Earthworks with Wet, Fine Grained Tropical Residual Soils

Master of Engineering Report

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ABSTRACT

Several dams and embankments have been constructed around the world with fine grained tropical residual soils, many with natural moisture contents slightly to well above the standard Proctor optimum moisture content. Nevertheless, the fills have generally been considered to perform well. Case histories of good performance from earthworks with wet fills are summarized and discussed. To aid in understanding the case histories, the terminology for describing and classifying residual soils is reviewed. The engineering properties of tropical residual soils are discussed, with an emphasis on differences from transported soils in temperate regions. Some reasons for the good performance of the wet fills are given in terms of these properties. As well, construction techniques and compaction control methods developed for working with overly wet soils are discussed.

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1. INTRODUCTION

Fine grained tropical residual soils have been used successfully in dams and embankments around the world, often with natural moisture contents greater than the standard Proctor optimum.

A review of the literature on residual soils in general and case histories of earthworks has been undertaken. The terminology associated with residual soils and their classification is presented, and followed with a review of their engineering properties. Case histories are presented, and the unusual aspects of construction with these soils relative to temperate soils are discussed.

As development has proceeded in tropical countries civil engineering works have been undertaken and classical soil mechanics have been modified based on local experience. Although some of the properties of tropical residual soils may be unknown to practitioners of soil mechanics in temperate regions, considerable expertise has been developed and documented in the technical literature over the last 30 years. According to Mitchell and Sitar (1982) the "volume of written material is so great that careful review and assimilation of all of it is a task of overwhelming magnitude". A bibliography arranged by subject is included in Appendix 1.

2. DESCRIPTION OF RESIDUAL SOILS

2.1 Definition of Residual Soil, Laterite, Saprolite, and Structure

A residual soil can be described as a soil-like material derived from the weathering and decomposition of rock which as not been transported from its original location (Blight, 1997). In this usage, residual soil is a broad term and includes saprolites and laterites. This general definition of a residual soil is used in this report. The term residual soil is sometimes used in a narrower sense to describe the mature soil, that part of the soil profile which has undergone physical and chemical weathering to the extent that no evidence of the parent rock's fabric or structures remain. The different horizons in a typical residual soil profile are further discussed in Section 2.3.

Saprolites are materials that have soil-like strength or consistency, but retain recognizable relics of the structure and fabric of the parent rock (Blight, 1997). As an example, a saprolite derived from a lava may contain flow bands, amygdules, and joints.

Fookes (1997) uses the following definitions in discussing the texture of residual soils:

- Structure – the fabric, texture and discontinuity patterns making up the soil-rock material, mass or unit;

- Fabric – the spatial arrangement of component particles;
- Discontinuities – the nature and distribution of surfaces separating elements of fabric, material or soil-rock mass.

Laterites are highly altered residual soils that have had the silica leached out and have some degree of cementation by sesquioxides (Blight, 1997). However, the term laterite is used very loosely and is sometimes applied to soils with little influence from sesquioxides. This cementation may give these soils a granular or nodular appearance. Laterization may occur in ancient transported soils as well as residual soils. As the concentration of sesquioxides increases the description of the soil progresses from lateritic to the soil being called a laterite. Because of the nature of their formation, laterites tend to occur near the surface and extend to limited depths. The progression with chemical weathering of a soil from saprolite to mature soil, to laterite will only occur under conditions conducive to laterization. Laterites are often excellent construction materials and may be a source of aggregate.

The description of a soil as a residual soil without including the parent rock is meaningless according to some researchers (De Mello, 1972). However, apart from generalizations it is difficult to relate the properties of a residual soil directly to the parent rock because of the superposition of effects from climate, topography, geologic age, and structure. For example, weathered granite from the warm, humid Malaysian peninsular has quite different properties from weathered granite from cooler, semi-arid South Africa (Blight, 1997).

2.2 Structure

The structure of tropical residual soils includes relict discontinuities found in saprolites which are derived from the parent rock, and microstructure or fabric.

Microstructure in residual soil includes the aggregation of clay particles, such as described by Terzaghi (1958), and other types of bonding and particle orientation as described by Filho et al (1989). The aggregation of clay particles may be from cementation by iron and aluminum oxides and may take many forms (Mitchell and Sitar, 1982). The simultaneous leaching and precipitation action may result in a highly porous soil structure.

Although microstructure is important to understanding the engineering behaviour of soils, particularly partially remolded soils such as compacted fills, the behaviour of the in situ soil mass is frequently more influenced by macroscopic features. Macroscopic features include structural orientation such as schistocity, as well as fissures, veins, joints, faults, and voids.

2.3 MINERALOGY

The mineralogical composition of residual soils is dependent on the composition of the parent rock and the climatic conditions. The mineralogy of tropical soils has engineering significance in the aggregation and cementation of soils, as well as in affecting index properties such as moisture content, and possibly even affecting field instruments such as nuclear densometers. Figure 1 shows that, in general, clay mineralogy varies in a predictable way with distance from the equator.

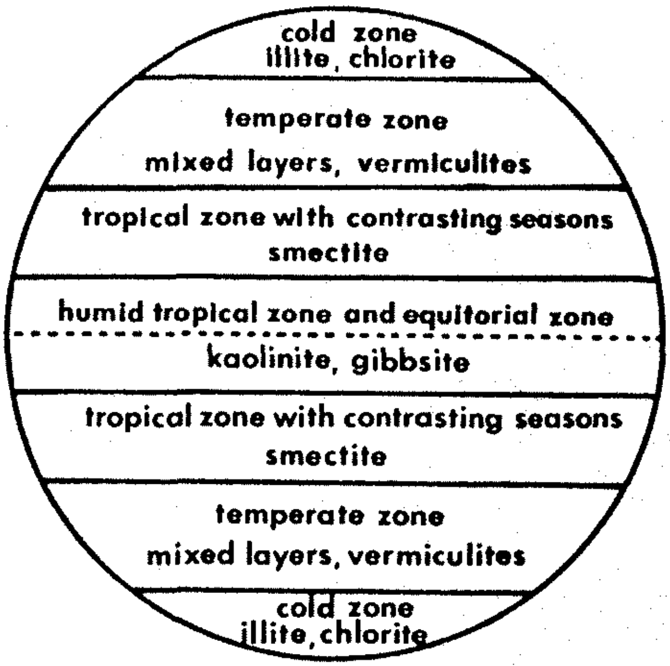


Figure 1 - The effect of climate on frequency of clay mineral occurrence with climate zones represented in a simplified manner as distance from the equator (after Millot, 1979; from Uehara, 1982).

Crystalline rock, such as igneous dykes, which cooled relatively slowly during their formation and are typically high in feldspars (alumino-silicates). In the tropics the feldspars weather initially to kaolinite, and hydrated iron and aluminum oxides, such as goethite ($Fe_2O_3 \cdot H_2O$) and gibbsite ($Al_2O_3 \cdot 3H_2O$), also referred to as sesquioxides. Other minerals which are more resistant to weathering, such as quartz (SiO_4) and mica ($KAl_3Si_3O_{10}(OH)_2$ - muscovite) may persist, often as individual sand grains in a clayey matrix. With further weathering the kaolinite content may decrease and the sesquioxides progressively alter to hematite (Fe_2O_3) and boehmite ($Al_2O_3 \cdot H_2O$) (Mitchell and Sitar, 1982). These soils are described as laterites and their high iron content usually gives them a reddish colour. Further chemical action may cement these materials and produce lateritic gravels.

The tropical weathering of volcanic rock and ash results in the formation of the clay minerals allophane and halloysite as well as concentrating iron and aluminum oxides (Mitchell and Sitar, 1982). Initially, the halloysite is found in a hydrated form which is characterized by a tubular morphology. Hydrated halloysite was first described by Terzaghi (1958), and is responsible for unusual soil properties, which are described later. The water layer present in halloysite is removed irreversibly by heating or air drying, leading to the formation of metahalloysite.

The notorious highly plastic 'black cotton' soils develop near the edge of the tropics in areas of decreased rainfall, distinct wet and dry seasons, and poor drainage. These soils are composed of high activity smectite clays. These soils are seldom considered for fill in earthworks.

2.4 Organic Matter

Tropical regions are generally covered in lush vegetation. However, the organic matter decomposes rapidly and is rarely incorporated below a thin surface layer.

2.5 Soil Classification

The first venture into the tropical soils literature can be confusing due to the breadth of the topic. The large number of soils described as tropical or residual soils requires some classification in order to organize their properties. A number of these classifications are discussed below.

Unfortunately, within much of the literature soils are not described beyond the Unified Soils Classification. This lack of a large database to draw on limits the usefulness of any classification system, and begets the engineer to carefully consider if the time invested in learning any system is warranted. However, in discussing the categories of the systems one is exposed to many different properties of tropical soils and this may help develop understanding of the nature of these soils, even if some of the divisions are of limited importance in engineering applications. A grouping of soils by comparisons with temperate soils proposed by Blight (1997), is also presented and may help organize thoughts about these soils.

2.5.1 Soil science classification

Soil sciences as developed for agricultural purposes routinely include pedological classifications in soil mapping. These classifications were developed for agricultural purposes but soil maps using this taxonomy can be used to deduce some engineering properties and are widely available. Three pedological classification systems are commonly used: the French classification, U.S. New Soil Taxonomy, and the FAO-UNESCO Classification.

A discussion of how the U.S. Soil Taxonomy can be used by engineers is presented by Uehara (1982). Some of the properties of the categories within this classification are summarized in Table 1. Figure 2 shows how soils classified using this system plot on the plasticity chart. Figure 3 shows similar information for soils classified according to the French classification.

Table 1 - Properties of soils classified according to the U.S. Soil Taxonomy system.

Classification (Order)	**Distribution**	**Minerals Present**	**Properties**
Oxisols, Ultisols, Alfisols	Humid tropics	Low activity clays: • Kaolin • Sesquioxides	• Less plastic • Tend to form aggregates • Less likely to shrink or swell
Andisols	Volcanically active areas	• Halloysite • Allophane • Imogolite	• Low dry density
Vertisols, Mollisols	Semiarid tropics	High activity smectites	• High plasticity • Shrinking / swelling
Histosol	Bogs	>30% organic matter	

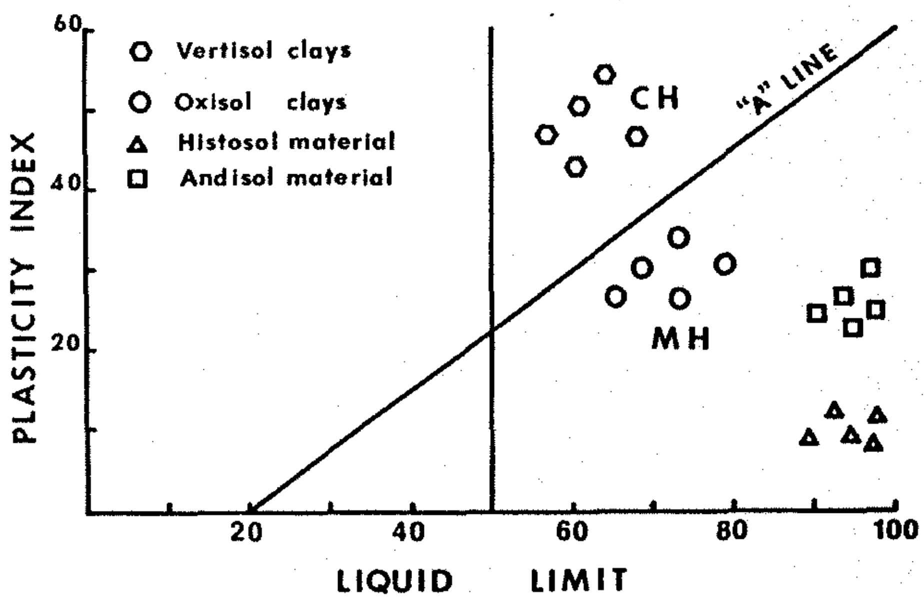


Figure 2- Plasticity chart showing the location of four different orders from the U.S. Soil Taxonomy (from Uehara, 1982).

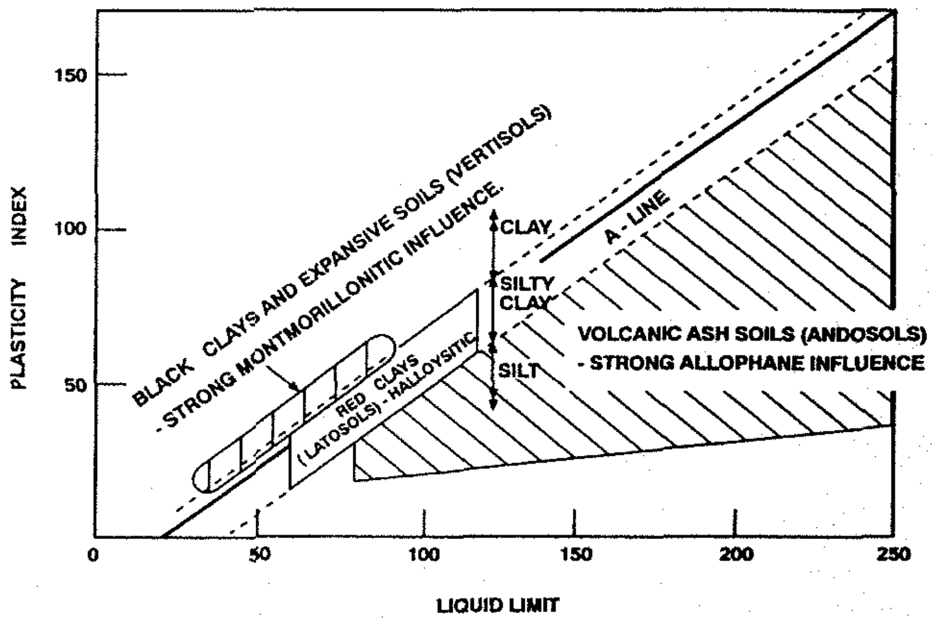


Figure 3 - Influence of mineralogical composition and soil classification with the French classification on position on the plasticity chart (from Blight, 1997).

Fookes (1997) uses the French classification (Duchaufour, 1982), as this system highlights the compositional soil characteristics more than the other systems that were developed for agricultural uses and use more subjective criteria. Table 2 shows the relationship of the major categories for the major classification systems. Morin and Todor (1975) discuss in greater detail how these systems correlate. Figure 4 shows the distribution of tropical residual soils around the world.

Table 2 - Approximate equivalents of various major classes of tropical residual soils (adapted from Fookes, 1997).

French (1982)	FAO-UNESCO (1988)	USA (1975, 1992)
Fersiallitic soils	Cambisols, calcisols, luvisols, alisols	Alfisols, inceptisols
Andosols	Andosols	Andisols, inceptisols
Ferruginous soils	Luvisols, alisols, lixisols, plinthosols	Alfisols, ultisols
Ferrisols	Nitisols, acrisols, lixisols, luvisols, plinthosols	Ultisols, oxisols
Ferrallitic soils	Ferralsols, plinthosols	Oxisols
Vertisols	Vertisols	Vertisols
Podzols	Podzols	Spodosols

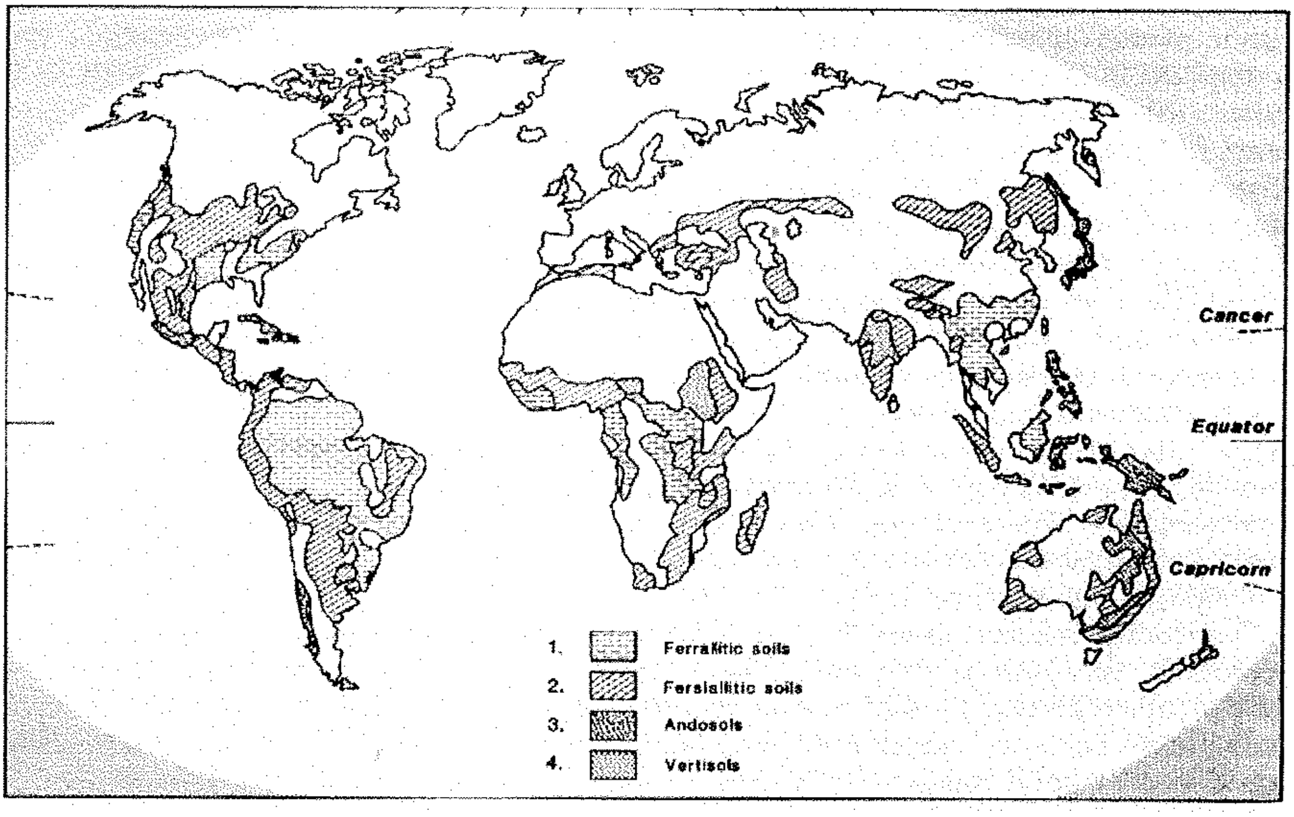


Figure 4 - Simplified world distribution of the principal types of tropical residual soils (from Fookes, 1997; based on F.A.O. World Soil Map). Broad classes of soils can be seen to extend beyond the tropics in favourable circumstances. Areas in the tropics shown blank include those where tropical residual soils are overlain by recent alluvial and aeolian deposits.

2.5.2 Grouping of soils by engineering properties relative to temperate soils

Blight (1997) describes a systematic overall grouping of tropical soils that is based on work done by Wesley (1988). The first level of grouping is based on mineralogical composition, primarily of the fines fraction. These groups are then subdivided according to the effects of structure, both on a macroscopic and microscopic scale. The three groups based on mineralogical composition are:

- Group A – soils without a strong mineralogical influence;
- Group B – Soils with a strong mineralogical influence deriving from clay minerals also commonly found in transported soils;
- Group C – Soils with a strong mineralogical influence deriving from clay minerals only found in residual soils.

Group A encompasses many soils, including most of the saprolite soils. Group A is further subdivided into those soils which have a strong macrostructure influence, those with a strong microstructure influence, and those with little structural influence. The behaviour of the soils with macrostructure control are best analyzed by using a combination of soil mechanics and rock mechanics. It is important to understand the bonding and likelihood of breakdown for the soils with microstructure control.

Group B is mainly composed of soils whose behaviour is controlled by the presence of smectite clays. This group includes the black cotton soils, which are dark coloured, highly plastic clays which form in poorly drained areas.

Group C soils is further divided into three subgroups depending on the minerals present. The first subgroup includes soils whose behaviour is dominated by the presence of allophane, which is typical of soils derived from volcanic ash. The second subgroup is based on the influence of the mineral halloysite. These soils tend to be derived from older volcanic rocks. The third subgroup includes soils whose behaviour is controlled by sesquioxides, such as lateritic soils. The plasticity typically varies from low to non plastic.

This classification system is good to keep in mind when comparing case histories. A difficulty with this system is the need to identify mineralogy, but this can done with laboratory testing, from interpreting agricultural pedological mapping, and from interpreting appearance and pl

3. WEATHERING CLASSIFICATION

The engineering properties of a soil will vary with depth even if it has developed from a uniform parent material. Near the surface, soil layers are affected by humus and seasonal wetting and drying. With depth the moisture content fluctuates less with the seasons and less organic matter is present. Also, with depth the groundwater movement is often slower and soil particles and solutes are less likely to be transported.

The nature of the contacts between soil horizons depends to some extent on the parent rock. Saprolites from ultramafic rocks tend to have a sharper transition to soil horizons. Profiles originating from quartzofeldspathic rocks tend to have a massive zone rich in illuvial clay and lacking the visible inherited rock fabric of the underlying saprolite (Fookes, 1997).

3.1 Pedological Based Weathering Classification

Typical weathering profiles for metamorphic and intrusive igneous rocks are shown in Figure 5. In Figure 5 these profiles are classified according to the weathering classification system proposed by Deere and Patton (1971). In this classification system the profile is divided into three zones:

- Zone I – Residual soil, which includes the saprolite horizon;
- Zone II – Weathered rock;
- Zone III – Unweathered rock.

