some results on the residual strengths of some compacted lateritic soils, showing little or no difference in peak and residual angles of shearing resistance, which range from 30 to 39 degrees for the Nigerian residual soils and which are about 10 degrees for the black cotton soils from Maiduguri. Like all the other geotechnical properties, the strength of the residual soils is a function of the degree of laterization/weathering as well as a function of the parent material. Rao and Ouoleye (*17*) have further shown that the strength of these soils is affected by the type of treatment (Table 1).

Many studies have been made on the compaction characteristics of Nigerian residual soils as a result of the specification requirements for base and subgrade materials in road construction in the country. For the soils under study, the compaction and CBR tests were conducted in line with the general specifications (Roads and Bridges), Federal Republic of Nigeria Highways (*27*), which prescribe West African standard compaction or modified AASHO compaction for the design of highway pavements in Nigeria. The West African standard compaction employs the standard CBR mold, 25 blows per layer for 5 layers using a 4.5 kg rammer falling 45.7 cm. This standard was developed in the early sixties when it was found that the standard proctor specification was not adequate for highway construction in West Africa. Because of the heavy rains and consequent flooding of the roads during the rainy season, design CBR values are usually those of soaked samples. Soaking of compacted samples from 24 to 98 hr is usually specified depending on the part of the country. The compaction characteristics of the soils and the CBR test results are contained in Table 2. The compaction curves for the group A and group C soils are shown in Figure 6, while Figure 7 shows the variation of CBR values with moisture content for the group A soils.

The maximum dry densities for the group A soils are all above 2.0 gm/cc except for A-4. The soils compact well and have relatively low optimum moisture contents. The results of linear shrinkage tests on five of the samples varied from 1.5 to 6 percent, and the optimum moisture contents were within the range of 6 to 12.5 percent. It is noteworthy that the natural moisture contents are 2–3 percent below the optimum moisture contents for the soils in this group. The CBR values (after a 48-hr soaking period) are relatively high (55–94). From the CBR curves (Figure 7), it can be seen that as the moisture content decreases below the optimum, the CBR value increases sharply; some increases are more than 100 percent when the moisture content is only 1–2 percent below the optimum moisture content. This observation points to the possibility of more economical pavement design on the basis of compacting the soils at moisture contents below the optimum if the pavement is well drained, especially in northern Nigeria with an average annual rainfall of about 750 m or less.

The good-to-excellent condition of the road pavements, based on visual inspection and surface deformation measurements after more than 5 yr of use (Table 2), is an additional indication that group A materials are good for road construction even when the CBR values at the optimum moisture content are less than 80 percent, the minimum specification of the Federal Ministry of Works. The inspection consisted of observing each section of the pavement for cracking, rutting, pot holes, and ridability. The surface deformation of the sections of the roads inspected met the Federal Ministry of Works General Specification (*27*), clause 6314, on "Final Surface of Carriage," which states that the accuracy of finish in the transverse profile shall not vary at any point by more than 0.63 mm. It should be mentioned, however, that the light-to-medium traffic in the zone A area must have affected the good ratings the roads (Table 2) received.

The maximum dry densities of the group B soils are low, varying from 1.33 to 2.0 gm/cc, while the CBR values ranged between 7 and 19 percent. The swell after a 48-hr soaking is generally low. The optimum moisture contents are relatively high, up to a maximum of 42.6 percent for soil B-5. However, unlike the natural moisture contents for group A soils, the natural moisture contents for the group B soils range from 8.1 to 58.2 percent and are generally higher than the corresponding optimum moisture contents. For this group of soils, the natural moisture would appear to be more critical than the optimum, justifying the present practice of basing design on soaked CBR values.

A plot of the dry density when compacted at optimum moisture content versus natural moisture content is shown in Figure 8. There seems to be a fairly good correlation between the dry density at optimum moisture content and at the natural moisture content. An approximate value of the dry density at optimum moisture content can be predicted from the natural moisture content, using Figure 8 for the group B soils or soils exhibiting similar compaction characteristics.

