TROPICAL RESIDUAL SOILS AS DAM FOUNDATION AND FILL MATERIAL

ICOLD COMMITTEE ON MATERIALS FOR FILLS DAMS
JANUARY 2009
BULLETIN No.___________

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TABLE OF CONTENTS

1 INTRODUCTION

A crucial component in the design of dams is the knowledge acquired by precedent. Extrapolation of this precedent in dam construction can only be utilized properly and successfully through a clear understanding of the conceptual assumptions, local conditions and construction methods. A clear difficulty to take advantage of precedent in many dam projects located in tropical regions is the presence of a particular type of soils: residual soils.

Unlike transported soils, residual soils originate from the in situ weathering and decomposition of the parent rock, which has not been transported from its original setting. The conditions present in humid and tropical climates provide the adequate moisture and temperature conditions to transform, through weathering processes, the underlying rocks into residual soils faster than they can be removed by erosion. Typically these regions have a high potential for hydraulic projects, either for hydropower, or flood control and irrigation. Figure 1-1 shows distribution of residual soils throughout the world.

Figure 1-1 Simplified world distribution of tropical residual soils (based on F.A.O)

Dam projects located in these areas, can encounter thick layers of residual soils at their foundation. For economical reasons, this material can also be used as fill material. The main difficulty in dealing with these soils for engineering purposes is that their characteristics are very different from those of transported soils (Blight 1997). Furthermore concepts and methodologies typically used in geotechnical engineering were developed in regions dominated by temperate climates. As a consequence many of the concepts of soil and rock behavior and properties, and geological conditions have been conditioned by earth materials found there. Despite these inconveniences, the widespread presence of residual soils does not allow us to avoid them, but challenges us, to deepen our understanding and knowledge about them, to be able to accept and employ them to our advantage in future projects. Success on this challenge is determined by a proper understanding and appreciation of the engineering properties of these soils and weathered rock encountered.

The purpose of this bulletin is to produce a state of the art report on "Residual soils as Dam Foundation and Fill Material for Dams". The intention is not to provide detailed geological descriptions or characterization methods. The geologic literature offers a plethora of articles on these issues. The bulletin rather focuses on the dam engineering implications of dealing with residual soils. To understand the properties of tropical soils requires rigorous study of their composition and structure, both of which are unique consequences of the climate conditions that prevail during their formation. Therefore, the formation, composition, and the micro and macro structures of tropical residual soils are reviewed first follow by a description of the unique aspects of soil profiles, sampling and testing, engineering properties, design criteria, construction techniques, and registered behavior. Finally an extensive gathering of case histories around the world of dams built on or with residual soils are presented to illustrate these topics in a pragmatic matter.

Hopefully the material included in this report will provide valuable information for dam engineers to design safer, good performing, more efficient and economical dam structures were residual soils are present.

2 RESIDUAL SOILS

2.1 ORIGIN

Unlike transported soils, mainly derived from alluvial, lacustrial, marine, aeolian or glacial depositional processes; residual soils originate in-situ. The formation of residual soils is controlled by other factors besides rock itself. Local climatic conditions and topography are the most relevant aspects, combined with other variables that provide certain environmental conditions for the development of residual soils from the parent rock to induce chemical (decomposition), physical (disintegration) and biological weathering processes.

Residual soils are mainly derived from metamorphic and igneous rocks, but residual soils developed from sedimentary rocks (e.g. shales and loess) are not rare.

2.1.1 Weathering Processes

Weathering results from the reaction of pre-existing materials to different conditions of pressure, temperature, fluids and stress regimes near the ground surface. In tropical regions of high temperature and abundant rainfall, rock chemical weathering is intensive. It is characterized by the rapid breakdown of the feldspars and ferromagnesian minerals, the removal of silica and bases (Na_2O, K_2O, CaO, MgO), and the concentration of iron and aluminum oxides. The process is often termed laterization, and the chemical reactions involved have been extensively described in the literature (Grant 1974, Gidigasu, 1972 and others). Usually physical weathering precedes chemical weathering. Exogenetic (climate and vegetation) and endogenetic (structure and composition of the material) determine the extent and the rate of weathering (Birkeland, 1984). Stress released by erosion, salt crystallization, differential thermal strain, swelling and ice are the most noticeable physical weathering agents. They expose fresh surfaces to the chemical agents and increase permeability allowing the flow of chemically active substances (Blight 1997).

2.1.1.1 Physical Weathering

Physical weathering terminates in the collapse of the parent material and the diminution of its grain size, as rock disintegrates and stress is exerted along planar structures (bedding, or fractures) by expansion of the rocks or minerals themselves, or by foreign agents in the voids (Birkeland, 1984). As depth increases, physical weathering is less effective due to the increase in confining stresses that inhibit the formation or opening of fractures.