Zone I is further divided using pedological terms. The A horizon is illuviated or depleted of certain minerals, and the B horizon is enriched by deposition of the minerals from the A horizon. The C horizon is in place soil and is unmodified except by weathering.

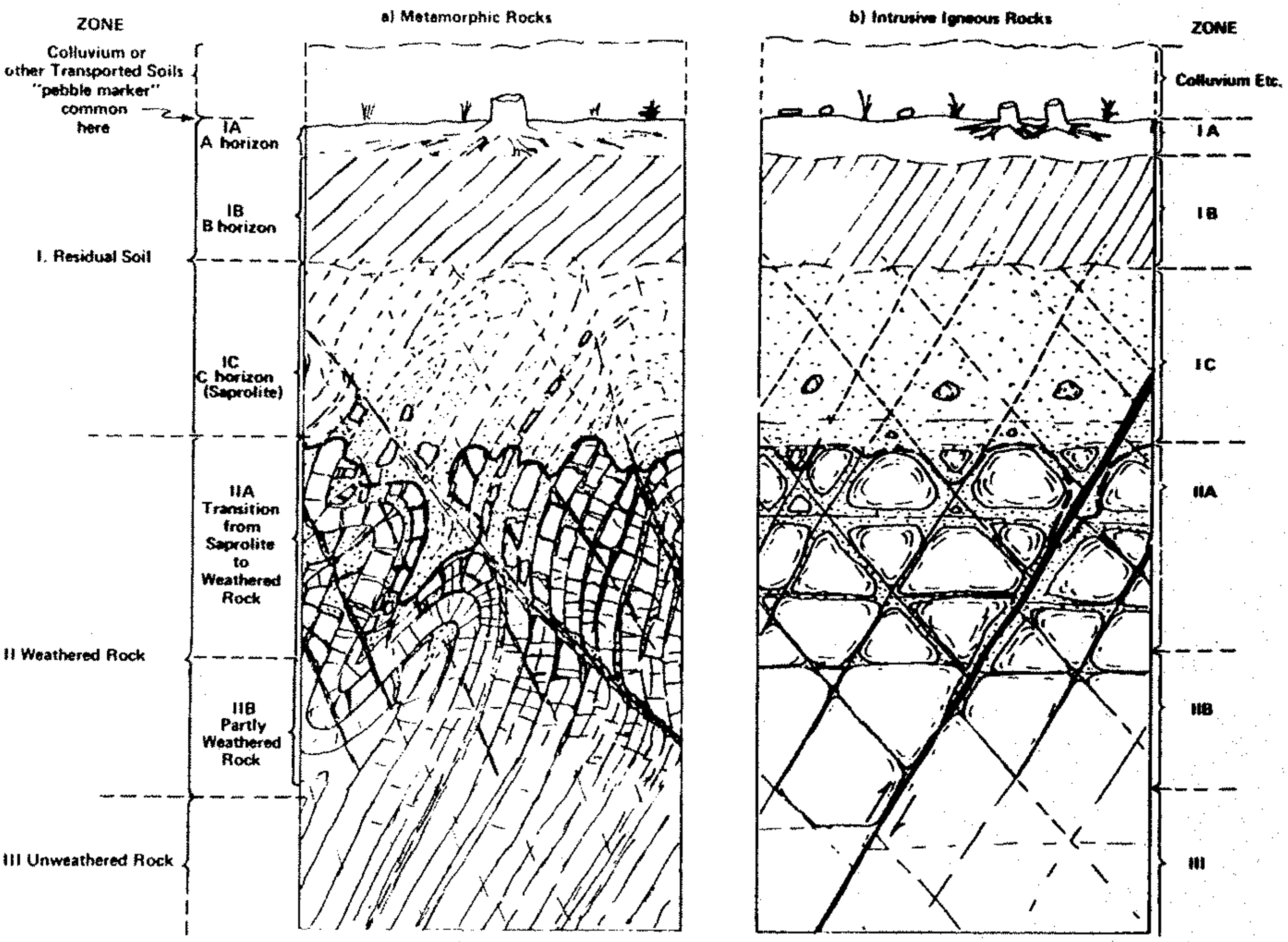


Figure 5 - Weathering classification system proposed by Deere and Patton (1971) with typical weathering profiles for metamorphic and intrusive igneous rocks (from Deere and Patton, 1971).

De Mello (1972) has proposed a reorganization of the system proposed by Deere and Patton (1971) based on the engineering significance of the different horizons. De Mello's proposed modification is shown with the typical weathering profile for intrusive igneous rock in Figure 6. This modification would dived Zone I and Zone II into mature soil, residual soil, and weathered rock. The mature soil encompasses overlying colluvium and the IA and IB horizons. The residual soil includes the saprolite and transition from saprolite to weathered rock. The partly weathered rock and unweathered rock are grouped together. Conventional soil mechanics is suggested to be applicable to the mature soil, rock mechanics to the weathered and unweathered rock, and a combination of these two disciplines to the residual soil.

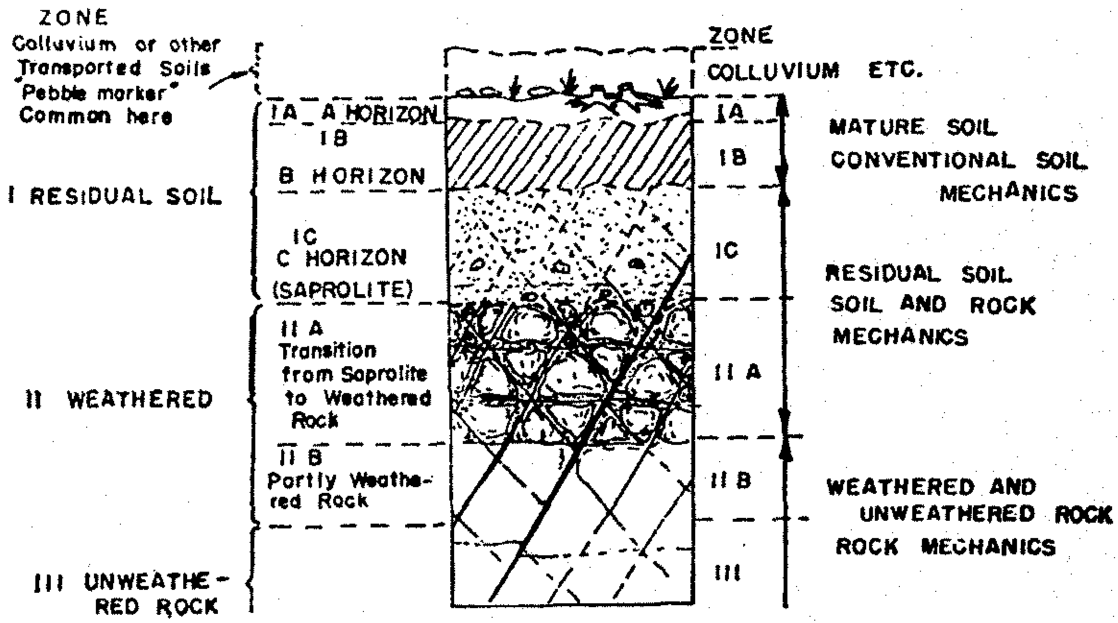


Figure 6 - Typical weathering profile from Deere and Patton (1971) with regrouping of profile into: mature soil, residual soil, and weathered and unweathered rock (from de Mello, 1972).

3.2 Classification based on Degree of Weathering

Classification systems based on the degree of weathering are readily applied, easily understood, and based on hardness which is significant for many engineering applications. Blight (1997) describes such a classification based on work done by Little (1969), shown in Figure 7.

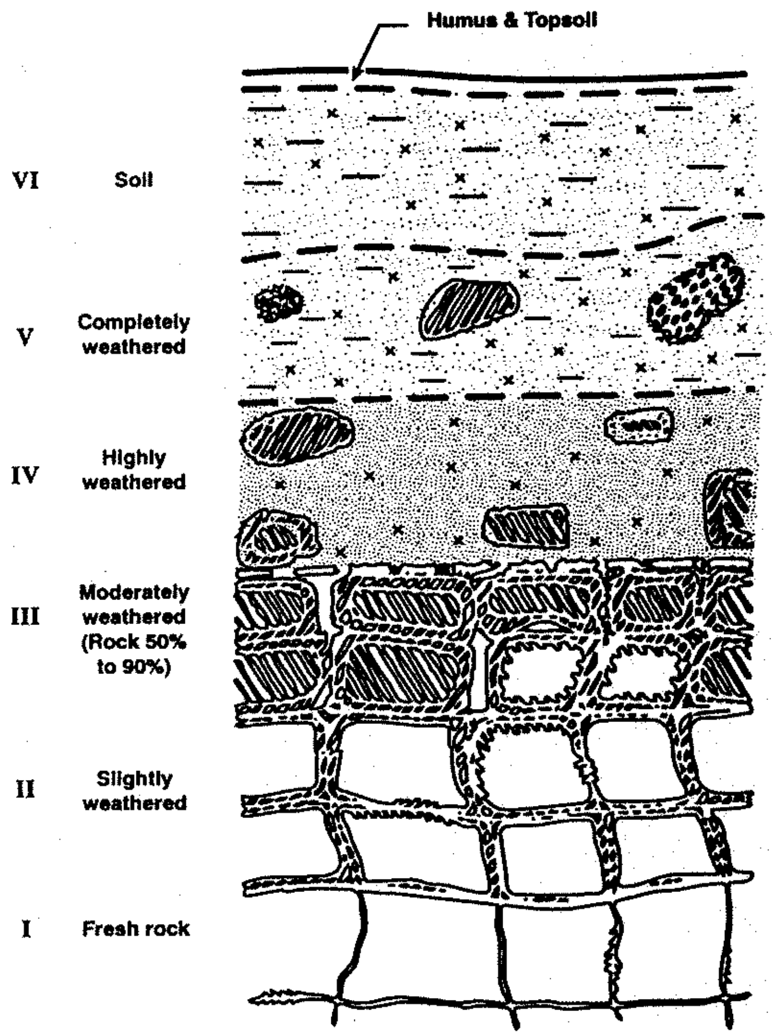


Figure 7 - Classification of a weathered rock mass profile (after Little, 1969; from Blight, 1997).

A very similar classification scheme favoured by Fookes (1997) is detailed in Table 3 and shown applied to a typical weathering profile in Figure 8.

Table 3 - Scale of weathering grades of rock mass (adapted from Anon, 1977, and Fookes, 1997).

Grade	Term	Description
I	Fresh	No visible sign of rock material weathering; perhaps a slight discoloration on major discontinuity surfaces.
II	Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering.
III	Moderately weathered	Less than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.
IV	Highly weathered	More than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.
V	Completely weathered	All rock material is decomposed and or disintegrated to soil. The original mass structure is still largely intact.
VI	Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

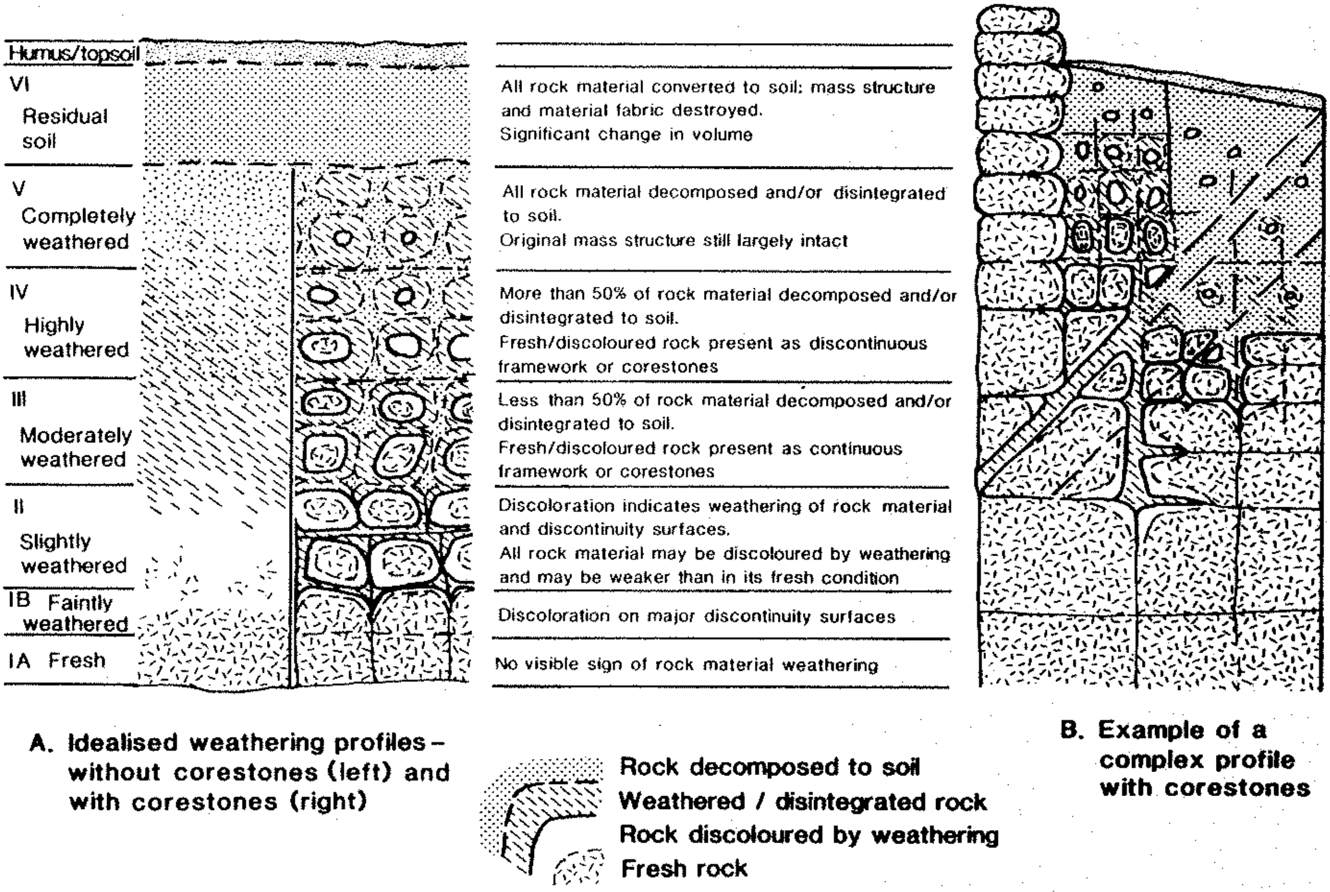


Figure 8 - Examples of idealized weathering profiles (from Fookes, 1997).

Engineering properties can be determined in a general way from the weathering profile. Figure 9 illustrates this by showing in schematic form how engineering properties vary through a typical weathering profile.

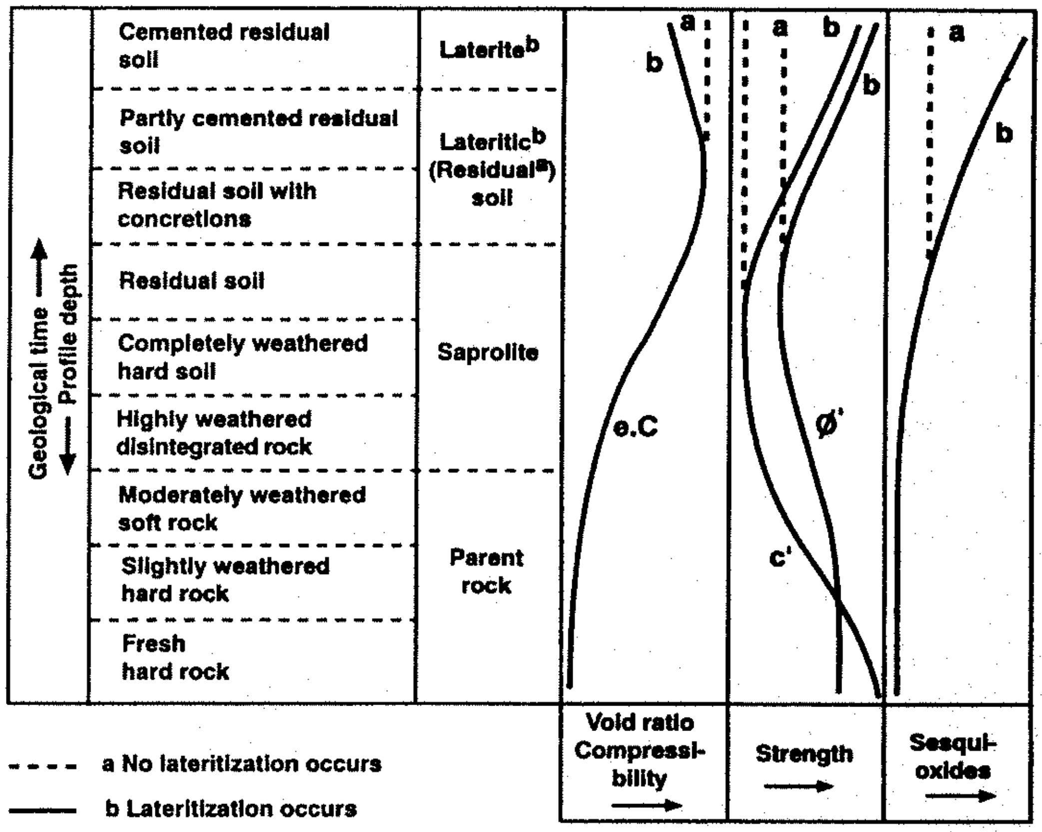


Figure 9 - Changes occurring with depth in a weathering profile. (adapted from Tuncer and Lohnes, 1977, and Sueoka, 1988; from Blight, 1997).

4. RESIDUAL SOIL PROPERTIES

The engineering characteristics of tropical residual soils are influenced by many different factors, including sample preparation. It is difficult to compare the results obtained by various investigators or to cite specific value ranges for different properties, especially if the methods of sample preparation are not known (Mitchell and Sitar, 1982). Further difficulty in comparison arises from small variations in the usage of terms such as laterite, lateritic, and residual soil. Thus the intent has been to provide a general review of residual soil properties which relate to the compaction behaviour rather than to catalogue typical values.

4.1 Cementation / Aggregation

In residual soils it is common to have the cementation of particles into clusters or aggregates by sesquioxides and the hydrated state of some of the minerals. This cementation or aggregation is responsible for high void ratios (low densities), high strength, low compressibility, and sometimes high permeability in relation to the high plasticity and small particle size that would be anticipated on the basis of the content of clay size particles of crystalline clay minerals, allophane, and oxides (Mitchell and Sitar, 1982).

For cohesive soils a position below the A line on the plasticity chart is an indication of aggregation. This evidence is particularly compelling when the soil is found to be mostly composed of clay minerals such as at Sasumua dam in Kenya and Tjipanoendjang dam in Java (Terzaghi, 1958).

In a discussion of the findings at Sasumua dam (Terzaghi, 1958) Evans contributed information about soils from Tasmania. Evans noted that the liquid limit of residual soils derived from dolerite talus was 10 to 20% lower than the liquid limit of residual soils from dolerite bedrock. He presumed that the mineralogy of each soil was similar and that the difference in engineering properties was due to a more advanced soil breakdown in the talus derived soils. Evans suggests that the aggregation described by Terzaghi (1958) may be due to the cementing action of iron oxide, and accentuated rather than caused by the presence of halloysite.

In recognition of the prevalence of cementation of clay minerals, some researchers in Brazil consider that the conventional characterization tests, such as grain size and plasticity limits, are an indication of the "clay-potentiality" for saprolites and not truly the clay content (De Mello, 1975).

4.2 Grain Size

In the Brazilian experience (Mori, 1979) the three main types of bedrock commonly found consistently weather to different grain sizes. The rock types and residual soil's grain sizes are:

- basalt – clayey;
- gneiss – silty, often micaceous;
- granite – sandy.

Typical grain size distributions of saprolites derived from different parent rocks in Brazil are shown in Figure 10.