The dry density at optimum moisture content of the group C soils ranged from 1.53 gm/cc to 2.07 gm/cc. The soaked CBR values (4-day soaking) are very low (2–5 percent). This confirms the expectation that the soils in

TABLE 2 CONSISTENCY LIMIT AND COMPACTION TEST RESULTS

			Gradation % Passing		Consistency Limits					Compaction						
NO.	Type	Group Classification	75µ	2µ	LL	PL	PI	GI	LS	OMC	MDD	CBR at OMC	MAX Swell	Free Swell	Organic Content	Condition of Pavement
1	A1	A-2-6	24		24	13	11	0	6	8.6	2.10	94				Good
2	A2	A-2-4	21.3		26.5	18	8.5	0	4.4	6.6	2.17	64				Good
3	A3	A-2-4	38		21.9	13.5	8.4	0	3.5	8.3	2.08	61				Good
4	A4	A-2-4	35		26	19	7	0	5.1	12.1	1.99	55				Good
5	A5	A-2-4	7		16	13	3	0		7.1	2.15	75				Good
6	A6	A-2-4	11		27	18	9	0	3.5	8.2	2.18	60				Good
7	A7	A-2-4	19		28.8	18.9	9.9	0		8.8	2.19	88				Good
8	A8	A-2-4	14		20	15.3	4.7	0		6.2	2.20	104				Exc.
9	A9	A-2-4	24		22.5	16.4	6.1	0		7.8	2.21	87				Exc.
10	A10	A-2-4	10		20.9	15.7	5.2	0		8.0	2.17	60				Exc.
																NMC (B)
11	B1	A-7-5	95		77	39	38	20		35.5	1.33	11	1.0		0.33	40
12	B2	A-7-6	40		61	28	33	7		14.9	1.74	9	3.8		0.00	18.5
13	B3	A-2-7	22		46	24	22	1		10.7	2.00	18	0.9		0.83	8.1
14	B4	A-7-5	98		72	39	33	20		34.3	1.32	10	2.5		0.61	42
15	B5	A-7-5	98		75	49	26	19		42.6	1.23	7	0.9		0.81	37.2
16	B6	A-7-5	94		72	39	33	20		37.6	1.29	13	1.0		0.78	40.4
17	B7	A-7-6	43		50	28	22	5		16.7	1.78	11	0.00		0.00	15.3
18	B8	A-7-6	37		45	27	18	2		13.4	1.85	12	2.8		0.00	16.2
19	B9	A-2-7	25		52	28	24	1		10.6	1.91	19	2.4		0.00	11.4
20	B10	A-2-7	28		55	28	27	2		11.7	1.88	15	2.2		0.85	11.7
																Activity Ratio (C)
21	C1	A-7-6			46	19	27	16		17	1.82	3.0	3.2	80		1.0
22	C2	A-7-6			67	23	44	20		24	1.63	2.0	8.3	110		1.54
23	C3	A-2-4		19	23	17	6	3		7	2.07			30		0.32
24	C4	A-7-6		20	44	20	24	14		17	1.76			70		1.20
25	C5	A-7-6		32	51	24	27	17		18	1.82			80		0.84
26	C6	A-7-5		58.5	78	36	42	20		25	1.53			140		0.72
27	C7	A-7-6		25	55	23	32	19		25	1.59			100		1.28
28	C8	A-7-6		26	50	14	36	18		17	1.58			90		1.38
29	C9	A-7-6		15	38	13	25	16		15	1.87			40		1.67
30	C10	A-2-4		20	26	16	10	0		11	2.65	5	2.5	30		0.50