2.1.1.2 Chemical Weathering

Due to constantly changing conditions at the Earth's surface, rocks and minerals decompose to reach equilibrium under new environments, altering original materials and generating new minerals (Birkeland, 1984). Oxidation, reduction, hydrolysis, solution and cation exchange play an important role in the chemical processes leading to chemical weathering. Residual Soils that are the product of chemical weathering are of great importance to earth-dam engineering. These type of soils are encountered more commonly that physical weathering soils and usually contain a considerable amount of clay minerals, which determine to a considerable extent their engineering properties. The clayley residual soils produced by chemical weathering are most extensively developed in tropical and semi tropical regions.

Biological weathering comprises both physical and weathering processes. Physical, in the form of splitting by root wedging, and chemical as bacteriological oxidation.

2.1.2 Factors affecting weathering processes

The parent rock involved is a crucial factor to determine what type of process or agent has a stronger weathering influence. Physical processes usually control weathering of sedimentary and metamorphic rocks and chemical processes dominate the weathering of igneous rocks. Although there might be a predominant process, there is always a presence and influence of the other weathering factors.

Factors affecting weathering can be divided in macro - factors and micro - factors as follows (based on Blight 1997):

Macro - factors	Micro - factors
Climate (moisture, temperature)	Lithology
Relief (topography)	Structure
Lithology. Goldich's Series (weathering susceptibility series)	Fluctuations of water table in recent geologic time
Time	

Table No. 2.1 – Macro and Micro-Factors Affecting Weathering

Among the local variables, climate is the one with the higher influence in the type and rate of weathering present in a certain region (Blight 1997, Weinert 1974). Physical weathering is prevalent in dry climates, decreasing in influence as higher moisture and temperature promote biological and chemical weathering.

Strakhov (1967) proposed a diagram relating the effects of global climate on the parent material (rock) weathering and the development of clay minerals and other weathering products (see Figure No. 2.1.) The model does not include local climatic variations or topographical influences, but nevertheless is a good approximation on the importance of climate in the weathering processes. It is important to note that even though general models are valuable, the evaluation of specific local and regional conditions is fundamental to understand the processes and effects of weathering at a particular site. It can be seen that at the tropic forest zone, high moisture and temperatures result in an extensive and rapid weathering and leaching of mobile constituents.

The unique aspects of profile descriptions, sampling and testing, engineering properties, design criteria, construction techniques, and registered behavior are outlined.