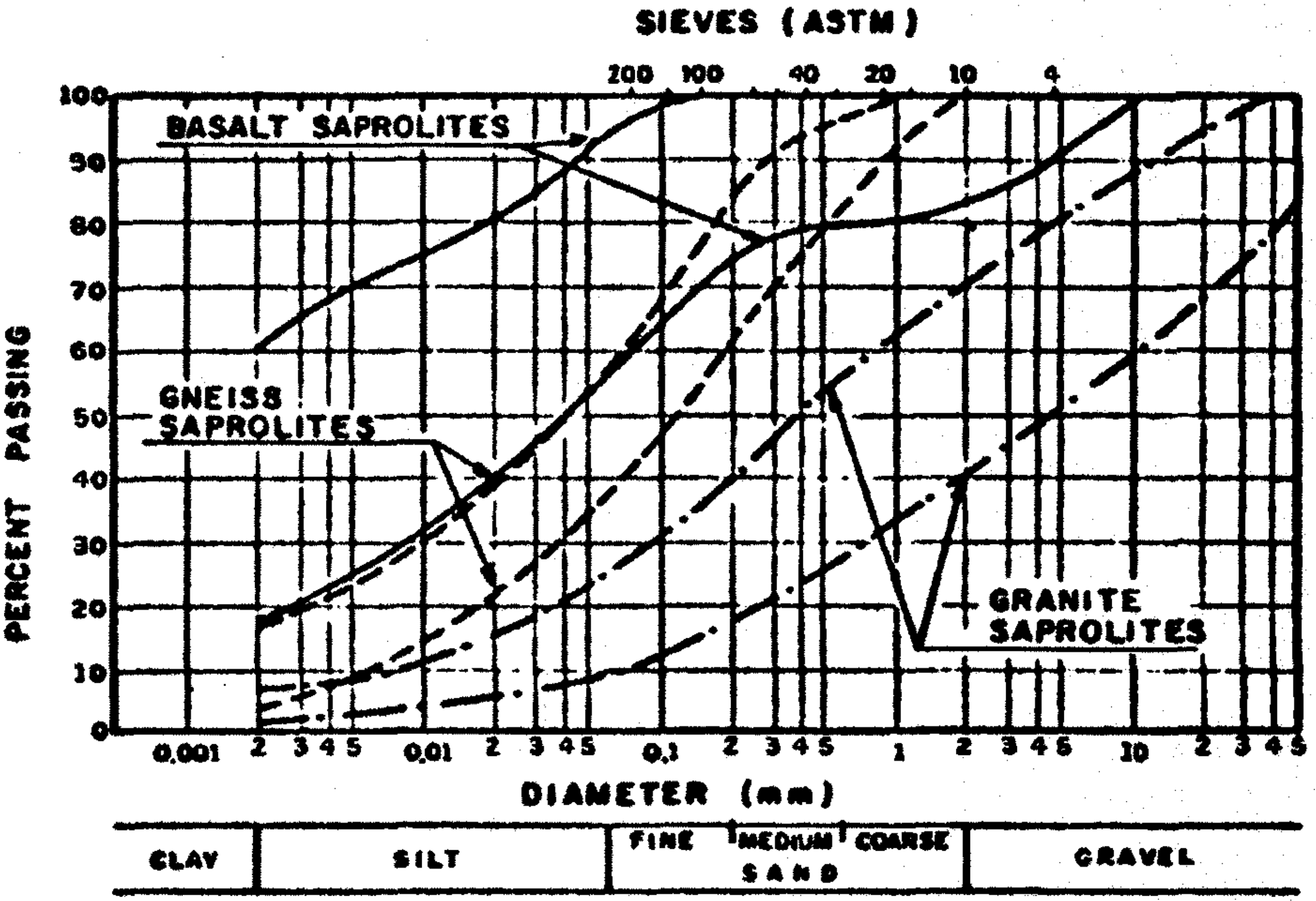


Figure 10 - Typical grain size distributions for saprolites from Brazil (from Mori et al, 1979).

The breakdown of soil particles with manipulation leads to ambiguity in determining the grain size distribution. The affect of remolding on the measured grain size of a soil is shown in Figure 11. In the soil with the sesquioxides removed, the change in grain size with remolding is less dramatic, which demonstrates that the sesquioxides were acting as cementing agents.

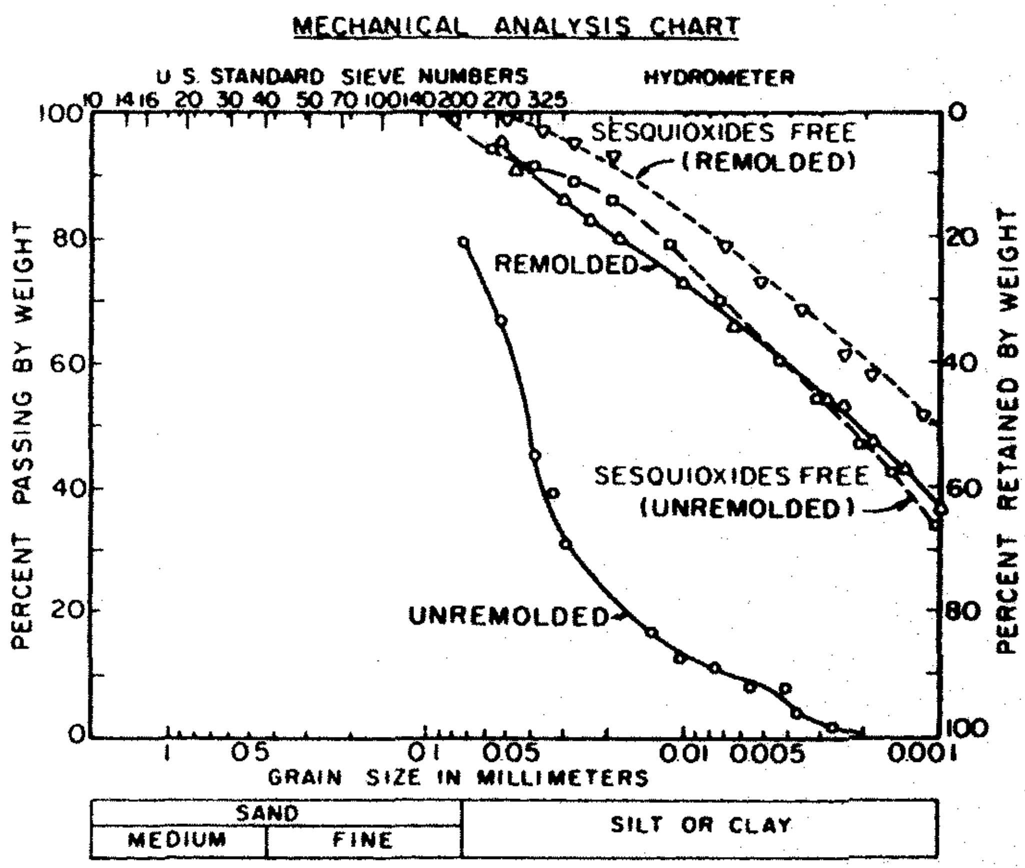


Figure 11 - Effect of remolding and sesquioxide removal on the grain size of lateritic soil (from Townsend et al, 1971; see Mitchell and Sitar, 1982)

4.3 Moisture Content

Drying of a soil may change its behaviour because of:

- cementation – by the sesquioxides;
- loss of water from hydrated minerals, changing their structure; or
- a combination of the above.

The changes in behaviour that result make it difficult to obtain reproducible results in classification tests. Many of the relationships developed for temperate zone soils do not seem to apply.

The loss of moisture from hydrated clays implies that moisture content determination in fills may be dependent on the drying procedure. Higher temperatures may drive off water that would be locked within the mineral structure at lower temperatures. The effects of drying temperature on the measured moisture content for some residual soils are shown in Figure 12. The effect is most pronounced for soils containing halloysite and allophane (Terzaghi, 1958).

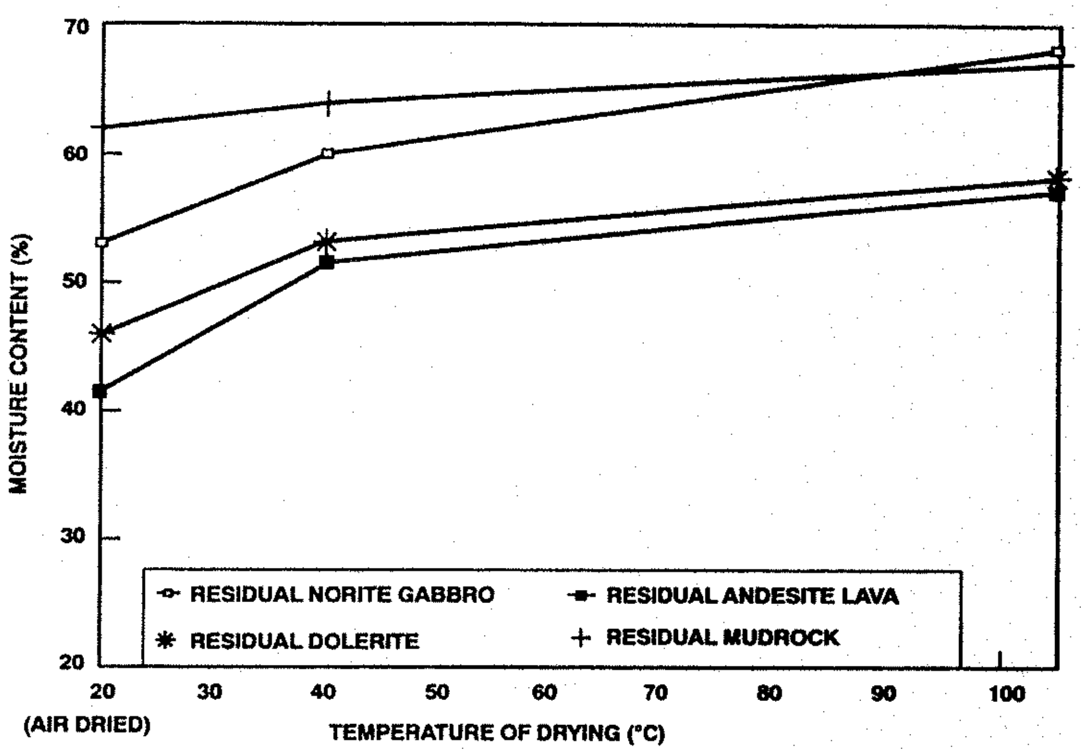


Figure 12 - Effect of drying temperature on apparent moisture content of four residual clays (from Blight, 1997).

4.4 ATTERBERG LIMITS

4.4.1 Typical Values

The position of some residual soils on the Unified Soils Classification has been discussed in Section 2.5 and shown in Figure 2 and Figure 3. Mitchell and Sitar (1982) have produced a broad grouping of residual soils on the Unified Soils plasticity chart, shown in Figure 13. However, the plasticity properties alone are not enough to uniquely identify the parent rock or genetic climatic conditions.

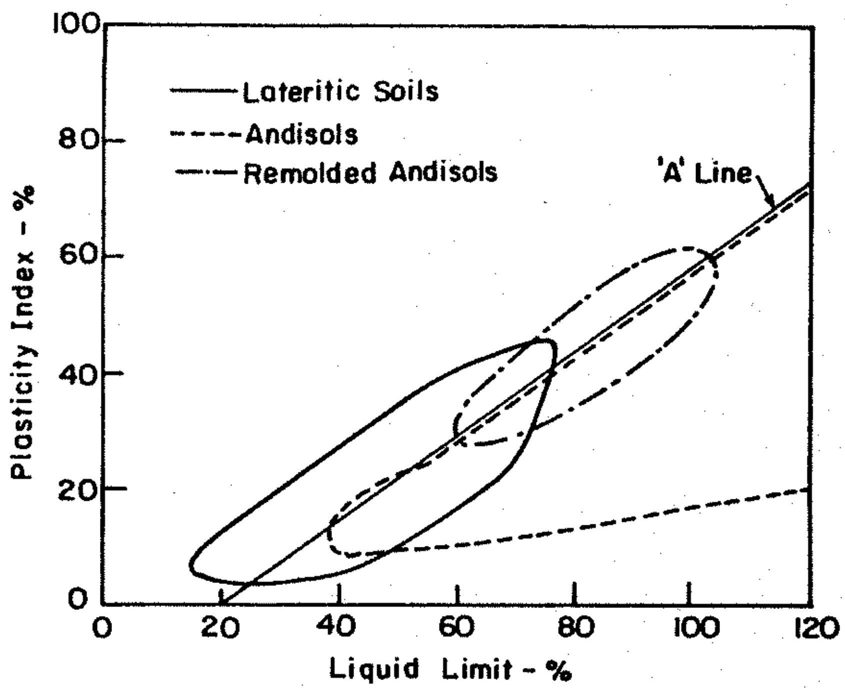


Figure 13 - Approximate ranges of Atterberg Limits for lateritic soils and andisols (from Mitchell and Sitar, 1982, compiled from many sources).

4.4.2 Effect of Particle Breakdown

The breakdown of the particles of many tropical soils with manipulation leads to ambiguity in determining the Atterberg Limits. The amount of mixing of a sample has been shown to influence the liquid limit but have limited impact on the plastic limit, as shown in Figure 14. Blight (1997) agrees that the greater the duration of mixing the larger the resulting liquid limit, and that the plasticity index is affected to a lesser extent.

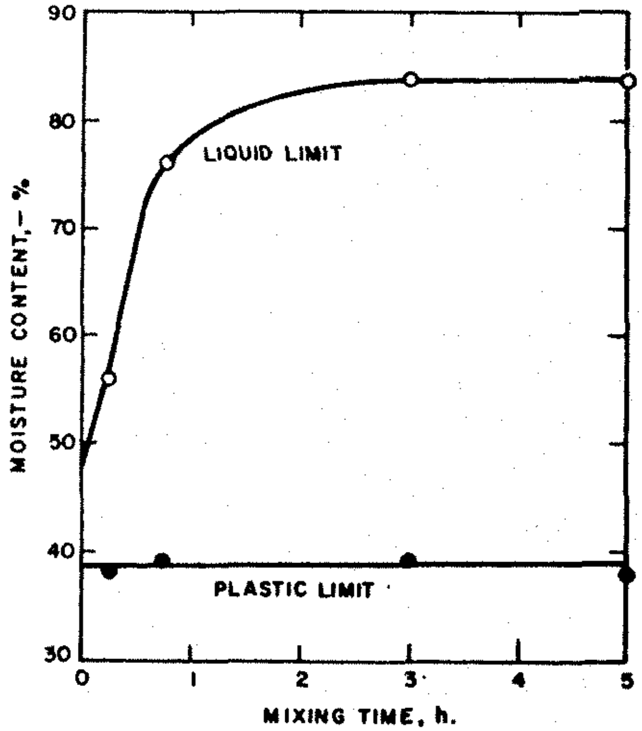


Figure 14 - Effect of sample mixing time on the liquid limit of a red clay from Kenya (after Millard, 1962; from Mitchell and Sitar, 1982)

4.4.3 Effect of Drying

The drying of a soil when measuring the Atterberg Limits may affect the results because of cementation from the sesquioxides and / or a loss of water from hydrated minerals.

Figures 15 and 16 show the affect of drying on the plasticity of residual volcanic and basaltic soils. It can be seen in this figure that drying these soils tends to reduce the liquid limit.

LEGEND

- ● Tested without air-drying
- X tested after drying (same soil)
- A Andosol
- L Other red soils
- ? Not known

NOTES;

New Guinea data from Wallace (1973)

Java data - from Wesley (1973, 1974)

NP = Non plastic

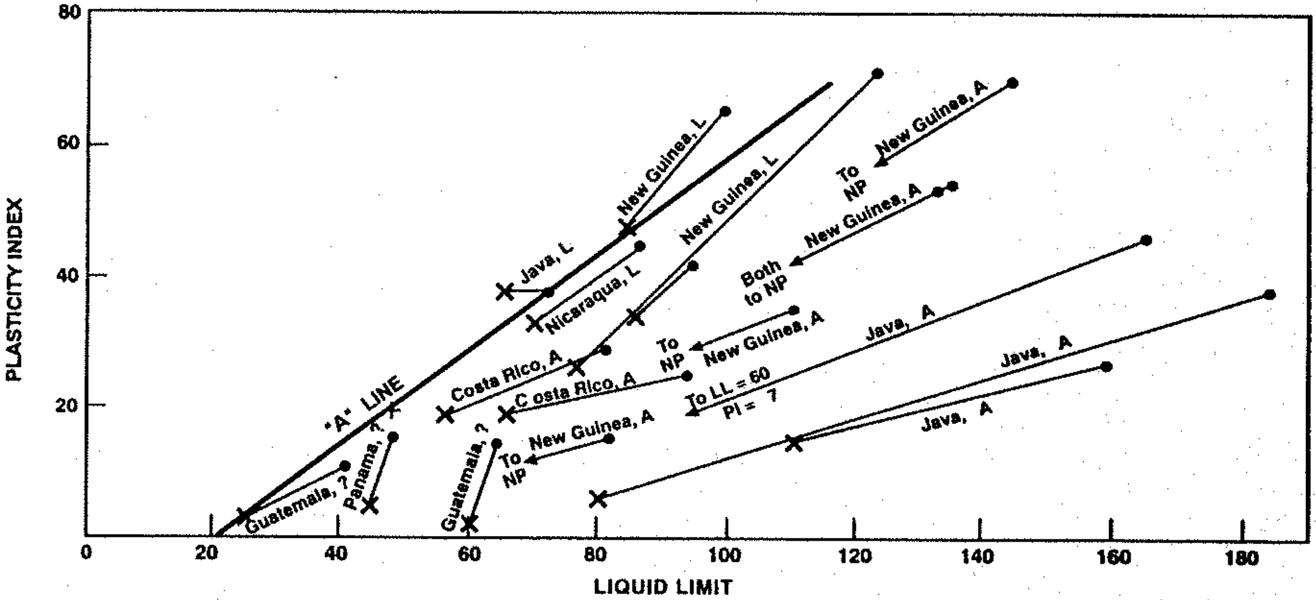


Figure 15 - Effect of drying on the plasticity of volcanic soils (after Morin and Todor, 1975; from Blight, 1997)

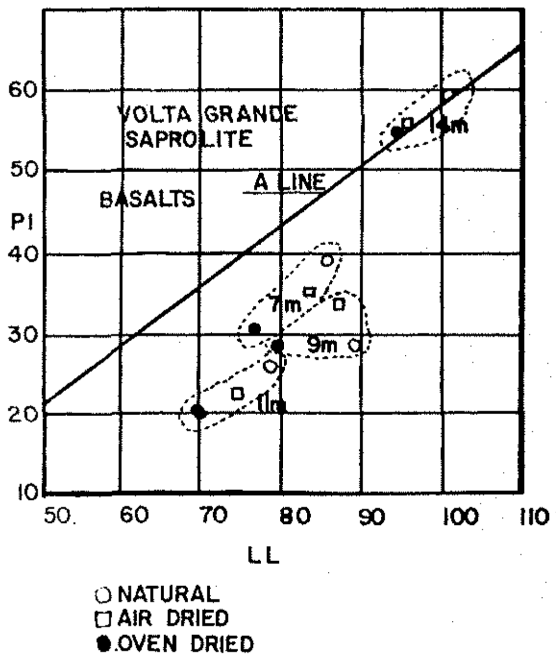


Figure 16 - Effect of drying method on the plasticity values of a basalt saprolite from Brazil (from de Mello, 1975).

According to Mitchell and Sitar (1982), use of classification tests is probably most important for identifying the soils which change their properties as a result of drying and remolding.

4.5 Shear Strength Parameters (for Compacted Material)

For lateritic residual soils and andisols, Mitchell and Sitar (1982) report that for compacted samples the majority of the friction angles range from $\phi' = 28°$ to $38°$, and cohesion values range from c' = 0 to 48 kPa.

Villegas and Mejia (1976) reported that for in situ residual quartz diorite soils in Colombia that the shear strength did not increase markedly with depth until near the top of the original rock.

For saprolites in Brazil, it appears that for shear strength and deformation the behaviour has been much better than would be expected from conventional soil mechanics correlations (de Mello, 1975).

4.6 Compressibility

Mori et al (1979) reported that for a variety of compacted saprolites from Brazil the Young's Modulus, E, for a confining pressure of 1 kg/cm^2 varied from 100 kg/cm^2 to 200 kg/cm^2 in a range of water contents above the optimum. The strength and compressibility of compacted saprolites was found to be closely related to the percentage of soft rock particles that remain intact in the fine soil matrix.

For Brazilian saprolites the compaction is manifested in the oedometer and triaxial tests as a preconsolidation pressure. A 96% compaction relative to standard Proctor maximum corresponds to approximately 400 kPa of preconsolidation pressure (de Mello, 1975). A compaction of 90% corresponds to about 150 kPa, and 102% compaction corresponds to about 550 kPa of preconsolidation pressure.

4.7 COMPACTION

4.7.1 General

The compaction characteristics of residual soils vary over a wide range. The factors which influence the moisture content / dry density relationship include: gradation, susceptibility to breakdown, method of pretreatment or sample preparation, mineral composition, and compactive effort.

Figure 17 illustrates the compaction curves of a wide range of residual soils subjected to the same method of compaction and compactive effort (Blight, 1997). The origin of the soil samples and the compaction method were not specified.

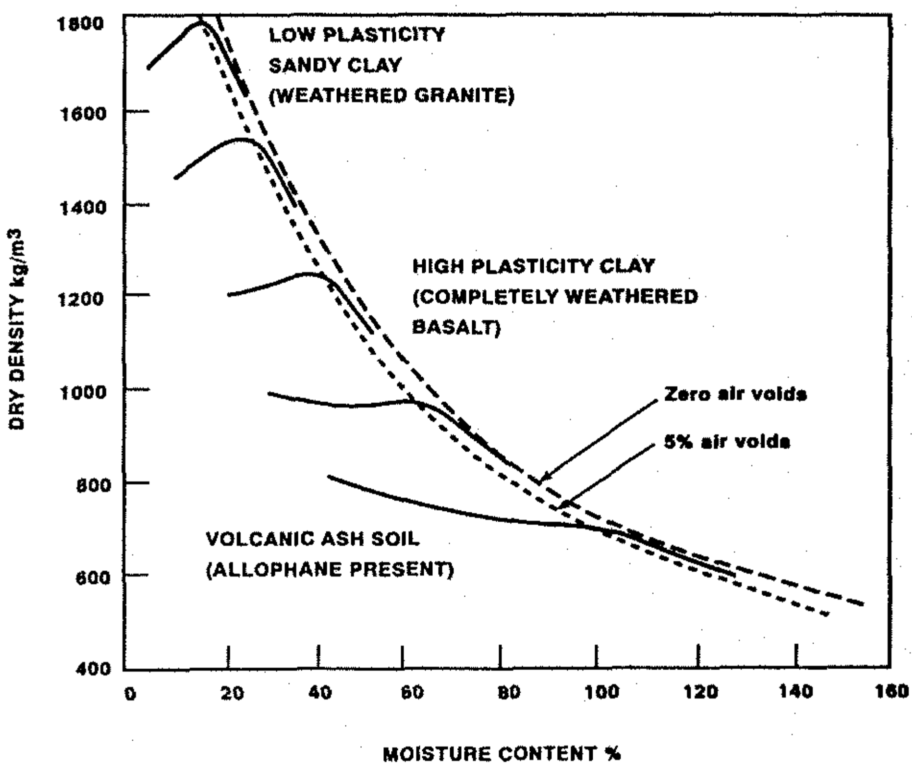


Figure 17 - Typical compaction curves for residual soils showing optimum characteristic (from Blight, 1997).

