



University of Pretoria

ENGINEERING GEOLOGICAL EVALUATION OF TRANSPORTED TROPICAL RED SOILS

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ENGINEERING GEOLOGICAL EVALUATION OF TRANSPORTED TROPICAL RED SOILS

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ABSTRACT

Tropical soils include a wide variety of pedogenic materials common to the tropical regions of the world, of which the material properties and engineering behaviour may vary significantly. These soils in many ways have unique characteristics that can mainly be ascribed to the compositions and microstructures of a material developed under hot, wet soil-forming conditions. Due to the variation in their properties and behaviour, the classification of soils formed under such conditions as distinct and well-defined soil types appears to have been avoided in the past. Unfortunately, this has brought about the grouping and classification of materials of significantly varying chemical, geological and/or geotechnical properties and engineering behaviour under a single, all-encompassing term of “tropical red soils”. The unique pedogenic environment and soil characteristics further mean that conventional soil mechanics concepts and geotechnical investigative procedures, developed almost exclusively from work completed on temperate soils, do not necessarily apply to soils formed in tropical environments.

This research reviews the typical and unique characteristics and behaviour of tropical red soils as summarised in the existing literature, including the application of existing soil classification systems and the possible ambiguity existing in the classification and nomenclature pertaining to soils formed in tropical environments. Significant inconsistencies in the identification and classification of tropical red soils and laterites were observed to exist throughout the available literature, with frequently overlapping ranges of geological, geotechnical and chemical properties reported for both lateritic and tropical red soil types. Much has been written in the existing literature regarding the unique material characteristics and engineering behaviour of tropical red soils and the inapplicability of conventional soil mechanics concepts. The majority of the available results, conclusions and recommendations in the existing literature refer to residual tropical red soils only, with very little information regarding the characteristics and behaviour of transported tropical red soils being reported.

The characteristics and behaviour of a number of transported tropical red soil horizons, from a selected area in the Mayoko region of the Republic of the Congo, were evaluated through conventional and specialised testing regimes during the completion of this study. A large number of disturbed and undisturbed samples were collected from the study area and submitted to geotechnical laboratories in South Africa to establish the chemical-, mineralogical-, index and mechanical properties of these soils. The test results, sample preparation and test procedures and effect of these on the engineering behaviour of the material were evaluated and compared to the typical results reported for residual tropical red soils in the literature. A number of the findings from this study, particularly the sensitivity of tropical red soils to change in moisture content and effect of the dispersing agent on the reported particle size distribution of such soils, confirmed the findings which have been reported in the literature for residual tropical red soils. However, the transported tropical red soils were also found to differ markedly from residual tropical red soils in certain aspects, most notably in terms of the apparent effect of the lack of remnant structure on the material mechanical properties, permeability, sample disturbance and relevance of tests completed on remoulded test specimens.

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1. General Introduction

Tropical soils in many ways have unique characteristics that can mainly be attributed to the compositions and micro-structures of material developed under humid soil-forming conditions. Most of these soils contain abundant iron and aluminium oxides due to the rapid breakdown of feldspars and ferromagnesian minerals, the removal of silica and bases and the concentration of iron and aluminium oxides or sesquioxides. Due to the high iron content and oxidation processes, these soils are more often than not red in colour.

The chemical alteration of primary minerals under widely varying degrees of weathering intensity will produce clay minerals that vary in both their chemical and physical properties. The primary weathering processes are characterized by the rapid breakdown of feldspars and ferromagnesian minerals, the removal of silica and bases and the concentration of iron and aluminium oxides. Different constituent elements are released into solution according to their solubility and resistance to weathering and are transported downward through the soil profile through percolating soil moisture (Scheffer and Schachtsabel, 1998). The overall decrease in percentage bases and silica content leads to the relative accumulation of weathering-resistant and stable iron and aluminium oxides and hydroxides such as goethite (FeOOH), hematite (Fe_2O_3) and gibbsite ($\text{Al}(\text{OH})_3$) (Hintermaier-Erhard and Zech, 1997). The distinctive red colour of many tropical soils can be attributed to the development of hematite in the material. Tropical red soils in this study refer to all non-lateritic tropical soils other than tropical “black” soils and which in reality may vary in colour (between shades of red, orange, yellow and brown), microstructure, mineralogy and grain size distribution.

Due to the fact that soil formation in the tropics is a highly variable and often complex process, the classification of soils formed under such conditions as distinct and well-defined soil types appears to have been avoided in the past. Much of the difficulty would also appear to stem from the inapplicability of ‘conventional’ soil mechanical concepts, developed almost exclusively from work on temperate soils, when applied directly to certain soils from tropical zones (Northmore *et al.*, 1992). Tropical red soils are further also frequently recorded and loosely referred to in the literature as either ‘laterites’, ‘laterite soils’, ‘latosols’ or ‘lateritic soils’. This approach has led to the grouping of materials of largely varying characteristics and engineering behaviour under a single, all-encompassing term, resulting in the categorization of materials that may have significantly different chemical, geological and/or geotechnical properties as a single soil type, namely “tropical red soils”. Often no distinction is made between reporting the origin of the material to which the results pertain and so distinguishing between the results of residual and transported tropical red soils. Where specified however, the majority of available test results pertain to residual soils.

Much has been written about the material characteristics and engineering behaviour of tropical soils and the sensitivity of these to the influence of test methods on the subsequent test results. The challenges associated with conventional index and strength tests for the identification and engineering classification of tropical soils as discussed in the literature exist primarily because of the dependency of such tests on the sample preparation process and the influence of the subsequent variation of the natural soil structure which occurs during preparation. The usefulness of conventional index and strength tests for the identification and engineering classification of

tropical soils has as such been questioned in the past, primarily because of the dependence of such tests on the sample preparation process and the variation of the natural soil structure which occurs during preparation. However, it has been shown that the unique engineering properties of tropical soils may be very useful for identifying and ‘grouping’ soils which are likely to possess distinctive or similar physical characteristics for specific or local application (Wesley, 1988).

Conventional engineering classification tests are largely concerned with the determination of the index properties of soils. Index property tests were developed for temperate soils and are generally sufficient for these purposes. The properties can be easily assessed, such as their particle size distribution, plasticity limits or density through the test procedures as defined in the British and ASTM standards. These index tests are carried out on disturbed or remoulded soil samples and are thus independent of the *in-situ* characteristics of the soil (Northmore *et al.*, 1992). It can therefore be argued that, by regarding the particle size and consistency limits as the basis of soil classification, the influence of mineralogy, chemistry and origin of a soil on its mechanical behaviour is adequately measured by these simple index tests (Schofield and Wroth, 1968).

This cannot necessarily be assumed to be the case for tropical red soils. The inherent mineralogical and chemical composition and structure (in the case of residual soils) of the material has a pronounced influence on the engineering behaviour and measurement of the index and mechanical properties of tropical red soils. Because of the mineralogical composition of tropical red soils, even partial air-drying at ambient laboratory temperatures may change the structure and physical properties of these soils. Some of these changes cannot be reversed when the soil is re-mixed with water. These structural changes are reflected in sometimes drastic changes in the index properties derived from plasticity, shrinkage and particle size tests or specific gravity. Conventional laboratory testing procedures, as defined in the British and ASTM standards, are therefore not necessarily applicable to tropical red soils without some modification or change in focus (Northmore *et al.*, 1992) to take into account the unique mineralogical and chemical composition and structure.

A detailed literature review was completed of the general characteristics and geotechnical properties of tropical red soils in an effort to evaluate and review the unique variations in the physical and chemical characteristics and subsequent engineering behaviour of tropical red soils compared to soils from more temperate climatic regions. An in-depth geotechnical investigation of a number of tropical red soil horizons of colluvial origin and identified in a selected area in the tropics is presented in the form of a case study, with the results and evaluations from the study compared to the findings and conclusions summarised from existing literature on tropical red soils.

The initial part of this thesis (Chapters 3 and 4) is a review of the researched physical and chemical characteristics and established engineering behaviour of a number of residual tropical red soils in the literature, as well as providing an overview of the selected study site and method of investigation. Chapters 5 to 8 quantify and evaluate the established characteristics of the tropical red soils covering the study area. The identification and evaluation process of the engineering characteristics of the material and any variations to standard test procedures, making provision for the differences in material properties of tropical soils, are discussed in detail. The

variation in soil suction pressures with a change in natural moisture content is investigated and the results obtained using two different methods of determination compared, as well as providing conclusions on the findings and results of the research completed in the study area against those detailed in the existing literature.

2. Overall Aim and Specific Objectives

In order to successfully evaluate and review the unique variations in the physical and chemical characteristics and subsequent engineering behaviour of tropical red soils, the following specific objectives were set:

- Investigate and review the physical and chemical characteristics and subsequent engineering behaviour of the tropical red soils summarised in existing literature, the shortcomings of existing soil classification systems in the classification of these soils and the possible ambiguity existing in the classification of and nomenclature pertaining to soils formed in tropical environments in the available literature;
- Investigate the development and distribution of the transported tropical soils in the study area in relation to origin, topography and mean elevation;
- Establish the chemical-, mineralogical-, index and mechanical properties of the transported tropical red soils in the case study area and investigate the effect of these properties on the established geotechnical properties and engineering behaviour determined through standard and/or specialised testing regimes. It should be emphasized that it is not the aim of this study to critically evaluate and comment on the suitability of specific tests and/or test procedures. Instead, wherever standard test procedures have been adapted to take into account the unique properties and behaviour of tropical soils, all variations are based on the findings and recommendations for the testing of residual tropical soils summarised in the existing literature;
- Determine the variation in these characteristics of the transported tropical red soils in the study area compared to those established and summarised in the literature for tropical red residual soils;
- Evaluate and correlate the soil suction pressure values which are determined using the traditional Filter Paper method with the values determined from the Soil Water Characteristic Curves;
- Provide recommendations for future geotechnical studies in the tropics based on the findings and conclusions of this study.

3. A Review of Tropical Red Soils

The aims and objectives outlined in this research first of all implies that tropical soils are sufficiently distinctive and unique to be considered different from soils in more temperate climatic regions. Tropical soils in many ways have unique characteristics that can mainly be attributed to the compositions and micro-structures of a material developed under hot, wet soil-forming conditions. This can be directly attributed to the chemical and mineralogical changes through weathering, as well as the change in physical characteristics. Geology, climate and topography are some of the major influencing factors in tropical soil formation and the subsequent properties of these materials. Keeping this in mind, it is possible to focus specifically on the role of geological and pedological maps as valuable sources of information to the engineer and geologist who want to identify the expected soil conditions in a specific region. However, the categorization of soils according to classification systems or parent rock types provides the engineer with very little or no indication of the expected geotechnical and engineering behaviour of the material, and the classification of tropical soils based on a number of their typical physical and chemical characteristics is therefore also discussed.

3.1 Soil Formation in the Tropics

Formally, tropical soils are broadly defined as “soils found in the vicinity and overlap the two tropics of Cancer and Capricorn” (Igwe, 2011), as illustrated in Figure 3-1.

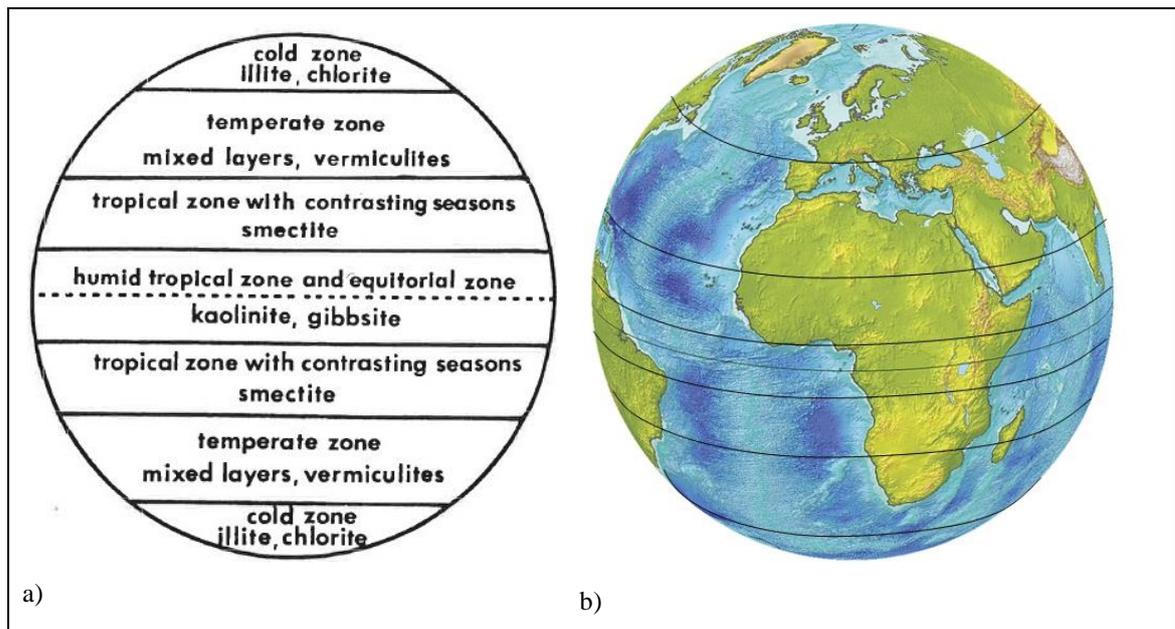


Figure 3-1 a) Effect of climate on clay mineral occurrence (Millot, 1979) and b) fitted to globe by the author

From the literature, soil formation or pedogenesis can be summarised as being predominantly driven by the following five factors:

- Parent material;
- Climate;
- Topography;
- Biological Factors and

- Time.

Pedogenesis takes place through either mechanical or chemical weathering processes of the parent material, with the prevailing weathering process essentially being governed by the climate. In humid tropical conditions, with high annual rainfall and high temperatures, this will take place through intense chemical weathering (Figure 3-2). The intensity of chemical weathering is controlled mainly by the amount, intensity and distribution of the annual rainfall and by the length of time the primary minerals have been exposed to the weathering process (Sherman, 1949).

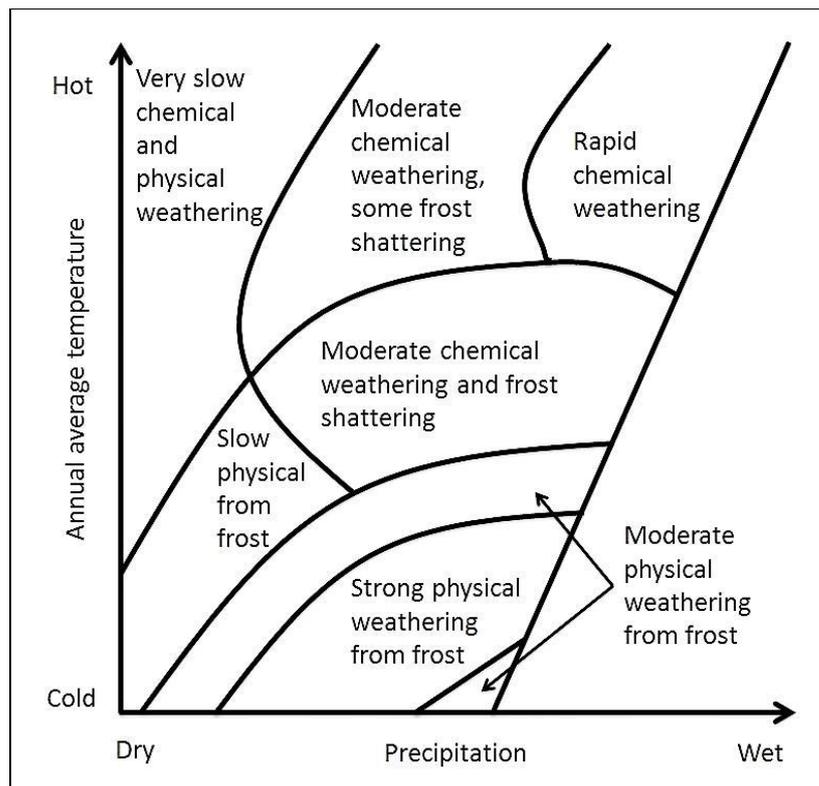


Figure 3-2 Influence of climate on weathering (Scharcks, 2013)

3.2.1 Chemical and Mineralogical Changes During Weathering

The chemical alteration of primary minerals under widely varying degrees of weathering intensity will produce clay minerals that vary in both their chemical and physical properties. The primary weathering process is characterized by the rapid breakdown of feldspars and ferromagnesian minerals, the removal of silica and bases and the concentration of iron and aluminium oxides. Different constituent elements are released into solution according to their solubility product and resistance to weathering (Figure 3-3) and are transported downward through the soil profile by percolating soil moisture (Scheffer and Schachtsabel, 1998). The overall decrease in percentage bases and silica leads to the relative accumulation of weathering-resistant and stable iron and aluminium oxides and hydroxides such as goethite (FeOOH), hematite (Fe_2O_3) and gibbsite ($\text{Al}(\text{OH})_3$) (Hintermaier-Erhard and Zech, 1997). The distinctive red colour of many tropical soils (Figure 3-4) can be attributed to the development of hematite in the material. Tropical red soils as the focus of this study refer to all non-lateritic tropical soils

other than tropical “black” soils and which in reality may vary in colour (between shades of red, orange, yellow and brown), microstructure, mineralogy and grain size distribution.

Some of the most common unweathered primary minerals identified from studies by Gidigasu (1974) of West African tropical red soils include quartz, garnet, staurolite, microcline, hornblende and zircon.

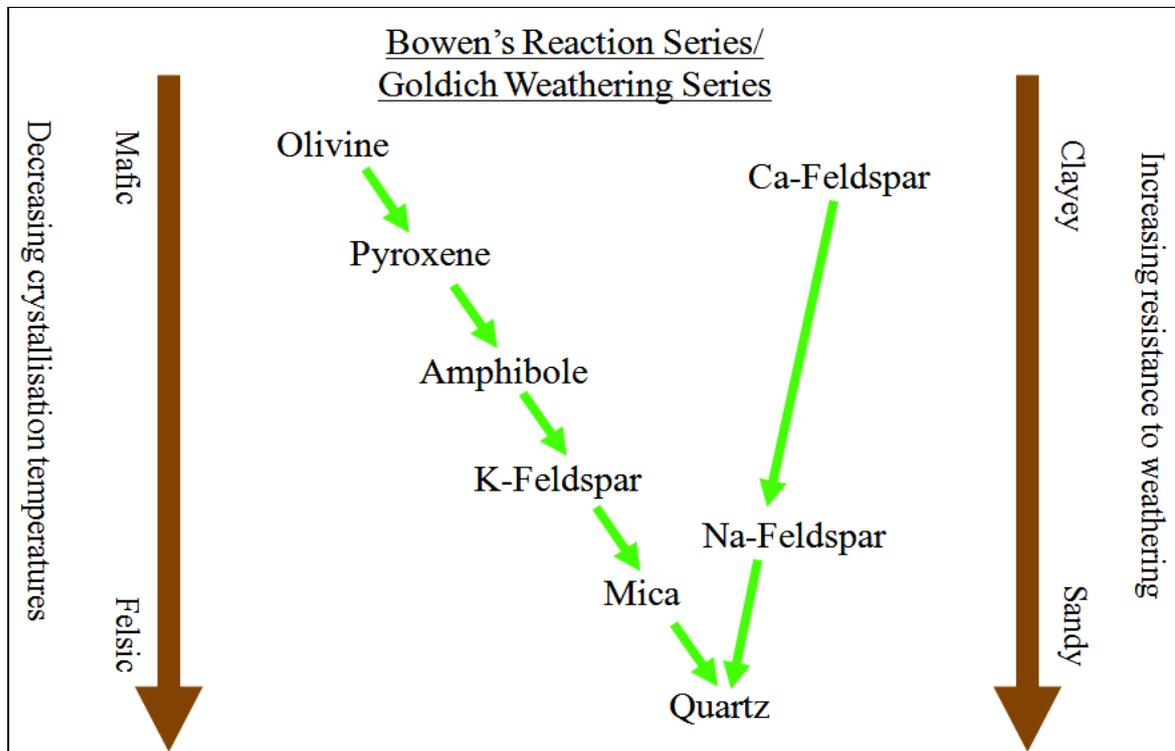


Figure 3-3 Summary of the Goldich Weathering Series (adapted from Goldich, 1938)

The development and formation of secondary minerals in the material occurs concurrently with the weathering of primary minerals. Important elements for the formation of many clay mineral types include Ca, Na, Mg, K and Si. With abundant rainfall and good drainage, the leaching-induced low concentrations or total absence of these elements lead to the continuing development of the low activity clay minerals such as kaolinite ($Al_2Si_2O_5(OH)_4$).



Figure 3-4 Distinctive red colouration typically associated with tropical soil development

Low-activity clay mineral development is not restricted to the tropics only, but from literature it is evident that the frequency of low-activity clay soils increase with proximity to the equator. A typical sequence of clay mineral development in a tropical environment (from volcanic ashes in Kenya) is summarised in Figure 3-5 below.

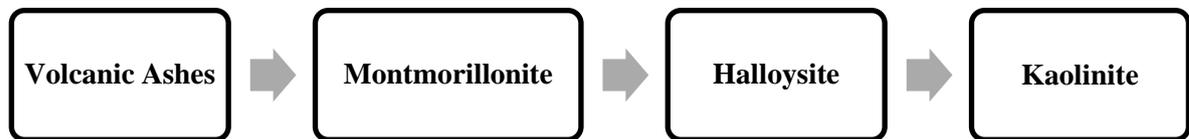


Figure 3-5 Different stages of volcanic clay mineral development (adapted from Gonzalez de Valejo *et al.*, 1981)

Low activity clays are typically characterized by an activity (Equation 3-1) of one or less. Table 3-1 below lists the typical activities of common soil minerals (Uehara, 1982).

$$Activity = Plasticity\ Index / \% Clay$$

(Eq. 3-1)

Table 3-1 Typical activity of common soils minerals (Skempton, 1953)

Clay Mineral	Activity
Quartz	0.0
Muscovite	0.18
Kaolinite	0.33-0.46
Illite	0.90
Montmorillonite (Ca)	1.5
Montmorillonite (Na)	7.2

Due to the high specific gravity associated with iron bearing minerals, the specific gravity of tropical soils may further be a useful indicator of the stage of pedogenesis and laterization (Table 3-2).

Table 3-2 Typical specific gravity values of common minerals (adapted from Mitchell and Sitar, 1982)

Primary Minerals	Specific Gravity
Orthoclase	2.5-2.6
Plagioclase	2.6-2.75
Quartz	2.65
Hornblende	2.9-3.3
Secondary Minerals	Specific Gravity
Kaolinite	2.2-2.6
Gibbsite	2.4
Goethite	3.3-3.5
Hematite	4.9-5.3

The secondary mineral development process can further be considered as a function of the position of the particular profile in the topography. For example, Nye (1955) has noted the concentration of kaolinite increases in well-drained soils near the crest and upper slopes of the topography, whereas montmorillonite is more likely to occur in the poorly drained lower slope and valley soils. These findings are summarised in Table 3-3 below.

Table 3-3 Mineral composition of the clay size fraction in a number of West African soil samples in relation to topographic position (adapted from Nye, 1955)

Depth (m)	Topographic Position	Mica (%)	Kaolinite (%)	Montmorillonite (%)
0.30-0.60	Upper middle slope	9	91	0
0.60-1.20		9	91	0
1.20-2.40		3	97	0
0.45-1.80	Lower middle slope	12	88	0
1.80-2.40		3	97	0
2.40-2.70		4	53	43
1.20-2.10	Lower slope	9	91	0
2.10-2.30		14	72	14
2.30-2.60		18	25	57
0.60-0.90	Valley	8	84	8
1.20-2.10		4	23	73
2.10-2.30		6	29	65
2.70-3.35		13	13	74

Northmore *et al.* (1992) have also recognised during their detailed mineralogical analysis of tropical red residual soils the presence of amorphous aluminium silicate gels, the presence of which appears to be restricted to constantly humid, well-drained environments. Optical and electron microscope studies showed that the ‘clay clusters’ are usually not “cemented” by iron oxide coatings, but at least partly bonded by amorphous gel, which sometimes coats entire pedes or develops as isolated areas at the ped contacts. In other parts of the same soil, individual pedes are in direct contact, which, in the absence of amorphous gel or iron oxide cement, are apparently joined by some form of ‘clay-like’ bond.

3.2.2 Leaching

Leaching of the dissolved primary minerals from the soil profile is vital in the development process of tropical red soils. Removal of bases and combined silica leads to further decomposition and the breakdown of aluminium silicate clay minerals and formation of hydroxides and sesquioxides (Loughnan, 1969). Vargas (1953) noted the influence of leaching on the vertical variation in geotechnical properties of residual soil profiles in Brazil. Leaching was found to be responsible for a major increase in the porosity of the material, in many cases to such an extent where the volume of voids comprised up to 60% of the total volume. This, in conjunction with low moisture contents, may lead to the development of highly compressible soil layers.

The Cation Exchange Capacity (CEC) of tropical soils is typically low, due to the predominance of so-called low activity clays such as kaolinite and a lack of 2:1 phyllosilicates (Marques *et al.*, 2004). Further, the CEC is a useful indication of the potential activity of a material. Low activity clays must have per definition a CEC of 24 milliequivalents or less per 100g of clay. This implies that these clays must have low specific surfaces, low surface charge densities or both. The CEC of clay minerals can be expressed as a function of the specific surface area per unit mass of the material and the surface charge density per square centimetre (Equation 3-2).

Table 3-4 summarises the typical values for montmorillonite, allophone and kaolinite. For a given quantity of clay, the activity of the clay increases with increasing surface charge density and specific surface.

$$CEC = Ss\bar{O} \quad (\text{Eq. 3-2})$$

Where:

CEC = Cation Exchange Capacity

Ss = specific surface in cm² per gram of soil

\bar{O} = surface charge density in charge per cm²

Table 3-4 Typical Cation Exchange Capacity, Specific Surface and Surface Charge Density for three common clay minerals (adapted from Uehara, 1982)

Mineral	Cation Exchange Capacity (me/100g)	Specific Surface (cm ² / g)	Surface Charge Density (me / cm ²)
Montmorillonite	90	750	12
Allophane	143	1000	14.3
Kaolinite	5	35	14.3

The strong leaching and depletion of bases due to the high annual rainfall associated with tropical regions leads to acidic conditions within tropical soils. At low pH values, aluminium contained in soil minerals becomes soluble, resulting in a concentration increase of Al³⁺ in the soil solution which can have toxic effects on plants and soil organisms. Under dense vegetation, such as tropical rainforest, this effect is often not as marked because of the rapid uptake of basic nutrients (K⁺, Ca²⁺, Mg²⁺) by the plant root system, causing higher concentrations of these elements in the upper horizon and therefore higher pH levels (Hintermaier-Erhard and Zech, 1997).

3.2.3 Colour and Structural Development

Pedogenesis in tropical environments leads to the development of soil structures that in many ways can be considered diagnostic of tropical soils. The colour of tropical red soils vary with the degree of weathering and the hydration of sesquioxides. For example, ferruginous materials change from red to brown with increasing weathering and age and soils in poorly drained areas often show mottles or streaks (Gidigas, 1974). The structure of fine-grained materials is considered to be variable, depending largely on the nature of the parent rock, pedogenesis and degree of sesquioxide enrichment (Alexander and Cady, 1962). The combined action of continuous leaching and cementation leads to the development of highly porous soil structures (Gidigas, 1974). In addition, Gonzalez de Valejo *et al.*, (1981) identified and classified a number of characteristic aggregation structures of andosols from the Canary Islands according to the system proposed by Collins and McGown (1974).

- a. Regular – Compacted, uncemented groups of spheroidal particles.
- b. Interweaving – Flow-like and orientated particles of halloysite.
- c. Clay Matrix – Randomly orientated compact structures.
- d. Oolitic – Characteristic of iron-oxide rich environment.

Table 3-5 below summarises the general morphological characteristics of materials identified in typical West African soil profiles (Gidigas, 1974).

Table 3-5 Morphological characteristics of materials identified in West African soil profiles (adapted from Gidigas, 1974)

Group	Colour and Structure	Particle Shape
Decomposed rocks	Discoloured parent rock, grey to yellowish and reddish grey, structure of parent rock preserved	Angular to sub-angular, varying in size from coarse sand to boulders
Fine-grained materials	Variable shades of red, structure depends on parent rock, degree of weathering and state of compaction	Isolated quartz and concretionary particles in fine grained matrix.
Gravelly materials	Yellowish-red to pink, structure depends on degree of weathering and (if present) laterite development	Irregular, angular to rounded particles
Laterite materials (boulders and cobbles)	Reddish-brown, dark brown, pinkish-brown. In residual materials indurated elements form a continuous, coherent skeleton; in non-residual materials the indurated elements cement pre-existing materials in secondary laterisation	Irregular quartz particles of different sizes

Variations in the morphological characteristics are caused mainly by the uncontrolled way in which pedogenesis is influenced by the soil-forming factors (Mohr and Van Baren, 1954). Topography and drainage conditions in particular exert considerable influence on the nature of tropical soil profiles. Ahn (1970) has suggested that in West Africa well-drained upper-slope soils frequently are reddish, red or brown or brown-red and that such red colours are mainly due to the presence of hematite (Fe_2O_3), a non-hydrated iron oxide, in the soil. In middle and lower slopes, soil drainage is poorer than in the upper slope and crest and hydrated iron oxides are produced, mainly in the form of goethite ($\text{FeO}(\text{OH})$) and limonite ($\text{FeO}(\text{OH}) \cdot n\text{H}_2\text{O}$). The presence of these minerals accounts for the change in colour down the slope, changing from dark reddish brown to brown to yellow brown.

Tuncer *et al.*, (1977) identified six stages of weathering of basaltic bedrock in a tropic environment, with each stage of weathering being characterized by unique engineering properties (Figure 3-6). The weathering of the parent rock with a low void ratio and high cohesion leads to rock disintegration, increasing the void ratio and leading to a decrease in the friction angle. The subsequent development of secondary minerals leads to an increase in clay content, resulting in an increase in cohesion and decrease in void ratio and permeability. The specific gravity increases due to the increase in sesquioxide development. Iron oxide content continues to increase, kaolinite content decreases and sesquioxides start cementing clay particles. The specific gravity increases even further due to the increase in iron oxide content. The continuous increase in iron oxide content leads to further cementation of aggregates and decreased void ratio. Specific gravity may either increase or decrease, depending on whether iron- or aluminium oxides dominate, and may thus provide a useful indication of iron content and stages of weathering. The final stage of weathering represents the development of a fully developed, continuous hardpan laterite.

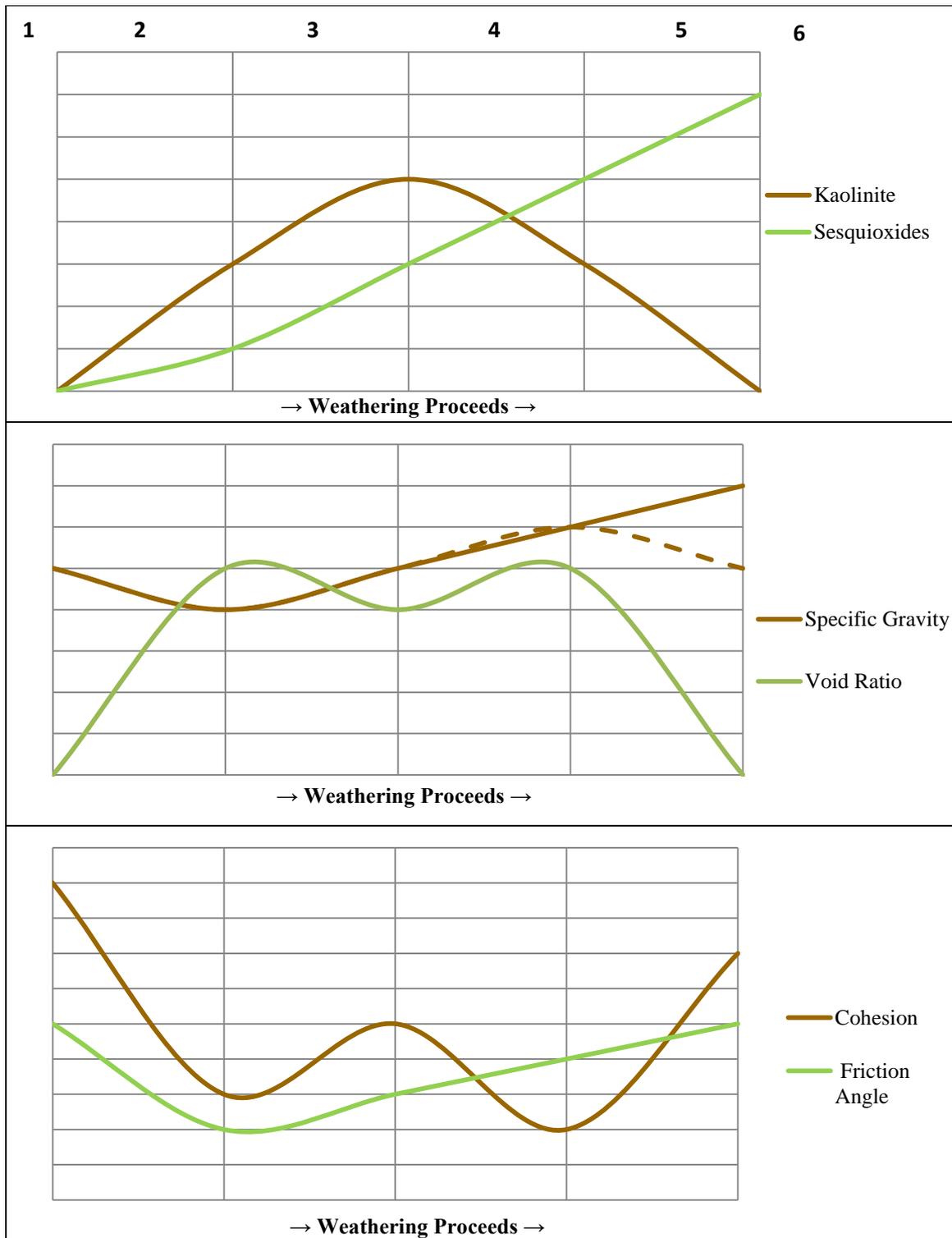


Figure 3-6 Variation in engineering properties of residual basalt with weathering (adapted from Tuncer *et al.*, 1977)

3.2 Identification and Classification of Tropical Red Soils

Pedological and even geological maps are frequently used in order to differentiate between and identify different soil types. It is however clear from Section 3-1 that soil formation in the tropics is a highly variable and often complex process. Significant variation may occur in the soil properties of the tropical red soil depending on (amongst others) the stage of weathering,

structural development, leaching and position in the landscape, parameters which are not provided for in existing classification systems.

3.2.1 Pedological Maps and Soil Classification Systems

The three most commonly used soil classification systems worldwide are the French classification system (Duchaufour, 1982), the American Soil Taxonomy system (USDA, 1975) and the Food and Agricultural Organization of the United Nations (FAO, 1988) scheme. Figures 3-7 and 3-8 respectively illustrates the distribution of the common soil orders for the world and African continent according to the American Soil Taxonomy classification system orders (Jones *et al.*, 2013).

ENGINEERING GEOLOGICAL EVALUATION OF TRANSPORTED TROPICAL RED SOILS

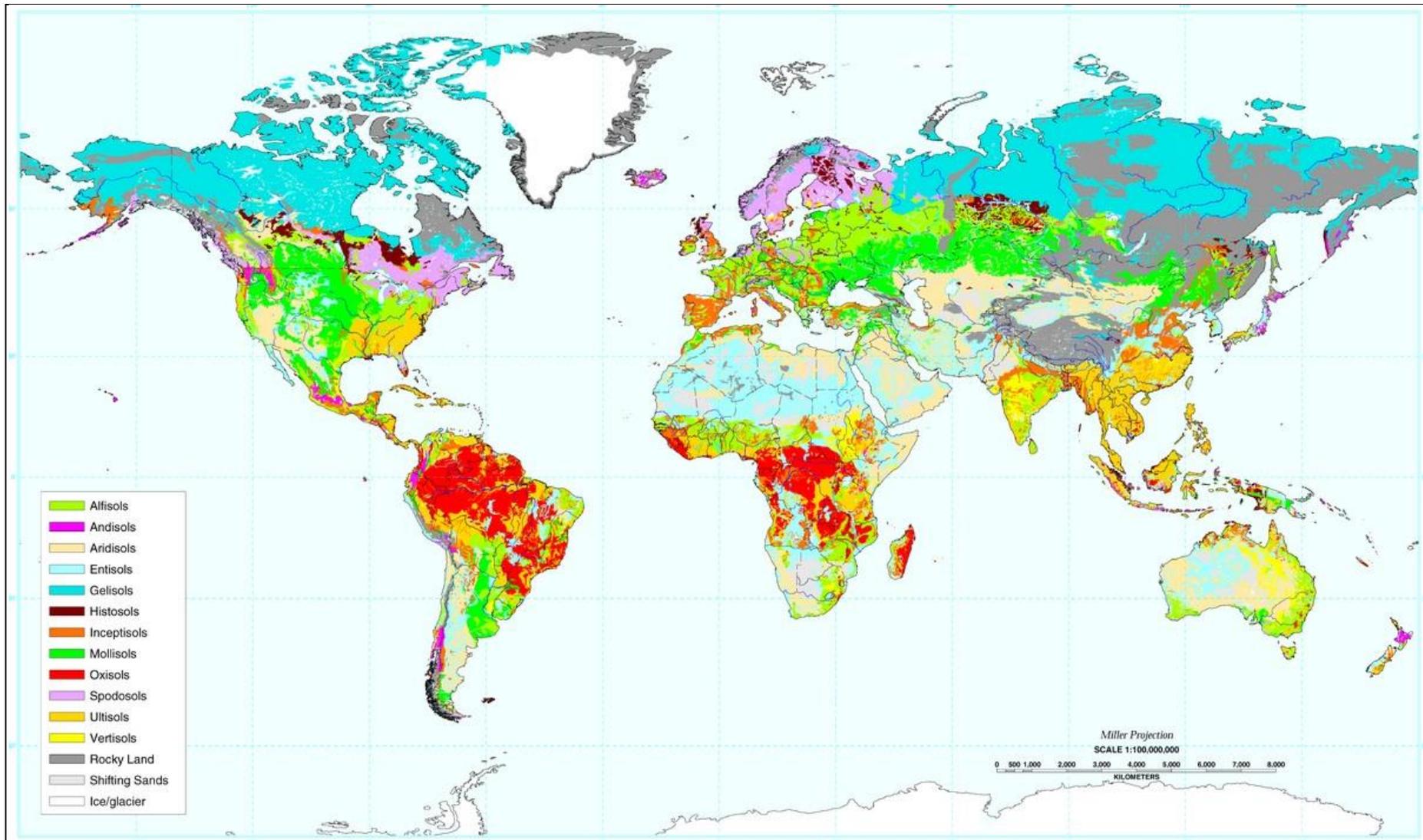


Figure 3-7 USDA Soil Map of the World (1975)

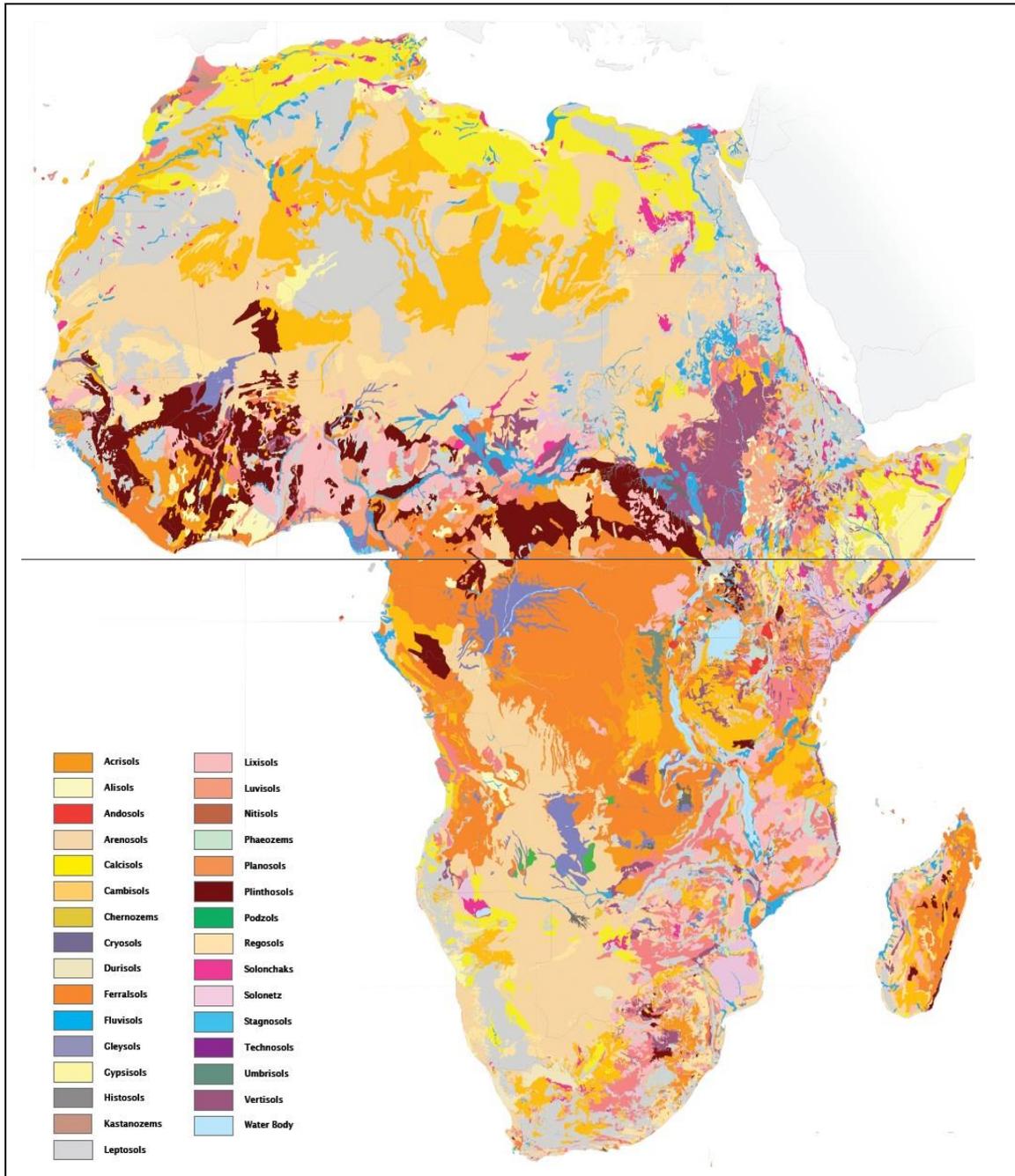


Figure 3-8 Distribution of soil orders for Africa (Jones *et al.*, 2013)

From the literature, the names of the main soil types recognised by each of the three main soil classification systems are summarised in Table 3-6 below.

Table 3-6 Summary of main soil classification system soil types

Duchaufour (1982)	FAO – UNESCO (1988)	American Soil Taxonomy (1975)
fersiallitic soils	cambisols, calcisols, luvisols, aliosols	alfisols, inceptisols
fersiallitic andosols	andosols	inceptisols
ferruginous soils	luvisols, aliosols, lixisols, plinthosols	alfisols, ultisols
ferrisols	nitisols, acrisols, lixisols, luvisols, plinthosols	ultisols, oxisols
ferrallitic soils	ferralsols, plinthosols	oxisols
vertisols	vertisols	vertisols
podzols	podzols	spodosols

The French classification system is based on the different phases of the weathering process and the resulting mineral composition of the soils. Consequently, this classification system might be the most readily used or understood by geotechnical specialists worldwide. The use of this scheme is particularly useful to clarify the often confusing terminology applied to tropical residual soils, particularly “tropical red soils”. A small percentage of sesquioxides will contribute to the overall red colouration and, because of the continuous nature of tropical weathering processes and its influence on soil development, there is no clearly defined transition between “red” and “non-red” soils. However, different concentrations of these iron and aluminium hydroxides will result in a wide variation of geotechnical properties, making it vital to distinguish between the different types or stages of tropical red soil development.

In the French system, tropical red soils are subdivided into *ferruginous*, *ferrisol* and *ferrallitic* profiles based on the characteristic properties associated with the various stages of soil development. Northmore *et al.* (1992) highlights that, based on the fact that *fersiallitic andosol* soil types can also have red colourations and are often associated with other red soils, these should be included as a typical stage of tropical soil development.

The typical aforementioned tropical red soil classes are briefly summarised below based on the descriptions from Northmore *et al.*, (1992):

- a. **Fersiallitic andosol soils** – are most commonly associated with sub-tropical climates, with distinct seasonal variations, annual rainfall levels of 500 to 1000 mm and mean annual temperatures of 13°C to 20°C. 2:1 clays predominate (particularly smectite), but kaolinite may develop where good drainage conditions are present. These “young” soils may develop within 500 to 1500 years (Yamada, 1977).

These soils are generally reddish brown in colour, porous, of low bulk density and have high water-retention capacities. Fersiallitic soils indurate on drying and prolonged desiccation of the material may lead to irreversible changes in material properties.

- b. **Ferruginous soils** (including **ferrisols**) – are associated with more humid sub-tropical zones or tropical zones with dry seasons and are more strongly weathered than fersiallitic soils. Kaolinite is the predominant 1:1 clay mineral and 2:1 clay minerals are

subordinate, occurring mostly in localised clay-rich horizons. The weathering of primary minerals is not yet complete.

On older land surfaces and permeable, base-rich parent rocks a transitional material, referred to as ferrisols, is formed during the transition from ferruginous to ferrallitic soils. These are deeper weathered than typical ferruginous soils, but also characterized by the incomplete weathering of the primary minerals. Ferruginous soils represent an intermediate stage of soil development between that of fersiallitic andosols and ferrallitic soils.

- c. **Ferrallitic soils** –are formed in the hot, humid tropics where annual rainfall levels exceed 1500 mm and mean temperatures are above 25°C, with short to no dry season. Ferrallitic soils represent the final phase of soil development in the tropics (Figure 3-9) and soil profiles may be several metres thick. Decomposition of all primary minerals, except quartz, is complete and much of the silica and bases have been removed in solution.

It is estimated that ferrallitic soils take approximately 10 000 or more years to form, depending on the parent rock composition. The silica-content of the parent rocks is usually reflected in the kaolinite development. Silica-rich parent materials are characterized by deep, poorly drained, often mottled kaolinite development, which can harden irreversibly on desiccation. On silica-poor parent rocks kaolinite development is not as deep, with a stable but friable aggregate soil structure of high porosity and which does not harden irreversibly on desiccation.

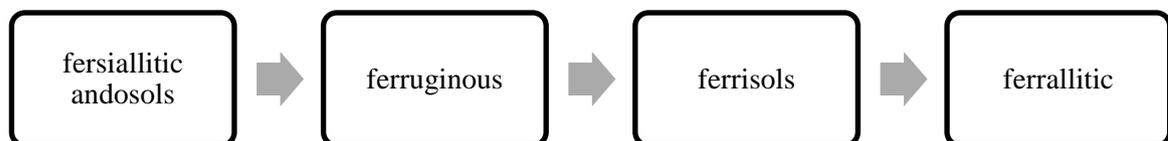


Figure 3-9 Different classes of tropical red soils based on the stages of soil development (Based on Duchaufour, 1982 and adapted from Northmore *et al.*, 1992)

3.2.2 Geological Maps

Different weathering profiles will develop over different rock types. The presence of faults, also identified from geological maps, may indicate zones of weakened, disturbed or broken rock where weathering and alteration of the rock mass may be further advanced.

When considering typical residual tropical red soils formed over various types of parent rock, the sesquioxides of iron and aluminium and kaolinite are formed through the weathering of basically all igneous rocks, with kaolinite clays representing the eventual end product of weathering. Pyroclastic and other extrusive igneous rocks may also produce allophanes, helictites and metahalloysites, particularly under cooler climatic conditions (Northmore *et al.*, 1992).

It is obvious that geological maps (Figure 3-10), detailing the expected bedrock type for a specific region, can give a general indication of the expected residual soil profile only, thereby excluding

the presence and/or distribution of transported soils and their expected characteristics in the region of interest. Further, it should be kept in mind that the properties of the residual soil are not only a function of the geology, but governed by the other soil forming factors as well. It is only once all these factors are taken into account when geological maps may provide useful information to the engineer regarding the expected residual soil characteristics.

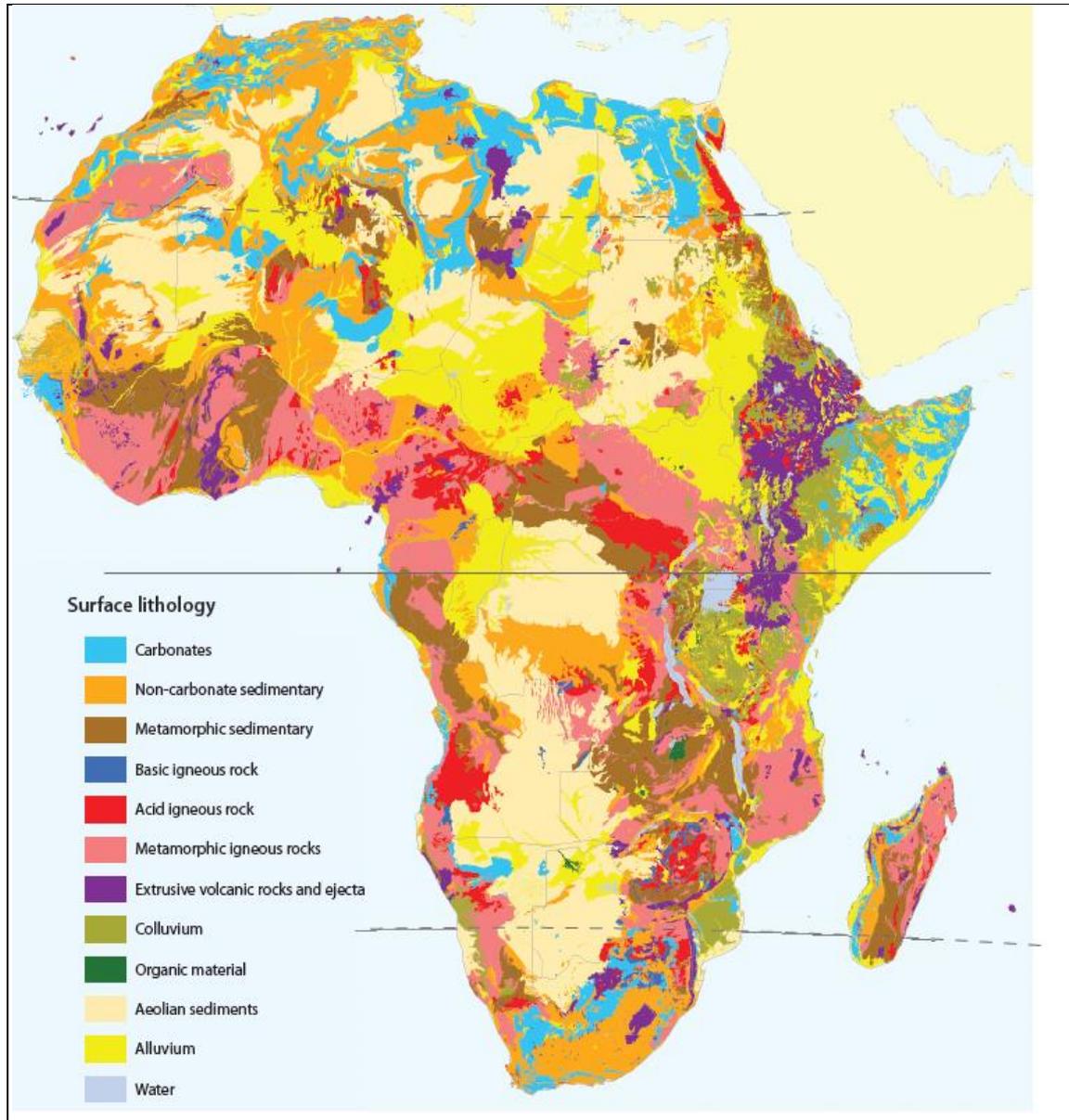


Figure 3-10 Surface geology of Africa (Jones *et al.*, 2013)

3.2.3 Application

For engineering purposes a practical soil classification system involves the categorization of different soil types into groups or classes of materials which possess similar properties. Such a classification system should provide a guide to the engineering performance of the soil type and a means by which soils can be identified quickly and easily. Essentially, therefore, soil classification tests attempt to provide an indication of its behaviour in various engineering situations and determine a benchmark by which it can be compared to other similar or dissimilar materials (Northmore *et al.*, 1992).

It is evident from the information summarised here that soil maps and associated pedological classification schemes can be very useful in evaluating the distribution and occurrence of tropical soils. These maps provide a very valuable indication of the occurrence of general broad soil types. However, not all pedological classification schemes follow the same classification process, making subsequent correlations (as summarised in Table 3-1) between the different schemes challenging. Considerable care should furthermore be exercised regarding the classification of soils using pedological maps as significant variation of the soil profile (laterally and vertically) may arise in the material due to stratification of the parent rock (Buurman and Subagjo, 1980). Localized pedogenic factors might also further influence the different stages of soil development, as well as the extent and distribution of the materials.

Due to the fact that soil formation in the tropics is a highly variable and often complex process, the classification of soils formed under such conditions as distinct and well-defined soil types appears to have been avoided in the past. Much of the difficulty would also appear to stem from the inapplicability of 'conventional' soil mechanics concepts, developed almost exclusively from work on temperate soils, when applied directly to certain soils from tropical zones (Northmore *et al.*, 1992). Unfortunately, this approach has resulted in the grouping of materials of largely varying characteristics and engineering behaviour under a single, all-encompassing term, resulting in the categorization of materials that may have significantly different chemical, geological and/or geotechnical properties as a single soil type, namely "tropical red soils". No distinction is also frequently made in the literature between reporting the results as residual and transported tropical red soils.

3.3 Tropical Red Soils and Laterites

Tropical red soils are frequently recorded and loosely referred to in the literature as either 'laterites', 'laterite soils', 'latosols' or 'lateritic soils'. The term 'laterite' was first proposed by Buchanan (1807) to describe the reddish ferruginous, unstratified and porous material occurring in Malabar, India. This material was used locally as building bricks and hence called 'laterite', derived from the Latin term 'later' meaning brick (Gidigas, 1974).

Numerous definitions and comprehensive chemical and textural systems of nomenclature of laterite materials exist in the literature. For example, Fermor (1911) and Lacroix (1913) developed systems on the basis of subdividing 'laterite' into different sub-groups based on the ratios of Fe, Al, Ti and Mn to silica and the relative hydroxide content of the material. Further, Martin and Doyne (1927, 1930) and Joachin and Kandiah (1941) defined three grades of laterite materials based on the ratio of silica to aluminium and silica to sesquioxides only. Pendleton and Sharasuvana (1946) defined "laterite" as soils in which a laterite horizon is present. In terms of the textural characteristics of the material, Nixon and Skipp (1957) suggested that only the concretionary coarse-grained materials are referred to as 'laterite' and finer-grained materials referred to as either red clays or latosols. The lateritic weathering profile can be defined as "soil profiles where laterite horizons exist or capable of developing under favourable conditions" (Gidigas and Kuma, 1987).

Where conditions are favourable for laterite development, hardened pedogenic layers may develop within tropical red soil horizons through the leaching and precipitation of sesquioxides with fluctuating soil moisture content. The degree or extent of laterite formation can partly be

described as a function of time, the presence of a suitable mobilizing agency and sufficient sesquioxide concentrations in the soil system. The final stage of development is typically represented by a continuous indurated sheet of hardpan ferricrete or laterite (Gidigas, 1974). Bell (1993) proposed using the silica sesquioxide ratio ($\text{SiO}_2 / (\text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3)$) of the soil, where ratios of between 1.33 and 2 indicate lateritic soils and values greater than 2 as indicating non-lateritic, tropically weathered soils.

From the previous examples and other cases encountered throughout the available literature, it is evident that varying definitions and classifications of laterite, ranging from being very specific to highly generalized, exist. The fact that the definition of “laterite” and all other derivative and associated terms has subsequently been differently interpreted and often generalized in the past, has resulted in the grouping of materials that may have significantly different chemical, geological and/or geotechnical properties as a single soil type. As a result numerous inconsistencies have developed in the identification and classification of tropical soils and laterites, with a significant range of geological, geotechnical and chemical properties being assigned to both lateritic and tropical red soils. The engineer or geologist who intends to use the available data in the literature to supplement or compare the data of his or her own studies, should therefore exercise extreme caution in ensuring the relevance of the data to the soil horizons identified in his or her current study area.

It is recommended that, and assumed for the purpose of this thesis, the term “laterite” is used solely to refer to those materials or soil horizons which either represent laterite formation in the form of hardened pedogenic horizons or contain distinct evidence of laterization in the form of nodules, concretions or distinct sesquioxide discolouration (mottling or staining). Tropical red soils as the focus of this study refer to all non-lateritic tropical soils other than tropical “black” soils and which in reality may vary in colour (between shades of red, orange, yellow and brown), microstructure, mineralogy and grain size distribution.

3.4 Description of the Tropical Weathering Profile

3.4.1 Morphological Properties

A typical tropical weathering profile representing a descriptive classification system of rocks based on degree of weathering is illustrated in Figure 3-11 and Table 3-7 below. It should be emphasised that the boundaries between the different horizons are not always as clearly defined as in the example shown. Field experience indicates that classification of the material using the morphological properties only, not taking into account the mechanical properties or expected engineering behaviour, is of little to no value as a source of information for the engineer (Duncan, 1970).