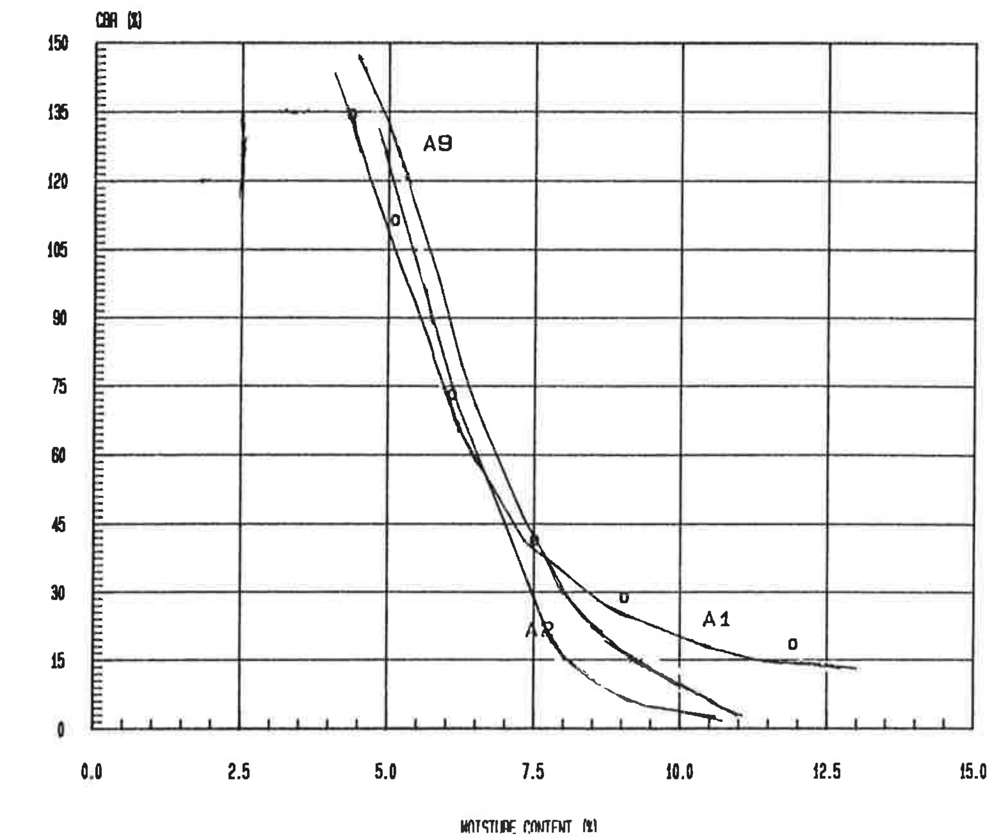


FIGURE 6 Effect of moisture content on CBR, Group A soils.

