TESTING OF LATERITIC MATERIALS FOR ROAD PAVEMENTS

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ABSTRACT

Laterites or lateritic soils are widespread in many countries internationally, including South Africa. These are often the only materials available within a reasonable haulage distance for road construction, but when tested in typical road material laboratories, they fall outside the limits of standard specifications for certain road layers and are rejected for use. However, experience has proved that many of these lateritic materials can be successfully used in low-volume (and even relatively high-volume) roads. Investigations have shown that the material properties determined using conventional test methods produce inaccurate results, often leading to the rejection of the materials. This paper reviews and discusses some of the unique properties of laterites and their influence on laboratory testing using conventional test methods. The use of the Dynamic Cone Penetrometer to assess the shear strength of laterites is proposed to eliminate many of these problems.

1. INTRODUCTION

In many sub-Saharan countries laterites are the only readily available construction material. However, testing of these laterites in normal road materials testing laboratories frequently produces results outside the necessary specifications, and the materials are rejected for use in upper structural layers of roads. Although it is known that the problem is related more to the test methods than the actual properties of the laterites, little differentiation between laterites and other materials is carried out in laboratories and laterites are tested according to the standard specifications.

Experience in several laboratories has shown that conventional test methods can produce misleading results that discriminate against using laterites. The problems related to the unique mode of formation and classification of laterites and their lack of compliance with traditional specifications are discussed in a previous paper (Paige-Green et al., 2015).

This paper highlights some of the problems associated with the testing of laterites and proposes possible causes as to why laterites do not comply with most specifications and yet perform well in road structures and should be read together with Paige-Green et al., 2015.

The term “laterite” or “laterite material” as used in this paper includes what are called laterites or ferricretes in geology and engineering and plinthite in pedology in southern Africa and does not infer any particular mode of formation.

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2. GEOTECHNICAL PROPERTIES AND TESTING

2.1 Properties

The geotechnical properties of lateritic and other pedogenic materials generally depend on three factors (Netterberg, 1969, 1985, 1994):

- 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 e.g. sesquioxide in laterites and carbonates in calcretes.

During their 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, 1969, 1971, 1985, 1994); various authors cited in Gidigasu, 1976, Morin & Todor, 1976). These particles or aggregations may or may not be broken down during laboratory testing and during construction. This affects the fundamental properties of the material in terms of, for example, grading and Atterberg limits. Moreover, both the clay mineral types and the cementing and replacing minerals are generally different from the minerals typically found in weathered materials in the temperate zone soils, which typically consist 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 1.

Table 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: typically comprise 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 the 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 with 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 potential 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 (e.g. LNEC et al, 1959, 1969; DNER, 2007), compared with the more traditional specifications such as those of AASHTO M147 (2011).

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

2.2 Testing

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

- Particle size distribution;
- Atterberg Limits and Shrinkage;
- Strength of the coarse particles, and
- Compaction and bearing strength, e.g. CBR and swell.

Another aspect of lateritic materials that may be relevant is the concept of self-hardening or a time-dependent improvement in their performance. 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. However, this property cannot be assumed to be present and needs to be quantified prior to relying on its presence.

2.3 Particle Size Distribution

Grading analyses are only applicable to the more immature types of laterite, such as relatively loose or soft soils like soft plinthite and nodular laterite in their natural state. Other varieties, which occur as boulder, hardpan or honeycomb laterite, are too coarse or occur as indurated horizons that 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 an average over the full range of particle sizes. This conversion may be misleading for nodular laterites, whose coarse fraction is iron-rich and whose fine fraction is kaolinitic. The coarse fraction usually has a specific gravity (relative density, RD) between 3.0 and 3.5 (and sometimes much higher), while the specific gravity of the fine fraction is about 2.7. The particle size distribution curve specified is based on proportions by mass retained between successive sieves, and only represents a particular packing arrangement for a soil of constant BRD. Given this possible variation in BRD, a conventionally calculated test applied, for example, to nodular laterites would underestimate the volume content of coarser particles, exaggerate any gap-grading in the material, and would not represent the true packing and mechanical stability of the compacted material as a whole.

