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Review of Specifications for the Use of Laterite in Road Pavements (Contract: AFCAP/GEN/124)

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ASANRA
ASSOCIATION OF SOUTHERN AFRICAN NATIONAL ROAD AGENCIES

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Project Management

The project was managed by the Crown Agents and carried out under the general guidance of the AFCAP Technical Manager, Mr. R. Geddes.

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Peer Review

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References

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LIST OF ABBREVIATIONS

AADT	Average Annual Daily Traffic
AASHTO	American Association of State Highway and Transport Officials
ACV	Aggregate Crushing Value
AFCAP	African community Access Programme
AIV	Aggregate Impact Value
ARD	Apparent Relative Density
BLS	Bar Linear Shrinkage
BRD	Bulk Relative Density
BS	British Standard
CBR	California Bearing Ratio
CIRIA	Construction Industry Research and Information Association
CSIR	Council for Scientific and Industrial Research
DCP	Dynamic Cone Penetrometer
DN	(DCP) Resistance to Penetration (mm/blow)
DNER	Departamento Nacional de Estradas de Rodagem
ESA	Equivalent Standard Axle
E80	Equivalent 80 kN Standard Axle
FACT	Fines Aggregate Crushing Test
GM	Grading Modulus
kg	Kilogram
kN	Kilo Newton
LL	Liquid Limit
LNEC	Laboratoria Nacional de Engenharia Civil
LS	Linear Shrinkage
LVR	Low Volume Road
LVSR	Low Volume Sealed Road
OMC	Optimum Moisture Content
MCT	Miniature Compaction Test
MDD	Maximum Dry Density
Mr	Resilient Modulus
NITRR	National Institute for Transport and Road Research
PI	Plasticity index
PL	Plastic Limit
PM	Plastic Modulus
PP	Plasticity Product
PSD	Particle Size Distribution
P425	Percentage material passing the 0.425 mm Sieve
P075	Percentage material passing the 0.075 mm Sieve
S/R	Silica-Sesquioxide Ratio
TMH	Technical Methods for Highways
vpd	Vehicles Per Day
WAACT	Western Australian Confined Compression Test
XRD	X-Ray Diffraction
XRF	X-Ray Fluorescence

1. INTRODUCTION

1.1 Background

In many countries in Africa there is a growing realization of the cost-effectiveness of upgrading gravel roads to a sealed standard even at relatively low traffic levels, often less than 200 vehicles per day. This has challenged road authorities to make optimum use of naturally occurring materials which are often rejected by traditional specifications for use in the upper layers of road pavements. One such naturally occurring material is ***laterite*** – a type of residual soil that occurs extensively in the humid tropical and sub-tropical zones of the world, including much of central, southern and western Africa. Fortunately, research carried out in the late 1960s in a number of countries, notably in Angola, Mozambique, Brazil, Australia and Nigeria indicates that the performance of laterite has often been better than expected on the basis of traditional specifications. However, if successful use is to be made of this material, the conditions under which it can be successfully used must be carefully specified – one of the key objectives of this report.

It is noteworthy at the outset that terms such as "laterite", "lateritic soils" and "ferricretes" are often used synonymously. However, such terms convey different meanings to different practitioners in that their application ranges from strict conformity to Buchanan's original definition (Buchanan, 1807) which confines the material to a fairly small group of red soils that *harden irreversibly on exposure* (described by Buchanan as "red clay used for air dried brick production"), to any variety of reddish, iron-rich, tropical residual soils. As a result, the confidence with which "laterite" can be used for road construction is diminished largely because the term may apply to a material with a range of geotechnical properties. Nonetheless, certain types of this material are eminently suitable for use in the construction of road pavements in Africa for which a commonly agreed definition is desirable so as to enable their engineering behaviour to be predicted.

Unfortunately, laterites have not been used to their fullest extent in the upper (base and subbase) layers of low volume paved roads (LVSRs) in the African region for a number of reasons including:

- The variability in their engineering properties and their failure to meet traditional specifications. For example, these materials commonly exhibit gaps in the grading curve (e.g. in the sand coarse fraction); high plasticity indices (PIs 15-20) and soaked CBR values lower than the minimum of 80 per cent normally specified.

- Lack of awareness of the more appropriate specifications that were first developed by the Portuguese in the 1950s and 1960s in countries such as Angola and Mozambique and subsequently adapted for use in other countries, notably Brazil and Australia

In view of the above, the use of neat (untreated) laterites for the construction of low volume sealed roads (LVSRs) in some African countries has been limited as the road authorities continue to use much tighter, restrictive standards that greatly suppress the use of this type of material. As a result, other more expensive options are adopted such as hauling over long distances other natural gravels which meet the traditional specifications; stabilizing the laterites with cement and lime or using crushed stone for the base.

Fortunately, there are some relatively recent examples of the use of laterites in a number of Southern African countries, such as Angola, Botswana, Kenya, Malawi, Mozambique, Zambia and Zimbabwe, where this type of material has been successfully used in the upper layers of both low and high-volume roads, despite its non-compliance with traditional specifications. Typical examples of some LVSRs are shown below (Source: Authors)

Nchisi road, Lilongwe, Malawi. Constructed in 2002.
Base properties:
PI = 16; P425 = 46; PM = 736; GM = 1.27
CBR_S @ 98% BS Heavy = 37%
Cum. Traffic to end 2012 = 0.026 MESA

Kwale road, Mombasa, Kenya: Constructed in 1986.
Base properties
PI = 15, P425 = 48; PM = 720
CBR_S @ 98% BS Heavy = 24%
Cum. Traffic to end 2003 = 0.28 MESA

Moreover, the high volume main road between Lilongwe and the Zambian border at Mchinji was constructed in about 1977 with a laterite base according to the 1974 Brazilian national specifications and has provided excellent service to date, as illustrated below (Source: Authors)

Lilongwe-Mchinji road, Malawi as at 3rd February, 2014

Despite the excellent performance of these roads, and of similar examples in other countries, the likelihood of adopting these designs in practice is limited, largely because the national standards of many countries in the region do not contain appropriate specifications for the use of laterites in road construction.

1.2 Purpose and Scope of Report

Against the above background, the main purpose of this report is to:

- Raise awareness of the existence of the performance-based specifications that have been developed specifically for the use of laterites in road construction in such countries as Angola, Brazil and Australia.

- Highlight the fact that such specifications are quite different to the more traditional ones that are still used in a number of countries in Africa, as a result of which unnecessary recourse is often made to the relatively expensive stabilization of these local materials for use in low volume roads (LVRs).

- Review existing specifications for the use of laterites in LVR construction based on the most recent/relevant documentation on the subject.

It is anticipated that the outputs of the project indicated above will enable road authorities to make informed decisions for the revision of their Special Technical Specifications for use of laterites in LVRs thereby leading to potentially significant cost savings in the construction of such roads.

Attainment of the objectives of the project requires, in part, that an extensive, but selective, review be undertaken of the vast body of literature available on the subject, the outcome of which is captured in this report.

The scope of the report includes the following:

- A review of all relevant research on the use of laterites in the African region and internationally.
- A review of existing specifications for the use of laterite in road base and sub-base in the region (southern, east and west Africa) as well as from such countries as Brazil and Australia.

- A review of the performance and properties of a number of laterite pavements constructed in Botswana, Ethiopia, Kenya, Malawi, Mozambique and South Africa.

1.3 Approach to Undertaking Literature Review

The starting point for the review was the preparation of a bibliography on the subject of laterite which was compiled on the basis of an extensive documentation held in the archives of Dr. F. Netterberg, supplemented by a literature survey that was carried out by CSIR. This bibliography of over 1200 titles is included as a separate report.

A review of the bibliography revealed the existence of a number of comprehensive geotechnical studies on laterites including those of Morin and Todor (1971, 1976), summarized by Morin (1982); Krinitzsky et al (1976), summarized by Townsend et al (1982); Gidigasu (1976); Mitchell and Sitar (1982); Netterberg (1985, 1994); the Committee on Tropical Soils (1985) which produced a detailed state-of-the-art review; Charman et al (1988) and Fookes (1997). Studies carried out in the southern African region include those of LNEC et al (1959, 1969, and 1972) for Angola and Mozambique; Van der Merwe and Bate (1971) for Zimbabwe; Mountain (1967) and Weinert (1980) for South Africa and, more recently, an unpublished review by Netterberg (1988). Some of the above include countries not specifically mentioned such as Brazil and Australia.

In addition to the above core references, much additional data is scattered in the literature or unpublished. All of this cannot be reviewed in this project. Thus, in compiling this review, southern African data has been used as far as possible, but recourse has had to be made to African and world data in a number of cases.

In keeping with the main objectives of the project, the review has concentrated more on the practical aspects of the use of laterites in road construction, such as the performance derived specifications recommended from regional research work in relation to prevailing specifications, rather than on the more academic aspects, such as the weathering processes leading to formation of laterites, for which information abounds in the literature.

1.4 Terminology

Because of the wide arange of terms used to discuss iron-rich material, for the purposes of this report, the term lateritic material is used as a generic term to describe materials that variously have been called plinthite, laterite, ferricrete and lateritic soil. These terms are discussed further in Section 2.3.3.

1.5 Structure of Report

This Literature Survey and Desk Study report is structured as follows:

Section 1 (this section): provides the background to the project and its main purpose and scope.

Section 2: Presents the general characteristics of lateritic gravels including the terminology used to describe them, their definition, formation, composition, description, classification and types.

Section 3: Presents the various properties of laterites that influence their selection for use in road construction and the related methods of testing.

Section 4: Reviews the various specifications that have been developed by a number of authorities for the use of laterites in road construction and compares them with the recommended Brazilian specfications. This section also evaluates the performance of a number of road projects in the southern Africa and compares the properties of the laterites used with those given in the Brazilian specifications.

Section 5: Provides examples of the use of laterites in road construction including the required quality control measures and performance-related studies carried out on laterites.

Section 6: Provides the conclusions and recommendations arising from the literature review and desk study.

2. GENERAL CHARACTERISTICS

2.1 Introduction

Despite its widespread occurrence and use in many African countries and, indeed, in many other tropical countries of the world, much confusion still exists concerning the general characteristics of what is often referred to loosely as “laterite”. Moreover, the numerous attempts made at definition and classification, to enable engineering behaviour to be predicted, have been unsatisfactory because the term has been so loosely applied without a detailed soil description related to the formation of the material.

What is better understood is that laterite is the result of a decomposition or weathering process, the consequences of which are of overriding significance in the formation of this material in the various tropical regions of the world. Thus, an appreciation of the tropical weathering process is fundamental to any system of classification or any attempt to identify the significant engineering characteristics of laterites.

In view of the above, this chapter addresses the following topics pertaining to the material “laterite”.

- Formation and development.
- Definition, description and distribution.
- Classification and composition.

2.2 Formation and Development

2.2.1 General

Laterite is the product of a humid tropical weathering process, current or past, which has the following effects:

- The parent material is chemically enriched with iron and aluminium oxides and hydroxides (sesquioxides)
- The clay mineral component is largely kaolinitic
- The silica content is reduced

The above processes usually produce yellow, brown, red or purple materials, with red being the predominant colour. While tropical weathering in oxidizing conditions generally leads to reddening, this does not necessarily produce a lateritic material – hence the widespread confusion concerning laterite and its behaviour.

Laterite formation requires particular conditions which concentrate the iron- and aluminium-rich weathering products sufficiently to allow concretionary development, often progressing to a cemented horizon within the weathering profile. Three phases of action are necessary to produce concretionary laterite:

- Humid tropical weathering to produce the minerals of laterite
- Concentration of these minerals in a discrete zone
- Concretionary development within the horizon.

2.2.2 Tropical weathering

In the humid, tropical regions of the world, chemical decomposition – the chemical alteration of the primary minerals into secondary and residual products - rather than physical disintegration, is the dominant mode of weathering which is especially effective in the presence of water and high temperature.

A schematic representation of a tropical weathering profile is shown in Figure 2-1 which indicates a gradual change from fresh rock to a completely weathered residual soil. The weathering process originates with rock exposure at or near the earth's surface in a physical environment, quite different from that in which it was originally formed. The minerals which constitute the rocks may react chemically with rainwater, ground water, and dissolved solids and gases of the new near surface environment to form new minerals which are more nearly in equilibrium with the surface conditions. The end result of these changes is to convert the upper portion of the rock into a residual debris more soil-like than rock-like in character and with chemical, mineralogical and physical properties entirely different from those of the original rock.

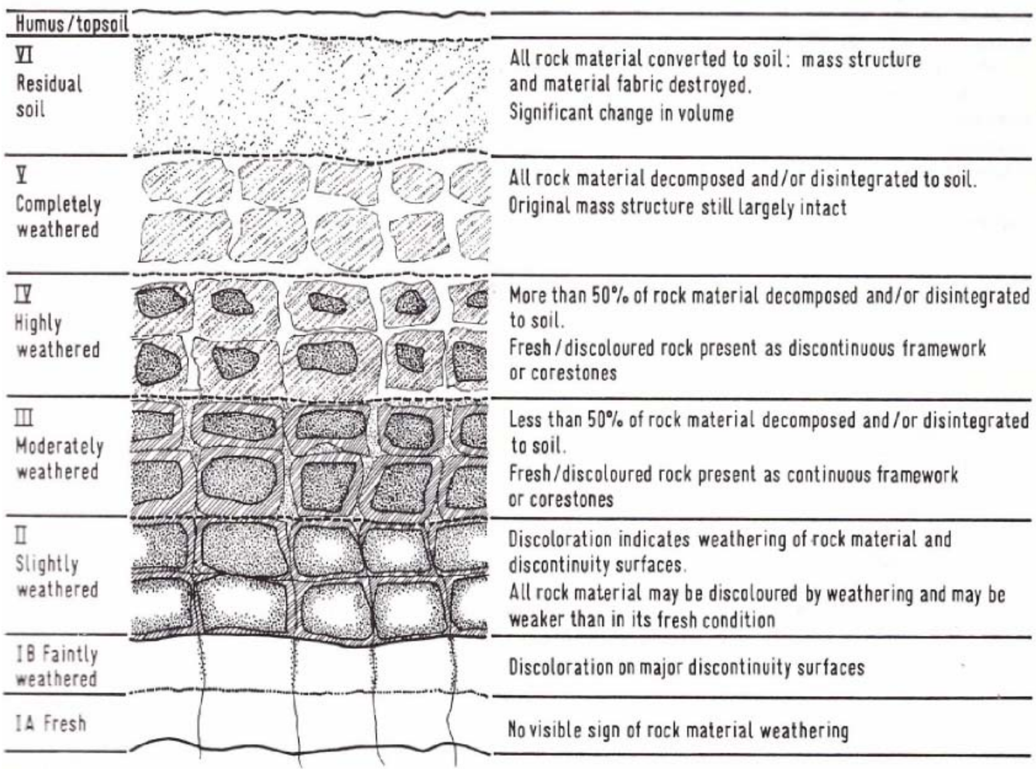

Figure 2-1: Schematic representation of tropical weathering profiles (descriptions based on Geological society Engineering working group Report (in Sharman, 1988)

The commonly used terms and adopted pedological classification to describe the various engineering weathering zones are presented in Figure 2-2. For example, the explanatory term "saprolite" is used to describe that part of the weathered mantle (behaving as a soil in engineering terms) which exhibits textural and structural features of the parent rock to the extent that identification of the parent rock could still be made.

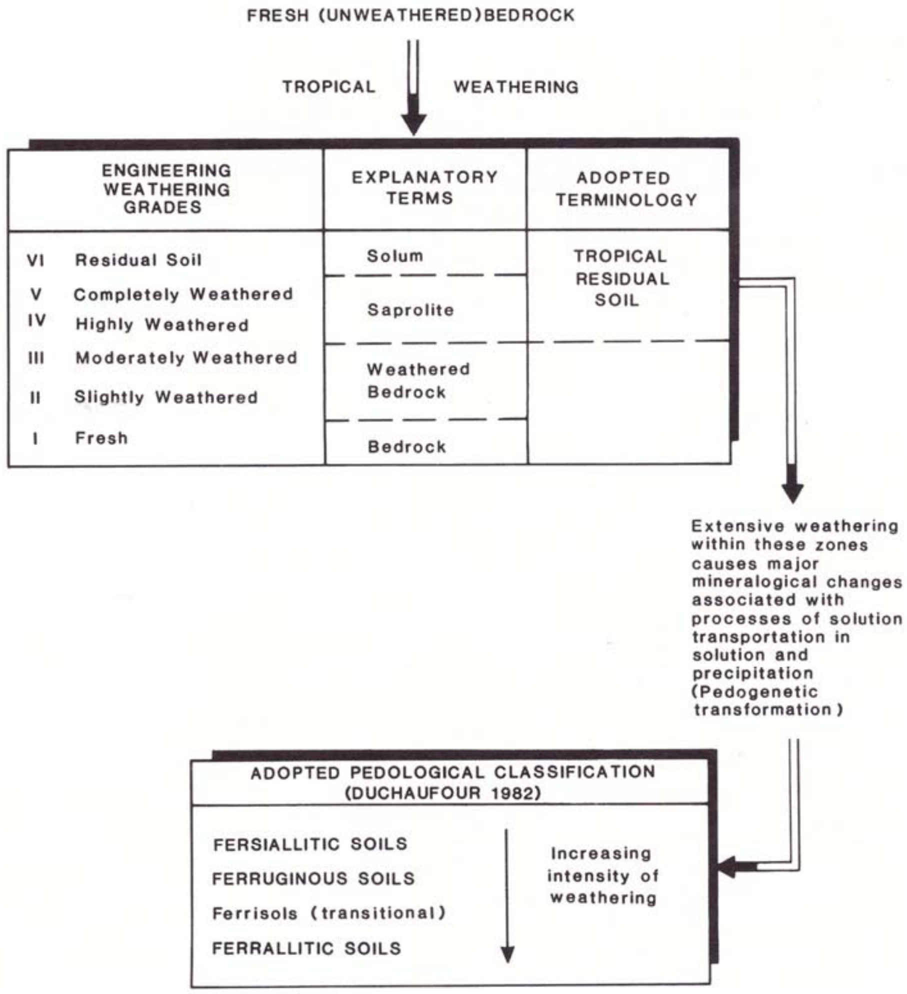

Figure 2-2: Commonly used terms and adopted pedological classification (Duchaufour, 1982 in Fookes, 1997).