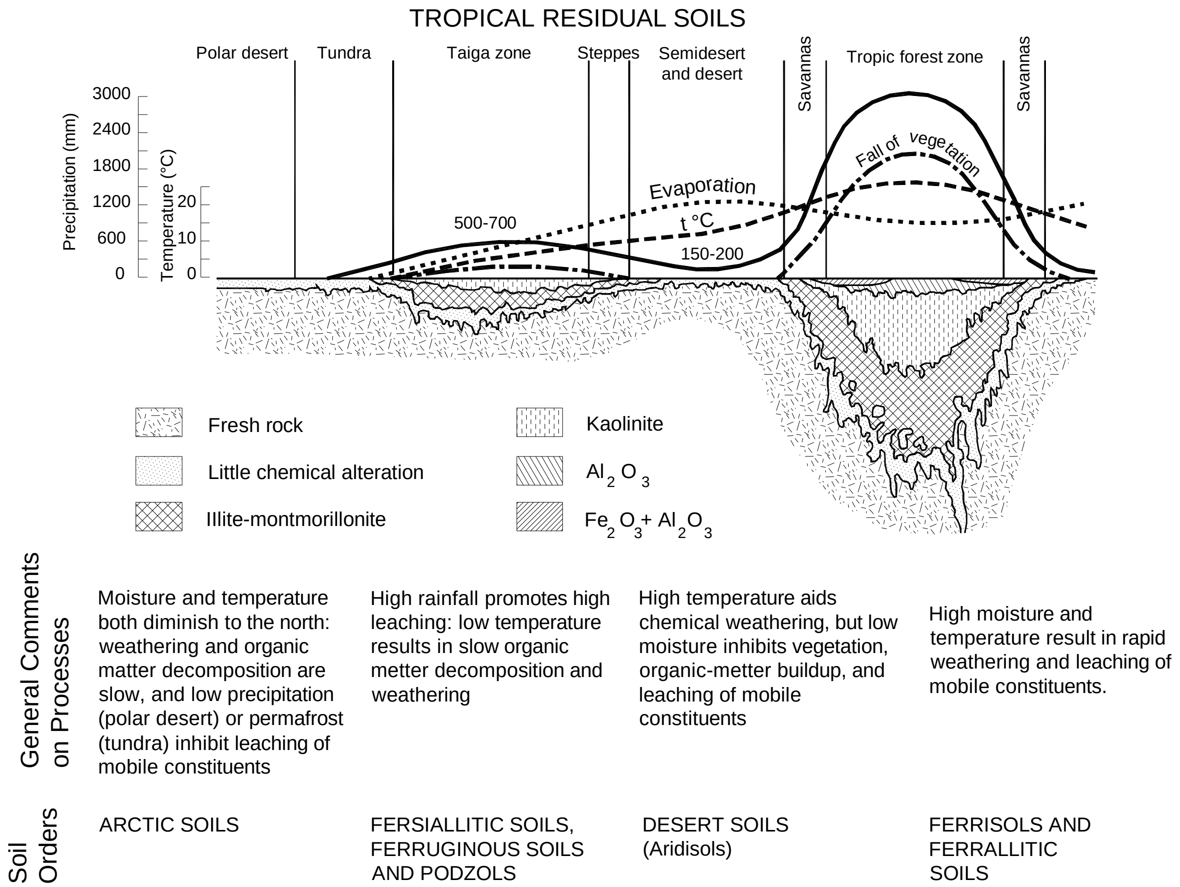

Figure No. 2.1 - Depth of Weathering Relative to Some Environmental Factors (After Strakhov, 1967)

The development of residual soil profiles is restricted by erosion removal rate. The weathering rate must be higher than the removal of weathering products. Residual soil profiles will generally be encountered in valleys and gentle slopes, rather than on steep slopes (Blight 1997). Studies (Van der Merwe, 1965 and Fitzpatrick et al, 1977) have shown that samples taken on high grounds and steep slopes have a greater concentration of kaolinite, while down slope a higher percentage of montmorillonite (smectites). Landsliding and general mass movements, promoted by steep slopes also arrest the development of deep residual profiles.

2.2 TERMINOLOGY

For the purposes of this bulletin laterites and saprolites are defined as follows:

2.2.1 Laterites and lateritic soils

Several definitions for Laterites have been proposed, and currently there is still no general consensus. In 1807 Buchanan (reference) defined a laterite as a soil with no stratification, presenting large amounts of iron, forming red or yellow “ocres”, cuttable when wet, but when air dried hard like a brick. The hardened crusts characteristic of some laterites have been associated with a general group named duricrusts (Fookes et al, 1997). In general terms lateritic soils are yellow and red tropical soils and concretionary hardened materials, part of the superficial soil horizon (Melfi, 1985). In Deere and Patton’s (1971) weathering profile, laterites and lateritic soils are generally part of zone IB (See Chapter 3).

Some engineers and geologists define laterites through low silica-sesquioxides relation. The definition of laterites through low silica-sesquioxides is represented by the molecular ratio of silica/Al-Fe sesquioxides (Kr) which is not frequently used by engineers because of the analytical procedure involved.

Sandstones and granites are the most common rocks subject to lateritization.

2.2.2 Saprolites

Saprolites are weak, friable, chemically weathered material that preserves the original structure and texture of the parent rock, in spite of the decrease in strength and replacement of original materials (mainly by clay). Despite having the original structure and fabric of the parent rock, the mechanical and engineering behavior of saprolites are very different as discussed in the following chapters. Saprolite or saprolitic soils characteristically present an early stage weathering process.

Their parent rocks are either metamorphic or igneous volcanic and sedimentary. In the weathering profile proposed by Deere and Patton (Chapter 3) correspond to horizon IC and the transition zone II A. In some cases their in-situ thickness may exceed 10 m, depending on the parent rock material and local conditions.

For the purposes of this bulletin a material is considered saprolitic if it is soil in the geotechnical sense. This differs from certain geological definitions.

3 RESIDUAL SOIL PROFILE

Physical-chemical factors affecting weathering are more readily present at the surface Therefore, the extent of weathering usually decreases with depth. In order to understand the engineering significance of the gradual transition of the physical properties of materials subjected to weathering, various authors have proposed typical weathering profiles. Among the most widely recognized, Deere and Patton's (1971) model is utilized for purposes of this bulletin. The profile is subdivided in three major zones: (I) residual soil, (II) weathered rock and (III) relatively unweathered and fresh rock. Zone I, the residual soil, is further divided in three, according to the pedologists classification: IA (Horizon A), IB (Horizon B), IC (Horizon C). In some instances, this model does not fulfill the requirements for an adequate description, especially if there are pronounced lateral variations. Table No. 3.1 presents some of the more relevant characteristics of the different zones of the weathering profile. Figure No. 3.1 illustrates a typical weathering profile for igneous and metamorphic rocks.