Therefore, when using grading analyses, it is important to inspect the material, assess its composition and decide if separate BRD determinations of the fine and coarse fractions should be made. If the BRDs 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 considerably improved (Netterberg, 1985, 1994).

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 fracture the coarse particles. It is equally important that the fines adhering to the coarse particles be separated. Therefore, it is 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.

Similar problems related to the variable particle density and variable disaggregation would also affect any sedimentation analyses conducted on the fine fractions.

2.4 Apparent Particle Density

Only limited data on the Apparent Relative Density (ARD) of sub-Saharan laterites appear to have been published. Values for 28 laterites on a worldwide basis (particle size fraction not stated) range between 2.67 and 3.46, with a mean of 3.06 (Krinitsky et al., 1976). Values of specific gravity of 2.7 to 2.9 for the fraction passing 2.00 mm of 15 west African and Indian laterite gravels in comparison with 2.9-3.5 for the whole soil were quoted by Gidigasu (1976), although a range of 2.2- 4.6 was found for the passing 2.00 mm fraction of 38 other Indian laterites. 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 varies with particle size, with the fines being the lowest (Gidigasu 1976; Charman 1988). The ARD of laterites, which is higher than the 2.65 usually assumed for most soils, is significant. It is often not 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 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, resulting in an unstable grading. Thus, in such cases it is necessary to determine the BRD of the relevant particle size fractions and to allow for these differences.

2.5 Atterberg Limits, Shrinkage and Swelling

Atterberg Limits (PI and LL), and 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 several 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 based on a visual assessment of its homogeneity to ensure that each stockpile is as homogenous as possible for testing purposes.

(2) Sensitivity to the 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 & Todor, 1976; Sharp et al, 2001). Some examples of the effect of remoulding are shown in Table 2, which suggest that mechanical processing (e.g. excavating, grading, compacting) of an in-situ laterite may actually degrade its engineering properties.

Table 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 the properties of many laterites. Drying is usually used 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. Typically, the Atterberg Limits are lowered after oven drying compared with air drying. Hight et al. (1988) 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 between 10 and 15 after oven drying. Lyon Associates Inc (1971) reports decreases in the PI of more than 40% on oven drying of the samples (e.g. 33 as-received sample and 19 after 24 h drying at 105°C).

Given the above findings, the form of drying employed in the laboratory should preferably represent the conditions that will apply in the field. Thus, laterites should generally be tested at their natural moisture content (Morin & Todor, 1976) or after air drying (LNEC et al, 1969, Morin & Todor, 1976). Brazilian practice requires air or oven drying at not more than 60ºC for all soils unless it can be shown that the soil is not affected by drying at a higher temperature. Similarly, MRWA (2002) requires 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 for road materials in South Africa

TMH1Method A1(a) (NITRR, 1986), which is to boild and oven dry the soil fines at 105 to 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, reducing the clay's ability to absorb water, thus effectively reducing the measured plasticity (Lyon Associates, 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 (1986) whilst Lyon Associates Inc (1971) suggest that a standard mixing time of 5 minutes (half of the South African and BS 1377 standards of 10 minutes) should be rigorously adhered to.

(5) Difference between BS 1377 and AASHTO T89 and ASTM D42/D4318 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 & Netterberg, 1984). The AASHTO/ASTM apparatus 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.

Given 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; 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 & Ventura, 1999).

(6) Plots of Atterberg Limits on Casagrande Plasticity Chart: Data for laterites on a world basis (Nixon & Skipp, 1957; Gidigasu, 1976; Morin & Todor, 1976) are somewhat contradictory, but it seems clear that laterites and lateritic soils as a group plot on both sides of the A-line (Mitchell & Sitar, 1982). Lateritic soils that plot below the A-line are likely to be troublesome (Gidigasu, 1976) as they might contain hydrated halloysite (exhibits unusual geotechnical properties). Figure 1 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 (Lyon Associates Inc, 1971).