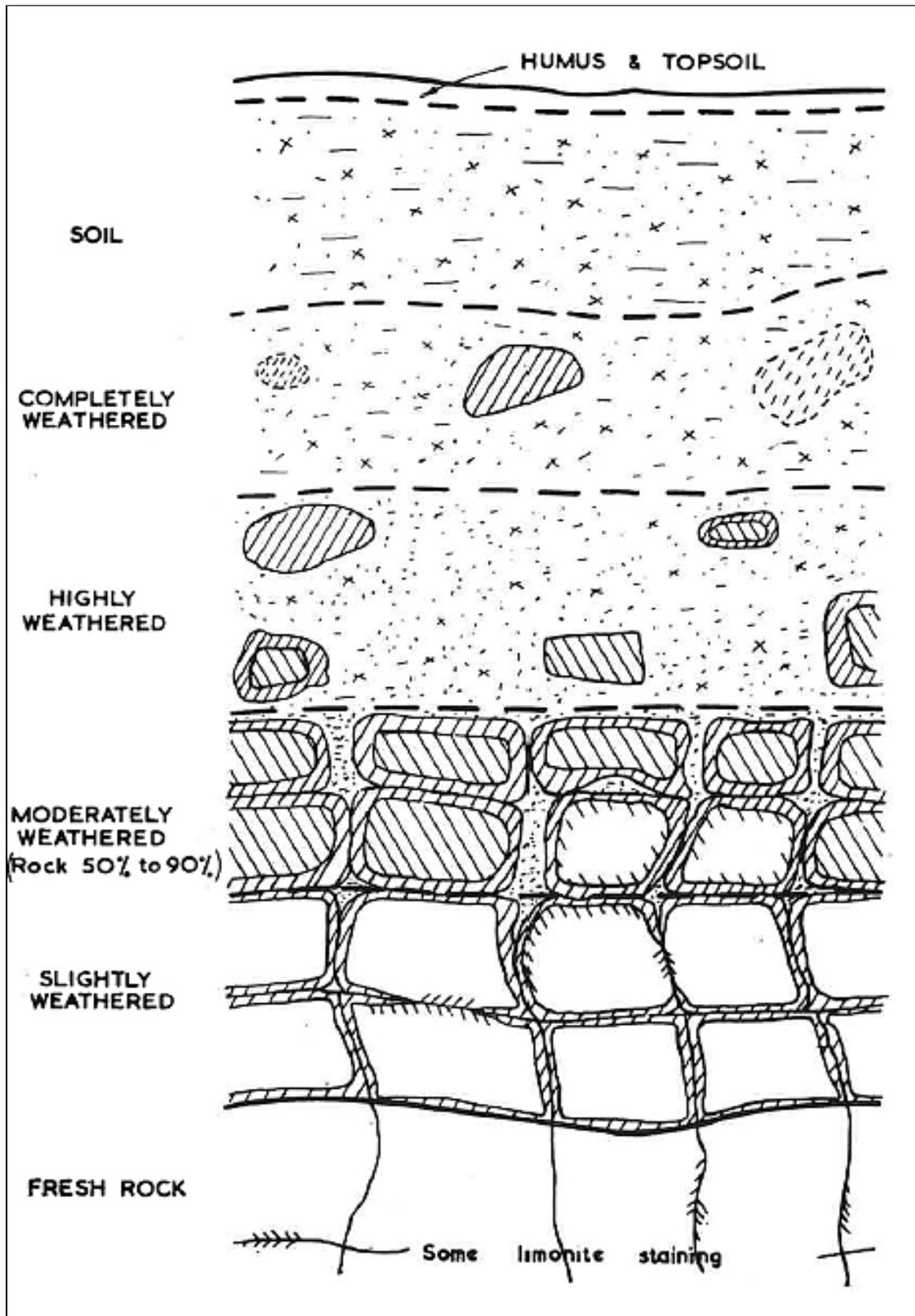


Figure 3-11 Typical tropical weathering profile (Little, 1969)

Table 3-7 Proposed classification system for residual tropical soils by degree of weathering (Little, 1969)

Grade	Degree of decomposition	Field recognition	Engineering properties
VI	Soil	No recognisable rock texture, surface layer contains roots and humus	Unsuitable for important foundations, unstable on slopes when cover is destroyed
V	Completely weathered	Rock completely decomposed by weathering in place but texture still recognisable, cannot be recovered as cores by conventional rotary core drilling methods	Can be excavated by hand or ripping without the use of explosives, unsuitable for the foundations of concrete dams or large structures, may be suitable for the foundations of earth dams and fill, unstable in high cuttings at steep angles, requires erosion protection
IV	Highly weathered	Rock so weakened by weathering that fairly large pieces can be broken and crumbled in hands, sometimes recovered as core by careful rotary core drilling, stained by limonite, less than 50% rock	Similar to grade V, unlikely to be suitable for foundations of large dams, presence of boulders makes it an unreliable foundation stratum for large structures
III	Moderately weathered	Considerably weathered throughout, possesses some strength - large pieces cannot be broken by hand, often limonite stained, 50-90% rock	Excavated with difficulty without the use of explosives, mostly crushes under bulldozer tracks, suitable for the foundations of small concrete structures and rockfill dams, may be suitable for semi-pervious fill, stability in cuttings depend on structural features
II	Slightly weathered	Distinctly weathered through much of the rock fabric with slight limonite staining, strength approaching that of fresh rock, more than 90% rock	Requires explosives for excavation, suitable for concrete dam foundations, highly permeable through open joints, often more permeable than the zones above or below, questionable as concrete aggregate
I	Fresh rock	Fresh rock may have some limonite stained joints immediately beneath weathered rock	Staining indicates water percolation along joints, individual pieces may be loosened by blasting or stress relief and support may be required in tunnels and shafts

3.4.2 Classification by Secondary Mineral Content

Scott (1955) proposed that the percentage secondary minerals present in decomposed rocks can be used to estimate the behaviour of the material as aggregate for pavement structures. The secondary mineral content can be determined through Equation 3-3 below (Gidigas, 1974).

$$\frac{100 \times M_{sec}}{M_{prim} + quartz} \quad (\text{Eq. 3-3})$$

Where:

M_{sec} = Number of secondary minerals counted

M_{prim} = Number of primary minerals counted

Claus (1967) and Weinert (1964) suggested and developed such classification systems based on the degree of decomposition and leaching of basic igneous rocks and the percentage secondary mineral content of the material. Table 3-8 below summarises this classification system as proposed by Weinert (1964).

Table 3-8 Classification of decomposed crystalline rocks by content of secondary minerals (Weinert, 1964)

Percentage Secondary Minerals	Stages of Decomposition
No lower limit – up to 100%	Residual soil
>30 %	Badly decomposed
15-30 %	Decomposed
<15 %	Fresh

3.4.3 Classification by Saturation Moisture Content

Studies carried out on decomposed rocks from temperate climates indicated that void ratio, porosity, apparent specific gravity and saturation moisture content can all be used to identify fresh and decomposed rock materials, as they directly reflect the state of the texture and other mechanical properties (Gidigas, 1974).

Duncan and Dunne (1967) suggest that as the saturation moisture content of the material approaches 15 %, the rock approaches the texture of a mature residual soil. In addition, the saturation moisture content has been used to evaluate the apparent specific gravity, resistance to boring, shear strength parameters, modulus of elasticity and seismic velocity (Figure 3-12).

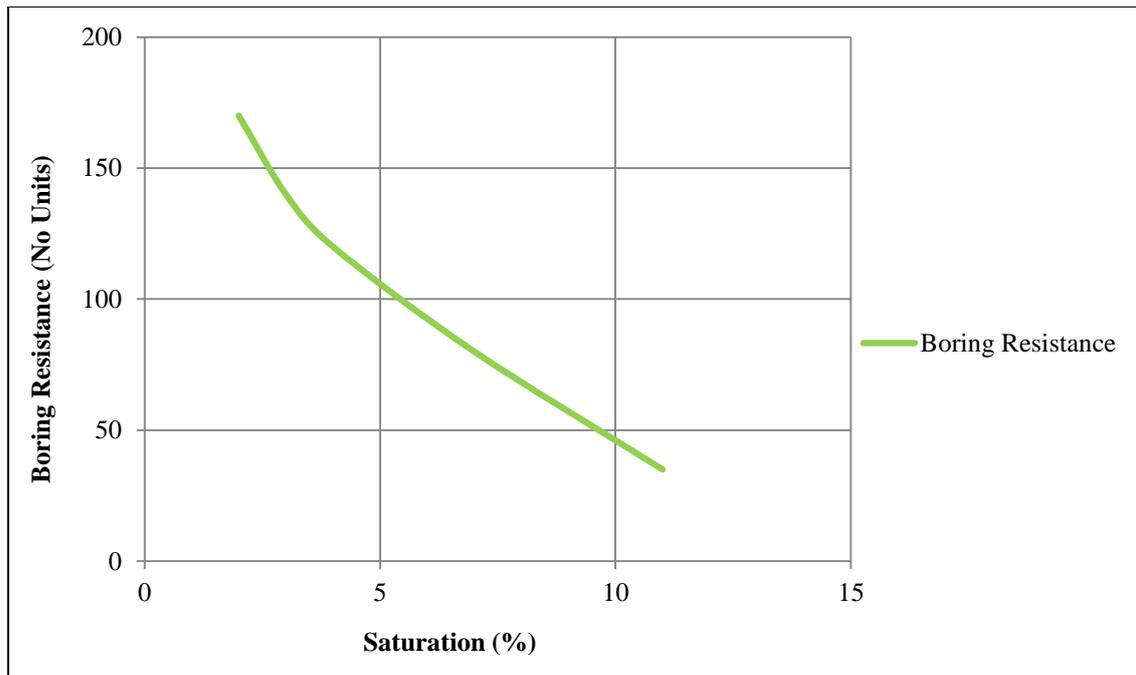


Figure 3-12 Boring resistance related to percentage saturation (Hamrol, 1961)

3.5 Engineering Characteristics of Residual Tropical Red Soils

The typical engineering characteristics of residual tropical red soils contained in the existing literature have been identified and summarised hereafter. It should be emphasised that the data and findings summarised hereafter represent tropical red soils of a residual nature of various stages of weathering and from a variety of settings and regions as summarised in the literature. This excludes any results and findings pertaining to lateritic and tropical black soils. Very little meaningful information on the typical engineering characteristics of transported tropical soils was found to exist and is therefore also excluded from the tropical red soil characteristics summarised hereafter.

The usefulness of conventional index and strength tests for the identification and engineering classification of tropical soils has been questioned in the past, primarily because of the dependence of such tests on the sample preparation process and the variation of the natural soil structure which occurs during preparation. However, it has been shown that the unique engineering properties of tropical soils may be very useful for identifying and ‘grouping’ soils which are likely to possess distinctive or similar physical characteristics for specific or local application (Wesley, 1988).

Many residual tropical red types of soil are reported as being highly sensitive to structural breakdown and manipulation. Manipulation of the material using heavy equipment may result in marked changes in characteristics and the ease with which it is handled on site. The influence of mixing or disturbance of samples prior to liquid limit testing is further testament to the susceptibility of residual tropical red soils to structural breakdown, as is illustrated by the effect of mixing time on the liquid limit of the material in Figure 3-13 and the influence of sample remoulding on other index properties of such soils in Table 3-9.

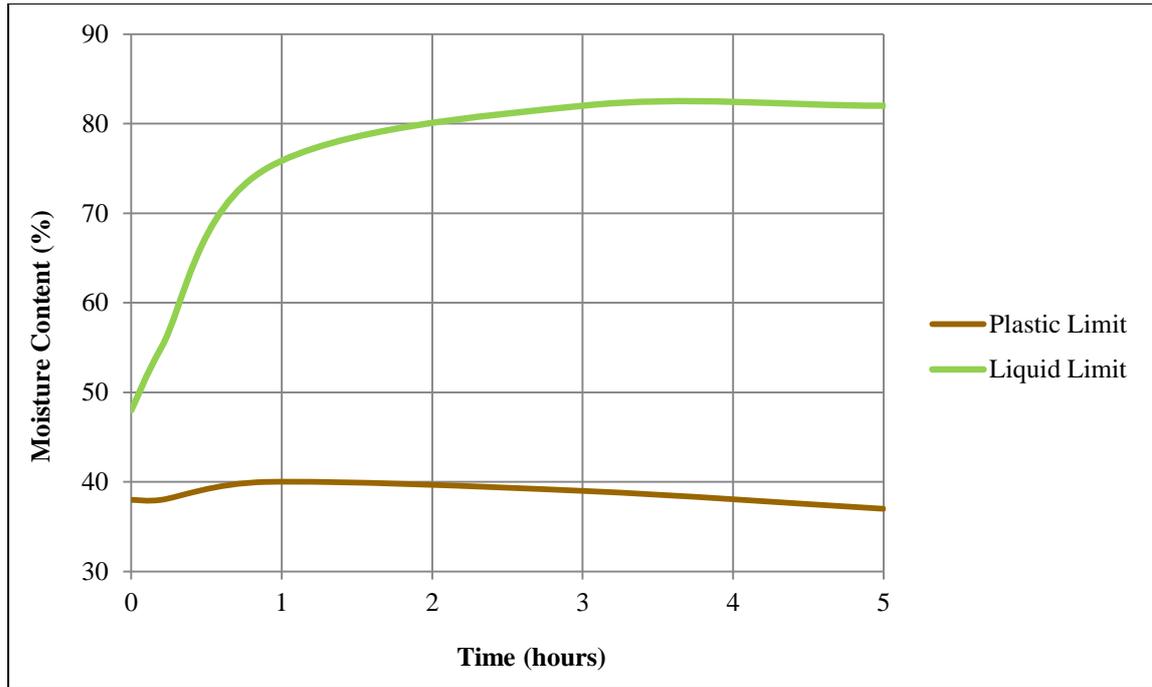


Figure 3-13 Effect of mixing time on liquid limit of a red clay from Kenya (adapted from Millard, 1962)

Table 3-9 Comparing physical properties of natural and remoulded samples (adapted from Townsend *et al.*, 1971)

Property	Sample in Natural State	Remoulded Sample
Liquid Limit (%)	57.8	69
Plastic Limit (%)	39.5	40.1
Plasticity Index (%)	18.3	28.0
Specific Gravity	2.8	2.8
Proctor Density (kg/m ³)	1330	1300
Optimum Moisture Content (%)	35	34.5

Mitchell and Sitar (1982) found that different dispersing agents (sodium hexametaphosphate versus sodium oxalate and sodium silicate) had an effect on the measured clay quantities, and considered this as evidence of the structural instability of these materials. It was however found by Northmore *et al.*, (1992) during their study of a number of tropical red soils from Kenya and Indonesia that the type of dispersing agent only has a limited influence on the overall test results and percentage clay. Additionally, tropical red soils are reported as being highly sensitive to changes in the natural moisture content of the material, with such changes typically resulting in irreversible changes in the physical properties of the material.

3.5.1 Index Properties

The well-known global classification systems, such as the Unified Soil Classification System (USCS), are all based on the grain size distribution and Atterberg Limits for soils from temperate climatic environments. The clay minerals in these soils are generally stable, allowing for the relatively accurate determination of the index properties and allowing the development of numerous relationships between index properties and engineering behaviour.

Due to the instability of clay minerals in tropical soils and the susceptibility of changes in material properties on drying (as indicated in Table 3-10), the relationships between the results of classification tests and the engineering behaviour are not as easy to obtain for tropical red soils (Mitchell and Sitar, 1982). Further, it should also be kept in mind that drying not only occurs during sampling and testing, but also depends on the local climatic conditions, drainage and profile position. This results in significant heterogeneity in the regional soil characteristics.

Table 3-10 Effect of drying on index properties of hydrated clay from Hawaiian Islands (Willis, 1946 in Gidigas, 1974)

Property	Natural Moisture Content	Partially Air Dried	Completely Air Dried
% Sand	30	42	86
% Silt	34	17	11
% Clay	36	41	3
Liquid Limit (%)	245	217	NP
Plastic Limit (%)	135	146	NP
Plasticity Index (%)	110	71	NP

*NP = Non Plastic

De Graft-Johnson *et al.* (1972) recommend that the data from index tests alone should not be used to distinguish between soils formed in different climatic regimes and from different parent rock types. However, it may be possible to make use of the classification tests, gradings and Atterberg Limits to establish a correlation with the expected engineering behaviour for specific application and based on local experience.

Studies by Northmore *et al.* (1992) have shown and confirmed that index property values of tropical red soils are extremely sensitive to pre-treatment of the samples prior to testing, particularly to pre-drying and, for plasticity and shrinkage tests, to the degree of manipulation or mixing of the test sample. Pre-drying of any sort leads was found to frequently lead to inconsistent results. Meaningful and repeatable plasticity-related tests were only achieved if the aggregated fabric, characteristic of these soils, was completely broken down prior to testing by thorough and complete remoulding of the test sample.

3.5.2 Strength and Consolidation Characteristics

From the available literature it was found that the data on the strength testing of undisturbed samples of tropical red soils is limited, with the majority of available test results reflecting the shear strength parameters obtained from remoulded specimens.

In their study of tropical red soils Northmore *et al.* (1992) determined the shear strength characteristics for a number of undisturbed and remoulded samples of tropical red soil from Kenya and Indonesia. Values of effective cohesion (c') were found to be highly variable, ranging

from 0 to 97 kPa and values of effective angle of internal friction (ϕ') in the range of 11° and 41° . These represent a big variation in values and are higher than those of temperate soils of similar plasticity or clay fraction. Wesley (1977) noted that allophane-rich soils generally result in soils of higher strengths than halloysite-rich soils. This could however not be confirmed from the data in the study by Northmore *et al.* (1992). Table 3-11 below summarises the average undisturbed shear strength characteristics from three other available studies of tropical red soils.

Table 3-11 Summary of shear strength characteristics of undisturbed samples of tropical red soils

Reference	Average Dry Unit Weight (kg/m^3)	Friction Angle (degrees)		Cohesion (kPa)	
		Range	Average	Range	Average
Vargas (1974)	-	23-33	28	0-59	24
Tuncer and Lohnes (1977)	1300	27-57	42	48-345	163
Fos (1973)	1150	36-38	37	22-28	25
Northmore <i>et al.</i> (1992) (Indonesia)	836	11-36	22	2-97	32
Northmore <i>et al.</i> (1992) (Kenya)	1112	12-44	25	0-55	24

The available residual friction angles of remoulded or destructured materials were found to vary between 18° and 41° (mostly 28° - 38°), with effective cohesion values ranging between 0 kPa and greater 50 kPa (De Graft-Johnson *et al.*, 1969; Gidigas, 1974; Saunders and Fookes, 1970 and Terzaghi, 1958). In an effort to establish the effect of remoulding on the engineering behaviour of the soil, Northmore *et al.* (1992) also determined the shear strength characteristics of two sets of destructured and compacted samples to compare with the results obtained from the testing of the undisturbed samples. The test results revealed largely similar stress-path and strength behaviour of disturbed and destructured materials. In both cases the compacted specimen revealed higher effective friction angles than the undisturbed samples. This implies that, provided these results are representative of the overall behaviour, reworked material will have greater slope stability than *in-situ* material.

The available undisturbed and remoulded results discussed here reflect a significant variation in the values in the shear strength characteristics of tropical red soils and, as was found to be the case with the index properties, the heterogeneity of the material characteristics, degree of weathering and cementation are the most likely reasons for the variations.

Northmore *et al.* (1992) concludes from their testing of tropical red soils from Indonesia and Kenya that there appears to be a positive relationship between the effective shear strength of tropical soils and its plasticity index and moisture content. This is the opposite of the established relationship in clayey soils from temperate climatic regions. There also appears to be a general positive relationship between the increases in the effective strength of the material with an

increase in depth below natural ground level due to either the development of material structure. The presence of yield points, representing points where significant structural change and densification occurs during the tri-axial test procedure, was also clearly established for the tested red soils. These points (one per test specimen) relate to the application of a particular applied stress range (most often between 100 and 200 kPa), representing single densification events.

Limited published results are available on the consolidation data of undisturbed tropical red soils. The available data indicates very variable initial void ratios, typically varying between 0.92 to up to 5.36, but those tropical red soils classified as “andosols” having very high initial void ratios and low dry densities in particular. It has always been assumed that the tropical red soils are fairly incompressible compared to clays from temperate climates. However, consolidation tests on undisturbed samples reflect very high rates of primary consolidation (indicating high rates of short term settlement), making the calculation of the consolidation coefficient (C_v) very difficult (Northmore *et al.*, 1992).

When evaluating the rate of consolidation according to the percentage of consolidation in the first minute of testing, it is evident that allophone-rich materials generally experience a reduction in the consolidation rate with increasing stress, whereas halloysite-rich samples tend to maintain a high rate of consolidation throughout (Northmore *et al.*, 1992). C_v values typically range between 6.3×10^{-6} cm²/sec and 2.5×10^{-2} cm²/sec. The consolidation coefficients of compacted or remoulded tropical red soils were found to typically range between 1×10^{-1} cm²/sec and 1×10^{-3} cm²/sec (Saunders and Fookes, 1970). The compressibility results (M_v) ranges from high to very low, with an overall decrease in compressibility with an increase in the applied stress. Secondary consolidation (C_a), which serves as an indication of the long term consolidation behaviour of the soil, was also found to increase with an overall increase in the applied stress (Northmore *et al.*, 1992).

The Compressibility Index (C_c) of the material provides an indication of the rate of decrease of the void ratio with an increase in the applied stress for the linear or “post-yield” portion of the void ratio versus log stress curve (Northmore *et al.*, 1992).

Using the calculated C_c , Vargas (1973) established an empirical relationship between the Compressibility Index and the Liquid Limit of 55 samples of tropical red soils from Brazil through:

$$C_c = 0.005 (LL + 22) \pm 0.1 \quad (\text{Eq. 3-4})$$

Where:

C_c = Compressibility Index

LL = Liquid Limit

The Compressibility Index values in the literature of tropical red soils vary between 0.20 and 2.10. The relationship between the C_c and LL values proposed by Vargas (1973) is compared to the relationship derived by Terzaghi and Peck (1967) for clays from temperate climate zones (Mitchell and Sitar, 1982). It is evident from the graphic representation in Figure 3-14 below that the relationship between the C_c and LL for the two different soil types does not vary significantly.

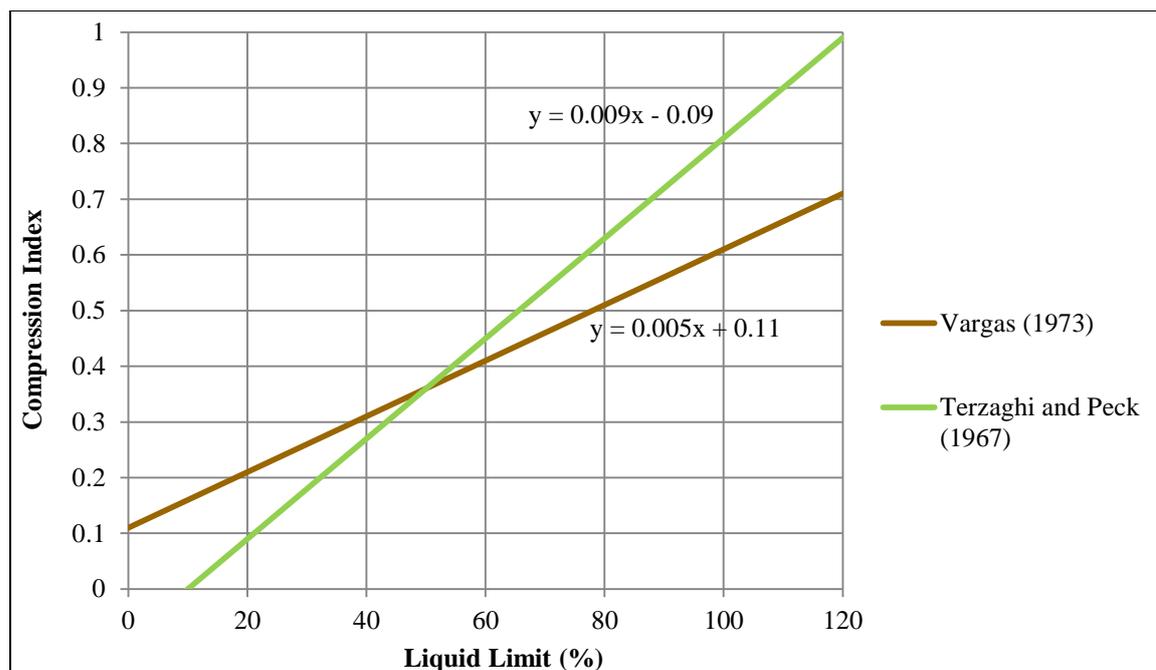


Figure 3-14 Compression Index as a function of Liquid Limit for tropical red soils (Vargas, 1973) and temperate climate soils (Terzaghi and Peck, 1967)

3.5.3 Compaction Characteristics

The compaction characteristics of tropical soils are influenced by a number of variable factors, leading to a wide variation in the characteristics. These factors include the grading distributions, sample preparation, crushing strength of any coarse fractions present, mineralogical composition, compaction method and effort. Table 3-12 summarizes the general compaction characteristics of some tropical red soils available from the literature.

Table 3-12 Summary of general compaction characteristics of some tropical red soils

Reference	Maximum Dry Density (kg/m^3)	OMC (%)
	Range	Range
De Graft-Johnson <i>et al.</i> (1972); Townsend <i>et al.</i> (1971)	1300-2280	6-22
Northmore <i>et al.</i> (1992) (Indonesia)	1090-1300	37-53
Northmore <i>et al.</i> (1992) (Kenya)	810-1560	27-82

From this data summarised in the table above, it is evident that the compaction characteristics of tropical red soils are directly influenced by the clay content of the material; higher clay contents result in higher values of Optimum Moisture Content (OMC) and lower maximum dry unit weight.

3.5.4 Permeability

It is generally assumed that the permeabilities of *in-situ* tropical red soils are high (up to 10^{-2} cm/sec), but as was the case with characterizing the strength of these materials the available data on the permeabilities of undisturbed tropical soils are very limited. Northmore *et al.* (1992) recorded values of 1.1×10^{-3} cm/s to 5.8×10^{-6} cm/s for the Kenya specimens and 5.6×10^{-4} cm/s to 2.2×10^{-6} cm/s for the specimens from Indonesia. This roughly equals the established permeabilities of silty materials or very fine sands from more temperate environments. As the permeability of a material is highly dependent on its structure, there is a significant variation in the available permeability results of undisturbed specimens and those permeabilities determined on remoulded specimens. The permeability of remoulded and compacted tropical red soils was found to vary between 1×10^{-6} and 1×10^{-8} cm/sec (Saunders and Fookes, 1970), with Northmore *et al.* (1992) reporting a “hundred to three hundred fold” decrease in the permeability of remoulded specimens compared to the values established for undisturbed specimens.

The enhanced permeability of tropical soils due sub-surface biotic activity should also be considered. Blight (1982) has highlighted the influence of termite activity on the overall permeability of the soil, as well as that of open channels from plant roots and other organisms. The presence of these cavities may increase the overall permeability by up to several orders of magnitude.

4. Investigation of Engineering Geological Properties of Transported Tropical Red Soil

4.1 Case Study Area

4.1.1 Introduction

The Republic of the Congo is a relatively scarcely populated country, located in central Africa between the parallels of 4° north and 5° south and the meridians of 11° and 19° east, and is bordered (clockwise) by Gabon, Cameroon, the Central African Republic, the Democratic Republic of the Congo (DRC), Cabinda (Angola) and by the Atlantic Ocean in the southeast (Figure 4-1). The country has an approximate surface area of 342 000 km². Relief is dominated by the western Congo basin and its surrounding plateau. The Mayombe highlands, the Niari-Nyanga depression, the Cataracts and Batéké plateaux separate the coastal zone from the large inland Congo Basin (Beernaert, 1999).

The proposed Mayoko Iron-Ore Project (hereafter referred to as “the study area”) is located approximately 300 kilometres northeast of the port city of Pointe-Noire in the Niari province in the southwest of the Republic of Congo (RoC), between 1° and 2° south of the Equator (Figure 4-1). The area can be accessed via the existing rail infrastructure, which is connected to the port of Pointe-Noire, or via the existing road network. The overall study area is further subdivided and discussed here in terms of the area of elevated elevation where mining activities will take place (“mining area”) and the lower-lying areas surrounding the mining area earmarked for proposed infrastructure development.

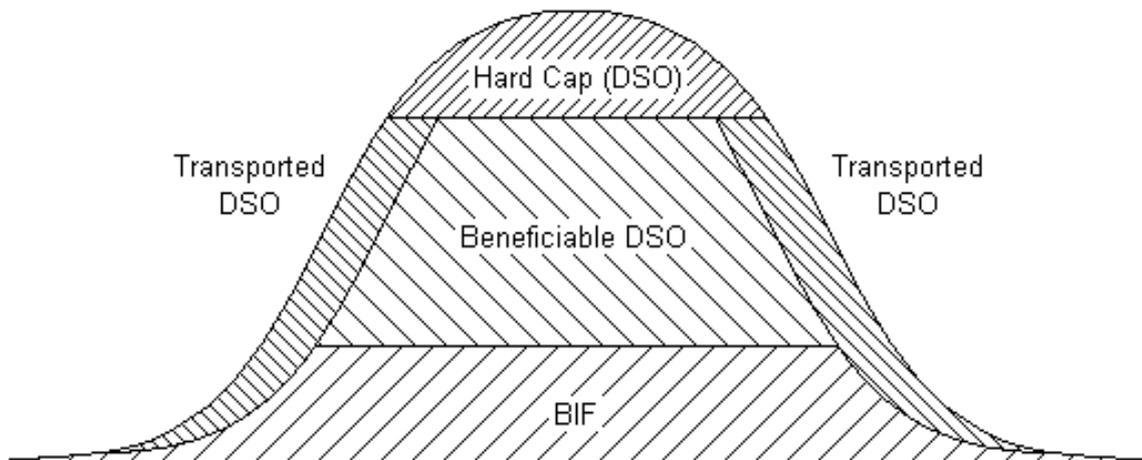
Mining activities will initially only focus on the Mt. Lekoumou deposit, represented by a topographic high-lying area of Banded Iron Formation (BIF) bedrock overlain by loose, unconsolidated iron-rich clayey gravel material (referred to as “transported ore”) and a hematite capping (Figure 4-2). The entire proposed mining area, apart from the mountain crest and uppermost slopes, is overlain by sandy silty clay colluvium of varying thickness.



Figure 4-1 Mayoko Iron-Ore Project location



a)



b)

Figure 4-2 a) Mt. Lekoumou and b) generalized section through the deposit (DSO = Direct Shipping Ore)

Infrastructure development will take place at selected sites in the lower-lying areas of the study area surrounding the Mt. Lekoumou mining area. The vegetation within the Mayoko region mainly consists of dense, evergreen tropical rainforests (Figure 4-3), with a total surface area of approximately 650 hectare of land that will be cleared for mining purposes.



Figure 4-3 Dense, tropical forest vegetation as found in the Mayoko region

4.1.2 Topography

Relief of the South West Congo comprises a series of parallel, southeast-northwest running mountainous ridges, which contain dissected plateau remnants of older planation periods. The Mayombe highlands and the Chaillu *massif* are prominent topographic units, with the syncline of

the Niari-Nyanga Depression lying between them. Parallel chains of hills rise from this low-lying area and average altitudes range from approximately 500 to 900 metres above mean sea-level (m.a.m.s.l).

Only the southern part of the Chaillu *massif* stretches to within the boundaries of the Republic of the Congo, and is bordered by the Bouenza river in the southeast and the N’Gounié, a tributary of the Ogowé river, in the west. Orientated in a north-westerly direction, the *massif* is about 500 km long and 150 km wide.

Rivers and streams tend to flow southeast in the valleys between ridges. The highest elevations reach 903 m.a.m.s.l. in the Bounou Mountains, keeping approximately this height on the watershed chain between the Louessé and the Mayoko region. The usually convex slopes become steeper further down slope and the valley bottoms are generally narrow. Only the main rivers (Lékoumou, Loyo, Louengo and Lélali) have alluvial plains (Beernaert, 1999).

The topography of the study area is undulating, representing a current geological erosional phase with Mt. Lekoumou (representing the uplift and tilting to which the BIF was exposed) forming the highest regional elevated point at approximately 820 m.a.m.s.l. The river valley sections are most often “V” to “U” shaped, with river valley banks being initially very steep rising from valley floor changing to sub vertical before flattening out towards the upper slopes and crests.

A Lidar elevation mapping survey of the mining area was completed early March 2012. The elevation measurements from the survey were used to create a Triangulated Irregular Network (TIN) Digital Elevation Model (DEM) in Arc GIS 10.1® (Figure 4-4).

From the slope topography presented in the DEM model and the distinct changes observed in the investigated soil profiles during the site investigation, it was possible to subdivide the slopes in the mining area into four main categories:

- Crest (approximately 820 m.a.m.s.l);
- Upper Slope (approximately 810 m.a.m.s.l);
- Middle Slope (approximately 720 m.a.m.s.l) and
- Lower Slope (approximately 650 m.a.m.s.l).

The transition between the different categories of slope listed above is often gradual and not clearly defined, and consequently reference is rather made hereafter to the transition zone between categories (for example “lower-to-middle slope”, “middle-to-upper slope” and upper slope-to-crest) than to the specific slope position itself.

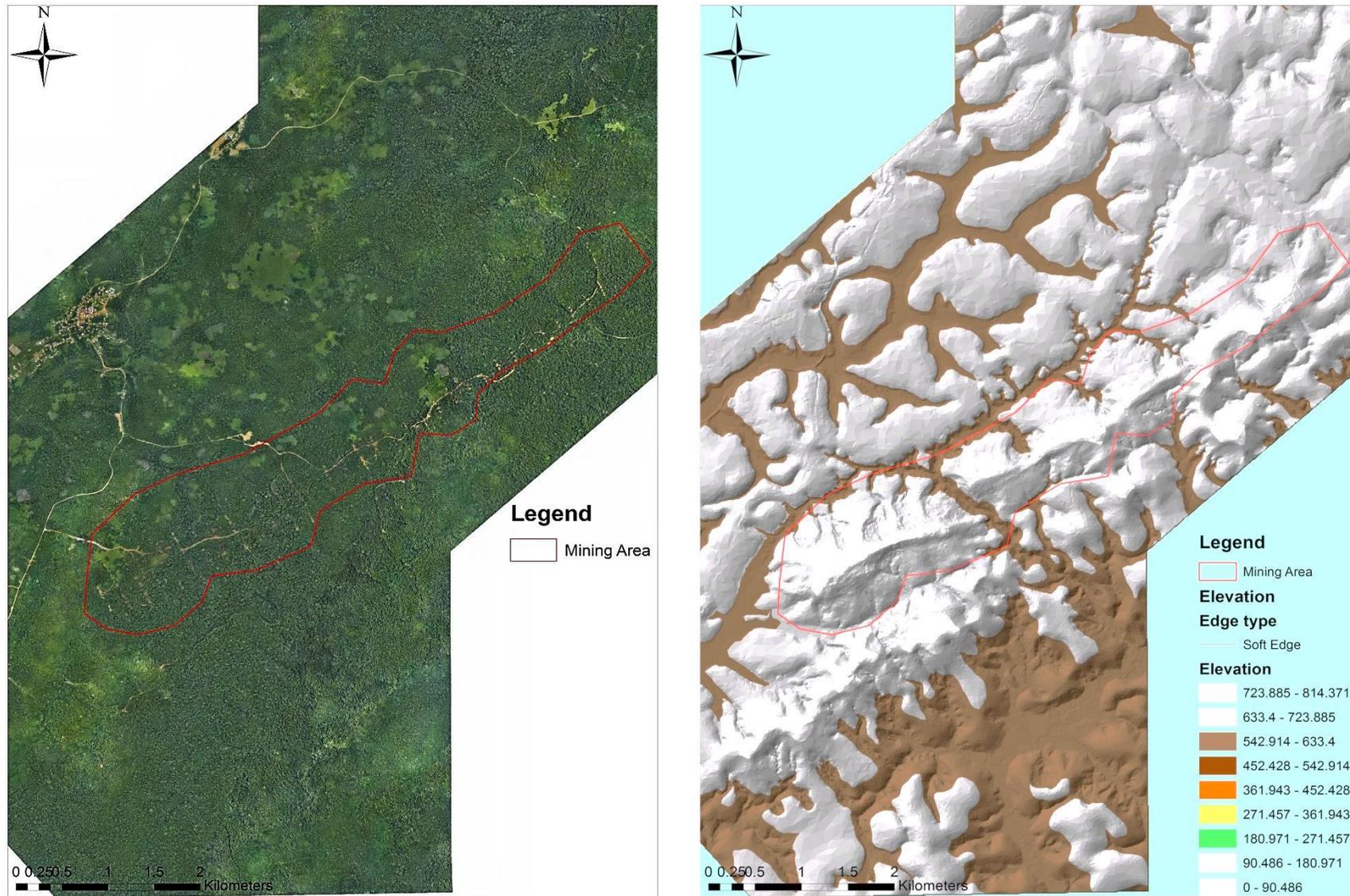


Figure 4-4 Lidar aerial photograph (*left*) and TIN (*right*) of the study area highlighting the position of the mining area

4.1.3 Climate

The predominant characteristic of the equatorial climate of this region is the absence of any seasonal winds, the lack of a well-defined dry season and consistently high humidity and temperatures. The study area is situated on the transition of the Tropical Wet and Tropical Wet and Dry zones as defined in terms of Köppen's Climatic Classification System (Shahin, 2002). Mean Annual Precipitation (MAP) levels of 1600 to 3200 mm/year has been recorded for the region, with dual peak wet seasons. The lower wet season occurs from March through May, while the higher equivalent occurs between August and November with an approximate rainy season of 9-10 months. The mean monthly precipitation levels reported by Golder Associates (2012) are summarised in Table 4-1 below and correlate well with the rainfall figures reported by Shahin (2002).

Table 4-1 Average monthly rainfall recorded for the Mayoko region (Golder Associates, 2012)

Month	Mean monthly rainfall (mm)
January	130
February	133
March	187
April	188
May	155
June	104
July	103
August	172
September	177
October	172
November	257
December	118
MAP	1896

Tropical Wet and Dry climates are characterised by distinct dry seasons and occur at the outer tropical zone. African equatorial air movement is dominated by a system of permanent pressure belts – the Inter Tropical Convergence Zone flanked to the north and south by two subtropical high pressure (descending air) belts. Air movement as a result of relative movement of these belts to the earth is the source of precipitation in the study area (Shahin, 2002). The only areas receiving greater amounts of precipitation (>3200mm) in Africa are two narrow belts along the Cameroonian, Nigerian and Liberian coastlines. Evapotranspiration is estimated to not be less than 1250 mm/year (Shahin, 2002). The study area therefore, contrary to the majority of Africa, receives a surplus of approximately 350 mm to 1950 mm of the MAP levels of 1600 to 3200 mm/year.

4.1.4 Geology

4.1.4.1 Regional Geology

The Republic of the Congo is largely underlain by rocks of Archean and Neoproterozoic age, with the eastern part of the country being largely covered by Quaternary alluvial sediments. The north-western part of the Congo Craton comprises a broad basement tilted upwards, constituting the foreland of the younger Congolian mobile belt. The mobile belt occurs as a vast granitoid *massif*, namely the Chaillu *Massif*, through Gabon to southern Cameroon (Schlüter, 2006).

The Chaillu *Massif* is characterized by north-south trending foliation, and contains two different types of granitoids, namely grey granodioritic to biotite amphibolite types and pink, potassium-rich migmatites. The granitoids are characterized by relicts of schists and greenstones that have not completely been transformed by granitization (Schlüter, 2006). Such a relict greenstone belt is preserved at the study area site.

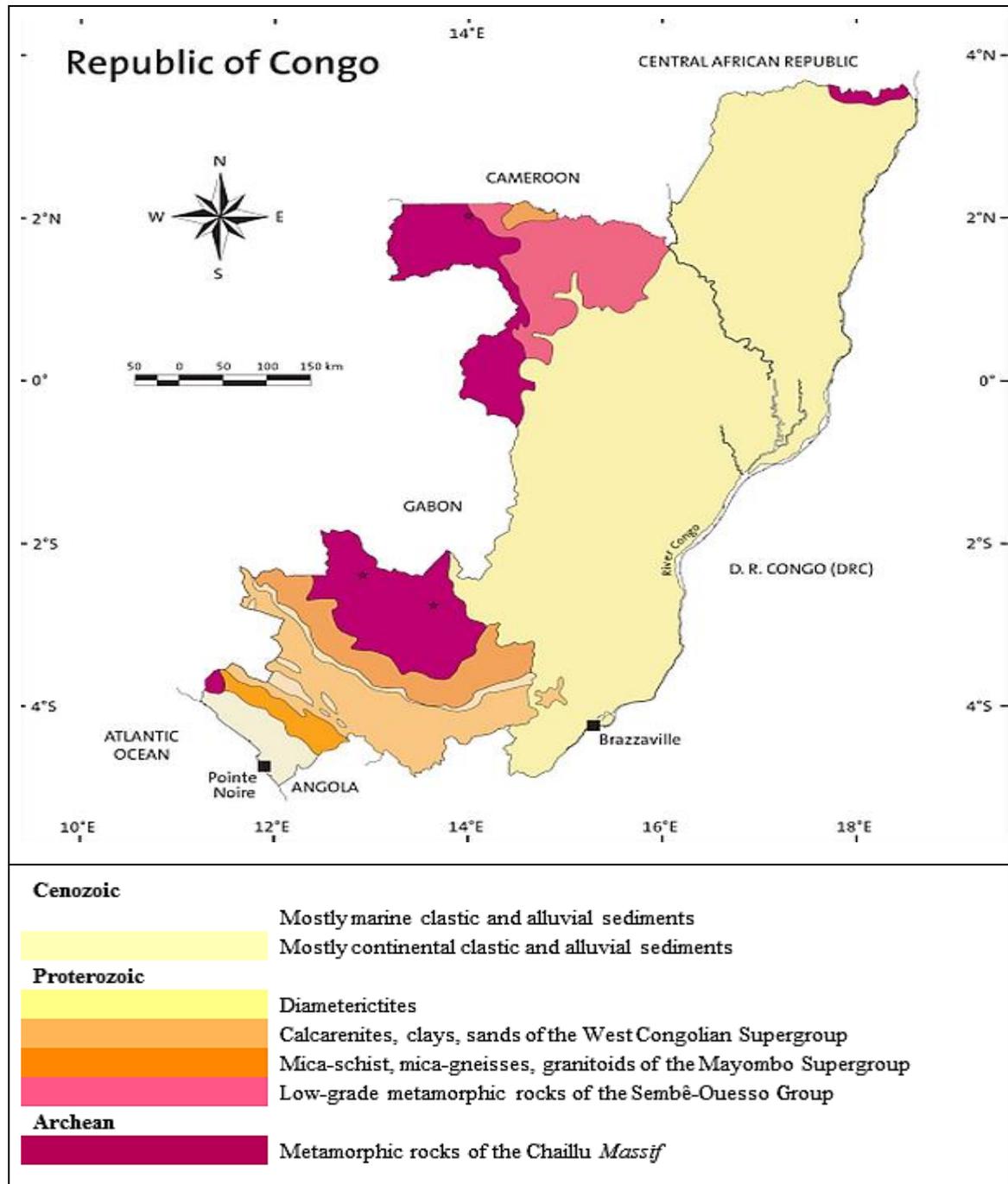


Figure 4-5 Regional geological map of the Republic of the Congo (Schlüter, 2006)

4.1.4.2 Local Geology and Stratigraphy

Due to the nature and location of the project site, the amount of existing site specific geological information is largely limited to the information gathered during the exploration and geotechnical drilling phases of the project.

The relict greenstone belt preserved at Mayoko predominantly consists of sub-vertically orientated Banded Iron Formation (BIF) and amphibolite surrounded by biotite-rich granitic gneisses (Figure 4-6). A prominent northwest-southeast trending strike-slip fault intersects Mt. Lekoumou, subdividing the mining area into 2 regions; namely Mt. Lekoumou South and Mt. Lekoumou North.

Three main phases of BIF can be recognised within the mining area (Figure 4-7). These are distinguished based on their relative state of weathering and enrichment in iron through the leaching of silica and bases. These phases comprise the Supergene Hematite (“transported ore”), Enriched BIF and Fresh BIF. The BIF is surrounded by amphibolite schist country rock, with a sharp contact between the two lithologies in the south, but separated by the northeast-southwest trending fault zone in the north. The general characteristics identified from the available exploration drilling data are summarised in Table 4-2 below for each phase of the BIF and the amphibolite schist.

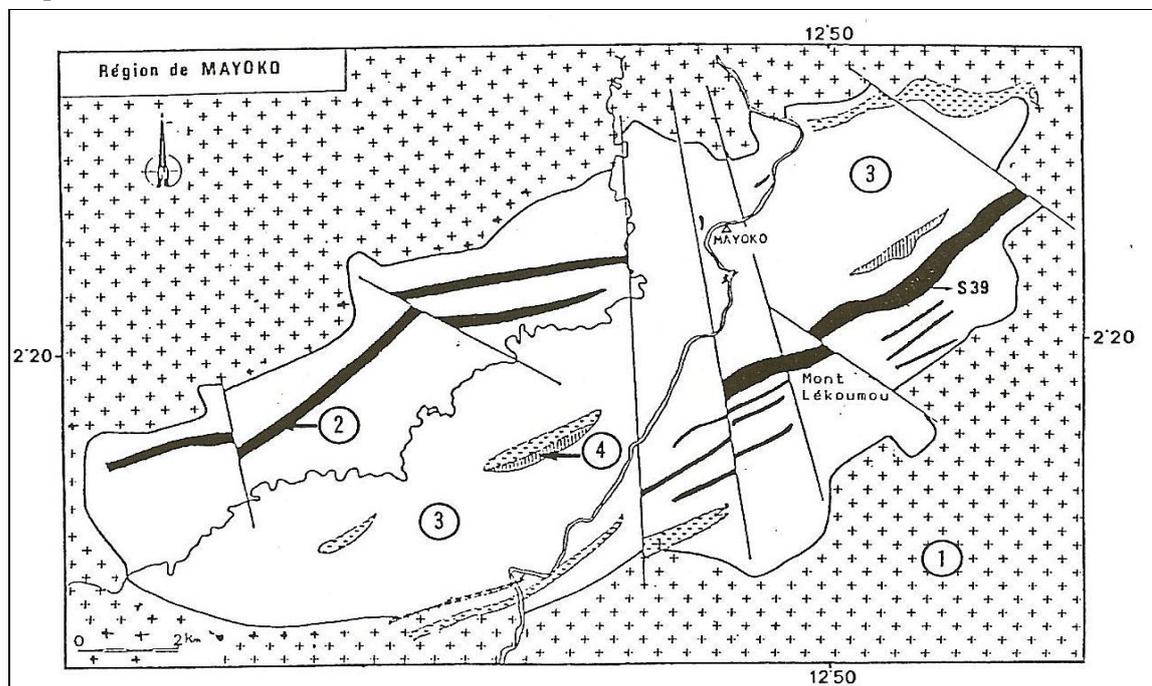


Figure 4-6 Schematic illustration of the Mayoko region geology where 1) granitic gneiss, 2) BIF, 3) amphibolite schist and 4) metasedimentary rocks (Boineau, 1956)

ENGINEERING GEOLOGICAL EVALUATION OF TRANSPORTED TROPICAL RED SOILS

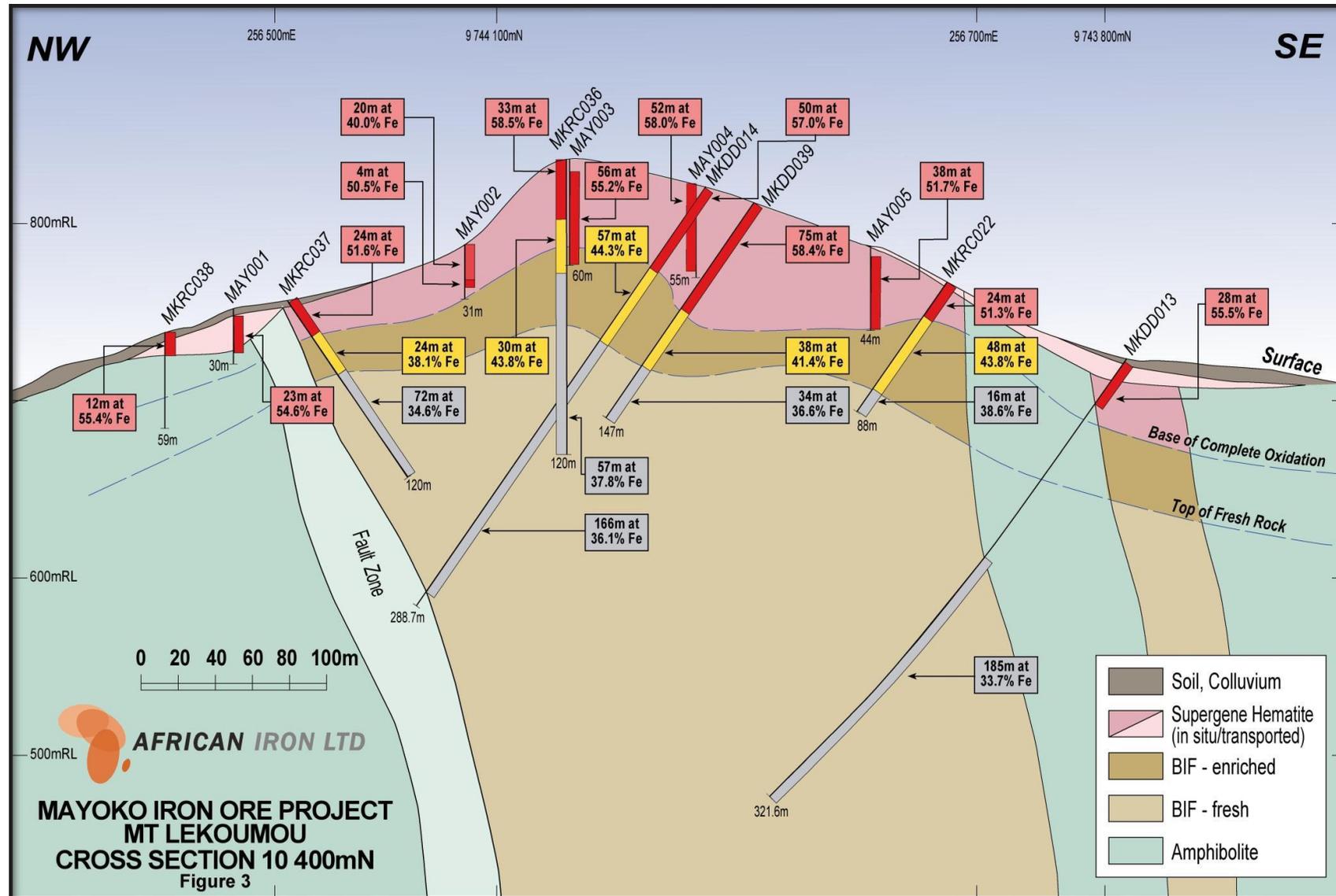


Figure 4-7 Cross-section through Mt. Lekoumou mining area (Exxaro Resources, 2013)

Table 4-2 General summary of characteristics of main rock types found in mining area

Phase	Description
	<p>Supergene Hematite (“transported ore”) – hematite/goethite or goethite/limonite/gibbsite blend. Abundant angular fine to coarse gravel in a moist, dark red, sandy clay matrix.</p>
	<p>Enriched BIF - mixture of hematite and magnetite. The enrichment is mainly the result of the weathering and depletion of silica. The result is a fairly friable BIF, which becomes harder downwards as weathering diminishes.</p>
	<p>Fresh BIF - represented mainly by alternating layers of magnetite and silica, hard rock.</p>
	<p>Amphibolite Schist– Dark greyish black banded light and dark grey, abundant quartz veins and garnet phenocrysts.</p>



Figure 4-8 Foliated BIF in exposed cliff face on northern slope of Mt. Lekoumou

4.2 Methodology

In an effort to effectively quantify and evaluate the engineering characteristics of the unconsolidated material which covers the study area, a number of detailed geotechnical investigations were completed. This include the profiling, logging and sampling of soil profiles and geotechnical boreholes on both the northern and southern slopes of the mining area, as well as in those areas proposed for infrastructure development. A detailed laboratory testing regime on all the collected samples was completed in an effort to establish and quantify the material characteristics and expected geotechnical behaviour. This includes the variation in characteristics of the transported tropical red soils throughout the study area i.e. both inside the mining area, where drainage on the slopes of Mt. Lekoumou is believed to be good, and those poorly-drained, lower lying areas outside the mining area where mining infrastructure development will take place.

The tropical red soils considered in this study are predominantly transported colluvial materials (“Hillwash”), formed through the intense chemical weathering and transport of the weathered product down slope of the study area. For the purpose of this thesis, the investigated materials are referred to as tropical “red” soils in order to distinguish this material from the other broad category of tropical soils, namely tropical “black” soils. In reality however these materials vary in colour (between shades of red, orange, yellow and brown), microstructure, mineralogy and grain size distribution.

Due to the aim of the study emphasis is primarily placed on the geotechnical characterization of the colluvium horizons, which classify as the traditional “tropical red soils”.

4.2.1 Profiling and Sampling

All soil profiles were investigated and described from cut faces during the construction of exploration drill pads and access roads between March and July 2013, as well as from the test pits excavated in the proposed foundation footprints of the planned mine infrastructure. The profiles varied in thickness between 0.5 m and 6.0 m, depending on the slope angle of the area and subsequent cutting and test pit depth. The profile positions were recorded using a hand held GPS instrument, which has an accuracy of about 3 metres and profiled according to the guidelines generally used in South Africa. Samples were collected from the various horizons identified in the profiles in an effort to investigate both the lateral variation (if any) in material properties over the mining area as well as the vertical variation in properties with the change in the elevation of the landscape.

Profiling and sampling of the cuttings took place immediately or very shortly after excavation and very little change in the natural moisture content of the material is expected to have occurred between excavation and profiling or sampling. The first 100 mm of the excavated profile face was removed prior to profiling and sampling using the geological pick and spade to obtain material representative of the *in-situ* conditions.

Undisturbed samples were retrieved using a geological pick and spade. The samples were tightly wrapped in multiple layers of cling film to prevent the block samples from disintegrating, then wrapped in tinfoil and again wrapped in multiple layers of cling film again in order to preserve the natural moisture content of the material during transportation of the samples back to South Africa (Figure 4-9).



Figure 4-9 Example of undisturbed sample (*left*) and bulk disturbed sample (*right*), both collected from the mining area

The size of the undisturbed block samples cut in the field varied depending on the material characteristics and the ease with which an intact block of material could be cut and removed without breaking. It was ensured that, due to the sensitivity of the tropical soil structure to sample disturbance, the dimensions of the retrieved undisturbed samples were often much larger than those typical undisturbed samples of soils from more temperate climates. This was done so as to ensure that laboratory testing specimens could be cut from the relatively undisturbed centre of the retrieved block.

Bulk disturbed samples (40-70 kg) and small disturbed samples (3-5 kg) were collected in an effort to evaluate the grading, Atterberg Limits, chemical and compaction characteristics of the material. These disturbed samples were sealed in heavy duty plastic sample bags, together with separately sealed natural moisture content samples, and transported back to South Africa for laboratory testing.

Four distinct horizons were identified throughout the study area (Figure 4-10). These comprised a horizon of slightly organic topsoil, a dark red brown colluvium horizon, a yellow brown colluvium horizon and transported ore. A typical description of each of these horizons is included below:

Slightly Organic Topsoil: Moist, dark brown, soft, micro-shattered, pinhole voided and voided with open root channels, silty clay with scattered fine-medium roots.

Dark Red Brown Colluvium: Moist, dark red brown, soft, micro-shattered and pinhole voided sandy to silty clay with scattered fine roots throughout.

Yellow Brown Colluvium: Moist, yellow brown, soft, micro-shattered and pinhole voided sandy to silty clay with scattered fine roots throughout.

Transported Ore: Abundant dark purple red, sub-rounded to sub-angular, fine to coarse Fe-ore gravel, cobbles and laterized boulders in a moist, dark red brown, silty clay matrix. Overall consistency is loose.

Despite the variation in colour description, both the dark red brown colluvium and yellow brown colluvium classify and are referred to as typical “tropical red soils”. The presence of transported ore and dark red brown colluvium horizons were found to be restricted to the upper slopes of the mining area only, with the lower-lying areas across the entire study area being covered solely by the slightly organic topsoil and yellow brown colluvium horizons at shallow depths. The presence and extent of the aforementioned identified horizons varied significantly, depending largely on the elevation and position of the profile in the slope (Figures 4-11 and 4-12).



Figure 4-10 Typical soil horizons (clockwise from top left); slightly organic topsoil, dark red brown colluvium, yellow brown colluvium and transported ore

ENGINEERING GEOLOGICAL EVALUATION OF TRANSPORTED TROPICAL RED SOILS

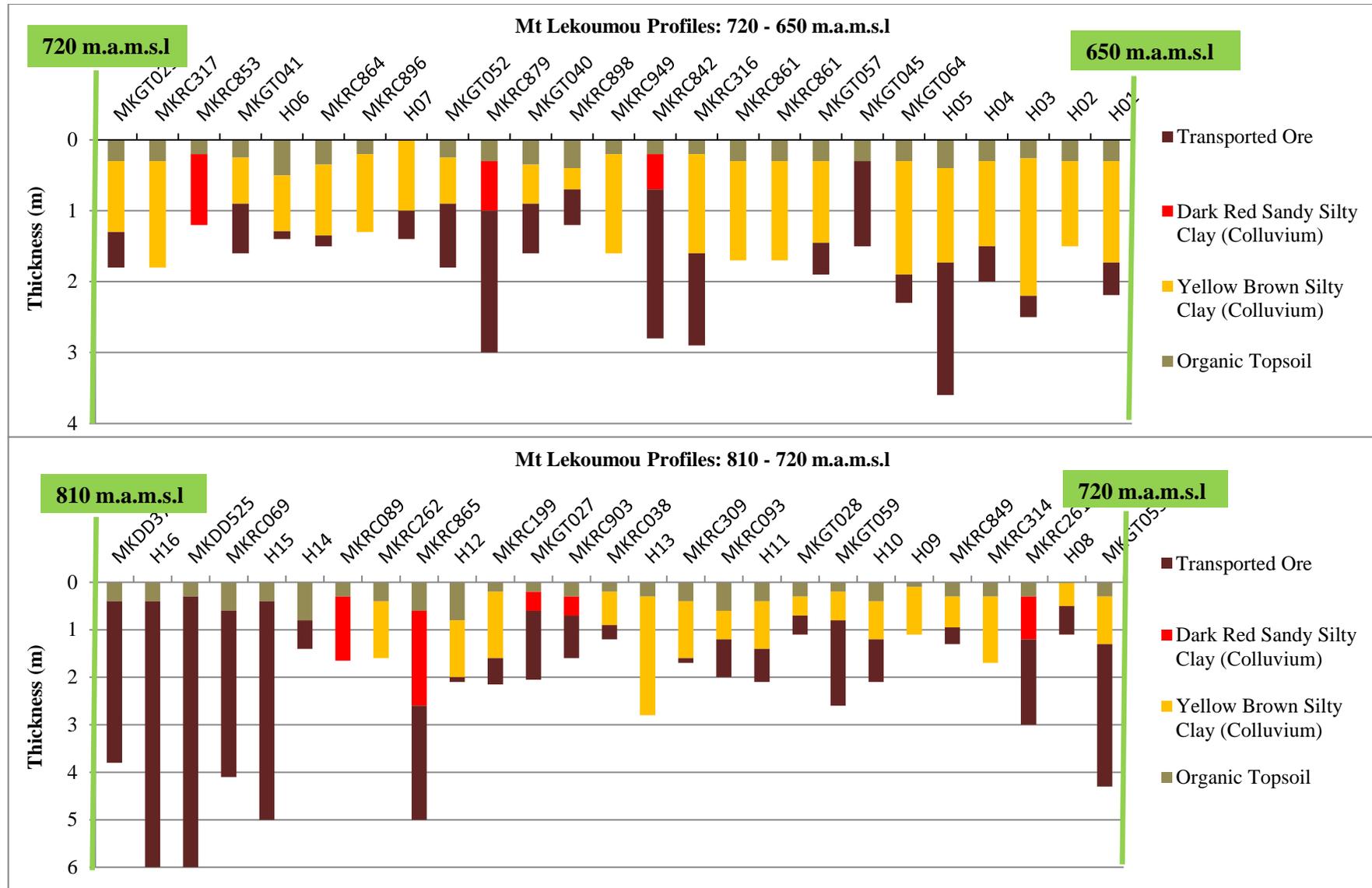


Figure 4-11 Generalized soil profiles – Mt. Lekoumou (northern slope)

ENGINEERING GEOLOGICAL EVALUATION OF TRANSPORTED TROPICAL RED SOILS

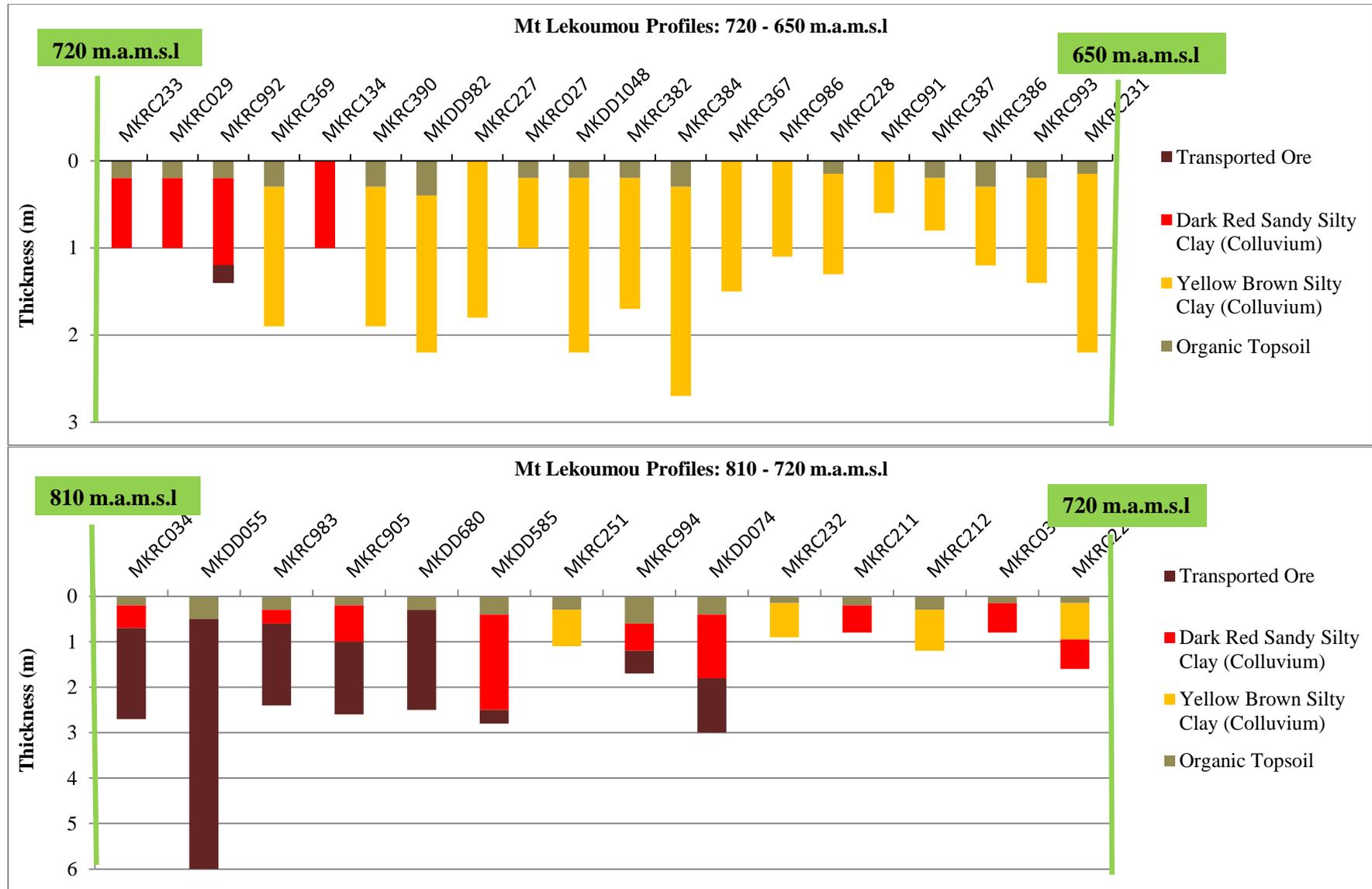


Figure 4-12 Generalized soil profiles – Mt. Lekoumou (southern slope)

The variation in the thicknesses of the soil profile and individual horizons across the Mt. Lekoumou area was found to vary according to the elevation of the profile, as well as the position of the particular profile on the northern or southern slope. The average horizon thickness with the standard deviation (STDEV) of the topsoil and colluvium horizons for each of the elevation intervals on the northern and southern side of the Mt. Lekoumou mining area are summarised in Table 4-3 below.