As explained in the publication on Tropical Residual Soils (Ed. Fookes, 1997), and illustrated in Figure 2-2, a residual soil profile can develop on any parent material, including fully formed in situ weathering profiles developed in a transported "parent" material such as a river terrace deposit. Thus, it may not be unusual to encounter one weathering profile grading with depth into a transported soil, which in turn rests on a horizon representing an old erosion surface within a second weathering profile.

2.2.3 Concentration of minerals

According to Charman (1988), before the concretionary development of true laterite can take place, an additional process is required – the concentration of the weathering products within the residual soil/completely weathered zones. A summary of the processes involved in the concentration of minerals into discrete horizons by leaching and residual accumulation or solution and precipitation is presented in Table 2-1.

Table 2-1: Summary of processes involved in the formation of laterite (Charman, 1988)

PARENT ROCK	*IGNEOUS AND METAMORPHIC ROCKS* ACID → DECREASING STABILITY → BASIC								
Light minerals	Quartz	Muscovite	Potash feldspar			Sodic plagioclase		Calcic plagioclase	
Dark minerals				Biotite	Amphibole		Pyroxene		Olivine

INITIAL WEATHERING UNDER TROPICAL CONDITIONS

Initial weathering products	No change but some solution	Vermiculite or Montmorillonite	Slow weathering to Kaolinite or Halloysite	Vermiculite or Montmorillonite	Chlorite calcite or Montmorillonite	Kaolinite or Halloysite	Chlorite calcite or Montmorillonite	Kaolinite or Halloysite	Serpentine or Montmorillonite

CONTINUATION OF WEATHERING UNDER WARM OXIDISING CONDITIONS

Further weathering products	No change but additional solution	Kaolinite and Gibbsite	Kaolinite or Halloysite	Kaolinite and Gibbsite	Kaolinite and Goethite limonite or Haematite	Kaolinite or Halloysite	Kaolinite and Goethite limonite or Haematite	Kaolinite or Halloysite	Kaolinite and Goethite limonite or Haematite

CONCENTRATION OF MINERALS INTO DISCRETE HORIZON BY LEACHING AND RESIDUAL ACCUMULATION OR SOLUTION AND PRECIPITATION

LATERITE

CONCRETIONARY GROWTH

2.2.4 Concretionary development

The degree of development of concretions in laterite materials significantly governs their engineering properties and is reflected in the appearance of the material. The harder these concretions are, the stronger is the material. The strength of the concretions depends mainly on the content of iron and aluminium oxide (sesquioxides) present and how much the concretionary particles have been dehydrated. In order for the concretions to develop, they need sufficient concentration of hydrated oxides of iron and aluminium to act as catalysts for cementation or precipitation growth to start.

The concentrated, uncemented, partially self-hardened, horizon of material is referred to as *plinthite* by pedologists – a term that has gained wide acceptance among soil scientists as an unambiguous one for Buchanan's laterite. In the South African system (MacVicar et al, 1977; Soil Classification Working group, 1991), the ability to self-harden is not required and the terms "soft plinthite" and "hard plinthite" are used for mottled or nodular horizons and continuously indurated (e.g. hardpan or honeycomb) horizons which can and cannot be cut with a spade when wet, respectively.

The hardening or concretionary development after the iron enrichment seems to proceed by a number of mechanisms including chemical precipitation, loss of water of crystallization (dehydration) and the development of a continuous fabric of cementing materials (Alexander and Cady, 1962). The physical condition in which the processes develop is thought to be due to the fluctuation of the groundwater level, or simply distinct wet and dry seasons, which causes alternately reducing and oxidizing conditions. Only under the oxidizing conditions, provided by drying or the lowering of the groundwater level, can precipitation and dehydration take place. The current climatic conditions at which concretionary laterites are found are not necessarily those at the time of formation. Conditions of drying or alternate wetting and drying appear to be necessary for the precipitation of the sesquioxides.

In light of the above, a laterite may be qualitatively described as a soil that has been impregnated with, cemented by or partly replaced by, another material. This process is mainly due to the precipitation of fine particles of sesquioxides which, with time, concentrate to the point whereby soft discrete nodules or concretions of soil cemented by the sesquioxides are likely to form. If the process continues the nodules coalesce and a spongy, hard mass full of cavities with a honeycomb structure is formed. The cavities may be filled with remnants of the host soil although the cementing material may have already become quite hard. The filling of these cavities with the precipitate eventually results in rock-like hardpan which, when weathering, breaks down into boulders (Weinert, 1890).

The filling and cementation of the material in the cavities eventually results in a massive, rock-like, hardpan. If this hardpan is subjected to a new cycle of weathering, a secondary deposit of lateritic material will be formed. Erosion and transportation play an important role in this process.

The relationship between concretionary development and climatic conditions in western Nigeria has been postulated on the basis of climatic indices as presented in Table 2-2. (Ackroyd, 1967 in Charman, 1988).

Table 2-2: Possible conditions for development of concretionary laterite

Annual rainfall (mm)	Max. 1000 - 1500	Max. 1500 - 2000
Thornthwaite Moisture Index	Max. 0	Max. 30
Length of dry season (months)	Min. 6	Min. 5
Type of laterite	Hard concretionary gravels	Min. requirements for concretions to develop

Source: Ackroyd, 1967 in Charman, 1988

According to Charman (1988), “a mean annual temperature of around 25°C is needed for laterite formation, and in seasonal situations there should be a coincidence of the warm and wet periods. If there is high rainfall during the cold season, laterites do not develop freely, but the minimum annual rainfall required for laterite formation is generally at least 750 mm. The higher the rainfall above this value, the greater is the leaching effect, which removes free silica, reduces the silica-sesquioxide (S/R) ratio and therefore increases the proportion of gibbsite ($Al(OH)_3$).

Ackroyd (1967) suggested a possible upper limit on rainfall of 1500 – 2000 mm per annum for the formation of concretionary laterites in Nigeria, while Newill and Dowling (1970) found laterites in Malaysia in areas with a current annual rainfall of about 2000 mm. However, under higher rainfall conditions, concretions do not appear to form and the result is rather a highly leached sandy, fersiallitic soil in which the silt and clay are weakly cemented into “pseudosand” or “pseudosilt” particles (Ackroyd, 1967).

In southern Africa, ferricretes in amounts and quality adequate for road construction material are largely limited to areas with a Weinert N-value of less than 5 (equivalent to a mean annual rainfall of about 550 mm in South Africa). Figure 2-3 shows the approximate distribution of the dominant two pedogenic materials, lateritic material and calcrete, in South Africa and where they have actually been used in road construction (Weinert, 1980). The predominance of lateritic materials in the wet areas (east and south of the N=5 contour) and the calcretes in the arid areas to the west of that contour can be clearly seen.

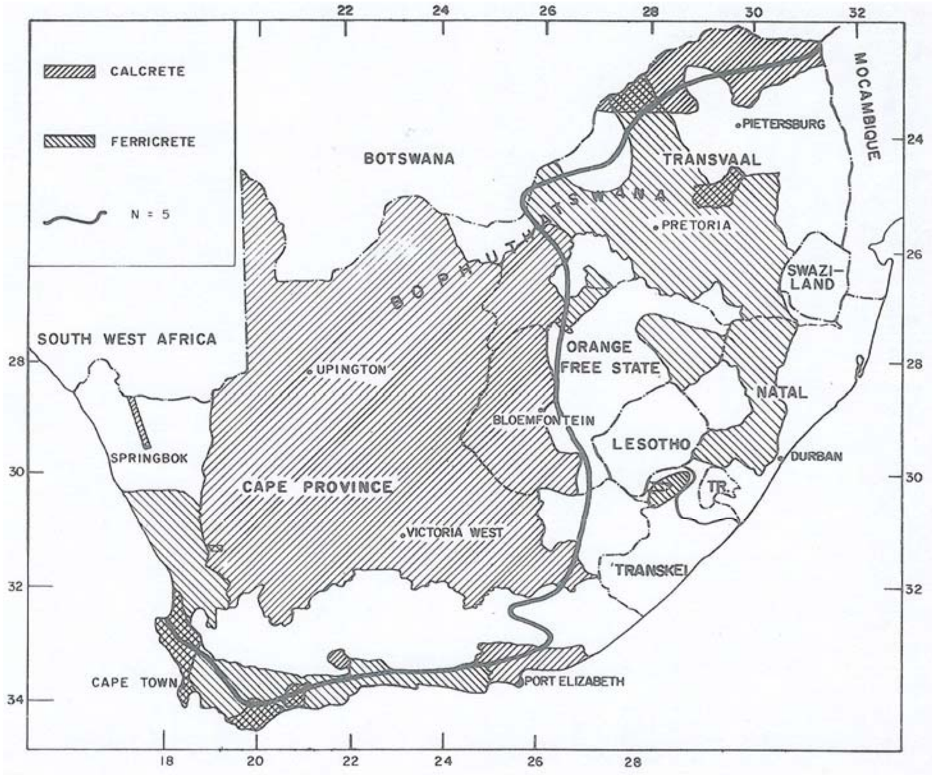

Figure 2-3: Distribution of lateritic materials and calcrete in South Africa (Weinert, 1980).

2.3 Definition, Description and Distribution

2.3.1 General

The term *laterite* first appeared in the scientific literature more than 200 years ago (Maignien, 1966) and seems to have been first used by Dr. Francis Buchanan-Hamilton to denote a building material used in the Malabar district of India. (Buchanan, 1807). Its appearance was described as “that of a ferruginous deposit of vesicular structure, apparently unstratified, and occurring not far below the surface”. Moreover, Buchanan observed that “when fresh, it can readily be cut into regular blocks with a cutting tool. However, on exposure to the air, it rapidly hardens and becomes highly resistant to weathering”. Thus, the unusual feature of the material first described by Buchanan under the name laterite, from the Latin word *later*, which means a brick, was that ***it had a soft consistency in situ but hardened rapidly on exposure*** – a phenomenon which led to the use of this material as a building brick (see photos 2-1 and 2-2 below). Modern pedological terminology would now perhaps describe Buchanan’s laterite as plinthite.

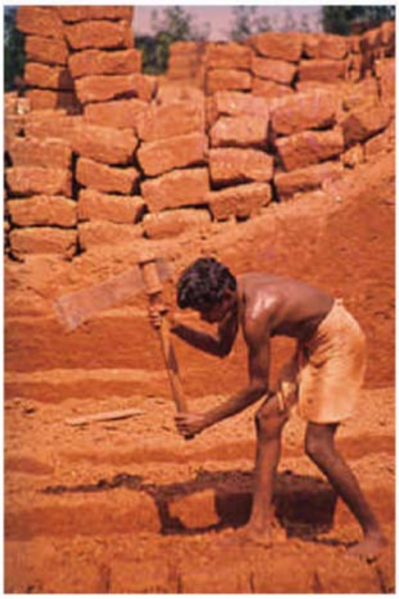

Photo 2-1: Cutting of laterite bricks in quarry.
Source: Werner Schellmann–own work.

Photo 2-2: Use of laterite bricks for house construction
Source: Anon

2.3.2 Review of terms and definitions

Since Buchanan's original use of the term laterite, in the context described above, many other terms have been used to refer to this type of material to include practically all reddish iron-rich soils of any origin, whether indurated or capable of induration or not. Table 2-3 lists commonly used terms to describe laterite (Charman et al, 1988):

Table 2-3: Commonly used terms to describe laterite

Brickstone (India)	Krusteneisensteine (Germany)
Cabook (Sri Lanka)	Laterite (India)
Canga (Brazil)	Mantle rock (Ghana)
Carapace (France)	Moco de hierro (Venezuela)
Cuirasse (France)	Murram (East Africa)
Eisenkrutse (Germany)	Picarra (Brazil)
Ferricrete (Southern Africa)	Pisolite (Australia)
Iron Clay (India)	Plinthite, soft (USA)
Ironstone (Nigeria)	Plinthite, soft or hard, (South Africa)

The tendency to indiscriminately use a wide array of terms to describe laterite, irrespective of the geotechnical characteristics and related engineering behaviour of the material, has caused much confusion amongst practitioners. To make matters worse, there appears to be no universally accepted definition for laterites and many differing, and sometimes, conflicting definitions have been proposed in the literature with each discipline (geology, pedology and engineering) tending to have their own definitions and terminology. In pedology the term "laterite" has long been abandoned and replaced by new, more closely defined terms, of which plinthite (Greek for brick) is one.

Examples of commonly used definitions include the following:

- **Laboratorio Nacional de Engenharia Civil (LNEC) et al, 1959, 1969):** (based on extensive work on laterites for road building in Angola, Mozambique and elsewhere):

  Laterite*: Material with a vesicular structure, very often coloured with hues ranging from yellow to red with more or less dark shades sometimes even black, made up of continuous cuirasses (crusts) with variable thickness and hardness, looking very frequently like a slag or also isolated pisolitic concretions of variable strength.*

  Lateritic soil*: Soil containing a clay fraction with a molecular silica-sesquioxide (SiO_2/R_2O_3) ratio of less then 2 and displaying a low expansibility."*

- **Gidigasu,1976):**
  Laterite soil*: All the reddish residual and non-residual tropically weathered soils which generically form a chain of materials forming from decomposed rock through clays to sesquioxide-rich crusts*.

- **6th Regional Conference for Africa: Soil mechanics and Foundation Engineering: Specialty Session on Pedogenic Materials (1976):**
  Laterite: *Applies to materials containing a minimum of about 50% of the cementing material (iron and aluminium oxides).*

  Lateritic soil*: Applies to materials containing less than about 50% of the cementing material (iron and aluminium oxides).*

- **Netterberg, 1985:** ferricrete is a soil which has been to a greater or lesser extent cemented or replaced by iron oxides and/or hydroxides. Is formed by absolute accumulation through upward, downward or lateral migration of iron and may form in a lateral weathering profile.

- **Charman (1988):**
  Laterite: *A highly weathered natural material formed by the concentration of the hydrated oxides of iron or aluminium. This concentration may be by residual accumulation of by solution, movement and chemical precipitation. In all cases it is the result of secondary physio-chemical processes and not of the normal primary processes of sedimentation, metamorphism, volcanism or plutonism. The accumulated hydrated oxides are sufficiently concentrated to affect the character of the deposit in which they occur. They may be present alone in an unhardened soil; as a hardened layer; or as a constituent such as concretionary nodules in a soil matrix or a cemented matrix enclosing other materials.*

As is apparent from the various definitions listed above, there is no internationally agreed definition for laterite. However, as has been pointed out by Vallerga et al (1969; **in** Gidigasu, 1976), the most important consideration for engineering purposes is "not what is its name, but what are its significant geotechnical characteristics and engineering behaviour".

One of the broadest definitions used to describe laterite is that by Gidigasu (1976) as "all the reddish residual and non-residual tropically weathered soils, which genetically form a chain of materials ranging from decomposed rock through clays to sesquioxide-rich crusts".

However, this definition can only be commended if it can be shown that geotechnical tests and criteria do in fact adequately predict performance and that geological, pedological and chemical tests and criteria (such as the S/R ratio) are of no geotechnical significance. This does not appear to have been shown to be the case. On the other hand, it also does not appear to have been shown that the S/R ratio is always a necessary part of a definition or of a relaxed material specification for laterites.

Notwithstanding the above uncertainties regarding the significance of the S/R ratio, in the absence of any evidence to the contrary, and in view of the fact that this ratio is used in specifications for lateritic soil and gravel bases and subbases in Angola and Mozambique (LNEC et al, 1959, 1969) and in Brazil (Departamento Nacional de Estradas de Rodagem (DNER) 1974), it must be concluded that the ***S/R ratio remains a necessary precaution at this stage of our knowledge for application of the "relaxed" specifications used in Brazil.*** Moreover, these requirements are generally in keeping with the more quantitative definition proposed at the specialty session on Pedogenic materials (Netterberg, 1976), as illustrated in Figure 2-4 (modified from Madu, 1975).

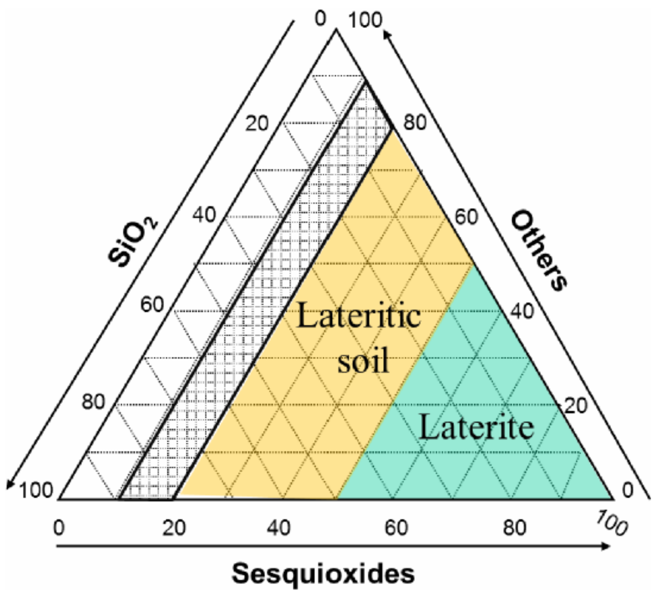

Figure 2-4: Ternary diagram showing composition of lateritic materials

2.3.3 Proposed definition

For purposes of this review, the following definitions for laterite are used:

(a) **Laterite:** should be used to refer to the more strongly cemented or replaced materials that contain a minimum of 50% of the cementing material (iron and aluminium oxides).

(b) **Lateritic soil:** should be used to refer to material:

- (i) that contains less than about 50% (but more than 20%) of the cementing material, and/or
- (ii) that has only been modified, and/or
- (iii) that is less well developed, and/or
- (iv) in which the parent or host material is still dominant

(c) **Lateritic material**: should be used as a generic term to refer to either laterite or lateritic soil as described above.

In light of the above discussion, laterite can generally be described as "a tropical soil that has been produced by advanced weathering accompanied by a relative enrichment in iron and aluminium sesquioxides due to the decomposition of primary minerals and the removal (eluviation) of bases and silica" (Netterberg, 1985). Ferricrete is probably better defined as a pedogenic material that has formed by "chemical precipitates deposited in a particular layer of the soil profile or in the pedological B-horizon by the soil forming process of eluviation" (Brink and Williams, 1964). In other words, applying D'Hoores' (1954) concepts, as here defined, a laterite is formed by a relative accumulation of sesquioxides and a ferricrete by an absolute accumulation as illustrated in Figure 2-5.