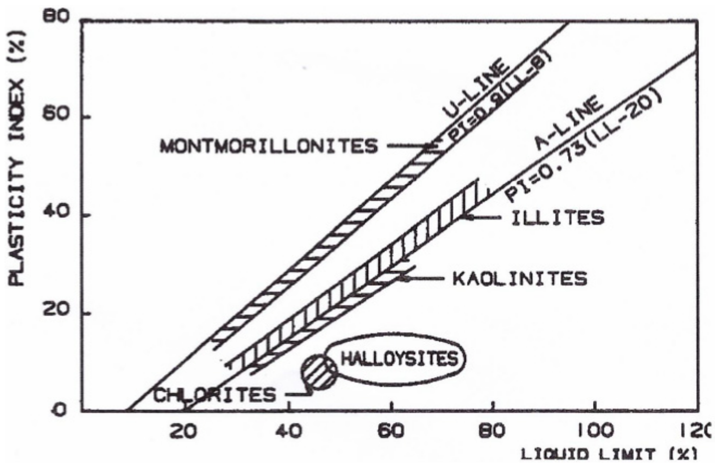

Figure 1: Location of common clay minerals on Casagrande's plasticity chart (Lyon Associates Inc 1971)

(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 determined on the fraction passing 2 μm) and might thus provide a more convenient alternative to this ratio.

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

2.6 Silica/Sesquioxide Ratio

A maximum silica sesquioxide ratio (S/R) of 2.0 has been used both to define a laterite and as part of the specification for its use as well as for the specification of their use (e.g. LNEC et al 1959, 1969; DNIT, 2007). A maximum value of 2.0 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.

A wide range of S/R values is found in the literature, but it would appear that not all values are derived using the same formula: some such as van der Merwe and Bate (1971) 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 (Equation 1).

The correct silica sesquioxide ratio (S/R) is calculated as follows (DNER, 2007):

$$\frac{S}{R} = \frac{\frac{SiO_2}{60}}{\frac{Al_2O_3}{102} + \frac{Fe_2O_3}{160}} \qquad (1)$$

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.

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 & 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) and 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 five samples giving ICP results higher than XRD/XRF and 8 giving lower results. They attributed the differences to the extent that the wet chemistry dissolves the oxides.

Brazil has a standard test method (DNER-ME 030-94) based purely on wet chemistry to extract and determine the components using titrations. Modern techniques such as

Inductively Coupled Plasma (ICP) mass spectroscopy or atomic absorption analyses should probably be used to replace the quantification portion of the Brazil analyses, retaining the extraction techniques, which have a sound basis.

Other than in Angola and Brazil, the S/R ratio has not been fully introduced into road material specification as it is not routinely carried out in standard engineering soils laboratories, is time-consuming, and is costly (De Graft-Johnson, 1975), and is not a requirement in the latest edition of Road Note 31 (Transport Research Laboratory; Undated) despite it being based upon studies which included laterite-based roads and intended for tropical and subtropical areas..

2.7 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 several workers that each point on the compaction curve and for CBR testing must be done on new material (i.e., materials should not be reused as their properties change) (Lyon Associates Inc, 1971; LNEC, 1959). It is also noted that 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, several authorities specify testing in the unsoaked condition only (De Graft-Johnson, 1975; Madu, 1975; Aggarwal & Jafri, 1987), but none seem to explicitly state that the materials should be allowed to equilibrate (condition) after compaction. Recent southern African experience (e.g. Netterberg, 2023), indicates that about four days between compaction and testing is necessary to allow relaxation of some of the compaction stresses and dissipation of pore pressures before testing.

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 shown in Equations 2 and 3 (De Graft Johnson et al., 1972; in Gidigasu, 1976).

$$CBR = 72.5 \log10 \frac{MDD}{16PI} - 7.5\ \% \text{ where MDD is in } kgm^{-3}\ (r = 0.68). \qquad (2)$$

The MDD and OMC of African lateritic soils are related as follows (Morin and Todor, 1976):

$$MDD = 2563 - 44.5\ OMC\ (kgm^{-3})\ (r = -0.84,\ SD = \pm 88\ kgm^{-3},\ n=81 \qquad (3)$$

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

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

All these factors being equal, South African CBRs of uncemented materials are on average about 80 % of those of most other countries due to our exclusive use of the CBR at a penetration of 2.54 mm, whereas the 5.08 mm CBR (which is usually higher) is used by most other countries (Pinard & Netterberg, 2012).