Table 4-3 Average total thicknesses of individual horizons on northern and southern slopes of Mt. Lekoumou

Horizon	Average Horizon Thickness (m)			
	Southern Slope (650-720 m) Lower-Middle Slope	Southern Slope (720-810 m) Middle-Upper Slope	Northern Slope (650-720 m) Lower-Middle Slope	Northern Slope (720-810 m) Middle-Upper Slope
Slightly organic topsoil	0.23	0.29	0.30	0.38
STDEV	0.069	0.138	0.083	0.179
Frequency (%)	80	100	96	96
Dark red brown colluvium	1.00	0.84	0.73	1.16
STDEV	0.115	0.559	0.252	0.679
Frequency (%)	20	60	8	14
Yellow brown colluvium	1.40	0.81	1.21	1.02
STDEV	0.534	0.063	0.382	0.535
Frequency (%)	80	29	84	60

Based on the observations summarised above the following can be concluded:

- A distinctly thicker topsoil horizon can be expected on the upper slopes of the northern face of Mt. Lekoumou compared to the average thickness of this horizon in other parts of the mining area;
- The colour and visual characteristics of the overall soil profile varies distinctly with elevation and position in the landscape. The middle-to-upper slope (810-720 m.a.m.s.l) soil profiles are characterized by the presence of thick transported ore horizons which a) either comprise the entire investigated soil profile (particularly near the crest of Mt. Lekoumou) or b) is overlain by a slightly organic topsoil and a dark red brown sandy silty clay colluvial horizon.
- Moving down the slope sees the distinct development of a yellow brown colluvium horizon from the middle-to-lower slopes (720-650 m.a.m.s.l) on both the northern and southern sides of Mt. Lekoumou. This can be explained by the secondary development of iron hydroxides and a subsequent decrease in the hematite content of the material. The transported ore horizons are not as frequently exposed in the lower profiles and, where exposed, are overlain by horizons of the yellow brown colluvium and topsoil.
- The average thicknesses of the colluvium horizons are similar in the middle-to-lower and the middle-to-upper slopes, but sees a variation in the frequency of how often these horizons occur

in the investigated profile. Yellow brown colluvium horizons were found in 45% of the middle-to-upper slope profiles, as opposed to 82% of the middle-to-lower slope profiles. Conversely, dark red brown colluvium horizons were found to be present in 37% of the middle-to-upper slope profiles, but only in 14% of the middle-to-lower slope profiles.

- All test pits excavated in the low-lying areas of the proposed footprint positions of the planned mine infrastructure were characterized by yellow-brown, in some instances weakly ferruginized, colluvium horizons grading into highly to completely weathered amphibolite schist bedrock with depth. These were found to be largely similar in characteristics as the yellow-brown colluvium described in the middle-to-lower slopes of Mt. Lekoumou, but at a further advanced state of weathering and subsequently higher iron sesquioxides content.

A distinct variation in the structure of the colluvial material was noted wherever the natural moisture content of the material was influenced by the mining and exploration activities on site, largely due to the removal of vegetation and excavations. In such areas the sandy silty clay tends to develop a micro-shattered structure, forming aggregates or “peds” approximately 1 – 10 mm in diameter. The aggregates were found to be partially to well-cemented, in some instances it was impossible to break down to the developed soil structure even after being soaked in water (Figure 4-13).



Figure 4-13 Characteristic shattered material structure of partially dried-out dark red brown colluvium

4.2.2 Geotechnical Drilling

Rotary core boreholes were set out and drilled at pre-determined locations across the study area in an effort to establish the depth and characteristics of the intact bedrock beneath the proposed infrastructure foundation footprints (Figure 4-14).

Core recovery of the transported ore was found to be poor due to the nature of the material (coarse gravel in a clayey matrix). In an effort to improve the core recovery and reduce the amount of fines that are “washed out” during drilling, the amount of drilling fluids used during drilling was significantly reduced. The best recoveries were achieved using as little as 1.5 litres of water per metre drilled and by shortening the drill run lengths to 0.5 m. The disadvantage of employing this technique is that by using less fluid during drilling the life of the drill bits are significantly reduced. The friction induced from drilling with such limited amounts of fluids lead to very high temperatures at the drill bit interface, leading to cracking and fracturing of the bits.

Standard Penetration Tests (SPT) were carried out according to BS 1377:1990 (Part 9). The tests were carried out in an effort to correlate the *in-situ* consistency of the material to the characteristics determined from laboratory testing. The SPT is an *in-situ* dynamic penetration test commonly used to provide soil consistency and give an estimate of the stiffness of the soil. The sample tube is driven 75 mm into the ground and the number of blows needed for the tube to penetrate each 75 mm up to a depth of 450 mm recorded. The first two readings are discarded as “settling in” on the loose material left at the bottom of the hole. The sum of the blows for the following four sets of 75mm penetrations is termed the “standard penetration resistance” or the “N-value”. Testing was carried out to the contact between the colluvium and transported ore, from where the amount of gravel and cobbles in the transported ore would have significantly influenced the determination of the N-value (Figure 4-15).

The typical ground profile described in the boreholes comprised sandy to silty clay colluvial soils underlain by either residual amphibolite schist (outside mining area) or residual BIF (Mt. Lekoumou mining area). Laterite, of varying degrees of development (varying from abundant fine to coarse, rounded laterite nodules in a dark red clayey matrix to well-cemented laterite boulders in a dark red clayey matrix) was found to be present in a number of boreholes, typically at or near the lower contact of the transported ore with the underlying residuum. Laterite development can be considered as a direct function of the leaching and precipitation of sesquioxides with fluctuating soil moisture content and is considered indicative of seasonal fluctuations in groundwater levels and/or the presence of a “perched” water table. Rain water is expected to drain from surface through the more permeable unconsolidated colluvium and transported ore and, upon reaching the contact of the lower, less permeable residual amphibolite and BIF, start ponding and flowing in a lateral direction. The general process is schematically illustrated in Figure 4-16.



Figure 4-14 Rotary core drilling rig on site



Figure 4-15 SPT Apparatus used during geotechnical drilling

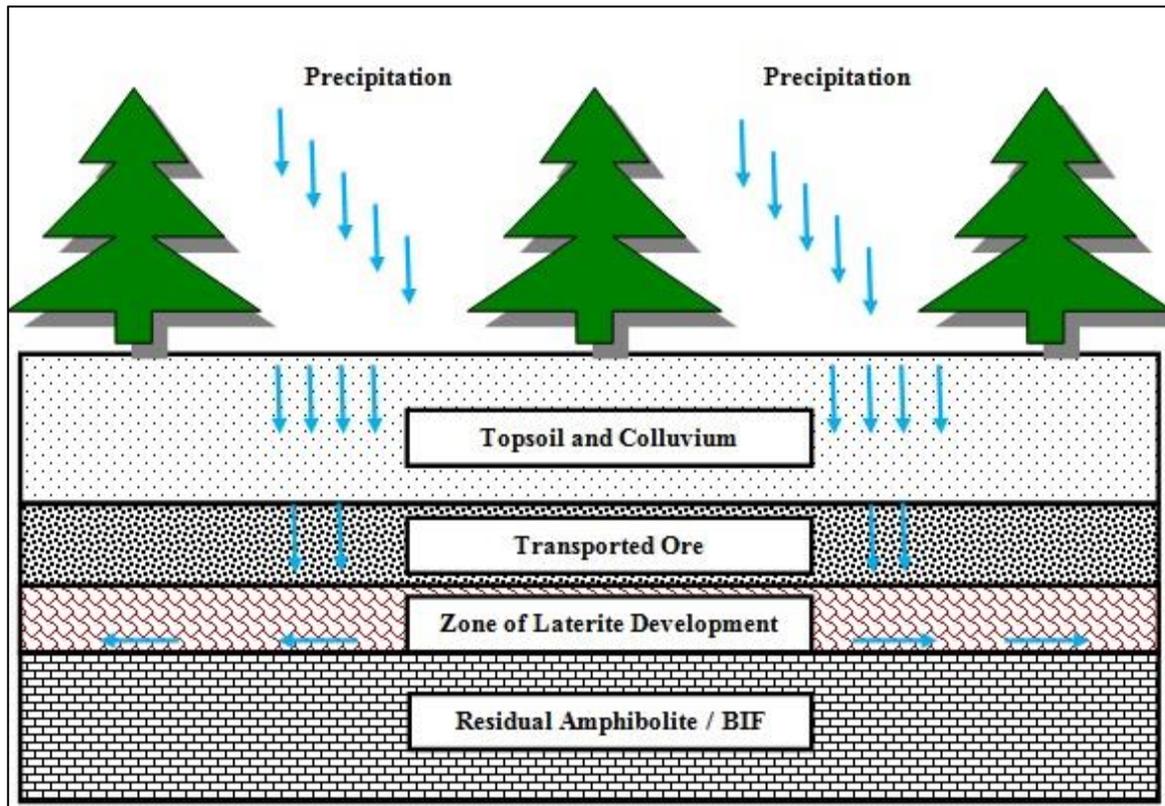


Figure 4-16 Schematic illustration of proposed process of laterite development at the overburden / residuum contact

4.2.3 Laboratory Testing

Conventional engineering classification tests are concerned with the determination of the index properties of soils. Index property tests were developed for primarily temperate soils and are usually sufficient for this purpose. The properties can be easily assessed, such as their particle size distribution, plasticity limits or density. Test procedures for the determination of these index tests are defined in the British and ASTM standards. Conventional index tests are carried out on disturbed or remoulded soil samples and are thus independent of the state of the *in-situ* characteristics of the soil (Northmore *et al.*, 1992). It can therefore be argued that, by regarding the particle size and consistency limits as the basis of soil classification, the influence of mineralogy, chemistry and origin of a soil on its mechanical behaviour is adequately measured by these simple index tests (Schofield and Wroth, 1968).

This cannot necessarily be assumed to be the case for tropical red soils. The inherent mineralogical and chemical composition and structure (in the case of residual soils) of the material has a pronounced influence on the engineering behaviour and measurement of the index and mechanical properties of tropical red soils. Because of the mineralogical composition of tropical red soils, even partial air-drying at ambient laboratory temperatures may change the structure and physical properties of these soils. Some of these changes cannot be reversed when the soil is re-mixed with water. These structural changes are reflected in, sometimes drastic changes in the index properties derived from plasticity, shrinkage and particle size tests or specific gravity. Conventional laboratory testing procedures, as defined in the British and ASTM standards, are therefore not necessarily directly applicable to tropical red soils without some

modification or change in focus (Northmore *et al.*, 1992) to take into account the unique mineralogical and chemical composition and structure.

The collected disturbed and undisturbed samples (Section 4.2.1) of the yellow brown and dark red brown colluvial material were subjected to laboratory testing analysis at two different commercial geotechnical laboratories, as well as specialised chemical and XRD/XRF analytical facilities.

The list of requested analyses and tests, together with the relevant standard test methodology used to complete the test, is summarised in Table 4-4 and Table 4-5 respectively. Any specified variations to the standard test procedures listed below are discussed separately for the individual test.

Table 4-4 Summary of scheduled engineering laboratory test work and relevant standards

Test	Standard / Method
Natural Moisture Content	BS 1377-2 (1990)
Atterberg Limits – Plastic Limit	ASTM Designation D424-59
Atterberg Limits – Liquid Limit (Falling Cone + Casagrande Apparatus)	BS 1377-2 (1990)
Linear Shrinkage	California Test Method 228 - A
Particle Size Distribution (wet sieving)	AASHTO Designation T146 - 49
Particle Size Distribution (hydrometer)	ASTM Designation D422-3
Particle Density / Specific Gravity	BS 1377-2 (1990)
Consolidation	BS 1377-8 (1990)
Swelling Pressure	BS 1377-5 (1990)
Compaction – Proctor Effort	BS 1377-4 (1990)
Dispersion – Double Hydrometer	BS 1377-5 (1990)
Permeability (during Tri-axial Testing)	BS 1377-6 (1990)
Tri-axial Testing (Consolidated-Undrained)	BS 1377-8 (1990)
Matric Suction Measurements – Filter Paper	Hamblin (1981)
Soil Water Characteristic Curves	ASTM D6836

Table 4-5 Summary of scheduled chemical analyses and relevant standards

Determinant	Method	Method References
pH Value	Electrometric	W044-08-W (A.P.H.A. 4500-H ⁺ B)
Conductivity	Potentiometric	W044-04-0 (A.P.H.A. 2510 B)
Total Dissolved Solids	Gravimetric	W044-03-W (A.P.H.A. 2540 C)
Total Alkalinity	Titrimetric	Auto Analyser or A.P.H.A. 2320 B
Calcium	Atomic Absorption Spectrophotometry	W044-15-W (A.P.H.A. 3111 B)
Magnesium	Atomic Absorption Spectrophotometry	W044-01-W (A.P.H.A. 3111 B)
Potassium	Atomic Absorption Spectrophotometry	W044-01-W (A.P.H.A. 3111 B)
Sodium	Atomic Absorption Spectrophotometry	A.P.H.A. 3111 B
Carbonate Hardness	By Calculation	A.P.H.A. 2340 A
Chloride	Titrimetric or Mercuric Nitrate Titration	Auto Analyser or A.P.H.A. 4500-Cl C
Sulfate	Gravimetric	A.P.H.A. 4500-SO ₄ C
Nitrate	Colorimetric	EPA 352.1
Nitrate	Nitrate Electrode	Auto Analyser (A.P.H.A. 4500-NO ₃ D)
Fluoride	Ion Selective Electrode	A.P.H.A. 4500-F C
Acidity/ P Alkalinity	Titrimetric	Auto Analyser or A.P.H.A. 2310/2320 B
Arsenic, Selenium, Titanium, Aluminium, Nickel, Manganese, Iron, Vanadium, Zinc, Antimony, Lead, Cobalt, Copper, Total Chromium, Silicon, Tin, Zirconium, Bismuth, Thallium, Beryllium, Cadmium, Boron, Phosphorus, Phosphorus as Phosphate, Uranium, Molybdenum, Barium, Silver, Thorium, Lithium, (also Ca, Mg, Na, K)	ICP Quantitative Scan	A.P.H.A. 3120 B

4.2.3.1 Chemical and XRD/XRF Analyses

In order to effectively investigate the chemical characteristics of the tropical red soils in the study area and the spatial variation of these characteristics, a number of disturbed samples of the yellow and red brown colluvium material were subjected to chemical analyses.

Samples from various parts of the study area were selected in an effort to investigate both the lateral variation (if any) in chemical properties as well as the vertical variation relating to change in elevation of the landscape. A contact leach test at a solid to liquid ratio of 1:5 was undertaken using deionised water to determine the soluble constituents that are present in the soil material. The samples were added to shake flasks and agitated for 24 hours. The extract was then submitted to the following analyses:

- pH at 19°C;
- Cation Exchange Capacity at 25°C;
- Total Dissolved Solids;
- Alkalinity;
- Acidity;
- Electrical Conductivity and
- Chloride (Cl), Sulfate (SO₄), Fluoride (F), Nitrate (NO₃), Calcium (Ca), Magnesium (Mg), Potassium (K) and Sodium (Na).

A number of guidelines and indices exist for the determination of the potential corrosivity of material. However, these guidelines tend to be variable in their respective interpretation of the results. For the purpose of this evaluation the following guidelines and indices were consulted:

- California Department of Transportation (CDT) Division of Engineering Services Materials Engineering and Testing Services Corrosion Technology Branch (2012).
- South African Water Quality Guidelines Volume 4: Agricultural Use - Irrigation (DWA, 1996).

The CDT use a simplified assessment of corrosion potential which assumes that corrosion is possible where chloride concentration is 500 milligrams per litre (mg/l) or greater, sulphate concentration is 2000 mg/l or greater, or the pH is 5.5 or less.

The South African guideline considers the potential to result in corrosion or the accumulation of scale to be a function of the equilibrium saturation point of calcium carbonate in solution. When the water is supersaturated, scale tends to accumulate whereas if it is under saturated it tends to be corrosive.

A number of indices were considered to quantify the corrosivity or scaling tendency of water, including the Langelier Index, Ryznar Index and Aggressiveness Index. The Langelier Index provides a measure with which to assess the potential to be either scale forming or corrosive, and it is calculated from the concentrations of calcium, alkalinity and total dissolved salts, the pH and the temperature of the water. Corrosion may however, occur in low salinity soft waters as a result of the dissolution of lime.

The Ryznar Index is similar to the Langelier Index in that it attempts to quantify the relationship between calcium carbonate saturation state and scale formation, while the Aggressiveness Index examines the corrosive tendency of chloride and sulfate ions to steel structures.

Bulk representative samples of the yellow and red brown colluvium and transported ore soils were submitted for Qualitative and Quantitative XRD (X-Ray Diffraction) and XRF (X-Ray Fluorescence) analysis.

The samples are prepared for XRD analysis using the back loading sample preparation method. They are analysed using a PANalytical X'Pert Pro powder diffractom with X'Celerator detector and variable divergence- and receiving slits with Fe filtered Co-K α radiation. The different mineral phases are identified using X'Pert Highscore plus software.

The relative phase amounts or weight percentages are estimated using the Rietveld method with help of the Autoquan ProgrAmerican Amorphous phases, if present, are not taken into consideration in the quantification of phases that are present.

For XRF analysis, the samples are heated at 1000 °C to determine the Loss On Ignition (LOI). One gram of the roasted sample is then placed together with six grams of Li₂ B₄ O₇ into a Platinum/Gold crucible and fused. The samples are then analysed using an ARL9400XL+ spectrom where analyses are conducted using Quantas software. The Quantas software analyses for all elements in the periodic table between Na and U, but only elements found above the detection limits are reported. The results are also monitored and filtered to eliminate the presence of the flux, wetting and oxidising agent elements.

In addition to the analysis of the aforementioned samples, XRD and XRF analysis were also completed in 2012 as part of the metallurgical characterization of the transported ore. The objective for the metallurgical sampling and testing was solely to evaluate the spatial variability of the ore (excluding all sandy silty clay colluvium horizons) over Mt. Lekoumou. The material was scrubbed to remove clay material and slimes and then screened into different size products to determine the predominant compositions of various standard size fractions of the material. These size products are:

- Run of Mine: -63 mm
- Lumpy: 32 x 6 mm
- Fine: 6 x 1 mm
- Ultra-fine: 1 x 75 μ m
- Slimes: -75 μ m

4.2.3.2 Index Properties

Laboratory test analysis of the index properties of the colluvium samples was completed in 2013. The representative samples which were collected from both the mining area and those areas of infrastructure development outside the mining area were selected in an effort to investigate both the lateral variation (if any) in index properties over the study area as well as the vertical change with the change in the elevation of the landscape. This included evaluating the particle size

distribution (sieve and hydrometer analysis) and Atterberg Limits, Natural Moisture Content (NMC) and particle density (“specific gravity”).

Based on the visual observations made during the investigation on site, it was assumed that the material structure is sensitive to drying. It was therefore specified that testing was done at the natural or “as received” moisture content as far as possible. Where drying was required, this was done through either air-drying or oven drying at a maximum temperature of 65 °C. Selected sub-samples were dried at both 100°C and 60°C in an effort to confirm the sensitivity of the material to different intensity drying programs. The samples were further stored in a temperature and moisture controlled environment before and after testing in an additional effort to limit the amount of change in the moisture content of the material. Before the aforementioned testing commenced, a number of variations to the standard test procedures were specified in an effort to take cognisance of and make allowance for some of the previously discussed unique characteristics of tropical red soils.

i) Particle Size Distribution

The particle size distributions of the respective samples were determined using a combination of the wet sieving and hydrometer analysis standard test methods (BS 1377-2, 1990).

This was done in an effort to determine the percentage of different grain sizes contained within the soil from various parts of the study area as well as from different slope positions within the mining area. The mechanical or sieve analysis was performed at natural moisture content (no pre-drying) to determine the distribution of the coarser, larger-sized particles and the hydrometer method to determine the distribution of the finer particles (< 0.075 mm). The size of the test sample largely depends on the amount of soil fines, and 400g of material is generally deemed being sufficient for the determination of the grain size distribution of fine-grained soils. However, the samples submitted for particle size distributions testing from the study area all varied between 3 and 5 kg in an effort to ensure that all the required test work could be completed without returning to site.

The samples were first quartered down by means of a riffler to the required size and the weight of each sieve and the bottom collection pan to be used in the analysis recorded. The material needed to be sufficiently dry so that it can be sieved through the 0.425 mm sieve without clogging the sieve. In those instances where the quartered test sample was too wet or damp to achieve this, the sample was first air-dried for a given period of time until this part of the test could be completed successfully and then weighed and sieved through the 0.425 mm sieve. The material fraction retained on the 0.425 mm sieve was then transferred to a mortar where the remainder of the soil aggregations were broken down and any material that might have adhered to the coarser fractions dislodged. The material was then sieved through the 0.425 mm sieve again. The material retained on the 0.425 mm sieve was sieved through a stack of sieves consisting of the following: 63.0 mm, 53.0 mm, 37.5 mm, 26.5 mm, 19.0 mm, 13.2 mm, 4.75 mm, 2.0 mm and 0.425 mm, with additional 32.0 mm and 6.0 mm aperture sieves also added to the stack. This was done for the metallurgical settling analysis of the material, which is not included in the scope of this research. Sieving was done by means of a mechanical sieve shaker and the material retained on each sieve weighed and the masses recorded to determine the grain size distribution.

The distribution of particle sizes smaller than 0.075 mm was evaluated using a calibrated hydrometer. When the soil specimen is dispersed in water, particles settle at different velocities depending on their shape, size and weight. In order to reduce the amount of variables the following assumptions are made during hydrometer analysis (Allen, 1975):

- The drag force on each particle is due entirely to viscous forces within the fluid. The particles must be spherical, smooth and rigid, and there must be no slippage between them and the fluid.
- Each particle must move as if it were a single particle in a fluid of infinite extent.
- The terminal velocity must be reached very shortly after the test starts.
- The settling velocity must be slow enough so that inertia effects are negligible.
- The fluid must be homogeneous compared with the size of the particle.

Two different dispersing agents were used during hydrometer analysis in an effort to evaluate the effect of the dispersing agent on the particle size distribution results. The colluvium samples from the mining area were dispersed using a dispersant solution of 36 grams Sodium Pyrophosphate Decahydrate per litre of deionized water. Colluvium samples from outside the mining area were dispersed using a mix of 35 grams Sodium Hexametaphosphate and 7 grams Sodium Carbonate per litre of deionized water.

ii) Particle Density

The particle density or specific gravity of the colluvium samples was determined according to the procedure set out in BS 1377-2 (1990) using the small pycnometer method. Particle densities were determined from the natural moisture content of the material unless specified otherwise, and no pre-drying allowed. These values are reported without any units.

The test method is based upon the determination of the dry weight of a sample of the soil, and the weight of the same sample plus water in a container of known volume. The volume of the container is determined through the weighing of the container empty, and filled with water (Clayton, 1995).

Selected samples were split into three representative sub-samples (A, B and C) at the laboratory. The particle densities were determined for each of the sub-samples; at the natural moisture content or as-received moisture content (A), partially dried (B) and completely dried (C). The respective sub-samples were air-dried in an effort to minimize the effect of drying on any inter-layer moisture in the clay component. Northmore *et al.* (1992) concluded that even with the presence of hydrated clay minerals in many of the tropical red soils, no significant differences in natural moisture content determinations were found between samples oven dried at 50°C or 105°C. It was still deemed prudent in this case to complete the test through the least “aggressive” form of drying in an effort to confirm the effect of *in-situ* drying on the hydrated state of the clay minerals and mimic the most natural form of drying the materials might be subjected to in its natural condition. Selected sub-samples were therefore dried at both 100°C and 60°C over a 24-hour period in an effort to confirm the sensitivity of the material to different intensity drying programs.

iii) Bulk density

The bulk densities of the material were determined from the respective undisturbed tri-axial test specimens. Significant effort was put into the wrapping and sealing of these samples so as to ensure the natural moisture content of the material is preserved up until the point of testing. It was therefore assumed that the bulk density values calculated for these specimens represent the natural bulk density values. These were determined through (Northmore *et al.*, 1992):

$$\gamma_b = \frac{W_0}{V_0} \quad (\text{Eq. 4-1})$$

Where:

γ_b – Bulk density (Mg / m³)

W_0 – Original, pre-test weight of test specimen (g)

V_0 – Original, pre-test volume of test specimen (cm²)

iv) Void ratio

The void ratio of a soil can be defined as the ratio of the volume of voids to the volume of solids in a mass of material (Das, 2005), and can be expressed as:

$$e = \frac{V_v}{V_s} \quad (\text{Eq. 4-2})$$

Where:

e – Void ratio

V_v – Volume of the voids (m³)

V_s – Volume of the solids (m³)

The initial void ratio of the material (before testing and consolidation) is calculated using the dry density and particle density value as determined for the material (Das, 2005):

$$e = G_s \frac{\gamma_w}{\gamma_d} - 1 \quad (\text{Eq. 4-3})$$

Where:

e – Void ratio

G_s – Particle density / Specific gravity

γ_w – Density of water (kg/m^3)

γ_d – Dry density of sample material (kg/m^3)

Once the sample is inserted into the loading cell and consolidation is in progress, the change in the void ratio is determined by the change in sample height in relation to the original sample height.

v) Linear Shrinkage

Linear or one-dimensional shrinkage is a function of the change in length of a semi-cylindrical bar sample of remoulded soil when dried from the moisture content at which the material reaches its liquid limit, and is expressed as a percentage of its initial length (Northmore, 1994). This can be expressed as:

$$LS = \left(1 - \frac{L_D}{L_O}\right) 100 \quad (\text{Eq. 4-4})$$

Where:

L_D - Length of the oven-dried specimen (mm)

L_O - Length of the original specimen (mm)

vi) Natural Moisture Content

From the description in BS 1377 – 2 (1990), moisture is considered to be present in most soils. The amount of moisture, expressed as a proportion of the mass of the dry solid particles, has a profound effect on the soil characteristics and engineering behaviour. In this context a soil is only considered “dry” when no further water can be removed at a temperature not exceeding 110 °C. Determination of the moisture content is further required as a guide to classification of soils and to evaluate the behaviour of soils during construction. This is most commonly determined by using the oven drying method.

In clay-rich soils the clay minerals contain water in several complex ways. For example, a layer of 'adsorbed' water of very small thickness is held on the surface of the clay particles by strong electrical attraction forces. It cannot be removed by oven drying and is normally considered to be part of the solid soil grain. Water which is not tightly bound, such as capillary water held by surface tension and gravitational water which can move in the voids or pores between the soil grains, can be removed by oven drying and air-drying. Water which can be readily removed by oven drying is often referred to as 'free' water which is available to play a part in the engineering behaviour of the soil. In conventional soil testing, therefore, the moisture content relates only to the water which is removable by oven drying at 105-110°C. In some minerals water may also be chemically combined in the form of water of hydration, or interlayer water, within their crystal structure. In most temperate soils (with the exception of gypsum) this water is not removable by oven drying, but in tropical red soils it is often assumed that this interlayer water in the hydrated

clay minerals can be removed irreversibly at the conventional drying temperatures, leading to apparently anomalous moisture content measurements (Northmore *et al.*, 1992).

Based on these findings, the natural moisture contents of the material was determined by placing and weighing a sample of material in a container of a known weight and then placed in the oven at 65 °C (Figure 4-17). Longer drying times than usual are required to reach a constant weight at the lower drying temperatures. The material was then weighed again inside the container once a constant weight has been achieved and the change in weight recorded. Any change in weight of the total sample was assumed to be the result of a change in the free water moisture content. The moisture content, expressed as a percentage, is then determined using Equation 4-5 as expressed in Das (2005).

$$Mc = \frac{W_w}{W_s} \times 100 \quad (\text{Eq. 4-5})$$

Where:

Mc – Moisture content (%)

W_w – Weight of the water, equal to the difference in weight of the material before and after oven drying (g)

W_s – Weight of the solids, equal to the total weight of the material after oven drying (g)

The calculated moisture contents can then be related to the degree of saturation of the material using Equation 4-6, as well as playing a vital role in the evaluation of the influence of suction pressures on the behaviour of the material (Section 3-6).

$$Se = wGs \quad (\text{Eq. 4-6})$$

Where:

S = Degree of saturation (%)

e = Void ratio

w = Moisture content (%)

G_s = Specific gravity



Figure 4-17 Drying oven (*left*) and samples being dried (*right*)

Additional to the drying procedure at a “lower” temperature as discussed previously, the effect of different drying temperatures on the calculated natural moisture contents of five separate yellow brown and dark red brown colluvium samples were determined. Northmore *et al.* (1992) concluded that even with the presence of hydrated clay minerals in many of the tropical red soils, no significant differences in natural moisture content determinations were found between samples oven dried at 50°C or 105°C. Five samples of the colluvium material were split and dried at two different drying temperatures of 100°C and 60°C for 24-hours in an effort to evaluate the sensitivity of the material to oven drying at varying temperatures.

vii) Atterberg Limits

Depending on the initial moisture content of the soil, the material may exist in four possible states: solid, semi-solid, plastic and liquid. In each state the consistency and behaviour of the soil is different, consequently influencing the engineering properties of the material. The boundary between each state signifies a change in the soil's characteristics and behaviour at that specific moisture content, and these are known as the Atterberg Limits (Figure 4-18). A wide variety of the engineering properties of soils from temperate climates have been correlated to the liquid and plastic limits, and these Atterberg limits are also used to classify a fine-grained soil according to the Unified Soil Classification system or AASHTO system.

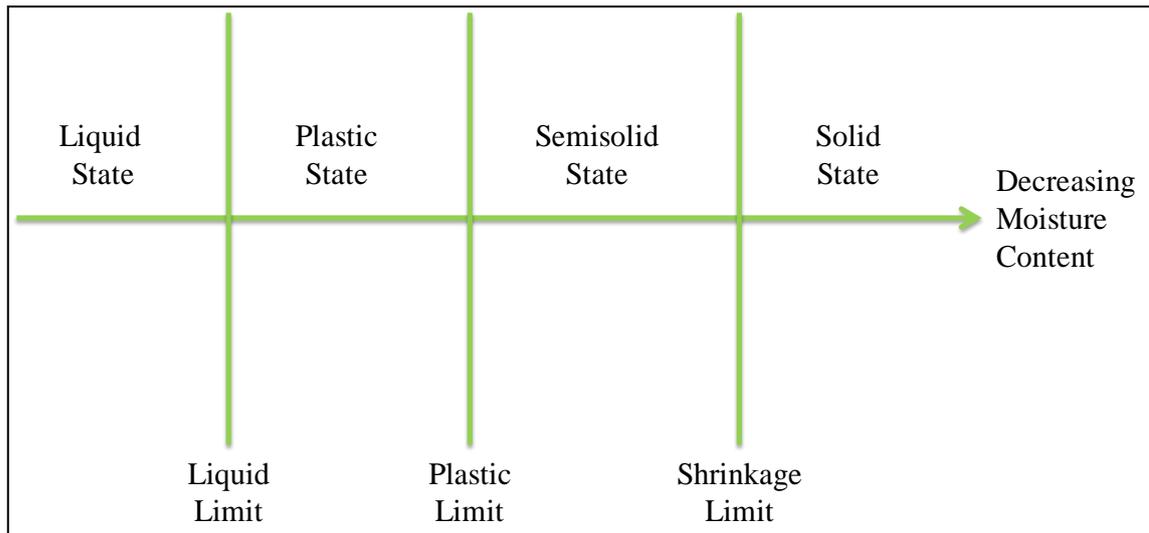


Figure 4-18 Relationship between soil moisture and various limit states (adapted from Das, 2005)

Liquid and plastic limit testing is carried out on remoulded mixes of wet soil passing the 0.425 mm sieve. Because even partial drying of the soil prior to testing may affect the structure and physical behaviour of some tropical soils due to irreversible dehydration of clay minerals, it was specified that testing should be completed at the natural or “as received” moisture content as far as possible and only air-dried where drying was necessary.

- Shrinkage Limit (SL)

The plastic limit is represented by the percentage moisture content at which point a soil on being dried ceases to shrink (or change in volume). The shrinkage limit values are not as often used in practice such as the LL and PL and were not determined as part of this research.

- Plastic Limit (PL)

The plastic limit is represented by the percentage moisture content at which point the soil passes from the plastic state to the solid state. The plastic limit is determined by hand-rolling a sample of remoulded soil into a ‘thread’, on a glass plate, until it begins to crumble due to gradual drying. The plastic limit is the moisture content of the ‘thread’ as it starts to crumble at a diameter of 3 mm. The range of percentage moisture content over which a soil is plastic is defined as the plasticity index (PI) and provides a measure of the overall plasticity of the material (Figure 4-19).



Figure 4-19 Evaluating the material plasticity in the field during the site investigation

- Liquid Limit (LL)

The liquid limit is represented by the percentage moisture content at which point the soil passes from a liquid to the plastic state. The liquid limits were determined using the falling cone penetrometer apparatus and according to the method as outlined in BS 1377-2 (1990), as well as the Casagrande method. Testing was done using both methods in an effort to evaluate the variability of the results obtained through the two different methods.

During the falling cone penetrometer test, the liquid limit of the soil can be defined as the soil moisture content at which a 30° cone of 80 gram weight sinks exactly 20 mm into a cup of remoulded soil in a 5 second time period. It was specified that cone penetrations should be determined for at least four different soil moisture contents. The cone penetration against the corresponding moisture content values was then plotted on a graph and a best-fit straight line drawn to determine the liquid limit (Figure 4-20).

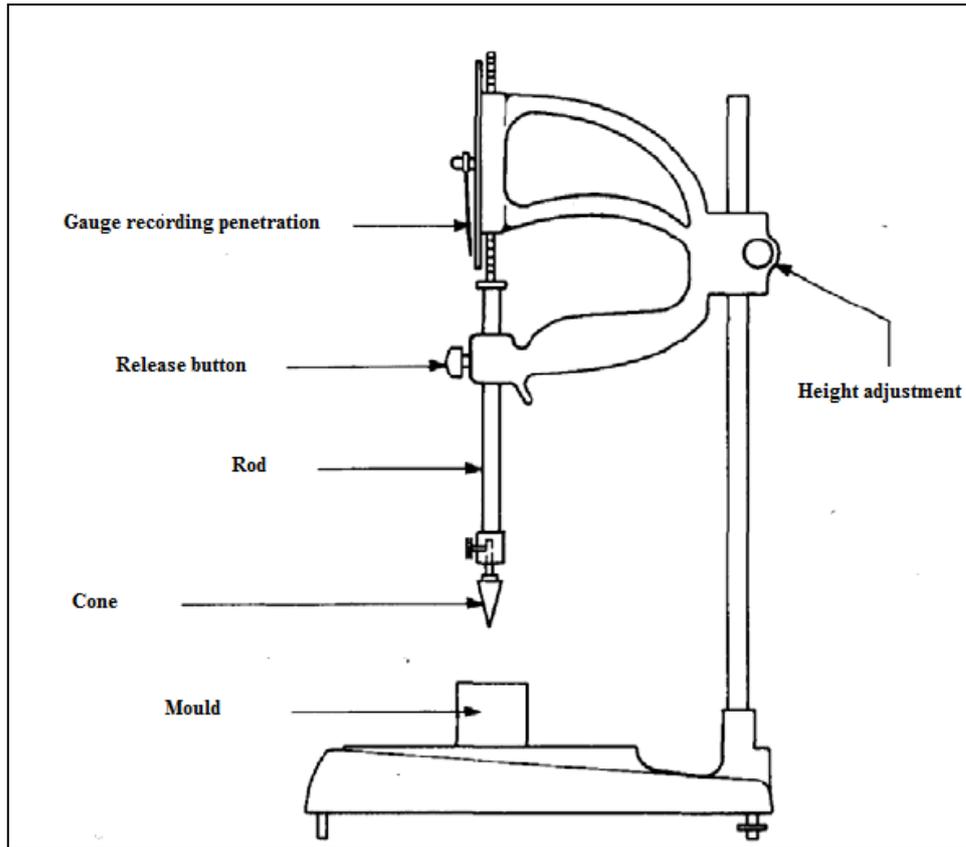


Figure 4-20 Falling cone penetrometer apparatus (adapted from Clayton, 1995)

When determining the liquid limit with the Casagrande apparatus, the base of the cup is filled with soil and a groove is then made through the soil to the base of the cup. The apparatus is set up to allow the metal cup to be raised repeatedly to a distance of 10 mm and dropped freely on to its rubber base at a constant rate of two drops per second. The liquid limit can be defined as the moisture content of a soil where 25 blows cause 13 mm of closure of the groove at the base of the cup (Figure 4-21). This point is determined by mixing soils to consistencies just wet and dry of the liquid limit and determining the liquid limit moisture content by interpolation between four points (Clayton, 1995).

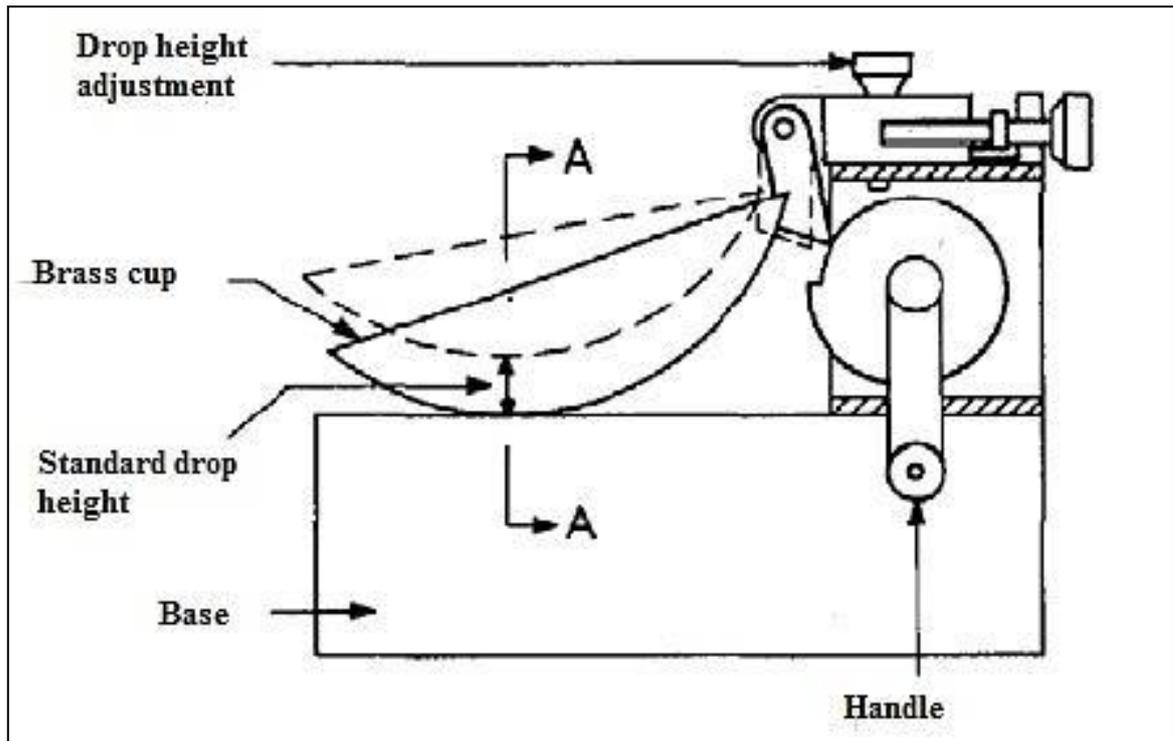


Figure 4-21 Casagrande apparatus (adapted from Clayton, 1995)

4.2.3.3 Mechanical Properties

The laboratory test analysis of the mechanical properties of a number of colluvium samples from the study area was completed between 2013 and 2014. This included evaluating the strength, deformability, consolidation, swelling characteristics, permeability, dispersivity and liquefaction potential of the material. These properties were then correlated back to the soil type, position in the mining area and slope.

i) Tri-axial Testing

Consolidated-undrained tri-axial tests were completed on undisturbed and remoulded samples of the yellow brown and dark red brown colluvium in an effort to determine the effective shear strength parameters, cohesion (c') and angle of internal friction (ϕ'), and to evaluate the influence of remoulding on these parameters. Three tri-axial setups were dedicated to the completion of this research and running parallel with one another. The test work was completed according to the procedure specified in BS 1377-8 (1990) and using the VJ Tech tri-axial equipment summarised in Table 4-6 and illustrated in Figure 4-22. The components as listed here all conform to the BS 1377 standards.

Table 4-6 Summary of tri-axial test equipment and specifications used during research (VJ Tech Limited, 2013)

Tri-axial Test Apparatus	Product	Specifications	
Loadframe	50 kN VJ TECH TriSCAN Advanced Load Frame	Load Capacity: Speed Range: Platen Adjustment Speed: Vertical Clearance: Horizontal Clearance: Platen Diameter:	50 kN 0-10.00000 mm/min 0-99.99999 mm/min 1000 mm 380 mm 158 mm
Tri-axial cell	VJ Tech 50 mm cell	Maximum Sample Diameter: Maximum Sample Height: Maximum Pressure: Height: Diameter:	50 mm 100 mm 2000kPa 320mm 170mm
Hydraulic pressure controller	VJ Tech Hydraulic APC 3000kPa	Medium: Maximum Pressure: Volume Capacity: Number of Pressure Outlets: Pressure Accuracy: Pressure Resolution: Volume Accuracy: Volume Resolution:	De-Aired Water 3000kPa 250cc 1 0.15% 1 kPa 0.25% 0.001cc
Data logger	VJ Tech MPX 3000	Analogue Channel Input: Digital Channel Input: Local Control via Keypad: Clock: Data Storage:	LSCT, Pressure or Load Cell Digital Dial Gauge Input Yes Real time clock Up to 12000 readings
Load cell sensor	VJ Tech S-Beam Load Cell 10 kN	Capacity: Rated Output: Accuracy: Excitation:	10 kN 2.0 +/-0.1mV/V <0.030% of load 15V DC (maximum)
Displacement sensor	VJ Tech 25 mm LSCT	Range: Resolution: Non linearity (best fit): Excitation:	0 - 25mm Infinite <0.1% of scale 2 to 10V AC/DC
Water pressure sensor	VJ Tech Pressure Transducer (10 bar)	Range: Excitation: Rated Output:	0 - 10 bar 10V DC 100.0 (+/- 0.1) mV DC



Figure 4-22 Tri-axial apparatus setup maintained during research

The specified test equipment was used together with VJT – cs TRIAX Clisp Studio software module to manage, record and analyse the tri-axial test data (Figure 4-23). The software conforms to the BS 1377-7 (1990) and BS 1377-8 (1990) standards and the saturation, consolidation and shear stages of up to four specimens or four multi-stages of a single specimen can be recorded and managed. The software further allows for the option of either isotropic or anisotropic consolidation, plotting of the Mohr circles and on-screen measurement of the determined cohesion and friction angle values (VJ Tech Limited, 2013).

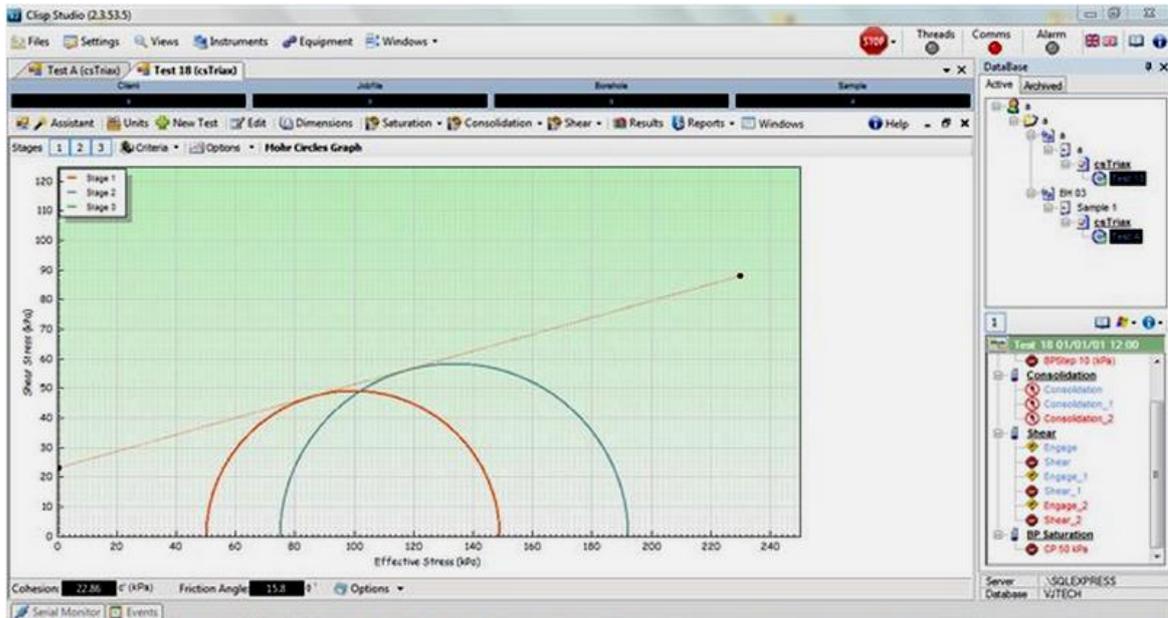


Figure 4-23 Example of Mohr circle graph on-screen in Clisp Studio Version 2.3 (VJ Tech Limited, 2013)

The consolidated-undrained tri-axial test procedure according to BS 1377-8 (1990) requires a cylindrical test specimen of soil (undisturbed or remoulded) with a length-to-width ratio of two. The specimen is sealed by a rubber membrane attached by rubber ‘O’ rings to a base pedestal and top cap. Once this sample set-up has been completed, the horizontal principal stresses are induced by water pressure inside the cell, while the vertical pressure at the top cap is developed through the cell fluid pressure and an applied ram force, driven into the tri-axial cell by means of the loading frame. The test specimen is then (normally) failed under compression conditions, under which the height of the specimen decreases during shearing (Clayton, 1995).

The consolidated-undrained tri-axial compression test is completed in multiple stages, involving the successive saturation, consolidation and shearing of each of three specimens. For the purpose of this research, consolidation pressures of 100, 200 and 400 kPa were specified. Saturation is carried out so as to ensure that the pore fluid in the specimen does not contain any free air and is normally carried out by leaving the specimens to swell against an elevated back pressure. The back pressure is applied through a volume change gauge at the top of the specimen, while a cell pressure of slightly higher value is also applied. Both cell pressure and back pressure are normally increased in increments, allowing time for equalization at each stage. When the consolidation cell pressure and back pressure are applied to the specimen, readings of volume change are made using a volume change device in the back pressure line. The rate at which volume change takes place depends on the effective pressure increment, the coefficient of consolidation of the soil and the drainage conditions at the specimen boundaries.

Three specimens are tested and consolidated at three different effective pressures in order to obtain widely spaced effective stress Mohr circles and subsequently the accurate determination of the effective shear strength parameters. The results of consolidation are further also useful to determine the minimum time to failure in the shear stage. The effective consolidation pressures are normally increased by a factor of two between each specimen, with the middle pressure

approximating the vertical effective stress at the depth of the retrieved sample in the ground (Clayton, 1995).

Remoulded tri-axial tests were completed in an effort to correlate the remoulded intrinsic shear strength parameters to the strength parameters of the material in its natural state. Using the calculated wax densities and natural moisture contents of the material, remoulded and compacted specimens of each of the specified samples were prepared for consolidated-undrained tri-axial testing.

It was emphasised prior to testing that the remoulding manipulation time be adjusted to ensure total remoulding and destruction of any material structure is achieved. This was achieved through moulding by palette knives on a glass plate, according to the procedure set out in BS 1377-2 (1990). The materials, of which the original structure is considered completely destroyed by this stage, were then moulded and compacted in layers of pre-determined thicknesses based on the calculated material properties and the height of the test specimen required. Tri-axial testing of remoulded samples was completed as close to the determined natural moisture content of the material as possible.

ii) Consolidation

The front-loading oedometer apparatus consists of a cell which can be placed in a loading frame and loaded vertically. In the cell the soil sample is laterally restrained by a steel ring, which incorporates a cutting shoe used during specimen preparation. The top and bottom of the specimen are placed in contact with porous discs, so that drainage of the specimen takes place in the vertical direction when vertical stress is applied; consolidation is then one-dimensional.

BS 1377 - 5 (1990) details the standard procedure for the test. In this procedure the specimen is subjected to a series of pre-selected vertical stresses, each of which is held constant while dial gauge measurements of vertical deformation of the top of the specimen are made, and until no further movements are registered. Dial gauge readings are taken at standard intervals of time after the start of the test. At the same time that the first load is applied, the oedometer cell is flooded with water, and if the specimen swells the load is immediately increased through the standard increments until swelling ceases. The results of each loading stage of an oedometer test are normally plotted as the dial gauge readings either as a function of square root of elapsed time, or as a function of the logarithm of elapsed time. The results of all the load stages are normally combined in one graph of void ratio as a function of the logarithm of effective pressure, plotted based on the calculated void ratios at the end of each of the load stages (Clayton, 1995).

iii) Proctor Compaction

Bulk disturbed samples (40-70 kg) with separately packaged and sealed natural moisture content samples were collected in an effort to evaluate the compaction characteristics of the colluvium material using the Standard Proctor method. Laboratory compaction tests are intended to model the field process, and to indicate the most suitable moisture content for compaction (the 'optimum moisture content') at which the maximum dry density will be achieved for a particular soil (Clayton, 1995). From the BS 1377 - 4 (1990) standards, the Standard Proctor test comprises the compaction of a soil specimen by a 2.5kg hammer falling through a standard distance of 300

mm on the various soil layers in the mould. Compaction of the specimen is completed through three standard “lifts” or layers of thicknesses equal to approximately one third of the height of the mould, with each layer subjected to 25 drops of the hammer. This process is repeated a minimum of five times for different specimens at different moisture contents. The moisture content versus the achieved dry density are then plotted as individual points and a "best fit" curve drawn through them in the order of increasing moisture contents to form a parabolic curve. The moisture content corresponding to the peak of the curve is the "optimum moisture" at which the maximum dry density is achieved.

As was specified by Northmore *et al.* (1992) during the compaction testing of a number of tropical soils, the following variations from the standard BS 1377 – 4 (1990) test procedure were specified:

- No pre-drying of the samples are allowed;
- Separate sub-samples are used for each individual compaction stage.

Although the single peak compaction curve is most common, an extensive laboratory study by Lee and Suedkamp (1972) showed that three irregularly shaped curves can also occur. From their research it is proposed that (1) soils with a liquid limit between 30 and 70 typically exhibited a single-peak moisture–density curve; (2) both double-peak and oddly shaped curves were present in soils with a liquid limit greater than 70; (3) soils with a liquid limit less than 30 usually produced either a double-peak or a one-and-one-half peak curve; and (4) the length of the moisture conditioning period substantially affected the moisture–density relationships in high-plasticity (high-liquid-limit) soils but had little influence on low-plasticity or non-plastic soils.

iv) Permeability

Tri-axial permeability tests were completed as part of this research under a consolidation pressure of 200 kPa and after full saturation of the specimen. This included tests on undisturbed soil specimens and remoulded specimens of the same material in an effort to evaluate the effect of remoulding and the inherent structure on the permeability of the material.

According to BS 1377 – 5 (1990), the permeability of a soil refers to its capacity to transmit water through the pore spaces between the soil particles. The coefficient of permeability is therefore expressed as a velocity. Permeability testing was completed simultaneous with tri-axial testing in the tri-axial cell according to BS 1377-6 (1990). This test method comprises the measurement of the permeability coefficient of a cylindrical soil specimen in a tri-axial cell under known conditions of effective stress and under the application of back pressure. The volume of water, passing through the soil specimen during a known period of time and under a constant hydraulic gradient is measured. Thus the coefficient of permeability (k) can be obtained from tri-axial tests through:

$$k = C_v M_v \gamma_w \quad (\text{Eq. 4-7})$$

Where:

k – Coefficient of permeability (cm/s)

C_v – Coefficient of consolidation (cm²/s)

M_v – Coefficient of compressibility (cm²/kg)

γ_w – Density of water (kg/cm³)

The expected typical permeability rates for various soil types are summarised from the available literature in Table 4-7 below. These rates are based on the values established for specimens of the different soil types found in more temperate climatic regions than the case study area.

Table 4-7 Typical permeability rates (k) for different soil types in temperate climatic regions (adapted from Spangler *et al.*, 1982)

Soil	k		
	(cm/s)		
	Lower Limit	Upper Limit	Average
Clean gravel	1.0E+00	-	1.0E+00
Gravel, clean coarse sand	1.0E-01	1.0E+00	5.5E-01
Graded sand, fine sand	1.0E-03	1.0E-01	5.1E-02
Silty sand, silt	1.0E-05	1.0E-03	5.1E-04
Dense silt, clayey silt	1.0E-07	1.0E-05	5.1E-06
Clay, silty clay	1.0E-11	1.0E-07	5.1E-09

The typical permeability rates for the various materials listed in Table 4-7 above can be subdivided into classes of intrinsic permeability in Table 4-8 below:

Table 4-8 Typical classes of permeability rates (k) (adapted from Bear *et al.*, 1968)

Log k (cm/s)	2	1	0	-1	-2	-3	-4	-5	-6	-7	-8	-9	-10	-11
Permeability	Pervious				Semi-Pervious				Impervious					

v) Dispersivity

Samples of the colluvium material were subjected to double hydrometer testing in an effort to evaluate the susceptibility of the material to dispersion. Testing was completed in accordance with BS 1377-5 (1990). The double hydrometer dispersion test involves the completion of a hydrometer sedimentation analysis (in accordance with BS 1377-2 (1990)) on two identical samples of soil; one with and one without the use of chemical dispersant and mechanical aggravation. The ratio between the measured clay fractions of the two samples are taken as an indication of the potential dispersiveness of the material.

Dispersion occurs in soils with a high percentage of Exchangeable Sodium Percentage (ESP), causing internal erosion and eventually piping through embankment dams. The tendency for dispersive erosion in a given soil depends on the mineralogy and chemistry of the clay and the dissolved salts in the soil water and eroding water.

Dispersion occurs when the repulsive forces (electrical surface forces) between individual clay particles exceed the attractive (van der Waal's) forces and when the clay mass is then in contact with water, individual clay particles are progressively detached from the surface and released into suspension. The dispersed clay particles are carried away by the flowing water, which causes internal erosion (piping) or surface erosion.

vi) Swelling Pressure

According to BS 1377-5 (1990), swelling pressure can be defined as the necessary vertical pressure applied to the test specimen in a consolidometer test setup to prevent the sample from swelling and recording a positive volume change when introduced to additional moisture.

The procedure to determine the swelling pressure involves balancing the induced swelling pressure of a prepared test specimen in a front-loading consolidometer (Figure 4-24) once water is added to the test setup. This is done by keeping the dial gauge reading stationary or in equilibrium at zero reading by the careful application of weights to the beam hanger of the consolidometer once it is evident that swelling occurs. This is done through a series of small calibrated weights to apply pressures in increments of 1 kPa to the specimen through the beam hanger. When equilibrium is established, the pressure applied to the specimen through the weights is calculated as the swelling pressure (kPa) of the material.



Figure 4-24 Front loading Gang consolidometers during testing

4.2.3.4 Unsaturated Soil Behaviour

It has been common practice in the geotechnical engineering world to consider most geotechnical problems in terms of the principles of saturated soil mechanics. The behaviour of unsaturated soils

encountered in engineering practice is not consistent with the principles and concepts of classical, saturated soil mechanics. It is argued by Fredlund and Rahardjo (1987) that when the degree of saturation of a soil is greater than 85%, saturated soil mechanic principles may be applied. When the degree of saturation is less than 85% however, the application of unsaturated soil mechanic principles becomes necessary.

Geotechnical engineering technology was primarily developed in the temperate climatic regions of the world. Consequently, research was largely dedicated towards soils which exhibit positive pore-water pressures (Fredlund and Rahardjo, 1993).

A number of theoretical frameworks have been developed to express the effective stress for unsaturated soil, with the transfer of theory from saturated soil mechanics to unsaturated soil mechanics being made possible through the use of stress state variables, defining the stress condition in a soil. These values are used along with measureable soil properties to form constitutive equations. The constitutive equations are used to express the relationship between the stress variables and shear strength or volume change during soil behaviour analysis (Ali and Rahardjo, 2004). Bishop (1959) formulated a relationship to quantify effective normal stress (σ') for unsaturated soil in terms of total stress (σ), pore air pressure (u_a) and pore water pressure (u_w):

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \quad (\text{Eq. 4-8})$$

The $(\sigma - u_a)$ term in the above equation refers to the net normal stress component and $(u_a - u_w)$ to the matric suction component. χ represents the constant parameter that quantifies the contribution of the matric suction to the effective normal stress, varying between 0 for a completely dry soil and 1 for a completely saturated soil. Some difficulties in accurately calculating the χ parameter have however been reported, including the fact that the parameter is test and stress path dependent and the fact that the parameter is essentially a function of the soil properties and not equilibrium conditions (Harrison, 1998). Following the difficulties encountered with accurately establishing the parameter χ , Fredlund and Morgenstern (1977) proposed the theoretical stress analysis based on the principle of multi-phase continuum mechanics. Based on this principle it is assumed that soil particles are incompressible and chemically inert, as it is the case with saturated soil mechanics.

For completely saturated or dry conditions the equation Eq. 4-8 may be further reduced to the classic Terzaghi equation ($\sigma' = \sigma - u$) for effective normal stress of a soil where the pores are completely filled either with air or water. These two components develop intergranular stress through different mechanisms, with the applied stress developing shear stresses and potential instability at the grain contacts while pore pressure is strictly a normal stress and hence increases the stability at grain contacts (Barden *et al.*, 1973).

$$\sigma' = \sigma - u \quad (\text{Eq.4-9})$$

Where:

σ' – Effective stress (kPa)

σ – Total stress (kPa)

u – Pore pressure (kPa)

The effective stress variable (σ') in the 1930's became central in the process of explaining and communicating the behaviour of saturated soils. Soil behaviour was related to effective stress which was independent of the soil properties. The basic calculations and considerations in classical soil mechanics was of a static and steady state nature. With time however, there has been an increasing need to consider a wide variety of unsteady state or transient related analysis (Fredlund and Rahardjo, 1993).