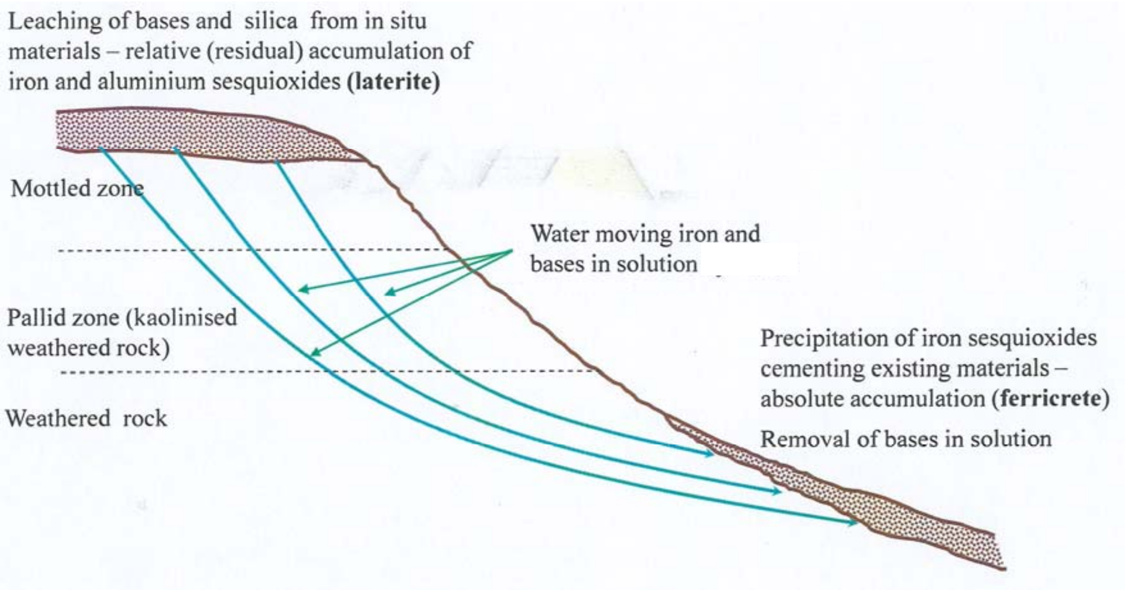

Figure 2-5: Formation of laterite and ferricrete

The above schematic reflects the explanation given by Widdowson (2003) who differentiated between ferricrete and laterite as follows: Ferricretes are those duricrusts which incorporate materials non-indigenous to the immediate locality in which the duricrust formed. In many instances the transported materials can be readily identified as pebbles or clasts derived from adjacent lithological terranes, or as fragments from indurated layers of earlier generations of laterite or ferricrete. By contrast, laterites are iron-rich duricrusts which have formed directly from the breakdown of materials in their immediate vicinity, and so do not contain any readily identifiable allochthonous component.

It is thus proposed that the term *ferricrete,* much as it may be associated with the genesis and properties of a laterite, is not used in the road engineering context as its usage is confined to limited geographic areas, notably Australia, South Africa and Zimbabwe. The presence of iron and aluminium hydroxides, common to both true laterite and ferricrete is the dominant factor that appears to affect the performance of these materials, and the generic term lateritic material. is thus proposed for general use by road engineers. It is suspected that many of the ferricretes may not actually have S/R values less than the prescribed 2 referred to in some specifications for the use of laterites in roads.

Although kaolinite is nearly always the predominant clay in laterites as defined above, ferricretes could contain any clay mineral present in the area of formation. This could often include smectites.

2.3.4 Distribution

The formation of the principal mineral components of laterite, the hydrated oxides of iron and aluminium, are mostly confined to the humid tropical and sub-tropical zones of the world, including Africa, India, South-East Asia, Australia, Central and South America. It should be emphasized that, because of shifts of climatic zone in the geological past, important areas of laterite can be found in areas now outside the tropics (Charman,1988). The geographical distribution of laterites is shown in Figure 2-6.

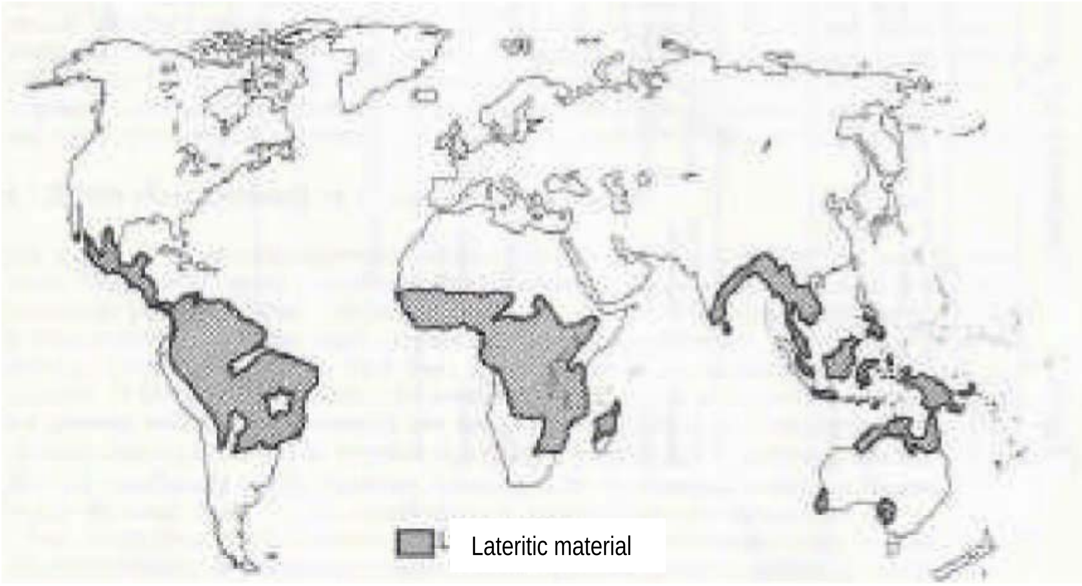

Figure 2-6: Generalized world map showing distribution of laterites (Quinones, Nixon and Skipp, Saunders and Fookes)

2.4 Classification and Composition

2.4.1 Classification

There are specific features or characteristics of tropical residual soils such as laterite that are not adequately covered by conventional methods of soil classification such as the Unified Soil Classification System. Among these features are the following (Blight, 1997):

- The unusual clay mineralogy of some tropical and subtropical soils results in characteristics that are not compatible with those normally associated with the group to which the soil belongs according to existing systems such as the Unified Soil Classification System.
- The soil mass in-situ may display a sequence of materials ranging from a true soil to a soft rock depending on the degree of weathering, which can not be adequately described using existing systems based on classification of transported soils in temperate climates.
- Conventional soil classification systems focus primarily on the properties of the soil in its remoulded state; this is often misleading with residual soils as their properties are likely to be most strongly influenced by in situ structural characteristics inherited from the original rock mass or developed as a consequence of weathering.

A number of systems have been proposed for classifying laterites. For example, Wesley (1988) proposed a practical system based on the mineralogical composition and soil micro and macro-structure. This system is intended to provide an ordinary division of residual soils into groups that belong together because of common factors in their formation and / or composition, which can be expected to give them similar engineering properties (Blight, 1997). The extrapolation of this approach for laterites has resulted in a classification system that depends on the degree of concretionary development in which the physical properties of laterites vary widely from soil to rock-like material, as shown in the recommended classification system presented in Table 2-4 (Charman, 1988).

Table 2-4: Charman's recommended classification system for laterite (adopted from Charman, 1988)

Age	Recommended name	Characteristic	Equivalent terms in the literature
Immature (young)	PLINTH ITE	Soil fabric containing a significant amount of laterite material. Hydrated oxides present at expense of some soil material. Unhardened, no nodules present, but may be slight evidence of nodular development.	Plinthite, laterite, lateritic clay
Mature (old)	NODULAR LATERITE	Distinct hard nodules present as separate particles.	Laterite gravel, ironstone gravel, pisolitic gravel, concretionary gravel.
	HONEYCOMB LATERITE	Nodules have coalesced to form a porous structure which may be filled with soil material.	Vesicular laterite, pisolitic iron-stone, vermicular ironstone, cellular ironstone, spaced pisolitic laterite.
	HARDPAN LATERITE BOULDER	Indurated laterite layer, massive and tough.	Ferricrete, ironstone, laterite crust, vermiform laterite, packed pisolitic laterite.
	BOULDER LATERITE	May be honeycomb or hardpan, but is the result of weathering of a pre-existing layer and may display brecciated appearance.	

Notes: The recommended names should be used as qualifying terms after a normal soil or rock description and do not replace the need for a full textural, strength and colour description in accordance with recommended practice, e.g.

(i) Soft yellowish-brown slightly sandy clay with occasional concretionary zones (up to 10 mm in dia.) hard to very weak material (PLINTHITE?).

(ii) Weak to moderately weak reddish-brown well-cemented porous textured medium gravel sized concretionary HONEYCOMB LATERITE.

The recommended names in Table 2-4 should not be used in isolation, but should be qualified by the necessary descriptive terminology developed, for example, in Brink and Williams (1964). Typical examples of complete descriptions are given in notes accompanying Table 2-4. Plinthite, in most cases, will not be positively identified and should be described fully in soil terms with a qualification '(Plinthite?)'.

Laterites, particularly nodular laterites, often contain quartz gravel. This can be an important factor governing their performance in pavements, particularly when the concretions or nodules are relatively weak. The relative proportion of quartz to concretionary nodules should be described in accordance with the guidelines given in Table 2-5.

Table 2-5: Description of nodular laterite containing quartz (In Charman, 1988)

Composition of gravel fraction	Description
<5% quartz	slightly quartzitic lateritic gravel
5% - 20% quartz	quartzitic laterite gravel
> 20% quartz	very quartzitic laterite gravel
About equal proportions of quartz and concretionary nodules	quartz/laterite gravel
> 20% concretionary nodules	very lateritic quartzitic gravel
5% - 20% concretionary nodules	lateritic quartz gravel
<5% concretionary nodules	slightly lateritic quartz gravel

e.g. Dense reddish-brown silty sandy quartzitic laterite GRAVEL (NODULAR LATERITE)

The above classification covers the proper material description of laterites. It must be emphasized that efficient exploitation of laterite deposits depends not only on a detailed material description but, on describing the field relationship of the laterite deposit. Such information would include, for example, variations in depth and thickness, lateral continuity and ease of excavation.

Illustrations of nodular and hardpan laterite are given in photos 2-3 to 2-6 (Source: International Focus Group: on Rural Road Engineering: Information Note-Laterite (www.ifgworld.org)

Photo 2-3: Nodular laterite (in profile)

Photo 2-4: Nodular laterite borrow pit

Photo 2-5: Hardpan laterite in quarry

Photo 2-6: Stockpiled hardpan laterite boulders

2.4.2 Composition

Laterites are essentially two-component mixtures of the original host or parent material and the authigenic cementing, replacing or relatively accumulated minerals (mostly sesquioxides but also certain clay minerals). As the laterite develops, so the authigenic mineral content increases until it may constitute almost the whole material. Thus, hardpan laterite can be expected to have a higher content of sesquioxides (Al_2O_3 + Fe_2O_3) than a nodular laterite. Table 2-6 shows the typical chemical composition of materials described as laterites (or ferricretes in southern Africa).

The citrate-carbonate-dithionite (CBD)-extractable iron content (a measure of the total **free** iron oxide and hydroxide minerals present) of hardpans laterites ranges between 43 and 77 % (Fitzpatrick, 1978; Fitzpatrick and Schwertmann, 1982). In the case of lateritic soils and gravels the content of Fe_2O_3 increases and that of Al_2O_3 decreases with particle size, while SiO_2 is highest in intermediate fractions (LNEC et al, 1969).

Table 2-6: Typical chemical composition of lateritic material (Netterberg, 1985)

Component	% By Mass	Main form of occurrence
SiO_2	5 - 70	Quartz, feldspar, clay minerals
Al_2O_3	5 - 35	Feldspar, clay minerals, gibbsite
Fe_2O_3 [2]	5 - 70	Goethite, hematite
TiO_2	0 - 5	Anatase, rutile
MnO	0–5?	?
P_2O_5	0-1	
H_2O +	5 - 20	Clay minerals, goethite, gibbsite
Loss on Ignition	5 - 30	Clay minerals, goethite, gibbsite, organic matter
Organic matter	0,2 - 2	Organic matter

Notes:

[1] Bauxites are excluded.
[2] Total iron as Fe_2O_3.

The minerals usually found in laterites are summarized in Table 2-7 (Netterberg, 2013).

Table 2-7: Sesquioxide minerals typically found in lateritic material

Major Element	Mineral [1]	Composition [2]	Colour [2]
Fe	limonite [3]	$Fe \cdot OH \cdot nH_2O$	yellow to brown
	goethite	α – FeO(OH)	yellow to brown to black
	lepidocrocite	γ– FeO(OH)	orange
	haematite	α – Fe_2O_3	red, reddish brown to black
	maghemite	γ - Fe_2O_3	reddish brown
	magnetite	Fe_3O_4	Iron black
	ferrihydrite	$Fe_5HO_8 \cdot H_2O$ [4]	reddish brown
Al	gibbsite	γ– $Al(OH_3)$	white, greyish, greenish or reddish white
	boehmite	γ – AlO(OH)	white, grey, pale lavender, yellow-green
	diaspore	α –-AlO(OH)	white grey, pale lavender, yellow-green
Mn	pyrolusite?	MnO_2	iron black
	manganite?	MnOOH	grey to black
Ti	anatase	TiO_2	red, reddish brown to black
	rutile	TiO_2	red, reddish brown to black
	ilmenite	$FeTiO_3$	Iron black

Notes:

[1] Compiled from the various authors quoted in the text and Dixon and Weed (1989). Other non-sesquioxide minerals include kaolinite, halloysite, metahalloysite, illite, smectite, chlorite, and allophane; whilst significant organic matter may also be present.

[2] Mostly from Klein and Hurlbut (1993) and Dixon and Weed (1989).

[3] A field term used to refer to natural hydrous iron oxides of uncertain identity (Klein and Hurlbut, 1993).

[4] Also given as $Fe_5O_7(OH) \cdot 4H_2O$, $Fe_2O_3 \cdot 2FeOOH \cdot 2.6\ H_2O$, $Fe_5HO_8 \cdot 4H_2O$, etc.

Studies on the mineralogy of laterites in Angola, and Mozambique (LNEC et al, 1959 1969), South Africa (Van der Merwe and Heystek, 1952; Maud, 1965; Frankel and Bayliss 1966, Fitzpatrick, 1974, 1978, 1983) all show the iron and aluminum in these materials to be dominantly in the form of goethite FeO(OH) (yellow-brown) with lesser haematite (Fe_2O_3), (red) and gibbsite (Al $(OH)_3$) (white), and rarely maghemite (Fe_2O_3) (reddish-brown). Traces of anatase (TiO_2) and rutile (TiO_2) may also be present.

No information is available on the S/R ratios of the laterites and lateritic soils actually used as roadbuilding materials in southern Africa. It is therefore recommended that as an interim measure, this ratio be determined both on the fraction passing 2.00 mm and on that passing 2 µm of a selection of such materials and that it also be determined before the relaxed Angolan or Brazilian specifications are applied to a particular case.

3. GEOTECHNICAL PROPERTIES AND TESTING

3.1 Introduction

The geotechnical properties of lateritic materials generally depend on three factors:

- The nature of the host or parent material (e.g. whether it was predominantly clay, sand or rock);
- The stage of development (i.e. the extent to which the host material has been cemented or replaced); and
- The nature of the cementing and/or replacing sesquioxide minerals

During development, the finer particles, such as clay, silt and sand, tend to become flocculated, aggregated, and cemented into silt to gravel-sized particles of varying strength and porosity (Netterberg, 1969a; 1971; various authors cited in Gidigasu, 1976; and Morin and Todor, 1976). These particles or aggregations may or may not be broken down during laboratory testing and during construction. Moreover, both the clay mineral and the cementing and replacing minerals are different from the minerals in the temperate zone soils consisting of discrete particles from which much of our geotechnical experience and specifications have been derived. Laterites can therefore be expected to exhibit certain differences in behaviour from "traditional", temperate zone, materials as illustrated in Table 3-1.

Table 3-1: Differences between traditional and pedogenic materials (Netterberg, 1976)

Property	Traditional	Pedogenic
Composition	Natural or crushed aggregate with fines	Varies from clay to rock
Aggregate	Solid, strong rock	Porous, weakly cemented fines
Fines	Rock particles with or without clay	Cemented, coated and aggregated clay and/or silt particles
Clay minerals	Mostly illite or smectite	Wide variety, e.g. kaolinite halloysite, palygorskite etc.
Cement	None (usually)	Iron oxides, calcium carbonate, etc.
Hydration	None	Variable
Chemical reactivity	Inert	Reactive
Solubility	Insoluble	May be soluble
Weathering	Weathering or stable	Forming or weathering
Atterberg limits	Stable	Sensitive to drying and mixing
Grading	Stable	Sensitive to drying and working
Salinity	Non-saline	May be saline
Self-stabilization	Non-self-stabilizing	May be self-stabilizing
Stabilization (cement)	Increase strength	Usually increases strength
Stabilization (lime)	Decreases plasticity	Usually decreases plasticity and/or increases strength
Variability	Homogeneous	Extremely variable
Climate	Temperate to cold	Arid, tropical, temperate
Traffic	High	Low

Note: "Traditional" materials: comprise typically fluvioglacial gravels found in temperate, northern hemisphere countries as well as crushed rock.
Pedogenic materials: comprise materials such as laterite and calcrete and are formed by pedogenic processes.

The presence of porous particles found in laterite, for example, will tend to increase all moisture content determinations, including Atterberg limits, whereas in traditional soil mechanics it is usually assumed that all the water is outside the particles. Kaolinite, the dominant clay in most lateritic materials, has a non-expansive lattice which, compared to other clay mineral types such as smectite, makes the material less susceptible to volumetric expansion in the presence of moisture. Moreover, the sesquioxides in laterites may be hydrated and/or amorphous, while clays such as hydrated halloysite and allophane may be present. The possible effects of these minerals have been well reviewed by Morin and Todor (1976) and Gidigasu (1976) and, to a large extent, account for the so-called “relaxed” specifications adopted for selecting laterites, compared with the more traditional specifications such as those of AASHTO (2011).