The andisols tend to have lower dry densities and higher optimum moisture contents than other residual and temperate soils. These soils also tend to have lower densities in situ and higher natural moisture contents.

4.7.2 Effect of sample preparation methods

The effects of drying samples on Atterberg limit and moisture content testing results has been described previously, particularly for soils with a high content of halloysite and allophane minerals. The effects of drying on the compaction properties of similar soils are shown in Figure 18 and Figure 19. These figures show that the optimum moisture content tends to be lower for soils that have dried before testing. Oven dried soils show an even greater reduction in the optimum moisture content than air dried soils.

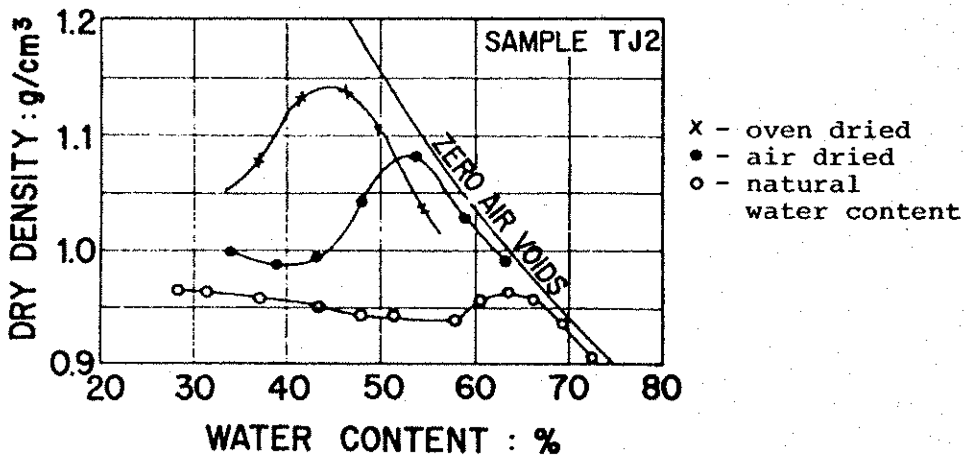


Figure 18 - Compaction curves of andosols obtained from Tjipanundjang Dam Site, Indonesia (after Wesley, 1973; from Mori, 1982)

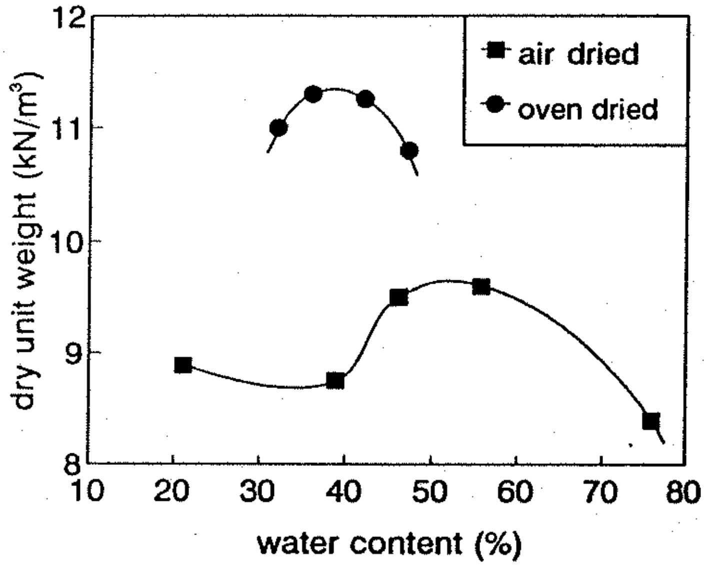


Figure 19 - Typical Proctor compaction curves for allophane soils (after Atlan, 1990; from Mitchell et al, 1991)

Figure 20 shows the affect of sample preparation on compaction test results for a soil from Salto Osorio dam in Brazil. This soil is described as a red porous soil with a Unified Soils Classification of MH. The location on the Unified Soils Classification plasticity chart is typical of Oxisols as described by Uehara (1982). When the sample was not dried out prior to testing and not reused the measured density was lower and the optimum moisture content was higher. The Hilf-Proctor test listed in Figure 20 is a method of determining the optimum moisture content using wet densities, the results of this interpretation correlate with standard Proctor interpretation results.

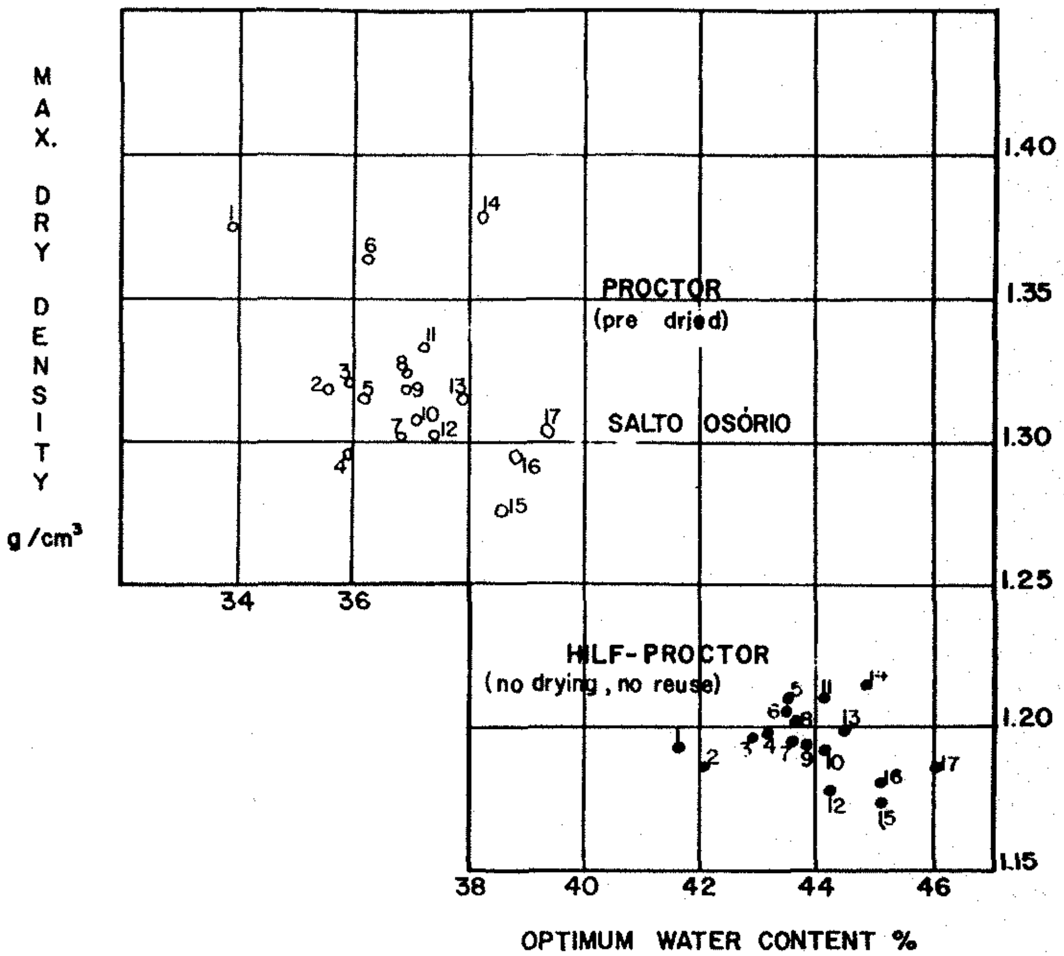


Figure 20 - Effect of drying samples during compaction testing of basalt saprolites from Brazil (de Mello, 1975).

4.7.3 Effect of variation in compactive effort

The compaction behaviour of a soil may be dependent on the compaction energy applied. In both temperate and tropical soils the compaction results achieved in the field may not be precisely duplicated by laboratory tests. This is illustrated in Figure 21 which shows compaction curves from weathered granite pegmatites for roller compaction in the field and laboratory results. In Figure 21-a the optimum moisture content in the field was 3% lower than the laboratory optimum. Material B depicted in Figure 21-b is from a greater depth than Material A and is sandier. The field compaction of Material B had a similar optimum moisture content as the laboratory tests, however, the maximum dry density which could be achieved in the field was only 98% of the laboratory maximum dry density. Mori et al (1979) also report different compaction results for roller compaction in the field compared to laboratory tests for Brazilian soils.

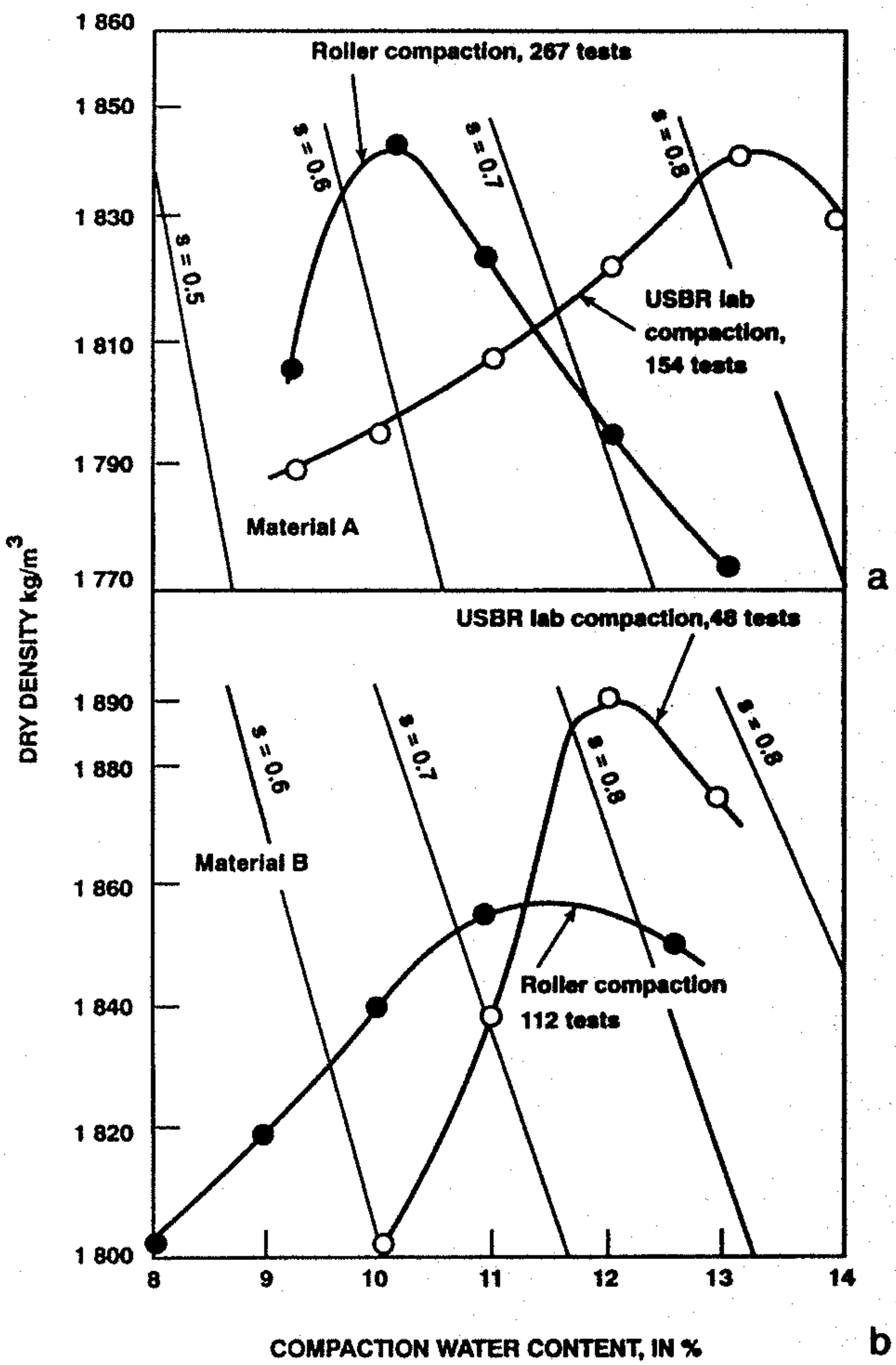


Figure 21 - Two average USBR laboratory compaction curves for weathered granite pegmatite are compared with corresponding average roller compaction curves obtained on the embankment (S = degree of saturation of compacted soil) (from Blight, 1962; see Blight, 1997).

5. MECHANICS OF COMPACTION

The shape of the density-moisture content curve for cohesive soils has been investigated by a number of researchers, including Proctor (1933), Hilf (1956), Lambe (1960), Olson (1963), and Barden and Sides (1970). The explanations offered in these references are based on the logical application of the current knowledge and theories of the time rather than reproducible measurements and thus are considered tentative. The process appears to be quite complex, involving "capillary pressures, hysterisis, pore air pressure, pore water pressure, permeability, surface phenomena, osmotic pressures, and the concepts of effective stress, shear strength, and compressibility" (Winterkorn and Fang, 1975).

The main factors affecting the engineering behaviour of a compacted soil are dry density, saturation, and microstructure. Each of these factors are in turn controlled by compaction water content, compaction energy and compaction method (Morgenstern et al, 1977).

To investigate the good engineering behaviour of some residual soils compacted wet of optimum it is important to understand some of the mechanics of compaction.

Proctor (1933) believed that in relatively dry soil capillary effects increased the frictional resistance. At a low moisture content these capillary effects resulted in a hard fill of high strength. Additional water increases the lubrication and causes a greater rearrangement of particles. The resulting soil is denser but less firm, presumably due to the loss of capillary suction. The benefits of lubrication are at a maximum at the optimum moisture content when the water, and any air that cannot be removed by compaction, just fill the voids that are left after compaction. With the addition of further moisture the volume of water filled voids at the end of compaction increases, lowering the strength and increasing plasticity.

Proctor's understanding gave him an excellent background for controlling compaction, particularly given the limited understanding of pore pressures and shear strength at the time.

Hilf (1956) used the concepts of pore air pressure and pore water pressure to refine Proctor's explanation of the shape of the compaction curve. In dry soils the frictional resistance to the compactive effort is increased by the high curvature of the menisci. Closer to optimum the capillary suction is less and the resistance to compaction is less, thus the soil reaches a denser state. On the dry side of optimum, air permeability is high and air is expelled easily. At the optimum moisture content the air voids are no longer connected (Winterkorn and

Fang, 1975). Wetter than optimum the trapping of air builds up pore air pressure and reduces the effectiveness of compaction.

Lambe (1960) explained the shape of the compaction curve using chemical theories.

Olson (1963) extended the use of effective stress concepts to explain the shape of the compaction curve. Increasing moisture content reduces the strength of the soil by reducing effective stress. Resistance to the compactive effort is built up as the effective stress increases as the soil particles slide over each other and build up stresses with the underlying previously compacted layer. The presence of a limiting density is attributed to an increase in pore air pressure from trapped air.

Some researchers have attempted to explain the engineering behaviour of clays with microscopic observations of the soil fabric. Barden and Sides (1970) showed that the moisture content during compaction greatly affected the resulting soil structure. Figure 22 shows scanning electron microscope photographs of a partially saturated low plastic clay compacted dry and wet of optimum. The shape of the compaction curve is explained in relation to the structure. The low density of dry fills is attributed to air filled macropores that exist between macropeds of soil, which are approximately 2 to 5 mm in diameter. The macropeds are held together by capillary forces. The macropeds have high strength and thus show limited deformation with the application of compactive effort. With increasing moisture content the macropeds become weaker and compaction results in lower void ratios as they deform. At the maximum dry density the macropores are completely filled by the deformed macropeds. The dry density of soils is reduced as more water layers are trapped between the soil particles with increasing moisture content. Barden and Sides (1970) also show that the air voids become non-continuous when the clay is compacted near optimum.

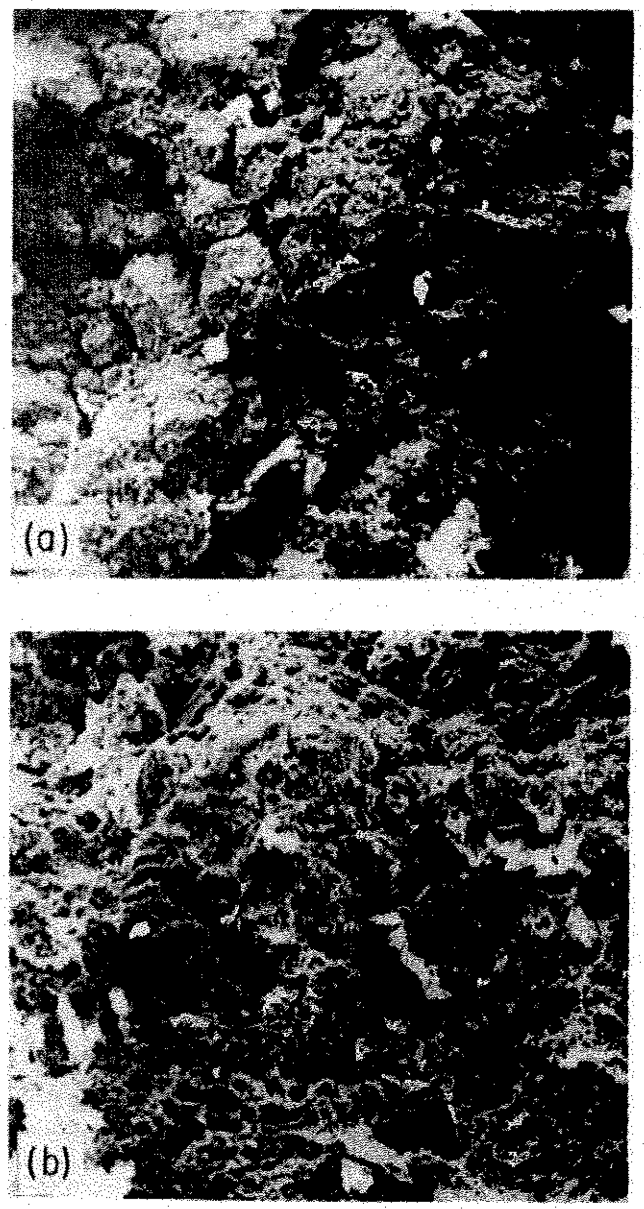


Figure 22 - Compacted clay air dried and viewed in scanning electron microscope at magnification of 110X. (a) Compacted 2.8% dry of optimum; (b) compacted 5.2% wet of optimum. (After Barden and Sides, 1970; from Winterkorn and Fang, 1975)

Typically soils that are compacted dry of the optimum moisture content have a greater shear strength and penetration resistance than soils compacted wet of the optimum moisture content, even if the dry densities are the same.

6. CASE HISTORIES OF EARTHWORKS WITH MOISTURE CONTENTS ABOVE OPTIMUM

6.1 GENERAL

Case histories of the construction of earthworks with wet, fine grained, tropical residual soils for dams and embankments from around the world have been reviewed and are summarized in Table 4. Further descriptions of some of these dams as well as general local experience are detailed by nation below.

6.2 BRAZIL

6.2.1 General

Residual soils and saprolites occur extensively in Brazil and with ongoing infrastructure development the geotechnical practice with these materials is well advanced. Because of the language difference as well as the difference in soils much of this knowledge has been slow in coming to general attention in North America. That is not to say it is not available as many papers by researchers from Brazil have been published in English in the Panamerican and International Society of Soil Mechanics conferences. Of particular note are the 12th International Conference on Soil Mechanics and Foundation Engineering held in Rio de Janeiro, Brazil, in 1989, and the 1st International Conference on Geomechanics in Tropical Lateritic and Saprolitic Soils held in Brasilia, Brazil, in 1985. The report from the Committee on Tropical Soils of the ISSMFE titled Peculiarities of Geotechnical Behavior of Tropical Lateritic and Saprolitic Soils – Progress Report (1985) and draws heavily on the Brazilian experience. More recently, da Cruz (1997) has summarized the case histories of 100 Brazilian dams.

De Mello has provided a history of the evolution of geotechnical practice in Brazil, from the early influence of teaching and consulting by Terzaghi and Casagrande, the importation of training from around the world, through the growth of national academic programs and professional practice (Varde et al, 1989).

Much of the advancements in the soil mechanics practices in Brazil have shown that many of the early recommendations based on conventional practice were overly conservative (de Mello, 1975). Much of the difficulties encountered in dam construction in Brazil have stemmed from extreme hydrologic events rather than difficulties with earthworks (Varde et al, 1989).