Soil suction is one of the most important independent parameters describing the stress state at different moisture conditions in an unsaturated soil. Generally, porous materials have the ability to attract and retain water. This ability is described as suction (Bulut *et al.*, 2001), and can thus be seen as the attraction the soil exerts on the moisture and the difference between the pore-air and pore-water pressures.

The total soil suction can essentially be considered the sum of two components (Equation 4-10); matric and osmotic suction (Fredlund and Rahardjo, 1993 in Bulut *et al.*, 2001).

$$\psi_t = \psi_m + \psi_o \quad (\text{Eq. 4-10})$$

Where:

ψ_t – Total soil suction (kPa)

ψ_m – Matric suction (kPa)

ψ_o – Osmotic suction (kPa)

Matric suction is equal to the difference between the soil pore-water pressure and the air pressure and can be considered a function of the combined effects of capillary forces and short-range adsorption of fine-grained soil particles. At high levels of saturation the capillary forces in the small pores of the soil dominate, arising from the surface tension of water at the air-water interface. Small pores act as capillary tubes, drawing pore-water above the water table which is held in menisci between the soil particles. The capillary pore-water has a negative pressure with respect to the air pressure (Westraad, 2004). At lower levels of saturation the adsorbed pore-water primarily occurs in the form of thin films which coat the soil particle surfaces. In these conditions the short-range adsorption effects arising from electrical and van der Waals forces at the soil pore-water interface is considered the dominant driving force (Lu and Likos, 2004).

Fredlund and Rahardjo (1993) mentions the fact also observed during this study that there appears to be some confusion in the literature regarding the spelling of the term “matric suction”. The 1963 report by the International Society of Soil Science to establish soil physics terminology, defined the term and spelled it as “matric suction”. In 1964, the Symposium-in-Print entitled, "Moisture Equilibria and Moisture Changes in Soils Beneath Covered Areas", was presented to the First International Conference on Expansive Soils (1965). The first article in this

publication was a statement of the Review Panel which sets forth the Notation and Definition of Terms for expansive soils. Here, the capillary suction component of the soil was spelled as matrix suction while the definition was the same as that given in 1963 by the International Society of Soil Science. This has subsequently led to confusion in the literature regarding the spelling of the capillary component of suction. As advocated by Fredlund and Rahardjo (1993), the spelling convention consistent with the soil physics literature will be followed during this study, referring to this component as “matric suction”.

Osmotic suction is concerned with the concentration of dissolved solutes in soil pore-water. A differential pressure occurs between two concentrations of dissolved solutes if the flow of water between them is restricted. The resultant potential for pore-water to flow due to this differential pressure inside the soil pore matrix is known as the osmotic suction component of total suction. Although the concentration of dissolved solutes may change with the degree of saturation, osmotic suction remains constant as long as the relative concentration of dissolved solutes remains unchanged (Lu and Likos, 2004).

The aforementioned effective stress variable for saturated soils can therefore be logically extended when considering the stress state of an unsaturated soil in terms of the two independent stress state variables; the net normal stress ($\sigma - u_a$) and matric suction ($u_a - u_w$). A change in total stress and pore-water pressure do not result in similar responses in an unsaturated soil as it would in the case of saturated material. However, as the pore-air pressure is assumed to be constant for the majority of geotechnical problems, the total stress and pore-water pressure can be independently referenced from the pore-air pressure (Fredlund and Rahardjo, 1993). The stress state at a particular point in a saturated soil is only a function of the total normal stress ($\sigma_{x,y,z}$) minus the pressure of the pore-water, whereas for an unsaturated soil the stress state is expressed as a function of the net normal stress ($\sigma - u_a$) and matric suction ($u_a - u_w$). This is graphically summarised in Figure 4-25.

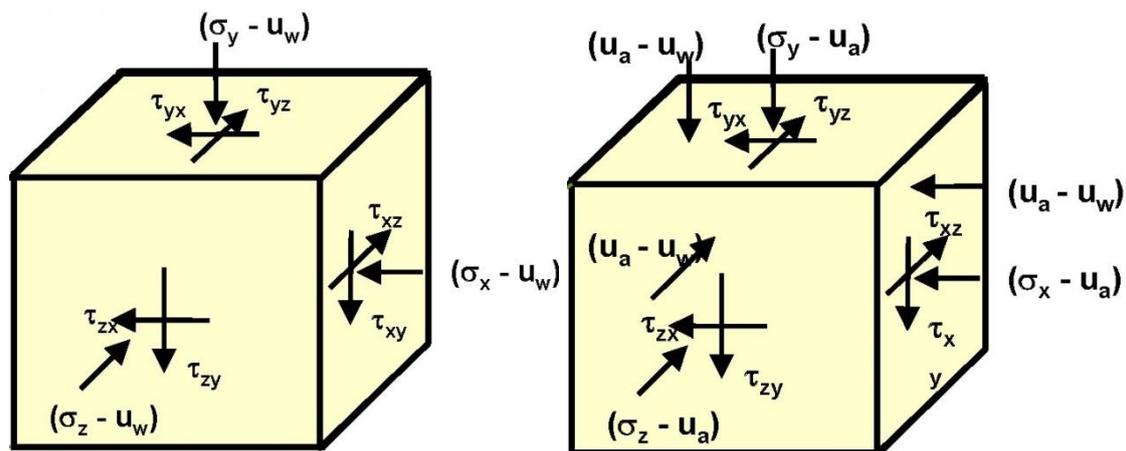


Figure 4-25 Stress state at a single point in a saturated (*left*) and unsaturated soil (*right*) (adapted from Fredlund and Rahardjo, 1993)

The change in matric suction due to the change in moisture content of the material therefore becomes an important, additional relationship to consider in quantifying unsaturated soil behaviour. This relationship is evaluated during this study using the Filter Paper method, as well as through pressure plate equipment to establish the Soil Water Characteristic Curve (SWCC) of the soil material.

The stress state variables for unsaturated soils (net normal stress and matric suction) can be used to establish constitutive relations for classic unsaturated soil problems. Volume change under isotropic loading conditions can be expressed in terms of the change in void ratio (d_e) through Eq. 4-11 and the moisture content (d_w) of the unsaturated soil (which is independent of a change in void ratio) through Eq. 4-12. Figures 4-26 and 4-27 illustrate these constitutive relations for the void ratio and moisture content in terms of their response to a change in total stress conditions of the unsaturated material (Fredlund and Rahardjo, 1993).

$$d_e = a_t d(\sigma_c - u_a) + a_m d(u_a - u_w) \quad (\text{Eq. 4-11})$$

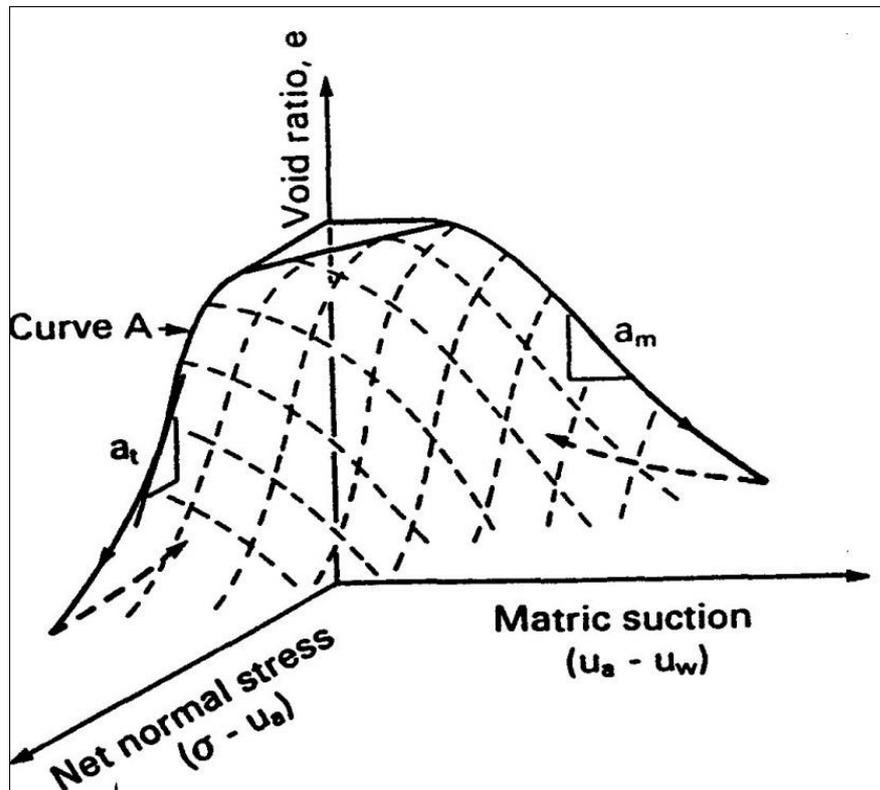


Figure 4-26 Constitutive relation between void ratio and stress state variables (adapted from Fredlund and Rahardjo, 1993)

$$d_w = b_t d(\sigma_c - u_a) + b_m d(u_a - u_w) \quad (\text{Eq. 4-12})$$

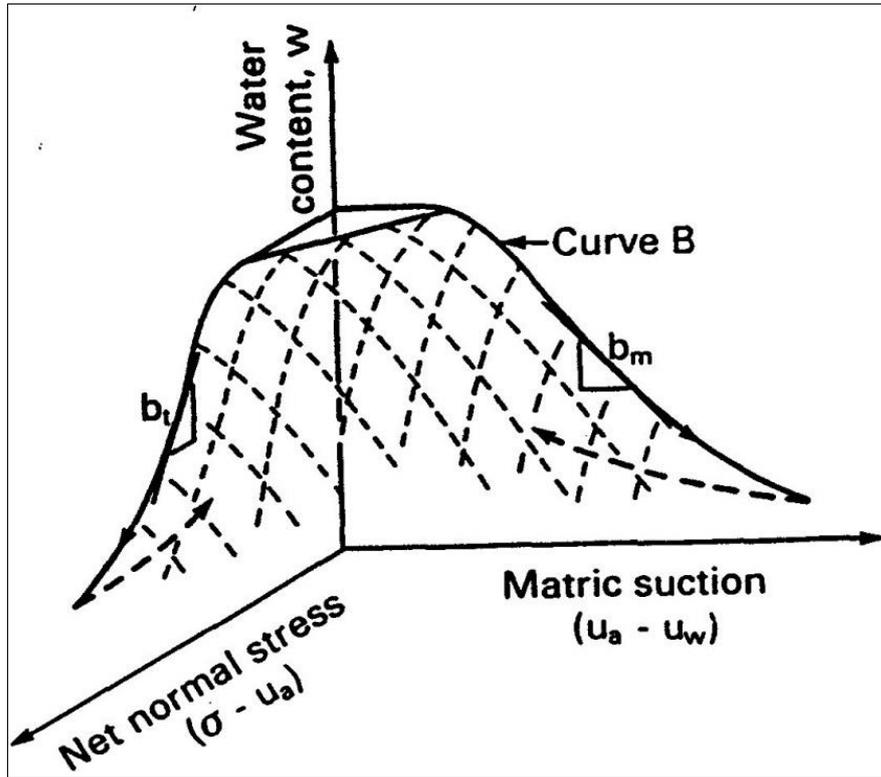


Figure 4-27 Constitutive relation between moisture content and stress state variables (adapted from Fredlund and Rahardjo, 1993)

From these established relationships it is evident that when the matric suction pressure in a soil equals zero, any change in void ratio and moisture content will be equivalent to the response of the materials to a change in the total stress conditions only.

The evaluation of the shear strength has been adapted for unsaturated soils as a linear combination of the stress state variables and the various shear strength parameters (Fredlund and Rahardjo, 1993):

$$\tau = c' + (\sigma_n - \sigma_a) \tan \phi' + (u_a - u_w) \tan \phi^b \quad (\text{Eq. 4-13})$$

Based on this relationship, it is possible to graphically represent and linearize the relationship of the change in shear strength of a material with a change in moisture content and subsequent change in matric suction pressures, as done by Fredlund and Rahardjo (1993) for a clayey silt and sandy soil respectively in Figure 4-28.

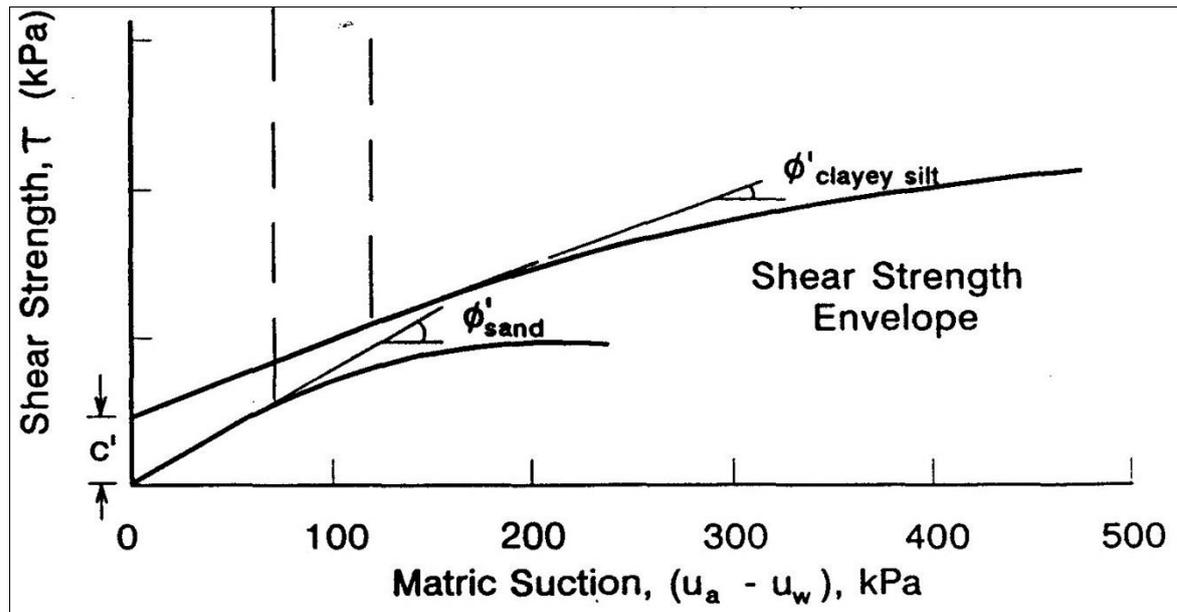


Figure 4-28 Relationship between shear strength and matric suction pressure (adapted from Fredlund and Rahardjo, 1993)

It is evident from this relationship that at low matric suction pressures, where the suction pressure is lower than the air-entry value of the soil, the soil is very near to saturation conditions and would be expected to behave as though it is saturated. As a result, an increase in the matric suction pressures would lead to a similar increase in shear strength as would have been experienced through an increase in the net normal stress ($\sigma - u_a$). At those matric suctions higher than the air-entry value of the soil however, the soil will tend to desaturate and negative pore-water pressure will not act throughout all the pores as in a saturated condition. At such high matric suction pressures the contribution to the overall shear strength of the soil is less than that of the net normal stress at the same stress level and the increase in shear strength through the effect of suction is less than the increase with respect to net normal stress (Ali and Rahardjo, 2004).

Additionally, consideration should also be given to the fact that an unsaturated soil will not have a constant coefficient of permeability (k) and will be a direct function of the matric suction pressures of the unsaturated soil material. The flow of water is however still proportional to the hydraulic head gradient as expressed through Darcy's Law and can therefore be adapted as:

$$V_w = -k_w(u_a - u_w)\partial h/\partial y \quad (\text{Eq. 4-14})$$

Using the SWCC derived from pressure plate testing (discussed hereafter), the permeability can therefore be calculated based on the saturated coefficient of permeability and SWCC.

i) Filter Paper Method

Soil suction measurements were conducted on selected yellow brown colluvium samples from the Mt. Lekoumou mining area using the filter paper method proposed by Bulut *et al.* (2001), in an effort to evaluate the relationship between the moisture content of the filter paper and suction pressures of the tested material. The filter paper method has long been used in soil science and engineering practice and has recently received much attention as an adaptable test method for soil suction measurements. This involves a selected specimen filter paper to reach equilibrium with

the soil through vapour (total suction) or liquid (matric suction) flow. The filter paper water content can then be used together with the established suction calibration curve or mathematical relationship to determine the corresponding suction value.

The filter paper method involves cutting a suitably sized test specimen, usually in a consolidometer ring of known dimensions. The sample is cut into two halves, with the surfaces that will be in direct contact with the filter paper smoothed and levelled. A piece of untreated Whatman's No. 42 filter paper is then removed from the packaging using tweezers, weighed and placed between two larger diameter filter papers on the surface of the prepared soil sample. The other half of the cut sample is then placed on top of the filter paper and taped together, whilst all the time ensuring the filter paper is sandwiched between and in intimate contact with the soil.

Once the test specimen has been taped together, a clean PVC O-ring is placed on top of the prepared test specimen and two Whatman's No.42 filter papers placed on top of the PVC ring. The test specimen is then sealed in a container of known weight and stored in a temperature controlled environment to prevent any exchange in moisture between the sample and environment. This process is repeated for a number of prepared samples of the material at different moisture contents to determine the matric suctions at the various moisture contents. The samples are left for one week, allowing the filter paper to reach equilibrium with the soil sample with an assumed magnitude of suction. The samples are then opened and dismantled after which the pieces of filter paper weighed again and the weights recorded. Considerable effort is made here to record these weights as efficiently and quickly as possible in order to minimise any change in moisture content and only handling the filter paper with tweezers.

Once the respective filter paper moisture contents are established, the total and matric suction pressure for the material at that particular moisture content can be determined. Total suction pressure is calculated using a simplified version of Kelvin's equation at a reference temperature of 25°C:

$$\psi_T = 137182 \times \ln(P/P_0) \quad (\text{Eq. 4-15})$$

Where:

P/P_0 – Relative humidity (%)

ψ_T – Total suction pressure (kPa)

Due to the dramatic decrease in total suction when the relative humidity approaches 100%, the total suction pressure is very sensitive to small changes in the filter paper water contents at these higher relative humidity values. Total suction pressure values can therefore only be accurately determined up to a point using the filter paper method. As proposed by Bulut *et al.* (2001), this minimum point from which reliable wetting suction measurement can be made was accepted as log 1.5 kPa.

Matric suction pressures are calculated using the filter paper placed between the two halves of the soil sample and using the relationship established by Hamblin (1981) expressed in Equation 4-16:

$$\ln \psi_m = -2.397 - 3.683 \ln F \quad (\text{Eq. 4-16})$$

Where:

F – Gravimetric water content of filter paper (%)

ψ_m – Matric suction pressure (kPa)

ii) Pressure Plate Extractors

The Soil Water Characteristic Curves (SWCC) of seventeen undisturbed samples of the colluvium material from the study area were determined using Pressure Plate Extractors (PPE) with a suction range of 0 to 1500 kPa. The tests were conducted in general accordance with ASTM D 6836 “Standard Test Methods for Determination of the Soil Water Characteristic Curve for Desorption Using a Hanging Column, Pressure Plate Extractor, Chilled Mirror Hygrometer, and/or Centrifuge” as summarised in Benson *et al.* (2003).

The SWCC can be described as a graphical representation of the mathematical relationship between the matric suction pressure of a particular soil and either its water content (gravimetric or volumetric) or degree of saturation. Originally developed in the field of soil and agricultural science, the SWCC represents the water storage capacity of a given material and allows for the determination of changes in matric suction with respect to changes in water content or degree of saturation. These curves are sigmoidal in shape, with soil suction presented on the x-axis as a log scale and the moisture content or percentage saturation on the y-axis. The soil water characteristic curve can be used to describe both the air pressure increase necessary to cause water to be expelled from the sample (desorption), and the pressure reduction needed for water to be absorbed into the soil (Fredlund and Rahardjo, 1993).

PPEs equipped with ceramic porous plates were used for measuring the portion of the SWCC corresponding to suctions between 0 and 1 500 kPa. A steel retaining ring with a 60 mm diameter and 25 mm height was used to trim the specimens. Once trimmed and inside the ring the excess soil on the top and bottom of the sample was scraped away and the surfaces flattened. The natural moisture content of the material was determined at this point, after which the specimen was saturated and placed in a chamber filled with de-aired water and the porous ceramic plate (Figures 4-29 and 4-30). A vacuum of 90 kPa was applied to the headspace in the chamber for at least 24 hours. Before the ring and the porous ceramic plate were removed from chamber, the vacuum was pulsed to remove any air bubbles that might be trapped in the system. Once saturated the specimens were placed into the PPEs. The outflow lines were saturated with de-aired water by adding the water through the Y-tube attached to the PPE. Before starting the test, the specimen was allowed to reach equilibrium under atmospheric pressure conditions for at least 48 hours.

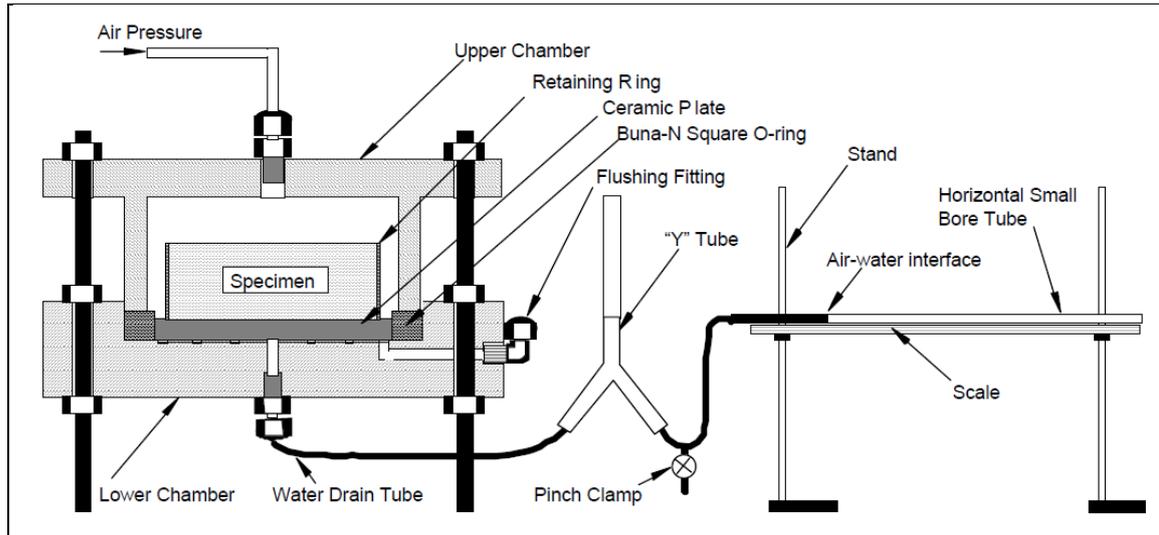


Figure 4-29 Pressure Plate Extractor Apparatus (Wang and Benson, 2002)

Suctions were then applied using the axis translation technique described in Fredlund and Rahardjo (1993). This comprised elevating the air pressure in the PPE whilst keeping the water pressure at zero kPa. Suctions were only increased once equilibrium has been reached at the previous increment.

The volumetric water content corresponding to each suction pressure is calculated by measuring the amount of water expelled and calculated by (Benson *et al.*, 2003):

$$\theta_i = \theta_{i-1} - \frac{\Delta L A_c}{V} \quad (\text{Eq. 4-17})$$

Where:

θ_{i-1} = Volumetric water content of previous suction pressure increment (mm^3)

ΔL = Distance meniscus moved in capillary tube between suction intervals (mm)

A_c = Cross-sectional area of the capillary tube (mm^2)

V = Volume of the sample (mm^3)

The volumetric water content and suctions measured with the PPE were combined and plotted to create the SWCC's for the respective samples.

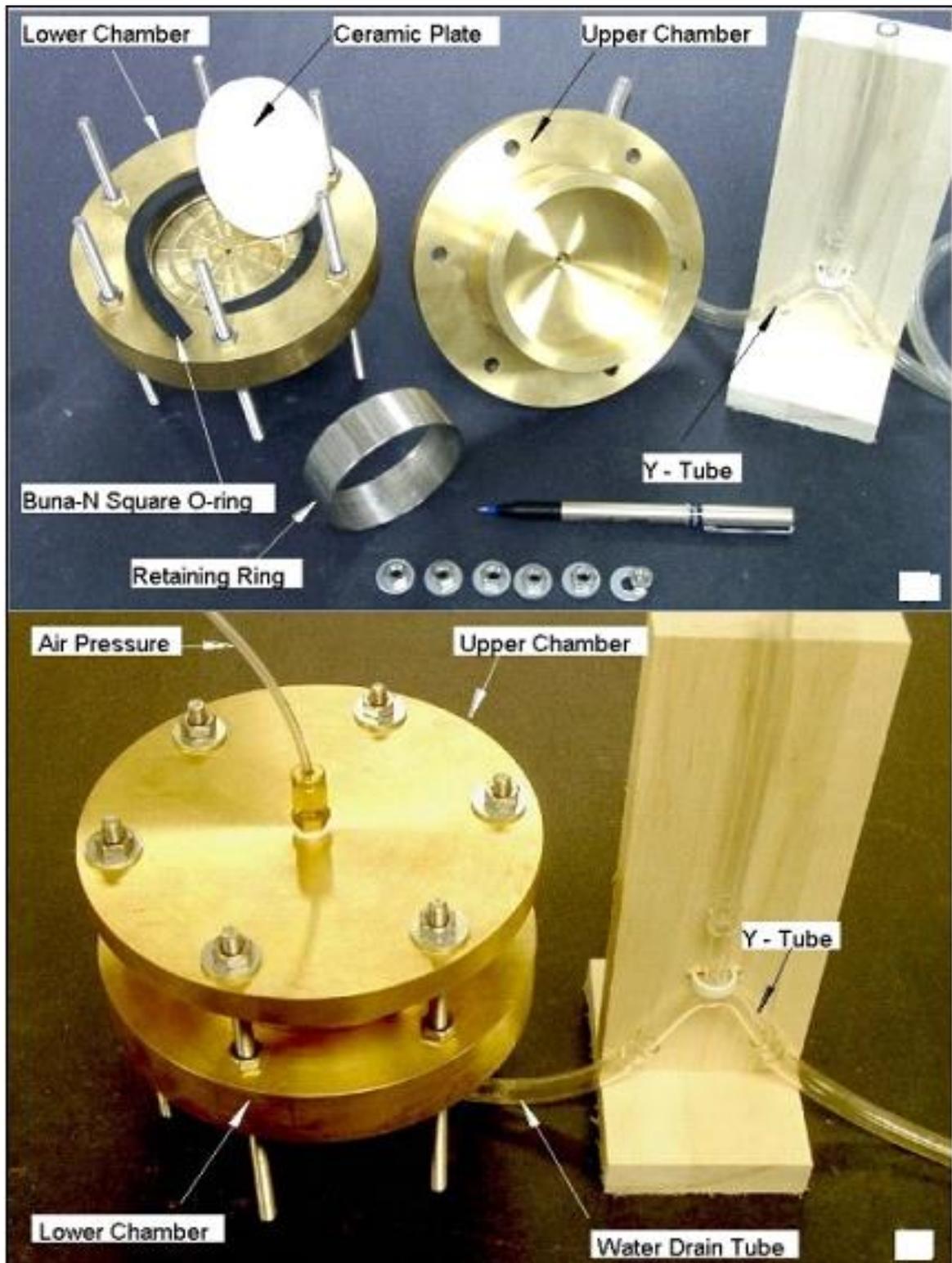


Figure 4-30 Example of Pressure Plate Extractor equipment used to determine the SWCC (adapted from Benson *et al.*, 2003)

5. Chemical and Mineralogical Properties

5.1 XRF Results

5.1.1 XRF Results – Metallurgical Characterization of Transported Ore

XRF analysis was completed separately on each of the different size products detailed in Chapter 4 of each of the samples as part of the metallurgical characterization of the transported ore. From the chemical results and the mass yield per fraction, the iron content of the various fractions of material forms the predominant chemical component for the bulk Run of Mine (ROM), lumpy and fine size products, with no clear correlation was established between iron content and elevation. The ultra-fine and slimes size-product is however characterized by a marked decrease in Fe and increase in SiO₂ and other elements (Exxaro Resources, 2012).

5.1.2 XRF Results – Geotechnical Characterization

From the mass yield calculations, silica (SiO₂) forms the major chemical constituent of the yellow brown colluvium from outside the mining area, typically varying between 40 and 50% in content (Table 5-1). The iron (Fe³⁺) content of the yellow brown colluvium is lower than that of the material found in the mining area, varying between approximately 20 and 30%, but with similar average aluminium oxide (Al³⁺) contents of 20 to 30%.

Table 5-1 Summary of XRF results for individual sample analysis from outside mining area

Element	Analyte	Unit	TSF 1-1	TSF 1-2	TSF 1-3	TSF 1-4	TSF 1-5
Silica	SiO ₂	% g/g	48.59	47.94	47.6	39.21	45.19
Titanium	TiO ₂	% g/g	1.06	1.25	1.31	0.95	1.17
Aluminum	Al ₂ O ₃	% g/g	27.45	28.96	28.87	24.15	27.73
Iron	Fe ₂ O ₃	% g/g	11.79	10.68	10.72	23.85	14.66
Manganese	MnO	% g/g	0.03	0.03	0.02	0.03	0.03
Magnesium	MgO	% g/g	0.38	0.33	0.22	0.15	0.23
Calcium	CaO	% g/g	0.03	0.03	0.03	0.02	0.03
Sodium	Na ₂ O	% g/g	<0.01	<0.01	0.13	0.23	<0.01
Potassium	K ₂ O	% g/g	1.33	0.81	0.49	0.25	0.38
Phosphorous	P ₂ O ₅	% g/g	0.07	0.05	0.05	0.04	0.06
Chromium	Cr ₂ O ₃	% g/g	0.07	0.06	0.05	0.08	0.07
Sulphur	SO ₃	% g/g	0.05	0.05	0.04	0.04	0.04
Loss on Ignition (1000 C)	LOI	% g/g	9.07	9.77	10.39	10.88	10.35
Total	Total	% g/g	99.92	99.96	99.92	99.88	99.94
	H ₂ O	% g/g	0.61	0.8	1.16	0.85	0.83

From the available results and the mass yield calculations, iron (Fe³⁺) forms the major chemical constituent of the red brown colluvium and transported ore materials from the mining area, typically varying between 40 and 60% in content for the red brown colluvium material and 60 to 80% for the transported ore (Table 5-2 and Figure 5-1). The iron content of the yellow brown colluvium is lower than that of the aforementioned horizons, varying between approximately 30 and 40%, overlapping partially with the observed iron oxide content of the red brown colluvium material. The average aluminium oxide (Al³⁺) content of the yellow brown colluvium is significantly higher (23%) than that of the red brown colluvium (16%) and transported ore

(13.5%). SiO_2 content of the different horizons varies significantly, with SiO_2 contents varying between 16 to 38% for the red brown colluvium, 6 to 27% for the transported ore and 22 to 27% for the yellow brown colluvium.

In addition to the variation in chemical compositions, the average intermolecular moisture content of the yellow brown colluvium is significantly higher (6.7% H_2O) than that of the red brown colluvium (1.0%) and transported ore (0.5%). In middle and lower slopes, where the yellow brown colluvium horizons are first encountered, soil drainage is poorer than in the upper slope and crest and hydrated iron oxides are produced, accounting for the elevated calculated H_2O content and the change in colour from dark red brown to yellow brown.

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Table 5-2 Summary of XRF results for bulk sample analysis from mining area

	SiO ₂	TiO ₂	Al ₂ O ₃	Fe ₂ O ₃	MnO	MgO	CaO	Na ₂ O	K ₂ O	P ₂ O ₅	Cr ₂ O ₃	SO ₃	LOI	Total	H ₂ O	*ID
MKRC1001	19.66	0.85	17.53	53.73	0.05	0.01	0.02	0.31	0.1	0.11	0.02	0.01	7.38	99.76	1.64	RBC
MKDD074	38.2	0.39	13.21	42.22	0.05	0.25	0.04	0.53	0.58	0.04	0.02	0.08	4.15	99.76	0.49	RBC
MKRC233	24.37	1.49	21.78	38.3	0.05	0.01	0.03	0.36	0.12	0.1	0.05	0.12	13.1	99.87	1.28	YBC
MKRC865	21.66	0.92	9.47	60.81	0.05	0.11	0.02	0.79	0.03	0.09	0.02	0.19	5.56	99.72	0.49	RBC
MKRC1189	23.8	0.17	11.08	60.41	0.06	0.19	0.02	0.58	0.18	0.06	0.01	0.01	3.19	99.74	0.57	TO
MKDD370	5.95	0.2	16.46	87.28	0.1	0.01	0.08	0.01	0.02	0.7	0.03	0.21	5.37	0	0.39	TO
MKRC367	27.21	1.98	24.3	31.38	0.05	0.01	0.06	0.64	0.09	0.11	0.07	0.1	13.88	99.87	7.76	YBC
MKRC905	16.61	0.82	16.66	56.53	0.07	0.01	0.03	0.26	0.05	0.1	0.02	0.1	8.59	99.84	1.74	RBC
MKRC231	22.27	1.52	21.56	40.88	0.05	0.01	0.03	0.34	0.11	0.1	0.05	0.15	12.82	99.88	5.55	YBC
MKRC842	24.83	1.08	14.86	50.51	0.13	0.09	0.05	0.64	0.06	0.11	0.02	0.02	7.3	99.7	0.61	RBC
MKRC1317	27.71	1.10	15.64	47.77	0.06	0.06	0.03	0.74	0.08	0.10	0.03	0.36	6.08	99.76	0.98	YBC
MKRC986	26.97	2.03	23.22	32.85	0.05	<0.01	0.03	0.34	0.09	0.16	0.08	0.40	13.88	100.10	1.78	YBC
MKRC212	26.40	1.93	23.42	33.45	0.05	<0.01	0.08	0.50	0.10	0.11	0.08	0.10	13.60	99.82	1.83	YBC
MKRC227	26.95	1.91	25.26	31.18	0.05	<0.01	0.03	<0.01	0.12	0.15	0.07	0.27	14.36	100.35	1.95	YBC
MKRC228	23.84	1.39	21.44	40.29	0.05	<0.01	0.04	0.49	0.12	0.11	0.04	0.26	11.76	99.83	1.67	YBC
MKRC905	18.53	0.87	16.24	55.00	0.06	<0.01	0.05	0.12	0.05	0.11	0.02	0.13	8.77	99.95	1.29	YBC
MKRC233	25.11	1.46	21.87	37.96	0.05	<0.01	0.03	0.17	0.12	0.10	0.05	0.13	12.91	99.96	2.38	YBC
MKRC027	27.68	1.60	24.13	32.91	0.05	<0.01	0.03	0.22	0.09	0.10	0.06	0.11	13.16	100.14	2.30	YBC
MKRC136	20.96	1.46	21.18	42.80	0.05	<0.01	0.03	0.13	0.10	0.11	0.04	0.17	12.99	100.02	1.85	YBC

***ID:**

RBC = Red brown colluvium

TO = Transported ore

YBC = Yellow brown colluvium

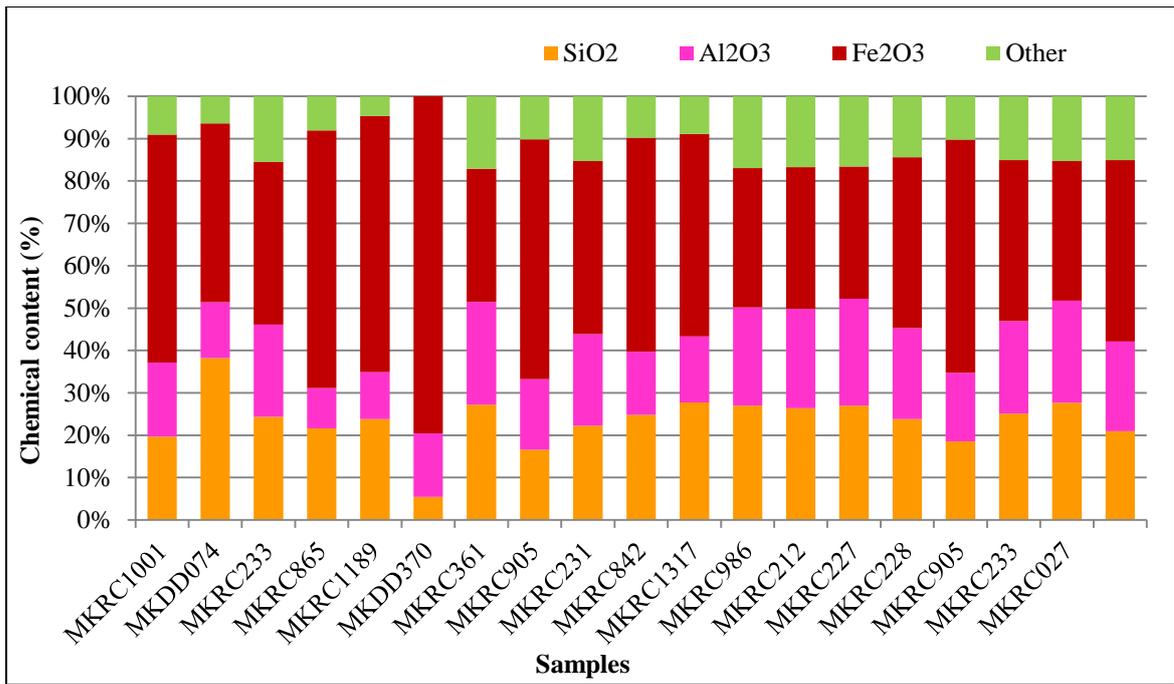


Figure 5-1 XRF results: chemical characterization of bulk samples

When analysing the variation in the major chemical constituents for the overall soil profile with change in elevation (Figures 5-2 and 5-3), it is evident that there is a marked decrease in Fe^{3+} with a decrease in elevation over the mining area. Conversely an increase in the overall SiO_2 is observed with decrease in elevation, with the Al^{3+} remaining fairly constant with change in elevation. Leaching of the dissolved primary minerals from the soil profile is vital in the development process of tropical red soils. Removal of bases and combined silica leads to decomposition and the breakdown of aluminium silicate clay minerals and the formation of hydroxides and sesquioxides.

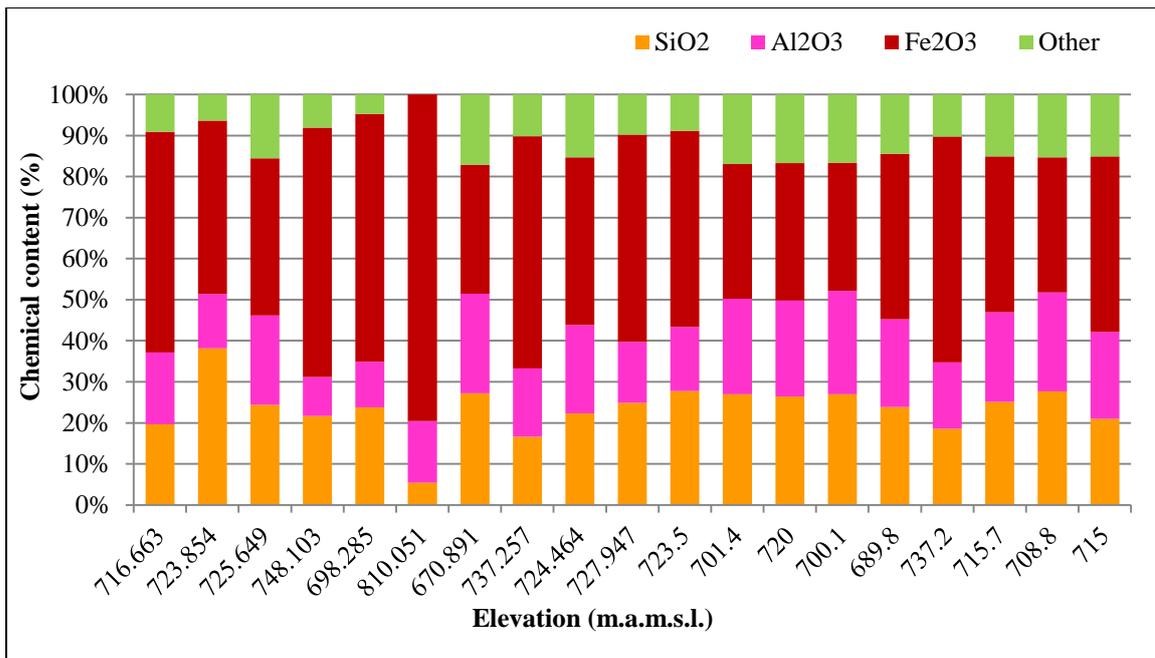


Figure 5-2 XRF results: chemical characterization of bulk samples according to elevation

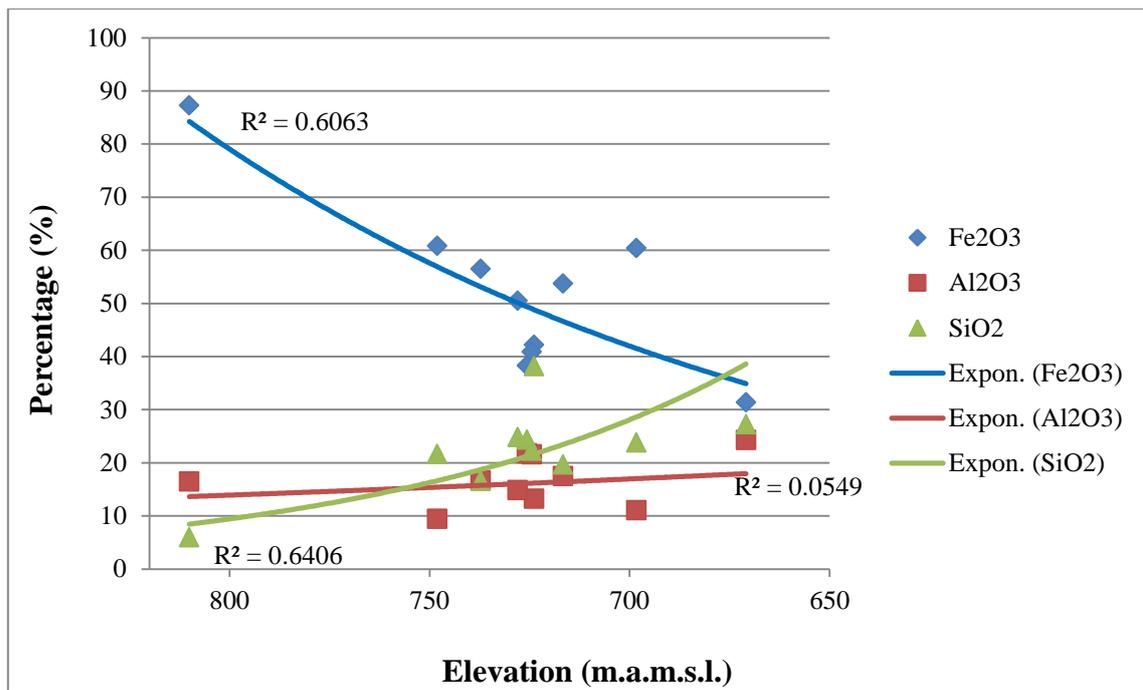


Figure 5-3 Variation in primary mineral content in relation to elevation

5.2 XRD Results

5.2.1 XRD Results – Metallurgical Characterization

XRD analysis was completed separately on the ultra-fine and slimes size products of each of the 14 samples as part of the metallurgical characterization of the transported ore. Quartz and haematite represent the major mineral components of the 75 μm fraction, whereas goethite typically comprises the major mineral proportion of the <75 μm fraction. However, goethite contents exceeding 50% in the 75 μm fraction were found to be present in some boreholes, believed to present oolitic development.

5.2.2 XRD Results – Geotechnical Characterization

From the XRD analysis of the bulk samples of the different geotechnical horizons from the Mt. Lekoumou mining area (Figure 5-4), it is clear that hematite forms the major mineralogical component of the transported ore, as well as containing higher concentrations of gibbsite and magnetite than the colluvium horizons. The XRD results of the fines fraction of the transported ore reflected a composition of predominantly goethite (50 – 60%), kaolinite (10 – 20%) and hematite (10 – 20%), similar to the red brown colluvium composition. Weathering of the iron-bearing minerals has however advanced slightly further in the red brown colluvium, hematite content has decreased and the iron-oxyhydroxide content increased in the form of goethite. Both the transported ore and red brown colluvium are also characterized by moderate to high percentages of quartz (10-30%).

The concentration of kaolinite is significantly higher in the yellow brown colluvium found from the middle to lower slopes of the topography due to the greater degree of weathering the material has been subjected to, with also moderate to high percentages of goethite. The quartz content of the yellow brown colluvium is however significantly lower than that of the transported ore and

red brown colluvium, which is contradictory to the increase in silica content noted from the XRF analysis of the yellow brown colluvium.

Table 5-3 XRD results for bulk samples from the mining area

MKRC	Gibbsite	Goethite	Hematite	Kaolinite	Magnetite	Muscovite	Quartz	*ID
367A	1.62	35.62	0.87	51.66	0.59	2.51	7.13	YBC
905A	2.23	51.63	22.31	5.86	1.86	1.48	14.65	RBC
1001A	0.38	41.94	27.31	6.8	2.56	2.2	18.81	RBC
231A	0.45	47.89	5.04	35.11	0	2.42	9.08	YBC
1189	3.41	12.98	52.4	8.34	3.25	1.59	18.04	TO
074A	0.56	23.07	33.83	5.74	2.55	2.96	31.29	RBC
842A	1.8	26.84	24.66	17.93	0.7	2.14	25.92	RBC
233A	1.93	23.95	5.81	56.56	0	1.08	10.67	YBC
865A	0.64	22.83	40.13	10.46	2.99	0.82	22.13	RBC
370A	4.83	29.36	57.1	0.31	5.62	0	2.78	TO
1317	0.38	38.94	25.69	8.54	1.33	25.12	0.38	YBC
986	3.66	37.27	1.22	50.2	0.58	7.05	3.66	YBC
212	1.52	47.31	22.48	9.28	2.45	16.96	1.52	YBC
227	0.63	39.2	6.68	39.1	0.71	13.64	0.63	YBC
228	0.39	43.4	5.82	39.2	0.4	10.79	0.39	YBC
905	3.67	39	0.53	50.7	0.31	5.81	3.67	YBC
233	3.51	41.5	1.66	44.49	0	8.85	3.51	YBC
027	0.84	47.11	6.12	35.71	0.46	9.75	0.84	YBC
136	3	39.18	2.24	47.06	0.57	7.95	3	YBC

*ID:

RBC = Red brown colluvium

TO = Transported ore

YBC = Yellow brown colluvium

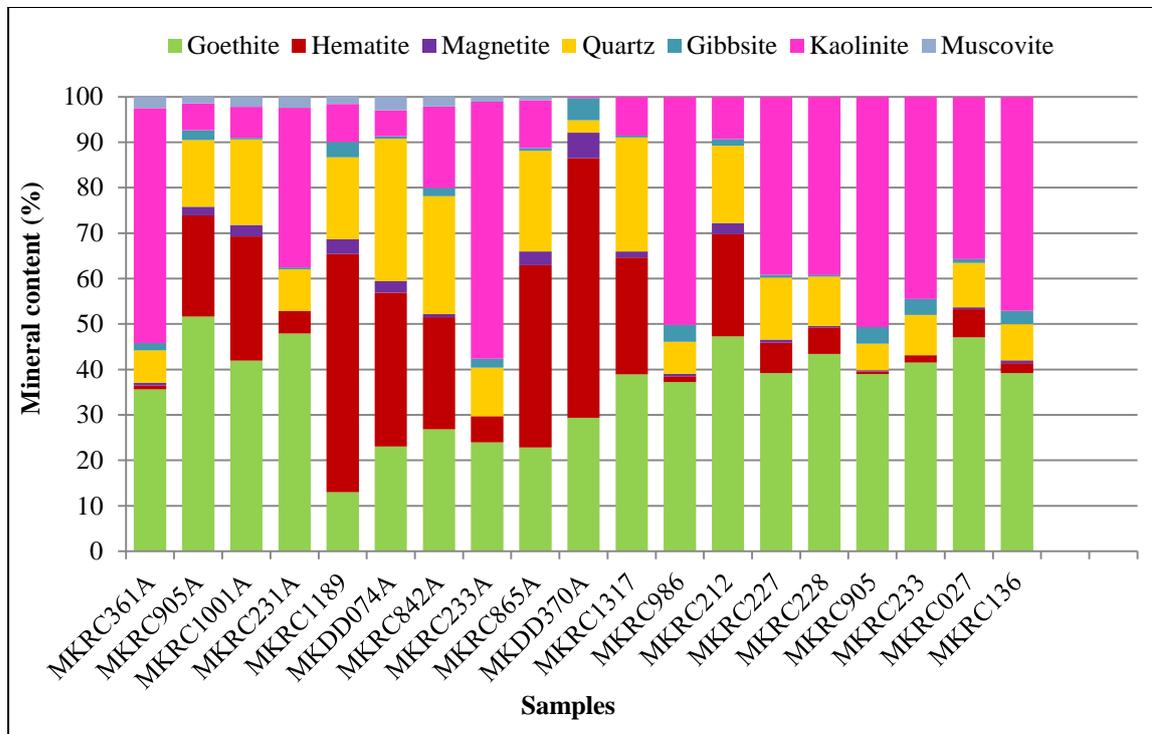


Figure 5-4 XRD results: mineralogical characterization of bulk samples from mining area

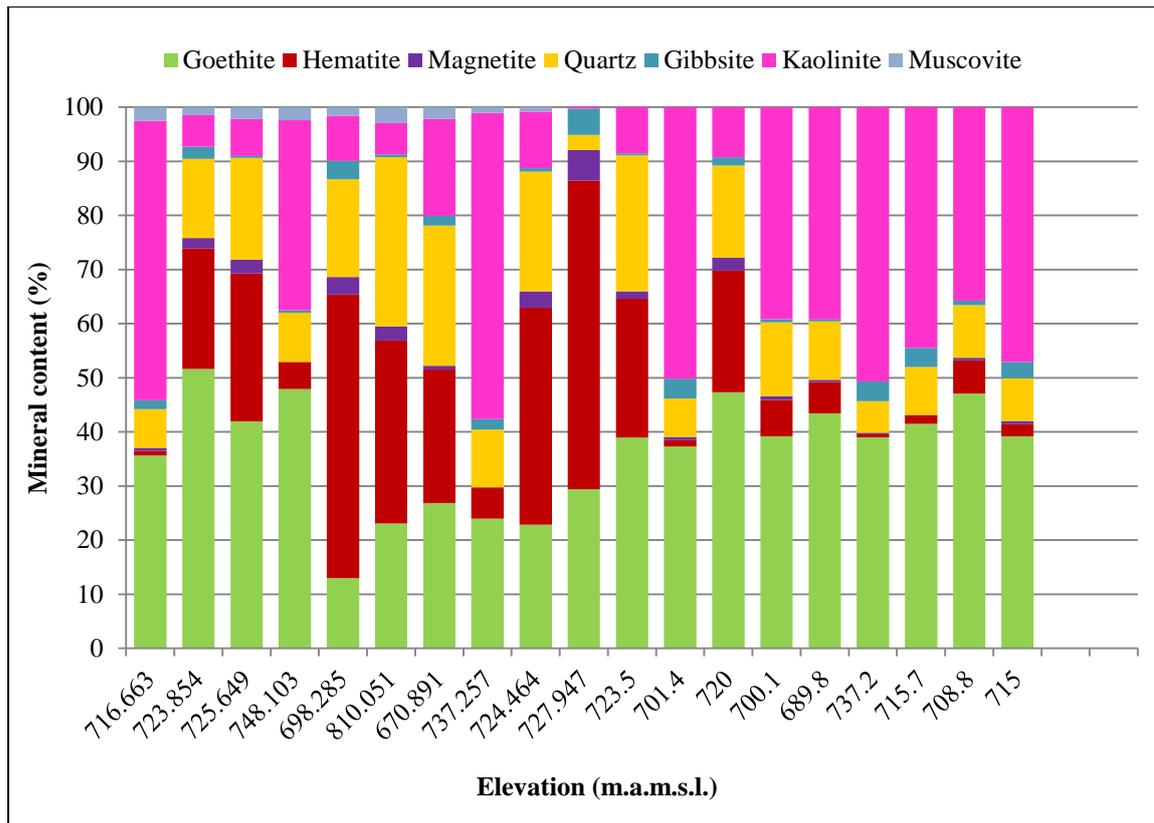


Figure 5-5 XRD results: mineralogical characterization of bulk samples according to elevation in mining area

From the XRD analysis of the samples of colluvium material from the lower-lying areas outside the mining area (Table 5-4 and Figure 5-6), it is clear that kaolinite and quartz form the major mineralogical components of the yellow brown colluvium material. The XRD results of this material reflected a composition of predominantly kaolinite (35 – 50%), quartz (35 – 50%) and goethite (5 – 15%). Very little hematite (< 1%) exists in the yellow brown colluvium found outside the mining area.

Table 5-4 XRD results of the yellow brown colluvium from outside the mining area

	Gibbsite	Goethite	Hematite	Kaolinite	Anatase	Muscovite	Quartz
TSF01	1.46	7.7	0.06	43.91	0	6.1	40.67
TSF02	2.49	8.77	0.18	45	0.23	7.7	35.61
TSF03	1.54	5.5	0.02	43.8	0.4	3.9	44.8
TSF04	1.43	16.5	3.52	38.5	0.34	2.56	37.1
TSF05	2.18	7.8	0.75	41.5	0.26	2.97	44.5

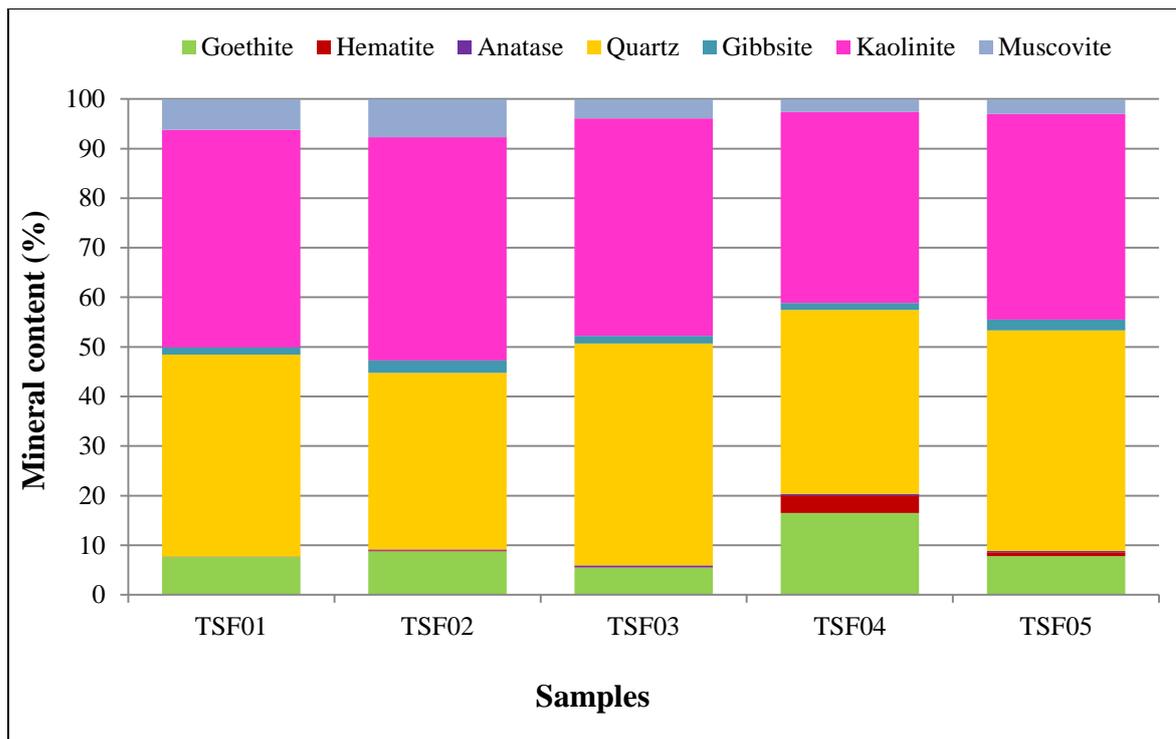


Figure 5-6 XRD results: mineralogical characterization of yellow brown colluvium from outside the mining area

5.3 Chemical Properties

The chemical analyses were completed on selected disturbed samples of the yellow and dark red brown colluvium. The analyses were carried out on 20% aqueous extracts of the received samples using deionised water.

5.3.1 Cation Exchange Capacity (CEC)

The CEC provides an indication of the ease with which nutrients can be absorbed and retained by charged substances in the soil. The CEC of the yellow and red brown colluvium materials are very low and less than 1.1 cmol_c/kg, with a maximum CEC of 4 cmol_c/kg. These values are very low compared to the CEC of temperate climate soils, where values of 4 to 8 cmol_c/kg classify as “low CEC”. This is believed to be the direct result of the high kaolinite content which, due to its 1:1 structure, is only able to allow for the minimal exchange of ions (Hintermaier-Erhard and Zech, 1997).

5.3.2 pH

The significant leaching of bases from the soil through the high levels of annual precipitation results in acidic conditions within tropical soils. This was confirmed during the chemical analyses from this study, where the average pH of the colluvium material equals 5.8 and the maximum and minimum pH equals 5.0 and 6.3 respectively. At low pH values, any aluminium contained in soil minerals becomes soluble, resulting in a concentration increase of Al³⁺ in the soil solution which may have toxic effects on plants and soil organisms. Under dense vegetation, such as in tropical rainforests, this effect may not be as obvious because of the rapid uptake of basic nutrients (K⁺,

Ca^{2+} , Mg^{2+}) by the plant root system, causing higher concentrations of these elements in the upper horizon and therefore higher pH levels (Hintermaier-Erhard and Zech, 1997).

5.3.3 Conductivity

Electrical conductivity of the colluvium samples averages 3.83 mS/m, with a minimum and maximum of 1.12 and 9.6 mS/m respectively. This indicates very low levels of solutes present in the material and subsequent potential corrosivity of the material, with conductivity values of temperate climate soils less than 10 mS/m classifying as “very low”.

5.3.4 Dispersivity

Dispersion occurs in soils with a high percentage of exchangeable sodium percentage (ESP), causing internal erosion and eventually piping through embankment dams. The tendency for dispersive erosion in a given soil depends on the mineralogy and chemistry of the clay and the dissolved salts in the soil water and eroding water. Based on the available chemical and double hydrometer test results, the colluvium from the case study area has a very low concentration of exchangeable sodium and classifies as having a potential dispersivity of 0%.

5.4 Discussion

From the chemical results and the mass yield calculations, iron (Fe^{3+}) forms the major chemical constituent of the red brown colluvium and transported ore materials, typically varying between 40 and 60% in content for the red brown colluvium material and 60 to 80% for the transported ore. The iron content of the yellow brown colluvium is lower than that of the aforementioned horizons, varying between approximately 30 and 40%, overlapping partially with the observed iron oxide content of the red brown colluvium material. The average aluminium oxide (Al^{3+}) content of the yellow brown colluvium is significantly higher (23%) than that of the red brown colluvium (16%) and transported ore (13.5%). SiO_2 content of the different horizons varies significantly, with SiO_2 contents varying between 16 to 38% for the red brown colluvium, 6 to 27% for the transported ore and 22 to 27% for the yellow brown colluvium.

When analysing the variation in the major chemical constituents for the overall soil profile with change in elevation, it is evident that there is a marked decrease in Fe^{3+} with a decrease in elevation over the mining area. Conversely an increase in the overall SiO_2 is observed with decrease in elevation, with the Al^{3+} remaining fairly constant with change in elevation. Leaching of the dissolved primary minerals from the soil profile is vital in the development process of tropical red soils. Removal of bases and combined silica leads to further decomposition and the breakdown of aluminium silicate clay minerals and the formation of hydroxides and sesquioxides. The concentration of kaolinite is significantly higher in the yellow brown colluvium found from the middle to lower slopes of the topography due to the greater degree of weathering the material has been subjected to, with also moderate to high percentages of goethite. The quartz content of the yellow brown colluvium is however significantly lower than that of the transported ore and red brown colluvium, which is contradictory to the increase in silica content noted from the XRF analysis of the yellow brown colluvium.