In essence, the differences between traditional and pedogenic materials (e.g. laterite) render the geotechnical behaviour of the latter less predictable for the interpretation of the results of fundamental engineering tests such as Atterberg limits and grading.

3.2 Properties and Testing

The geotechnical properties of laterites generally considered to be the most relevant to their performance include:

- Particle size distribution
- Atterberg Limits
- Strength of the coarse particles
- Compaction and bearing strength

Another aspect of lateritic materials which may be relevant is the concept of self-hardening, or a time-dependent improvement in performance in traditional specifications. Thus, if certain types of laterite can be shown to exhibit a time-dependent or construction-dependent improvement in performance, then this property could play a role in their selection for use in road pavements.

3.2.1 Particle size distribution

Grading analyses are only applicable to the more immature types of laterite such as relatively loose or soft soils like plinthite, nodular laterite and honeycomb laterite in their natural state. Other varieties occur as either boulder or hardpan laterite which are too coarse, or as indurated horizons which require excavation and processing before they can properly be said to have a grading. Moreover, such grading may also be changed by construction processes and by the test method adopted. A clear understanding of the assumptions implicit in the test and calculation methods is therefore fundamental to the assessment of any analysis of particle size.

One of these assumptions is that of a constant bulk relative density (BRD) for the soil particles: when determined by measurement this value is usually, on average, over the full range of particle sizes. For nodular laterites, whose coarse fraction is iron-rich and whose fine fraction is kaolinite, this convention may be misleading. The coarse fraction usually has a specific gravity between 3.0 and 3.5 (but sometimes very much higher), while the specific gravity of the fine material is about 2.7. The particle size distribution curve is based on proportions by mass retained on successive sieves, and only represents a particular packing arrangement for a soil of constant bulk particle relative density. Given this possible variation in BRD, a conventionally calculated test applied, for example, to nodular laterites would underestimate the volume content of coarser particles and exaggerate any gap-grading in the material, and would not represent the true packing and mechanical stability of the material as a whole.

It is important, therefore, when using grading analyses, to inspect the material, assess its composition and decide if separate BRD determinations of the fine and coarse fractions should be made. If the specific gravities are significantly different, the grading should be calculated by volume proportions as well as by mass proportions. Nodular laterites tend to be relatively poorly graded by mass, displaying an apparent deficiency of coarse sand and an excess of fine sand. However, if the mass gradings are corrected to a volumetric basis (which is what really matters) they may be found to be improved. Figure 3-1 shows the difference in grading curves when calculated on the basis of mass proportions and volume proportions.

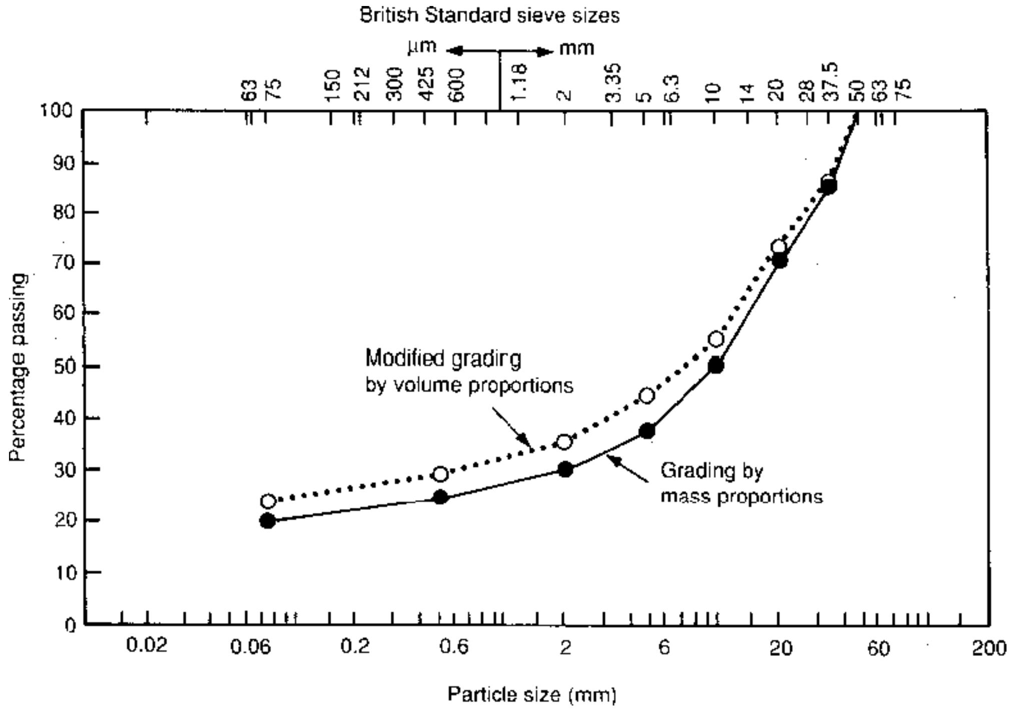

Figure 3-1: Modification to particle size distribution required with lateritic materials (Charman, 1988)

Relatively weak particles may also cause problems in grading analyses. For the analysis to represent the source material, the sample preparation and test procedures should not facture the coarse particles. It is equally important that the fines adhering to the coarse particles be separated. It is therefore recommended that the particles should be soaked until the coating material is fully softened; that only the wet sieving be used; and that a “closed system” of washing be maintained so that no material is lost in the process. Any tendency for the coarse particles to fracture should be recorded on the test reports.

3.2.2 Apparent particle density

No figures for the apparent relative density (ARD) of southern African laterites appear to have been published. Values for 28 laterites on a world-wide basis (size fraction not stated) range between 2,67 and 3,46, with a mean of 3,06 (Krinitsky et al, 1976). Figures of 2.2 to 4.6 for the fraction passing 2 mm are quoted by Gidigasu (1976). Most figures for laterite are higher than those for other soils, reflecting their content of iron minerals.

An important feature of laterites is that the apparent relative density (ARD) and BRD variety with particle size, that of the fines being the lowest (Gidigasu 1976). The ARD of laterites, which is higher than the 2.65 usually assumed for most soils, is significant. ***It is insufficiently appreciated that the grading requirements usually specified on a mass basis assume that the particle BRD does not vary with particle size.*** If this assumption is not met, then the grading specified is actually incorrect. For example, if a laterite gravel with a particle BRD of say 3.0 was mechanically stabilized with sand with a BRD of 2.65 to meet a maximum density-type grading, then 13% too much sand would be added, and an unstable grading would result. Thus, in such cases it is necessary to determine the BRD of the relevant particle size fractions and to allow for these differences.

3.2.3 Atterberg limits, shrinkage and swelling

Atterberg Limits (PI and LL), together with shrinkage and swell limits, are used in most traditional specifications as selection criteria for road construction materials. However, for laterites, the determination of these limits is fraught with a number of complications and the results are also atypical of those associated with traditional (non-pedogenic) materials as summarized below:

(1) **Material variability**: The plasticity of laterites varies widely, both from borrow pit to borrow pit and within a borrow pit. This makes it necessary to stockpile the material very carefully on the basis of a visual assessment of its homogeneity to ensure that each stockpile is as similar as possible for testing purposes.

(2) **Sensitivity to preparation of soil fines**: The results of the Atterberg Limit tests are very sensitive to the manner of preparation of the soil fines in terms of mechanical reworking and drying, and these actions may cause irreversible changes in their engineering properties (e.g. Morin and Todor, 1976; Sharp et al, 2001). Some examples of the effect of remoulding are shown in Table 3-2 which suggest that mechanical processing (e.g. excavating, grading, compacting) of an in situ laterite may degrade its engineering properties.

Table 3-2: Effect of remoulding on Atterberg Limits of lateritic soils (in Townsend, 1985).

Soil	Liquid Limit		Plasticity Index		Source
	Natural	Remoulded	Natural	Remoulded	
Red clay, Kenya	74	84	36	45	Newill (1961)
Red clay, Kenya	77	91	16	32	Newill (1961)
Lateritic soil, Cuba	46	53	15	22	Winterkorn et al (1951)
Lateritic soil, Cuba	60	70	21	30	Townsend)1969)

(3) **Air drying versus oven drying:** Conventional oven drying at 105°C can irreversibly change many of the properties of laterites. Drying is necessary for all of the common tests (grading, Atterberg Limits and compaction characteristics) but it is suspected that oven drying removes some of the water of hydration, which does not affect the material properties but is reflected in higher moisture content determinations. Recommendations of maximum drying temperatures of between 50 and 60°C for laterites are made in the literature.

The data in Figure 3-2 indicate a lowering of Atterberg Limits after oven drying in comparison with air drying. This is in keeping with the findings of Hight et al (1988) who indicated that the difference in PI between undried, air-dried and oven-dried lateritic gravels was significant and that the undried PI of their samples was greater than 30% decreasing to between 15 and 25% after air drying and to between 10 and 15% after oven drying. Lyon Associates Inc (1971) report changes in the PI of more than 40% on oven drying of the samples (e.g. 33.4% as received sample and 18.9% after 24 h drying at 105°C).

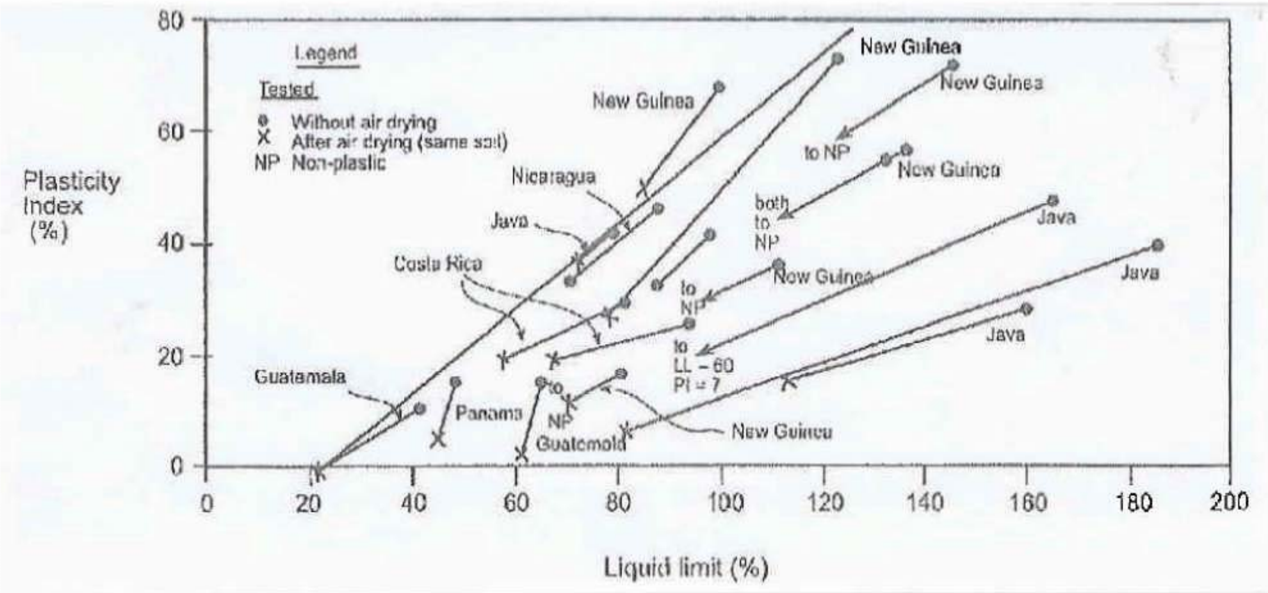

Figure 3-2: Effects of drying on the properties of lateritic soils (Townsend, 1985)

In view of the above findings, ***the form of drying employed in the laboratory should preferably represent the conditions which will apply in the field.*** Thus, laterites should generally be tested at their natural moisture content (Morin and Todor, 1976) or after air drying (LNEC et al, 1969, Morin and Todor, 1976). Brazilian practice appears to require air or oven drying at not more than 60 º C for all soils unless it can be shown that the soil is not affected by drying at a higher temperature. Similarly, MRWA (2002) require that all Atterberg limit testing on lateritic materials be carried out on samples that have been air dried at 50°C. These recommendations are in contrast to standard practice in South Africa TMH1 (NITRR, 1986) which is to oven dry the soil fines at 105 - 110°C.

It is also noteworthy that sesquioxide coatings can cause an irreversible change in plasticity upon drying. It is thought that the sesquioxides in the fine fraction of laterites coat the surface of individual soil (particularly clay) particles, which reduces the ability of the clays to absorb water, thus effectively reducing the measured plasticity (Lyon, 1971).

(4) Period of mixing: Materials with friable and aggregated particles such as laterite are sensitive to the degree and period of mixing. A mixing time of 10 minutes is specified in NITRR (1979; TMH1 (1986) whilst Lyon Associates Inc (1971) suggest that a standard mixing time of 5 minutes (half of the South African and BS standards of 10 minutes) should be rigorously adhered to.

(5) Difference between BS and AASHTO test: An important factor in Atterberg limit testing which is often not appreciated is that the British type of Casagrande and cone liquid limit devices yield LLs **and therefore also PIs** on average about 4 units higher than the American AASHTO and ASTM type (Sampson and Netterberg, 1984). The latter is the type specified in South Africa (NITRR, 1986), but the British type is also encountered, especially in Commonwealth countries such as Kenya and Malawi.

In view of the above problems associated with the determination of Atterberg Limits, the use of the bar linear shrinkage has been suggested as a substitute for the PI (Ackroyd, 1960; Easterbrook, 1962, both in Madu, 1975; Netterberg, 1971 and Gidigasu, 1976). This is in line with non-related work indicating that the South African bar linear shrinkage test (slightly different from the BS test – Paige-Green, 2007) is better than many other indicator tests for predicting performance (Paige-Green and Ventura, 1999).

(6) Plotting of Atterberg Limits on Casagrande Plasticity Chart: Data for laterites on a world basis (Nixon and Skipp, 1957; Gidigasu, 1976; Morin and Todor, 1976) are somewhat contradictory, but it seems clear that laterites and lateritic soils as a group also plot on both sides of the A-line (Mitchell and Sitar, 1982). Lateritic soils which plot below the A-line are likely to be troublesome (Gidigasu, 1976) as they might contain hydrated halloysite (exhibits unusual geotechnical properties). Figure 3-3 shows the location of common clay minerals on Casagrande's plasticity chart with the kaolinites plotting just below, and the halloysites well below, the A-line (Lyons, 1971).

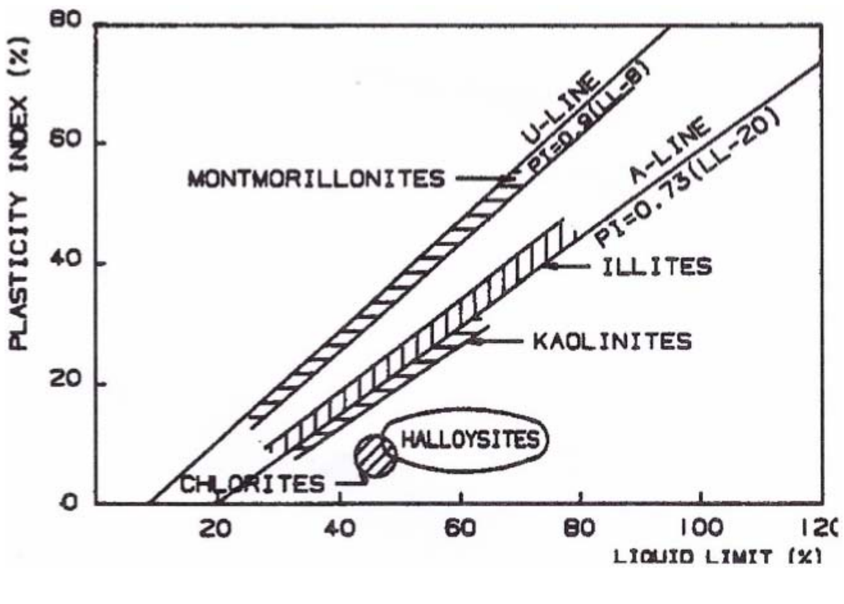

Figure 3-3: Location of common clay minerals on Casagrande's plasticity chart (Morin and Todor, 1976)

(7) Swell: The swell of lateritic soils is low even when the Atterberg limits are high (LNEC et al, 1969) and the De Castro (1969) swell test offers an alternative or supplementary method of assessing the properties of the fines. The maximum swell of the fraction passing 0.425 mm is approximately equal to 8 times the molecular silica / sesquioxide ratio (apparently of the fraction passing 2 µm), and might thus provide an alternative to this ratio.

The various characteristics of laterites described above may explain in large part why apparently high plasticity materials appear to perform satisfactorily, i.e. the plasticity determinations are not representative of the actual material's performance.

3.2.4 Silica/sesquioxide ratio

The silica sesquioxide ratio (S/R) has been used for the classification of laterites as well as for the specification of their use. A value of not larger than 2 has been proposed to define a laterite (Charman, 1988) although Persons (1970 after de Medina (undated)) uses 2 as a maximum for lateritic soil and 1.33 for a laterite. This value should also be related to actual performance in roads to gain the most benefit.

A wide range of values is found in the literature, but it would appear that not all values are derived using the same formula: some use the total [$SiO_2/(Al_2O_3 + Fe_2O_3)$], while others are normalized for the molecular masses prior to calculation as shown below.

The silica sesquioxide ratio (S/R) is calculated as follows: (DNER-ME 030/94):

$$\frac{S}{R} = \frac{\frac{SiO_2}{60}}{\frac{Al_2O_3}{102} + \frac{Fe_2O_3}{160}}$$

The values provided by Madu (1975) and Lyon associates Inc (1971) for instance are not corrected for the individual molecular masses and yet use the same ratios of 1.33 and 2.0 as described above. However, only the molecular ratio should be used, rendering the above ratio incomplete and incorrect.

Another of the problems with this test is the fraction that is tested. Sometimes the testing appears to only be done on the fine fraction (2 mm, 0.425 mm or even 0.002 mm), while others use the entire grading (ABP, 1976; Cocks and Hamory, 1988). There are likely to be significant differences in the results. The Brazilian method is based on an air-dried sample lightly crushed (to break down aggregated lumps and not particles) sieved through a 2 mm screen. Cocks and Hamory (1988) used the minus 0.425 mm fraction for their work.