Zone		Description
I. Residual Soil	IA Horizon	Topsoil, roots, organic material, zone of leaching and eluviation. May be porous. Sandy textures. Medium to high permeability. Low to medium strength.
	IB Horizon	Characteristically clay enriched; accumulation of Fe, Al, and Si. May be cemented or susceptible to irreversible hardening. No relict structure present. Deposition of solid materials from IA. Low permeability and strength (high if cemented).
	IC Horizon	Relic rock structures retained; silty grading to sandy material; less than 10% corestones. Often micaceous. Most minerals of parent material (except quartz) altered. Saprolite. Medium permeability. Low to medium strength.
II. Weathered Rock	IIA Transition	Transition from saprolite to weathered rock. Highly variable, soil like to rocklike; matrix medium to coarse sand or silty and micaceous; 10 to 95% corestones; spheroidal weathering common. Wide range of engineering properties. High permeability. Relic structures control strength. RQD 0-50%.
	IIB Partly weathered rock	Soft to hard rock; joints stained to altered; some alteration of feldspars and micas. Lower strength, lower modulus and higher permeability than original material. Medium to high permeability and strength. RQD 50-75%.
III. Unweathered rock		No iron stains to trace along joints; no weathering of feldspars or micas. RQD > 75%.

Table No. 3.1 - Description of Weathering Profile for Igneous and Metamorphic Rocks (Adapted from Deere and Patton, 1971)

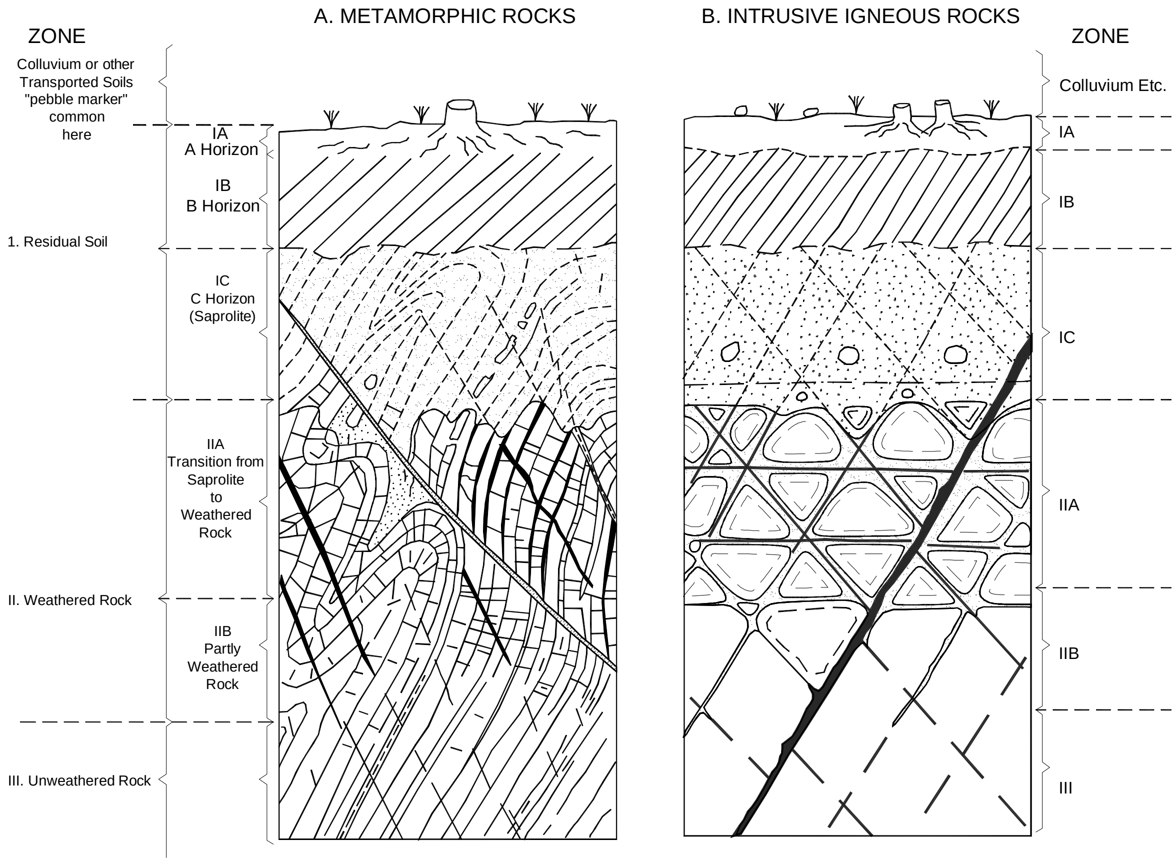

Figure No. 3.1 - Typical Weathering Profile for Igneous and Metamorphic Rocks (Deere and Patton, 1971)

Among the other widely known and recognized models for weathering profiles, the most referred models are those proposed by Little (1969), Hough (1957).