Amorphous or poorly ordered forms of silica are common constituents of tropical soils, particularly in constantly humid, well-drained environments (Northmore *et al.*, 1992). The

recognition and characterization of such forms during XRF and XRD analysis may however prove challenging for a number of reasons (Drees *et al.*, 1989). The foremost of these is that amorphous silica has a nondescript morphology and often occurs as a thoroughly diffused cementing agent in a clay-rich matrix so that it cannot be effectively isolated and analysed by commonly used techniques such as XRD. Silica could be deposited from soil solution within the soil voids during drying of the soil. Silica polymers may be adsorbed on the hydroxyl planes exposed on the (001) face of kaolinite crystals which provide an opportunity for further deposition of silica (Chadwick *et al.*, 1987).

Based on this information and XRF data, it is possible that some of the high quartz contents reported from the XRD analysis for the transported ore and dark red brown colluvium may rather represent amorphous silica. The “decrease” in quartz content in the yellow brown colluvium may in that case represent amorphous silica going into solution and deposited onto the kaolinite crystal faces, accounting for an apparent decrease in “quartz”, but increase in general silica content of the lower slope, yellow brown colluvium material.

Based on the available analysis of the samples of colluvium material from the lower-lying areas outside the mining area, it is clear that kaolinite and quartz form the major mineralogical components of the yellow brown colluvium material, with minor goethite and very low hematite contents.

6. Index Properties

Laboratory test analysis of the index properties of the colluvium (dark red and yellow brown) found across the study area was completed in 2013. Representative samples were collected from both the proposed mining area and from those lower-lying areas where mine infrastructure development outside the mining area will take place in an effort to investigate the lateral variation (if any) in index properties over the study area, as well as the vertical change with the change in the elevation. This included evaluating the particle size distribution (sieve and hydrometer analysis) and Atterberg Limits, Natural Moisture Content (NMC) and particle density (“specific gravity”).

From the available index property test results, the majority of the index characteristics of the red brown and yellow brown colluvium horizons were found to be very similar and are discussed only as “colluvium”, unless distinguished otherwise.

6.1 Particle Size Distribution

From the available particle size distribution results, the gradings of the red brown and yellow brown colluvium horizons were found to be very similar. The average ranges of the various grain size fractions for both the red and yellow brown colluvium both inside and outside the mining area is summarised in Table 6-1 below.

Table 6-1 Average range of various grain size fractions for colluvium horizons

Fraction	Range (%)
Gravel	0-15
Sand	20-30
Silt	10-60
Clay	10-55

From these results, the colluvium material can be considered as being gap-graded, with a very low gravel-sized fraction and low average grading modulus value of 0.60.

It is evident from the grain size distribution curves that two distinct “clusters” of results exist based on the clay content for the colluvium outside the mining area, with one group of samples averaging 50% clay content and the other 16% clay (Figure 6-1). It is however clear that for those samples where the clay content of the material is higher, the overall silt content is significantly lower compared to those samples of lower clay content and consequently the overall fines content (silt + clay fraction) of all colluvium samples is found to be similar. Additionally, a distinct and sudden decrease of between 10 and 20% in the particle distribution curves occurs between sieve and hydrometer analysis at the 0.075mm particle size. This sudden drop is believed to largely be a function of the dispersing agent used during hydrometer analysis.

No distinct grouping of colluvium material according to individual grain size distributions were observed in the mining area and the transition between sieve and hydrometer analysis is gradual (Figure 6-2).

ENGINEERING GEOLOGICAL EVALUATION OF TRANSPORTED TROPICAL RED SOILS

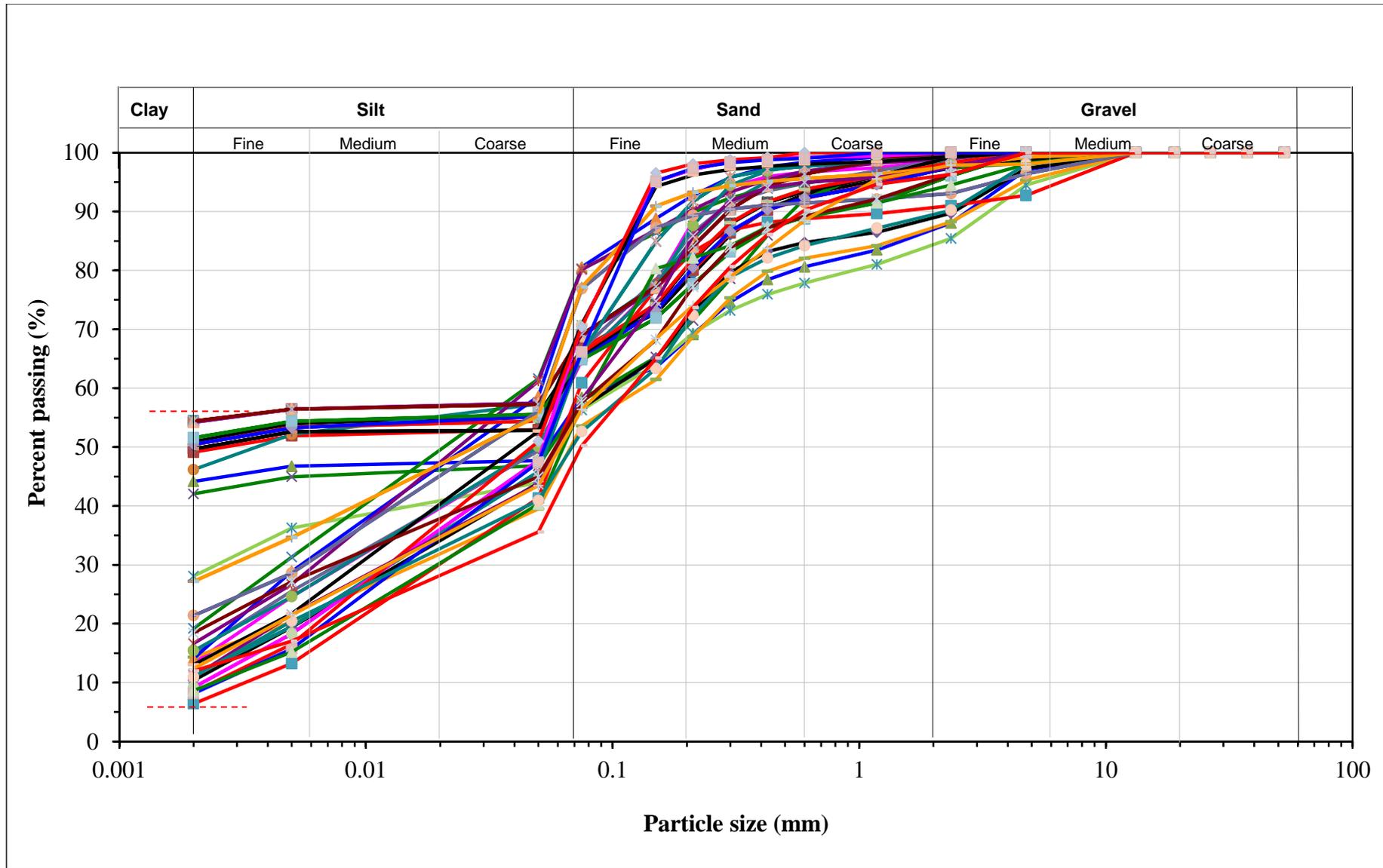


Figure 6-1 Distribution of particle sizes of sampled colluvium horizons from outside mining area highlighting the two distinct clusters of results

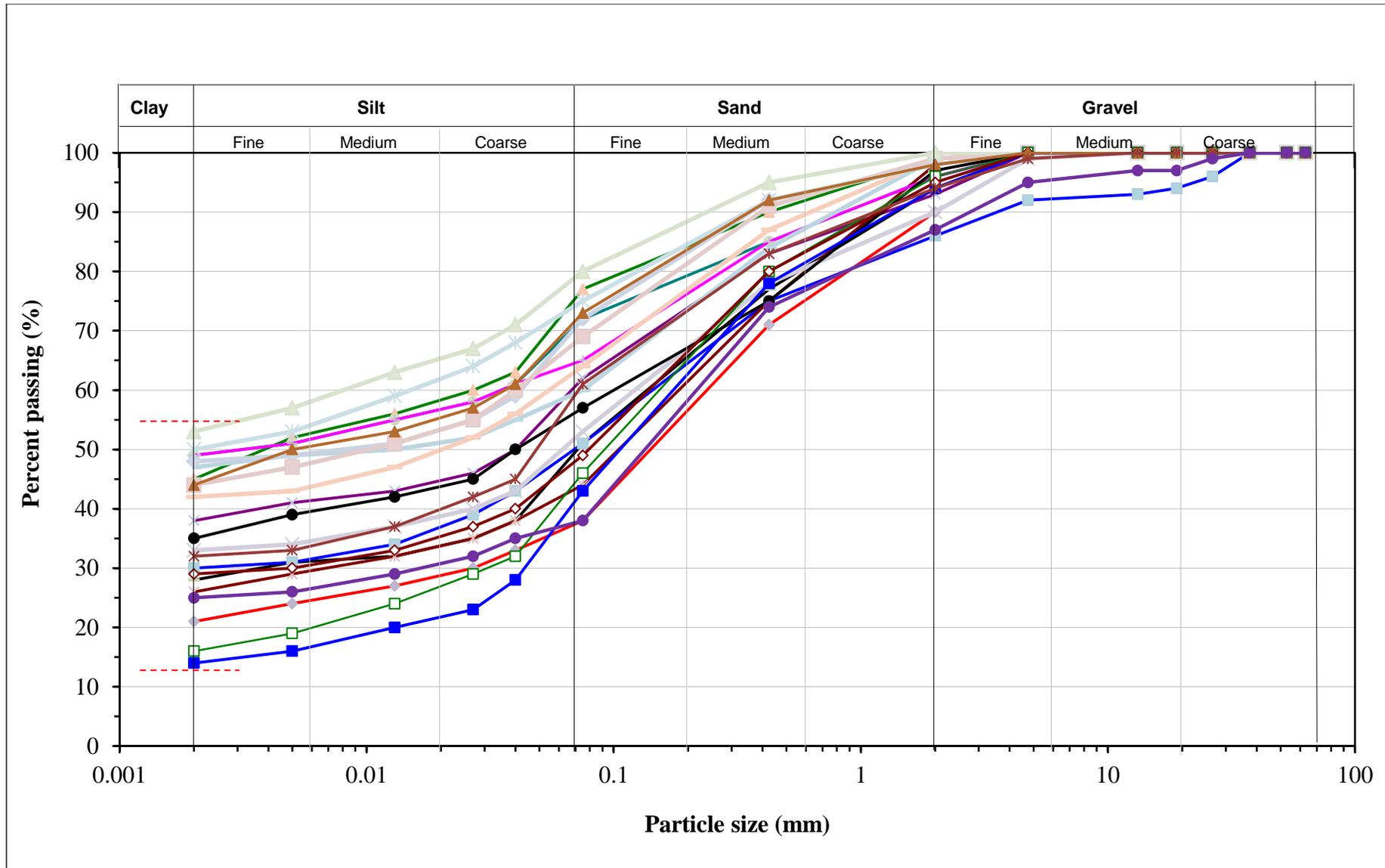


Figure 6-2 Distribution of particle sizes of sampled colluvium horizons in mining area

From the available results, the matrix classifications of the respective colluvium samples predominantly vary between clayey sand, sandy clay and sandy silt (Figures 6-3 and 6-4) and full classifications of sandy fines (Figures 6-5 and 6-6).

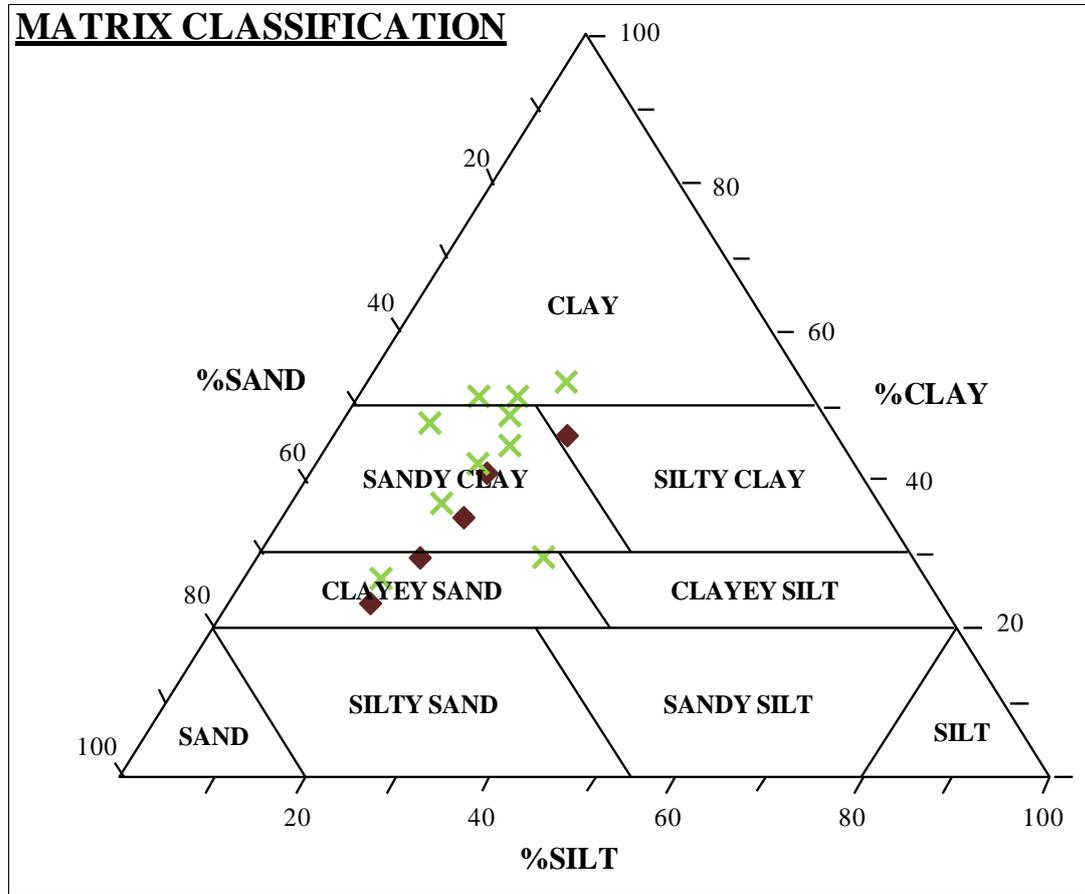


Figure 6-3 Matrix classification of colluvium samples from mining area

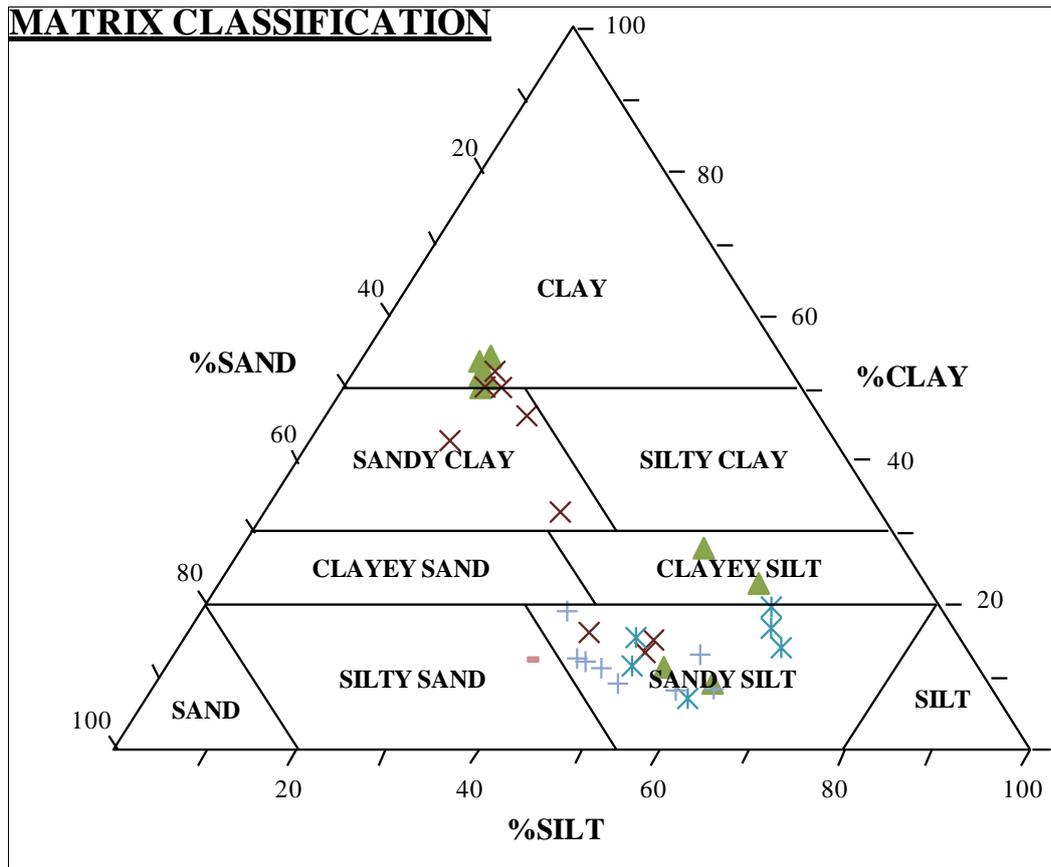


Figure 6-4 Matrix classification of colluvium samples from outside mining area

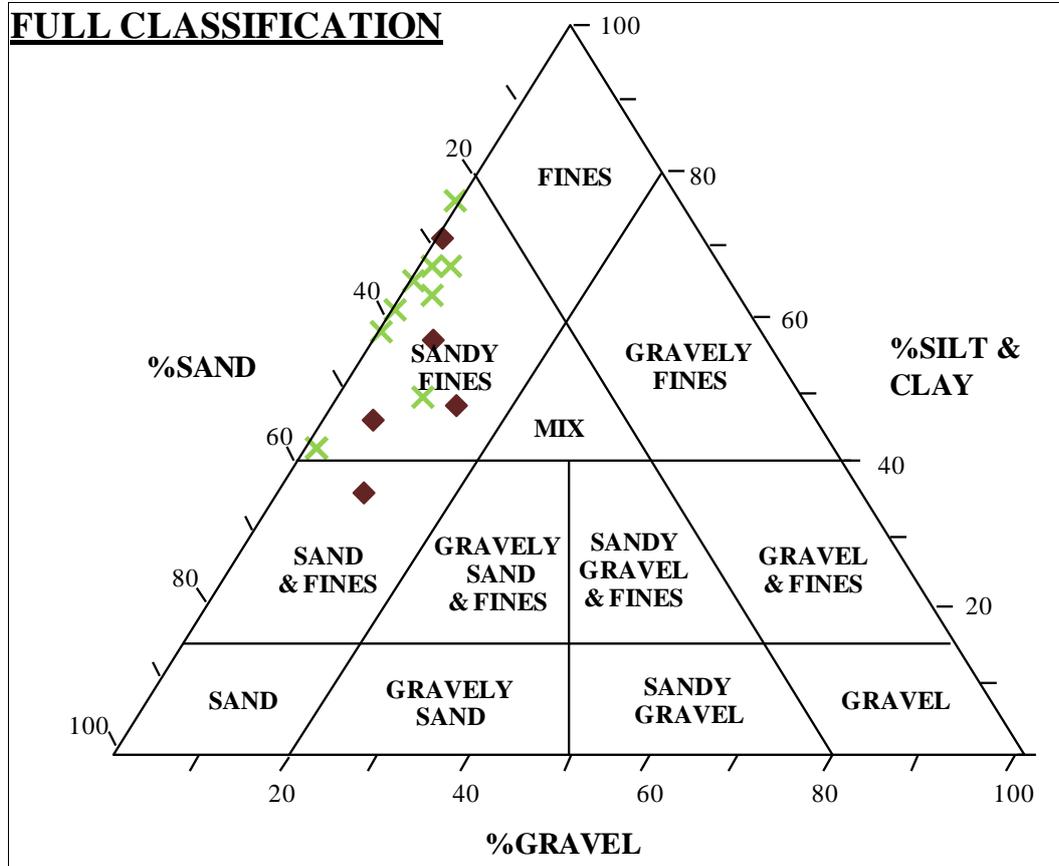


Figure 6-5 Full classification of colluvium samples from mining area

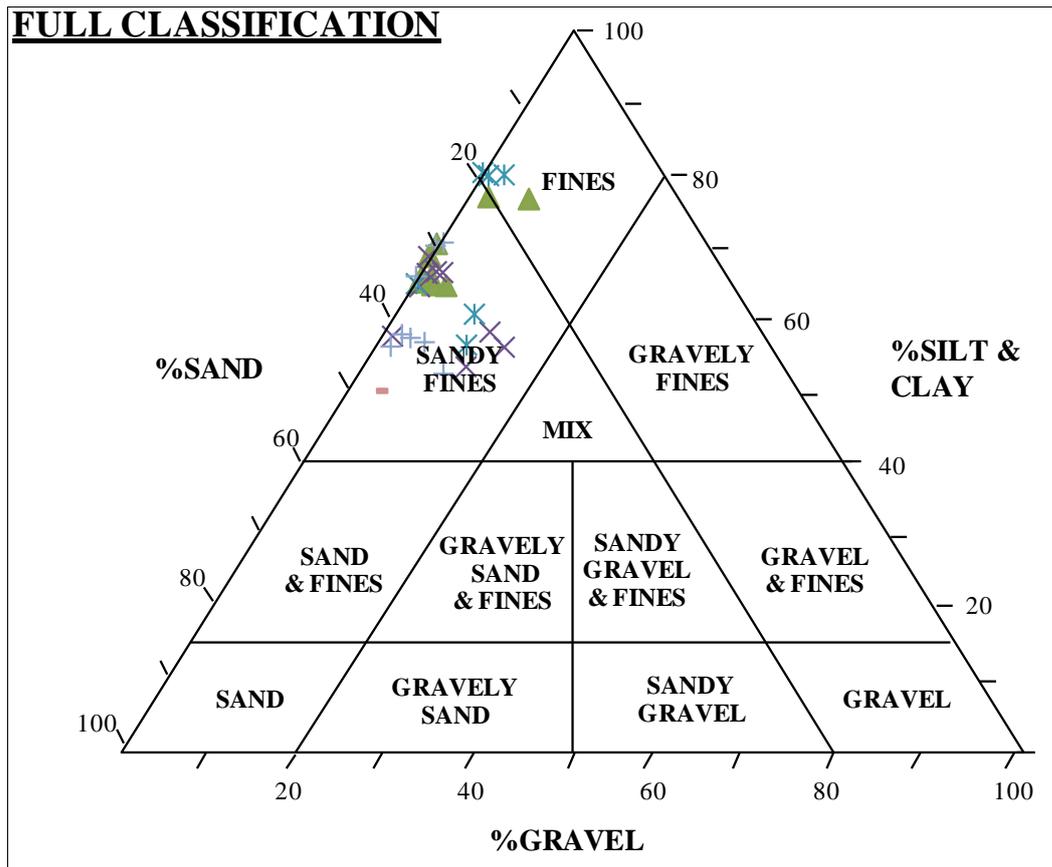


Figure 6-6 Full classification of colluvium samples from outside mining area

It is evident from these classification results that those colluvial horizons found in the lower-lying areas outside the mining area either classify as sandy silt or sandy clay to clay, comprising either large percentages silt or clay with similar lesser percentages of sand.

The colluvium material found in the mining area was sampled from near the crest of Mt. Lekoumou to the lower slopes. Consequently the grain size distributions of the colluvium material vary greatly, with matrix classifications varying from clay to clayey sand to sandy clay (Figures 6-7 and 6-9).

When considering the variation in the particle size distributions of the colluvium materials according to the position of the investigated profile in the slope of the mining area and subsequent degree of weathering, it is evident that those horizons near the bottom of the slope exhibit higher clay contents than the colluvium horizons found at higher elevations. The colluvium typically classifies as silty sand to sandy clay at elevations between 720 to 810 m.a.m.s.l. and as sandy clay to clay at elevations 650 to 720 m.a.m.s.l. (matrix classification).

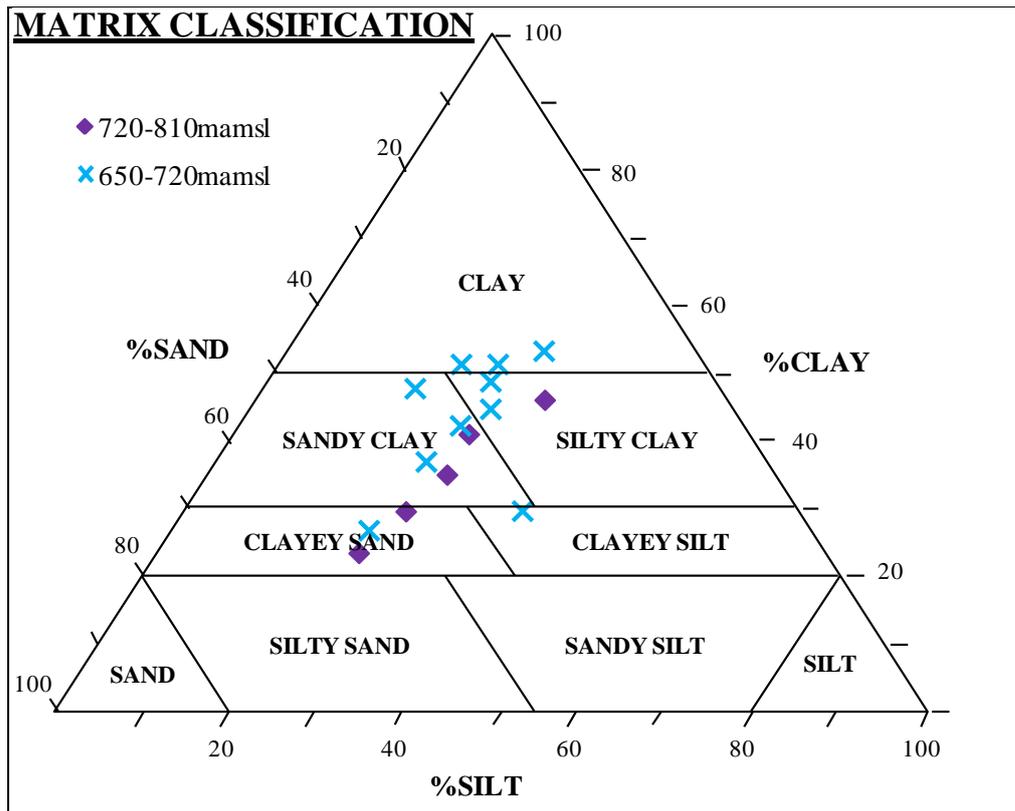


Figure 6-7 Matrix classification of yellow brown colluvium samples in mining area according to elevation

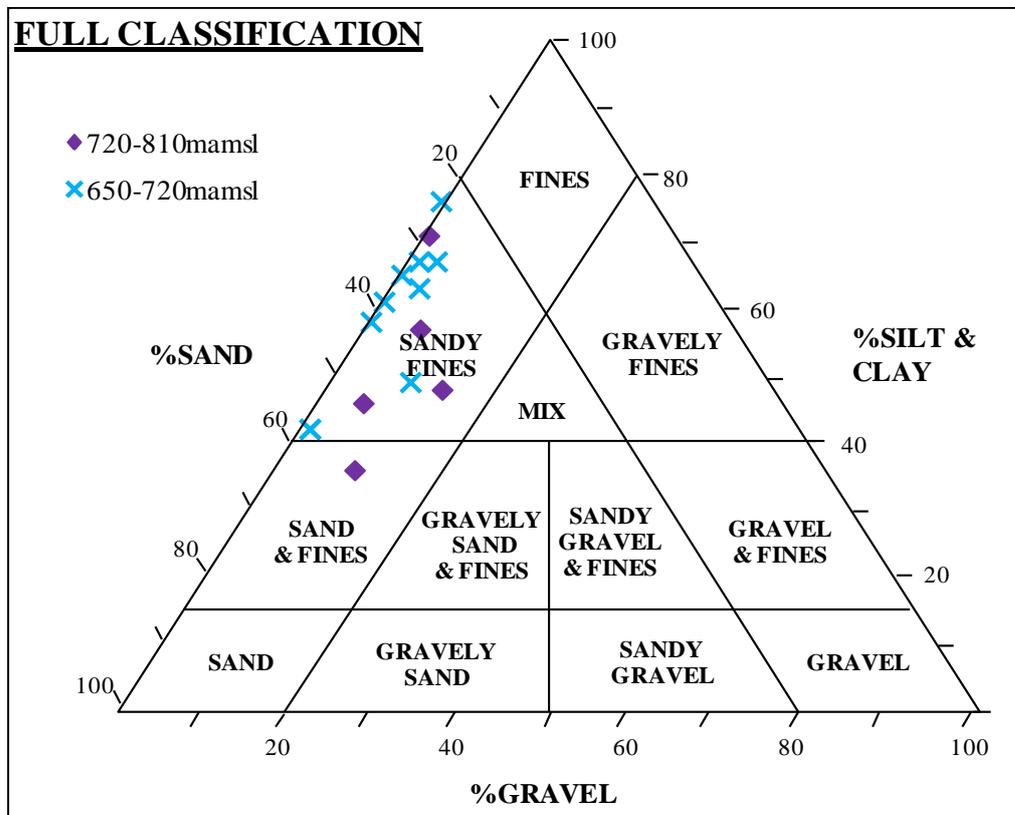


Figure 6-8 Full classification of yellow brown colluvium samples in mining area according to elevation

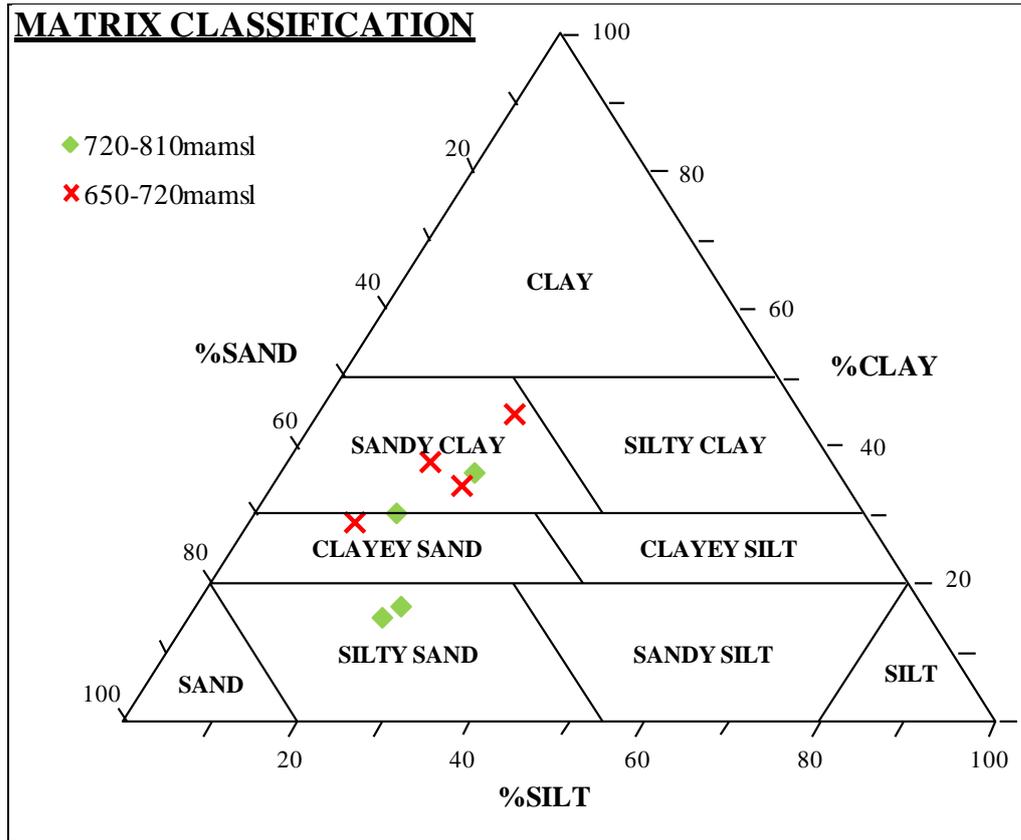


Figure 6-9 Matrix classification of red brown colluvium samples in mining area according to elevation

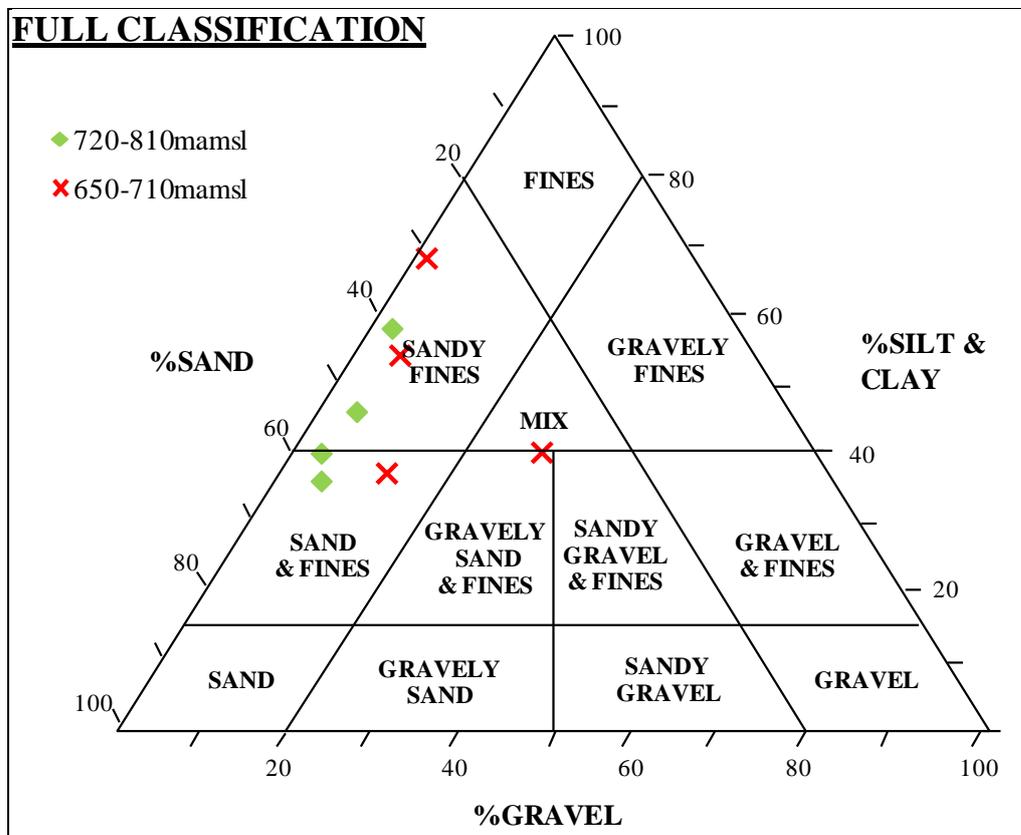


Figure 6-10 Full classification of red brown colluvium samples in mining area according to elevation

No pre-drying was done prior to testing due to the expected sensitivity to change in natural moisture content and based on the visual observations of the material behaviour on site and particle size distributions were determined from the natural or “as-received” moisture content.

6.2 Atterberg Limits

The calculated Atterberg Limits of the colluvium horizons were plotted on the Casagrande Chart as a function of the respective sample’s plasticity index and liquid limit (Figures 6-11 and 6-12). Based on the established relationship of these parameters for temperate climate soils, at liquid limit values of 50% or greater the A-line separates clays of high plasticity, or fat clays (above line), from inorganic silts and organic clays of high compressibility (below line). At liquid limits less than 50%, the A-line separates inorganic clays of low to medium plasticity (above line) from inorganic and organic silts of low to medium compressibility (below “A” line).

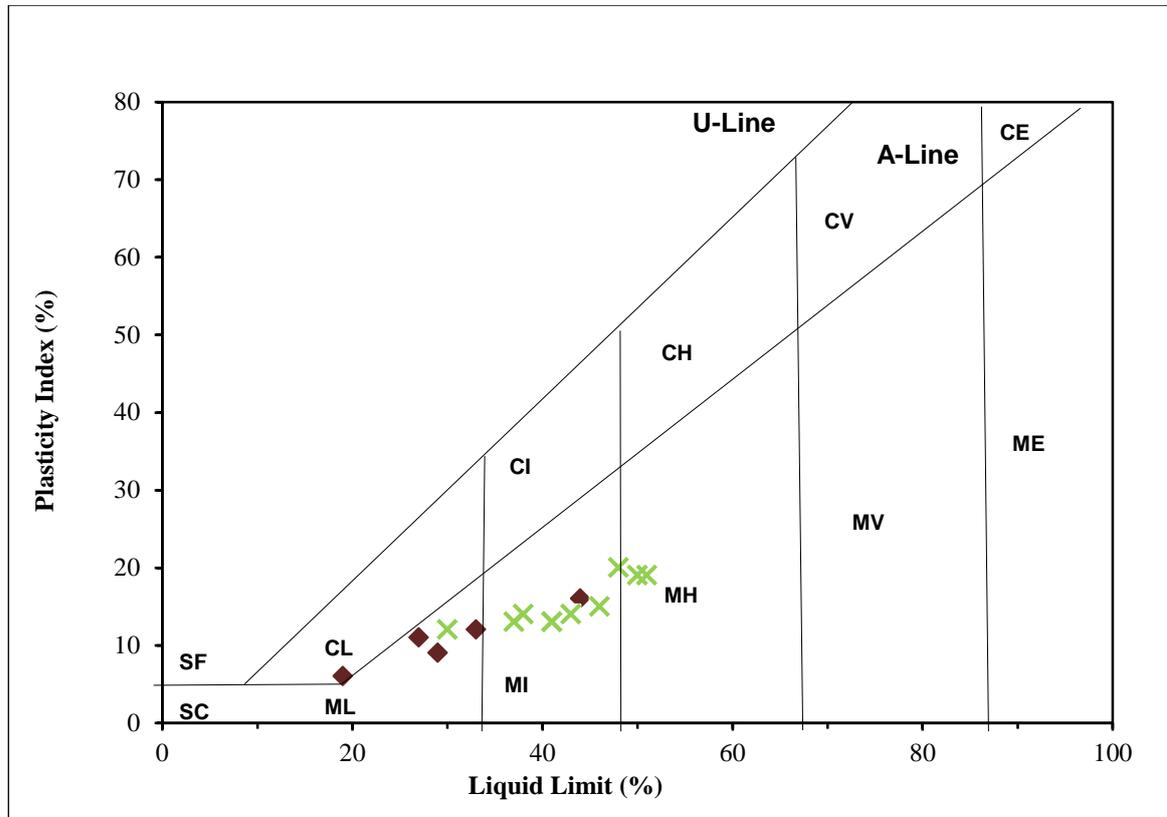


Figure 6-11 Casagrande Chart of colluvium samples in mining area

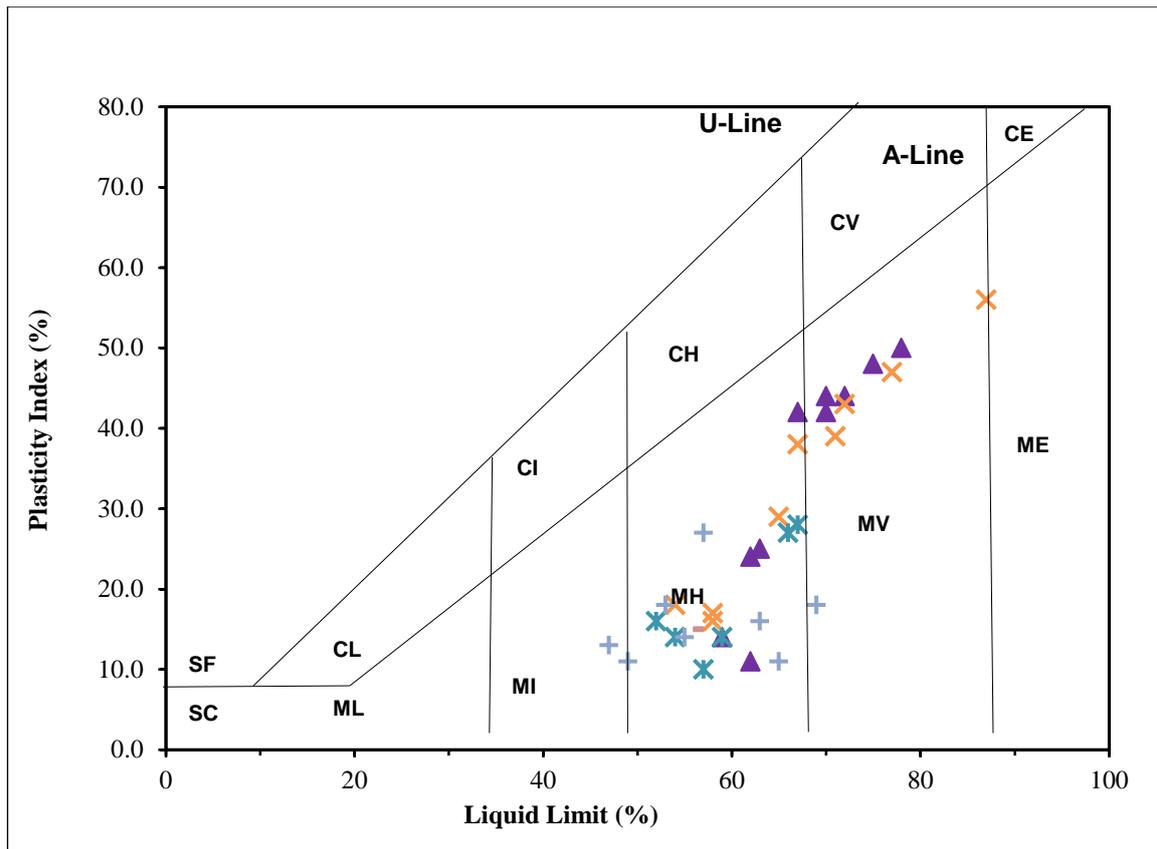


Figure 6-12 Casagrande Chart of colluvium samples from outside mining area, with two distinct sets of results indicated

The calculated plasticity indices and liquid limits of the colluvium from the mining area were found to be representative of the predominantly kaolinitic mineralogical composition derived from the XRD and XRF analysis. The colluvium material is characterized by very high liquid limit values, but comparatively low to moderate plasticity index values for such high liquid limit values. Consequently, when plotted on the conventional Casagrande Chart the colluvium material tends to always plot below the A-line. When assuming the relationship established for those soils from temperate climates holds true and can be applied to tropical soils, at liquid limit values of 50% or greater the colluvium can be classified as inorganic silts and organic clays of high compressibility (MH and MV) and at liquid limits of less than 50% as inorganic silts of low to medium compressibility (ML and MI). A general increase in plasticity index and liquid limit values is apparent with a change in elevation in the mining area, where there is a general increase in the plasticity index and liquid limit of the colluvium horizons with decrease in mean elevation. This trend is illustrated in Figure 6-13 and Figure 6-14 below.

It is evident that the range of plasticity index, liquid limits and the ratios of these values to one another varies significantly for the colluvium material outside the mining area, classifying from MI to significantly higher and closer to the A-line in the MH-MV categories.

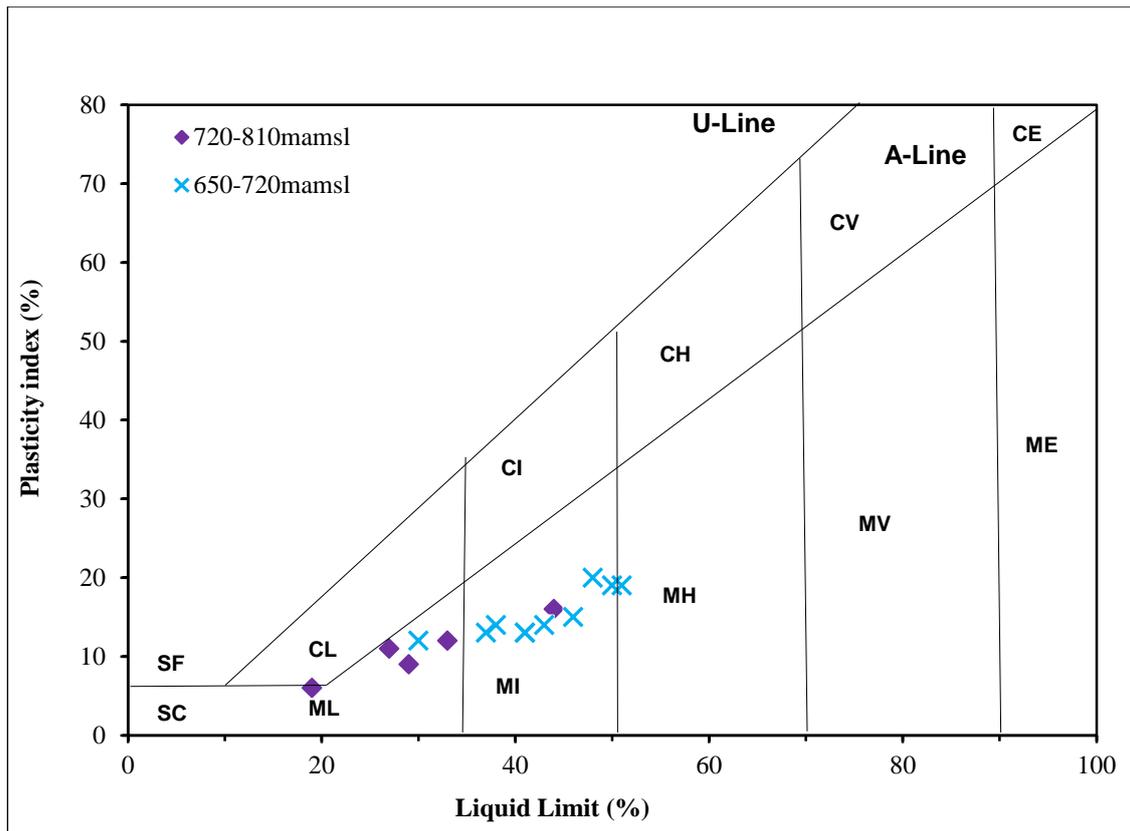


Figure 6-13 Casagrande Chart of colluvium samples in mining area according to elevation

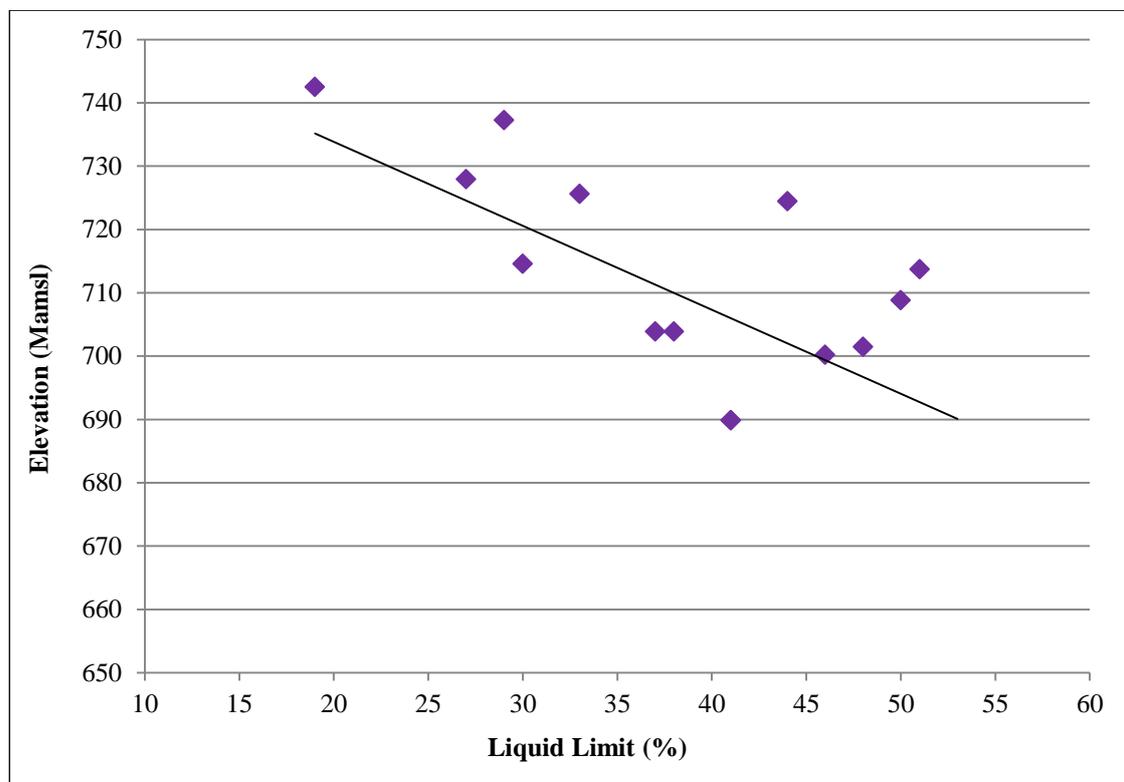


Figure 6-14 Relationship between elevation and liquid limit of colluvium in mining area

The liquid limits of a number of colluvium samples were determined using both the cone penetrometer and the Casagrande apparatus. The results are summarised in Figure 6-15 below.

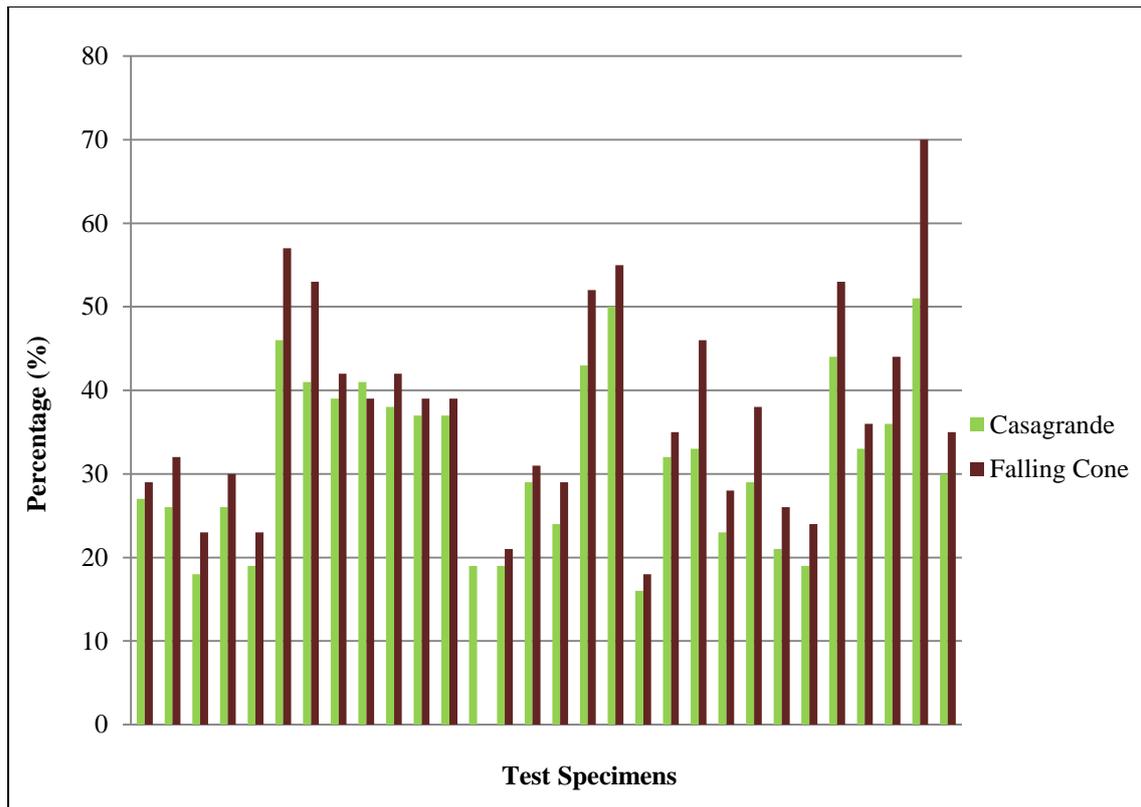


Figure 6-15 Evaluation of the liquid limits established using a) the Casagrande cup and b) Falling Cone apparatus

Linear shrinkage (or one-dimensional shrinkage) tests give an indication of the amount of shrinkage by determining the change in length of a cylindrical sample of soil upon change in moisture content. It does not measure volumetric shrinkage, but does give an indication of the amount of shrinkage that can be expected on drying. From the available results the linear shrinkage of the colluvium varies between 2% and 18%.

The observed shrinkage is additionally found to be irreversible due to the observed changes in material characteristics that take place on change in moisture content. The calculated linear shrinkage values were plotted for all the colluvium samples against their calculated plasticity indices in an effort to evaluate the potential relationship between these properties (Figures 6-13 and 6-14). Northmore *et al.* (1992) established the relationship between linear shrinkage and plasticity index for a wide range of soils as:

$$IP = 2.5L_s - 15.2 \quad (\text{Eq. 6-1})$$

Where:

IP = Plasticity Index

Ls = Linear Shrinkage

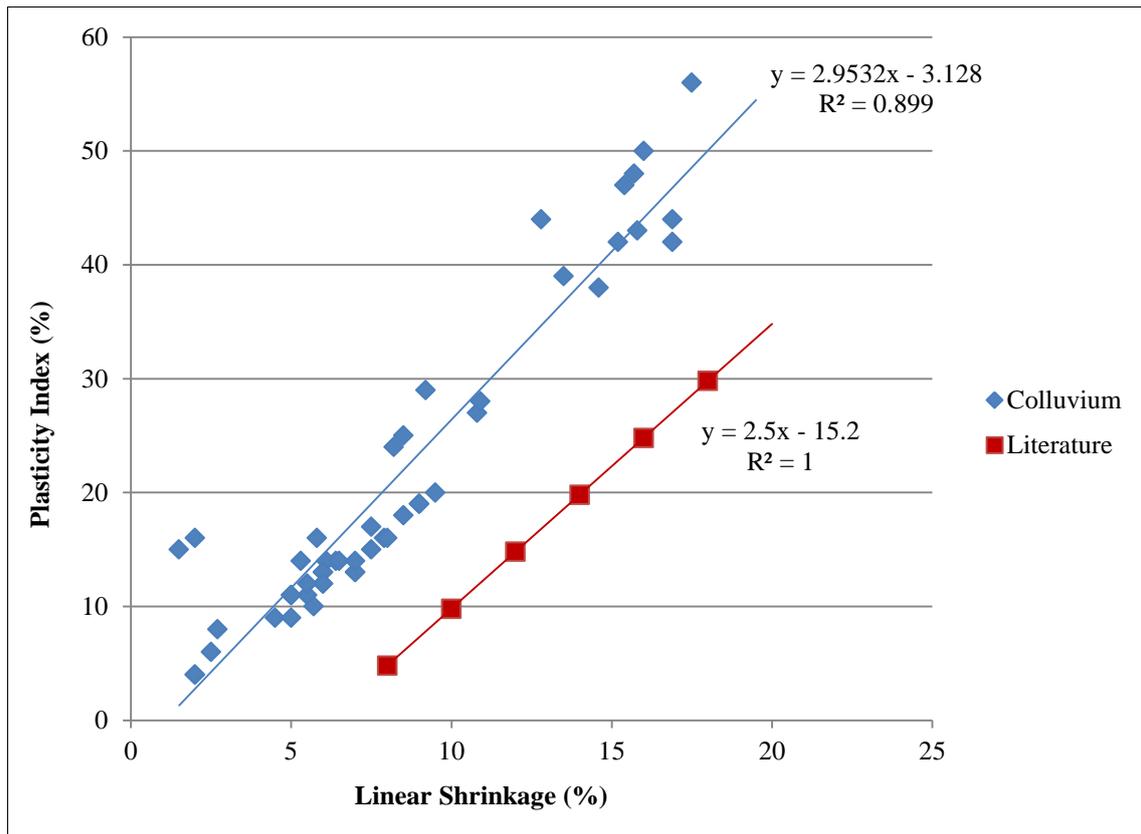


Figure 6-16 Relationship between plasticity index and linear shrinkage of all colluvium samples compared to the relationship established in the literature

It is evident that there exists a near linear relationship between the linear shrinkage and the calculated plasticity index for the colluvium samples, but the ratio of the plasticity index to linear shrinkage for colluvium from both inside and outside the mining area is higher than that established in the available literature for tropical red soils. The established plasticity index of the colluvium is on average 10 to 20% higher than the plasticity index established from the relationship in the literature for a particular linear shrinkage value. The calculated linear shrinkage values were plotted against the calculated liquid limits. Northmore *et al.* (1992) established the relationship between linear shrinkage and liquid limit for a wide range of tropical red soils (excluding andosols) as:

$$LL = 3.9L_s + 6.3 \quad (\text{Eq. 6-2})$$

Where:

LL = Liquid Limit

Ls = Linear Shrinkage

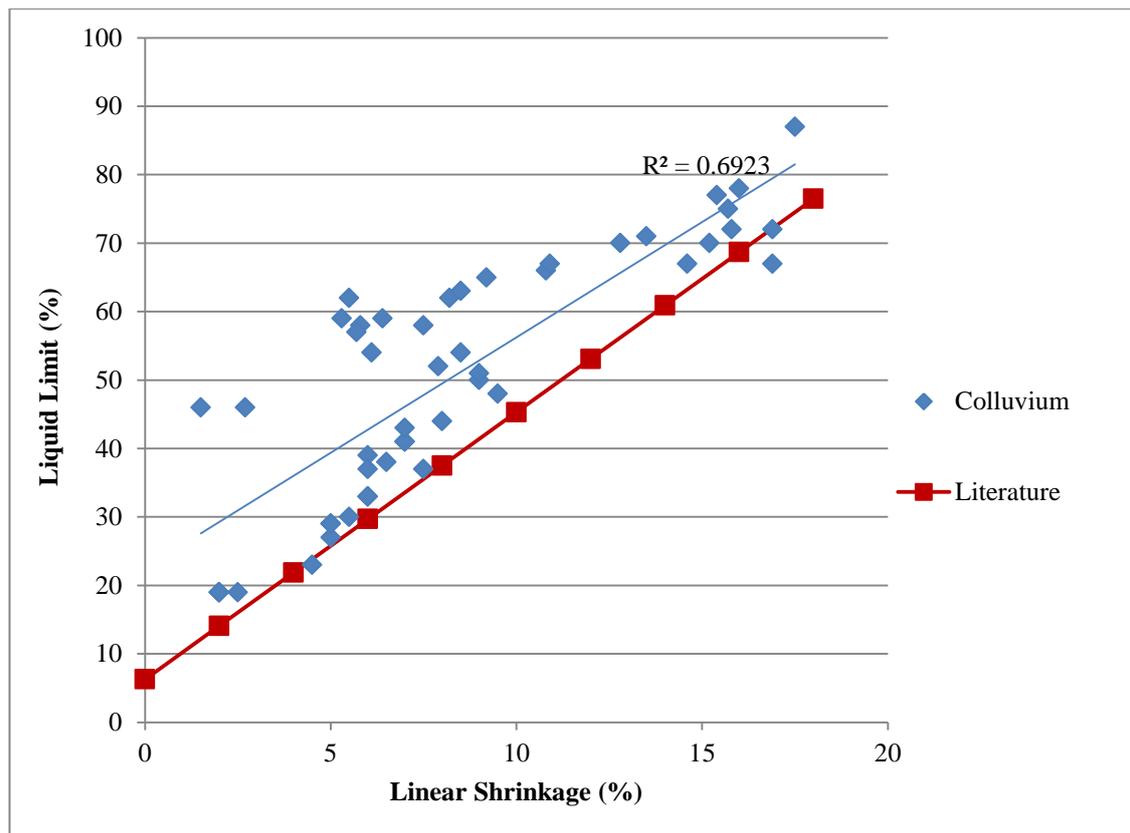


Figure 6-17 Relationship between liquid limit and linear shrinkage of all colluvium samples compared to the relationship established in the literature for different tropical red soils

The relationship between the linear shrinkage and the calculated liquid limits for the colluvium samples generally exceed the relationships established for these parameters in the available literature for tropical red soils at various stages of weathering, but correlates better than the relationship established for the plasticity index and linear shrinkage.