Cocks and Hamory (1988) used two methods to determine the S/R. The first method used X- ray fluorescence (XRF) to determine the Al_2O_3, Fe_2O_3 and SiO_2. The quartz content was determined by X-ray diffraction (XRD) and the combined silica for the S/R was the difference between the XRF total silica and the XRD quartz. The second method used acid to dissolve the laterite combined with inductively coupled plasma spectrometry (ICP) to quantify the sesquioxide and silica proportions. The results were mixed, with 5 samples giving ICP results higher than XRD/XRF and 8 giving lower results. They attributed the differences to the extent that the wet chemistry actually dissolves the oxides.

Brazil has a standard test method (DNER-ME 030-94) based purely on wet chemistry for the extraction and determination of the components using titrations. Modern techniques such as Inductively Coupled Plasma (ICP) mass spectroscopy or atomic absorption analyses should be used to replace the quantification portion of the Brazil analyses, retaining the extraction techniques, which seem to have a sound basis.

Other than in Brazil, the S/R ratio has not been fully introduced into road material specification as it is not routinely adopted in standard engineering soils laboratories, is time consuming and is costly (De Graft-Johnson, 1975).

3.2.5 Compaction and CBR

The moisture content at the time of compaction can have a critical influence on the CBR test results. Lyon Associates Inc (1971) show that compaction at only slightly higher than OMC drastically reduces the CBR values.

It is noted by a number of workers that each point on the compaction curve and for CBR testing must be done on new material (materials should not be reused as their properties change) (Lyon Associates, 1971; LNEC, 1959). It is also noted that the pre-treatment can affect the results with oven-dried materials having the highest MDDs and lowest OMCs compared with those at natural moisture content (Lyon Associates Inc, 1971).

Laboratory CBR values are often extraordinarily high when compacted to Maximum Dry Density (MDD) at Optimum Moisture Content (OMC). Many laterites also retain their strength on soaking and have low swells (< 0.5%). However, a number of authorities specify testing in the unsoaked condition only (De Graft-Johnson, 1975; Madu, 1975; Aggarwal and Jafri, 1987), but none seem to explicitly state that ***the materials should be allowed to equilibrate (condition) after compaction***. This appears to be a significant omission based on recent southern Africa experience where strengths of most materials (not only laterites) seem to increase on equilibration/conditioning, allowing relaxation of some of the compaction stresses and dissipation of pore water pressures.

Rough correlations for laterites between CBR and the product of the PI and the percent passing 0,425 mm have been given by Morin and Todor (1976) and Gidigasu (1976). For example, if this product is 160 or less the Modified AASHO CBR is at least 80. The soaked CBRs of laterite-quartz gravels and their MDDs and OMCs are related as follows (DeGraft Johnson et al, 1972; **in** Gidigasu, 1976).

CBR = $72.5 \log_{10} \frac{MDD}{16PI} - 7.5$ % where MDD is in kg m^{-3} (r = 0.68).

The MDD and OMC of African lateritic soils are related as follows (Morin and Todor, 1976):
MDD = 2563 – 44.5 OMC (kg m^{-3}) (r = – 0.84, *SD* = ± 88 kg m^{-3}, *n*=81

CBRs at OMC are on average about 50 % higher than soaked CBRs of laterites at intermediate compaction (Gidigasu and Bhatia, 1971), and calcretes at Modified AASHO compaction.

CBR swells are generally low, almost regardless of PI and gradings.

3.2.6 Triaxial and resilient properties

With modern mechanistic-empirical analysis and design techniques, the use of parameters such as the resilient modulus and Poisson's ratio is increasing. This type of testing is, however, relatively specialized and costly and is restricted mostly to research laboratories and is not routinely carried out in commercial laboratories. However, these properties can assist in understanding the behaviour of materials under loading conditions in roads and have been used for this purpose (Nogami and Villibor, 1991; MRWA, 2002).

MRWA (2002) makes use of the West Australian Confined Compression Test (WACCT) which is essentially a triaxial test in which the shear strength of the material at various moisture and density conditions is assessed. It is similar to the Texas Triaxial Test and is used in a number of countries and states for routine pavement design (e.g., Zimbabwe, Texas, various states in Australia). However, the revised and updated document (MRWA, 2014) has dropped the WACCT test results as selection criteria for lateritic materials and only the CBR results are now used.

3.2.7 Hardening and self-stabilization

Some pedogenic materials possess the ability to undergo self-hardening. Indeed, the original definition of laterite (i.e. Buchanan's laterite or the originally American definition of plinthite) was just such a material. Alexander and Cady (1962) indicated that some laterites, when wetted and dried, harden with time as a result of solution and crystal growth. They hypothesized that microcrystalline goethite is adsorbed onto kaolinite crystals, rendering this iron ineffective for self-hardening. This area (i.e. the relationship between kaolinite and iron content) should be addressed in search on potential self-stabilizing materials.

Charman (1988) indicates that oxisols (most intensely weathered of all soils rich in iron and aluminium oxides and kaolinitic clays) become irreversibly cemented when exposed to repeated wetting and drying. This is evidence for self-cementation. Materials that may harden on repeated wetting/drying cycles are classed as ferralsols by the FAO-UNESCO classification. Sweere et al (1988) also discuss a weak material with a high iron content that performed better than much stronger materials under wet conditions and attributed the good performance to the alternate wetting and drying in situ.

Although MacVicar et al (1977) doubt whether self-hardening plinthite occurs in South Africa, Du Toit (1954, p.451) stated that it does, and that this property is valued in roadbuilding. As pointed out by Grant and Aitchison (1970), only actively forming laterites can [probably] be expected to possess this property.

In light of the above, it is apparent that there is clear evidence that some laterites possess the ability to undergo self-hardening, and that large increases in soaked CBR strength may be attainable in the laboratory after several wetting and drying cycles or even simple curing (Rosseau, 1982). Nonetheless, documented evidence of the value of self-stabilization in road construction is generally lacking (Netterberg, 1975) although some success has been claimed in Australia (Morin and Todor 1976). It should not be forgotten that it is quite possible that simply drying a plastic material could induce a measure of apparent self-cementation, due to shrinkage-induced development of a structure with an increased density, water resistance and/or reduced permeability which would be largely or completely lost on re-wetting.

Test methods for a potentially self-stabilizing material include the Petrification Degree test (Nascimento et al, 1965) and the soaked CBR after subjecting the material compacted in the mould to drying, moist curing or to wetting and drying cycles. (DaSilva et al, 1967; Novais-Ferreira and Meireles, 1969; Netterberg, 1969a, 1971; 1975; Van der Merwe and Bate, 1971). Comparison of the soaked CBR of untreated and dried, cured or subjected to five cycles of wetting and drying appears to be the most reliable and easy to interpret of these tests (Netterberg, 1975). As the CBR sometimes **decreases**, it is probably also a useful durability test. The development of self-stabilization in a pavement layer cannot, however, be assured at this stage and this should be looked upon rather as an added (and uncertain) safety factor.

Drying back an unstabilized pavement layer either deliberately or incidentally to its equilibrium moisture content before covering it with the next one is in any event generally good practice and may induce self-stabilization in certain laterites. However, wetting and drying cycles seem to be necessary in the case of calcretes and other laterites and this is difficult to achieve in practice. Morin and Todor (1976) concluded that the property of self-hardening can rarely be used to advantage in construction.

A period of dry curing was deliberately included in the Texas Triaxial test method to simulate the usual predrying during construction before priming and the associated irreversible hardening reported by construction personnel (C McDowell 1975; Texas Highway Dept., pers. comm.).

After subjecting several Botswana lateritic and non-lateritic gravel samples to the five-cycle wet/dry CBR test, Overby (1990, a, b) concluded that the results were erratic, the increase in strength negligible in comparison with the increases attained at lower moisture contents, and that it was the latter together with the low equilibrium moisture contents which accounted for the good performance of the roads monitored. However, perusal of the results of XRD analyses of the samples (Overby, 1990 b) shows that the main components of the "laterites" were quartz and feldspar and that only traces of goethite were present. The greatest increase found in soaked Mod. AASHO CBR was from about 70 to 105 for a material simply described as laterite (also with quartz and feldspar as the main components and only a trace of goethite) with a PI of 7.

If the poor reproducibility of the CBR test is considered, then probably nothing less than doubling of the soaked CBR should be regarded as indicating a significant potential for self-cementation.

Further research on cycled CBR test for potential self-stabilization appears justified, for example, on the effects of drying temperature and time, compactive effort, number of cycles, and removing the weight and swell plate during cycling.

After subjecting five laterites to the Petrifaction Degree test and one to the cycled CBR test, Van der Merwe and Bate (1971) concluded that the Zimbabwean laterites used in road construction did not appear to possess significant self-cementing properties and that their outstanding performance was due rather to the sandy nature of the parent material and the rough surface texture of the coarse aggregate.

Induration is not necessarily permanent, and under the right conditions pedocretes will weather like any other rock. Thus, disintegration and solution takes place during weathering, and softening of laterite crusts occurs on reforestation (Maignen, 1966; Morin and Todor, 1976). In essence, the compatibility of a pedogenic material with any change in its environment needs to be assessed for all important works. This is perhaps most obvious with materials containing soluble salts, gypsum and carbonates, but under reducing conditions such as might occur in earth dams iron might also go into solution (Donaldson, 1967) and at least one such case appears to have occurred (Anagosti, 1969).

The first indication of a cycled CBR being used to predict self-stabilization is by Da Silva et al (1967) where cycles of 24 hours drying and 4 days soaking were used to identify "petrifaction". They also carried out petrifaction degree testing as proposed by Nascimento et al (1964). Significant increases in CBR strength were obtained after 10 CBR cycles (from 37/38% to between 48 and 67%). Netterberg (1975) concluded that self-stabilization seemed to be likely to occur in practice and that the cycled CBR test appeared to be a promising indicator of the potential for self-stabilization.

3.2.8 Aggregate strength/hardness/durability

Various tests are specified for establishing the aggregate strength and durability of laterites, although in most documents, these two properties are not differentiated, with aggregate strength test results being synonymous with durability. This is not necessarily the case. The Los Angeles Abrasion test is probably (and incorrectly) used as an indicator of durability most commonly (Ruenkrairergsa, 1987).

The Aggregate Impact Value (AIV) test appears to be the most used test in African countries for the estimation and specification of the strength of aggregate particles (De Graft Johnson et al, 1972; Gidigasu and Dogbey, 1980). This is a simple, cheap test and is recommended when a knowledge of particle strength is necessary as discussed under *Specifications.*

In South Africa and Zimbabwe, the Aggregate Crushing value (ACV) and 10% Fines Aggregate Crushing Test (FACT) have always been the standard tests for aggregate strength, although the Durability Mill Index test (Sampson and Netterberg, 1989) is now preferred in South Africa for natural gravels. It is noteworthy, however, that South African experience has shown that material durability is seldom a problem in low volume roads, with Durability Mill Index (DMI) values in excess of 1000 being determined on a number of roads investigated that had performed well (Paige-Green, 1999).

3.2.9 DN values

Because of the inherent problems associated with testing laterites and the fact that many of the tests are not really appropriate for establishing their potential performance, it would probably be wiser to make use of more direct laboratory and field test methods. These include properties such as DCP DN values. The DCP DN value can be easily and cheaply obtained on a large number of samples and well-quantified statistically. It is usually done on standard CBR moulds in the laboratory (at any required moisture and density condition and best after 4 days of equilibration which allows the moisture to be equally distributed, pore water pressures to be dissipated and some of the compaction stresses to be released). It is suggested that this test be investigated further and a standard protocol developed specifically for the use of lateritic materials. .

4. SPECIFICATION

4.1 Introduction

Laterites have certain unique properties that make them different from other road construction materials. This often results in their test results not conforming to conventional specification requirements and the materials being rejected for use, despite performing well when used in roads. For this reason, a number of countries have specifications specifically tailored for the use of laterites. Despite their properties not complying with many of the existing specification requirements, their performance in service is surprisingly good. Gidigasu and Dogbey (1980) note for instance that the lateritic gravels in Ghana have much poorer gradings than the local residual granitic gravels but, nonetheless, have performed satisfactorily.

Apart from the naturally poor gradings, the particle size distributions of laterites are affected significantly by the material preparation techniques used prior to testing. Ackroyd (1967) indicates that the aggregations of silt and clay caused by iron oxides are broken down during normal laboratory testing, but not during construction, giving results that are not directly related to the as-built condition.

Where the materials have weak aggregate particles in the gravel sized range, a good particle size distribution is necessary to ensure that the gravel particles have as many grain to grain contacts as possible with minimal voids. This will improve the “support” for the aggregate particles and reduce the internal stresses allowing them to bear larger loads without fracturing. An alternative method of protecting the coarse aggregate is for it to float in the fines, but such a grading is normally inferior to the continuously graded material. Most limits for the fine material have been derived in temperate climates and have been based on the need to avoid frost damage.

4.2 Review of Existing Specifications

4.2.1 General

Numerous specifications have been proposed in the literature emanating from a variety of authorities all, not surprisingly, with differing requirements, presumably based on local experience. Some of the differences in specification requirements are summarized below:

- Most South African road authorities, other than KwaZulu Natal who allow a PI of up to 10, do not appear to relax their normal requirements in the case of lateritic materials.

- Zimbabwe (Van der Merwe and Bate, 1971; Zimbabwe Ministry of Roads and Road Traffic, 1979), Angola and Mozambique (LNEC et al, 1959; 1969; Dos Santos 1971), have found that relaxations of grading, Atterberg limits and aggregate strength are possible provided certain other criteria are met (Table 4-1).

- In the case of Zimbabwe a 50 % relaxation in PI and PP is permitted for “certain” pedocretes (laterites and calcretes: Mitchell et al, 1975; Zimbabwe Min. of Roads and Road Traffic, 1979) provided the strength (usually the Triaxial Class) is met.

- In the case of the LNEC et al (1969) the other criteria are the swell in the De Castro (1969) test as well as the silica/sesquioxide ratio. Plasticity indices of up to 18 are permitted (LNEC et al, 1969), but values of up to 21 can be used for up to 300 vpd (50 % heavy vehicles) provided an impermeable seal is used and construction to specification is strictly controlled (Meireles, 1967). Although CBR requirements are shown, it appears from Dos Santos et al (1971) that these two PI requirements would ensure minimum CBRs of 60 and 50, respectively. However, these authors emphasize that the quality of the seal is more important than that of the base course.

- The Brazilian specification (Associacao Brasileira de Pavimentacao, 1976), by comparison with their specification for other materials, shows that considerable relaxations are allowed, even for relatively heavy traffic.

- In the Angolan specification the De Castro (1969) swell test is omitted in preference to a maximum CBR swell of 0.5 % (LNEC et al, 1969). Brazilian (DNER) practice is to require a maximum CBR swell of 0.2 % (at Modified AASHO or intermediate compaction) unless the DeCastro (1969) swell is less than 10 %, in which case 0.5 % is permitted (De Souza et al, 1984). The molecular silica/sesquioxide ratio of < 2 is still required.

- Specifications for lateritic materials in subbase and other applications are also given by these authorities, while Zimbabwe also has specifications for modified and stabilized materials.

- Specifications for laterites have also been formulated by others such as Morin and Todor (1976) and Gidigasu (1983), while sections of roads with laterite bases comparable to those in Table 4-1, have performed successfully in Botswana (Overby, 1982), Kenya and Malawi (Grace, 1991).

4.2.2 Typical Specifications

Traditional materials specifications mostly developed in North America (e.g. AASHTO M147-65, 2011) have generally been found to be too conservative for pedogenic materials in terms of their strict grading and plasticity requirements, while they do not control certain other constituents of pedogenic materials such as salts. It has therefore been necessary to derive empirical specifications for these materials locally for which some examples are given in the tables below:

Table 4-1: Some laterite and lateritic soil base specifications In comparison with the Brazilian general granular base specification (Netterberg and Paige-Green, 1988)

Material	**Granular**	**DNER-ER-P10-71**	**Laterite**	**DNER-ES- P47-74**	**Laterite LEA-CE-11-62**	**Fine sandy lateritic soils [2]**
Authority	**DNER [1] (1974)**	**Brazil**	**DNER (1974) Brazil**	**Brazil**	**LNEC et al (1969)**	**Utiyama et al(1977) Brazil**
Percent passing (mm)	B	C	A	B		
50.8	100		100		Up to 1.4 times that in ASTM D1241-55T	-
25.4	75-90	100	75-100	100		-
9.5	40-75	50-85	40-85	60-95		-
4.8	30-60	35-65	20-75	30-85		-
2.0	20-45	25-50	15-60	15-60		-
0.42	15-30	15-30	10-45	10-45		85-100
0.074	5-15	5-15	5-30	5-30		25-45
LL %	≤25	≤25	≤40		≤40	20-30
PI %	≤6	≤6	≤15		≤15	6-9
Swell[3]%	-	-	≤10		≤10	-
Aggregate Strength%	LAA (>2mm fraction) ≤55	≤55	LAA(>2mm fraction) ≤65		LAA ≤65	- -
CBR % (soaked)	≥60(<5ME80)		≥60(<5ME80)		-	>80@OMC
	≥80(>5ME80)		≥80 (>5ME80)		-	
CBR % Swell	≤0.5		≤0.5		-	<0,1
S/R [4]	-		<2		<2 [5]	<2? [6]
Rainfall (mm)	Mostly 1 000 - 2 500 mm				>700?	1100-1500
Surfacing	Surface treatment (S.T.) to 50 mm asphalt				S.T.	S.T. to 50 mm asphalt
Traffic limitations	None?		None?		None?	<1 000vpd or <10 M E80

Notes:

1. DNER = Department of National Roads standard specification.
2. Mostly B horizons (some A and C) of orthic and acric ferrasols, ferric luvisols, ferralic arenosols in Sao Paulo state of Brazil.
3. De Castro (1969) swell test (LNEC Technical Paper 235) or Brazilian Method DNER-ME 29- 74
4. S/R = silica/sesquioxide molecular ratio (on <**2 mm** fraction) by Brazilian Method DNER-ME 30-72.
5. S/R on **<5 µm** fraction by LNEC (1959) methods
6. Not specified, but probably advisable (preferably on <2 µm fraction) if used outside Brazil.
7. Normal CBR swell requirement 0.2 %; however, up to 0.5 % allowed if Castro swell < 10 %.