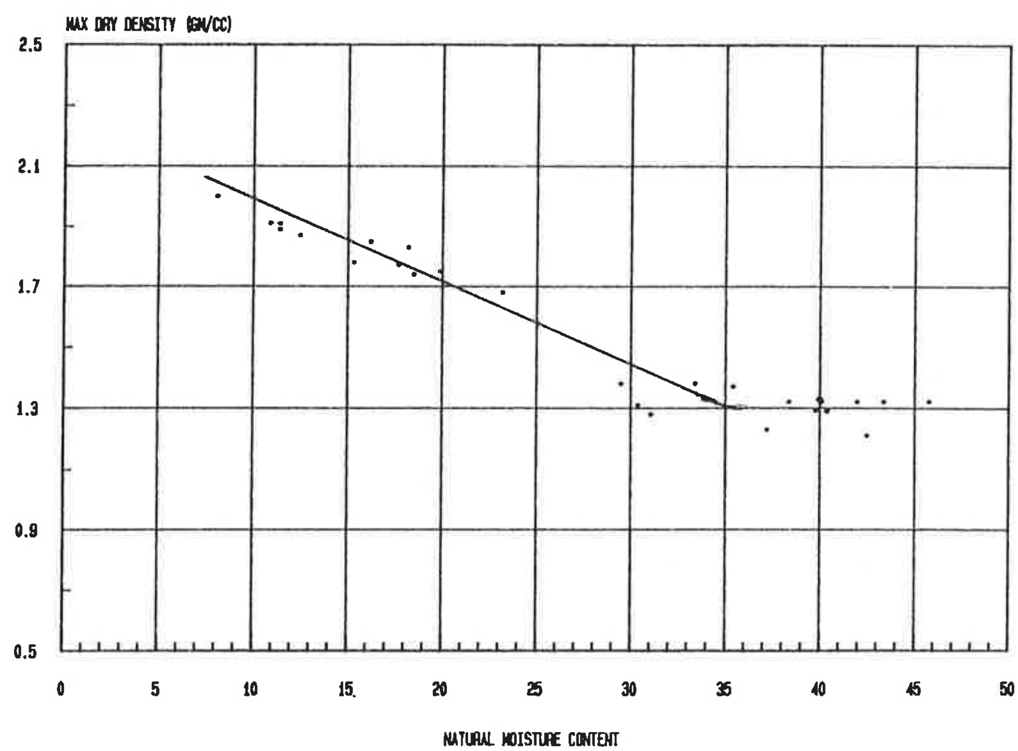


FIGURE 7 Density versus natural moisture content, Group B soils.

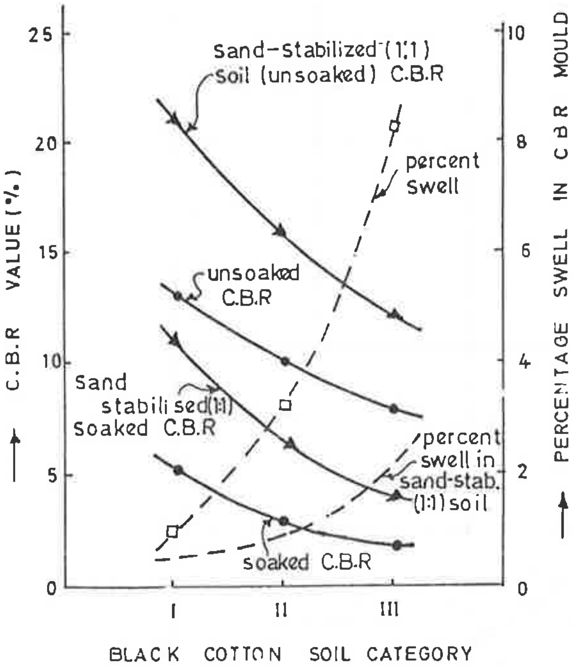


FIGURE 8 CBR values versus percent swell, Category C soils.

this group are not suitable for road construction. The conclusion can also be inferred from the study of the swelling characteristics of the soils. The soils are very expansive, with free swell varying from 30 to 140 percent, and swelling during soaking in the CBR mold was as much as 8.3 percent (sample C-10). The variation of CBR and percent swell for three categories of the black cotton soils are shown in Figure 9 (*10*).

USES

Residual soils are very important and useful as construction materials for a variety of engineering projects. They form the foundation for most engineering structures including buildings, airfields, and earth and rockfill dams and are often used as base and subgrade material for roads. The most notable drawback of these soils when used for unpaved rural roads is their tendency to corrugate in fine weather and to erode in the rain. This tendency can be overcome by providing a satisfactory watertight topping, which is usually achieved through bituminous stabilization of the soils.

Stabilization for the coarser materials (group A soils) is best achieved by the addition of cement. When the proportion of fines is high (≥ ?), plasticity may be reduced by the addition of lime. For example, Ola (*28*) found that a 2 percent addition of lime substantially reduced the plasticity index of laterite, but further addition caused no change.