The Working Party of Geological Society of London (Fookes, 1997) adapted a scale based on different weathering stages, initially proposed by Little (1969), classifying the weathering grades in six different groups (Figure No. 3.2). This weathering scale and Deere and Patton's (1971) weathering profile have many aspects in common, and both can be applied for engineering purposes.

If a sharp basal weathering front is not present, the total depth of weathering may be difficult to determine, particularly for sedimentary and metamorphic fissile rocks. In fractured crystalline rocks the task of determining the depth of weathering can be more precise. Fresh outcrops can evidence an influence 15 to 20 m deep, with occasional abrupt gullies up to 50 m deep.

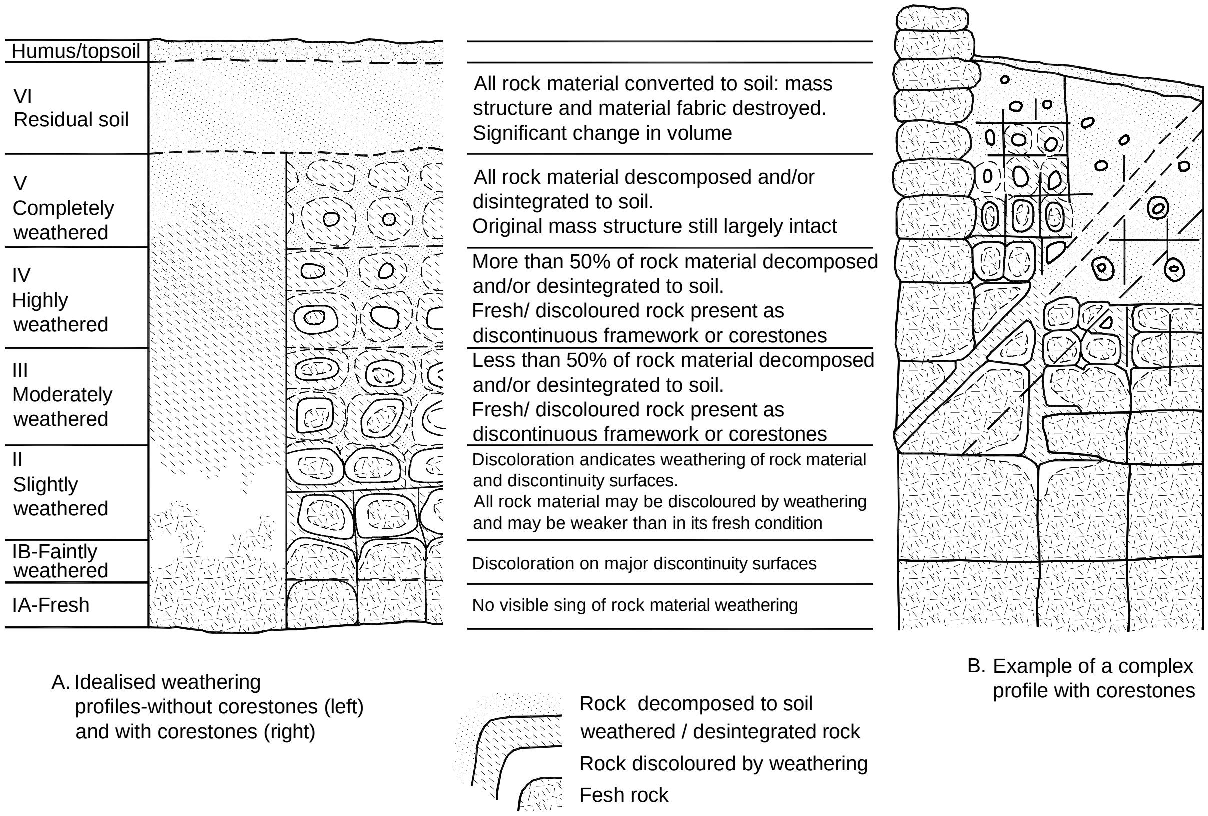

Figure No. 3.2 - Weathering Grades proposed by the Geological Society of London (Fookes, 1997)

A typical weathering profile, showing the different weathering horizons of the residual soil is presented in Figure 3.3. This profile corresponds to materials found at the foundation of the Goro Nickel Project in New Caledonia.

Figure No. 3.3 - Weathering Profile found at the Goro Nickel Project – New Caledonia

4 MINERALOGIC COMPOSITION

Mineralogy is responsible in great part for the wide range of engineering properties exhibited by residual soils. The influence of micro structure on the engineering properties of tropical residual soils, and andisols in particular, was first brought to attention by Terzaghi (1958). Their mineralogy is to certain extent inherited from their parent rock materials and in part due to weathering processes present in their formation. Granites generate tan and yellow silts and silty sands, with varying quantities of mica and clays of the kaolinite family. Basalts (rich in ferromagnesian minerals) produce highly plastic montmorillonite clays with iron oxide coloration varying from deep red to dark browns, typical of “cotton soils” from India (Sowers, 1970).