De Mello (1989) stated that criteria for moisture content ranges for compaction based on fixed values are conceptually wrong. He found that for most soils dimensionless graphs of relative compaction versus difference of natural

Table 4 - Comparison of Engineering Properties of Tropical Residual Soils from Embankments and Fills That Have Performed Adequately in the Past

		INDEX PROPERTIES									PROPERTIES MEASURED IN THE LABORATORY									PROPERTIES MEASURED IN THE FIELD					
Ref. No.	Project	Soil Description	Parent Rock	P.L. %	L.L. %	P.I. %	L.I. %	ω natural %	Unified Soils Classification	Comments	ω optimum %	Δω from optimum	γd max Standard Proctor Opt. (kg/m³)	γd max in situ	Cu kPa	φ' (deg) compacted	φ' in situ	c' (kPa) compacted	c' in situ	ω borrow %	Δω from optimum	γd fill (kg/m³)	Relative (Standard Proctor)	Cu kPa	Comments
1	Omai Tailings Dam Guyana, 1996	saprolite, mostly quartz and kaolinite		35	48	13	-0.2	33	ML / MH	major clay mineral is labradorite	27 avg	6 avg	1443 avg			32 avg		0		32	5	1440			
2	Arenal Hydro, Costa Rica, 1977	highly variable, mostly well graded silty sand	Pyroclastic					24 to 32	SM	Test Fill	17 to 20		1650 to 1680		low when remolded	41.4 (in situ)		0							
				69	110	41	0.5	91		-from 5 samples			723												
				42	63	21	0.4	51		-from 5 samples			1097												
				28	41	13	0.7	37		-from 5 samples			1330												
				47	74	27	0.5	60		-from 5 samples			961												
				NP	NP	NP		19		-from 5 samples			-												
																					2.8		92%		Borrow Area 1, 1-A, 2 passes D6 and 2 passes vibratory roller
																					1.2		94%		Borrow Area 2, overcompaction reduces performance after 2 to 4 passes
3/*4	Quebradona Hydro, Columbia, 1956	silty sand 24% M, 6% Cl	quartz diorite (batholith)	31	35	4	-1.3	26	ML	quartz = 35 to 45%, kaolinite = 35 to 60% illite = 5 to 10%, gibbsite = 10 to 16%	23*	3	1590*	1500 to 1600		28-35		0-30 (50 short term)		26	3	1540* (avg)			-rainfall of 2.3 to 5.3m / yr, 3 month dry season -high construction pore pressures, which
3/*4	Troneras Hydro, Columbia, 1959	silt 69% M, 10% Cl	quartz diorite (batholith)	34	43	9	-0.4	30	ML	"	24 to 28*	4	1440 to 1510*	"		"		"		30	4	1470* (avg)			dissipated rapidly -considerable scatter in placed densities
3/*4	Miraflores Hydro, Columbia, 1962	silty 41% M, 5% Cl	quartz diorite (batholith)	31	37	6	-1.5	22	ML	"	19 to 20*	3	1620 to 1680*	"		"		"		22	3	1660* (avg)			and moisture contents
3	La Fe Hydro, Columbia, 1970	silt 61% M, 7% Cl	quartz diorite (batholith)	30	40	10	-0.5	25	ML	"	19	6	"	"		"		"		25	6				
3	Santa Rita I Hydro, Columbia, 1966	silty 43% M, 7% Cl	quartz diorite (batholith)	32	38	6	-1.8	21.5	ML	"	16.5	5	"	"		"		"		21.5	5				
3/**5	Santa Rita II Hydro, Columbia, 1972	silt 49% M, 7% Cl	quartz diorite (batholith)	30	38	8	-0.5	26	ML	"	17.5 to 21.5**	5.5	"	"		"		"		26	5.5	1750** avg (est)			6.21 m of rain during year of construction
6	Punchina Cofferdam Hydro, Columbia, 1980	Saprolite 43% fines	quartz diorite (batholith)	25.4	35.6 avg	10.2	-0.1	24.2	SM		20.1	4.1	1660	1570		31		15 (to 50)		24.2	4.1				1.5m of bulging, stopped with pore pressure dissipation
7	Netzahualcoyotl Hydro, Mexico, 1959	residual soil 70% fines	marine sedimentary	27 est	35-70 45 avg (est)	18 avg est	0.2	24 to 38 est	ML / MH	2.3 and 2.7 m of rain/yr during construction	16 to 33	+7 to 9	1650 est		60	22 (φ tot)				24 to 38 est	+7 to 9	1625 est	95% avg		-20cm loose lifts, 8 passes sheepsfoot -density in embankment less than lab
8	Sasumua Water Supply, Kenya, 1949	Saprolite 20% to 50% clay	basalt	45 to 58 avg	85 to 90 avg est	28 to 53 avg est		63 to 65 avg	MH	clay content depended on dispersing agent 58% halloysite	50	13 to 15	1120			34 to 36		26 to 30							
8	Tjipanoendjang Hydro, Java	6% to 33% clay	andesites and tuffs	47 to 53 56, 56	64 to 69 112, 118	11 to 22 54, 62		95 to 97	MH	two sets index tests were done, both from the borrow areas; 76% halloysite	48 to 49	47	1140 to 1160								above ωopt (interpreted)				14 in. loose lifts (thinner not possible), 4 to 6 passes of 5 / 6 ton rollers, compacted thickness of 8 in.
8	Silvan Australia	Saprolite 25 to 50% clay	basalt	42 avg	64 avg	22 avg		"close to P.L."		80% halloysite less than 1 μ	"close to P.L."	-	1270							"close to P.L."		> 1270			6 in. lifts compacted to 4 in. with 4 passes of 8, 13, and 18 ton grooved rollers
8	Wayatinah B Hydro, Tasmania	Saprolite	dolerite	33.1 avg	54 avg	20.9		42	MH	Residual dolerite talus plots above A line	31.1	10.9	1400			30 to 38		40		42	soil dried to +2% opt.				
9	Las Cristinas Mining Prospect, Venezuela	Saprolite	andesite, tuff	30 to 55	50 to 70	5 to 30		20 to 55 35 avg	MH		23 to 36	-1 to +12 avg	1340 to 1680		125	30	29	32	30						
	BRAZIL																								
10,*11	Itapanca	Lateritic, clayey	claystone	21	57	42		20 to 26 est	CL		23	-3 to +3				18.5				20 to 26 est	-3 to +3	1605	97 to 100		
		Saprolite	granite	NP	NP 20 to 35*	NP 7 to 17*		12.5 to 15.5 est	CL*		14.5 7 to 12 est*	-2 to +1	1770 to 2030 est*			34 to 37 38 to 40.5*				12.5 to 15.5 est	-2 to +1	1879	95 to 103		
10	Tres Irmaos	Lateritic, clayey	basalt	31	73	31		28.5 to 31 est			30	-1.5 to +1				24				est	-1.5 to +1	1451	95 to 98		
		Colluvium Residual	sand and basalt	21	33	12		13.9 to 16.4 est			15.4	-1.5 to 1				29				13.9 to 16.4 est	-1.5 to 1	1847	98		
10	Juquia I	Lateritic, clayey	migmatite	35	66	21		23.2 to 28 est			25.6	-2.4 to +2.4				29				23.2 to 28 est	-2.4 to +2.4	1457	95 to 98		
10	Juquia II	Lateritic, clayey	migmatite	23	44	18		17.6 to 21.4 est			19.5	-1.9 to +1.9				31.5				17.6 to 21.4 est	-1.9 to +1.9	1696	95 to 98		
10	Pocos de Caldas	Lateritic, clayey	alkaline rocks	28	48	33		25.4 to 31.4 est			28.4	-3 to +3				32.5				25.4 to 31.4 est	-3 to +3	1375	95 to 97		
10	Itaipu	Lateritic, clayey	basalt	26	59	np		27.3 to 31.3 est			29.3	-2 to +2				28.5				27.3 to 31.3 est	-2 to +2	1494	101 +/-2		
10, ***12	Nova Avanhandava	Saprolite	basalt	55	86 45 to 90***	31 12 to 42***		36 to 43.5 est 25 to 53 est***	MH		38.5 25 to 40 est***	-2.5 to +5 0 to +20***	1220 to 1490 est***			28				36 to 43.5 est	-2.5 to +5	1280	95 to 97		
10	Euclides da Cunha	Saprolite	gneiss	29	48	19		15.4 to 18.4 est			16.4	-1 to +2				28				15.4 to 18.4 est	-1 to +2	1714	100		borrow material dried; 12 ton smooth-wheel rollers, careful to scarify between lifts
10	Passauna	Saprolite	migmatite	36	50	12 to 17		23.7 to 27.7 est			25.7	-2 to +2				36				23.7 to 27.7 est	-2 to +2	1540	95 to 98		
10	Emborcacao	Saprolite	schist	27	48	19		16.9 to 19.9 est			18.9	-2 to +1				29.5				16.9 to 19.9 est	-2 to +1	1605	95 to 100		
10	Rosana	Colluvium Residual	sand and basalt	14	26	12		10 to 12 est			11	-1 to +1				31.5				10 to 12 est	-1 to +1	1953	95 to 99		
10	Porto Primavera	Colluvium Residual	sandstone	np	np	np		8.9 to 10.4 est			9.9	-1 to +0.5				36.5				8.9 to 10.4 est	-1 to +0.5	1990	98		
11	Paraibuna	Saprolite	gneiss		35 to 50 est	7 to 20 est		"well above optimum"	ML		13 to 20		1610 to 1820 est			29-32		0							
11	Paraitinga	Saprolite	gneiss		35 to 50 est	7 to 20 est		"well above optimum"	ML		13 to 20		1610 to 1820 est			29-32		0							
11, ***12	Itumbiara	Saprolite	basalt		58 to 90 est	15 to 45 est		35 to 57***	MH		28 to 42 est 40 to 47 est***		1220 to 1480 est 1190 to 1260 est***			28 to 29		0						28 to 29	
11, ***12	Capivara	Saprolite	basalt		45 to 65 est	12 to 30 est		"well above optimum"	MH		28 to 42 est 18 to 28 est***		1220 to 1480 est 1490 to 1750 est***			28 to 29		0						28 to 29	
11, ***12	Monday	Saprolite	basalt		50 to 95 est	10 to 40 est		"well above optimum"	MH		28 to 42 est 35 to 42 est***		1220 to 1480 est 1220 to 1340 est***			28 to 29		0						28 to 29	

Notes: - 'est' denotes values estimated from graphs
- 'avg' denotes average values
- *, **, *** denotes source reference

Reference List for Table 4

Reference No.	Reference
1	Golder Associates, unpublished reports
2	Rodda, K.V., Perry, C.W., Roberto Lara, 1982. Coping with dam construction problems in a tropical environment. Engineering and Construction in Tropical and Residual Soils, A.S.C.E. Geotechnical Division Special Conference, Honolulu, Hawaii, p. 695-713.
3	Villegas, F., and Mejia, O., 1976. Experience on earth dams built of silt in Colombia. 12th International Congress on Large Dams, Mexico, Q. 44, R. 29, p. 581 - 597.
4	Li, C.Y., 1967. Construction pore pressures in three earth dams. Proceedings of the A.S.C.E., Vol. 93, No. SM2, March, p. 1-27.
5	Villegas, F., Mejia, O., and Sanchez, G., 1976. Construction of an earth dam built of silt in Colombia under extremely rainy weather. 12th International Congress on Large Dams, Mexico, Q. 47, R. 13, p. 931-947.
6	Villegas, F., 1982. Difficulties during construction of the Punchina Cofferdam. 14th International Congress on Large Dams, Brazil, Q. 55, R. 60, p. 1081-1101.
7	Instituo de Ingenieria, UNAM, 1976. Behaviour of Dams Built in Mexico, Vol. 1. Ciudad Universitaria, Mexico, 587 p.
8	Terzaghi, K., 1958. Design and performance of Sasumua dam. Proceedings Institute of Civil Engineers London, Vol. 9, p. 369-394.
9	Bruce Geotechnical Consultants Inc., 1995. Unpublished reports.
10	Nogami, J.S., et al. *(26 names)*, 1985. Peculiarities of tropical lateritic and saprolitic soils uses as construction materials - dams. Progress Report, Tropical, Lateritic and Saprolitic Soils. Committee on Tropical Soils, International Society of Soil Mechanics and Foundation Engineering. Brazilian Society for Soil Mechanics, p. 329-404.
11	Mori, R.T., de Bare, F.R., Pan, Y.F., 1979. Properties of some typical compacted saprolites. Proceedings of the 6th Panamerican Conference on Soil Mechanics and Foundation Engineering, Lima, Peru, Vol. 2, p. 583-591.
12	Mori, R.T., 1979. Engineering properties of compacted basalt saprolites. Proceedings of the 6th Panamerican Conference on Soil Mechanics and Foundation Engineering, Lima, Peru, Vol. 2, p. 575-583.

moisture content from the optimum divided by the optimum moisture content (PC% vs. $\Delta\omega/\Delta\omega_{opt}$) could be superimposed for optimum moisture contents from 8% to 45%. No reference was provided for these dimensionless graphs. For basalt clays the acceptable range of compaction moisture contents was found to be from 0.92 to 1.10 of ω_{opt}, and if rock fragments were present then the moisture content could go up to 1.15 ω_{opt}. Since these clays are described as having an optimum moisture content of approximately 40%, this relationship would predict satisfactory performance with moisture contents up to 4 to 6% above optimum. Although it is not stated, presumably this relationship is derived from and applicable to regional practice in Brazil.

According to Nogami et al (1985), quoting work by Pinto et al (1970) and Mori et al (1979), Brazilian lateritic and saprolitic soils compacted up to about $1.1w_{opt}$ usually develop small construction pore pressures.

Plasticity tests are considered misleading in predicting performance because the behavior of the fill is essentially granular as opposed to the remolded state which is tested. De Mello (1975) has found that requirements based on a soil's position on the plasticity chart have proved to be academic and are not given great weight when selecting borrow material. For many years dams in Sao Paulo state incurred very significant additional costs in scraping surficial material to obtain clay from above the A-line to satisfy concerns about internal erosion.

During construction of the Tres Marias dam the shear strength measured in laboratory compaction test samples were at least 10 to 20% weaker than the strengths in field block samples (de Mello, 1975).

6.2.2 Dams from Southern-Central and North-Eastern Brazil

In general, the saprolites of Brazil have proven to be very satisfactory construction materials. Not all of the residual soils have natural moisture contents that are above the optimum moisture content. The typical natural water content of the granite saprolites from the North-Eastern region of Brazil, where the climate is mainly dry, is far below the optimum (Mori, et al, 1979). However, the natural water content of basalt and gneiss saprolites from the Southern-Central region, where the rainy season precipitation is intense, is typically well above the optimum water content.

The basalt saprolite is typically clayey, the gneissic saprolite is typically micaceous and silty, and the granite saprolites tend to be sandy. The optimum moisture contents of the residual soils from different parent rocks are shown in Figure 23.

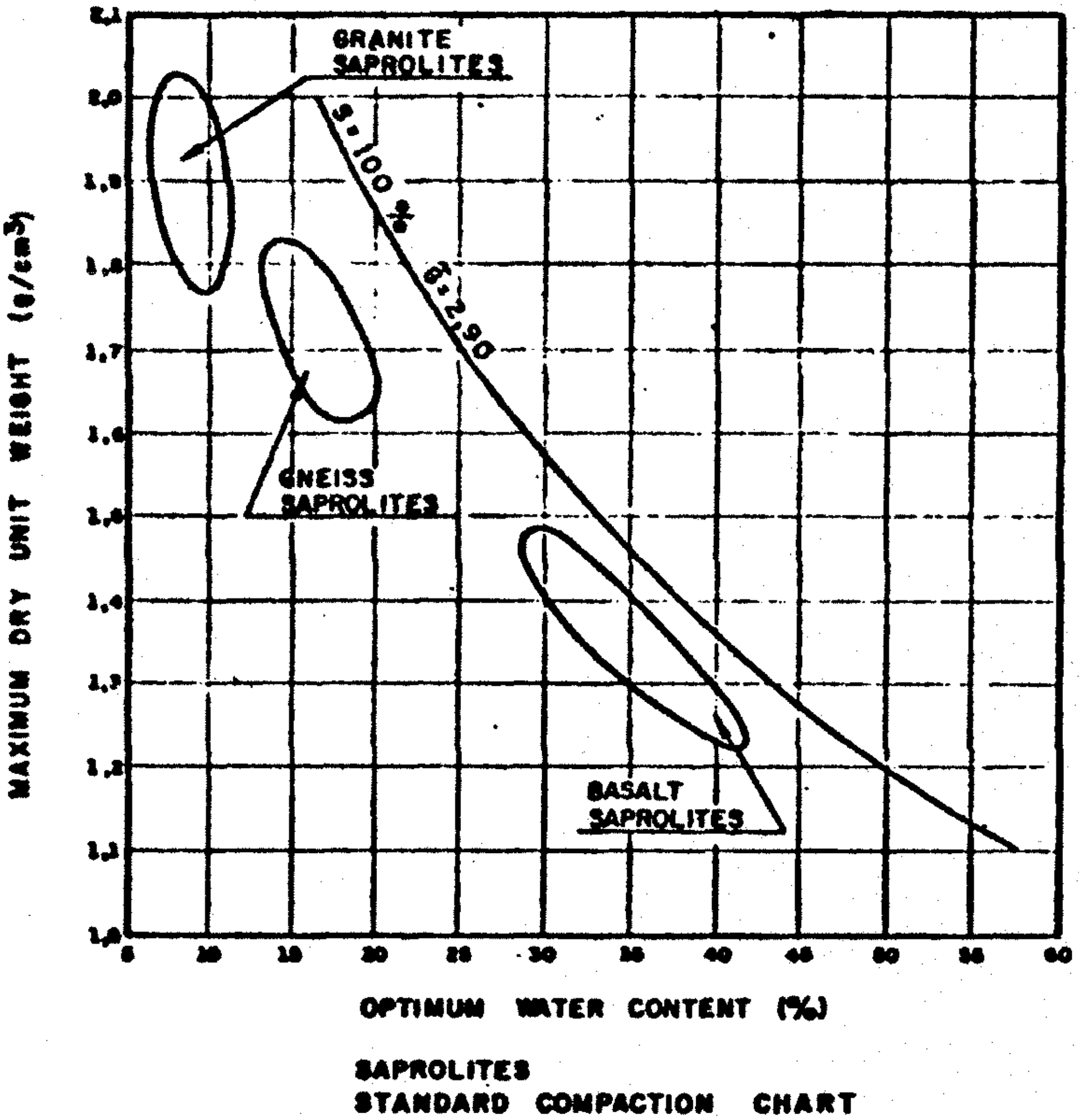


Figure 23 - Optimum moisture content and maximum dry density for granite, gneiss, and basalt derived residual soils from Brazil (from Mori et al, 1979).

These saprolites were found to exhibit adequate and homogeneous strength and compressibility even when compacted in water contents well above or below the Proctor optimum water content (Mori et al, 1979).

Part of the good engineering characteristics of these saprolites may be related to the optimum moisture content often falling well below the plastic limit (Mori, 1979). The relationship between the optimum moisture content and the plastic limit are shown in Figure 24. From this figure it appears that the greater the plasticity index the greater the amount that the optimum moisture content falls below the plastic limit. The basalt saprolites from Southern-Central Brazil have natural moisture contents that are significantly above optimum, up to 20%, but are within ±10% moisture content of the plastic limit. It is not stated if the mineralogy of these saprolites is such that sample preparation would affect these plasticity limits and Proctor density testing results, which were presumably done according to ASTM standards.

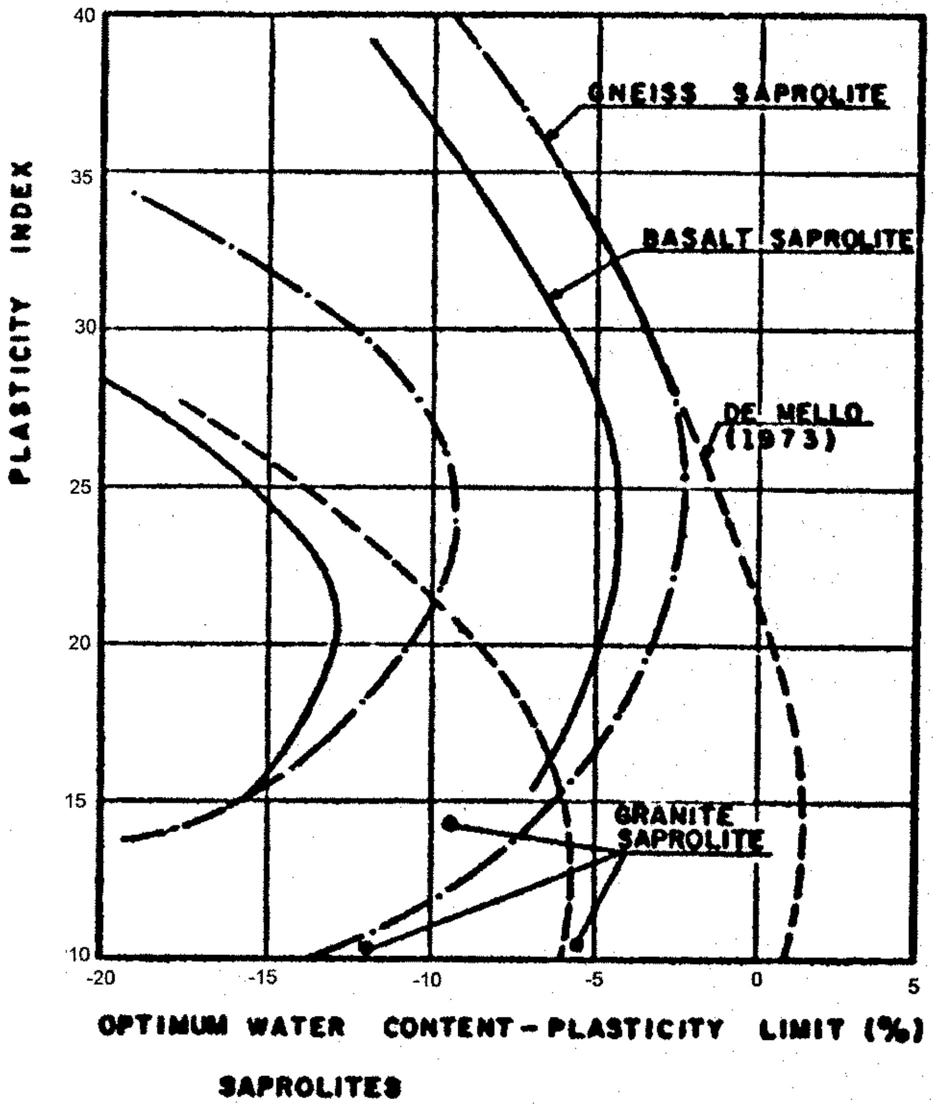


Figure 24 - Plasticity index versus (optimum moisture content - plasticity limit), showing that the optimum water content is often well below the plastic limit for three different saprolites from Brazil (adapted from Mori et al, 1979).