For group B soils, stabilization with cement, lime, and lime pozzalan or bitumen may not be practical, as experience has shown that soils with a liquid limit greater than 45 percent and a plasticity index over 20 percent cannot normally be economically utilized in soil-cement or soil-lime construction. For bitumen construction, the above limits are 30 and 18 percent respectively. The foregoing is true also of the group C soils. However, laboratory tests show that stabilizing the group C soils with local sand can have beneficial effects. As reported by Madedor and Lal (*10*), there were appreciable increases in the CBR values for the soils (C-1, C-2, C-10) when stabilized with sand in the ratio of 1:1, and there was a corresponding decrease in the swelling potential of the sand-stabilized soils (Figure 9). For the three groups, the soaked CBR values were, as expected, less than the unsoaked CBR values. It is conjectured that the group B soils might benefit from blending with local materials; however, no definitive experimental data exist to prove this possibility.

Another use of residual soils is as slopes for roads, railways, and other embankments. But roads and other cuttings in residual tropical soils can present some problems if they are not well protected. Embankments are usually made as steep as possible to minimize the problem of erosion on flat slopes. In some cases, slopes are as high as 60 degrees, indicating a high angle of shearing resistance for these soils. Gidigasu (*12*) reported studies on Ghana soils that suggest that slope angles of 60–80 degrees may be stable for cuts up to 15 ft high in fairly indurated gravelly laterite soils; for cuts between 15 and 25 ft, the stable angles drop to 50 and 60 degrees. Slopes in loam and clay laterite soils range between 50–60 degrees for cuts less than 15 ft, and 35–50 degrees for higher cuts. However, it is necessary to protect the exposed slope surfaces from erosion. Ways to protect the slopes include spraying them with bitumen, covering them with cement plaster, or grassing them to reduce erosion and penetration of water into the soil. Berms are often used in high embankments to limit the risk of deep-seated slope failure.

Slopes in coarse-grained soils are usually 50–60 degrees; and in fine-grained soils, slopes are limited to 40–50 degrees.

CONCLUSION

The group A soils are generally good base and subgrade materials for road construction. However, under the influence of wetting, the structure of residual soils often changes, leading to loss of strength. Loss of strength can lead to the failure of road base, subgrade, and road embankments made of these soils, particularly during the rainy season. The strength can be improved and preserved by both mechanical and chemical stabilization.

Group B soils have expansive tendencies and high plasticity indices. Their high plasticity and low CBR values do not recommend their use as road materials. The black cotton soils are very compressible soils and are not considered suitable for road construction because of their low strength and swelling characteristics.