Climatological factors involved in the genesis of tropical soils generate geochemical processes that result in soils with less silica and cations, but richer in iron and aluminum (typical of saprolites). Laterites or lateritic soils tend to be richer in oxides. As mentioned earlier, laterites are regarded as having a low silica-sesquioxides relation (Morais Leme, 1985).

Clay minerals are generally found, sometimes as residual minerals from weathering from the parent materials when claystones and mudstones are present (Fookes et al, 1997). Tropical climate promotes the formation of the clay mineral kaolinite, making it the most common clay mineral in tropical residual soils (Queiroz de Carvalho, 1997). Particles of kaolinite are larger and less active than most clay minerals, but are also platey with a low coefficient of interparticle friction. When shearing occurs kaolinite orient their longer sides and present slickensided surfaces and low residual angles of friction (Fookes et al, 1997).

Under certain conditions halloysite is also present, and as a consequence soils containing this mineral exhibit irreversible changes in index properties if dehydrated. Since tropical residual soils are frequently found partly saturated, if they contain halloysite and are wetted (Queiroz de Carvalho, 1997). Unlike other clay minerals, halloysite and allophane (another mineral present in residual soils) are not of platey nature and consequently do not exhibit a sharp decrease in their residual strength. They display low swelling and small strains when wetted or dried (Fookes et al, 1997). Allophanic soils present satisfactory engineering properties, despite having liquid limits ranging from about 80 to 250% (Wesley, 1997). Residual soils found at Tjipanundjang and Sasumua dams, mainly composed of halloysite and allophane, exhibited satisfactory resistance, even though their clay content was very high (Terzaghi, 1958; Wesley, 1974).

Smectites (montmorillonite) are found in specific types of residual soils (i.e. vertisols or cotton soils). When their presence is important, small changes in effective stress cause large volume changes, and when desiccated and wetted result in significant volume changes and heave displacements (Fookes et al, 1997).

Table No. 4.1, presents typical angles of shearing resistance and residual angles of shearing resistance for different types of minerals found in tropical residual soils:

Clay mineral	ϕ'	ϕ_r'
Smectites	15-20	5-11
Kaolinites	22-30	12-18
Allophane	30-40	30-40
Halloysite	25-35	25-35

Table No. 4.1 - Angles of Shearing Resistance for the Different Clay Minerals Present in Residual Soils (After Fookes et al, 1997)

5 CLASSIFICATION, SAMPLING AND TESTING

5.1 CLASSIFICATION

The Unified System and the AASHTO system are the two engineering soil classifications more common to geotechnical engineers. Both are based on grain size and grain size distribution for coarse-grained soils and plasticity properties, as measured by Atterberg limits, for fine-grained soils. These classification systems have worked well for the temperate zone soils because the clay minerals encountered are stable in their environment and therefore grain size distributions and plasticity properties can be generally determined unambiguously. As a result, numerous relationships have been successfully developed relating the results of the classification tests and various engineering properties. However, in the case of residual soils, reproducible results in classification tests are difficult to obtain, and the relationships between the classification tests and engineering properties developed for temperate zone soils do not apply. The most evident factors which may account for this difference in behavior are the changes in the soil properties due to drying and the presence of cementing agents, which are generally not found in the temperate zone soils. Several authors have formulated some of the conflicts of "conventional" classification systems for residual soils (Queiroz de Carvalho 1985-1997; Wesley, 1997; Fookes et al, 1997):

- The physical properties of residual soils are not reproduced successfully in a laboratory environment, due to the importance of their structure in their engineering behavior.

- The unusual clay components sometimes present in residual soils, generate abnormal behavior if compared to those associated with traditional classification (See Chapter 4). Also, the in-situ characteristics of these materials and their sequence are not adequately reflected in small samples.

- The conventional classification systems are focused on the properties of the soil in a remolded state, with good results for transported soils. Residual soils are not found naturally in a remolded state, and this procedure eliminates inherited relic structures of the parent material, crucial for predicting behavior. Moreover, standard soil classification systems were developed for soils from temperate areas and transported soils in tropical zones, limiting their use to the distinctive characteristics of tropical residual soils.

- Typical tests (according to ASTM procedures) used for transported soils are not always applicable, relevant or representative of residual soils characteristics as explain previously. Accordingly, relationships and correlations are difficult to establish, and empirical relationships have to be adapted to local experience.

- Frequently, residual soils exhibit more competent engineering properties than index tests suggest, mainly when index tests results are correlated to the engineering behavior of transported soils as in the USCS classification system.