Table 4-2: Charman's (1988) recommended material selection criteria for lateritic gravel sub-bases and road-bases under thin bituminous surfacing in the tropics

USE	ROAD BASE			SUB-BASE
TYPE OF ROAD	MINOR	MAJOR RURAL	AND URBAN	ALL
TRAFFIC (X10^6 esa)	<0.3	0.3-1	1-3	<3
Grading	Grading Modulus ≥1.5[1]	Grading envelopes in Charman, 1988.		specifications
Plasticity index (%)				
Moist, wet tropical	≤ 12	≤ 10	≤ 6	≤ 25
Seasonal tropical	≤15	≤12	≤10	≤25
Semi-arid & arid	≤ 20	≤ 15	≤ 12	≤ 25
Plasticity modulus [2]				
Moist, wet tropical	≤ 300	≤ 200	≤ 150	≤ 500
Seasonal tropical	≤ 400	≤ 250	≤ 200	≤ 750
Semi-arid & arid	≤ 500	≤ 350	≤ 250	≤ 1 250
CBR (%)	≥ 45 [3]	≥ 65 [4]	≥ 80 [4]	≥ 25 [3]
Durability				
Los Angeles Abrasion value (%) [5]	≤ 65	≤ 50	≤ 50	n.s
10% fines value (saturated) (kN)	n.s	≥50	≥50	n.s.

NOTES:

n.s. = not specified

[1] Grading modulus = $\frac{300 - (P2.0 + P0.425 + P0.075]}{100}$

Where P2.0 is percentage passing 2 mm sieve
P0.425 is percentage passing 0,425 mm sieve
P0.075 is percentage passing 0,075 mm sieve
Values of grading modulus are between 0 and 3.

[2] Plasticity modulus = plasticity index multiplied by percentage finer than 0.425 mm sieve.

[3] CBR on samples compacted to 95% of modified AASHO or BS (heavy) MDD and soaked for 4 days.

[4] CBR on samples compacted to 100 % of modified AASHO or BS (heavy) MDD and soaked for 4 days.

[5] Los Angeles abrasion value on fraction coarser than 2 or 2.36 mm.

Krinitzsky et al (1976) discuss various "relaxations" of specifications for laterite bases from selected countries. The main relaxations are for plasticity, but in the case of Thailand, PI x percentage passing 0.075 mm limits are proposed with values of 15 to 20 depending on traffic. This effectively is much stricter than most conventional specifications. Ghana had at that stage developed specifications for laterite bases for 3 different traffic categories, class I to III. These included slight relaxations of the grading envelopes over the AASHTO requirements, a move away from the LAA specification preferred by many countries towards the ACV (values between 35 and 50%), CBR requirements of 100, 70 and 50% for Class I, II and III roads respectively and requirements for the products of PI and P075 (200 to 600) and LL and P075 (600– 1250).

Based on their review, Krinitzsky et al (1976) suggest various requirements for laterite base materials (Table 4-3).

Table 4-3: Krinitzsky et al's (1976) recommended criteria for laterite base course materials

Criteria	Class I (Heavy)	Class II (Medium)	Class III (light)	Agency
CBR (min %)	100 80	70 60	50	BRRI DNER
Liquid Limit (max %)	40 35	 40	N/S	LEA DNER
LLxP075(max)	600	900	1250	BRRI
PI (max %)	10 15	12	N/S	DNER LEA
PI x P075 (max)	200	400	600	BRRI
ACV (max %)	35	35-40	40-50	BRRI
LAA (max %)	65	65	N/S	LEA and DNER

BRRI – Building and Road Research Institute, Kumasi, Ghana.
DNER – Departemento Nacional de estradas de Rodagem, Brazil
LEA – Laboratorio de Engenharia de Angola.
Note: These recommendations do not take different test methods, in situ densities, etc into account.

Krinitzsky et al (1976) also compared the specifications for gravel (not specifically laterite) base course materials from 13 African countries where laterites are common and relaxations of certain requirements had been permitted. These are summarized in Table 4-4. It is interesting to note that these requirements are for gravels, and although laterites probably predominate in these countries, no distinction appears to be made between the requirements for laterites and other natural gravels.

Table 4-4: Krinitzsky et al's (1976) summary of base course specifications for selected African countries

Country		Malawi	Niger	Kenya	Nigeria	Mali	Ivory Coast	Senegal	Cameroon	Gabon	Gambia	Zambia	Uganda
Type of material		Gravel	Gravel	Gravel	Gravel	Gravel	Gravel	Gravel	Gravel		Gravel	Gravel	Gravel
Grading envelopes	Sieve size (mm):					Standard AASHTO grading							
	37.5	100	100		100						80-100	100	100
	19	60-90			80-100						60-100		85-100
	9.5	45-75			55-80			<60% passing 0.246 mm			35-83		68-1 00
	4.75	30-60			40-60						28-62		54-1 00
	2.0	20-50			30-50		30-65				25-50		43-90
	0.425	10-30									22-44		30-57
	0.075	5-15	≤ 25		5-15		16-30	20-35			13-28	15-20	19-38
Plasticity	Liquid limit (%) max	30			25						20-37	30	37-48
	Plasticity Index (%) max	6	12		10	6-16	12	10-25	10-25	20	13-22	6	16-25
	PIxP425 (max)			250									
CBR	4-day soaked (%) min	85	80	80	80	50	60	80	80	60		120	

Grace and Toll (1987) looked at the use of laterites in Kenya and Malawi and referred to the specifications in tropical countries that were summarized by Krintzsky et al (1976). They indicate that the laterites used on two successful roads generally had high PIs (7-21%), wide ranging laboratory soaked CBRs (12 to 96% at 95% BS compaction) and percentages finer than 0.063 mm of 15 to 49%).

Ruenkrairergsa (1987) discussed the use of laterites on lower volume roads in Thailand, where traditional AASHTO specifications used by the roads authorities result in extremely high construction costs. It is interesting to note, however, that the philosophy in Thailand was to reduce the plasticity of the laterites, rather than to change the specification limits. This results in higher processing costs rather than making use of materials that are probably appropriate and thus appears to be the incorrect path to follow. Ruenkrairergsa also indicates that a Los Angeles Abrasion value of 50% is the upper limit for ensuring durability of laterites for low to medium volume roads.

Akpokodje and Hudec (1992) assessed the possible improvement of determining the aggregate strength using the standard Aggregate Impact Value (AIV). They found that the most important predictors of aggregate strength and abrasion resistance were water absorption and bulk density. Unfortunately, they had no comprehensive performance-related data and compared their data with a typical upper limit of 30% for the AIV identified from previous studies (mostly Bhatia and Hammond, 1970).

De Graft Johnson et al (1969) developed the Suitability Index for the selection of lateritic soils. This parameter is derived from the ratio of the percentage larger than 2 mm and the liquid limit multiplied by the log of the PI. This can be used to predict the CBR (CBR = (SI x 35 – 8)). Typically a value of between 2.1 and 4.0 is required for potential base course materials.

De Graft Johnson et al (1972) discussed a laterite rock aggregate classification system for determining the potential for use of the materials in roads proposed by Bhatia and Hammond, (1970). This was based on non-traditional selection criteria in Ghana as presented in Table 4-5.

Table 4-5: Criteria used in Ghana for selecting laterites (De Graft Johnson et al, 1972)

Specific gravity	Water absorption after 24h soaking (%)	Aggregate Impact Value (%)	Los Angeles Abrasion value (%)	Rating based on probable in situ behaviour
>2.85	<4	<30	<40	Excellent
2.85–2.75	4–6	30-40	40–50	Good
2.75–2.58	6–8	40–50	50–60	Fair
<2.58	>8	>50	>60	Poor

Gidigasu and Mate-Korley (1984) also reviewed the Ghanaian materials and road performance and came up with various specifications for different environments. In the arid zones, a minimum soaked CBR of 69 (at 100.5% MDD) and maximum PI of 8% was suggested. In the dry sub-humid zones, a PI of 7 ± 4% was suggested and in the moist sub-humid zones a maximum PI of 10% was suggested with maximum products of the percentage passing 0.075 mm and the LL and the PI of 300 and 200 respectively.

Aggarwal and Jaffri (1987) studied various laterite roads in Nigeria and based on their performance suggested a relaxation of the PI up to 12% and a minimum CBR of 65%, provided that the compaction specification is achieved. No detail regarding the compaction specification or whether the CBR is determined at OMC or soaked is given, although in their prior discussion, it is stated that the CBR is carried out at OMC unless the standing water level is within 1.2 m of the road surface.

Gourley and Greening 91997 and 1999) give guidelines for reducing the requirements for laterite bases, after carrying out research on various low volume roads in southern Africa. Relaxations of PI and plasticity modulus (PM) of up to 25% and more than 800m respectively are permitted for very low volume roads (Table 4-6).

Table 4-6: Gourley and Greening's (1999) proposed guideline for selection of lateritic gravel base materials for low volume roads with unsealed shoulders.

Subgrade CBR		Design traffic class					
		≯0.01	0.05	0.1	0.3	0.5	1.0
S2	I_P	≯15	≯15	≯12	≯9	≯9	≯6
	PM	≯400	≯250	≯150	≯150	≯120	≯90
	GE	B	B	B	A	A	A
S3	I_P	≯18	≯15	≯15	≯12	≯9	≯6
	PM	≯550	≯320	≯250	≯180	≯120	≯90
	GE	B	B	B	B	A	A
S4	I_P	≯20(1)	≯18	≯15	≯15	≯9	≯9
	PM	≯800	≯450	≯320	≯300	≯200	≯90
	GE	GM 1.6-2.6	B	B	B	B	A
S5	I_P	≯25(1)	≯20	≯18	≯15	≯12	≯9
	PM	n/s	≯550	≯400	≯350	≯250	≯150
	GE	GM 1.6-2.6	B	B	B	B	B
S6	I_P	≯25(1)	≯20	≯20	≯18	≯15	≯12
	PM	n/s	≯650	≯550	≯400	≯300	≯180
	GE	GM 1.6-2.6	B	B	B	B	A

Notes:
(1) Ip maximum = 8 x GM
n/s = not specified
Unsealed shoulders are assumed

Ip = plasticity index
PM = plasticity modulus
GE = grading envelope
GM = grading modulus

The most recent full specification for the use of laterite gravels is that of Western Australia Main Roads Department (MRWA, 2002). The requirements for lateritic gravel and crushed laterite (hardpan) for different applications in roads are summarized in Tables 4-7 to 4-19. The current revision of the MRWA document (MRWA, 2014) has dropped the WACCT test results and uses only the CBR results. It is interesting to note that MRWA only specify a requirement for Fe and Al oxides for crushed hardpan laterite (not laterite gravels) and then in the form of combined Al2O3 + Fe2O3 (≥ 20%) and not silica sesquioxide ratio (S/R). Earlier work within the Department (Cocks and Hamory, 1998) specified a minimum limit for the Al2O3 + Fe2O3 content for all lateritic gravels of 10%. Table 4-7 specifies the materials according to the material class based on PI according to Table 4-8 (i.e. maximum PI of 6%, etc).

Table 4-7: Typical selection criteria for lateritic gravels based on grading and classification tests (1) (MRWA, 2002)

Type of Material	Lateritic Gravel				Crushed Rock (2)
Designation		Lt6	Lt10	Lt16	NS
	Sieve Size mm	% Passing	% Passing	% Passing	% Passing
	37.5	100(4)	100(4)	100(3)	
	26.5				100
	19.0	71-100	95-100	95-100	95-100
Grading (4)	13.2				70-90
	9.5	50-81	50-100	50-100	60-80
	4.75	36-66	36-81	36-81	40-60
	2.36	25-53	25-66	25-66	30-45
	1.18	18-43	18-53	18-53	20-35
	0.60				13-27
	0.425	11-32	11-39	11-39	11-23
	0.30				8-20
	0.15				5-14
	0.075	4-19	4-23	4-23	5-11
	0.0135	2-9	2-11	2-11	
Dust Ratio(6)		0.3-0.7	0.3-0.7	0.3-0.7	0.35-0.6
Liquid Limit (7) %		≤25	≤30	≤35	≤25
Plasticity Index %		≤6	≤10	≤16	NS
Linear Shrinkage %		≤3	≤5	≤8	0.4-2.0
P0.425xLS(7)		≤150	≤200	≤250	NS(10)
Expected maximum Dry Compressive Strength(8) kPa		≥1700	≥1700	≥1700	≥1700
Particle Toughness		Note (9)	Note (9)	Note (9)	35
Dryback%		≤85(11)	≤85(11)	≤85(11)	≤60(11)

Notes:

NS = Not specified

(1) Selection criteria apply to base course roads with a thin bituminous surfacing with a 20 year design traffic loading of up to $5x10^6$ ESA. For higher traffic loadings, specialist advice should be sought, or Table 4-10 should be used.

(2) Non lateritic, included for comparison purposes only. MRWA document 6706/02/1 321

(3) Most deposits of lateritic gravel contain some oversize material which must be broken down by grid rolling.

(4) Dry Sieving and Decantation, Test Method WA 115.1.

(5) Dust Ratio = P0.075/P0.425

Material with a low dust ratio is likely to be harsh and the achievement of a satisfactory surface may prove difficult.

(6) Liquid Limit (using the cone apparatus), Plasticity Index and Linear Shrinkage tests on samples air dried at 50°C.

(7) For materials approaching the upper limit for plasticity index or P 0.425 x LS confirmation of suitability by strength testing is recommended.

(8) Maximum Dry Compressive Strength, Test Method WA 140.1

(9) No particular test for particle toughness is specified at this time. However the lateritic pebble must be hard and durable.

(10) NS = not specified.

Base course should be dried back to a moisture content of less than 85% (~ 60% for Crushed Rock) of OMC prior to application of bituminous surfacing.

Table 4-8: Typical selection criteria for lateritic gravel based on strength and classification tests [1] (MRWA, 2002)

DESIGNATION		Lt6	Lt10	Lt16
WACCT (2),(4)				
	Class No	≤2.0	≤2.0	≤2.3
	Cohesion[3] kPa	≥85	≥85	≥85
	Tensile Strength kPa	≥55	≥55	≥55
CBR[5]	Soaked	≥80	≥60	≥60
	Unsoaked	≥80	≥80	≥80
Maximum Size[6] mm		37.5	37.5	37.5
Grading Modulus[7]		≥1.5	≥1.5	≥1.5
Dust Ratio[8]		0.3-0.7	0.3-0.7	0.3-0.7
Plasticity Index %		≤6	≤10	≤16
Linear Shrinkage %		≤3	≤5	≤8
$P_{0.425}$ x LS		≤150	≤200	≤250
Particle Toughness		Note[9]	Note[9]	Note[9]
Dryback[10]%		≤85	≤85	≤85

Notes:

(1) Selection criteria apply to base course roads with a thin bituminous surfacing with a 20 year design traffic of up to $5x10^6$ ESA. For traffic exceeding $5x10^6$ ESA, specialist advice should be sought or Table 4-10 should be used.

(2) West Australian Confined Compressive Test, Test Method WA 142.1. Class number, cohesion and tensile strength assessed at specified density for project and at the design moisture content for the site.

(3) Cohesion and tensile strength may be reduced to 45 kPa and 30 kPa respectively provided the angle of shearing resistance is >60°. These parameters are not as critical where the shoulders are sealed.

(4) The criteria for the assessment by WACCT may not be met when testing specimens immediately after compaction. In these cases the specimens should be compacted at 100% OMC, dried to design moisture content and cured for three weeks without further loss of moisture prior to testing.

(5) CBR specimens compacted to OMC to the specified density for the project and tested at design unsoaked moisture conditions. Test method WA 141.1.

(6) Most deposits of lateritic material include some oversize material which must be broken down by grid rolling.

(7) Grading Modulus = $\frac{300-(P2.36 + P0.425 + P0.075)}{100}$

(8) Dust Ratio $= \frac{P_{0.075}}{P_{0.425}}$

(9) No particular test for particle toughness is specified at this time. However the lateritic pebble must be hard and durable.

(10) Base course should be dried back to a moisture content of less than 85% of OMC prior to application of bituminous surfacing.

Table 4-9: Typical selection criteria for crushed lateritic caprock for warm humid and warm sub humid zones (MRWA, 2002)

Traffic ESA[(1)]		$<5x10^6$	$<10^6$	$<10^5$
	SIEVE SIZE	% Passing	% Passing	% Passing
Grading[(2)]	37.5	100	100	100
	26.5			
	19.0	71-100	71-100	71-100
	13.2			
	9.5	50-81	50-81	50-81
	4.75	36-66	36-66	36-66
	2.36	25-53	25-53	25-53
	1.18	18-43	18-43	18-43
	0.60	13-32	13-32	13-32
	0.425	11-28	11-28	11-28
	0.30	9-25	9-25	9-25
	0.15	6-20	6-20	6-20
	0.075	4-16	4-16	4-16
	0.0135	2-9	2-9	2-9
Dust Ratio[(3)]		0.3-0.7	0.3-0.7	0.3-0.7
Liquid Limit[(4)]		≤25	≤30	≤35
Plasticity Index %		≤6	≤8	≤10
Linear Shrinkage		≤3	≤4	≤5
$P_{0.425}$x LS[(5)]		≤150	≤200	≤250
MDCS[(6)] kPa		≥ 1700	≥ 1700	≥ 1700
Al_2O_3+ Fe_2O_3[(7)]%		≥ 20	≥ 20	≥ 20
Los Angeles Abrasion%		≤50	≤50	≤50
Point Load Index (MPa)[(8)]		≥1.0	≥0.5	≥0.5
Flakiness Index %		≤20	≤20	≤20
CBR (Soaked) [(9)]		≥80	≥80	≥80
Dryback(10)%		≤85	≤85	≤85

Notes:

(1) Selection criteria apply to base course on well drained roads with a thin bituminous surfacing with a 20 year design traffic loading of up to 5 MESA. Use with higher traffic loadings, or adverse drainage conditions is not precluded. However, specialist advice should be sought, or Table 4-10 should be used.

(2) Dry sieving and Decantation, Test Method WA 115.1

(3) Dust Ratio = $P_{0.075}/P_{0.425}$

Material with a low dust ratio is likely to be harsh and the achievement of a satisfactory surface prove difficult.

(4) Liquid Limit (using the cone apparatus), Plasticity Index and Linear Shrinkage tests on samples air dried at 50°C.