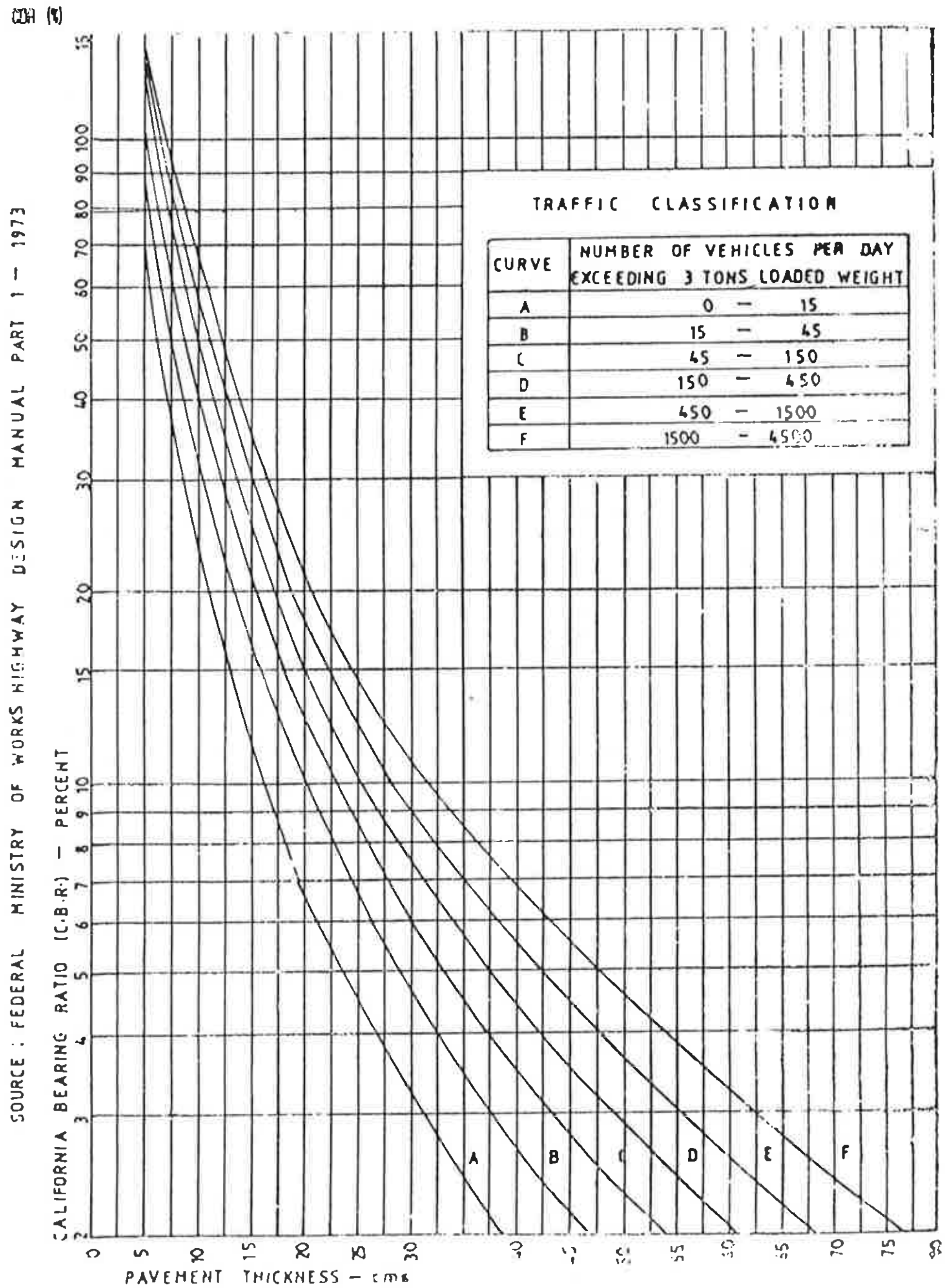


FIGURE 9 Flexible pavement design curves.

REFERENCES

1. B. Durotoye. Geomorphology and Quarternary Deposits of Nigeria. In *Tropical Soils of Nigeria in Engineering Practice* (S. A. Ola, ed.), Balkema/Rotterdam, 1983.
2. A.L. Little. Laterites. *Proc., Third Asian Regional Conference, SM&FE,* Haifa, Vol. 11, 1967, pp. 61–71.
3. A.L. Little. The Engineering Classification of Residual Tropical Soils. Specialty Session on Engineering Properties of Lateritic Soils. *Proc., 7th International Conference of SM&FE,* Mexico City, Vol. 1, 1969, pp. 1–10.
4. L.L. Fermor. What is Laterite? *Geological Magazine,* Vol. 5, No. 8, 1911.
5. J.W.S. De Graft-Johnson, H.S. Bhatia, and M.D. Gidigasu. The Engineering Characteristics of Lateritic Residual Clay of Ghana for Earth Dam Construction. *Proc., Symposium on Rock and Earth Fill Dams,* New Delhi, Vol. 1, 1968, pp. 94–107.
6. M.D. Gidigasu. Parameters for Classification of Fine-Grained Laterite Soils of Ghana. *Proc., 50th Meeting of the Committee on Exploration and Classification of Earth Materials,* ISSMFE, 1971, pp. 57–79.
7. L.W. Akroyd. Formation and Properties of Concretionary and Non-Concretionary Soils in Western Nigeria. *Proc., Fourth Regional Conference for Africa of SM&FE,* Cape Town, 1967, pp. 47–52.