Due to the aforementioned limitations of traditional classification systems, alternative schemes have been proposed. These systems are based on characteristics sometimes different from grading and typical physical properties, with the objective of providing additional information important to reflect behavior of residual soils that is usually lacking in conventional classification systems, like the observed site characteristics.

The attempts to develop an adequate classification system for residual soils focus in the pedological and the physical-chemical characteristics.

In any case, when analyzing soil profiles including residual soils, and their classification is either based on the Unified Soils Classification System, it should be clear to the engineer which of the properties can be different under real conditions. Some of the classification systems that have been proposed are:

Wesley (1997), presented a classification system to complement and use in conjunction with the Unified Soil Classification System (USCS), including an analysis of the mineral components, and based on the following points:

- Soil composition, including mineralogy and physical properties of the soils.

- Structure in its undisturbed in- situ state, both macro-structure and micro-structure.

The proposed system by Wesley classifies soils in three main groups that tend to exhibit similar engineering properties:

Group A: Residual soils without a strong mineralogical influence.

Group B: Residual soils with a strong mineralogical influence deriving from commonly occurring clay mineral.

Group C: Residual soils with a strong mineralogical influence deriving from special clay minerals only found in residual soils.

The three main categories are further subdivided into sub-groups to limit the variables affecting engineering behavior. As pointed by Wesley, one of the main difficulties applying this method is its use of mineralogical composition as a starting point for classification. On the other hand the scheme has the advantage of including most residual soils, denoting simplicity and applicability.

The Geological Society suggests the use of the formal French Classification System for residual soils. Although this method of classification is intended for engineering purposes, anticipating typical behavior, the terminology is more complex and less practical than Wesley's System. See Fookes (1997) for a detailed description of this classification system.

Several other classification systems have been proposed. They can be grouped in three main categories (Queiroz de Carvalho (1985):

- *Classifications based on chemical, pedological and morphological criteria* (Clare's - 1957, based on the silica/ alumina and silica/ sesquioxides ratios- 1951, Ruddock's - 1969, Little's – 1967, Gidigasu's – 1971).

- *Orthodox Classifications* (USCS, Highway Research Board, Vallerga, 1969; Lal and Bindra's,1981; Eklu, Natei and Muller's, 1981; Medina and Preussler's, 1980 and Vargas, 1982).

- *Classifications based on non-orthodox tests* (De Graft Johhnson, Arulanandan, 1969; Queiroz de Carvalho's, 1981; Lohnes and Demirel, 1973; Tuncer's, 1976; Nogami and Villibor,1981; EPF Zurich,1979).

Every abovementioned classification system has specific benefits, but it is believed the use of Wesley's scheme in combination with the USCS provides the most complete and practical classification system. For that reason the other classification systems are only referenced and not explained in detail.

5.2 FIELD IDENTIFICATION

Due to the sensitivity of residual soils, an adequate field classification is fundamental to properly classify and correlate engineering properties of residual soils.

The identification of saprolite and the other residual soils shall be mainly performed in the field. Test holes and wide borings are the most suitable means for establishing the soil and weathering profile. Since most residual soils are cohesive and frequently the water table is found deeper than in other soil layers, the holes tend to be stable and provide time for an adequate description (Simmons Blight, 1997).

The information that should be collected in the field includes the following aspects (Fookes et al, 1997, Simmons Blight,1997):

- Origin (rock type)
- Texture
- Fabric or structure
- Color
- Resistance
- Natural Moisture
- Density
- Apparent behavior
- Mineralogy
- Strength

In addition, factors like precise location, landforms characteristic of the site, climate and vegetation should be recorded, since they can give a hint to the extent and variation of weathering.

In most cases, where saprolitic soils are encountered, engineering behavior is controlled by inherited geological features from the parent material. As a result it is very important to detect and describe any preferential planes, joints, or any other features remaining from the parent rock.

Conventional geophysical methods, may encounter problems, since it is normal that residual soils in higher layers (highly weathered), have higher wave velocities, making a correct interpretation of the refraction tests difficult. Occasionally, cross-hole tests have been used, but they also present a number of limitations, including complications to determine layer thickness. In any case, if any of these tests is used, it should be calibrated and verified with profiles inferred from drill- holes.

In some places, such as Brazil, SPT tests are used to separate the collapsible materials from the more competent levels.

5.3 SAMPLING

This section emphasizes the special sampling practices and specific criteria applicable to residual soils. Sampling techniques developed for transported soils not specifically commented in this chapter can be applied to residual soils.

Two main matters are fundamental for an adequate sampling program: representativity and disturbance. Typical anisotropy, heterogeneity and variability of tropical residual soils complicate the planning of a sampling program. Sampling for testing of many engineering properties (i.e strength) requires large specimens to ensure that variables like structure and relic features are representative of the mass. The weak cementation and bonding of particles of residual soil also sets additional hurdles, because the common brittle structure of this type of soils makes them very sensitive to disturbance (Fookes et al, 1997).