In an effort to assess the activity of the colluvium soils through simple index test results, the plasticity indices of the different colluvium samples were plotted against the clay content of the material. Assuming the potential activity can be taken as the ratio of the plasticity index to the percentage clay, Skempton (1953) distinguished between four main categories of clay minerals based on the ratio between the calculated plasticity index (I_a) and clay content:

- Inactive Clays – Activity < 0.75
- Normal Clays – 0.75 < Activity < 1.25
- Active Clays – 1.25 < Activity < 2.0
- Highly Active Clays – Activity > 2.0

Figure 6-18 and Figure 6-19 illustrate the potential activity of all the colluvium samples from both inside and outside the mining area according to the aforementioned potential activity classes.

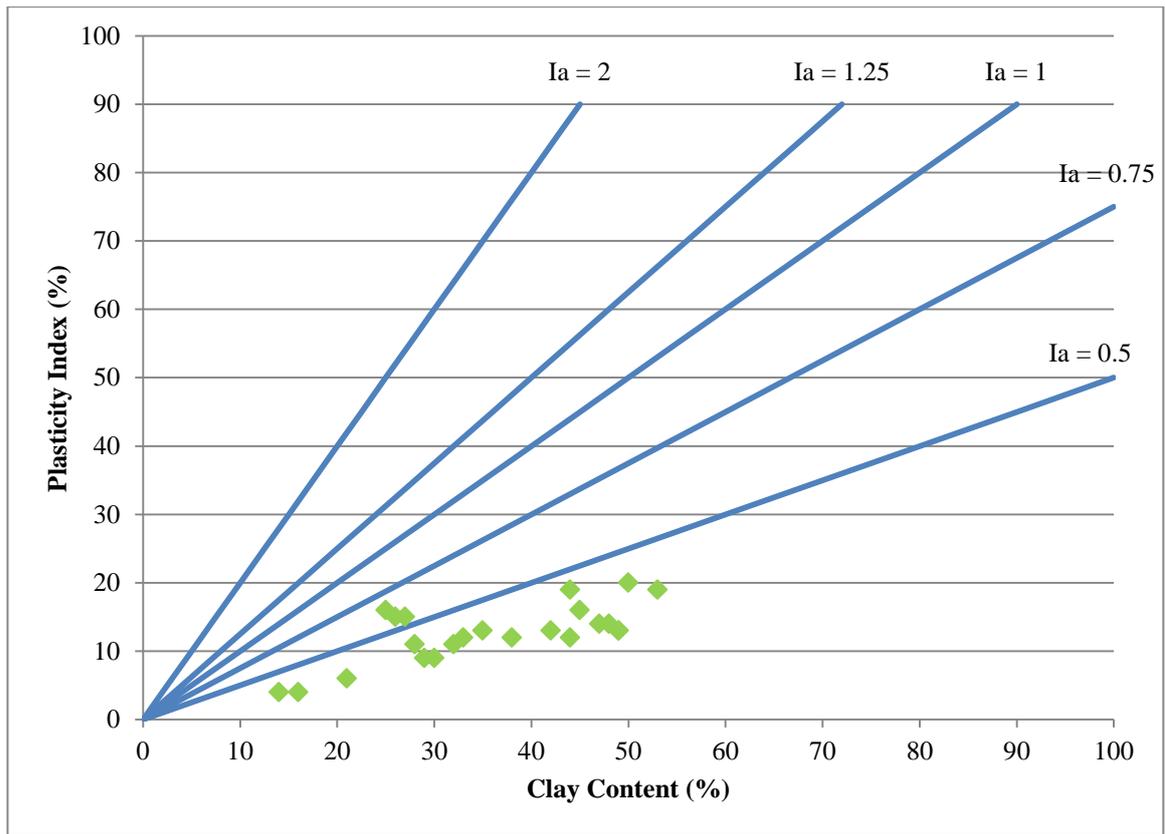


Figure 6-18 Activity chart of colluvium samples in mining area

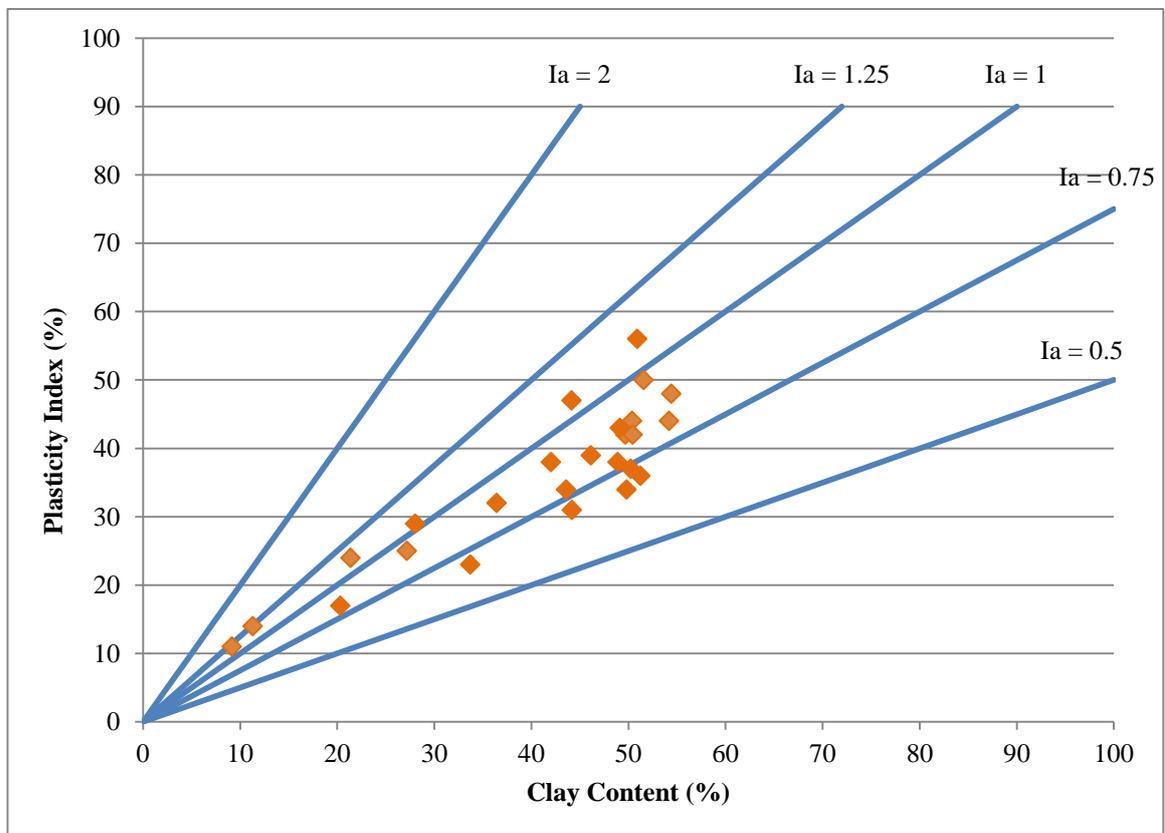


Figure 6-19 Activity chart of colluvium samples from outside mining area

Based on this relationship the average activity of the colluvium material inside the mining area equals 0.34, typical of kaolinite-rich material (Table 3-7). The average activity of the colluvium outside the mining area equals 0.87, but was found to generally vary between 0.6 and 1.23. Maeda et al. (1977) quoted andosol activities noted in the literature as ranging from 1.5 to below 0.6, which are echoed by the data presented here. The soils are therefore expected to be of higher potential activity than those kaolinite-rich soils from the mining area, but still classify as “Normal Clays” according to the classes derived by Skempton (1953).

The calculated potential activities are compared to the actual linear shrinkage percentages in an effort to gauge the accuracy of the potential activity estimation. The potential activity values were compared to the linear shrinkage values rather than the calculated swelling pressures as the swelling pressures are believed to represent a gross under-estimation of the potential activity of the material due to the high degree of saturation of the material at its natural moisture conditions. The linear shrinkage values were established on samples of which the natural moisture content was decreased through air-drying of the material, therefore believed to represent a more accurate actual representation of the maximum potential activity of the material.

Based on this relationship, the activity classification through Skempton’s method is believed to rather underestimate the potential activities of those samples of high clay contents and high plasticity indices. For such samples the actual linear shrinkage values of the particular samples reflect material for which significant shrinkage occurs on change in moisture content, but which classify as normal clays according to the aforementioned classification system.

Van der Merwe (1964) investigated the potential of clays to swell and drew up a chart of Total PI (Pg) versus total clay fraction in which zones of swell potential were defined ranging from low – medium – high – very high by a series of straight lines. Using this approach the activity of the colluvium can be evaluated according to Figure 6-20 and Figure 6-21.

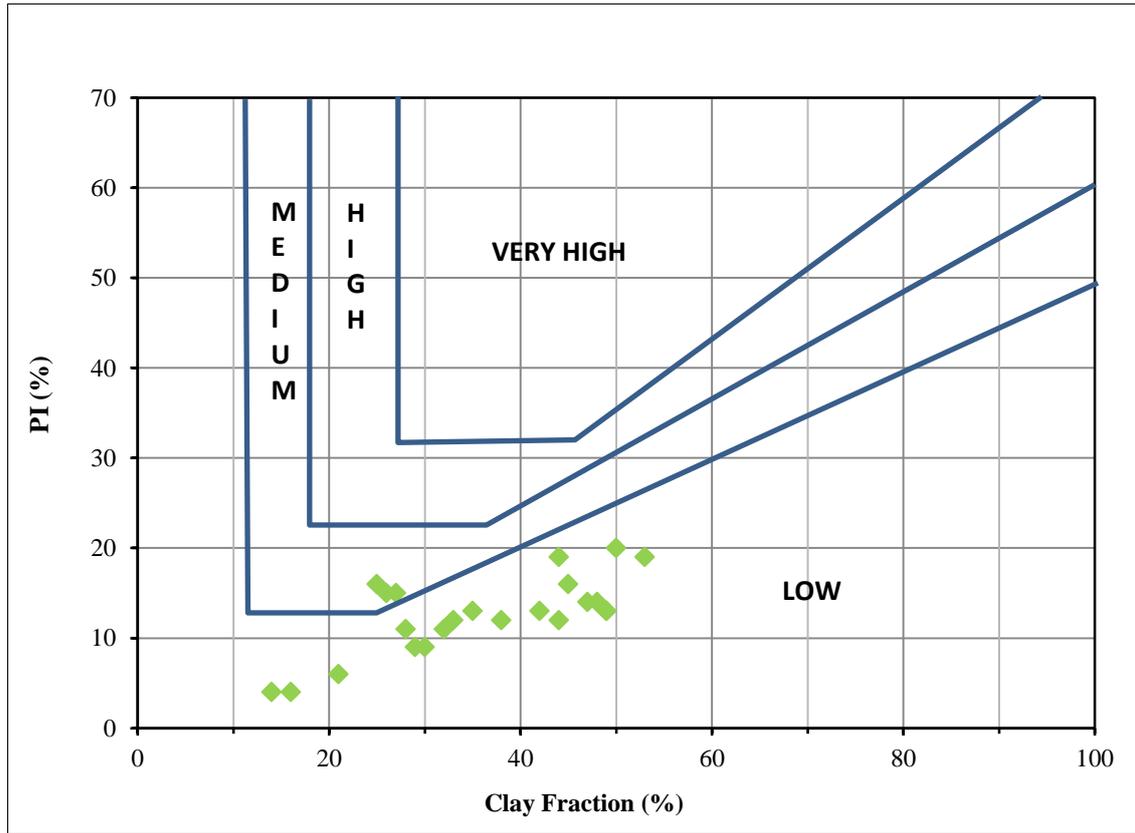


Figure 6-20 Van der Merwe activity chart of colluvium samples in mining area

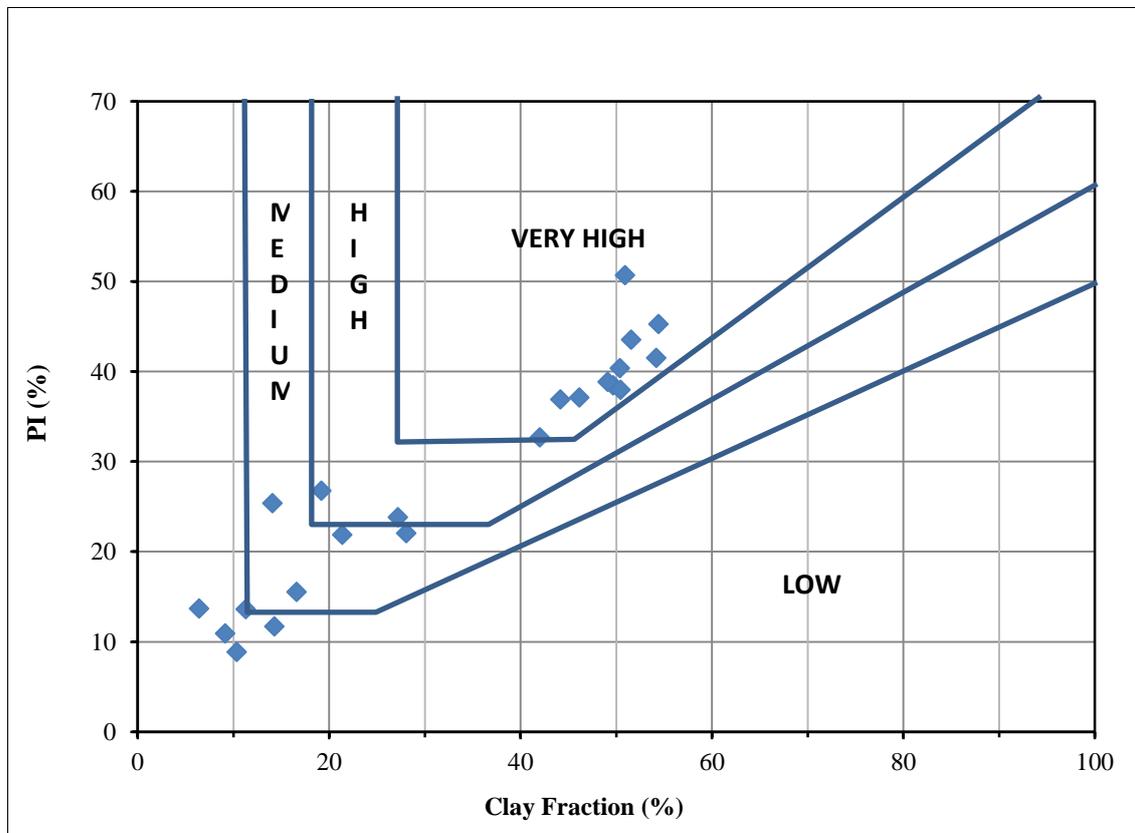


Figure 6-21 Van der Merwe activity chart of colluvium samples outside mining area

Van der Merwe's method of evaluating the potential activity of a soil uses the same parameters as that proposed by Skempton, but the overall plasticity index is adjusted to the weighted, or "gross", plasticity index of the entire sample. It is evident from these graphical presentations that the colluvium soils from outside the mining area are expected to be of higher potential activity than the colluvium found in the mining area. The evaluation through the Van der Merwe method, and contrary to Skempton's activity rating, is however believed to overestimate the potential activities of those samples of low to moderate clay contents and their equivalent gross plasticity indices, where the actual linear shrinkage values of the particular samples do not reflect material of equal activity.

No pre-drying was done on the majority of samples prior to testing due to the expected sensitivity to change in natural moisture content and based on the visual observations of the material behaviour on site and Atterberg Limits were determined from the natural or "as-received" moisture content.

In an effort to evaluate the sensitivity of the calculated Atterberg Limits to pre-drying of the sample prior to testing, the Atterberg Limits of 3 different colluvium samples from the study area were determined, each exposed to varying periods of air-drying. Table 6-2 below summarizes the average variation in Atterberg Limits for the different periods of air-drying and subsequent resultant moisture contents.

Table 6-2 Variation in Atterberg Limits when exposed to different drying periods

	Natural Moisture Content	Air Dried (1 Day)	Air Dried (3 Days)
Liquid Limit	58	51	48
Plastic Limit	30	31	30
Plasticity Index	28	20	18

6.3 Natural Moisture Content

The natural moisture contents of the colluvium materials over the study area were determined in an effort to evaluate the behaviour of soils during construction and as part of investigating the effect of soil suction pressures on the *in-situ* behaviour of the soils. Contrary to the air-drying method specified to mimic the effect of *in-situ* drying on the particle density and grading of selected sub-samples, the natural moisture contents of the material were determined using the oven drying method at a below normal drying temperature of 65°C.

No to very little change in moisture content of the material occurred during transit from site to the geotechnical laboratory in South Africa. Natural moisture contents of samples subjected to the same drying temperature (65°C) and for the same drying time period (24hours) in drying ovens on site and on arrival in South Africa vary by a maximum of 0.5%. Proper sealing of the undisturbed samples using a combination of two protective layers of clingfilm and tinfoil is therefore deemed to provide suitable preservation of the original natural moisture content from the time of sampling to testing, representing a duration of approximately two months.

From the graphical representation (Figure 6-22) of the natural moisture contents of the colluvium materials, it is evident that the moisture contents of the red and yellow brown colluvium horizons in the mining area may vary significantly.

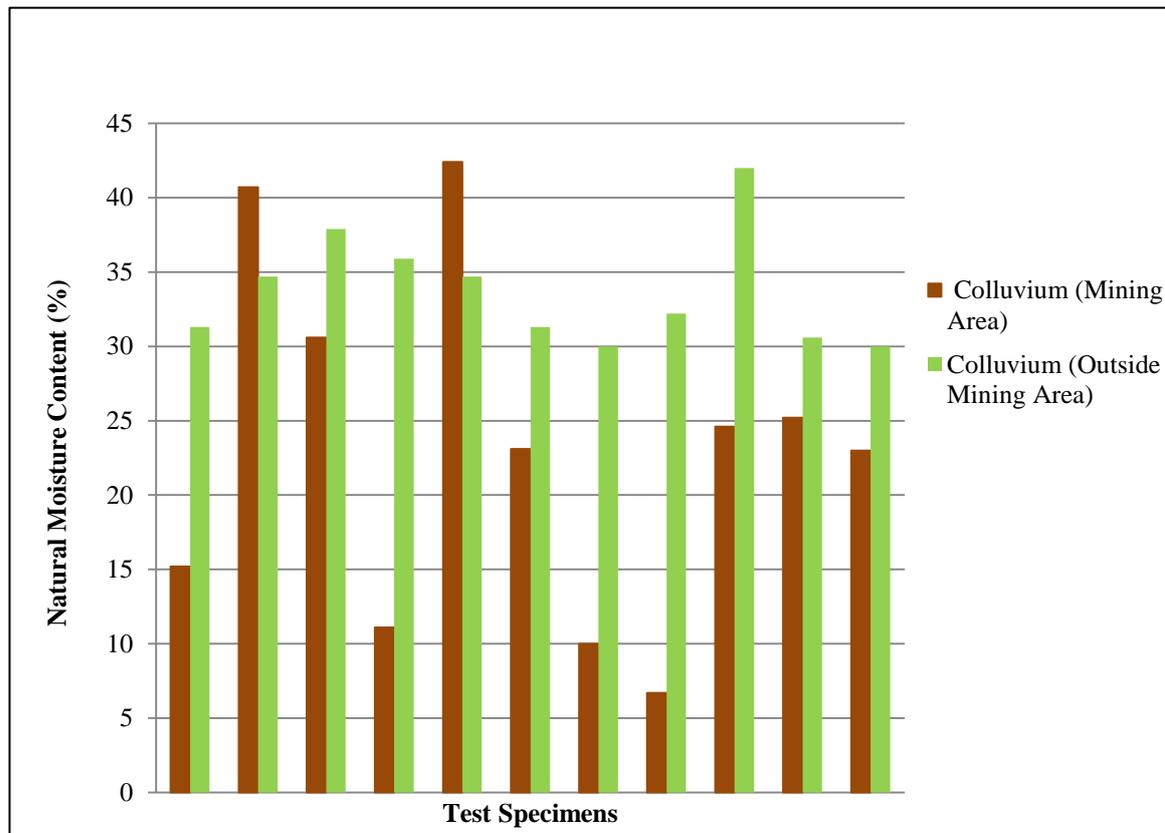


Figure 6-22 Natural moisture contents of colluvium samples

The colluvium samples from the low-lying, poorly-drained areas outside the mining area is characterized by a higher average moisture content (Average = 33.7%) than the colluvium material from the mining area (Average = 27.7%). In an effort to relate the natural moisture content of the colluvium material (for all material both inside and outside the mining area) to the calculated Atterberg Limits, it is necessary to correct to the equivalent moisture content (W_a), using the percentage material passing the 0.425 mm sieve (P_a):

$$W_a = \frac{100w}{P_a} \% \quad (\text{Eq. 6-3})$$

The weighted moisture contents of the colluvium material generally exceed the calculated plastic limits of the material, but are significantly lower than the high upper state boundary liquid limits. Using the principles of established phase relationships and along with the specific gravities and natural void ratios of the material (discussed in Section 7.4), the degree of saturation of the colluvium materials could be calculated. This was found to vary slightly for the colluvium material both inside and outside the mining area, but on average to be very high for all samples varying between 80% and 95% saturation at its natural moisture content.

The moisture contents of 5 yellow and dark red brown colluvium samples were established on site at two different drying temperatures of 105 °C and 65°C after a period of drying of 24-hours in an effort to evaluate the sensitivity of the material to oven drying at varying temperatures (Table 6-3).

Table 6-3 Variation in moisture contents determined at different drying temperatures

	NMC (%)		% Variation
	105°C	65°C	
Sample 1	20.22809	15.69525	4.532836907
Sample 2	18.00217	14.41787	3.584299577
Sample 3	23.50687	19.19934	4.307527455
Sample 4	29.62717	21.9934	7.633771835
Sample 5	26.73922	19.46488	7.274335627

6.4 Particle Density

Particle density or specific gravity (SG) gives an indication of the intrinsic density of the solid phase of the soil sample, and is independent of moisture content and voids. The particle density or specific gravity of the colluvium samples from the study area was completed according to the procedure set out in BS 1377-2 (1990), but from the natural or “as-received” moisture content and with no pre-drying of the material allowed apart from on those samples selected to evaluate the effect of drying on particle density (Figure 6-23).

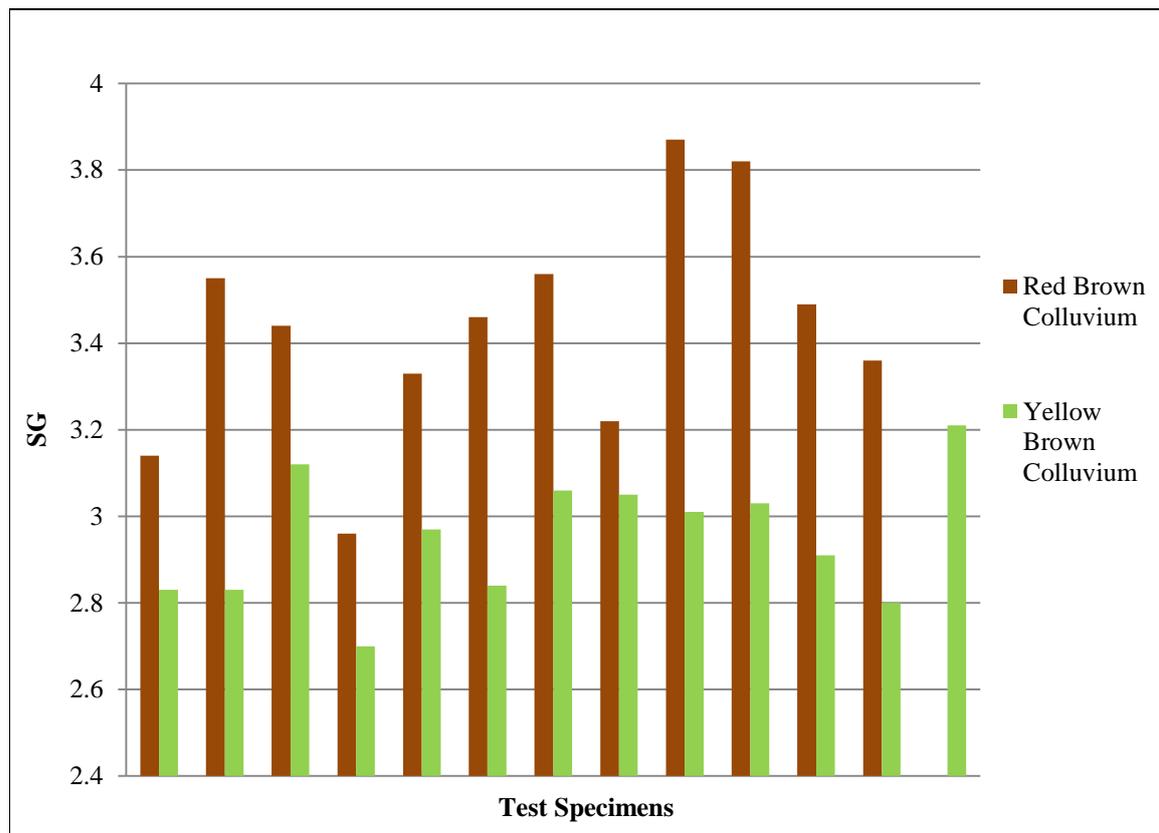


Figure 6-23 Calculated particle densities of yellow brown and red brown colluvium material – Mt. Lekoumou

From these specific gravity results, it is clear the dark red brown colluvium is expected to have a higher average particle density than that of the yellow brown colluvium material as found in the middle and lower slopes of the mining area and throughout the study area, with an average SG value of 3.45. The yellow brown colluvium has a lower average specific gravity value of 2.95.

The average specific gravity of the colluvium found in the lower-lying areas outside the mining area equals 2.75 and represents the lowest of all the materials in the study area (Figure 6-24).

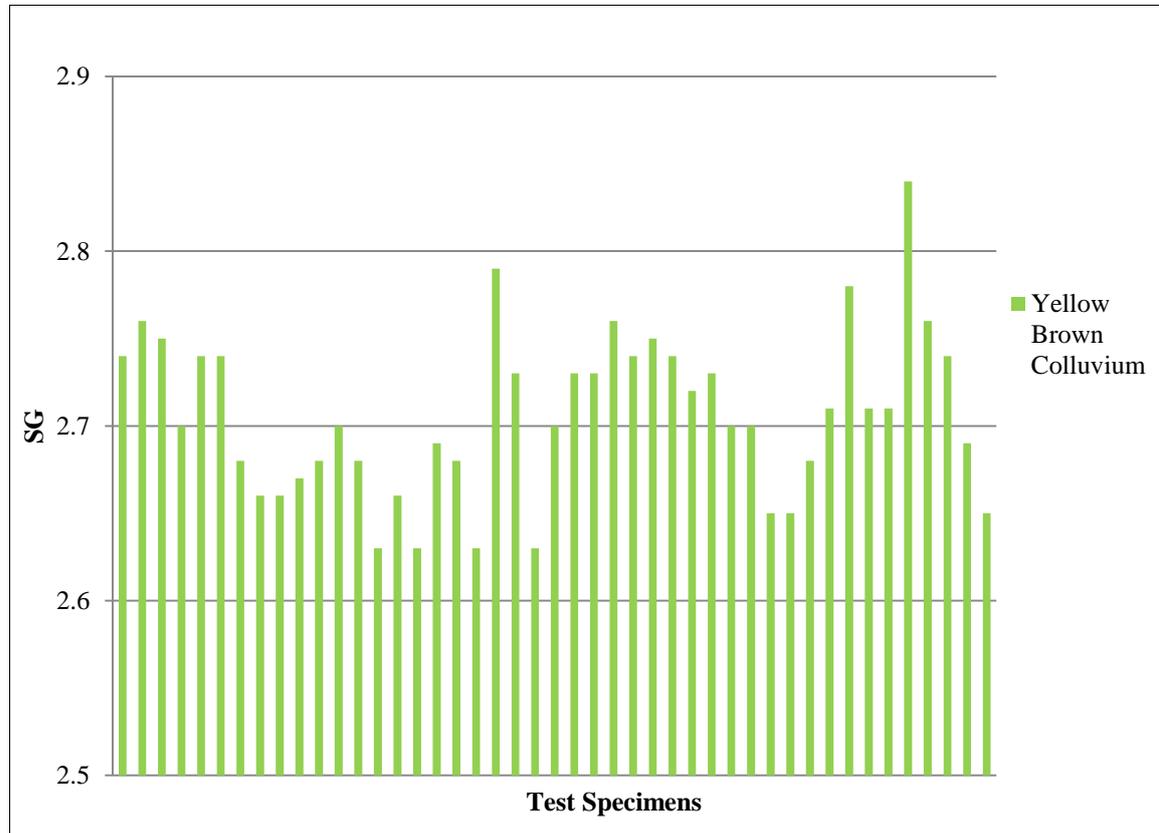


Figure 6-24 Calculated particle densities of yellow brown colluvium material outside mining area

In an effort to establish the influence of a change in moisture content on the specific gravity of tropical red soils investigated as part of this study, the specific gravities of selected colluvium samples from both inside and outside the mining area were determined at its natural moisture content and after being allowed to air-dry for an extended period of time (Figures 6-25 to 6-27). Air-drying was specified in an effort to simulate or “mimic” the natural desiccation process the *in-situ* material may be subjected to and being the least aggressive form of drying. These tests were completed on both disturbed and undisturbed colluvium samples. Control samples of the same material and accompanied by the same test instructions were submitted to a second commercial geotechnical laboratory in order to confirm the observation and influence of drying on the specific gravity of the respective colluvium horizons.

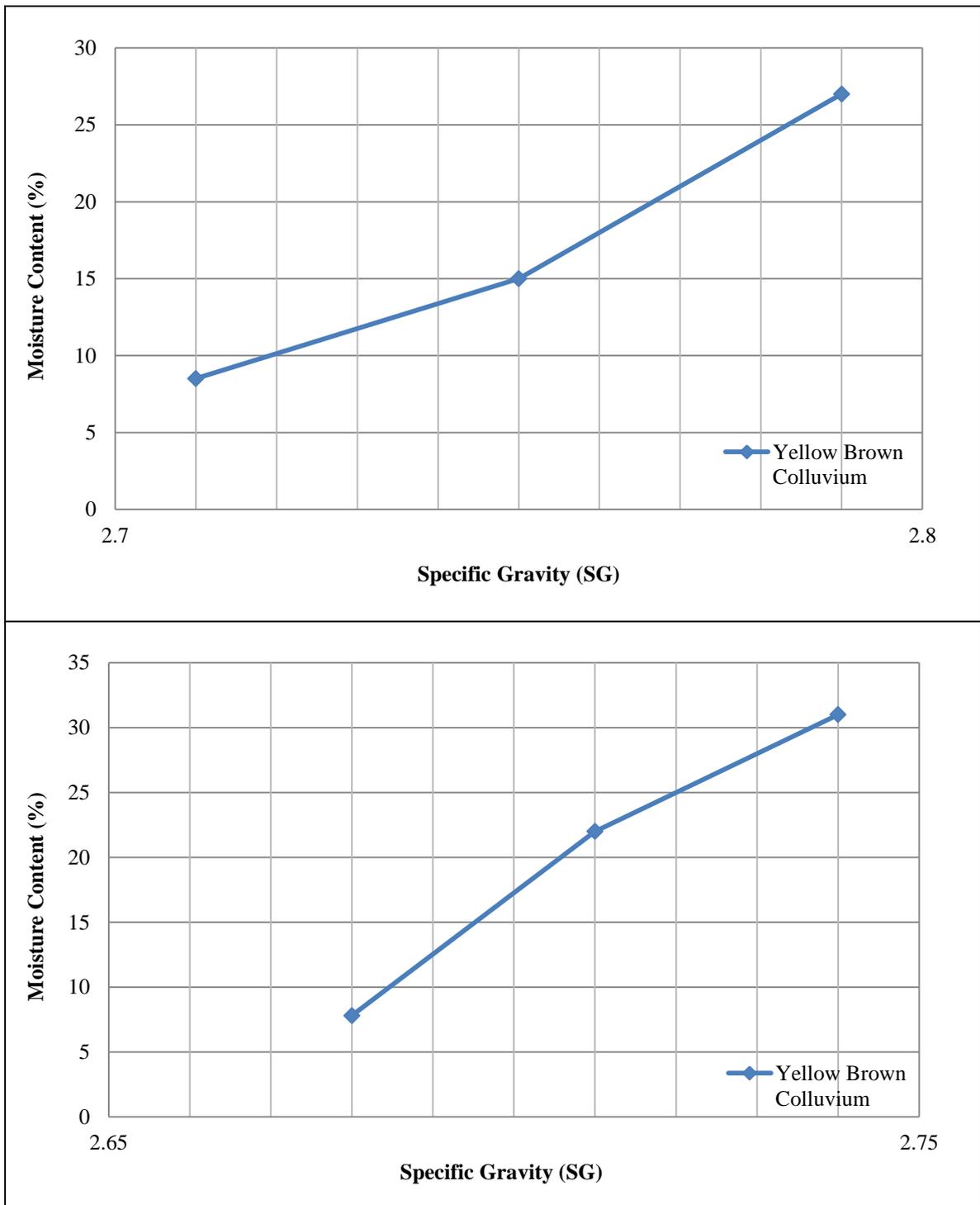


Figure 6-25 Variation in SG of two yellow brown colluvium test specimens from outside the mining area with decreasing moisture content

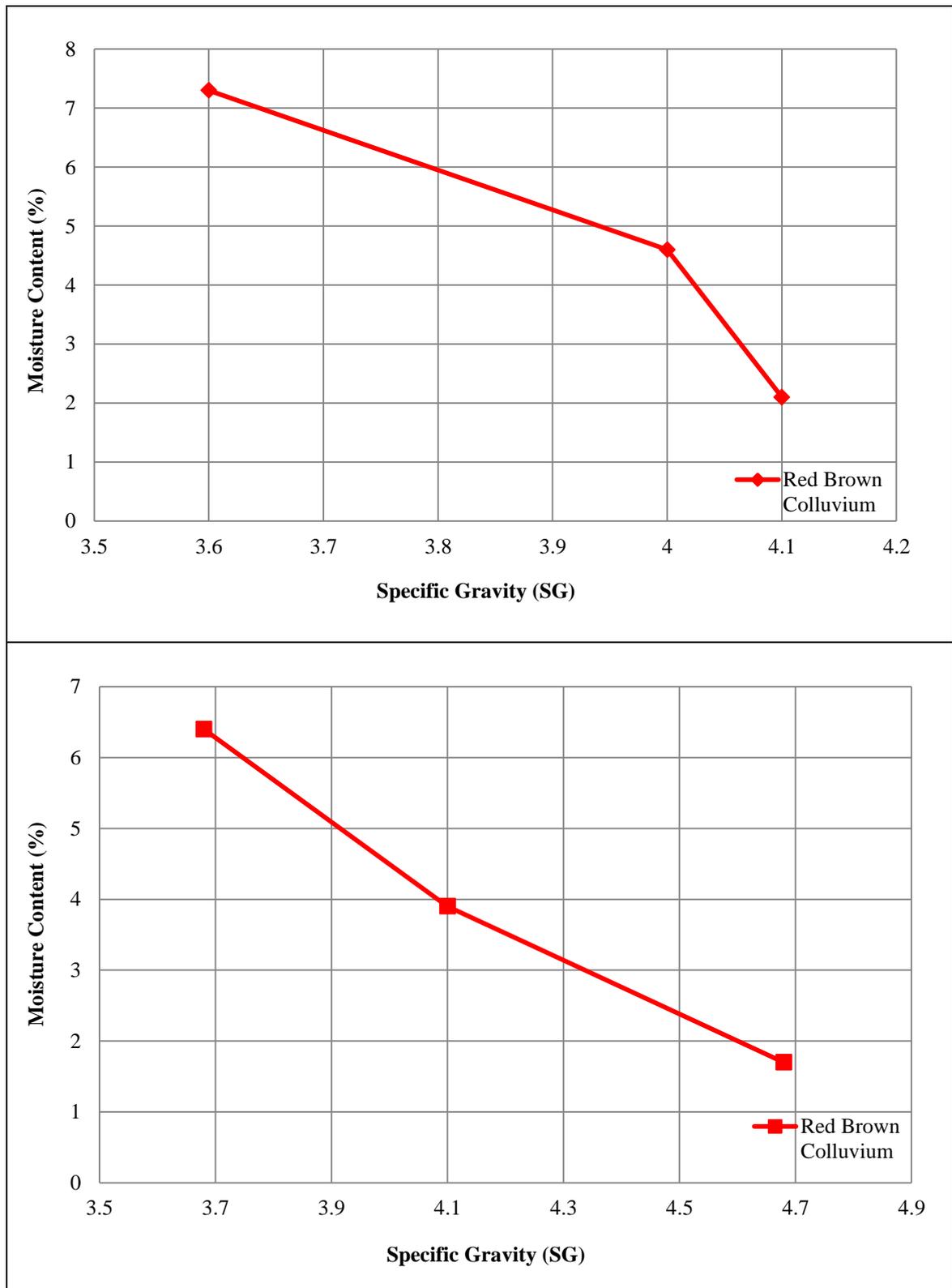


Figure 6-26 Variation in SG of 2 red brown colluvium from Mt. Lekoumou test specimens with decreasing moisture content

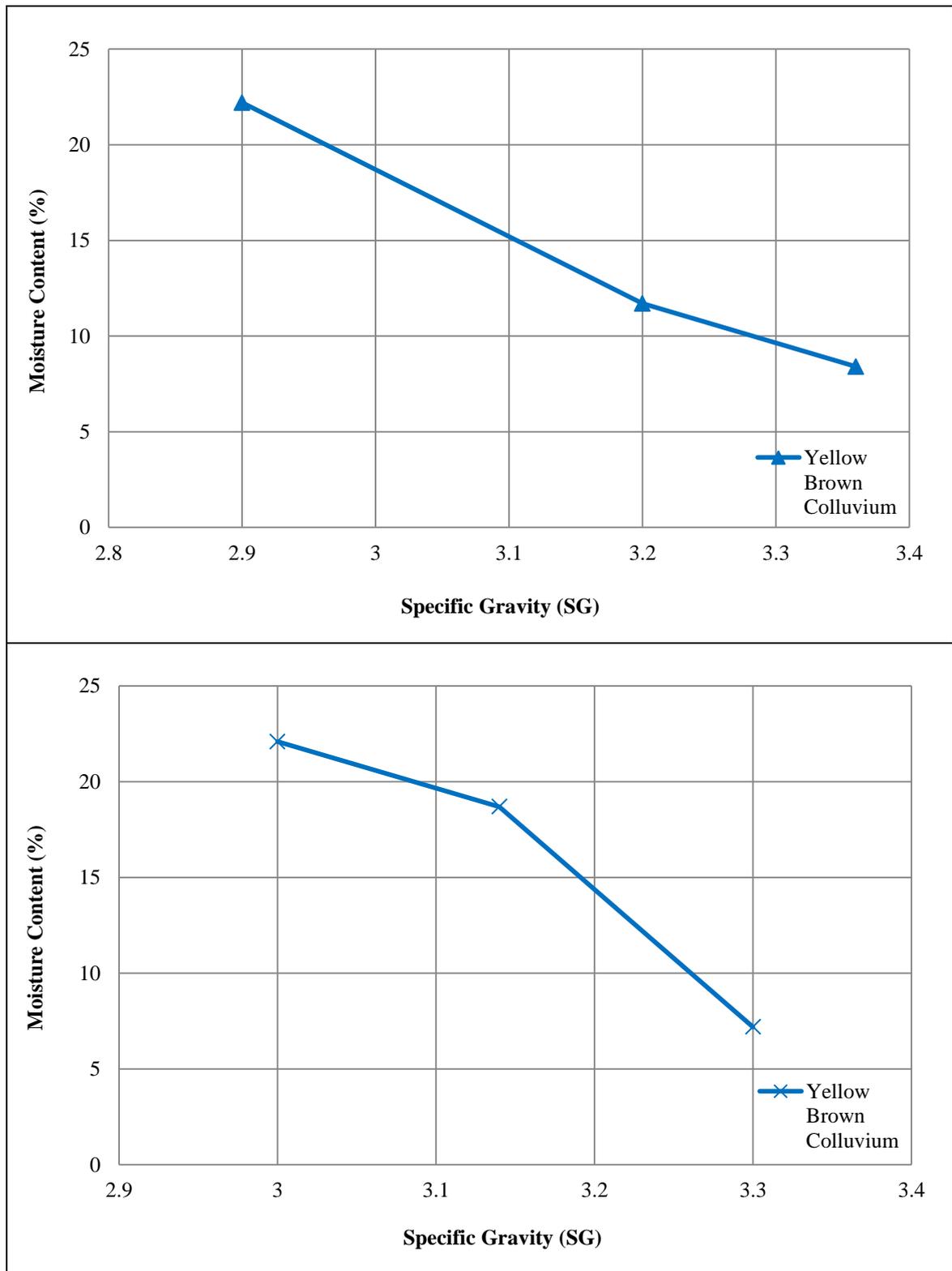


Figure 6-27 Variation in SG of 2 yellow brown colluvium from Mt. Lekoumou test specimens with decreasing moisture content

Based on the results graphically summarised here it is clear that minor variation in the specific gravity of the colluvium specimens outside the mining area does occur, even when drying of the specimens only takes place through air-drying at atmospheric conditions. SG values decrease by a maximum of 0.08 from SG = 2.79 to SG = 2.71, and is believed to possibly represent the loss of

interlayer water from any hydrated minerals that may be present. It should be noted that the final recorded change in SG achieved through air-drying is only marginally less than the changes recorded in the literature and at the particular moisture content, but achieved using oven drying methods.

Contrary to the behaviour observed for the colluvium materials outside the mining area and previous findings discussed in the literature, a significant increase in the SG values of both the disturbed and undisturbed yellow and red brown colluvium specimens from the slopes of the mining area occur as the moisture content of the material is decreased.

The magnitude of the increase in SG does not depend on either the initial moisture content or the overall percentage change in moisture content of the material, with the SG values of the red brown colluvium increasing by similar or even greater magnitude than those of the yellow brown colluvium specimens for which the overall change in moisture content was significantly greater.

This increase in particle density was recorded for the test specimens submitted at both commercial laboratories and no evidence of operator or test procedure error leading to the results reported here (Table 6-4).

Table 6-4 Variation in SG with drying observed for colluvium specimens from mining area

Sample	SG at NMC	SG after air drying (1 day)	SG after air drying (3 days)
1	3.6	4	4.1
2	3.68	4.1	4.68
3	2.9	3.2	3.36
4	3	3.14	3.3

6.5 Discussion

6.5.1 Particle Size Distribution

From the available particle size distribution results, the gradings of the red brown and yellow brown colluvium horizons throughout the case study area were found to be very similar. No pre-drying of the samples occurred prior to testing due to their expected sensitivity to drying based on the visual observations of the material behaviour on site and all particle size distributions were determined from the natural or “as-received” moisture content using the wet-sieving method.

When considering the variation in the particle size distributions of the colluvium materials with respect to their position in the slope and subsequent degree of weathering, it is evident that those horizons near the bottom of the slope have higher percentages of clay than the colluvium horizons found at higher elevations. The colluvium typically classifies as silty sand to sandy clay at elevations between 720 to 810 m.a.m.s.l. and as sandy clay to clay at elevations 650 to 720 m.a.m.s.l. (matrix classification).

It is evident from the grain size distribution curves that two distinct “clusters” of results exist based on the clay content for the colluvium outside the mining area, with one group of samples averaging contents of 50% clay and the other 16% clay.

From the findings of this study, the different dispersing agents used during hydrometer analysis appear to have a definitive effect on the resultant grain size distribution results. A distinct and sudden decrease of between 10 and 20% in the particle distribution results occurs between the transition from sieve to hydrometer analysis (i.e. in the coarse silt fraction) when using the Sodium Hexametaphosphate and Sodium Carbonate dispersing agent, after which the curve either completely flattens out (average 50% clay) or for the second group of results gradually continues downward to the final fines fraction (average 16% clay). Conversely, grading results obtained using the Sodium Pyrophosphate Decahydrate mixture continues from the final sieve aperture without any distinct drop-off in results. The variation observed in the overall percentage fines for the mining area colluvium horizons is largely considered to be a function of the sample position in the landscape, with those horizons near the bottom of the slope generally having much higher percentages of clay than the colluvium horizons found at higher elevations and not the effect of the dispersing agent.

Incomplete dispersion is therefore believed to have resulted in significantly lower clay contents and the apparent concentration of particles in the coarse silt-size range of the second group of colluvium samples from outside the mining area. Consequently, the matrix classification of these samples classifies as “sandy silt” rather than the “sandy clay” results observed for the properly dispersed material from the mining area. Proper dispersion was however also achieved for a number of samples from the same area using the same dispersing agent, resulting in higher final clay contents similar to the yellow brown colluvium found in the lower slopes of the mining area.

When considering the available chemical and XRD/XRF results and the variation in the mineralogical composition of the colluvium from the different parts of the study area, it is clear the Sodium Pyrophosphate Decahydrate dispersing agent mixture is suitable for the goethite- and kaolinite-rich colluvium material from the Mt. Lekoumou mining area. When using the Sodium Hexametaphosphate and Sodium Carbonate dispersing agent mixture to disperse the kaolinite- and quartz-rich colluvium from outside the mining area, incomplete dispersion was found to occur for a number of test specimens.

6.5.2 Atterberg Limits

The calculated Atterberg Limits of the different colluvium horizons were evaluated and compared to those relationships established for soils from temperate climatic zones. The calculated results were plotted on the Casagrande Chart as a function of the respective sample’s plasticity index and liquid limit.

The calculated plasticity indices and liquid limits of the colluvium from the mining area were found to be representative of the predominantly kaolinitic mineralogical composition derived from the XRD and XRF analysis. The colluvium material is characterized by very high liquid limit values, but comparatively low to moderate plasticity index values for such high liquid values. Consequently, when plotted on the conventional Casagrande Chart the colluvium material tends to always plot below the A-line. When assuming the relationship established for those soils from temperate climates holds true and can be applied to tropical soils, at liquid limit values of 50% or greater the colluvium can be classified as inorganic silts and organic clays of high compressibility (MH and MV) and at liquid limits of less than 50% as inorganic silts of low to medium

compressibility (ML and MI). A general increase in plasticity index and liquid limit values is apparent with a change in elevation in the mining area, where there is a general increase in the plasticity index and liquid limit of the colluvium horizons with decrease in mean elevation. The concentration of kaolinite is significantly higher in the yellow brown colluvium found from the middle to lower slopes of the topography due to the greater degree of weathering the material has been subjected to, compared to the predominantly hematite and goethite found in the younger red brown colluvium soils found in the upper slopes. The results therefore provides an indication of the influence and variation of soil genesis at different positions in the landscape, and how far the soils have progressed along the tropical weathering path, on the engineering properties of these soils.

It is evident that the range of plasticity index values, liquid limits and the ratios of these values to one another varies significantly for the colluvium material outside the mining area, classifying from MI to significantly higher and closer to the A-line in the categories of MH-MV.

The liquid limit results determined for a number of samples using both the falling cone and Casagrande cup method (Figure 6-15) indicate that the liquid limits established through the falling cone method consistently exceed the liquid limit value established for the same material using the Casagrande cup apparatus. The difference between the values tends to be further exaggerated with an increase in the overall liquid limit, on average exceeding the Casagrande liquid limit by 14% for liquid limits less than 30% and by 21% for liquid limits greater than 30%.

From the available results the linear shrinkage of the colluvium varies between 2% and 18%. The observed shrinkage is additionally found to be irreversible due to the observed changes in material characteristics that take place on change in moisture content. Additionally, there exists a linear relationship between the linear shrinkage and the calculated plasticity index for the colluvium samples, but the ratio of the plasticity index to linear shrinkage for colluvium from both inside and outside the mining area is higher than that established in the available literature for tropical red soils. The established plasticity index of the colluvium is on average 10 to 20% higher than the plasticity index established from the relationship in the literature for a particular linear shrinkage value. This implies that even though the investigated colluvium material may have high plasticity indices, the amount of linear shrinkage would be significantly lower than would be typically expected for a material of that plasticity index based on the existing relationship from the literature.

As the plasticity index represents the range between the liquid limit (upper boundary condition) and plastic limit (lower boundary condition) and the calculated liquid limits (discussed below) largely agree with the relationship established for similar tropical soils in the literature, the high plasticity index is considered to be the product of a significantly lower than expected plastic limit and that the material will therefore behave in a plastic fashion at a much lower moisture content.

When plotting the calculated linear shrinkage values as a function of the liquid limit of the material, it is apparent that a number of the colluvium samples have higher liquid limit to linear shrinkage ratios than the relationship established in the literature. This ratio however decreases with increasing values of linear shrinkage, with more of the liquid limits for a high linear

shrinkage value resembling the liquid limits that would be expected from the relationship from the existing literature.

In an effort to assess the activity of the colluvium soils through simple index test results, the plasticity indices of the different colluvium samples were plotted against the clay content of the material according to Skempton (1953) and Van der Merwe (1964). It was however found that the activity classification through Skempton's method tends to rather underestimate the potential activities of those samples of high clay contents and high plasticity indices. For such samples the actual linear shrinkage values of the particular samples reflect material for which significant shrinkage occurs on change in moisture content, but which classify as normal clays according to the aforementioned classification system. The evaluation through the Van der Merwe method, and contrary to Skempton's activity rating, is again believed to overestimate the potential activities of those samples of low to moderate clay contents and their equivalent gross plasticity indices, where the actual linear shrinkage values of the particular samples do not reflect material of equal activity.

By using two different methods that each use different versions of the same index properties to evaluate the potential activities, two significantly different conclusions were made of the potential activity of the colluvium found outside the mining area, neither of which agree with the established linear shrinkage values. Both methods however correctly predict the low potential activity of the kaolinite-/goethite-rich material found on the slopes of Mt. Lekoumou.

The potential activities predicted using either of the aforementioned methods are directly influenced by the level of dispersion achieved during hydrometer analysis. Those samples of colluvium not properly dispersed during hydrometer analysis will result in the erroneous reporting of a lower clay content and, when plotted as a function of the corresponding plasticity index (weighted or total) of the sample, result in a higher potential activity classification than actually the case. Cornell University (1952) summarised the following typical values (for temperate soils) as shown in Table 6-5.

Table 6-5 Average limit states of common clay minerals (Cornell University, 1951)

Clay Mineral	Exchangeable Ions	Liquid Limit	Plastic Limit	Plasticity Index	Shrinkage Limit
Montmorillonite	Na	710	54	650	9.9
	K	660	98	562	9.3
	Ca	510	81	429	10.5
	Mg	410	60	350	14.7
	Fe	290	75	215	10.3
Illite	Na	120	53	67	15.4
	K	120	60	60	17.5
	Ca	100	45	55	16.8
	Mg	95	46	55	14.7
	Fe	110	49	49	15.3
Kaolinite	Na	53	32	21	26.8
	K	49	29	20	-
	Ca	38	27	11	24.5
	Mg	54	31	23	28.7
	Fe	59	37	22	29.2

Using this data, Savage (2007) proposed that the ratios of the liquid limit (LL), plastic limit (PL) and plasticity index (PI) can well be used as a potential clay type indicator (Table 6-6).

Table 6-6 Average limit state ratios of common clay minerals (adapted from Savage, 2007)

Clay Mineral	Relationship	Average
Montmorillonite	LL/PI	1.19
	PI/PL	4.96
	LL/PL	5.9
Illite	LL/PI	1.91
	PI/PL	1.15
	LL/PL	2.16
Kaolinite	LL/PI	2.68
	PI/PL	0.61
	LL/PL	1.59

As the ratio LL/PI is the product of LL/PI and PI/PL, the ratio LL/PL is therefore assumed as a clay type indicator. This relationship is represented as $LL/PL = R$ and is referred to as the plasticity ratio. In addition, Savage (2007) further identified a mathematical derivation of swell potential through a factor K , relating gross PI with the clay content of the sample through:

$$(P_{002} - 0.73K)(Pg - 0.16P_{002}K^{0.4}) - K = 0 \quad (\text{Eq. 6-4})$$

Where:

P_{002} = Clay Content (%)

P_g = Gross Plasticity Index

K = Constant

The swell potential is then defined by K as (Savage, 2007):

- $K \leq 16$ – Low swell potential (Inactive Clays);
- $16 \leq K \leq 27$ – Medium swell potential (Normal Clays);
- $27 \leq K \leq 37$ – High swell potential (Active Clays);
- $K > 37$ – Very high swell potential (Highly Active Clays).

However, it is possible to substitute the clay content parameter using the assumption that $P_{002} = 6.25P_gR^{-2.13}$, where R is the plasticity ratio and thereby evaluating the potential activity using the calculated Atterberg Limits only. Equation 6-4 can then be re-written as (Savage, 2007):

$$P_g(1 - K^{0.4} \times R^{-2.13})(6.25P_gR^{-2.13} - 0.73K) - K = 0 \quad (\text{Eq. 6-5})$$

For graphical evaluation of the swell potential, this mathematical relationship can be represented as follows (Figure 6-28):

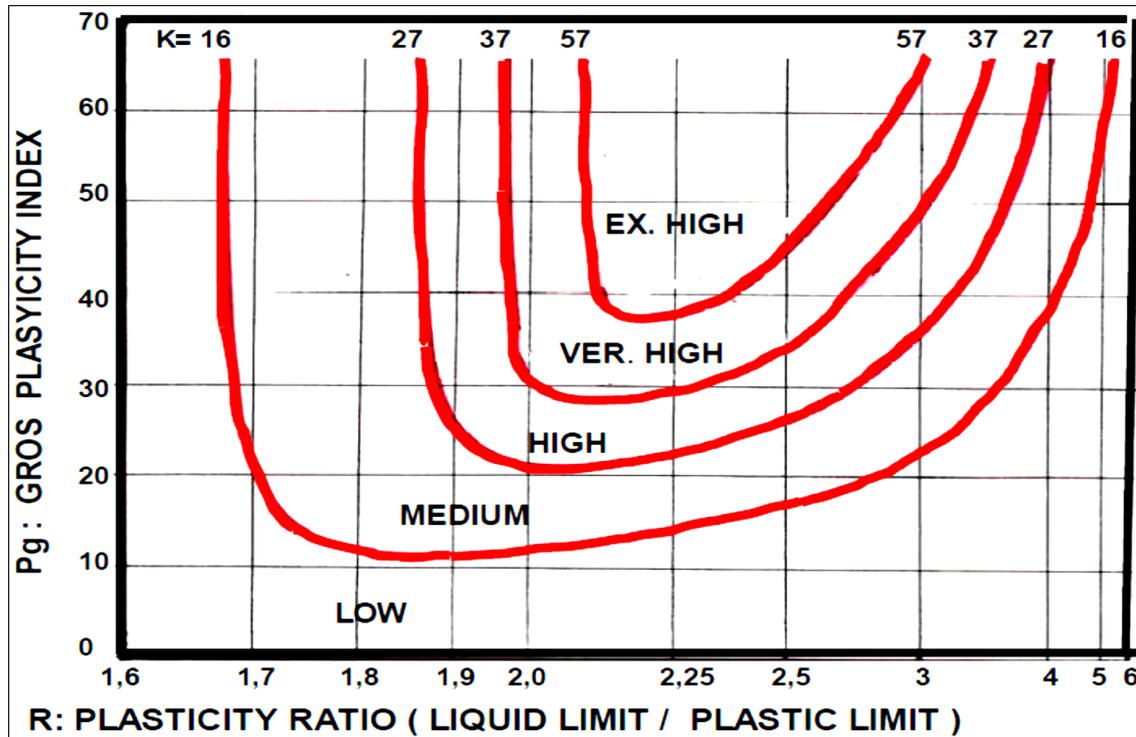


Figure 6-28 Potential activity based on Atterberg Limits alone (adapted from Savage, 2007)

Following this approach, the potential activity of the colluvium samples can be plotted as a function of the plasticity ratio and plasticity index, excluding the influence of the overall clay content (Figure 6-29).

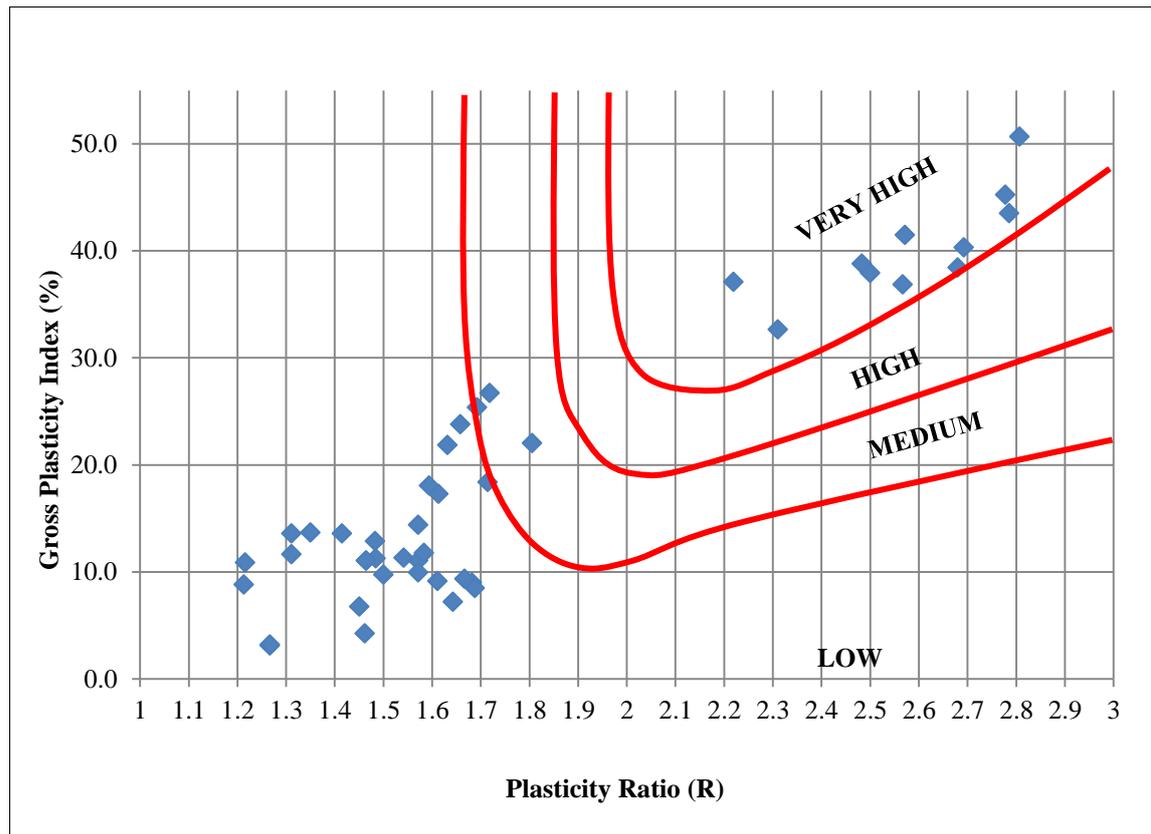


Figure 6-29 Potential activity of all colluvium samples based on plasticity ratio and PI

The potential activity determined using each of the different discussed methods compared to the actual linear shrinkage values are summarised below (Table 6-7). This represents how many times the potential activity derived for the particular sample is the same class as the actual activity rating obtained from the linear shrinkage.

Table 6-7 Comparison between measured linear shrinkage and predicted activity

Linear Shrinkage (%)	Predicted Potential Activity (Plasticity Ratio)	Predicted Potential Activity (Skempton)	Predicted Potential Activity (Vd Merwe)
5 – 10% (Low)	100% Low	59% Low, 41% Medium	94% Low, 3% Medium, 3% High
10 – 15% (Medium)	66% Medium, 34% High	100% Medium	50% High, 50% Low
>15% (High)	100% High	0% High, 100% Medium	100% High
Correlation	96%	51%	82%

When evaluating the potential activities determined using the plasticity ratio and weighted plasticity index, a much improved correlation between the potential activity and the actual linear shrinkage values was observed for all the colluvium samples, in particular the samples from outside the mining area.