6.2.3 Dams from all areas of Brazil

Da Cruz et al (1985) discuss the soil characteristics from several dams throughout Brazil, this data has been summarized in Table 4. The data analyzed by da Cruz et al (1985) came from more than 50 dams built in Brazil between 1945 and 1985.

Most of the compaction done for earth dams in Brazil is dry of optimum and thus construction generated pore pressures tend to be low. However, high construction pore pressures have been measured, such as at Capivara dam which was constructed of fill with a natural moisture content 5% above the optimum and had pore pressures go up to 50% of σ_1 (Cruz, 1985).

The standard test in Brazil for predicting pore pressure response has been the PN test, which is a UU test with the ratio of maximum and minimum principle stresses, σ_1/σ_3, held constant. A satisfactory agreement between this test and Skempton's A pore pressure parameter has been found (da Cruz et al, 1985).

In discussing the properties of Brazilian residual soils da Cruz et al (1985) have found it convenient to categorize the soils into four groups. Groups I and II are residual soils, whereas Groups III and IV are weathered colluvial soils and transported soils respectively.

Group I soils are described as lateritic clayey soils or residual mature soils. These soils tend to be unsaturated and have moisture contents close to the standard Proctor optimum. The workability is reported to be quite good for moisture contents up to $0.2\omega_{opt}$ above optimum.

The Group II soils are saprolites which have not been modified by laterization. They tend to have higher natural moisture contents than the Group I soils. Compared to the Group I soils the workability is good at higher moisture contents and the final compacted product is more heterogeneous.

In comparing the shapes of standard Proctor curves for laterite and saprolite soils to temperate soils, Nogami et al (1985) commented that:

- The lateritic soils generally have steeper slopes on the dry side of optimum;
- The silty and clayey saprolitic soils have similar slopes on the dry side of optimum;
- The lateritic soils attain higher degrees of saturation for similar maximum dry densities as saprolitic soils;
- Saprolitic curves tend to have less pronounced peaks than lateritic curves.

Mori et al (1979) carried oedometer and triaxial compression tests on compacted saprolite samples at several different compaction degrees and water contents. The results indicate good engineering performance with high strength and homogeneous compressibility, over a much wider range of placement water contents than the mature soils.

6.3 Colombia

6.3.1 Medellin Dams

A series of earth dams have been completed for the hydroelectric and water supply systems of the city of Medellin, Colombia. These dams have all been constructed primarily of silty residual quartz diorite with high natural water contents placed in high rainfall (Villegas and Mejia, 1976). These dams include, in chronological order from the earliest, Quebradona, Troneras, Miraflores, Santa Rita (Stage I), La Fe, and Santa Rita (Stage II) dams. Also beginning construction in the mid 1970's were Punchina, San Lorenzo, and Playas dams.

The characteristics of the soils used in construction of the Medellin dams are given in Table 4. The properties of the silt and sandy silt borrow materials include high water content, low natural density, medium to high compressibility, rather low permeability, and high shear strength given the grain size distribution and low density (Villegas and Mejia, 1976). The nature of the materials and the rapid construction due to a short construction season has contributed to the generation of unusually high pore pressures. The high pore pressures have necessitated the use of unconventional design and construction processes.

On the surface, the dam sites usually had a 0.3 to 0.5 m thick layer of high plastic clays, unified classification of MH. The horizon used for the bulk of the embankment fill underlies the surface horizon and averages 5 to 20 m thick. This horizon typically is classified as ML and corresponds to zone 1B of the weathering classification proposed by Deere and Patton (1972). The typical mineralogical composition of the silts is:

- Quartz, 35 to 45%;
- Kaolinite, 35 to 60%;
- Illite, 5 to 10%,
- and gibbsite, 10 to 16%.

The moisture content of the bulk of the fine grained borrow material ranged from 20% to 40%. The typical effective strength parameters are $c' = 0$ to 30 kPa and $\phi' = 28°$ to $35°$.

Villegas and Mejia (1976) hypothesized that the apparent cohesion in these soils is not only due to pore pressure effects but also due to the structural bond of the original rock remaining in the soil skeleton. They also thought there could be some cementation from iron oxides.

Each of the Medellin dams had in common the following construction conditions (Villegas and Mejia, 1976):

- high annual rainfall, from 2300 to 5300 mm,
- short dry season, about 3 months, which is not present in some years,
- limited choice of borrow material – typically silt with a natural water content of 2 to 10% above optimum,
- a scarcity of materials suitable for filters,
- concerns with the foundations due to relict structures.

The construction approaches adopted to deal with the materials and difficult climate included:

- use of almost all the material in the borrow pits regardless of moisture content, excluding organic material,
- placing fill at the maximum rate possible during the dry season,
- minimizing infiltration at the borrow area with ditching and by avoiding stockpiling,
- minimizing infiltration at the fill by sealing the surface with low ground pressure tracked equipment,
- adjusting the design and compaction method continuously in the field to respond to the foundation and borrow area conditions,
- minimizing foundation preparation,
- continuously monitoring the fill performance with pore pressure cells, cross-arm settlement devices, surface monuments, etc.,
- designing the embankment for high pore pressures.

The construction philosophy has been to use the materials at their natural water content and vary the compaction effort to obtain densities near the theoretical maximum for that water content. The compacted dry densities achieved at three of the dams using this methodology are shown in Figure 25 (Li, 1967). It was found necessary to keep at least two types of compaction equipment, usually sheepsfoot rollers and bulldozers, at each project to deal with variation in moisture content. The hauling equipment were specially selected to cope with the slippery road conditions. Placement is directed by the engineer in the field to account for the weather, borrow area condition, equipment trafficability, pore pressures, material availability, and stability assessments. The construction procedures are outlined in more detail in Section 7.

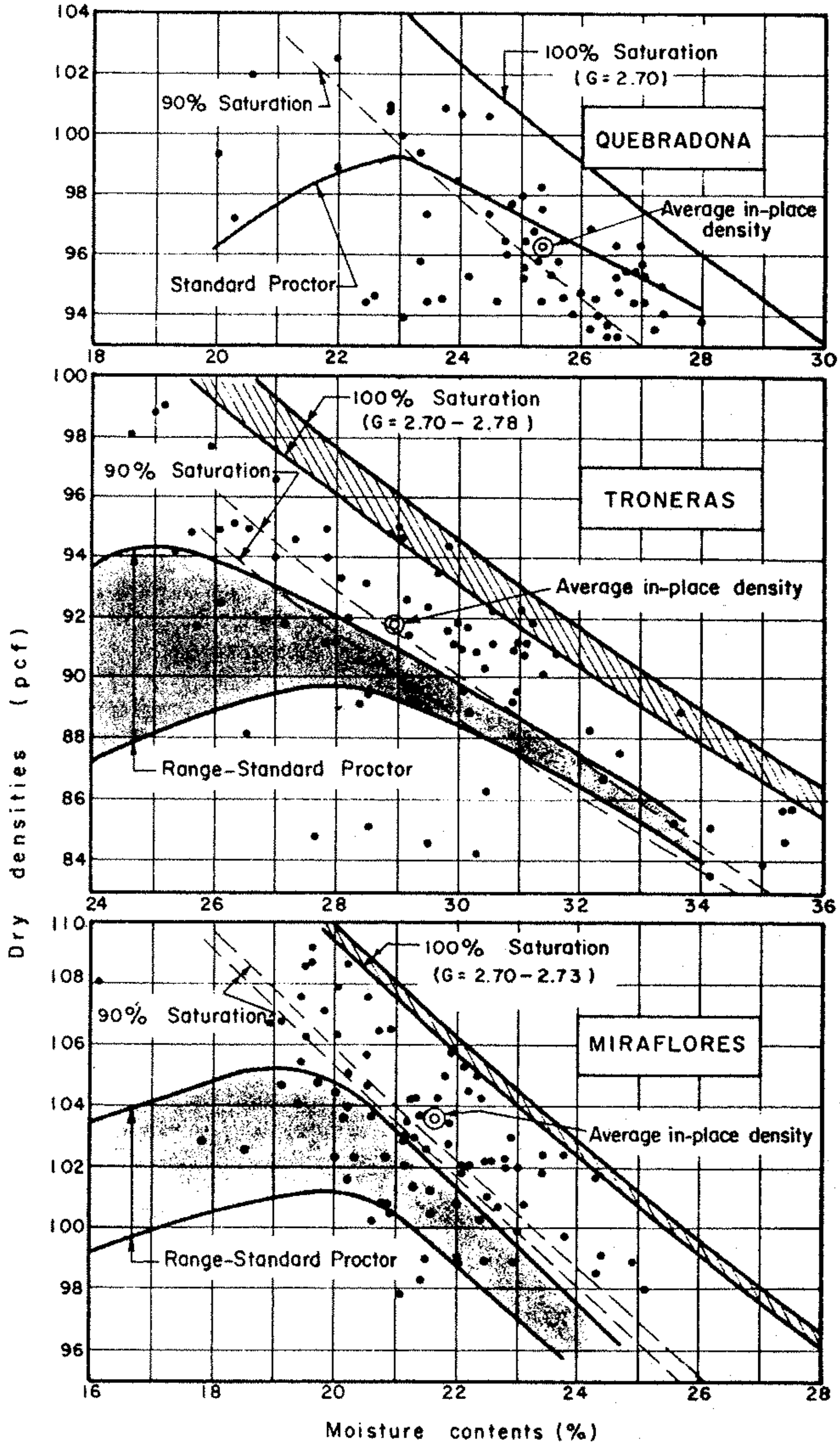


Figure 25 - Average in-place densities achieved in three earth dams in Colombia (from Li, 1967).

Design adjustments for the high pore pressures included flatter slopes, counterweight fills at the toe of the slopes using waste, and in some cases drainage blankets or trenches.

The factors influencing the generation of high pore pressures in these dams are discussed in detail by Li (1967). The highest pore pressures were recorded in the silty zones of Miraflores, Troneras, and Santa Rita Stage 1 dams. Lower pore pressures were measured in the dams that were constructed slower - in the sandier fills of Quebradona, Santa Rita Stage 2 dams, and the fine grained La Fe dam.

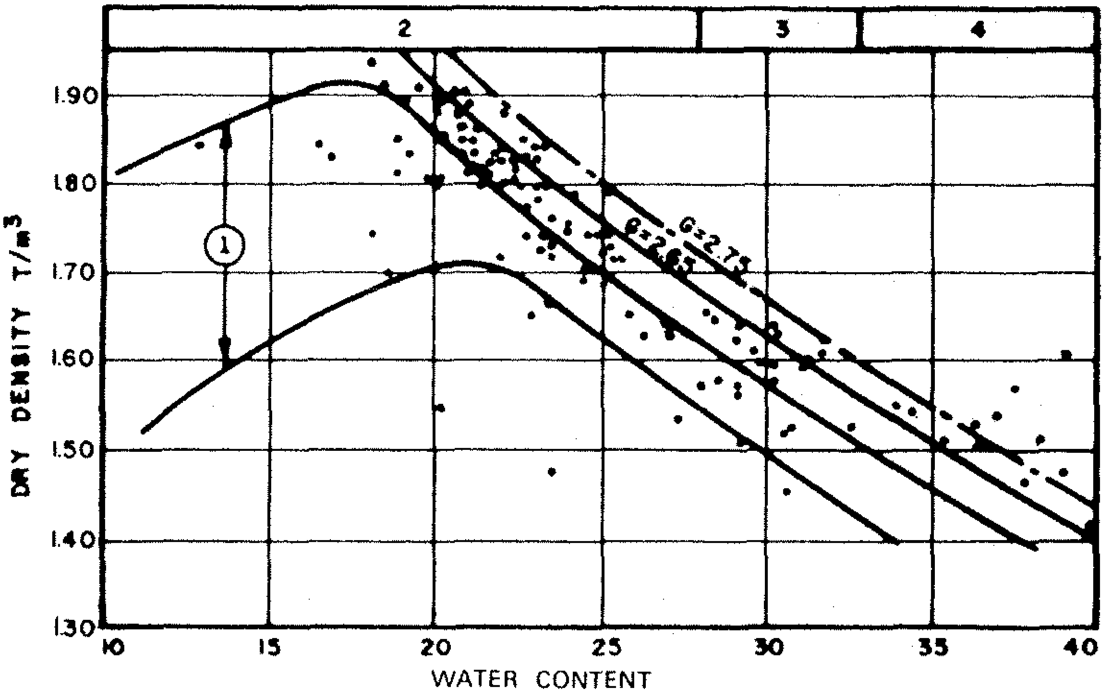
Villegas and Mejia (1976) noted that there was a pore pressure decrease due to bulging at all of the dams, but mainly at Troneras, Miraflores, and La Fe. This was even before the dramatic example of the Punchina dam, which is discussed later.

6.3.2 Santa Rita

The Santa Rita dam is part of the system of dams for the city of Medellin, Colombia, which is described above. The 60 m high dam warrants a separate discussion because of the adverse weather conditions during its construction. During the two to three years of construction for Stage II the average annual precipitation was 6210 mm, which far exceeded the precipitation recorded during the construction of the other dams (Villegas et al, 1976).

The geology and soil conditions at the Santa Rita dam are generally the same as the other dams in the Medellin system.

The unconventional construction procedures derived from experience with earlier dams described previously were utilized. During construction it became evident that for various reasons that for the dam to be completed on schedule that fill would have to be placed during the entire year. As well as the unconventional procedures previously adopted, it was found that extra attention had to be given to the working surface of the embankment and haul roads to ensure that the contractor's equipment could operate. It was also important to maintain as many working areas as possible for the greatest flexibility to respond to the weather conditions and stability monitoring.



Compaction results.

(1) Range of Standard Proctor Curves.
(2) Sheepsfoot roller.
(3) Self-propelled tamping roller Caterpillar 825 B.
(4) Caterpillar D 8 tractor.

Figure 26 - Compaction results from Santa Rita Stage II dam, Colombia. The types of compaction equipment used and dry densities achieved for each range of moisture content are shown (from Villegas et al, 1976).

The performance of the embankment has been entirely satisfactory. Although the fill placed had a moisture content that was on average 7% above optimum the construction pore pressures were tolerable. In fact, the pore pressures were lower than those recorded for the Troneras and Miraflores dams that were constructed of similar soils but under more favourable weather conditions. The lower pore pressures are attributed to raising the embankment over the entire year rather than the rapid advancement during the 3 month dry season that was used at Troneras and Miraflores (Villegas et al, 1976).

6.3.3 Punchina cofferdam

The construction of the Punchina dam as part of the Medellin hydroelectric system described above required a 45 m high cofferdam for flood control. The unusually large displacements observed during construction of the cofferdam from the use of overly wet material have been described by Villegas (1982). The properties of the fill material are listed in Table 4.

The bulk of the fill is residual quartz diorite material with unified soil classification of ML and SM. The majority of the borrow came from a layer corresponding to horizon IB of the profile proposed by Deere and Patton (1972). The natural moisture content of the soil averaged 28% and was 4 to 6% above optimum.

The cofferdam was built on an extremely tight schedule due to the short dry season and the threat of overtopping with the floods of the wet season. The schedule precluded drying out the borrow material or using staged construction. The project site has high rainfall - the average annual precipitation is 3100 mm.

During construction the downstream slope of the cofferdam bulged between 1.2 and 1.5 m over a period of 10 days. Some cracks were evident on the surface of the cofferdam. Ongoing raises were able to take advantage of the stability of the upstream side. Pore pressure monitoring and deformation monitoring were undertaken throughout the construction. The deformations stopped virtually immediately when the cofferdam was completed.

The deformation was attributed to high pore pressures arising from the rapid fill placement and high natural moisture content of the soils. The stopping of the deformations was due to an increase in shear strength gained with the rapid dissipation of the excess pore pressures. The dissipation of pore pressure is thought to be due to the dilation of the fill material with large strains. A drop in the piezometric levels was observed to coincide with most of the deformation. Undrained triaxial strength testing of the fill material showed a plastic behaviour with no definite peak strength or drop to a residual strength. Measurements during the testing showed that pore pressures began to decrease at 3 to 4% strain.

Stability analyses were undertaken using effective stress strengths and the piezometric readings, Factors of Safety of close to 1 and even below, to 0.97, were found. Horizontal filter blankets were incorporated into the fill and found to be effective in controlling the generation of high pore pressures and in accelerating their dissipation. It is thought that the effectiveness of the blankets was underestimated in the stability analyses that had a Factor of Safety below unity. The lowest factors of safety corresponded with the initiation of the most rapid deformation.

6.4 Kenya

The description of the construction of the Sasumua dam in Kenya by Terzaghi (1958) is one of the earliest reports of the acceptable engineering behaviour of tropical residual soils that would be considered undesirable by conventional criteria for fills. The Sasumua dam was constructed between 1949 and 1956.

The majority of the fill is composed of residual soils derived from basaltic lavas, tuffs, and breccias.

The designers of the dam were concerned about the suitability of this soil for dam fill, particularly given the abnormally high plastic limits and low dry density at optimum moisture content. A thorough experimental study of the engineering properties and mineralogical nature of the soils confirmed their suitability for use as fill.

The engineering parameters of the Sasumua clay are summarized in Table 4.

The Sasumua clay plots below the A line and this was taken to account for the low dry density achieved at the optimum moisture content. The measured Atterberg limits were found to depend on the testing method. This was attributed to the tubular structure of the halloysite containing a large amount of structural water. The standard deviation of the Atterberg limits during the investigation was found to be no greater than a relatively homogeneous sedimentary deposit. The liquid limit of the Sasumua clay had a standard deviation of ±14.8%, versus the corresponding value of the London Clay of ±13.4%.

The optimum moisture content of the Sasumua clay averaged 50%, the natural moisture content averaged 63%, and the plastic limit averaged 45% to 58%. The clay was eventually dried and placed near the optimum moisture content, although at the time of the investigation and even during early construction it was not certain that this level of drying could be achieved.

The mineralogical investigation of the clay found that it was composed of at least 70% clay minerals, primarily halloysite. The halloysite had a size of less than 1 μ, but when the grain size of the sample was determined using mechanical means the percentage of particles smaller than 2 μ was below 25 to 30% with regular treatment and 40 to 50% with a powerful dispersing agent. Hence the clay was determined to be composed of spongy aggregates of crystals rather than individual crystals. The aggregation was judged to account for the position of the soil below the A line on the plasticity chart, as well as their high permeability, low compressibility, low dry density at optimum moisture, and high shear strength parameters.

It was judged that the clay was not likely to deflocculate under operating conditions of the dam. Terzaghi (1958) discusses the performance of the Sasumua dam as well as the Tjipanoendjang dam in Java, and the Silvan dam in Australia which were constructed of similar soils. The performance of all these dams has been satisfactory, equal to the best of dams built with "normal" clays. The stability of the Tjipanoendjang dam and Silvan dam with relatively steep slopes over the long term, 25 years at the time of the Sasumua construction, indicates that the shearing resistance of these soils does not decrease with time even though it is the product of crystal aggregation.

6.5 JAPAN

The construction of a highway embankment out of sensitive volcanic clay in Japan is described by Kuno et al (1978). The dominant clay minerals are allophane and halloysite. The natural water content is extremely high, ranging from 60% to 180%. The optimum moisture content was not determined.

Several construction procedures were developed to overcome the high sensitivity of the soils. Figure 27 shows the susceptibility of these soils to overcompaction by showing a decrease in cone penetration resistance with increasing compactive effort. Compaction was undertaken using bulldozers with low ground contact pressure to limit remolding the fill. For short hauls the bulldozers were used for excavation, hauling and placement. Medium length hauls, less than 300 m, were undertaken using towed scrapers. Shovels and dump trucks were used for longer hauls. Haul roads of sandier material were constructed to allow the dump trucks to reach the spreading bulldozers. Infiltration of rainwater and surface water into the fills and borrow areas was limited by compacting the surface, using ditches, and maintaining a sloped grade from the center of the embankment. The compacted fill showed an increase in strength with time.