8. J.O. Jackson. Geotechnical Properties of Biu Black Cotton Soil. *Proc., International Conference of SM&FE*, Vol. 1, Stockholm, 1981.
9. L.A. Ajayi. The Laterites and Lateritic Soils of Nigeria. *Geotechnical Practice in Nigeria: ISSMFE Golden Jubilee Commemorative Publication*, August 1985, pp. 7–15.
10. A.O. Madedor and N.B. Lal. Engineering Classification of Nigerian Black Cotton Soils for Pavement Design and Construction. *Geotechnical Practice in Nigeria: ISSMFE Golden Jubilee Commemorative Publication*, August 1985, pp. 49–67.
11. K.E. Clare. *Road Making Gravels and Soils of Central Africa*. Overseas Bulletin 12. British Road Research Laboratory, 1966, 43 pp.
12. M.D. Gidigasu. *Development in Geotechnical Engineering: Laterite Engineering*. Elsevier Scientific Publishing Company, Amsterdam, 1976.
13. H. Pern and K.S. Baua. Expansive Soils. *Proc., Conference on Materials Testing, Control and Research on Roads and Formation Problems*, FMW, Lagos, 1984.
14. R.D. Denman. Some Notes on Expansive Soils with Particular Reference to Northern Nigeria. *Proc., Conference on Materials Testing and Control*, FMW, Kaduua, 1966.
15. A. Ibrahim. Road Design and Construction. Research on Expansive Black Cotton Clay Areas of Maiduguri-Gamborw-Walgo Road in Borno State of Nigeria. *Proc., Conference on Materials and Testing*, Federal Ministry of Works, Kano, 1979.
16. S.A. Ola. Peak and Residual Strengths of Compacted Lateritic Soils. *Geotechnical Practice in Nigeria: ISSMFE Golden Jubilee Commemorative Publication*, August 1985, pp. 79–92.
17. R.R. Rao and O.A. Ouoleye. A Study of the Variation of Consistency Limits in Relation to Strength Properties. *Geotechnical Practice in Nigeria: ISSMFE Golden Jubilee Commemorative Publication*, August 1985, pp. 65–77.
18. K. Terzaghi. Design and Performance of the Sasumua Dam. *Journal, Institution of Civil Engineers*, London, Vol. 9, 1958, pp. 369–394.
19. H.F. Winterkorn and E.C. Chandrasekharan. Laterite Soils and Their Stabilization. *Highway Research Board Bulletin No. 44*, 1951, pp. 10–28.
20. P. Lumb. The Residual Soils of Hong Kong. *Géotechnique*, Vol. 15, No. 2, 1965, p. 180.
21. *Highway Manual*, Part 1. Nigeria, Federal Ministry of Works and Housing, 1973.
22. Transport and Road Research Laboratory. *A Guide to the Structural Design of Bitumen-Surfaced Roads in Tropical and Sub-tropical Countries*. Road Note 31, Department of Transport, United Kingdom, 1977.
23. H.R. Aggarual and R.H. Jafri. A Preliminary Study to Modify Material Specifications for Base Laterites for Secondary Roads. In *Geotechnical Practice in Nigeria: ISSMFE Golden Jubilee Commemorative Publication*, August 1985, pp. 109–131.
24. M.J.A. Sadiku. The Use of Sub-standard Soil Materials in Road Construction. *Geotechnical Practice in Nigeria: ISSMFE Golden Jubilee Commemorative Publication*, August 1985, pp. 133–137.
25. British Standard Institution. *Methods of Testing Soils for Civil Engineering Purposes*. British Standard 1377, London, 1975.
26. A. Casagrande. Classification and Identification of Soils. *Proc., ASCE*, Vol. 113, pp. 783–816, 1948.
27. *General Specifications for Roads and Bridges*, Vol. 11. Nigeria, Federal Ministry of Works, 1970.
28. S.A. Ola. *Soil Stabilized Compressed Blocks*. Geotechnical Report No. 1, University of Benin, 1983.

Publication of this paper sponsored by Committee on Soil and Rock Properties.