(5) For materials approaching the upper limit for plasticity index, or P0.425 x LS, confirmation of suitability by strength testing is recommended.

(6) Maximum Dry Compressive Strength, Test Method WA 140.1.

(7) Al_2O_3 + Fe_2O_3 determined on the fraction passing a 0.425mm sieve.

(8) Average of 20 tests on rock fragments or core.

(9) CBR specimens compacted at OMC to the specified density for the project, typically 96% modified. Test Method WA 141.1.

(10) Base course should be dried back to a moisture content of less than 85% of OMC prior to application of bituminous surfacing.

Table 4-10: Typical selection criteria for lateritic gravels used on heavy duty pavements based on grading and classification tests (MRWA, 2002)

Lateritic gravel for heavy duty pavements[1]		Target grading[2]	Range
Grading[3]	**Sieve Size**	% Passing	% Passing
	37.5	100	100
	19.0	80	72-100
	9.5	57	50-78
	4.75	43	36-58
	2.36	31	25-44
	1.18	23	18-35
	0.60	18	13-28
	0.425	15	11-25
	0.30	13	9-22
	0.15	9	6-17
	0.075	7	4-13
	0.0135	4	2-9
Liquid Limit[4]		≤25	
Linear Shrinkage (%)		≤2	
MDCS[5] kPa		≥ 2300	
CBR (Soaked) [6]		≥ 80	
Dust ratio[7]		0.3 - 0.7	
Dryback % [8]		≤85	

Notes:

(1) Selection criteria apply to base course on roads with a thin bituminous surfacing with a 20 year design traffic loading of up to 1 x 10^7 ESA. For higher traffic loadings, specialist advice should be sought.

(2) Dry sieving and Decantation, Test Method WA 115.1

(3) Dust Ratio = P 0.075/P 0.425

Material with a low dust ratio is likely to be harsh and the achievement of a satisfactory surface prove difficult.

(4) Liquid Limit (using the cone apparatus), Plasticity Index and Linear Shrinkage tests on samples air dried at 50°C.

(5) For materials approaching the upper limit for plasticity index, or P0.425 x LS, confirmation of suitability by strength testing is recommended.

(6) Maximum Dry Compressive Strength, Test Method WA 140.1.

(7) Al_2O_3 + Fe_2O_3 determined by ICP on the fraction passing a 0.425mm sieve.

(8) Average of 20 tests on rock fragments or core.

(9) CBR specimens compacted at OMC to the specified density for the project, typically 96% modified. Test Method WA 141.1.

(10) Base course should be dried back to a moisture content of less than 85% of OMC prior to application of bituminous surfacing.

4.2.3 Summary of specification requirements

It has become clear in recent years that the most important criterion for the selection of natural gravels and soils and especially pedogenic materials, for pavement materials is a test for compacted strength (e.g. a CBR or Texas Triaxial) at the likely in-service moisture content, and that grading and Atterberg limit requirements are of less importance. However, in addition to a strength requirement it also appears wise to control the CBR swell. Another important point is to **ensure that one is actually dealing with a lateritic material**, and a material with a S/R ratio < 2. This is not always as easy as it appears, and as an interim measure it is suggested that the molecular S/R ratio on the fraction passing 2 µm (or possibly 0.425 or 2.00 mm) be used in the case of laterites and lateritic soils.

Grading also remains important with respect to compactability and finish and plastic gravels cannot be compacted when too wet.

Once compacted, **aggregate strength** is probably only important for the relatively more heavily trafficked road categories. In Zimbabwe laterites with dry/soaked 10 % FACT values of 50/40 kN have been used successfully and a minimum soaked value of about 50 kN seems adequate for traffic up to about 2000 vpd (Mitchell and Sitar, 1982).

In applying the specifications shown here – or any foreign specifications for that matter – the test methods employed must be comparable with those of the authority concerned. For example, while oven drying of soil fines at 105 – 110 º C is standard in South Africa, drying at not more than 60 º C or even air drying is common in countries where lateritic materials are used extensively, such as Angola and Brazil.

The CBR may also be increased by oven drying (LNEC et al, 1969) before compaction. Moreover, it is even less appreciated that in South Africa only the CBR at 2.54 mm penetration is used, whereas in practically all other countries the higher of the CBR at 2.54 and 5.08 mm penetrations is used. Except in the case of stabilized materials, southern African experience indicates that the CBR at a penetration of 5.0 mm is almost always significantly higher (on average 20%) than that at 2.54 mm (Pinard and Netterberg, 2012).

In summary, relaxations in Atterberg limits, grading and CBR requirements seem to be the norm when using laterites. The use of combinations of plasticity and grading indices is also prevalent. However, the specification that has probably been the most successfully applied to the greatest length of roads is that used in Brazil, making use of the CBR for lateritic gravels and the MCT test procedure for fine-grained lateritic soils.

The standard Brazilian specifications for base courses for roads (DNIT 141/2010) are fairly conventional with requirements for particle size distribution (typical Fuller-type requirements), Atterberg limits (PI< 6%), dust ratio, CBR (60% for less than 5 million axles and 80% for more) and Los Angeles Abrasion loss (max of 55%). However, they also have a separate specification for base courses using lateritic gravels (DNIT 098/2007) which is summarized in Table 4-11.

Table 4-11: Summary of Brazilian national specifications (DNIT 098/2007) for laterite base course

Property	Limits		Comments	
Silica-sesquioxide ratio	≤ 2			
California Bearing Ratio (%)	≥ 60		DNER ME 49/74 (56 blows per layer compaction at design moisture content)	
Liquid limit (%)	≤ 40			
Plasticity Index (%)	≤ 15			
Los Angeles Abrasion	≤ 65			
Grading (% passing sieve)	Grading A:	Grading B:	Tolerances specified:	
50.8 mm	100			
25.4 mm	75 - 100	100	Sieve size:	%age passing:
9.5mm	40 - 85	60 - 95		
4.8 mm	20 - 75	30 - 85	9.5 – 25.4	± 7
2.09 mm	15 - 60	15 - 60	0.42 – 4.8	± 5
0.42 mm	10 - 45	10 - 45	0.075	± 2
0.075 mm	5 – 30	5 – 30		
Grading modulus	1.65 – 2.70	1.65 - 2.70		
Dust ratio	≤0.67	0.67		
Sand equivalent (%)	≥ 30			

4.2.4 Recommended specification

Based on a review of a number of regional and international specifications for the use of laterite in road pavements, it is apparent that the Brazilian specifications (Associacao Brasileira de Pavimentacao, 1976) permit considerable relaxations, even for relatively heavy traffic. Thus, in view of the successful use of laterite in the construction of both high and low volume roads in Brazil over the past 40 years, the Brazilian specifications are recommended as an interim measure until local performance-related specifications have been developed.

4.3 Regional Performance-Related Studies

4.3.1 General

A number of roads have been constructed in the region in which laterite has been used in the pavement layers, both as base course and subbase. An evaluation of the performance of these road projects has been carried out with the objective of comparing the engineering properties of the laterites used in these projects with the Brazilian specifications presented in Table 4-11. Only the Atterberg Limits, gradings and CBRs are available for most of the roads that were reviewed. It should also be noted that the CBRs determined for all of the African examples are the soaked laboratory values. However, indications are given in some documents regarding in situ CBR values, determined using a Dynamic Cone Penetrometer.

4.3.2 South Africa

During an assessment of the performance of 57 roads constructed with marginal materials in their bases carried out between 1990 and 1994 in South Africa (Paige-Green, 1999), a number of the roads were described as being constructed of laterite by the relevant road authorities. During the investigation, 4 of the road bases were described as laterite, one as a quartzitic laterite and one as a manganocrete/laterite.

The roads were between 4 and 8 years old, had carried between 13 000 and 92 000 ESAs and had generally performed satisfactorily. Localized rutting up to 20 mm was observed and apart from the road that had carried 92 000 ESAs with significant bleeding and cracking in the wheel tracks the roads were generally in a good condition. The average riding quality in Quarter Car Index was 43 (approx. IRI of 3.3 which is even better than the terminal riding quality of a freeway (Category A road in TRH 4). All of the roads were certainly significantly better than the previous unpaved roads and all provided very satisfactory performance.

Photo 4-1: South Africa: Example of low volume road constructed with laterite base course (Road 466 – km 16 and 22)

The summarized properties in relation to the relevant Brazilian specifications are presented in Table 4-12.

Table 4-12: Comparison of South African material properties from six base courses (20 test results) with Brazilian specification

Property	Brazilian specifications (DNIT 098/2007)	South African Properties			Comments
		Mean	Max	Min	
Liquid limit (%)	≤ 40	30	38	22	All samples complied
Plasticity Index (%)	≤ 15	12.7	20.5	5.7	8 higher than 15%
P425 P075 mm Grading modulus Dust ratio	10 –45 (± 5) 5–30 (± 2) 1.65 – 2.70 ≤ 0.67	49 25 1.59 0.52	59 34 2.04 0.68	29 11 1.27 0.38	14 > 45% and 8 >50% 2 > 30% 12 < 1.65 1 > 0.67
CBR (%)	≥60				
Soaked CBR (%)	Not specified	56	127	11	11 <60%
DCP CBR (%)	≥ 60	48	106	19	13 < 60%

It is clear that under South African conditions, materials with PIs significantly higher than the Brazilian standard have performed successfully. This is illustrated in Figure 4-1, where a plot of the condition of the roads (1 is very good and 5 is very poor) against the PI is shown.

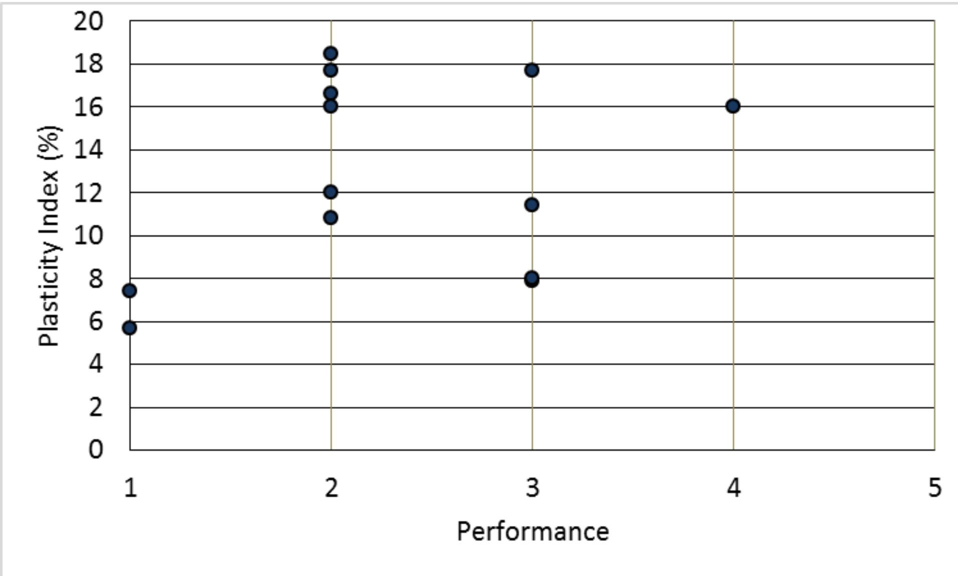

Figure 4-1: Plot of road condition against PI

The grading analyses show similar requirements to the Brazilian specifications although the materials are considerably finer, with more than half of the materials having grading moduli less than the minimum required in Brazil.

It is difficult to compare the CBR values but in general the South African measured CBRs are less than the Brazilian requirement. The relationship between soaked CBR and performance is shown in Figure 4-2.

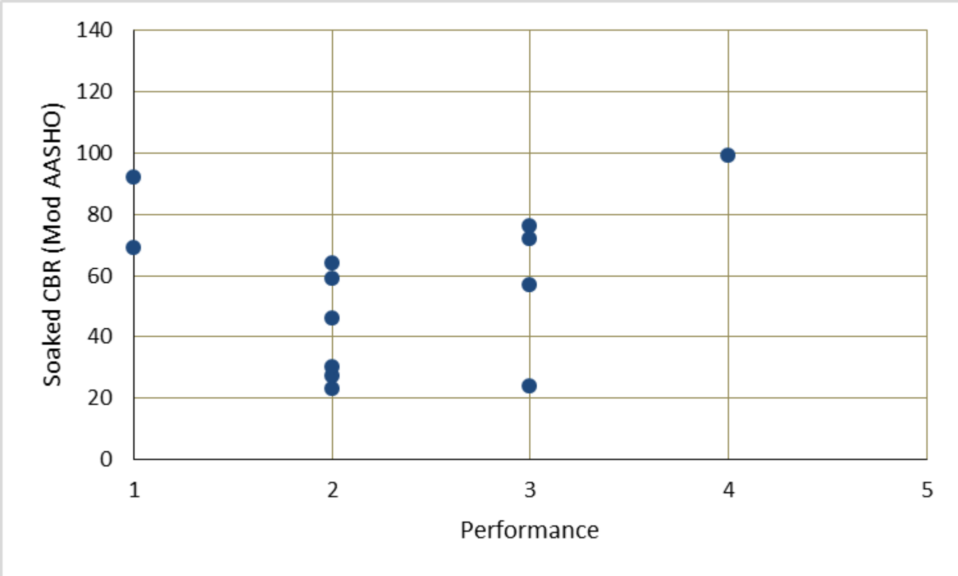

Figure 4-2: Plot of Soaked CBR (Mod AASHO comp. versus performance of the road

Plots of the in situ CBR show similar trends with CBR values of less than 40% performing well in most cases. (Figure 4-3).

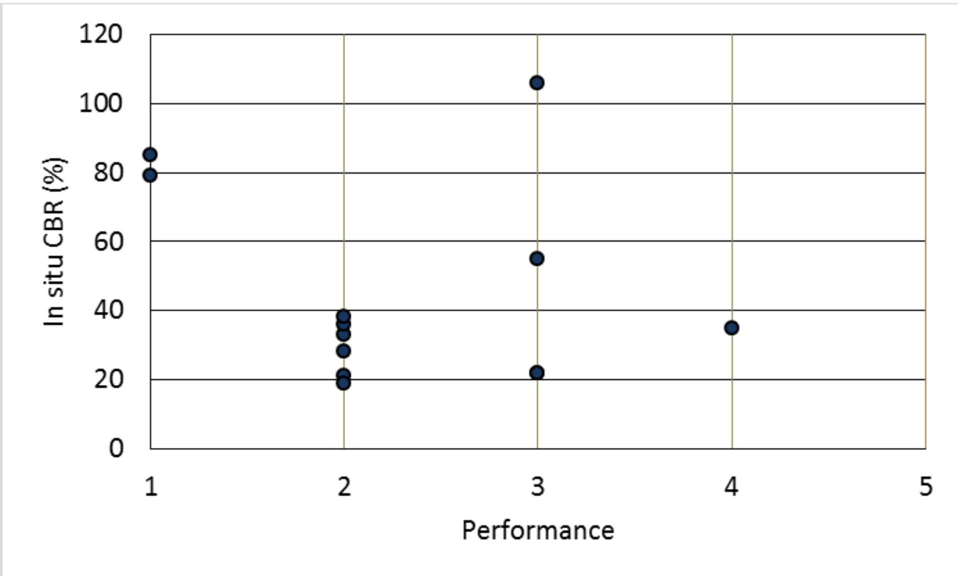

Figure 4-3: Plot of DCP CBR of base versus performance of the road

4.3.3 Botswana

An analysis was carried out of the performance of 23 roads in Botswana that were monitored over a period of 16 years (Paige-Green, 2010). Seven of the roads were described as having base courses of laterite, lateritic gravel or lateritic quartz gravel and had nearly all performed well carrying an estimated traffic of between 400 000 and 2 million standard axles.

Photo 4-2: Botswana example of low volume road constructed with laterite base course

The summarized properties in relations to the relevant Brazilian specifications are as follows:

Table 4-13: Comparison of Botswana material properties from seven laterite base courses with Brazilian specification (Paige-Green, 2010)

Property	Brazilian specifications (DNIT 098/2007)	Botswana Properties			Comments
		Mean	Max	Min	
Liquid limit (%)	≤ 40	28	37	NP	All samples complied
Plasticity Index %	≤ 15	12.3	15	NP	None higher than 15%
% age passing 0.425 mm %age passing 0.075 mm Grading modulus Dust ratio	10 –45 (± 5) 5 – 30 (±2) 1.65 – 2.70 ≤ 0.67	41 16 1.94 0.43	69 26 2.38 0.55	22 12 1.08 0.31	All comply All comply 3 < 1.65 All comply
CBR (%)	≥ 60				
Soaked CBR (%)	Not	85	165	36	6 <60%
DCP CBR (%)	≥ 60	163	235	111	All > 60%

Under the more arid conditions in Botswana, the in situ strengths are significantly higher than the South African ones as well as the Brazilian requirements. The materials are generally a little finer than the Brazilian specifications require but generally all of the materials comply with the main requirements of the Brazilian specifications.

4.3.4 Kenya

Limited data was obtained from a number of experimental roads with laterite bases constructed in Kenya. The results are summarized in Table 4-14:

Table 4-14: Comparison of Kenya material properties from seven laterite base courses with Brazilian specifications (Grace and Toll, 1987)

Property	Brazilian specifications (DNIT	Kenya Properties			Comments
		Mean	Max	Min	
Liquid limit (%)	≤ 40				
Plasticity Index %	≤ 15	19.2	2	15	All higher than 15%
%age passing 0.425 mm % age passing 0.075 mm Grading modulus Dust ratio	10 – 45 (± 5) 5 – 30 (± 2) 1.65 – 2.70 ≤ 0.67	41	48 37	36 30	1 does not comply None comply
CBR (%)	≥ 60				
Soaked CBR (%)[a]	Not	36	6	30	Only 1 > 60%
DCP CBR (%)	≥ 60				
a CBR at 95% BS heavy compaction only					

Based on these results the following specifications for laterites in base courses were proposed for Kenya (Grace, 1991). .

- Neat lateritic gravel can be used on roads with very light traffic (AADT <500 and cumulative ESAs <100 000).
- CBR >25 and 35% and PI <25 and 20% respectively for dry (less than about 700-800 mm rainfall per year) and wet areas.
- Percentage passing 0.075mm <40% and LAA/ACV <70/45.

In the same report, the following relaxed specifications are suggested for very light traffic, provided that the pavement is well drained under all circumstances.