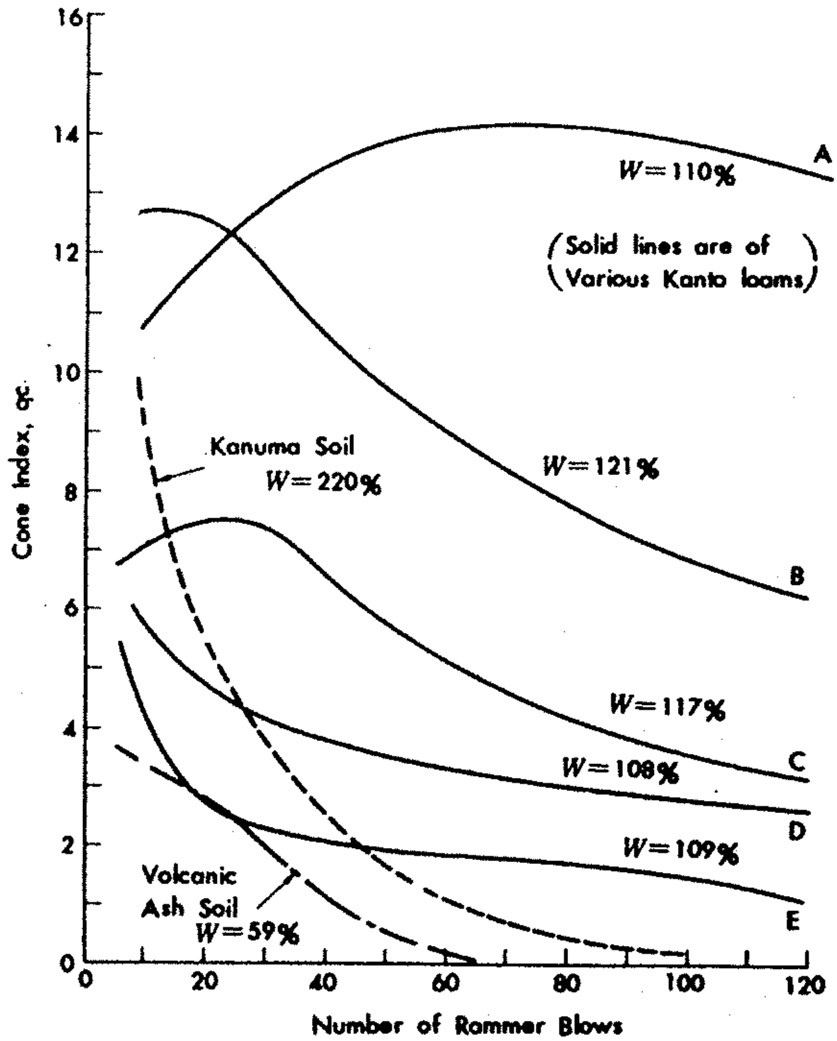


Figure 27 - Relationship between cone resistance and compactive effort, in the form of number of hammer blows, for volcanic soils of varying moisture content from Japan (from Kuno et al, 1978).

6.6 Costa Rica

Arenal Dam is a 60 m high hydroelectric dam located in the northwestern part of Costa Rica. The dam was completed in 1977 and is described by Rodda et al (1982). The foundation material is composed of highly variable pyroclastic (volcanic) deposits, which are described as having a "fused" structure, which results in a high drained strength and low compressibility.

The shells of the dam were constructed of alluvium, but the core of the dam was constructed of processed well-graded, non-plastic, silty sand material. The properties of the core material are summarized in Table 1. No pedological description of the core material was given but given its volcanic origin these soils most likely belong to the andisol / andosol family. The mineralogical composition of the embankment fill varied. Some of the samples examined contained substantial halloysite, and most contained significant amounts of the clay mineral labradorite as well as unaltered volcanic ash.

Upon completion of the first stage it was found that foundation settlements, pore pressures in the fill, and transverse movements were much lower than expected.

The specifications for the core material originally called for compaction to not less than 95% of Standard Proctor maximum dry density. The test fills found that this level of compaction was not possible at the natural moisture content in the first borrow area. Compaction was undertaken using two passes of a D-6 bulldozer followed by 2 passes of a vibratory roller to seal the surface against infiltration. Eventually a new borrow area was developed and the compaction results improved.

The average moisture - density line measured during construction was found to coincide with the computed 90% saturation line.

Although the core material was placed at a relatively low placement density it was found to consolidate rapidly under subsequent lifts.

During laboratory compaction testing moisture loss from the mineral structure led to overestimation of the maximum dry density and underestimation of the optimum moisture content. The testing procedures were modified such that the samples were dried from the natural moisture content to the test moisture content.

6.7 Fiji

Monasavu dam, at Monasavu Falls in Fiji, was started in 1979 and was due to be completed in 1982 (Knight et al, 1982). The dam was to be a 85 m high rock fill hydroelectric dam with a clay core.

The clay core was constructed of a very wet, halloysitic residual clay that is weathered from a sandstone. The borrow material in situ is described as weathering grade IV – "firm to stiff red-orange to brown silty clay with gravel-sized rock fragments in banded and small block form. Completely weathered rock, with relic structures" (Knight et al, 1982).

The natural moisture content of the clay core material is typically 20% above optimum. Climatic conditions at the site preclude drying the soil.

The sensitivity of halloysite to moisture content testing procedure was seen in these soils. Oven drying samples to determine moisture content, which was the standard procedure, returned values approximately 13% higher than air drying.

Initially the construction specifications were to be based on end-product requirements including unconfined compressive strength combined with method requirements. During construction a change was made to a wholly method-related specification that aimed at the densest and driest homogeneous core achievable. Compaction of the wetter material was done in 100 mm layers with low ground pressure Caterpillar D-6 bulldozers. The bulldozers sank into the fill and left V-shaped grooves. The compacted fill comprised of "discrete uncrushed nodules up to 100 mm size surrounded by a remolded clay matrix" (Knight et al, 1982). Compaction is stopped when excessive nodule breakdown results in a fill with poor trafficability. The bulldozer compaction is supplemented with a towed pneumatic tired roller for the driest fill.

6.8 Venezuela

The Las Cristinas mining prospect is currently under development in eastern Venezuela by Placer Dome Technical Services Inc.. A test embankment was constructed as part of the design process for flood control dykes built on soft, saturated tailings (Bruce Geotechnical, 1995). The properties of the soils in this embankment are listed in Table 4. The embankment was constructed during intermittently rainy weather. Fill was delivered to the site using rear wheel drive tandem dump trucks. Trafficability of the dump truck construction equipment became impossible when the fill moisture content was greater than 35%. At this moisture content deep ruts developed in the fill. Compaction was undertaken using various combinations of truck traffic and smooth drum rollers. Pore pressure dissipation was rapid in the fill but pore pressures in the saturated tails rose rapidly and dissipated very slowly. The embankment failed at a height of 11.5 m. Distress in the embankment was attributed to foundation failure.

7. UNCONVENTIONAL ASPECTS OF COMPACTION WITH RESIDUAL SOILS

7.1 MICROSTRUCTURE / AGGREGATE BEHAVIOUR AND BREAKDOWN

Many of the laterites and andisols are subject to breakdown with manipulation (Mitchell and Sitar, 1982). The strength and compressibility of compacted saprolites are closely related to their percentage of soft rock particles that remain intact in the fine soil matrix (Mori et al, 1979).

As discussed in section 4.8.2, reusing a sample in a Proctor test may lead to a prediction of a lower optimum moisture content compared to the less worked natural sample.

Nogami et al (1985) discuss the case of two lateritic soils compacted using the standard Proctor procedures. With each cycle of compaction the physical properties of the soil were altered. The properties that were altered were the uniformity coefficient, clay content, cohesion, and maximum dry density, which increased, and the liquid limit, friction angle, and permeability which decreased. Saprolitic soils were also shown to be affected by compaction cycles but with greater variability due to the more complex structure.

A soil with a high water content may have a high in situ strength due to its bonded structure, yet may have a low remolded strength. These soils will lose their structural strength with reworking during excavation, placement, compaction, and bearing traffic. This is demonstrated in Figure 28, which shows a loss of strength with repeated compaction. These soils may require special methods of construction to optimize performance, as discussed in Section 7.7. These soils may present problems when encountered in sub-base for roads and in fills.

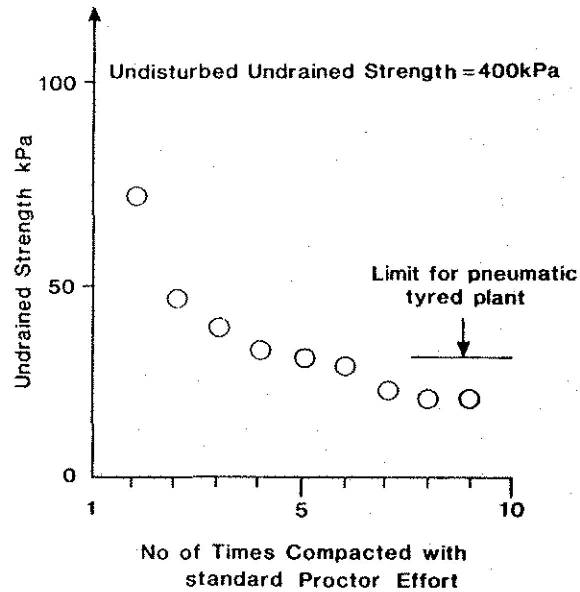


Figure 28 - Effect of repeated laboratory compaction on undrained strength of porous residual soil from basalt from Mauritius (from Fookes, 1997).

This problem can be evaluated by determining the remolded strength at the highest water content that is likely to exist. Due to seasonal fluctuations in moisture content it may be prudent to consider the water content based on full saturation of the in situ void ratio.

The breakdown of microstructure with the application of compactive forces beyond a certain level has been reported for many soils. Mitchell and Sitar (1982) report that overcompaction may transform a predominantly granular soils to a "sticky mass that cannot be handled easily".

Mitchell and Sitar (1982) hypothesize that excessive field compactive effort results in a decrease in the optimum water content relative to already high natural water contents, which may make further work difficult. In the same text Mitchell and Sitar report from work by Gidigasu (1972) that the optimum moisture content increases with increasing clay content.

At Arenal dam, Costa Rica, field compaction trials described by Rodda et al (1982) showed evidence of decreasing density with increases in compactive effort. The density relative to standard Proctor maximum dropped 2 percent when the number of passes with a pneumatic roller was increased from 2 to 5. A vibratory roller showed a drop of 3 percent in relative density when the number of passes was increased from 2 to 4. When compacting at the natural moisture content any attempt to increase the density beyond the 90% saturation line with additional compactive effort resulted in poor fill performance.

Kuno et al (1978) found that for a volcanic soil in Japan that applying more than 2 to 3 passes of an 11 to 13 ton bulldozer, or more than 4 passes of a low ground pressure bulldozer, resulted in a loss of strength in the compacted fill.

At Miraflores dam, Colombia, Li (1967) considered overcompaction and the "skip graded" soil to be significant factors in the generation of high pore pressures in conjunction with the fast rate of construction.

7.2 Dilatant Behaviour

Hundreds of triaxial tests on compacted soils from Brazil have been summarized by da Cruz et al (1985). Figure 29 shows typical stress-strain and pore pressure-strain relationships from these tests. The Type III tests for low confining pressures have pore pressure responses that show an initial increase, then decrease with strain, even becoming negative, which is indicative of dilatant behaviour. This behaviour favourable for earthworks as the mobilization of shearing resistance occurs at lower pore pressures that tend to decrease during the failure process. Dilatant behaviour was reported for the deformation in the Punchina cofferdam as discussed in Chapter 6.

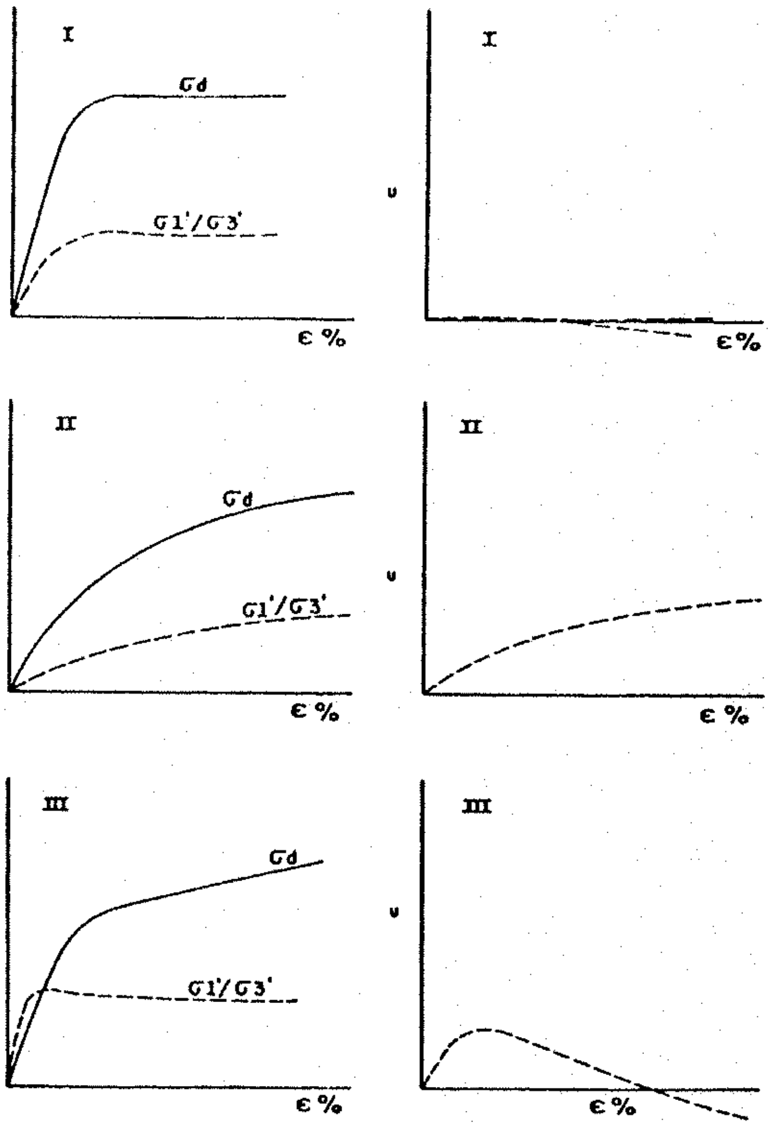


Figure 29 - Three of the typical stress-strain curves and pore pressure-strain curves from hundreds of tests on compacted soils of varying origin from Brazil (from da Cruz et al, 1985).
Type I – CD tests for any soil for any pressure range;
Type II – characteristic of samples at or above optimum in CU or UU tests with a confining pressure above 600 kPa;
Type III –characteristic of compacted and saturated soils, CU and UU for cell pressures less than 300 kPa.

7.3 Mineralogical Effects

7.3.1 Halloysite

The water layer present in halloysite is removed irreversibly by heating or air drying, leading to the formation of metahalloysite. This change is partly responsible for the unusual characteristics of some soils.

The engineering properties of halloysitic soils are good, despite a high clay fraction, and fairly high values of natural water content in terms of the Atterberg limits (Blight, 1997). The halloysitic soils are commonly described as tropical red clays, Latosols, Oxisols, or Ferralsols.

7.3.2 Allophane

Allophonic soils are probably the most distinctive of all residual soils due to the very unusual properties of the amorphous mineral allophane. The influence of allophane is both dramatic and puzzling, in that it results in soils having natural water contents ranging form about 80% to 250%, but which still perform satisfactorily as engineering materials (Blight, 1997). They are commonly much superior to other soils with water contents of only a fraction of the above values.

Allophanic soils always appear to be associated with volcanic ash soils (Andisols, Andesols, Andepts) as parent material. These soils show considerable variation in the degree to which they are affected by microstructure, which is reflected in a wide variation in sensitivity.

7.3.3 Sesquioxides

Sesquioxides affect the behaviour of soils by acting as a cementing agent to bind clay minerals into aggregates or clusters.

7.4 Heterogeneity

In situ layers of residual soils may be extremely heterogeneous laterally and vertically due to the high geochemical activity and heterogeneity in the parent rock. The nonhomogeneity of saprolites is also due to variations in relict structures.

Drying may cause changes in the properties of soils during sampling and testing, as well as in situ due to local climate change, drainage, and soil profile distribution (Mitchell and Sitar, 1982). These changes contribute to heterogeneity within the soil mass.

The heterogeneity of many residual soils in situ implies that care is required when developing borrow areas and determining compaction requirements.

7.5 Optimum Moisture Content Determination

The optimum moisture content measured in the laboratory may not be representative of the optimum moisture content for the field conditions. The relevance of laboratory compaction to field compaction has long been discussed, mostly due to differences in compactive forces between the field equipment and the hammer used in the standard Proctor test. Microstructure in a compacted sample will affect the strength in that samples compacted with different compaction methods at the same moisture content and to the same dry density exhibit different UU behaviour under identical test conditions (Morgenstern et al, 1977). The effect of microstructure is strongest when the compaction moisture content is above optimum and strength is defined as that developed under small strains. Tropical residual soils also have differences between laboratory results and field compaction which arise from other sources such as mineralogy.

At Arenal dam in Costa Rica the standard testing procedure was modified to better correspond to the field conditions. It was found that by increasing the maximum particle size in the laboratory test to 1.9 cm (3/4 in.) and drying the sample back to each compaction moisture content, as opposed to drying and then rewetting for each test, that the measured optimum moisture content increased by 4.8% (Rodda et al, 1982).

"The compaction with hammer blows creates an activation of the clay content produced by the breakdown of the relict rock structure that will not be so hardly processed in the field compaction (Mori et al, 1979).

Some effects of sample preparation on compaction results can be seen in Figure 30. Oven dried samples tend to indicate lower optimum moisture contents and higher dry densities than samples which are air dried from the natural moisture content to the test moisture content. Reuse of a sample for all the test points leads to particle breakdown and higher maximum dry densities and lower optimum content.

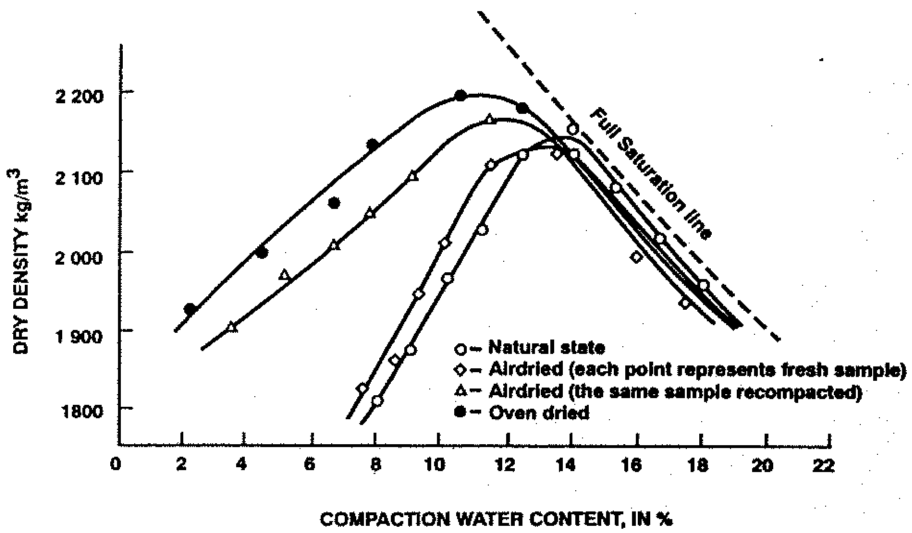


Figure 30 - The influence of method of sample preparation and laboratory procedure on compaction characteristics of lateritic soil from Ghana (after Gidigasu, 1974; from Blight, 1997).

7.6 Compaction Control

Specifications for compaction commonly prescribe either performance requirements or compaction methods. Performance based specifications typically restrict the allowable moisture content for the fill to a percentage of the Proctor optimum moisture content. Similarly, the required density is prescribed relative to the Proctor maximum density. For example, a common specification would require 95% of Proctor maximum density compacted at a moisture content within ±2% of the optimum moisture content. A performance specification is easy for the designer and allows for flexibility in the field to respond to changing conditions. The disadvantage of a performance specification is the large amount of testing which is required, which is standard practice on a large project but may be onerous for a smaller project. A performance specification for a large project should include an allowance for testing errors and material variability. Performance specifications are sometimes criticized because the designer writer yields decision making capacity to the field inspector (Bazett, 1993).

In a method specification the range of acceptable moisture contents is specified along with the number of passes and type of compaction equipment. An example method specification is 4 passes of a sheepsfoot roller of a specified size, at the natural moisture content. This type of specification requires that the designer has a good knowledge of the soil conditions prior to writing the specification. This type of specification requires less testing and limits the field interpretation required. The construction of a trial embankment can be used to optimize compaction methods and equipment and to finalize the specifications. Caution must be exercised in the use of trial fills to ensure that the soil is sufficiently uniform that the behaviour of the trial embankment is truly representative of the entire borrow area. De Mello (1975) describes how an overly conservative analysis of a test embankment led to design recommendations for the Tres Marias dam in Brazil that were later found unnecessary.

Blight (1997) outlines five basic soil parameters which can be monitored for compaction control:

- In situ density as compacted;
- In situ moisture content as compacted;
- In situ strength as compacted;
- In situ permeability as compacted;
- Laboratory strength properties correlated to in situ measurements.

The first three of these control parameters are shown on a standard density-moisture content plot in Figure 31. It can be seen that it is necessary to specify more than one parameter in order to completely bound an acceptable range.