It is therefore evident that a different approach needs to be followed to evaluate the potential activities of tropical soils with greater accuracy. At this point it is also very important to keep in mind the high ratio between the recorded plasticity index and the actual linear shrinkage that was observed for the colluvium samples. The established plasticity index of the colluvium is on average 10 to 20% higher than the plasticity index established from the relationship in the literature for a particular linear shrinkage value due to lower than average plastic limit values. This implies that even though the investigated colluvium material may have a high plasticity index and high clay content, the amount of actual shrinkage would be significantly lower than would be typically expected for a material of that particular plasticity index based on the existing relationship. Additionally, the colluvium samples tend to be highly sensitive to test procedure (drying and mixing) and the dispersant agent used during hydrometer analysis.

It is proposed that, on the basis of the results of this study, in order to evaluate the potential activity of the tropical soils with a greater degree of certainty using indicator data alone, an alternative approach other than relating the plasticity index to overall clay content is followed. By making allowance for the consideration of the variation in the plastic limit of the material due to variation in sample mineralogy and excluding the influence of the incorrect determination of the clay content, a much improved correlation between the potential activity and the actual linear shrinkage values was observed.

Three samples of yellow brown colluvium material from the mining area were each subdivided into three different specimens of which the Atterberg Limits were established at their natural moisture content, after one day of air-drying and three days of air-drying. The plastic limits of the material were found to remain fairly constant regardless of the exposure of the material to drying. The liquid limits and plasticity indices of the material were however found to decrease by up to 10% from the natural moisture content sample and the sample allowed to air-dry for three days. It is evident from the results of this study that the Atterberg Limits are influenced by the degree of drying the test specimen has been exposed to.

6.5.3 Natural Moisture Content

The natural moisture contents of the colluvium materials over the study area were determined in an effort to evaluate the behaviour of soils during construction and as part of investigating the effect of soil suction pressures on the *in-situ* behaviour of the soils. Contrary to the air-drying method specified to mimic the effect of *in-situ* drying on the particle density and grading of selected sub-samples, the natural moisture contents of the material were determined using the oven drying method at a below normal drying temperature of 65°C.

The moisture contents of the red and yellow brown colluvium horizons in the mining area may vary significantly, but a clear trend exists of increasing average moisture content with decreasing elevation towards the lower, “flatter” slopes where drainage conditions are expected to be poorer than along the steeper upper slopes of the mining area. This increase in moisture and the change in drainage conditions is the main controlling factor behind the change in soil colour and mineralogy with change in elevation.

During air-drying of selected red and yellow brown colluvium samples a variation in sensitivity of the different materials to drying became apparent. The moisture contents of samples of both colluvium horizons were determined at their natural moisture contents, after being allowed to air dry for one day and the final measurement after three days of drying. All samples were allowed to dry at the same time and therefore at the same atmospheric conditions. The moisture contents of the kaolinite-rich yellow brown colluvium material decreased in the order of 10% to 15% from the natural moisture content established prior to drying. The goethite- and quartz-rich red brown kaolinite material on the contrary only experienced a loss in moisture of 4% to 5% during the same period and conditions.

From the results summarised in Table 6-3 for those samples subjected to different oven-drying temperatures, it is evident that the yellow brown and dark red brown colluvium in the study area indeed appear sensitive to the drying temperature, with those samples dried at higher oven drying temperatures reporting natural moisture contents of between 3.5% and 7.6% higher than the same material dried at a lower temperature of 60°C.

The moisture content of a particular material is largely dependent on adsorption and capillary forces prevailing in the material. From the available literature the water adsorption force of a particular soil is widely accepted and expressed as a function of the clay content of the material, with increasing clay content used to emphasize the increase in magnitude of these forces. However, as discussed the red and yellow brown colluvium materials have largely similar particle size distributions and can the variation in sensitivity to drying not solely be attributed to the elevated content of a particular grain size. Instead, it is rather proposed that the observed difference in sensitivity to drying be considered a function of the variation in mineralogical composition of the material and the subsequent difference in cation exchange capacity (CEC) observed for the different horizons. The CEC is considered a function of the net negative charge of the colloidal fraction of a soil and is therefore expected to correlate positively with the attraction forces between particle surfaces and water (Nagahori and Haccho, 1981). The red brown colluvium specimens typically have CEC values exceeding that of the yellow brown colluvium by a minimum order of magnitude of 10^2 cmolM⁺/kg and is therefore believed to have significantly better water-holding capacities during non-aggressive drying procedures (such as air-drying) than the kaolinite-rich yellow brown colluvium, irrespective of the total clay content of the specimen.

6.5.4 Particle Density

Specific gravity or particle density gives an indication of the intrinsic density of the solid phase of the soil sample, and is independent of moisture content and voids. The particle density or specific gravity of the colluvium samples from the study area was completed according to the procedure set out in BS 1377-2 (1990), but from the natural or “as-received” moisture content and with no pre-drying of the material allowed apart from on those samples selected to evaluate the effect of drying on particle density.

From these specific gravity results it is clear the dark red brown colluvium is expected to have a higher average particle density than that of the yellow brown colluvium material as found in the middle and lower slopes of the mining area and throughout the study area. This directly correlates

with the mineralogy of the material, as was established during XRD and XRF analysis. Weathering of the iron-bearing minerals has not advanced as far in the red brown colluvium as in the yellow brown colluvium, with the hematite content varying between 20% and 40% and goethite between 20% and 50%, resulting in an average specific gravity value of 3.45 for the red brown material. The yellow brown colluvium has a lower average specific gravity value of 2.95. This material is characterized by significantly lower hematite contents (<6%), but high kaolinite and goethite contents of 30% to 50% due to the greater degree of weathering.

The average specific gravity of the colluvium found in the lower-lying areas outside the mining area equals 2.75 and represents the lowest of all the materials in the study area. This is be ascribed to the increased distance from the heavy mineral-rich mining area and the advanced stage of weathering of these horizons.

Mitchell and Sitar (1982) describes the average specific gravities of hematite, goethite and kaolinite as varying between 4.9-5.3, 3.3-3.6 and 2.2-2.6 respectively. Comparing the specific gravities of the materials summarised here and the occurrence of the various minerals in the calculated XRD results, it is clear that the specific gravities of the materials correlate well with the results established for the various minerals in the literature. Due to the high specific gravity of iron bearing minerals and low specific gravity of kaolinite and other clay minerals, the specific gravity may be an useful indicator of the stage of pedogenesis, the locality within the study area and subsequent expected engineering characteristics material.

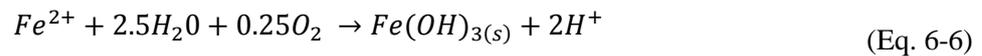
During their investigation of the influence of oven drying on the material properties of tropical soils, Northmore *et al.* (1992) found that despite the fact that oven drying of tropical red soils appears to have little influence on the inter-layer moisture content of the soils, a clear trend of higher specific gravity for those samples at natural moisture content prevails. From their study it is concluded that a decrease in SG with change in moisture is the result of the loss of inter-layer moisture from the hydrated minerals on oven drying, ultimately leading to a reduction in the relative density of the solid matter. Additionally, Newill (1961) completed specific gravity tests on two red clay specimens from Kenya at a series of moisture contents ranging from natural moisture content to a complete oven-dried state. One specimen, rich in hydrated halloysite, showed a significant decrease in SG with decrease in moisture content. Newill (1961) also suggested this marked decrease in SG to be the result of the loss of interlayer water from the hydrated halloysite minerals, subsequently transforming to more stable metahalloysite.

Based on the results from this investigation it is clear that minor variation in the specific gravity of the colluvium specimens outside the mining area does occur, even when drying of the specimens only take place through air-drying at atmospheric conditions. SG values decrease by a maximum of 0.08 from SG = 2.79 to SG = 2.71, and is believed to represent the loss of interlayer water from any hydrated minerals that may be present. It should be noted that the final recorded change in SG achieved through air-drying is only marginally less than the changes recorded in the literature and at the particular moisture content, but in those instances achieved using oven drying methods.

Contrary to the behaviour observed for the colluvium materials outside the mining area and previous findings discussed in the literature, a significant increase in the SG values of both the

disturbed and undisturbed yellow and red brown colluvium specimens from the slopes of the mining area occur as the moisture content of the material is decreased. SG values were observed to increase by between 0.3 and 1.0 after 3 days of continuous air-drying.

It is proposed that this increase of the SG values on drying represents the precipitation of sesquioxides (predominantly iron) from solution within the soil voids or on the crystal planes exposed on the face of the constituent clay mineral crystals during drying of the soil. From the chemical, XRD and XRF analysis (Section 5) completed on de-ionised water extracts and selected samples of the colluvium material, it is evident that at any point within the Mt. Lekoumou mining area any moisture present in the soil profile will contain very high concentrations of Fe^{2+} in solution under the acidic and largely anaerobic soil conditions. This holds particularly true for the red brown colluvium horizons, which were found to be weathered and leached to a lesser extent than the yellow brown colluvium horizons. The rapid change in soil moisture conditions upon drying of the sample will lead to a change the aeration state of the soil material. In the presence of oxygen, aqueous ferrous iron is converted to ferric iron (precipitate) through (Bigham *et al.*, 1990):



The increase observed in SG values on drying appears to be irreversible and not influenced by the subsequent rewetting of the sample during soaking as part of the SG test procedure.

7. Mechanical Properties

Tri-axial (consolidated-undrained)-, shearbox-, free-swell, oedometer consolidation- and Proctor compaction tests were carried out on the tropical red soil colluvium from the study area. The scheduled test regime was completed in an effort to detail the shear strength, stiffness, compaction characteristics, permeability and consolidation of the soils with respect to their position in the landscape, the degree of chemical weathering and subsequent influence of the weathering on the mechanical properties and how the determined properties relate to the mechanical properties established for tropical and temperate soils in the literature.

7.1 Shear Strength

Consolidated-undrained tri-axial and shear box tests were carried out on a number of remoulded and undisturbed samples retrieved of the different colluvium materials from various elevations both inside and outside the mining area. It was emphasised prior to testing that the remoulding manipulation time is adjusted to achieve total remoulding and destruction of the natural material structure. This was achieved through moulding by hand palette knives on a glass plate, according to the procedure set out in BS 1377-2 (1990). The materials, of which the original structure is considered completely destroyed by this stage, were then moulded and compacted in 5 layers of 20 mm thickness. Disturbed samples were either remoulded to a density representative of the *in-situ* density or to 95% of the maximum dry density of the material.

Three shearbox tests were completed to supplement the available tri-axial test data on destructured and remoulded (natural density) samples from outside the Mt. Lekoumou mining area in an effort to determine the effective residual shear strength parameters of the remoulded specimens. The values of the calculated effective shear strength parameters c' and ϕ' are summarised in Table 7-1 below.

Table 7-1 Summary of effective shear strength parameters

Undisturbed Samples (Mining Area)	Effective Cohesion (c')	Effective Angle of Friction (ϕ')
Sample 1	17 kPa	27.6°
Sample 2	7 kPa	36.7°
Sample 3	0 kPa	38.1°
Sample 4	0 kPa	36°
Sample 5	0 kPa	36°
Sample 6	12.9 kPa	31.8°
Sample 7	1 kPa	37.5°
Remoulded Samples (Mining Area)	Effective Cohesion (c')	Effective Angle of Friction (ϕ')
Sample 1 (<i>In-situ</i> density)	3 kPa	38°
Sample 2 (<i>In-situ</i> density)	0 kPa	36.5°
Sample 3 (<i>In-situ</i> density)	2 kPa	36°
Sample 4 (<i>In-situ</i> density)	0 kPa	38.1°
Sample 5 (<i>In-situ</i> density)	3.8 kPa	37°
Sample 6 (<i>In-situ</i> density)	4.4 kPa	34.5°
Sample 7 (<i>In-situ</i> density)	2.2 kPa	38.4°
Sample 8 (<i>In-situ</i> density)	0 kPa	39°
Sample 9 (<i>In-situ</i> density)	0 kPa	37°
Sample 10a (<i>In-situ</i> density)	0 kPa	37°
Sample 10b (<i>In-situ</i> density)	0 kPa	38.8°
Sample 11a (<i>In-situ</i> density)	3.7 kPa	36.2°
Sample 11b (<i>In-situ</i> density)	0 kPa	36.2°
Sample 12a (<i>In-situ</i> density)	0 kPa	38.1°
Sample 12b (<i>In-situ</i> density)	0 kPa	36.6°
Remoulded Samples (Outside Mining Area)	Effective Cohesion (c')	Effective Angle of Friction (ϕ')
Sample 1 (<i>In-situ</i> density)	31 kPa	27°
Sample 2 (<i>In-situ</i> density)	10 kPa	26°
Shear Box Test on Remoulded Sample (Outside Mining Area)	Effective Cohesion (c')	Effective Angle of Friction (ϕ')
Sample 1 (95% MDD)	13.4 kPa	27.8°
Sample 2 (95% MDD)	8.4 kPa	32.9°
Sample 3 (95% MDD)	0 kPa	30.2°

From these results, values of effective cohesion (c') vary between 0 and 31 kPa (average = 5.3 kPa) and effective angle of internal friction (ϕ') between 26° and 39° (average = 34.5°). The yellow and red brown colluvium from the mining area generally have lower values of effective cohesion and higher effective angles of friction than the yellow brown colluvium material found in the lower-lying areas outside the mining area. Shearbox test specimens remoulded to 95% of the maximum dry density reflect an increase in the effective angle of friction (average = + 2.6°) compared to those samples that were remoulded to natural density. The test results revealed largely similar stress-path and strength behaviour of disturbed and destructured materials. Colluvium test specimens from the mining area remoulded to *in-situ* densities on average however exhibit higher effective friction angles than the undisturbed samples of the same material (average = +2.3°).

Figures 7-1 and 7-2 indicate the T-S peak strength plots for the interpreted peak strengths of the remoulded and undisturbed samples from the consolidated-undrained tri-axial data.

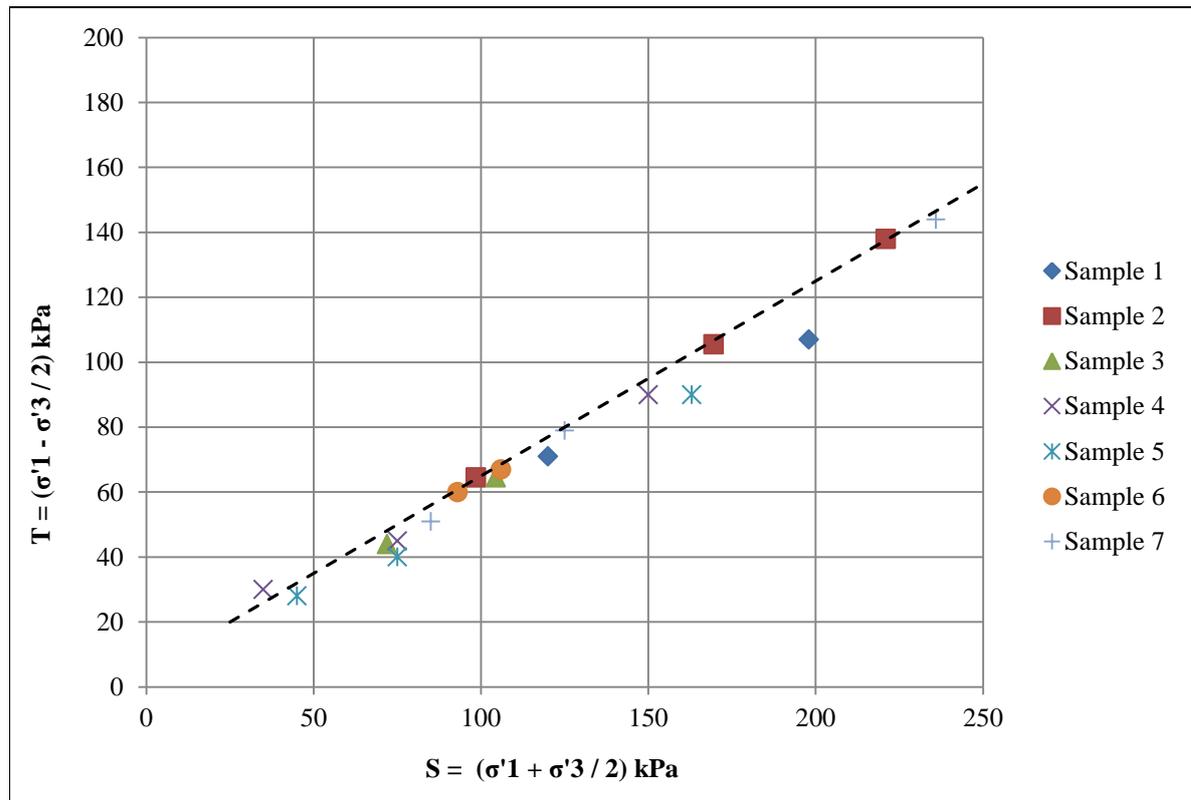


Figure 7-1 t-s' (peak strength) for undisturbed colluvium samples from mining area

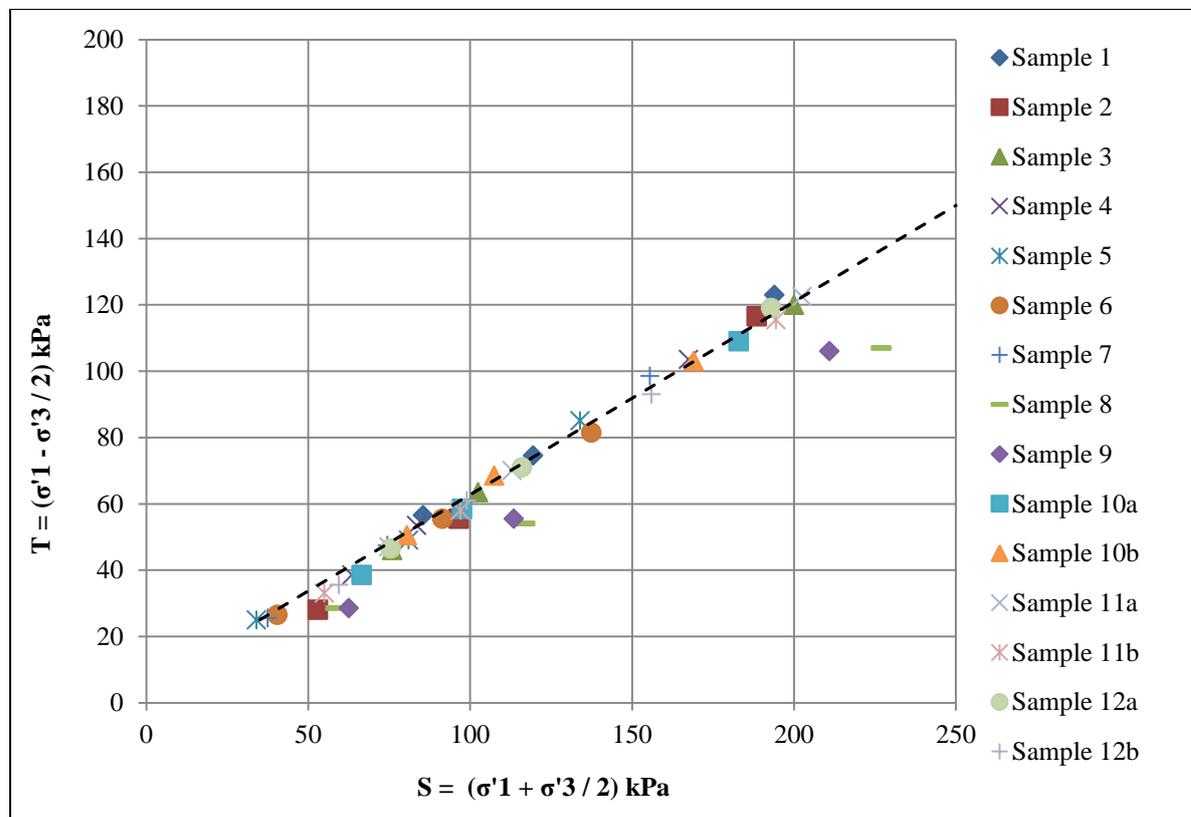


Figure 7-2 T-S (peak strength) for remoulded colluvium samples from mining area

These plots show fairly consistent results for all test specimens. A trend line, representing the “best-fit” critical state line for the data, can be added for both the disturbed and undisturbed specimen results. The angle of the trend line (α) from horizontal may be used to calculate a generalized effective friction angle (ϕ') assuming $\sin \phi' = \tan \alpha$. This can be summarised as:

Table 7-2 Best-fit effective friction angle from undrained stress-path plots

	α	$\tan \alpha$	ϕ'
Best-fit (Undisturbed)	31°	0.6	36.8°
Best-fit (Remoulded)	30.1°	0.58	35.5°

Typical deviator stress and axial strain responses are presented for two representative test specimens in Figure 7-3 and 7-4. Peak deviator stress exists at low axial strain values for the majority of the test specimens, varying between 60 kPa ($\sigma_3' = 100$ kPa) and 240 kPa ($\sigma_3' = 400$ kPa).

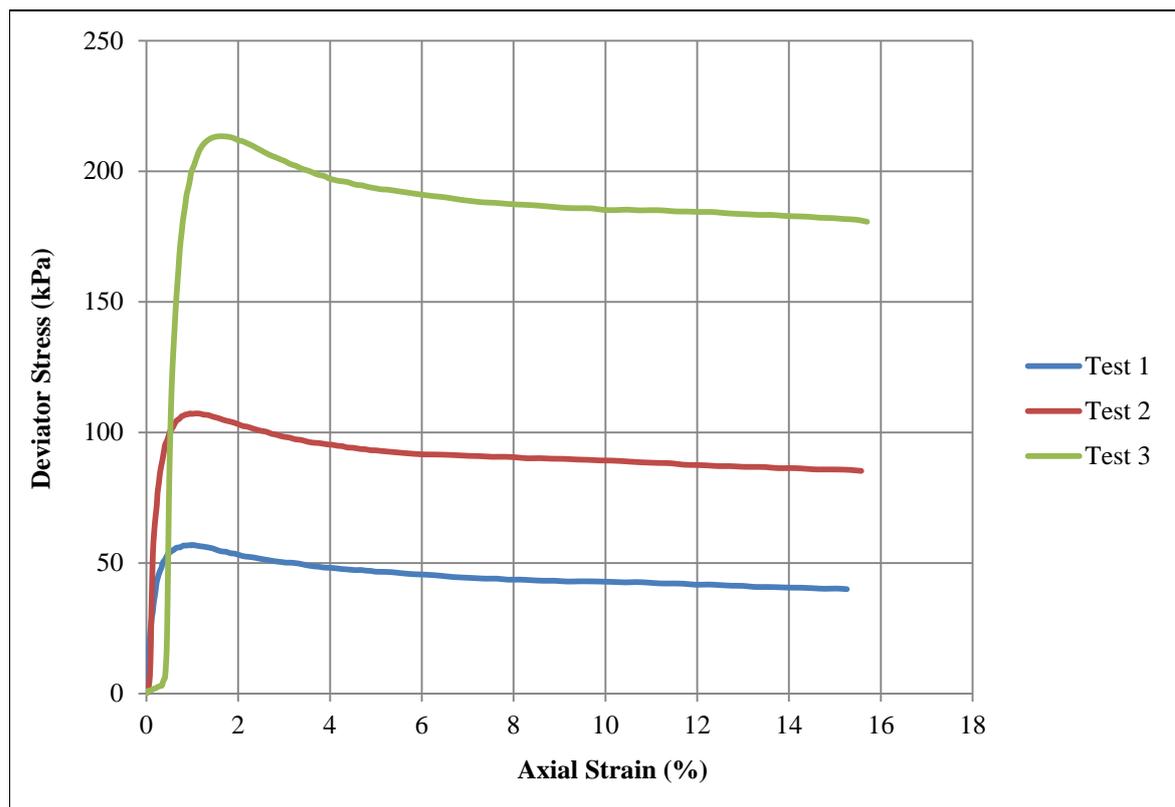


Figure 7-3 Typical deviator stress versus axial strain relationship for remoulded test specimen

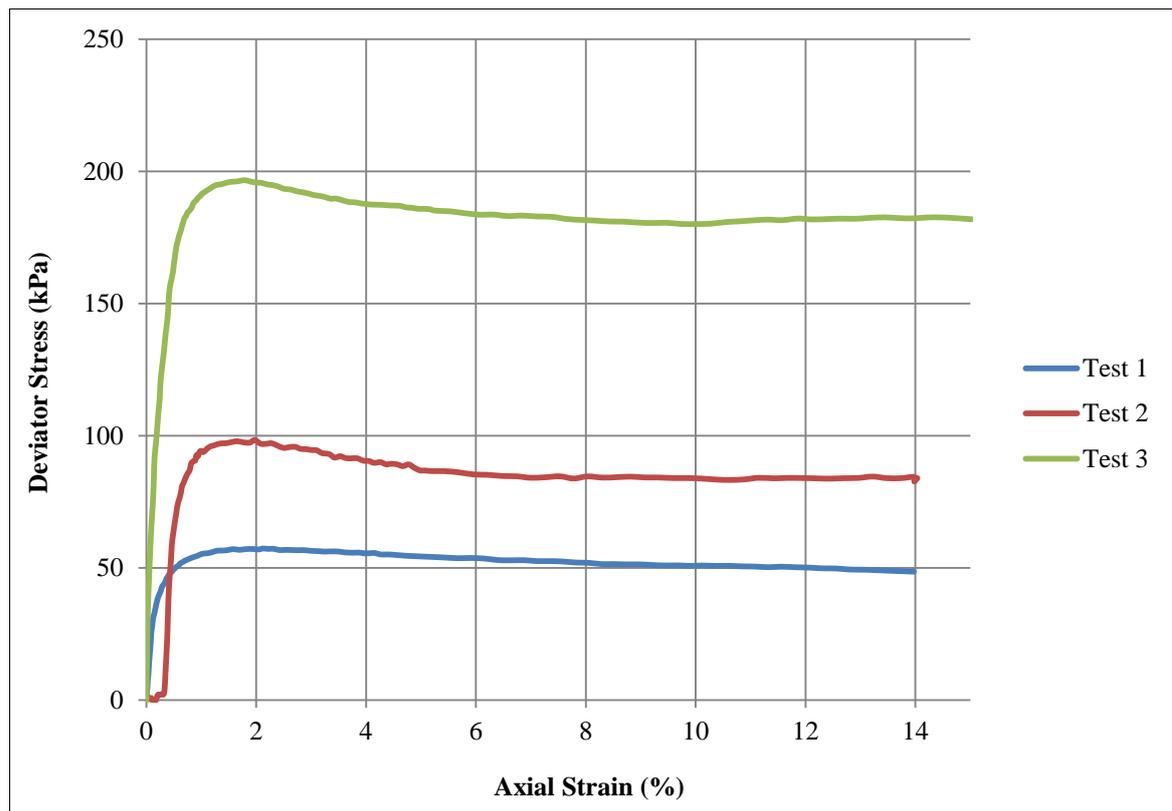


Figure 7-4 Typical deviator stress versus axial strain relationship for undisturbed test specimen

Pore water pressure increases continuously for all test specimens from the start of the test in the form of a smooth curve with increasing axial strain. Peak pore pressure is achieved at between 60 kPa ($\sigma_3' = 100$ kPa) and 340 kPa ($\sigma_3' = 400$ kPa) and just before failure of the test specimen, after which it decreases slightly before continuing at a constant value to the end of the test.

7.2 Compaction

Proctor Compaction tests were carried out for a number of bulk samples of the yellow and red brown colluvium from inside and outside the mining area. As was specified by Northmore *et al.* (1992) during the compaction testing of a number of tropical soils, the following variations from the standard BS 1377 – 4 (1990) test procedure were specified:

- No pre-drying of the samples were permitted;
- Separate sub-samples were used for each individual compaction stage.

However, due to the high natural moisture content and degree of saturation of the material, the colluvium in its un-dried state was found to be wet of optimum moisture content. In an effort to effectively determine the optimum moisture content and maximum dry densities of the material, air-drying of selected separate sub-samples was specified to effectively plot the variation of achieved density with moisture content.

The moisture versus density relationships of the various samples are compared to the 0%, 5% and 10% air voids curve for each of the materials using the average specific gravity established for the particular test specimens.

$$\rho_d = \frac{\rho_w}{\left(\frac{w}{100} + \frac{1}{G_s}\right)} \quad (\text{Eq. 7-1})$$

Where:

ρ_d = Dry density (kg/m³)

ρ_w = Density of water (kg/m³)

G_s = Specific gravity

w = Moisture content (%)

The 0% air voids or 100% saturation curve represents the maximum possible density that the soil can be compacted to at the particular moisture content. This degree of compaction is however widely accepted to be unattainable in practice and reported compaction efforts must lie completely to the left of the 0% air voids curve.

The compaction test results are summarised in the Figures 7-5 and 7-6 below.

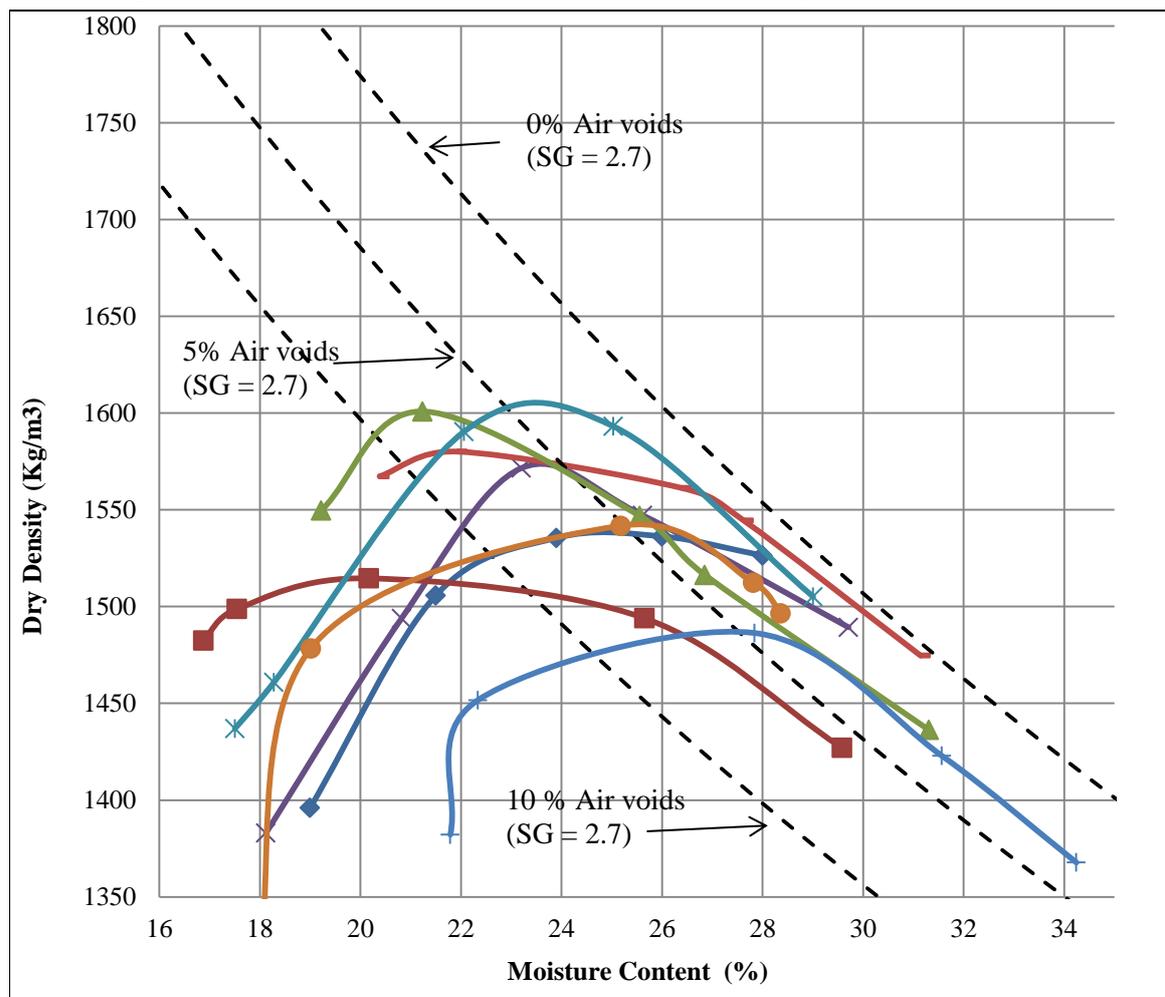


Figure 7-5 Summary of Proctor compaction curves of yellow brown colluvium material outside mining area at SG = 2.7

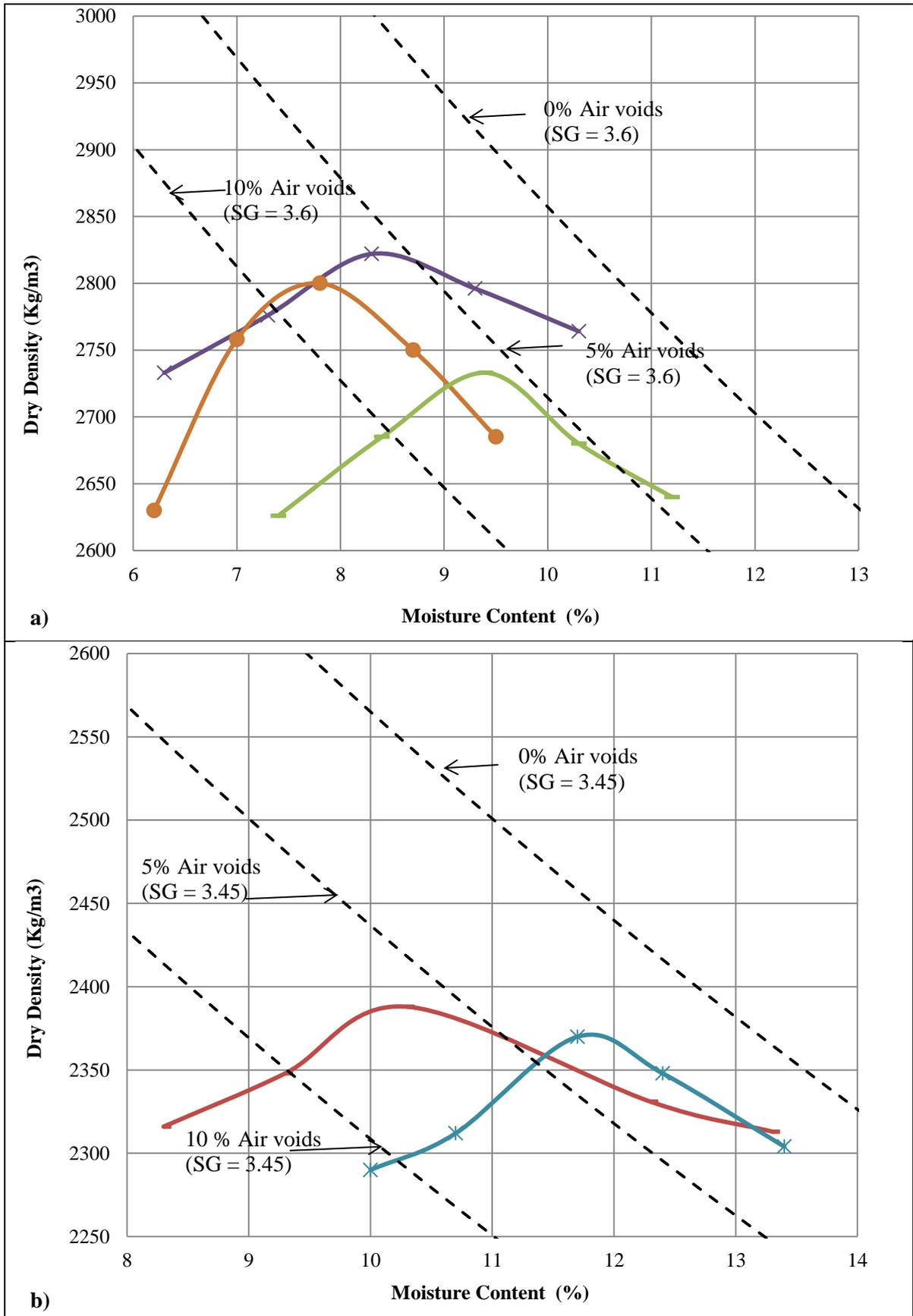


Figure 7-6 Summary of Proctor compaction curves of red brown colluvium material from Mt. Lekoumou mining area at a) SG = 3.6 and b) SG = 3.45

From the results of the compaction tests summarised above, the optimum moisture content (OMC) and maximum dry density (MDD) values of the yellow brown colluvium material found in the low-lying areas away from the Mt. Lekoumou mining area vary between OMC = 20% to OMC = 27% and MDD = 1486 kg/m³ to MDD = 1605 kg/m³.

The MDD values achieved for the red brown colluvium material from the mining area vary between MDD = 2370 kg/m³ to MDD = 2822 kg/m³ at OMC = 7.8% to OMC = 11.7%. Only a single Proctor Compaction test was completed on yellow brown colluvium material from the mining area, with a calculated MDD of 2109 kg/m³ at an OMC of 13.3%. The colluvium materials from the mining area have significantly higher MDD values than those densities achieved for the colluvium away from the mining area and which would be expected from temperate soils of similar plasticity and USCS classification. The OMC at which these densities are achieved is however significantly lower than the OMC values of the colluvium outside the mining area and the natural moisture contents of the material.

The compaction curves of the different samples show that in general, the maximum dry density of the majority of the colluvium specimens is attained between the 10% (90% saturation) and 5% (95% saturation) air voids curves. This is slightly lower levels of saturation than what would be expected for temperate climate soils.

7.3 Consolidation

The consolidation characteristics of the yellow brown colluvium material from the oedometer tests are plotted as a semi-logarithmic function of the consolidation (Δe) of the sample with increase in load.

No to very little collapse settlement (average of 0.2 mm) was observed from the double oedometer test results. This is believed to largely be the result of the high degree of saturation of the material at its natural moisture content, with the majority of settlement occurring on loading of the material prior to the additional water being added.

From the oedometer and consolidation stage of the tri-axial test data it was possible for the following consolidation parameters to be determined:

- Coefficient of Consolidation

$$C_v = \frac{(0.026 \times (H)^2)}{t_{50}} \quad (\text{Eq. 7-2})$$

Where:

C_v = Coefficient of Consolidation (m²/year)

H = Mean height of sample during test stage (mm)

t_{50} = Time to achieve 50% consolidation (min)

- Coefficient of Volume Compressibility

$$M_v = \frac{(\delta e / \delta p) \times 1000}{1 + e_1} \quad (\text{Eq. 7-3})$$

Where:

M_v = Coefficient of Volume Compressibility (m^2/MN)

δe = Incremental change in void ratio

δp = Incremental change in pressure (kN/m^2)

e_1 = Starting void ratio

- Compression Index

$$C_c = - \frac{\Delta e}{\Delta \log P} \quad (\text{Eq. 7-4})$$

Where:

C_c = Compression Index

Δe = Incremental change in void ratio of normal consolidation stage of specimen

$\Delta \log P$ = Incremental change in pressure of the normal consolidation stage of specimen (kN/m^2)

- Recompression Index

$$C_e = - \frac{\Delta e}{\Delta \log P} \quad (\text{Eq. 7-5})$$

Where:

C_e = Recompression Index

Δe = Incremental change in void ratio on unloading of specimen

$\Delta \log P$ = Incremental change in pressure on unloading of specimen (kN/m^2)

Void ratios of the yellow and red brown colluvium from the Mt. Lekoumou mining area and the yellow brown colluvium material from the lower-lying areas outside the mining area were found to vary between 0.87 and 1.17, with an average void ratio of $e = 1$ (porosity = 0.50).

Based on the primary consolidation curves obtained from oedometer testing and using the \sqrt{t} method, the C_v values for the undisturbed colluvium samples in the study area vary between 5 and 10 mm^2/min , but in some instances as low as 1 mm^2/min to as high as 20 mm^2/min at the various applied loads (Table 7-3). This relates to coefficient of consolidation values of between 1 and 15 m^2/year . Consolidation rates of the undisturbed specimens were found to increase for the applied pressure increments up to 100 – 200 kPa, from where the relationship remains linear and the ratio

of the rate of consolidation and increasing pressure remains constant up to 800 kPa due to the densification under continuous loading (Figure 7-7).

Table 7-3 Typical range of C_v for undisturbed and remoulded samples from oedometer test results

Sample	Pressure Range (kPa)	Average C_v (mm ² /min)
Sample 13 (Undisturbed)	12.5-25	4.57
	25-50	10.15
	50-100	13.69
	100-200	9.59
	200-400	9.15
	400-800	5.94
Sample 01 (Remoulded)	12.5-25	5.42
	25-50	6.50
	50-100	8.47
	100-200	10.01
	200-400	7.56
	400-800	6.90

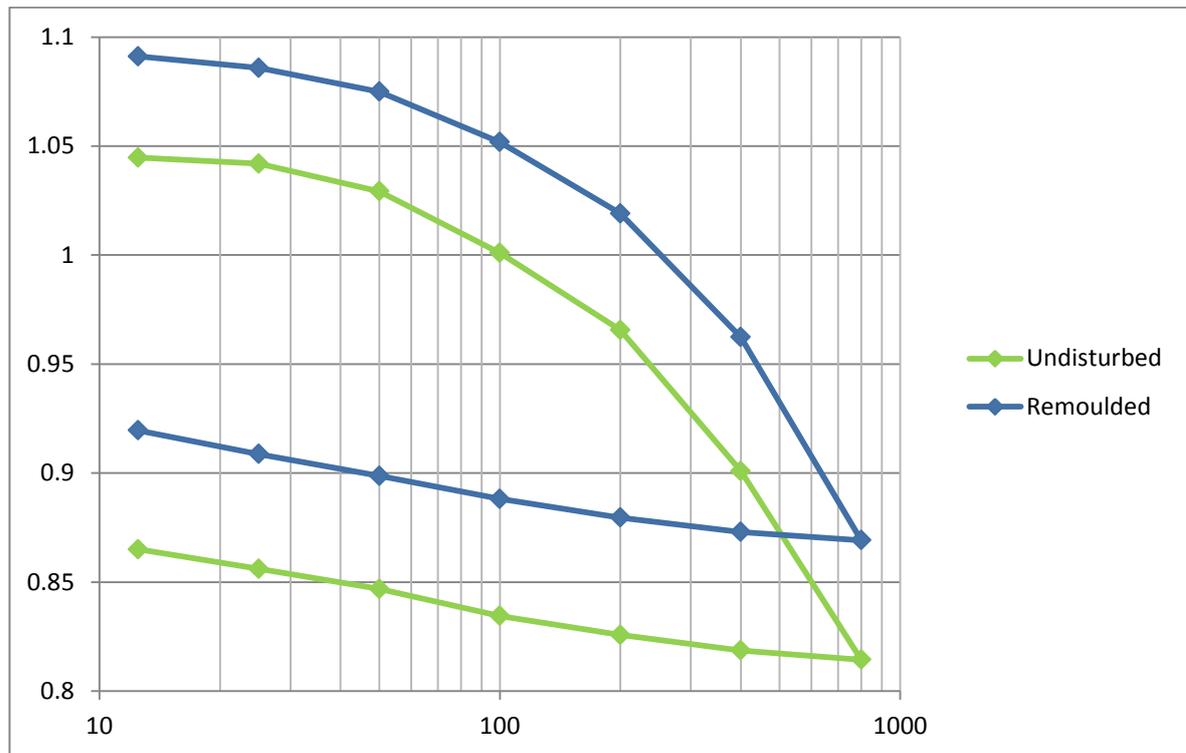


Figure 7-7 Typical change in void ratio versus log applied load during oedometer testing for remoulded and undisturbed test specimens

Typical C_v values obtained using different test methods and for a variety of different materials are summarised in Table 7-4 below.

Table 7-4 Typical C_v values obtained from various methods (after Robinson and Allam, 1996)

Soil	Range of Pressure σ' (kPa)	c_v (m^2/yr)		
		Logarithm-of-time method	Square-root-of-time method	Early stage log t method
Red earth	25-50	1.46	1.72	1.93
	50-100	2.03	2.52	2.84
	100-200	2.31	3.15	3.60
	200-400	2.57	3.44	3.96
	400-800	2.55	3.78	4.04
Brown soil	25-50	1.20	1.40	1.71
	50-100	0.95	1.19	1.20
	100-200	0.90	1.07	1.11
	200-400	0.66	0.70	0.86
	400-800	0.41	0.46	0.43
Black cotton soil	25-50	1.60	2.07	3.07
	50-100	0.97	1.16	1.51
	100-200	0.63	0.79	1.09
	200-400	0.36	0.50	0.64
	400-800	0.18	0.20	0.25
Illite	25-50	0.52	0.71	0.79
	50-100	0.42	1.00	1.05
	100-200	0.69	1.00	1.15
	200-400	0.99	1.45	1.62
	400-800	1.31	1.84	2.03
Bentonite	25-50	0.02	0.04	0.05
	50-100	0.01	0.03	0.04
	100-200	0.01	0.02	0.03
	200-400	0.01	0.01	0.01
	400-800	0.00	0.01	0.01
Chicago clay	12.5-50	7.92	14.35	14.51
	25-50	6.34	7.54	9.93
	50-100	4.32	5.49	6.37
	100-200	1.00	1.49	1.57
	200-400	1.44	1.39	1.55
	400-800	1.91	2.03	2.34
	800-1600	2.24	2.72	2.87

A significantly higher rate of primary consolidation is obtained from calculating the C_v values using the tri-axial consolidation stage test results for both the undisturbed and the remoulded specimens, with drainage during consolidation taking place from one end only. Readings during tri-axial testing typically reached 50% of total consolidation during the first 60 seconds of testing and 90% within the first 10 minutes (Figure 7-8). The resultant calculated C_v values from the tri-axial test results exceed $100 m^2/year$ for all test specimens.

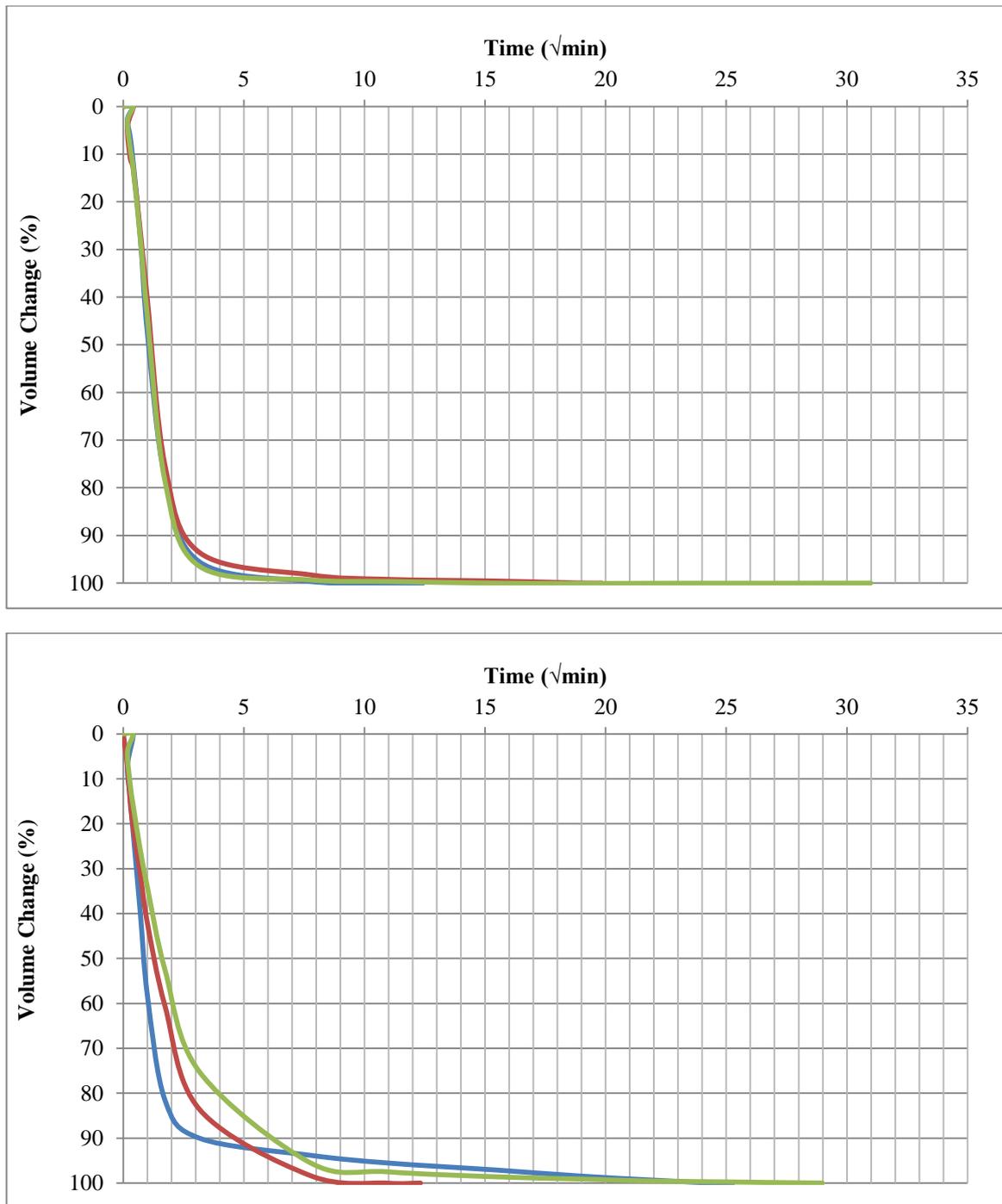


Figure 7-8 Typical rate of volume change during tri-axial consolidation stage for remoulded (above) and undisturbed (below) test specimens

Compressibility values (M_v) of the colluvium samples range between 0.06 and 0.45 m^2/kN . This represents material varying between low to high compressibility and typical of moderately over-consolidated to normally consolidated soils (Table 7-5).

Table 7-5 Typical m_v values and compressibility (Carter and Bentley, 1991)

Type of Clay	Description	m_v		
		(x10 ⁻³ m ² /kN)		
		Lower Limit	Upper Limit	Average
Heavy over-consolidated boulder clays, stiff weathered rocks and hard clays	Very low compressibility	0.05		
Boulder clays, marls, very stiff tropical red clays	Low compressibility	0.05	0.1	0.075
Firm clays, glacial outwash clays, lake deposits, weathered marls, firm boulder clays, normally consolidated clays at depth and firm tropical red clays	Medium compressibility	0.1	0.3	0.2
Normally consolidated alluvial clays such as estuarine and delta deposits, and sensitive clays	High compressibility	0.3	1.5	0.9
Highly organic alluvial clays and peats	Very high compressibility	1.5		

The Compressibility Index (C_c) of the material provides an indication of the rate of the decrease in the void ratio with an increase in the applied pressure for that linear portion of the void ratio versus log stress curve (Northmore *et al.*, 1992). C_c values determined from the oedometer and tri-axial consolidation test results vary between 0.25 and 0.50 for the remoulded and undisturbed colluvium material samples.

Recompression Index (C_e) values of all the colluvium material varies between 0.01 and 0.05, which is considered very low when compared to the typical values of soils from temperate climates.

7.4 Swell Pressure

From the definition in BS 1377-5 (1990), swelling pressure can be defined as the necessary vertical pressure applied to the test specimen in a consolidometer test setup to prevent the sample from swelling and recording a positive volume change when introduced to additional moisture.

No swell (0%) was recorded from any of the samples with the increase in moisture. This lack of volume change with an increase in moisture content is due to the high natural moisture content already causing separation of the constituent clay minerals.

7.5 Permeability

Measurements of permeability of the yellow and red brown colluvium materials from the mining area were evaluated during tri-axial testing (Table 7-6). Flexible wall permeability testing was carried out during the second consolidation stage at a 200 kPa consolidation pressure. This included evaluation of both undisturbed and completely destructured and remoulded to either the *in-situ* density or 95% of maximum dry density specimens in an effort to critically evaluate the

effect of remoulding and the inherent structure on the permeability of the material. All tri-axial test specimens were subjected to saturation under elevated back pressure and all analyses assume full saturation of the test specimens. The saturation time is taken as the moment at which the pressure at the top of the specimen equals the back pressure and subsequently the time required for the applied pressure to diffuse through the specimen. Theoretically, if the system is saturated there is no flow during this stage, and Skempton's pore pressure coefficient $B = 1$. Practically however, pore pressure coefficients varying between $B = 0.95$ and $B = 0.97$ were assumed to represent complete saturation of the specimen at the particular back pressure.

Table 7-6 Calculated permeabilities of undisturbed and remoulded colluvium test specimens

Sample	Permeability (m/s)	Sample State
1	1.30×10^{-6}	Remoulded (<i>In-situ</i>)
2	5.90×10^{-6}	Remoulded (<i>In-situ</i>)
3	4.10×10^{-6}	Remoulded (<i>In-situ</i>)
4	3.70×10^{-6}	Remoulded (<i>In-situ</i>)
5	4.60×10^{-6}	Remoulded (<i>In-situ</i>)
6	2.00×10^{-5}	Remoulded (<i>In-situ</i>)
7	1.50×10^{-5}	Remoulded (<i>In-situ</i>)
8	3.80×10^{-6}	Remoulded (<i>In-situ</i>)
9	2.00×10^{-6}	Remoulded (<i>In-situ</i>)
10	6.1×10^{-6}	Remoulded (<i>In-situ</i>)
11	3.9×10^{-6}	Remoulded (<i>In-situ</i>)
12	3.6×10^{-6}	Remoulded (<i>In-situ</i>)
13	$9.5E \times 10^{-6}$	Remoulded (<i>In-situ</i>)
14	7.60×10^{-9}	Undisturbed
15	7.80×10^{-8}	Undisturbed
16	2.00×10^{-7}	Undisturbed
17	8.70×10^{-8}	Undisturbed
18	1.20×10^{-8}	Undisturbed
19	4.5×10^{-7}	Undisturbed
20	4.70×10^{-8}	Remoulded (95% MDD)
21	7.20×10^{-9}	Remoulded (95% MDD)

From the flexible wall permeability test results summarised in Table 7-6, a distinct variation in the permeability values of the undisturbed samples and those specimens remoulded to *in-situ* density is obvious. The permeability values of the destructured and remoulded specimens typically exceed the permeability values of the undisturbed specimens of the same material and the same density between 1 and 3 orders of magnitude. The permeability values of the samples remoulded to 95% of the maximum dry density are however in the same range and up to an order of magnitude lower than the values determined for the undisturbed specimens (Figure 7-9).

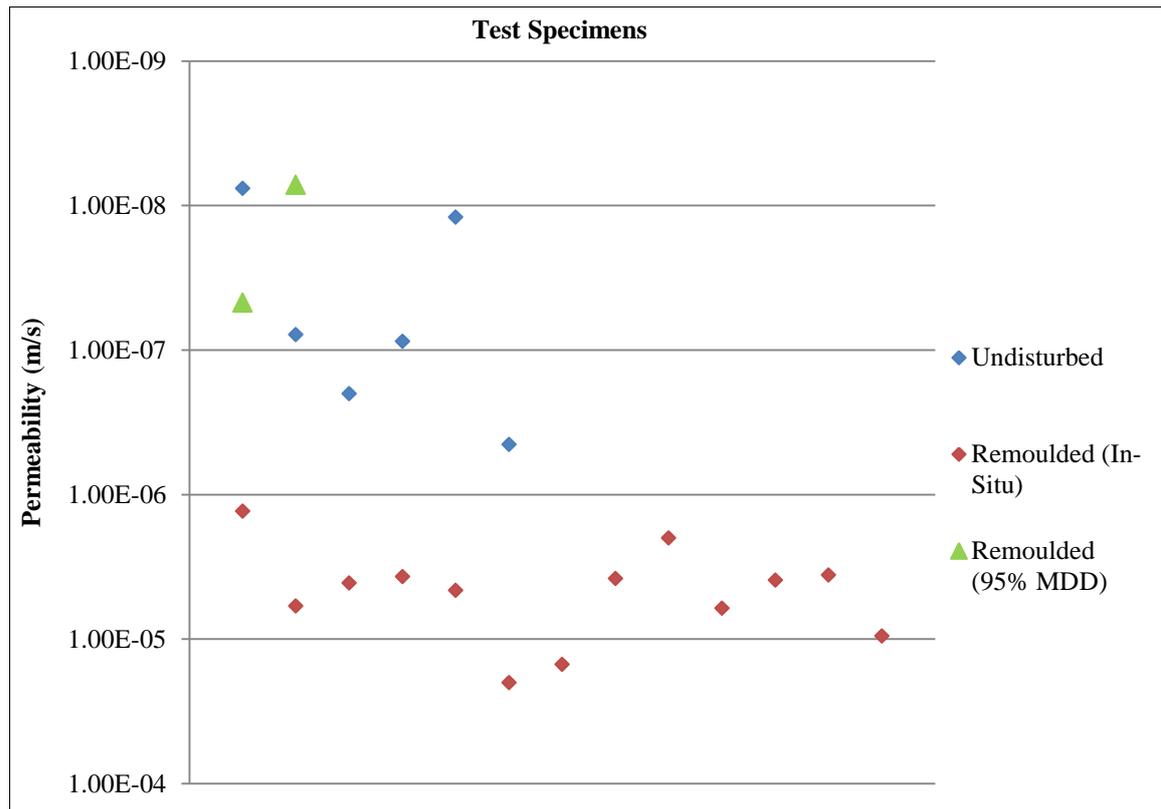


Figure 7-9 Variation in permeability of undisturbed and remoulded samples

7.6 Discussion

7.6.1 Shear Strength

From the consolidated-undrained tri-axial and shear box test results of this study, the yellow and red brown colluvium materials from the Mt. Lekoumou mining area generally exhibit lower values of effective cohesion and higher effective angles of friction than the yellow brown colluvium material found in the lower-lying areas outside the mining area. The test results revealed largely similar stress-path and strength behaviour of disturbed and destructured materials. Colluvium test specimens from the mining area remoulded to *in-situ* densities on average exhibit slightly higher effective friction angles than the undisturbed samples of the same material (average = +2.3° higher). Samples remoulded to 95% of the maximum dry density report a further increase in the effective angle of friction (average = + 2.6° higher) compared to those samples that were remoulded to natural density. Last mentioned are however based on results using the shearbox test apparatus, the difference in testing methods which could also possibly account for the observed variation in effective friction angle.

The majority of test specimens were found to exhibit slight strain-softening behaviour after failure at low total axial strain percentages varying between 1 to 3%. Yield and/or deformation under increasing stress therefore occur in a similar fashion than moderately over-consolidated clays.

From the undrained stress-path plots, the majority of the tested specimen curves are seen to initially veer slightly to the right due to apparent axial dilation. The majority of the test specimens failed in contraction and very near to the point of first contact with the critical state line, characteristic of materials of high percentage clay content. No distinct variation between the

stress-path geometries and failure modes of the undisturbed and remoulded specimens exists, or between those of the test specimens from and outside the mining area.

Wesley (1977) and Northmore *et al.* (1992) reported a positive relationship between moisture content and the effective friction angle for a range of tropical soil samples. This is the opposite of the typical relationship established between these parameters in temperate soils. No distinct and direct correlation could be established between these parameters during this study, with a wide variety of effective friction angles reported for a particular range of sample moisture contents.