2.8 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. However, these properties can assist in understanding the behaviour of materials under loading conditions on 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 is assessed at various moisture and density conditions. It is similar to the Texas Triaxial Test which is used in a number of countries and states for routine pavement design (e.g., Zimbabwe, Texas, and various states in Australia).

2.9 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 (Latin *later*, brick), now called plinthite (Greek *plinthos*, brick) (e.g. Soil Survey Staff, 1994; FAO-Unesco, 1997) in pedology, all of which, by definition, harden irreversibly to a hardpan or to irregular aggregates after repeated wetting and drying, is just such a material. Alexander and Cady (1962) also stated that some laterites, when wetted and dried, harden with time due to 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 research of potential self-stabilizing materials.

Plinthic horizons occur for example in some oxisols (the most intensely weathered of all soils) in Soil Taxonomy (1994); and some ferrasols, and in all plinthosols in the FAO-Unesco (1997) system. In the South African soil classification system (Soil Classification Working Group, 1991; Fey, 2010) plinthite is **not** required to be self-hardening and is described either as soft (can be cut with a spade when wet - e.g. a nodular gravel) or hard cannot (a continuous hardpan). Hard plinthite occurs in such soil forms as Wasbank and Glencoe, soft plinthite in Longlands, Avalon and Bainsvlei forms.

Although MacVicar et al. (1977) doubted that self-hardening plinthite occurs in South Africa, Du Toit (1954:451) 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.

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.

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 (Rossouw, 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 & 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 wetting and drying cycles. (DaSilva et al, 1967; Novais-Ferreira & Meireles, 1969; Netterberg, 1969, 1971; 1975; Van der Merwe & 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 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 pre-drying during construction before priming and the associated irreversible hardening of caliche (calcrete) 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 (1990a & 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, 1990b) shows that the main components of the “laterites” were quartz and feldspar and that only traces of goethite were present. The greatest increase that was 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 the soaked CBR should be regarded as indicating a significant potential for self-cementation.

Further research on cycled CBR tests 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 take place during weathering, and the softening of laterite crusts occurs on reforestation (Maignien, 1966; Morin & 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. Still, under reducing conditions such as those that 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 (1965). Significant increases in CBR strength were obtained after 10 CBR cycles (from about 38 to between 48 and 67). Netterberg (1975) concluded that self-stabilization seemed likely to occur in practice, that the cycled CBR test appeared to be a promising indicator of the potential for self-stabilization and has since summarised the proposed test methods for it (Netterberg, 2023)

2.10 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 taken as synonymous with durability. This is not necessarily the case. The Los Angeles Abrasion test is probably most commonly (and incorrectly) used as an indicator of durability most commonly (Ruenkrairergsa, 1987).

The Aggregate Impact Value (AIV) test appears to be the most commonly used test in African countries for the estimation and specification of the strength of aggregate particles (De Graft Johnson et al., 1972; Gidigasu & Dogbey, 1980; Charman, 1988). This is a simple, cheap test and is recommended when a knowledge of particle strength is necessary. The Treton is a similar test used in South Africa.

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 1 000 being determined on a number of roads investigated that had performed well (Paige-Green, 1999).

2.11 Dynamic Cone Penetrometer (DCP) DN Values

Because of the inherent problems associated with testing laterites and the fact that many of the tests are not 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 is well-quantified statistically. It is also strongly correlated with the shear strength of a material, as determined by the Repeated Load Triaxial Test (Ayers et al, 1989). The DCP DN test 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). A standard test and analysis technique has been developed for the laboratory test (Pinard and Hongve, 2020), but this should be developed into a protocol specifically for the use of lateritic materials.

3. CONCLUSIONS

There is no doubt 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 maximise their use, standardised methods for their testing need to be developed.

Brazil 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 test methods would probably be the first approach for both laterite gravels and fine-grained lateritic soils.

One way of minimising the wide range of problems with the testing of laterites is to concentrate on developing tests such as the DCP DN value, which measures the actual in situ or laboratory strength at any required moisture/density combination and after subjecting the material to typical environmental variations that may prevail in the field.

However, because of the importance of ensuring that the material is actually a laterite before ignoring traditional test methods and specifications it is essential to standardize on a test procedure and specification that uniquely classifies the material as a laterite. The silica/sesquioxide ratio is probably the best test for this but needs to be modernised and simplified.

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