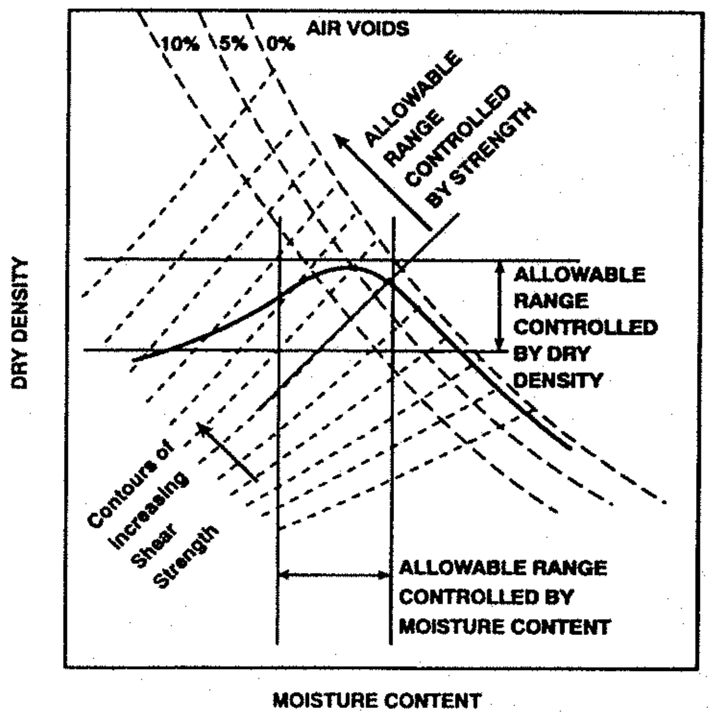


Figure 31 - Selection of compaction control methods to define acceptable range of end product (from Blight, 1997).

In situ density has traditionally been the most common control parameter, partly due to its long history of use. In theory, measuring density is simple, but in practice it may be subject to procedural errors. Measuring volume may be time consuming and nuclear densometers and moisture meters may be subject to errors in residual soils. Density offers an indirect assessment of the strength, stiffness, and permeability of the fill. Interpretation of the in situ density is usually done by correlation with laboratory tests. A sufficient amount of testing must be done to ensure that material variability is accounted for and the field measurements are interpreted correctly.

The use of a nuclear densometer is rapid and convenient for measuring density but calibration is critical and can be difficult. The mineralogy of some tropical soils can cause an offset in the nuclear density readings which is locally consistent but varies over a project site (Bruce Geotechnical, 1995). At Arenal dam in Costa Rica density measurements using the sand cone method were found to overestimate the compaction due to squeezing in of the holes. This was overcome by using driven steel tube samples.

In situ moisture content is easily measured and can be used as a control parameter if correlated with strength and permeability. Strength and permeability are also a function of the density, and thus another control parameter must also be specified.

The shear strength of the compacted fill is generally a direct indicator of performance. The undrained shear strength can be measured rapidly using hand held vane shear devices. Because of the shear strength's dependence on moisture content the two are often specified together. Blight (1997) states that in situ strength is often the most appropriate control parameter for residual soils. Trafficability is an indirect assessment of the shear strength.

The permeability of a fill is also often a direct indicator of performance. However, its measurement is prone to procedural errors and scale effects. Also, it is not easily correlated with shear strength, which must also then be tested for.

The laboratory undrained shear strength is often correlated to the in situ moisture content. This method of compaction control works well when the material variability is well understood.

According to Wesley (1988), traditional compaction control using optimum moisture content and maximum dry density is unsuitable for residual soils because of the inherent heterogeneity in residual soils, and because some of the compaction curves, notably volcanic ash soils, do not show distinct peaks. Wesley (1988) describes a method of compaction control which is widely used in New Zealand and has been used on many projects in Southeast Asia. This control method specifies a minimum undrained shear strength, commonly 100 kPa to 150 kPa, and a maximum percentage of air voids, commonly 8% to 12%. Figure 32 compares this method with a standard performance based specification. In this example the performance based range of acceptable compaction is 95% or greater of Proctor maximum density, and the range of moisture content is –2% to +1% of optimum moisture content. The range of acceptable compaction from strength and air void criteria are shown stippled. The ranges in this example assume that soil wetter than optimum +1% does not meet the minimum strength criteria. In applying this method, shear strength measurements are made using a hand operated vane shear device. The percentage of air voids are calculated from density and moisture content measurements. Using this method the actual optimum moisture content and maximum dry density do not need to be determined. These standard compaction tests are often done, however, for record keeping and to determine if the soil will require wetting or drying for the best compaction.

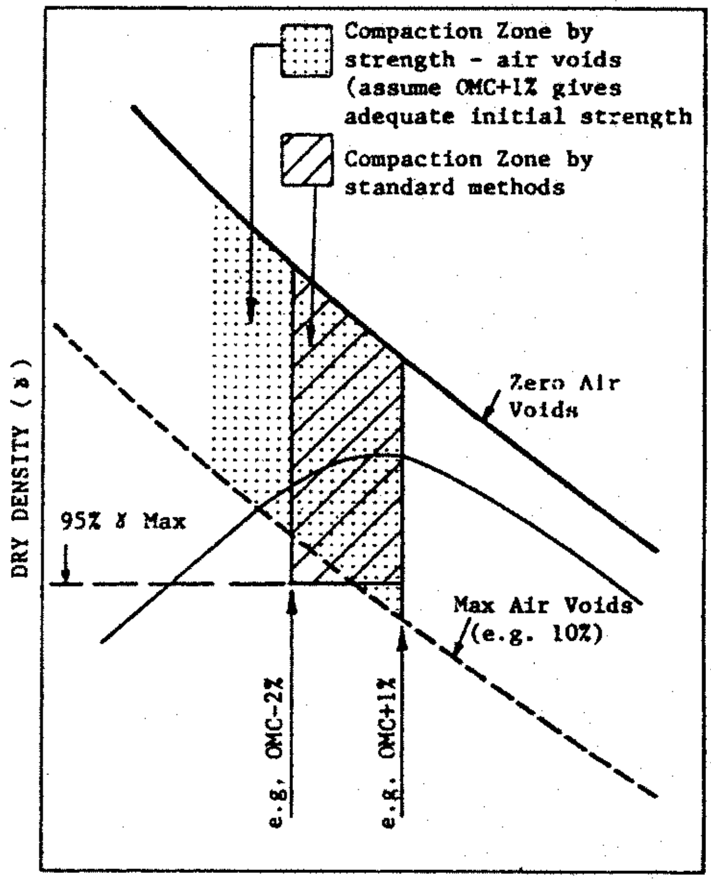


Figure 32 - Comparison of specification types for compaction control (after Pickens, 1980; from Wesley, 1988).

A comparison of traditional compaction control and the methods used for some projects in areas of high precipitation in Colombia are shown in Figure 33 and described by Li (1967). These projects are described in greater detail in Section 6.3. The specifications called for the maximum density attainable for a wide variety of borrow moisture contents rather than a specific density range. The compaction equipment used in the field was dictated by the moisture content of the fill. The maximum density achievable was determined based on trafficability of the compaction equipment.

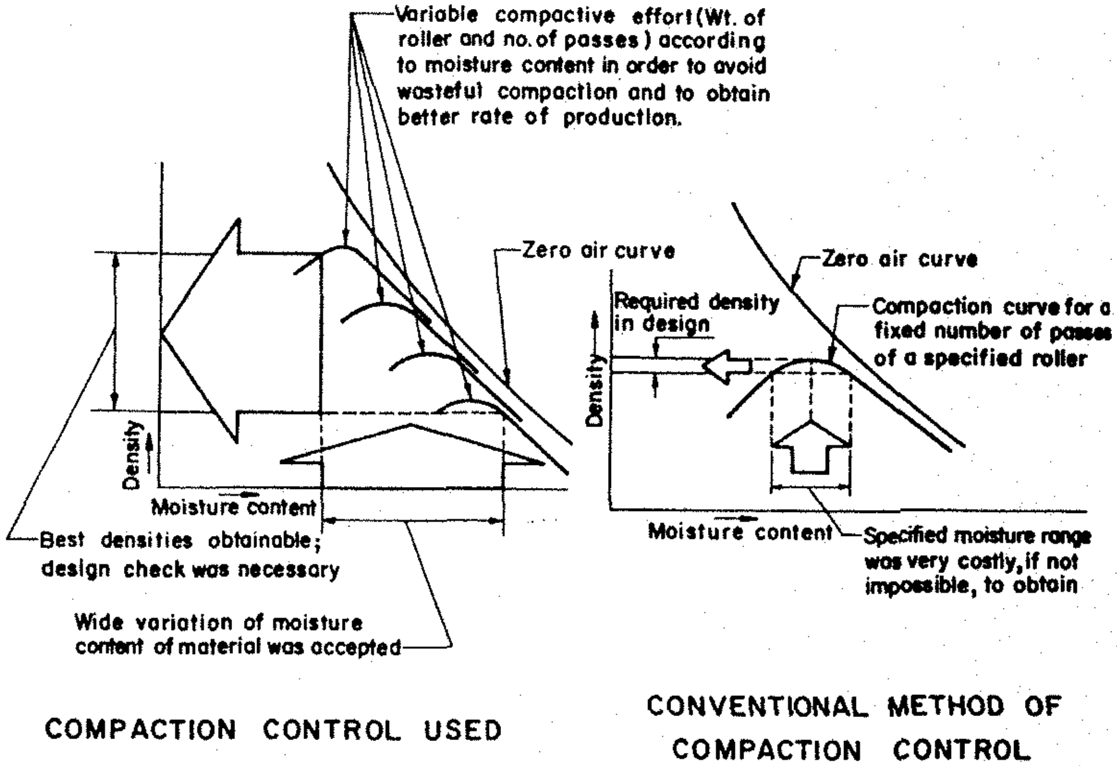


Figure 33 - Compaction control methods used for three dams in Colombia compared with conventional compaction control (from Li, 1967).

In reviewing Brazilian experience, de Mello (1975) notes that if the borrow material was found to expand in the Proctor mould then the material is rejected. Field compaction control is conventional, stating that the dry density of the compacted material should be equal to or greater than 95% of the standard Proctor maximum.

Some Brazilian researchers prefer the concept of a "workability water content" for saprolitic soils to replace the conventional optimum water content criteria (da Cruz et al, 1985). The workability water content is the water content at which compaction equipment is able to travel on the fill.

During construction of Monasavu dam in Fiji the initial specification based on compacted density relative to standard Proctor maximum was changed to a wholly method based specification. This specification designated that compaction was to be undertaken with a rubber tired roller when possible, and by bulldozers otherwise. The dry density was monitored using core cutters to measure the density.

Specification in which the degree of compaction is evaluated in terms of the degree of saturation is popularly adopted in Japan for earthworks with overly wet volcanic soils (Kuno et al, 1978). Typically the specification is that the degree of saturation of the compacted fill be within 85 to 95%. A maximum water content is also specified to ensure the mobility of the construction equipment.

7.7 Construction Methods

At Arenal Dam in Costa Rica compaction equipment with low ground contact pressure was required. The wet core material was compacted using two passes of a D-6 bulldozer followed by 2 passes of a vibratory roller to seal the surface against infiltration. The bulldozer did not attain the highest compaction but "did not damage the fill surface" (Rodda et al, 1982). A tamping roller "failed" the surface of the fill with rutting and shoving the fill, as did pneumatic rollers, although to a lesser extent. Compaction equipment with pneumatic drive plants had difficulty maintaining traction at times. Stripping of the borrow areas was limited to two days worth of fill. Surface runoff in the borrow areas was strictly controlled. Excessively wet material was disposed of. The surface was crowned heavily and no fill was placed during periods of rainfall.

At Monasavu Dam in Fiji the borrow areas were developed by bulldozers operating on slopes of approximately 4 horizontal to 1 vertical (Knight et al, 1982). This slope was steep enough to allow good drainage. The fill material was hauled to the dam by 20 t dump trucks which backed onto steel framed mats with timber running surfaces. Spreading was done by low ground pressure Caterpillar D-6 bulldozers in 100 mm layers. Rain infiltration of the core was minimized by maintaining a 10% fall to the core, running the bulldozers parallel to the core crest, and removing water from the tracks by hand. Occasionally tarpaulins were also used.

Figure 34 depicts the method adopted for construction of a highway embankment in Japan for haul distances of greater than 300 m (Kuno et al, 1978). The methods adopted for this project were designed to limit the remolding of the extremely wet and sensitive soils. Haul roads of 4 to 5 m in width were constructed of well graded gravel, sometimes reinforced with mats, to allow the dump trucks to reach the spreading bulldozers.

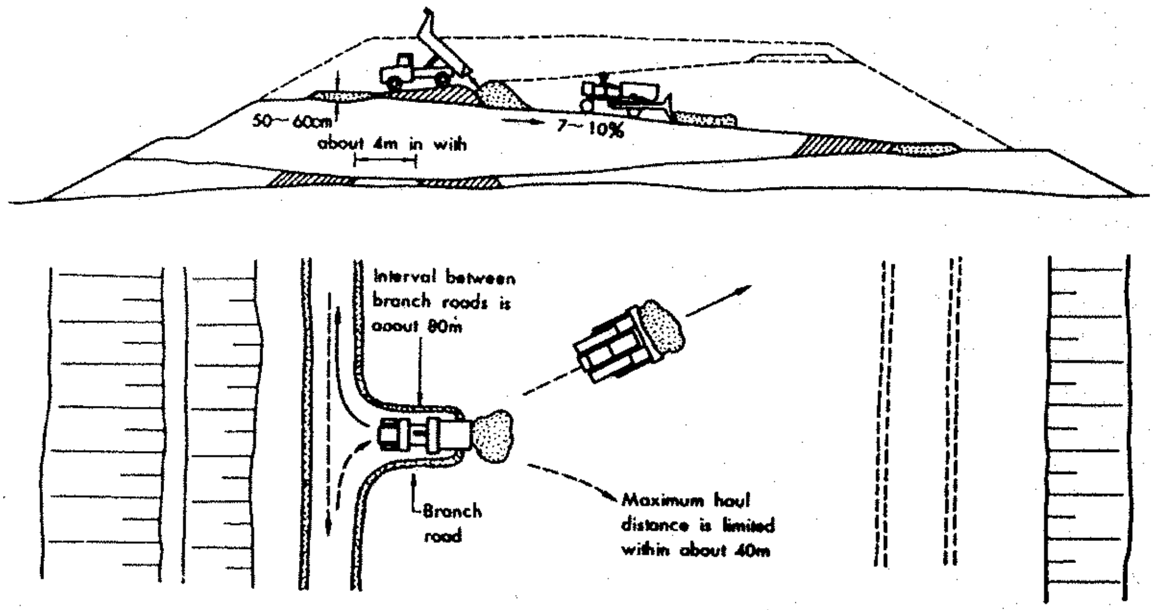


Figure 34 - Layout of hauling and spreading operations for a highway embankment in Japan (from Kuno et al, 1978).

The construction of the Santa Rita dam in Colombia during extremely wet weather is described in Section 6.3.2. In order to provide working areas for fill placement the contractor sometimes found it necessary to mix in "decomposed rock" material with the previous lifts. This was done with Caterpillar D6 and D8H bulldozers. Silt was then placed on the stabilized surface. Compaction was done with self-propelled tamping rollers (Caterpillar 825B, weight of 29 t), and followed by sheepsfoot rollers, weighing 32 t, if possible. If the fill was too wet for the sheepsfoot rollers then low ground pressure D8H bulldozers weighing 22 t were used. The ranges of moisture contents over which each method was used and the densities that were achieved are shown in Figure 26. The haul roads required surfacing with weathered and hard rock in order to minimize the time between the rain stopping and the commencement of fill placement. During dry periods all of the work activity was directed towards fill placement, otherwise work was directed towards road maintenance and the more workable zones of the embankment such as the filters and decomposed rock zones. Drainage of the borrow areas and fill surface was carefully maintained.

At Salto Osorio dam, Brazil, it was found necessary to restrict the loads and tire pressure of the standard earth moving equipment in order to avoid excessive lamination of the fill (de Mello, 1975). Mori et al (1979) recommend the use of a roller for compaction as it "minimizes the breakdown of the saprolite soil blocks".

Some construction sites have reported problems with borrow pits being excavated below the water table, unbeknownst to the construction crew.
De Mello (1989) recommends that saturation testing be done as part of borrow source investigations.

8. CONCLUSIONS

Several dams and embankments have been constructed around the world with fine grained tropical residual soils, many with natural moisture contents slightly to well above the standard Proctor optimum moisture content. Nevertheless, the fills have generally been considered to perform well. Soil properties, natural moisture contents, and optimum moisture contents gathered from 32 projects that exhibit adequate strength and deformation properties, are summarized in Table 4.

The good engineering behaviour of residual soils compacted at moisture contents above what would commonly be considered acceptable for transported soils, may be due to a number of factors. These include material properties unique to residual soils, specialized compaction control methodologies, and construction techniques modified for tropical soils and climates, as noted below.

1) Material Properties

- Although a soil may have a significant proportion of clay minerals, the soil is aggregated by relict bonds from the rock structure and by cementation developed during the weathering process, primarily by sesquioxides. The soil therefore does not behave as a saturated clay unless broken down, as the excess water that is trapped within the aggregated structure affects the moisture content but does not impact the engineering properties;

- Some soils rich in halloysite and allophane, such as the volcanic ash soils, have extremely high moisture contents. However, the water is incorporated into the clay structure, and thus has a limited impact on the shear strength;

- The soils often perform better than would be expected based on experience from temperate soils because many of the tropical residual soils have high friction angles as a result of mineralogy and aggregation;

- Some of the aggregated soils tend to behave in a dilatant fashion, particularly at lower confining pressures, which limits the generation of pore pressures with shearing, even in soils with high natural moisture contents;

- The moisture content measured at the borrow area may be overestimated because of test procedures, such as oven drying, although this is likely offset by an equal error in determining the moisture content during compaction testing.

2) Compaction Control

The actual optimum moisture content acting in the field may be greater than the optimum moisture content measured in the laboratory because in the ASTM standard Proctor test procedure:

- drying of the sample prior to testing causes changes in the structure of the clay minerals so that the sample tested in the laboratory is no longer representative of the soils in the field;

- pulverizing the sample prior to testing breaks down bonds within the soil matrix, thus "activating" the clay;

- the compactive effort used in the standard proctor test is not representative of the compactive effort in the field.

Various methods of compaction control other than density testing have been used. Compaction monitoring using undrained shear strength and trafficability has the advantage of being directly linked to fill performance and avoids the difficulties inherent in measuring density and moisture content and correlating it with performance.

3) Construction techniques

Working with wet soils in climates where drying is not feasible has necessitated the development of some innovative construction procedures. In order to obtain the maximum compaction possible at a given natural moisture content the compaction equipment is often varied in response to the soil conditions. Low ground pressure tracked equipment is often used for the compaction of material too wet for conventional equipment. Care is taken to avoid overcompacting the soil to avoid the loss of strength and density which results from breaking down the soil aggregates. Although this may not achieve the conventionally required density relative to the standard Proctor maximum the end result is a stable, useable structure. Great care is taken to limit infiltration at the borrow area and on the fill surface.

9. RECOMMENDATIONS

Based on this review of the literature the following methodology for undertaking earthworks in tropical residual soils is recommended. The intent is for all of the measurements to be indicative of the field behaviour and correlated with fill performance.

Investigation

- Assess the mineralogy. Determine if minerals which are known to impact the engineering behaviour of soils, such as halloysite, allophane, and sesquioxides, are present. This may be done using x-ray difraction, scanning electron microscope, various chemical tests, or by interpreting field behaviour or agricultural maps.

- Assess the microstructure. Relict rock bonds and aggregation influence the engineering properties. This can be determined by comparing grain size distribution before and after compaction, and by evaluating the sesquioxide content.

- Assess the macrostructure. Determine if relict discontinuities will impact on the borrow area or foundation stability and permeability.

- Assess the weathering distribution. Determine the soil types present and their distribution. Contacts between laterites, mature soils, saprolites, and rock may be irregular and structurally controlled, making borrow resource estimation and exploitation difficult.

Testing

- Assess the suitability of ASTM standard tests. Moisture content measurement, Atterberg Limit testing, and Proctor compaction testing may all be affected by changes in the samples resulting from drying or manipulation. The use of nonstandard procedures, such as air drying samples to the test moisture content, avoiding reusing samples, or testing using various compaction efforts at the natural moisture content, may give results which are more indicative of the soil's actual properties.

- Consider the use of strength index tests, possibly vane shear tests, to assess soil suitability.

Design / Specification Writing

- Review relevant literature and assess local experience.
- Ensure that performance based specifications are representative of the field conditions. Consider method based specifications.
- Construct a test fill to evaluate fill behaviour and determine the most effective method of compaction. Verify that performance based specifications can be obtained.

Construction

- Consider varying compaction equipment to respond to soil conditions.
- Limit infiltration of precipitation at the borrow site and the embankment.
- Place fine grained fill during the best weather and granular material during precipitation. Maintain flexibility in the construction schedule to accommodate soil and weather conditions.
- Avoid overcompacting and breaking down aggregated soil.

Compaction Monitoring

- If density and moisture content are measured as control parameters ensure that they are correlated with performance.
- Consider the use of shear strength and moisture content as compaction controls as they are directly linked with performance and may be less prone to errors in measurement than density in residual soils.

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APPENDIX 1

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