Kenney (1959) developed an empirical approximation relating the effective friction angle ($\sin \phi'$) to the plasticity index of normally consolidated soils, assuming a general decrease in the value of the effective angle of friction with an increase in the plasticity index value (Figure 7-10):

$$y = -0.102\ln(x) + 0.8146 \quad (\text{Eq. 7-6})$$

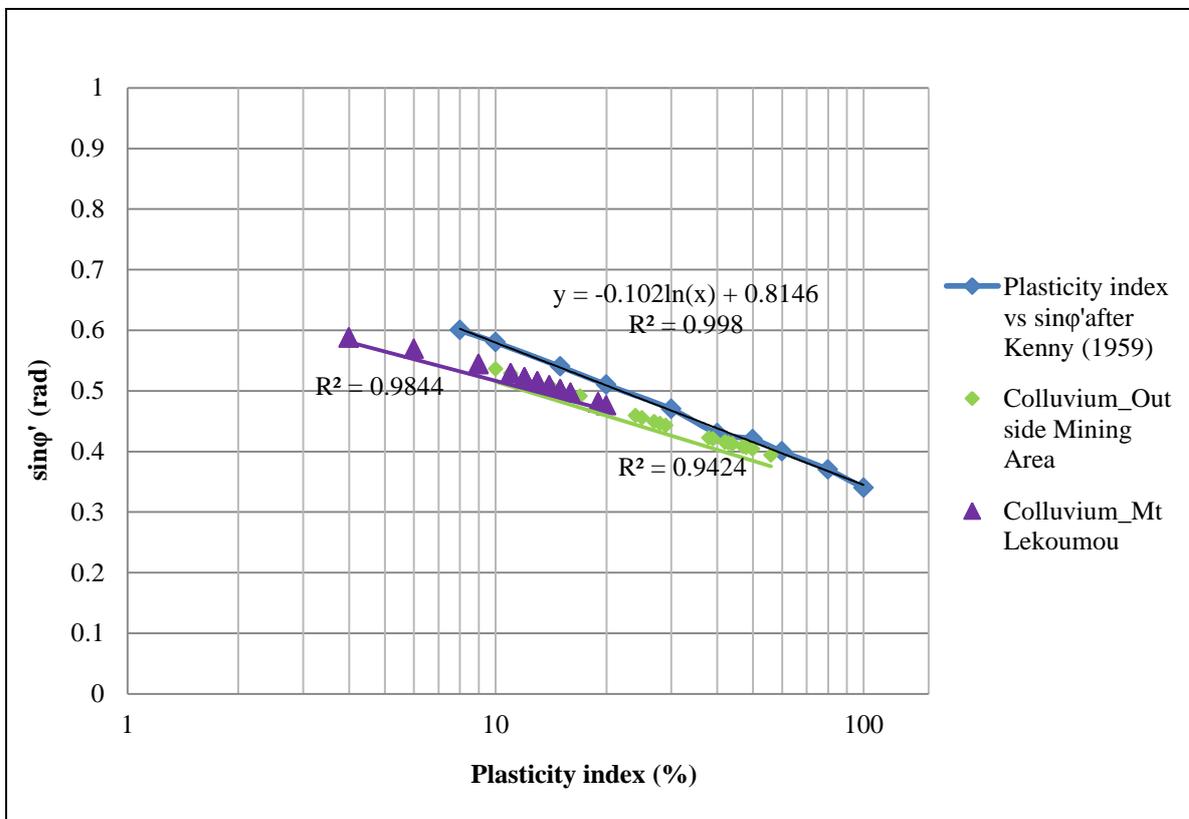


Figure 7-10 Plasticity index versus $\sin \phi'$ (Kenney, 1959) for all colluvium samples

The minimum and maximum plasticity indices of the yellow and red brown colluvium in the mining area equal 5 and 20% respectively. From the aforementioned relationship established by Kenney (1959), these minimum and maximum plasticity index values would equal effective angle of friction values of 35.3° to 28.4° respectively. These correlated values are very similar to the angles of friction established for the undisturbed test specimens from the mining area, but are slightly lower than what was recorded for the samples from the mining area which were remoulded to *in-situ* densities.

The minimum and maximum plasticity indices of the yellow brown colluvium from outside the mining area equal 10 and 50% respectively, correlating to effective angle of friction values of 32.4° to 23.8° respectively. These correlated values are also very similar to the range of effective friction angle established during tri-axial and shear box testing.

7.6.2 Compaction Characteristics

Proctor Compaction tests were carried out for a number of bulk samples of the yellow and red brown colluvium from inside and outside the mining area. Due to the high natural moisture content and degree of saturation of the material, the colluvium in its un-dried state was found to be consistently wet of the actual optimum moisture content. In an effort to effectively determine the optimum moisture content and maximum dry densities of the material, air-drying of selected separate sub-samples was specified to effectively plot the variation of achieved density with moisture content.

From the results of the compaction tests summarised above, the optimum moisture content (OMC) and maximum dry density (MDD) values of the yellow brown colluvium material found in the low-lying areas away from the Mt. Lekoumou mining area vary between OMC = 20% to OMC = 27% and MDD = 1486 kg/m^3 to MDD = 1605 kg/m^3 . This represents slightly lower MDD values than what would be expected from temperate clays of similar plasticity and USCS classification, but significantly higher OMC.

The MDD values achieved for the red brown colluvium material from the mining area vary between MDD = 2370 kg/m^3 to MDD = 2822 kg/m^3 at OMC = 7.8% to OMC = 11.7%. Only a single Proctor Compaction test was completed on yellow brown colluvium material from the mining area, with a calculated MDD of 2109 kg/m^3 at an OMC of 13.3%. The colluvium materials from the mining area have significantly higher MDD values than those densities achieved for the colluvium away from the mining area and which would be expected from temperate soils of similar plasticity and USCS classification. This can solely be ascribed to the mineralogy of the mining area material and the subsequently significantly higher SG values due to the high hematite and goethite contents. The OMC at which these densities are achieved is however significantly lower than the OMC values of the colluvium outside the mining area and the natural moisture contents of the material. Significant air-drying of the Mt. Lekoumou colluvium materials was necessary to determine the optimum moisture content, with the calculated OMC in some instances being between 20% and 30% lower than the natural moisture content of the material.

The majority of the tested specimens have liquid limit values in the range of 30% to 70%. The resultant compaction curves generally plot as single-peak curves, with a number of specimens that have highly irregular shapes and distributions. No correlation could however be found between the graph shape and liquid limit of the specimen as was found by Lee and Suedkamp (1972). Samples of high liquid limits in the range of 50% to 80% were found to just as frequently have irregularly shaped moisture content to dry density relationships as that it plots as well-defined single-peak curves. The colluvium from the mining area however plot only as single-peak and well-defined curves, regardless of the liquid limit of the material. It is interesting to note that there does seem to be some correlation between the distribution of the fines fraction and the shape of

the compaction curve. Whenever the silt and clay fractions are present in roughly equal proportions, the single-peak curve tends to be well-defined whereas whenever a particular fines fraction dominates the particle size distribution of the specimen, the resultant compaction curve tends to be irregularly shaped.

The compaction curves of the different samples show that in general, the maximum dry density of the majority of the colluvium specimens is attained between the 10% (90% saturation) and 5% (95% saturation) air voids curves. This is slightly lower levels of saturation than what would be expected for temperate climate soils.

7.6.3 Consolidation

The consolidation characteristics of the colluvium material were evaluated and plotted from the oedometer tests as a semi-logarithmic function of the consolidation (Δe) of the sample with increase in load as well as plots of percentage consolidation versus \sqrt{t} time derived from the tri-axial consolidation stage. The void ratio over the logarithmic applied pressure consolidation graphs suggest the clays behave as moderately over-consolidated soils, with an approximate pre-consolidation pressure of between 100 and 200 kPa for all the samples. When plotted on a conventional or linear scale, the presence of a pre-consolidation or yield pressure is no longer evident and the consolidation with increasing applied pressure is largely linear.

No variation in the consolidation behaviour of the remoulded and undisturbed specimens was also found to exist. This similarity in behaviour was observed a number of times during the study of the available literature. The majority of tropical red soils exhibit a stiff undisturbed structure which is destroyed by remoulding the sample, resulting in significant change in the compression curves of the remoulded and undisturbed samples. A number of these soils however tend to exhibit virtually no change after remoulding and de-structuring, appearing to exist naturally in a structureless state.

The initial, undisturbed void ratios of the colluvium material were found to be highly variable and no relationship exists between the *in-situ* void ratio of the colluvium material and the elevation or position in the landscape. Void ratios of the yellow and red brown colluvium from the Mt. Lekoumou mining area and the yellow brown colluvium material from the lower-lying areas outside the mining area were found to vary between 0.87 and 1.17, with an average of $e = 1$ (porosity = 0.50). Using the established phase relationships between particle density, moisture content and bulk density, the initial void ratios of the remoulded samples were found to be higher than that of the undisturbed specimens, believed to be largely due to the change in material properties (particle density and size distribution) through the change in moisture content during remoulding.

A significantly higher rate of primary consolidation is obtained from calculating the C_v values using the tri-axial consolidation stage test results. This is true for both the undisturbed and the remoulded specimens. Readings during tri-axial testing typically reached 50% of total consolidation during the first 60 seconds of testing and 90% within the first 10 minutes. The resultant calculated C_v values from the tri-axial test results exceed $100\text{m}^2/\text{year}$ for all test specimens. The high rate of primary consolidation typically gives rise to a “head-and-shoulders”

stress path geometry due to bedding between the load cap and sample. These errors and subsequent stress path geometry was observed without exception for all the samples from effective stresses of 200 kPa and greater.

The Compressibility Index (C_c) of the material provides an indication of the rate of the decrease in the void ratio with an increase in the applied pressure for that linear portion of the void ratio versus log stress curve (Northmore *et al.*, 1992). C_c values determined from the oedometer and tri-axial consolidation test results vary between 0.25 and 0.50 for the remoulded and undisturbed colluvium material samples. Vargas (1973) established the relationship between the Compressibility Index and the liquid limit of tropical red soils from Brazil, similar to the relationship derived by Terzaghi and Peck (1967) for clays from temperate climate zones (Mitchell and Sitar, 1982). It is however evident when comparing these two relationships that for those instances where the liquid limit of the material exceeds 50%, the typical compressibility of tropical soils is expected to be lower than would be expected for temperate soils of the same liquid limit. Conversely, where the calculated liquid limit is less than 50%, the correlations established by Vargas and Terzaghi and Peck means that at lower liquid limits tropical soils will have higher values of compressibility than would be expected for temperate soils of the same liquid limit.

From the calculated C_c values and corresponding liquid limits, it is evident that the colluvium investigated during this study have even lower compressibility index to liquid limit ratios than that established for tropical red soils in Brazil by Vargas (1973). Additionally, C_c values of tropical soils at liquid limits greater than 50% are expected to be significantly lower than those of temperate climate soils of similar liquid limits (Figure 7-11).

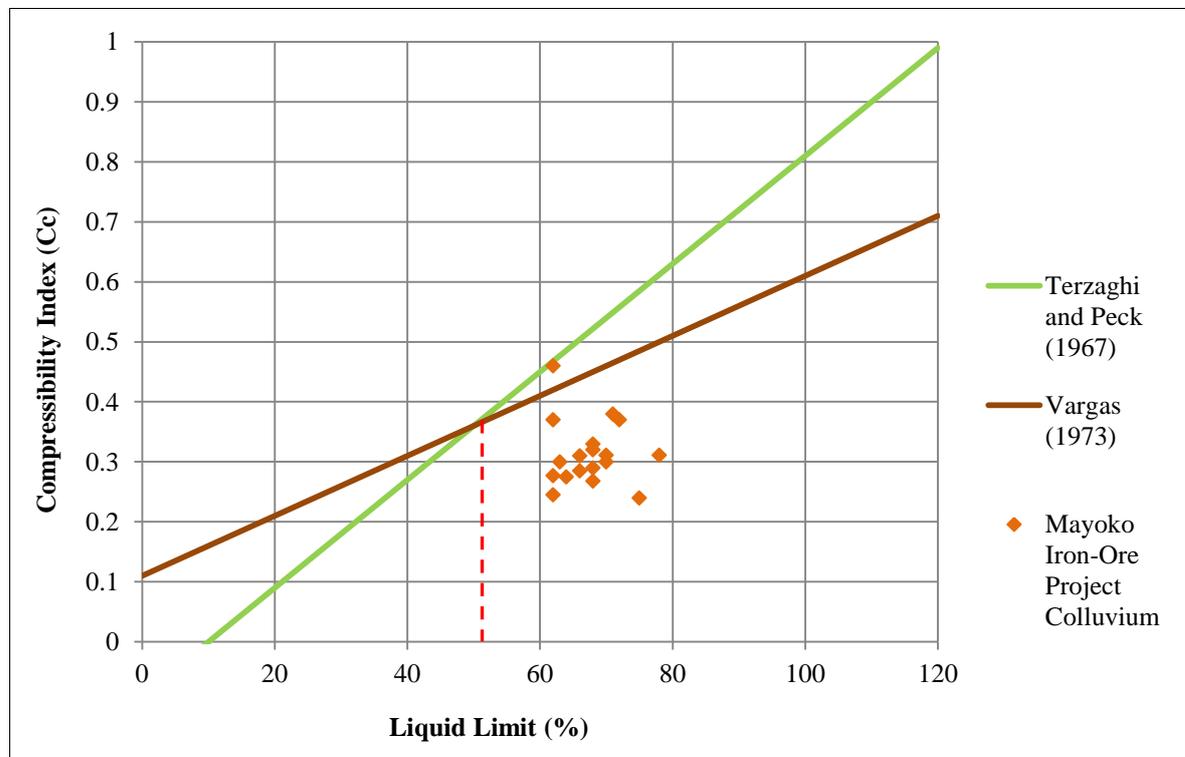


Figure 7-11 Compressibility Index as a function of Liquid Limit for tropical red soils (Vargas, 1973) and temperate climate soils (Terzaghi and Peck, 1967), also indicating critical 50% liquid limit and relationship established for the colluvium material during this study

The Recompression Index (C_e) provides an indication of the increase in void ratio of the material on unloading of the soil. The low C_e values calculated for the colluvium material indicates that very little “rebound” of the material structure will occur on unloading of the material, and the change in material structure due to loading can be considered permanent.

7.6.4 Permeability

From the flexible wall permeability test results reported during this study, a distinct variation in the permeability values of the undisturbed samples and those specimens remoulded to *in-situ* density is obvious. The permeability values of the destructured and remoulded specimens typically increased from the permeability values of the undisturbed specimens of the same material and the same density between 1 and 3 orders of magnitude. Those permeability values of the samples remoulded to 95% of the maximum dry density are however in the same range and up to an order of magnitude lower than the values determined for the undisturbed specimens.

This is contradictory to the findings of Northmore *et al.* (1992) who during their study of tropical red clays from Kenya and Indonesia found that the permeability of the compacted and destructured samples typically exhibited a decrease of 2 to 3 orders of magnitude from those in the undisturbed sample state. It is suggested that this observed decrease is due to the complete destruction of the natural soil structure and any biologically derived voids which may exist, leading to an increase in the overall permeability of the material in its natural or *in-situ* state.

In an effort to evaluate the actual permeability values observed from the tri-axial flexible wall test results, as well as the significant variation observed between the permeabilities of the undisturbed specimens and those specimens remoulded to *in-situ* density, the empirical permeability values of the colluvium material were determined using the Fair-Hatch (1933) approximation:

$$K = \left(\frac{pg}{\mu}\right) \left[\frac{n^3}{(1-n)^2}\right] \left[\frac{1}{m \left(\frac{C}{100} \sum \frac{P}{d_m}\right)^2}\right] \quad (\text{Eq. 7-7})$$

Where:

K = Permeability (m/s)

n = Porosity (Assumed $n = 0.5$)

m = Packing Factor (Assumed $m = 5$)

C = Shape Factor (Assumed $C = 6$)

P = Percentage material held between adjacent sieves (%)

d_m = Geometric mean of adjacent sieve sizes

Based on the aforementioned relationship and the available particle size distribution test results, the empirically determined permeability values of the colluvium material varies between 1.1E-07 and 1.0E-08 m/s (Figure 7-12). These permeability values are similar to those values observed for the undisturbed tri-axial test specimens and consequently also lower than those of the specimens remoulded to its *in-situ* density.

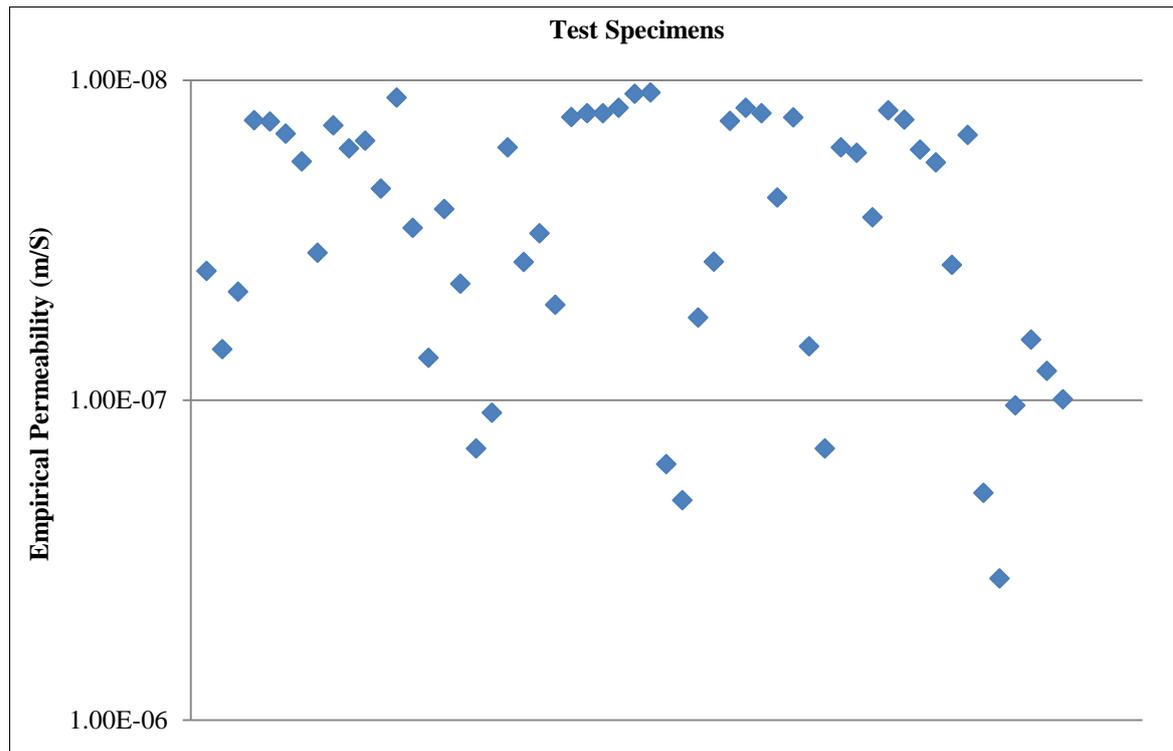


Figure 7-12 Range of empirical permeability values based on Fair-Hatch approximation

Based on these empirical results and the fact that the void ratio, particle density and bulk density of the remoulded and undisturbed specimens are the same, it is therefore assumed that the significantly higher degree of permeability observed for those specimens remoulded to the *in-situ* density to be a direct result of the remoulding and sample preparation process. Due to the high natural moisture content and degree of saturation of the colluvium material in its natural state, remoulding of the material to a density similar to the *in-situ* density at similar moisture contents was often found impossible without the material behaving in a complete liquid-like fashion. Consequently, the final moisture content of the remoulded specimens were necessitated to, on average, be reduced to lower than the natural moisture content of the colluvium material in its undisturbed state in order to achieve an overall density similar to that of the undisturbed specimens. This may directly result in the undisturbed samples having a higher initial degree of saturation than the specimens remoulded to the same density.

Permeability test results by Mitchell *et al.* (1965) of remoulded test specimens show that the permeability of a particular remoulded material is significantly influenced by the moulding moisture content. From their results and for different remoulded specimens of the same void ratio, the permeability of a specimen remoulded at a moisture content dry of optimum is typically in the range of 10^{-7} m/s, whereas the permeabilities of those specimens remoulded wet of optimum moisture have permeability values in the range of 10^{-10} m/s. A review of factors influencing the hydraulic conductivity of compacted clays is also discussed by Benson *et al.* (1994). During their study, the hydraulic conductivity of specimens collected from 67 compacted clay liners throughout the United States was examined. Their findings also indicate that the hydraulic conductivity of the test specimens depends greatly on the moulding water content and dry unit weight achieved during compaction. In particular, specimens with combinations of water content

and dry unit weight yielding higher degrees of saturation tend to have lower hydraulic conductivity values. To illustrate this behaviour, the hydraulic conductivity versus initial saturation was plotted in Figure 7-13 (Benson and Trast, 1995).

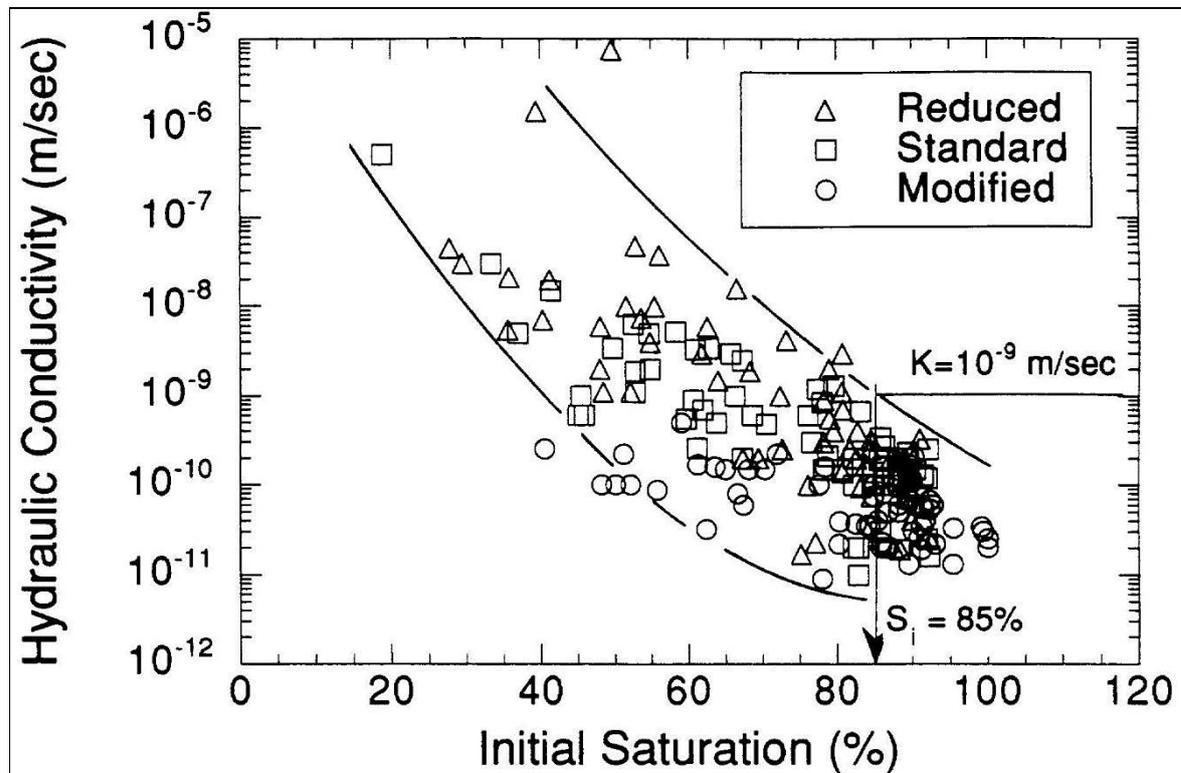


Figure 7-13 Hydraulic conductivity as a function of the initial degree of saturation (Benson and Trast, 1995)

A definitive trend of decreasing hydraulic conductivity with increasing initial saturation exists. It was also noted that those specimens compacted at higher degrees of compactive effort also frequently have lower hydraulic conductivity values (Benson and Trast, 1995).

Considering the sensitivity of the colluvium material to any drying, as well as the influence of the initial degree of saturation and remoulding water content of the material, it is proposed that the variation in permeability is the direct result of a combination of the following factors:

- Due to the sensitivity of the colluvium material to drying, any drying of the material resulted in a change in the specific gravity/particle density of the material. As all of the remoulded colluvium specimens represent colluvium specimens from the Mt. Lekoumou mining area, an increase in the SG of the material (discussed in Section 6) is expected due to the precipitation of sesquioxides out of solution from the groundwater on a decrease in moisture content. In order to maintain the same bulk density than the *in-situ* material (of lower SG), the remoulded specimens will consequently need to be remoulded at a higher porosity in order to achieve the same bulk density, even at lower moisture contents.
- A change in the natural moisture content of the colluvium material will result in a direct change of the overall particle size distribution due to the aggregation and cementation of the particles, resulting in so-called “bi-modal” behaviour of the material. This increase in average grain size is

expected to lead to an increase in the dimensions of the inter-particle void dimensions, acting as significantly better conduits for fluid transfer than the *in-situ* micro-structure.

- The combination of the increase in porosity and necessitated decrease in remoulding moisture content from the level of natural moisture content further directly results in a significant decrease in the initial degree of saturation of the remoulded specimens. By remoulding at lower degrees of saturation, any aggregates that will have developed during drying will be stiff and difficult to remould (Benson and Daniel, 1990) and clay particles flocculated (Lambe, 1958). This will add to a more permeable micro-structure of the remoulded samples.

8. Suction

8.1 Evaluation of Soil Suction – Filter Paper Method

Matric soil suction measurements were conducted on six undisturbed samples of the yellow brown colluvium from the mining area using the filter paper method proposed by Bulut *et al.* (2001). Once the moisture contents were established for the various filter paper test specimens, the matric suction pressures for the colluvium samples at that particular moisture content could be determined using the relationships discussed in Section 4 and summarised in Figures 8-1 and 8-2 below.

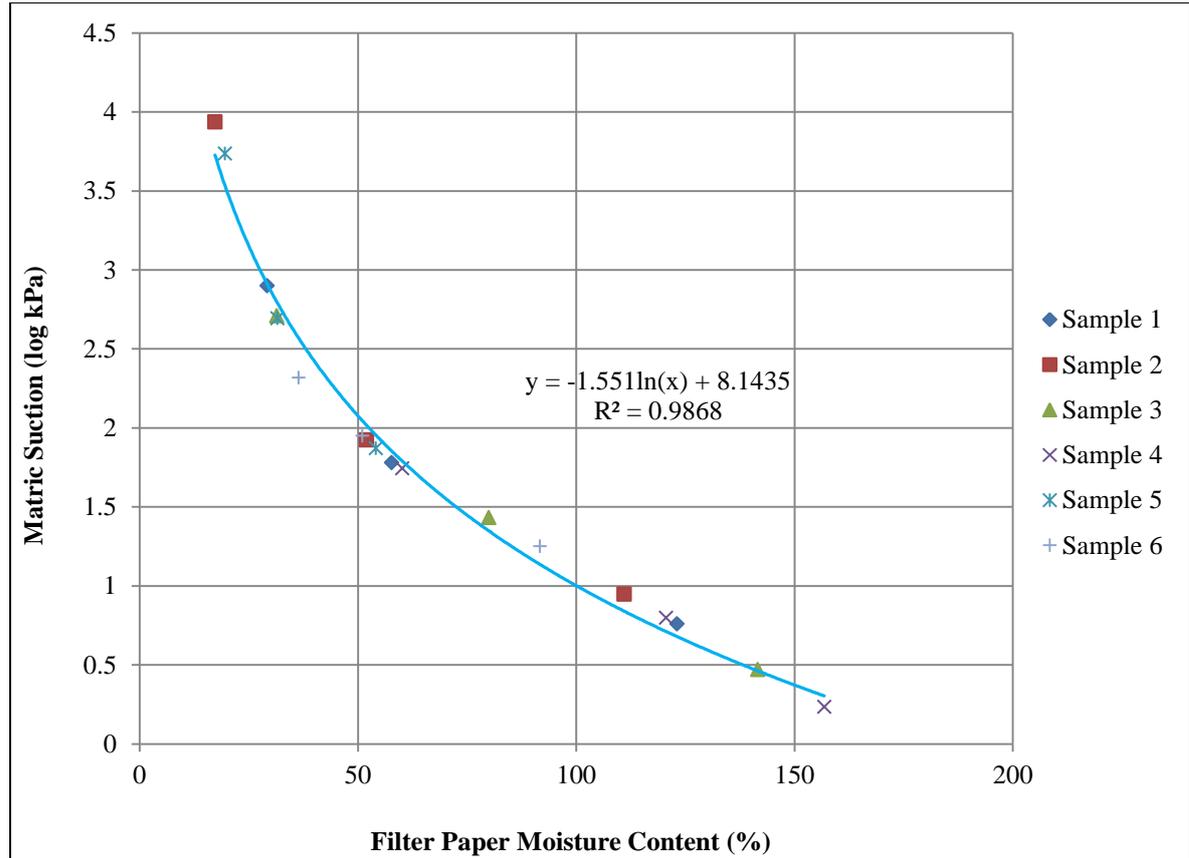


Figure 8-1 Wetting curve for the matric suction pressures at various filter paper water contents

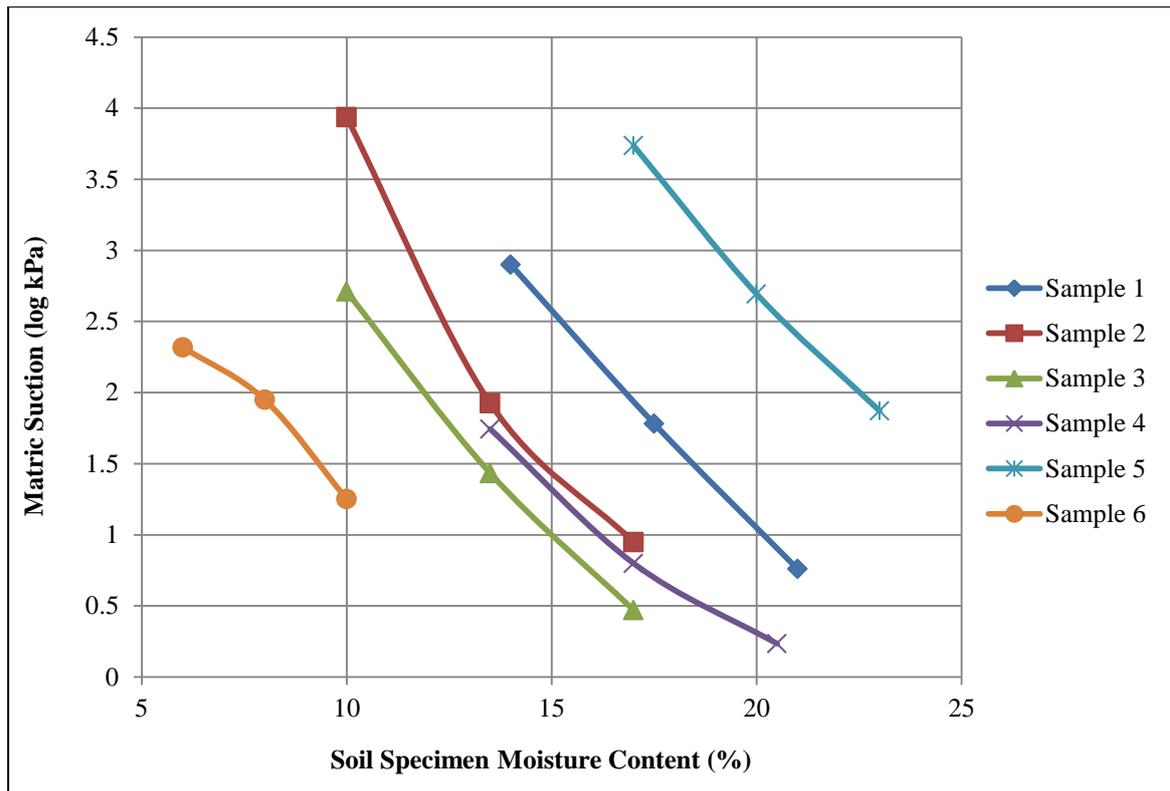


Figure 8-2 Filter paper established relationship between matric suction pressures and soil moisture content

From the filter paper measurement results summarised here, it is evident the matric suction pressures of the colluvium materials tend to decrease in an almost linear fashion with increase in the soil specimen moisture content up to 15 – 20%. From this range of moisture contents, the extent at which matric suction pressure decreases with increasing moisture content appears to decline for a number of samples. It is clear that there exists a significant variation in matric suction pressure values for different samples of the same colluvium material and at the same moisture content, i.e. at 10% moisture content matric suction pressures of 1.25, 2.75 and 3.9 log kPa have been recorded for different sample specimens.

8.2 Evaluation of Soil Suction – Pressure Plate Extractor Apparatus

The soil suction characteristics of seventeen undisturbed samples of the colluvium material from the Mt. Lekoumou area were also determined using a Pressure Plate Extractor (PPE) apparatus with an applied suction pressure range of 0 to 1500 kPa (Figures 8-3 and 8-4).

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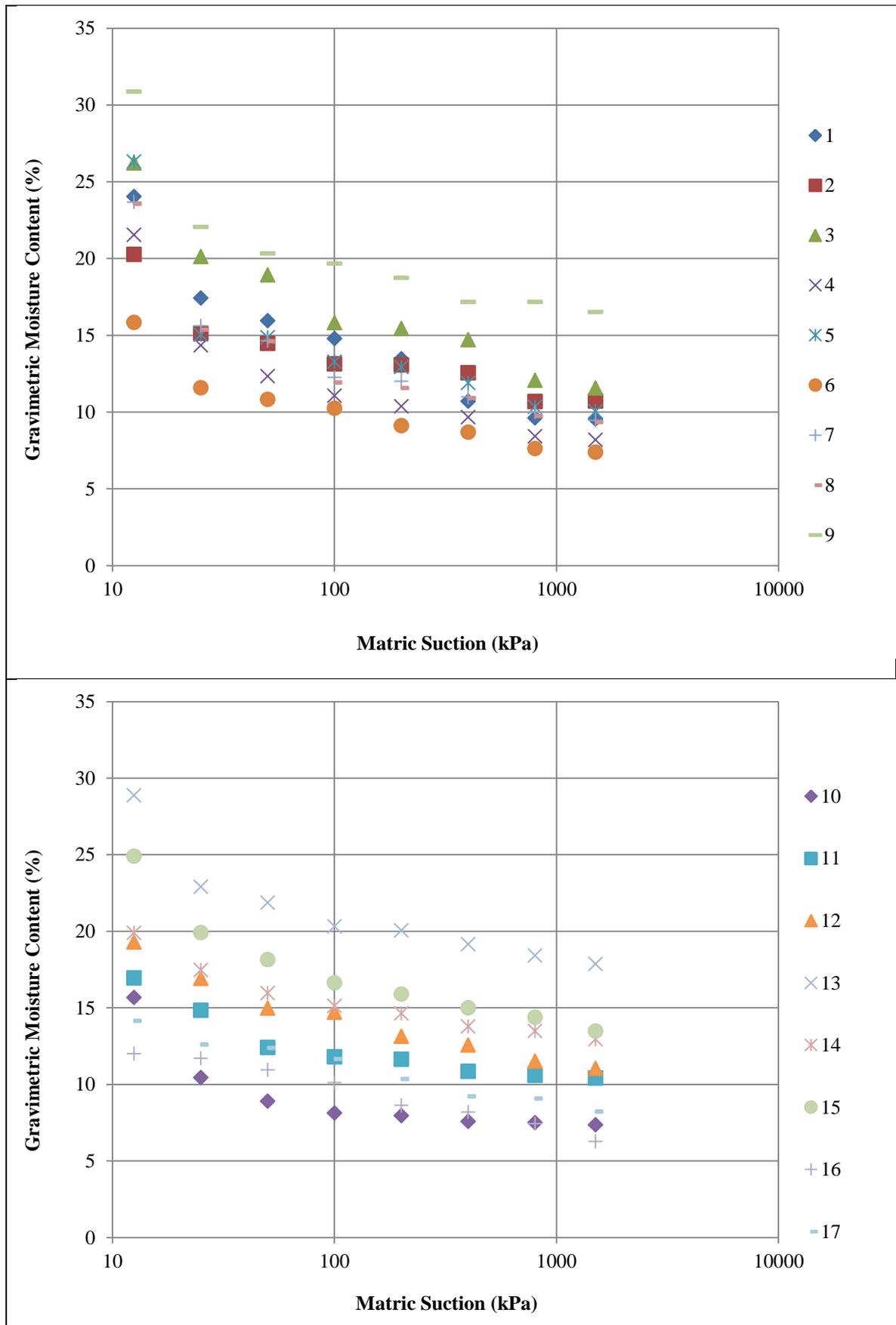


Figure 8-3 SWCC of colluvium samples from Mt. Lekoumou mining area

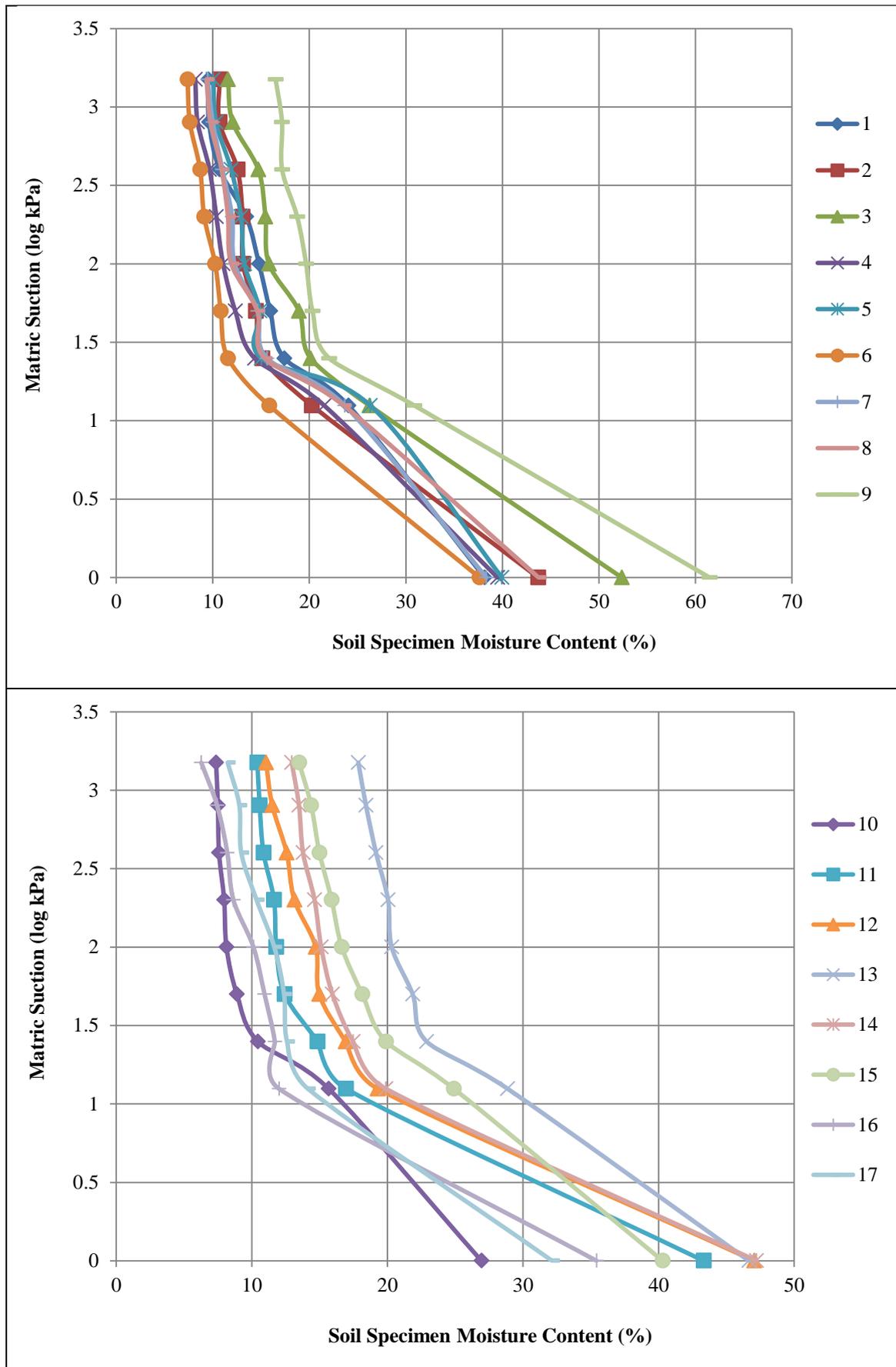


Figure 8-4 Relationship between matric suction pressures and soil moisture content determined from PPE

From the PPE measurement results summarised here, it is evident the matric suction pressures of the colluvium materials tend to decrease in an almost linear fashion with increase in the soil specimen moisture content up to 15 – 25%. Above this range of moisture contents, the extent at which matric suction pressure decreases with increasing moisture content declines for all the tested specimens up to the final moisture content. It is again clear that there exists a significant variation in matric suction pressure values for different samples of the same colluvium material and at the same moisture content.

8.3 Discussion

From the filter paper and PPE measurement results summarised here, it is evident the matric suction pressures of the colluvium materials tend to decrease in an almost linear fashion with increase in the soil specimen moisture content up to 15 – 25%. From this range of moisture contents, the extent at which matric suction pressure decreases with increasing moisture content declines for all the tested specimens up to the final moisture content. It is again clear that there exists a significant variation in matric suction pressure values for different samples of the same colluvium material and at the same moisture content.

Figures 8-2 and 8-4 indicate the variation in matric suction pressures with variation in the soil specimen moisture content. A very good correlation exists between the matric suction values for specific moisture content obtained using the PPE apparatus and those determined using the filter paper method, as well as between the behaviour of the matric suction pressures with increasing moisture content.

Matric suction pressures contribute to the shear strength of the soil via the water inter-aggregate contact area (Fredlund *et al.*, 1995), with very little change occurring in the moisture content of the soil below the air-entry value and the contribution of matric suction pressure to the shear strength of the material above the air-entry value decreasing with increasing moisture content.

It can therefore be argued that for the tested colluvium samples and at those moisture contents higher than 25%, the effect of a change in moisture content on the matric suction pressure and overall shear strength of the colluvium material will be significantly less than what the result would be at lower moisture contents. For all samples at moisture contents lower than 15%, a small variation in moisture content results in a significant variation in matric suction pressures and potentially the overall shear strength of the colluvium material.

The extent to which the shear strength of the colluvium material is influenced by the matric suction pressure is not investigated and reported on here. Considering the high natural moisture contents of the colluvium material in its *in-situ* state and the low matric suction pressures existing at these moisture contents, it is expected that the additional contribution of matric suction pressures to the shear strength of the colluvium material will be negligible.

9. Conclusions and Recommendations

The characteristics and behaviour of a number of transported tropical red soil horizons in the selected study area were established and evaluated according to conventional and specialised testing regimes. The findings of the research are compared to the typical properties for soils from temperate climatic regions and residual tropical red soils as summarised in the literature, including the chemical, mineralogical, index and mechanical properties of these soils and the sensitivity to variation in standard test methods.

9.1 Classification and Nomenclature of Tropical Soils

Tropical red soils are very common throughout the tropical regions of the world, comprising a wide variety of soils of which the material and engineering properties may vary significantly. Due to the large variation in the aforementioned properties, classifying these materials as distinct and clearly defined soil types is avoided. Unfortunately, this approach has resulted in the grouping of materials of largely varying characteristics and engineering behaviour under a single encompassing term, resulting in significant confusion in the evaluation and correlation of material properties and the expected behaviour of seemingly “similar” soils.

From the examples and other studies encountered throughout the available literature, it is evident that varying definitions and classifications of laterite, ranging from being very specific to highly generalized, exist. The fact that the definition of “laterite” and all other derivative and associated terms has subsequently been differently interpreted and often generalized in the past, has resulted in the grouping of materials that may have significantly different chemical, geological and/or geotechnical properties as a single soil type. As a result numerous inconsistencies have developed in the identification and classification of tropical soils and laterites, with a significant range of geological, geotechnical and chemical properties being assigned to both lateritic and tropical red soils. The engineer or geologist who intends to use the available data in the literature to supplement or compare the data from his or her own studies, should therefore exercise extreme caution in ensuring the relevance of the data to the soil horizons identified in his or her current study area.

9.2 Sampling and Sample Disturbance

Mechanical sample disturbance, either during sampling or transport of the samples from site to the laboratory, is frequently ascribed as to playing a significant role in the engineering behaviour of the respective test specimens. The stress-path geometries and consolidation data obtained for all the undisturbed and remoulded colluvium test specimens were however largely similar and subsequently result in very similar shear strength and consolidation characteristics. This may indicate that these transported soils may possibly exist in a naturally unstructured state, or very near to unstructured, in which case sample disturbance or reworking of the material will have a minimal impact on the material behaviour during testing or on site. This however excludes the effect of the change in the natural moisture content of the material on the material properties and subsequent behaviour.

No to very little change in moisture content of the material was found to have occurred during transit from site to the geotechnical laboratory in South Africa. All undisturbed samples were retrieved from the exposed soil profile using a geological pick and spade. The samples were tightly wrapped in multiple layers of cling film to prevent the block samples from disintegrating, wrapped in tinfoil and then wrapped in multiple layers of cling film again in order to preserve the natural moisture content of the material during transportation of the samples back to South Africa. Natural moisture contents of samples subjected to the same drying temperature (65°C) and for the same drying time period (24hours) on site and on arrival in South Africa vary by a maximum of 0.5%.

9.3 Effect of Dispersing Agent on Particle Size Distribution

Numerous and often conflicting arguments and findings regarding the effect of the type of dispersing agent on the particle size distribution test results exist in the literature. The different dispersing agents used during hydrometer analysis appear to have a definitive effect on the resultant grain size distribution results of the transported soils investigated during this study. A distinct and sudden decrease of between 10 and 20% in the particle distribution results occurs between the transition from sieve to hydrometer analysis (i.e. in the coarse silt fraction) when using the Sodium Hexametaphosphate and Sodium Carbonate dispersing agent, after which the curve either completely flattens out (average 50% clay) or for the second group of results gradually continues downward to the final fines fraction (average 16% clay). Conversely, grading results obtained using the Sodium Pyrophosphate Decahydrate mixture continues from the final sieve aperture without any distinct drop in results when starting with hydrometer analysis.

Incomplete dispersion is believed to have resulted in significantly lower clay contents and the apparent concentration of particles in the coarse silt-size range of the second group of colluvium samples from outside the mining area. Consequently, the matrix classification of these samples classifies as “sandy silt” rather than the “sandy clay” results observed for the properly dispersed material from the mining area. Proper dispersion was however also achieved for a number of samples from the same area using the same dispersing agent, resulting in higher final clay contents similar to the yellow brown colluvium found in the lower slopes of the mining area.

9.4 Sensitivity to Change in Natural Moisture Content and Drying of Test Specimens

The sensitivity of tropical red soils to any change in its Natural Moisture Content (NMC) was confirmed during this study, with significant variation in the transported material properties and subsequently influencing the engineering behaviour of those samples subjected to various periods of drying. These changes to the material properties appear to occur regardless of the aggressiveness or period of drying to which the sample is subjected. Additionally, the changes in material characteristics due to the change in moisture content were found to be largely irreversible once the moisture content is increased back to the original or NMC even with considerable reworking of the material.

When determining the NMC of test specimens, conflicting views on the effect of the oven drying temperature on the calculated moisture content exist in the literature. Based on the results from

this study, the colluvium material appears to be highly sensitive to the temperature at which it is dried in the oven. Samples dried at higher oven drying temperatures (105 °C) reporting natural moisture contents of between 3.5% and 7.6% higher than the same material dried at a lower temperature of 65°C. The high moisture contents reported at higher drying temperatures could represent the removal of moisture from the structures of those hydrated minerals that are present in the material due to the aggressiveness of the drying method.

It was further observed that the different transported horizons exhibit varying levels of sensitivity to change in moisture content during air-drying. During air-drying, the yellow brown colluvium was prone to larger changes in moisture content and subsequently greater changes in the material properties and engineering behaviour than the red brown colluvium material air-dried for the same period of time. From the established NMC (using oven drying) and particle size distribution results it is evident that both the yellow brown and red brown colluvium have similar percentages clay content and initial natural moisture contents. Instead, it is rather proposed that the observed difference in sensitivity to drying be considered a function of the variation in mineralogical composition of the material and the subsequent difference in Cation Exchange Capacity (CEC) observed for the different horizons. The CEC is considered a function of the net negative charge of the colloidal fraction of a soil and is therefore expected to correlate positively with the attraction forces between particle surfaces and water (Nagahori and Haccho, 1981). The red brown colluvium specimens typically have CEC values exceeding that of the yellow brown colluvium by a minimum order of magnitude of 10^2 cmolM⁺/kg and is therefore believed to have significantly better water-holding capacities during non-aggressive drying procedures (such as air-drying) than the kaolinite-rich yellow brown colluvium, irrespective of the total clay content of the specimen.

9.5 Index Properties

When attempting to assess the activity of the colluvium soils through simple index test results, the plasticity indices of the different colluvium samples were plotted against the clay content of the material according to Skempton (1953) and Van der Merwe (1964). It was however found that the activity classification through Skempton's method tends to rather underestimate the potential activities of those samples of high clay contents and high plasticity indices and the Van der Merwe method to overestimate the potential activities of those samples of low to moderate clay contents and their equivalent gross plasticity indices. This is believed to be the direct result of the mineralogy of the material, where the percentage clay and expected activity do not correlate as well as that correlation established for temperate soils.

A significant variation in the distribution of the various particle sizes occurs on drying of the material, varying from a total classification of sand with fines to gravely sand with fines. This change in the overall grain size distribution upon drying was found to be the result of the aggregation of fine particles to form coarser particles, being largely irreversible and staying bonded when re-wetted using only water. Plastic limits were found to remain largely similar on drying, but the liquid limits and plasticity indices of the material were however found to decrease by up to 10% from the natural moisture content sample and the sample allowed to air-dry for three

days. It is evident from the results of this study that the Atterberg Limits are influenced by the degree of drying the test specimen has been exposed to.

In an effort to establish the influence of a change in moisture content on the specific gravity of tropical red soils investigated as part of this case study, the specific gravities of selected colluvium samples from both inside and outside the mining area were determined at its natural moisture content and after being allowed to air-dry for an extended period of time. Based on the results from this investigation it is clear that minor variation in the specific gravity of the colluvium specimens outside the mining area does occur, even when drying of the specimens only take place through air-drying at atmospheric conditions. SG values decrease through the loss of interlayer water from any hydrated minerals that may be present. It should be noted that the final recorded change in SG achieved through air-drying is only marginally less than the changes recorded in the literature and at the particular moisture content, but achieved using oven drying methods

The unexpected increase in SG values occurring with a decrease in moisture content of the colluvium samples from the mining area is argued to represent the precipitation of iron oxides from solution within the soil voids or on the crystal planes exposed on the face of the constituent clay mineral crystals during drying of the soil. From the chemical, XRD and XRF analysis completed on de-ionised water extracts and selected samples of the colluvium material, it is evident that at any point within the Mt. Lekoumou mining area any moisture present in the soil profile will contain extremely high concentrations of Fe^{3+} in solution. This holds particularly true for the red brown colluvium horizons, which were found to be weathered and leached to a lesser extent and therefore contain still higher percentages Fe^{3+} than the yellow brown colluvium horizons. The subsequently greater concentration of Fe^{3+} in solution within the soil moisture of the red brown colluvium material directly accounts for the similar and greater increases in SG observed with drying, even if the overall change in moisture was significantly less than that observed for the yellow brown colluvium samples.

9.6 Mechanical Properties and Permeability

Consolidated-undrained tri-axial and shear box tests were carried out on a number of remoulded and undisturbed samples retrieved of the different colluvium materials from various elevations both inside and outside the mining area. The majority of test specimens were found to exhibit slight strain-softening behaviour after failure at low total axial strain percentages. Yield and/or deformation under increasing stress therefore occur in a similar fashion than moderately over-consolidated clays. No distinct variation between the stress-path geometries and failure modes of the undisturbed and remoulded specimens exists.

Similarly, no variation in the consolidation behaviour of the remoulded and undisturbed specimens was found to exist. Extremely high rates of primary consolidation were recorded for all colluvium test specimens. Readings during tri-axial testing typically reached 50% of total consolidation during the first 60 seconds of testing and 90% within the first 10 minutes. The resultant calculated C_v values from the tri-axial test results exceed $100\text{m}^2/\text{year}$ for all test specimens.

The high rate of consolidation observed corresponds with the findings in the literature that, according to Northmore *et al.* (1992), a typical feature of the consolidation characteristics of tropical red soils when compared to temperate soils, is the extremely high rate of primary consolidation. Up to half of the total consolidation settlement may occur within the first couple of seconds of the test. At such high rates of consolidation the determination of a meaningful C_v is highly unlikely. Additionally, Clayton *et al.* (1995) acknowledges the fact that the C_v values obtained from those tests completed on the relatively small oedometer soil specimen will normally represent significant under-estimates of the actual rate of primary consolidation. Even when considering the rate of primary consolidation of only the completely destructured and remoulded samples, values of C_v still vary significantly and remain very high. It is therefore proposed that the use of the C_v parameter alone as an indication of the expected rate of consolidation of these tropical soils should be avoided.

When comparing the relationship of the calculated liquid limit and compressibility index (C_c) values, it is evident that where the liquid limit of the material exceeds 50%, the typical compressibility of tropical soils is expected to be lower than would be expected for temperate soils of the same liquid limit. Conversely, where the calculated liquid limit is less than 50%, the correlations established by Vargas and Terzaghi and Peck means that at lower liquid limits tropical soils will have higher values of compressibility than would be expected for temperate soils of the same liquid limit.

From the calculated C_c values and corresponding liquid limits, it is evident that the colluvium investigated during this study have even lower compressibility index to liquid limit ratios than that established for tropical red soils in Brazil by Vargas (1973). Additionally, C_c values of tropical soils at liquid limits greater than 50% are generally expected to be significantly lower than those of temperate climate soils of similar liquid limits.

From the flexible wall permeability test results reported during this study, a distinct variation in the permeability values of the undisturbed samples and those specimens remoulded to *in-situ* density is obvious. The permeability values of the destructured and remoulded specimens typically increased from the permeability values of the undisturbed specimens of the same material and the same density between 1 and 3 orders of magnitude. Those permeability values of the samples remoulded to 95% of the maximum dry density are however in the same range and up to an order of magnitude lower than the values determined for the undisturbed specimens. Keeping in mind the sensitivity of the colluvium material to any drying, as well as the influence of the initial degree of saturation and remoulding water content of the material, it is proposed that the variation in permeability is the direct result of a combination of the following factors:

- Due to the sensitivity of the colluvium material to drying, any drying of the material resulted in a change in the specific gravity/particle density of the material. As all of the remoulded colluvium specimens represent colluvium specimens from the Mt. Lekoumou mining area, an increase in the SG of the material (discussed in Section 6.4) is expected due to the precipitation of sesquioxides out of solution from the groundwater on a decrease in moisture content. In order to maintain the same bulk density than the *in-situ* material (of lower SG), the remoulded specimens will

consequently need to be remoulded at a higher porosity in order to achieve the same bulk density, even at lower moisture contents.

- A change in the natural moisture content of the colluvium material will result in a direct change of the overall particle size distribution due to the aggregation and cementation of the particles, resulting in so-called “bi-modal” behaviour of the material. This increase in average grain size is expected to lead to an increase in the dimensions of the inter-particle void dimensions, acting as significantly better conduits for fluid transfer than the *in-situ* micro-structure.
- The combination of the increase in porosity and necessitated decrease in remoulding moisture content from the level of natural moisture content further directly results in a significant decrease in the initial degree of saturation of the remoulded specimens. By remoulding at lower degrees of saturation, any aggregates that will have developed during drying will be stiff and difficult to remould (Benson and Daniel, 1990) and clay particles flocculated (Lambe, 1958). This will add to a more permeable micro-structure of the remoulded samples.

9.7 Suction

A good correlation between the specific moisture contents obtained using the PPE apparatus and those determined using the filter paper method was obtained, as well as the behaviour of the matric suction pressures with increasing moisture content. The extent to which the shear strength of the colluvium material is influenced by the matric suction pressure is not investigated here. Considering the high natural moisture contents of the colluvium material in its *in-situ* state and the low matric suction pressures existing at these moisture contents, it is expected that the additional contribution of matric suction pressures to the shear strength of the colluvium material will be negligible.

9.8 Recommendations and Concluding Remarks

- It is clear that the need exists for a simplified, standardised and independent classification system to effectively classify tropical red soils. Numerous attempts have been made in the past to group and/or classify residual tropical soils, excluding any and all transported soils, as the residual soil structure is reported as to significantly influence the engineering behaviour of the material. Based on the available information from the literature and the results from this study, it however appears that the micro- and macro structure of the transported tropical red soils does not play a prominent role in governing the soil behaviour and that these soils exist in or very near to a naturally unstructured state. Therefore, by attempting to classify or categorize tropical red soils based on the effect of its structural development may not necessarily represent the ideal approach in developing a classification appropriate for these soils.

It is recommended that, and assumed for the purpose of this thesis, the term “laterite” is used solely to refer to those materials or soil horizons which either represent laterite formation in the form of hardened pedogenic horizons or contain distinct evidence of laterization in the form of nodules, concretions or distinct sesquioxide discolouration (i.e. mottling or staining). Tropical red soils as the focus of this study refer to all non-lateritic tropical soils other than tropical “black”

soils and which in reality may vary in colour (between shades of red, orange, yellow and brown), microstructure, mineralogy and grain size distribution.

Because the geotechnical properties and engineering behaviour of both residual and transported tropical soils appear to be heavily influenced by their mineralogical and chemical composition, the most appropriate approach to developing a practical classification system will have to involve establishing and evaluating these properties and getting a clear understanding of the influence of these on the geotechnical properties and engineering behaviour. With continuous infrastructure and mining developments in the tropical regions of the world, the need for detailed XRD and XRF analysis to be included as standard geotechnical analysis for mining development in the tropics is advocated in particular. Establishing and understanding the mineralogical and chemical characteristics of the soils will enable the engineer or geologist to specify or adapt geotechnical test procedures that are appropriate to the individual soils themselves and a correlation with the expected engineering behaviour can be established for a site-specific, localized application. It is believed that only after this approach has been adopted and applied for geotechnical investigations in these regions it will be possible to develop a practical classification system for tropical red soils.

- Based on the results of this study sealing of undisturbed samples using a combination of multiple protective layers of clingfilm and tinfoil provides suitable preservation of the original natural moisture content from the time of sampling to testing, in this case representing a time-span of approximately two months.
- Due to the observed sensitivity of the material to the type of dispersing agent used and subsequent effect on the hydrometer test results, it is recommended that prior to the scheduling of all particle size distribution analysis the most appropriate agent mixture is first identified for the particular material(s) on site. This selection should incorporate the XRD/XRF and chemical test results to establish the effect of variation in mineralogical/chemical properties over the study area (if any) on the suitability of a particular dispersant agent mixture. When considering the available chemical and XRD/XRF results and the variation in the mineralogical composition of the colluvium from the different parts of the study area, it is clear the Sodium Pyrophosphate Decahydrate dispersing agent mixture is suitable for the goethite- and kaolinite-rich colluvium material from the Mt. Lekoumou mining area. When using the Sodium Hexametaphosphate and Sodium Carbonate dispersing agent mixture to disperse the kaolinite- and quartz-rich colluvium from outside the mining area, incomplete dispersion was found to occur for a number of test specimens.
- Due to the sensitivity of tropical soils to change in its NMC and the apparent irreversible nature of the changes in material properties occurring once the NMC is altered, it is therefore recommended that testing be carried out at or as close as possible to the preserved natural moisture content of the material. The importance of the preservation of the natural moisture content should be emphasised in particular, as re-wetting and/or remoulding of the material to the original moisture content after drying of the material is unlikely to yield representative results due to the irreversible changes occurring on drying of the test specimen.

When determining the NMC of test specimens it is further recommended that, as is proposed throughout the literature for residual tropical red soils, extreme care is taken to not expose the test

specimen to the standard, more “aggressive” temperatures conventionally used during oven-drying. However, when test specimens are air-dried it should be considered that, particularly in those materials with higher CEC values, there may in fact remain some moisture in the specimen unable to be removed through air-drying alone. The moisture content even after prolonged periods of air-drying therefore is unlikely to represent zero moisture.

- It is proposed that, on the basis of the results of this study, when attempting to evaluate the potential activity of the tropical soils with a greater degree of certainty using indicator data alone, an alternative approach other than relating the plasticity index to overall clay content is followed. By instead considering the ratio between the liquid limit and plastic limit of the material, a much improved correlation between the potential activity and the actual linear shrinkage values was observed.
- The increase observed in SG values on drying appears to be irreversible and not influenced by the subsequent rewetting of the sample during soaking as part of the SG test procedure. It is proposed that additional future research is focused on confirming whether the observed increase in particle density is due to the process of precipitation suggested here, other variation in the material characteristics during drying or the standard test procedure.

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