- Maximum size: 10 to 40mm
- Percentage passing 0.075mm maximum 35%
- Plasticity index maximum 20% (?) in dry areas
  maximum 15% (?) in wet areas.
- CBR (4 days soak): maximum 35 in dry areas
  maximum 50 in wet areas
- Los Angeles Abrasion: maximum 60%
- Aggregate Crushing Value: maximum 40%

These are mostly somewhat relaxed compared with the Brazilian requirements, although the CBR values are soaked, unlike the Brazilian requirements that the CBR is tested at the expected in situ moisture content.

Grace and Toll (1987) also discussed the performance of some laterite bases in Kenya. The following test results on materials removed from roads were presented.

Table 4-15: Properties of laterites used in low volume roads in Kenya (Grace & Toll, 1987)In comparison with Brazilian specification

Property	Brazilian specification	Kenya Properties			Comments
		Mea	Max	Min	
Liquid limit (%)	≤ 40				
Plasticity Index (%)	≤ 15	17.6	21	7	Majority don't comply
%age passing 0.425 mm %age passing 0.075 mm Grading modulus Dust ratio	10 –45 (± 5) 5–30 (± 2) 1.65 – 2.70 ≤ 0.67	28	37	15	Majority comply
CBR (%)	≥ 60				
Soaked CBR (%)	Not specified	54	96	21	@ 95% BS heavy
In situ CBR (%)	≥ 60	89	209	47	Majority comply

4.3.5 SADC region (selected countries)

During the 1990s, TRL (Gourley and Greening, 1999) carried out investigations into a number of roads in various countries in southern Africa, including Botswana, Malawi and Zimbabwe. A number of the materials used for the bases of the roads in Malawi and Zimbabwe were described as laterite or lateritic, the results of which are compared with the Brazilian specification below.

Table 4-16: Properties of laterites used in low volume roads in Zimbabwe (Gourley and Greening, 1999) in comparison with the Brazilian specification.

Property	Brazilian specifications (DNIT 098/2007)	Zimbabwe Properties			Comments
		Mean	Max	Min	
Liquid limit (%)	≤ 40	20	20	NP	All comply
Plasticity Index %	≤ 15	5	5	NP	All lower than traditional specification
Percentage passing 0.425 mm Percentage passing 0.075 mm Grading modulus Dust ratio	10–45 (± 5) 5 –30 (± 2) 1.65 – 2.70 ≤ 0.67	37 16 1.85 0.44	40 20 2.0 0.54	35 13 1.7 0.35	All comply All comply All comply All comply
CBR (%)	≥60				
Soaked CBR (%)	Not specified	97	135	50	Only 1 <60%
In situ CBR (%)	≥ 60	78	117	52	Only 1 <60%

It is interesting to note that nearly all of the Zimbabwe base materials investigated that comply with the Brazilian specifications are in fact only marginally outside traditional specifications.

Table 4-17: Properties of laterites used in low volume roads in Malawi (Gourley and Greening, 1999) in comparison with the Brazilian specification

Property	Brazilian specifications (DNIT 098/2007)	Malawi Properties			Comments
		Mean	Max	Min	
Liquid limit (%)	≤ 40	32	35	NP	All comply
Plasticity Index %	≤ 15	17	19	NP	5 of 7 higher than
Percentage passing 0.425 mm	10 –45 (± 5)	46	67	33	1 does not comply
Percentage passing 0.075 mm	5 –30 (± 2)	25	37	17	4 > 30%
Grading modulus	1.65–2.70	1.68	2.00	1.16	3<1.65
Dust ratio	≤ 0.67	0.55	0.59	0.50	All comply
CBR (%)	≥ 60				
Soaked CBR (%)	Not specified	55	85	40	Only 2 of 7 > 60%
In situ CBR (%)	≥ 60	81	140	51	

4.3.6 Mozambique

TRL back analyzed a number of low volume roads that had been constructed in Mozambique over a number of years, primarily to relate their performance to their material properties. The results were summarized in a report, which indicated that one of the roads (Nametil-Angoche) was constructed with a laterite base course. However, a table later in the text of material properties with their descriptions indicates that 4 roads contained laterite with a total of 9 “laterite” samples being tested. The following compares the results from these samples with those specified in Brazil.

Table 4-18: Properties of laterites used in low volume roads in Mozambique (TRL, 2013) in comparison with the Brazilian specification

Property	Brazilian specifications (DNIT 098/2007)	Mozambique Properties			Comments
		Mean	Max	Min	
Liquid limit (%)	≤ 40				
Plasticity Index	≤ 15	1.5	3.7	NP	All comply
Percentage passing 0.425 mm	10 –45 (± 5)	25	30	20	All comply
Percentage passing 0.075 mm	5 –30 (± 2)	25	37	17	All comply
Grading modulus	1.65 – 2.70	2.0	2.2	1.6	1 <1.65
Dust ratio	≤ 0.67	0.68	0.73	0.50	3 don’t comply
CBR (%)	≥ 60				
Soaked CBR (%)	Not specified	64	86	49	
In situ CBR (%)	≥ 60		150+	30	9 of 28 don’t comply

4.3.7 Ethiopia

The results of an experimental section of road constructed in Ethiopia have been included in an AFCAP report and are compared with the Brazilian requirements as follows:

Table 4-19: Properties of laterites used in low volume roads in Ethiopia (TRL, 2013) in comparison with the Brazilian specification

Property	Brazilian specifications (DNIT 098/2007)	Ethiopia Properties			Comments
		Mean	Max	Min	
Liquid limit (%)	≤ 40		47	40	None comply
Plasticity Index %	≤ 15		12	11	All comply
Percentage passing 0.425 mm Percentage passing 0.075 mm Grading modulus Dust ratio	10 –45 (± 5) 5–30 (± 2) 1.65 – 2.70 ≤ 0.67		17.4	9.4	All comply
CBR (%)	≥ 60				Probably don't comply
Soaked CBR (%)	Not specified		69	60	
In situ CBR (%)	≥ 60	90	212	31	Most complied

4.3.8 Malawi

Toll and Grace (1987) discussed the satisfactory performance of some laterite bases in Malawi. The following test results on materials removed from roads were presented.

Table 4-20: Properties of laterites used in low volume roads in Malawi (Grace and Toll, 1987) in comparison with the Brazilian specifications.

Property	Brazilian specifications (DNIT 098/2007)	Malawi Properties			Comments
		Mean	Max	Min	
Liquid limit (%)	≤ 40				
Plasticity Index (%)	≤ 15	16.3	18	14	Most don't comply
Percentage passing 0.425mm Percentage passing 0.075 mm (0.063) Grading modulus Dust ratio	10–45 (± 5) 5 – 30 (± 2) 1.65 – 2.70 ≤ 0.67	37.6	49	14	Majority don't comply
CBR (%)	≥60				
Soaked CBR (%)	Not specified	31.7	45	12	@ 95% BS heavy
In situ CBR (%)	≥ 60	87	120	58	Most comply

From recent investigations of LVRs carried out in Malawi (Pinard, 2010), it was found that none of the laterites used in the construction of the LVSRs complied fully with the traditional standards and specifications normally applied in Malawi in terms of the commonly specified plasticity, grading and strength requirements. PIs were in the range of 15-20 and CBRs less than 40% at 98% BS heavy. Nonetheless, the road sections all performed very satisfactorily after more than ten years in service. These findings support those reported by Grace (1988) and are manifested in no better way than by comparing two adjacent sections of one of the roads which was constructed to "traditional" standards and the other to "LVR" (lower) standards as shown in the photo below.

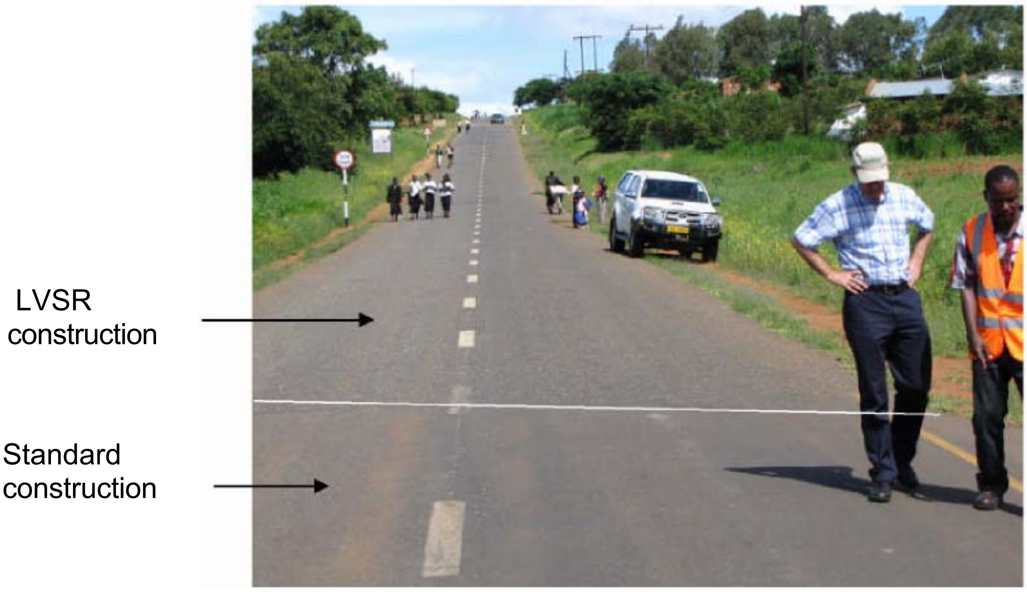

**Photo 4-3: Malawi: Nchisi Boma road: Standard construction (foreground)
LVSR construction (background)**

4.3.9 Conclusions

It is clear that many of the materials that have performed well in a number of African countries do not comply with the Brazilian national specifications for lateritic gravel bases for roads designed to carry up to 5 MESA.. For this reason, it is suggested that a detailed investigation of African roads constructed with laterites be carried out and the specifications adapted to account for the local conditions. In general, the main factor governing performance of base courses is the in situ strength (in relation to overall pavement balance) and being able to maintain this through varying moisture conditions. The potential benefits associated with laterites (and probably pedogenic materials in general) with respect to self-cementation appear to be clearly exhibited in the field, but cannot be relied on, particularly during the early life of the roads where the traffic effects may influence the road prior to any self-cementation occurring.

5. CONSTRUCTION AND USE

5.1 Introduction

In general, layers constructed with lateritic materials can be constructed using standard methods and equipment. However, a number of aspects that may require specific attention have been identified in the literature.

Typically laterites are located and then samples from pits in the proposed borrow area are taken for testing and approval. However, once the borrow pit is approved and winning has commenced, material should be stockpiled and representative samples from the stockpile should be tested as a number of properties may change during winning. Stockpiling also minimizes the variability of the material when placed on the road. The use of scrapers in borrow pits is not recommended as the variability with depth can be large. Stockpiling with a dozer in the borrow pit followed by windrowing and mixing on the road using graders is suggested (Cocks and Hamory, 1988). In Zimbabwe it has been standard practice for many years to only approve a stockpile for a particular layer on the road.

5.2 Examples of Use

Mukerji and Bahlmann (1978) indicated that laterites are difficult to use in road construction as their properties vary considerably and this variability makes their use difficult. They do note the fact that the materials achieve relatively high strengths and water resistance on drying. Similarly when excavated in “vertical cuts, highly laterized soils can be self-stabilizing (after hardening on exposure to the air).”

Nogami and Villibor (1991) indicate that fine grained lateritic soils were used only for subbase (or stabilized with cement for base) until the early 1970s when trial sections were done with neat soils. Routine use of such lateritic soils for base for low to medium traffic in Sao Paulo State started in the 1980s. In many cases they were blended with gap-graded crushed stone for more heavily trafficked roads. The area has rain in all months with between 1000 and 2000 mm annually (Thornthwaite 5 – 100). Although Charman (1988) suggested their use for low volume road bases only, they are used in Brazil for up to 1500 vpd and 5 MESA (Villibor, 2006) – their use clearly not being limited to low volume roads. .

In Zimbabwe, the excellent performance of many laterites has been attributed to the hardness, roughness and shape of the particles more than possible improvement in quality through “self-cementation” (van der Merwe, 1971).

MRWA (2002) recommend curing after addition of the compaction moisture to OMC. Typically a low plasticity lateritic gravel requires one day of curing while lateritic sands may require 4 or 5 days (Cocks and Hamory, 1988). This may be done in windrows on the road or in the borrow pit. They also suggest that mixing on the road (using a motor grader) should be carried out to minimize variability. Good compaction at OMC is essential with many of the non-traditional gravels to ensure a strong upper surface.

Laterites have been shown to have high in situ Resilient Modulus (Mr) values (5 GPa from road samples) and> 700 MPa for laboratory samples (Nogami and Villibor, 1991 after Motta et al, 1985).

Grace and Toll (1987) indicate that a high degree of compaction is essential in order to ensure that the laterite retains its strength even when saturated. Cocks and Hamory (1988) confirm this stating that compaction must take place at close to OMC. They also recommend that the top of the base be well shaped to ensure drainage before surfacing, the surface must be well-compacted and firm before sealing and sealing should be delayed (with the base trafficked) until the base has dried out, all cracks are filled and a rough compact surface has developed. This may take 1 to 2 months. The subgrade should also be well-compacted to at least 1 m depth. Cocks and Hamory (1988) indicate that if the material is compacted dry of OMC, the surface is often poor and the strength is not achieved. Slushing is also advised for certain materials in order to achieve a tight, impermeable surface.

Concretionary and hardpan laterites often contain large particles, which when used as base material, need to be broken down with a grid roller to ensure an adequate surface finish. The use of paving machines is usually ineffective as they result in segregation of coarse and fine materials. The high plasticity often seen with these materials also makes the use of paving machines difficult.

It has been noted in Brazil that some laterites are difficult to compact and this was attributed to poor grain-size distributions (ABP, 1976). In some cases precautions need to be taken during compaction. Vibratory, sheepsfoot and pneumatic tyred rollers appear to give most success with the materials generally drier than optimum.

Compacted bases should be dried back to at least 80% of OMC before sealing.

One of the unique characteristics of laterites is the performance in service, even with high plasticity and naturally low CBR values. One of the contributing factors appears to be the uncharacteristically low permeability of unsaturated compacted laterites. Hight et al (1988) indicate that the unsaturated permeabilities are often three to four orders of magnitude lower than the already very low saturated permeabilities (2.5×10^{-7} to 4.5×10^{-8} cm/s). This results in the high suctions in the unsaturated state but means that suction cannot be relied on to maintain high strengths if water is allowed access to the material layer (Hight et al, 1988). However, this risk is mitigated by the very low permeability and resistance to moisture penetration, especially, fine-grained laterites, and the attendant high stiffness at equilibrium moisture content (Grace and Toll, 1987) – typically at or below OMC with reasonable drainage, even in the wet season.

The impact of suction on strength is discussed by Hight et al (1988). The big difference in strength developed through the suction as a result of compaction at OMC and allowing the material to dry back compared with directly compacting the material at its equilibrium moisture content is highlighted. They describe how a well compacted material, even under close to saturated conditions will have a high strength as a result of the tendency to dilate under shear stresses, thus producing a negative pore water pressure.

5.3 Quality Control

In order to achieve the specified requirements in the road, suitable quality control systems need to be in place. Problems with equating the field conditions to the laboratory outputs have been reported (Gidigasu and Mate-Korley, 1984; Gidigasu, 1988b). A specific example is the change in properties during construction where, for instance a laboratory MDD and OMC of 2177 kg/m^3 and 7% respectively became 2288 kg/m^3 and 3.3% in the field after compaction. This makes compaction control difficult. Gidigasu and Mate-Korley (1984) also highlight the problems with defining the field compaction.

It is also known that the presence of excessive iron in soils can affect nuclear density measurements. This is typically given at greater than 35 to 40% iron oxide (not uncommon in African laterites) but it is likely that variations in iron contents at levels lower than these within materials actually placed on the road can cause variations in readings.

Similar problems regarding the determination of moisture contents and Atterberg limits after compaction may be encountered if the drying temperatures are not taken into account.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

There is irrefutable evidence that lateritic materials that do not comply with standard specifications can perform particularly well when used in road construction, even as base course. In order to construct cost-effective roads, particularly those classified as low volume roads, it is essential that maximum use is made of these local materials. This will require a standardized method for their testing and the development of appropriate specifications for their selection.

Extensive research on lateritic materials has been carried out in a number of countries and the science of their use is fairly well advanced. However, it is necessary to assess the test and specification limits that are currently being applied internationally and optimize these for use in sub-Saharan Africa.

Brazil, for instance, has a wide range of innovative and appropriate tests but even these are not used nationally, with only local (regional) use apparently being made. Based on the literature, it is recommended that the Brazilian methods would probably be the first approach but many of these would need to be translated and calibrated for wider use.

It is interesting to note that the need for a high degree of compaction is considered essential by most practitioners. In order to achieve this, it is necessary to have a well graded aggregate and the normal Fuller type particle size distributions are proposed in most specifications. This tends to go against the need to simplify the specifications and material testing. . Grading, of course also affects the compactability and surface finish of the layer. The use of in situ strength instead of the wide range of other material requirements certainly simplifies the material selection and specification process

6.2 Recommendations

In order to optimize the use of laterites in road construction for developing tropical and subtropical countries the following are recommended:

- A standard suite of test methods should be prescribed. These could consist of a combination of existing methods, modifications to these methods or possibly (but unlikely) the development of new methods.

- Comparative laboratory testing of a number of materials will be necessary to identify the most suitable test methods and sample preparation techniques.

- The Brazilian MCT method should be investigated as a standard for wider use when fine grained lateritic soils are being considered.

- A range of materials (from different countries) that are known to perform both poorly and satisfactorily should be obtained and their properties using the available and proposed test methods should be assessed and compared with their performance.

- The roads in South Africa, Malawi, Zimbabwe and Botswana that have been investigated in the past should be used as the nucleus for any further investigations of laterite performance that are to be carried out, with additional in situ, laboratory and performance testing being carried out. These should be supplemented by other roads exhibiting a range of different performances.

- The phenomenon of self-stabilization should be investigated with a view to inducing it in the road and being able to predict it reliably. As a start, a full literature review of the subject should be carried out.

- The relationship between laboratory DN values and various standard properties of laterites should be investigated in order to optimize the use of the DCP design foe evaluating lateritic materials and controlling construction.

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