The large voids commonly present in residual soils and the fact they are frequently found partly saturated generates inevitable volumetric expansion when sampled, especially in saprolites, where the stress changes are greater (Fookes et al, 1997).

Testing requirements vary depending on the type of engineering property the sample is going to be tested for. Disturbed samples can be utilized for tests related to density, water content, particles size, and compaction. For these cases, SPT samples, open-drive tube samples or piston samples are satisfactory. The assessment of strength and stiffness in the laboratory requires high quality samples; hand cut samples from trial pits or shafts. The importance of this aspect has been reported by Prusza (1984) at the Guri Dam (Venezuela), where he noticed important differences between shear tests performed on samples obtained by Denison samplers and undisturbed blocks. In Guri, where $70x10^6$ m^3 of earthfill was placed, testing on undisturbed samples obtained by using Denison samplers and/or Shelby tubes, indicated that the decomposed granitic gneiss had almost the same strength ($\phi' = 30^o - 40^o$) and similar stress paths as that of the laboratory compacted fill material. However, undisturbed block samples obtained from shallow excavated trenches, revealed a lower strength (ϕ'=25º), indicating that the Denison Sampling and/or sample extraction procedure destroyed the sample structure increasing the density and moisture content. Subsequent testing was performed on specimens cut from block samples obtained from shallow and deep trenches.

Generally, special care should be taken to preserve the sample's in-situ water content, especially when the soil exhibits the minerals halloysite and allophane, since irreversible changes can occur if the material is allowed to desiccate.

Wolle et al, separate residual soil in three groups according to the difficulty of sampling. Lateritic soils are considered easy to sample, because of their relative homogeneity and less sensitivity to disturbance; thus samples tend to be representative of the in-situ conditions. On other hand, saprolites are regarded as difficult to sample. Relic structures, boulders and sensitivity to disturbance can even inhibit any sampling, particularly in the soil- rock transition. In general terms, with increasing depth greater heterogeneity is found, and consequently samples are less representative and difficult to extract.

Shelby tube samplers have been more commonly used than Denison samplers, but even this type of sampler becomes unfeasible to use in saprolitic soils when resistance increases.

5.4 INDEX TESTS

Despite the fact that traditional index test do not properly typify residual soils, their extensive use for geotechnical analysis worldwide has promoted different studies to adapt these tests (or their results) to the different behavior and characteristics of residual soils, in order to allow for better interpretations and correlations of their engineering properties, based on index tests.

A basic principle in geotechnical testing is to reproduce in-situ conditions in the laboratory to anticipate the behavior of soils for design purposes. This fundamental requirement for useful and successful testing is usually not satisfied for residual soils. As de Mello (1972) pointed out, only their behavior after complete disaggregation, remoulding and saturation is modeled. Furthermore, these soils neither have been, nor will be subjected in-situ to these conditions.

The single individual factor that has a greater effect on index properties is *drying*. Due to the mineralogical composition of residual soils, drying can induce irreversible chemical changes, leading to alteration of the clay minerals and aggregation of fine particles (Fookes et al, 1997). This circumstance can have profound effects in the index properties of residual soils, that is in their moisture content, plasticity, shrinkage and particle size distribution. The main effects of drying are increased cementation due to oxidation of the iron and aluminum sesquioxides and dehydration of halloysite and allophane minerals (Townsend, 1985).

5.4.1 Natural Moisture Content

Fourie et al (1997) and Fookes et al (1997) recommend a specific methodology for determining the natural moisture content of residual soils, to subtract any structural water present in the soil, from the actual moisture content.

Two samples should be used to determine natural moisture. One of them should be dried to a standard temperature of 105 degrees Celsius, calculating its natural moisture with the conventional method. The second sample should be dried either with air, or oven-dried at a temperature of 50 degrees Celsius and a relative humidity of 30%.

If a significant difference is observed, it indicates that structural water is present within the soil sample. If such a difference exists, the moisture content of all other samples need to be measured following the second procedure.

The dams built on the “Antioqueño” Batholith residual soils in Colombia (Piedras Blancas, Quebradona, Troneras, Miraflores, Santa Rita I and II, Punchiná and San Lorenzo) exhibited high water contents, ranging between 25-40%. The data shown was obtained from conventional testing.

5.4.2 Atterberg limits

To avoid alteration of natural properties, the determination of Atterberg limits should be performed without any form of drying a priori. The Atterberg limits, as well as the natural water content, should be measured with the fraction of the soil passing the 4,75 mm sieve (Fookes et al- 1997, Fourie et al-1997). Morin and Todor (1975) gathered reported effects of drying on the plasticity of volcanic soils in different parts of the world (Figure No. 5.1).

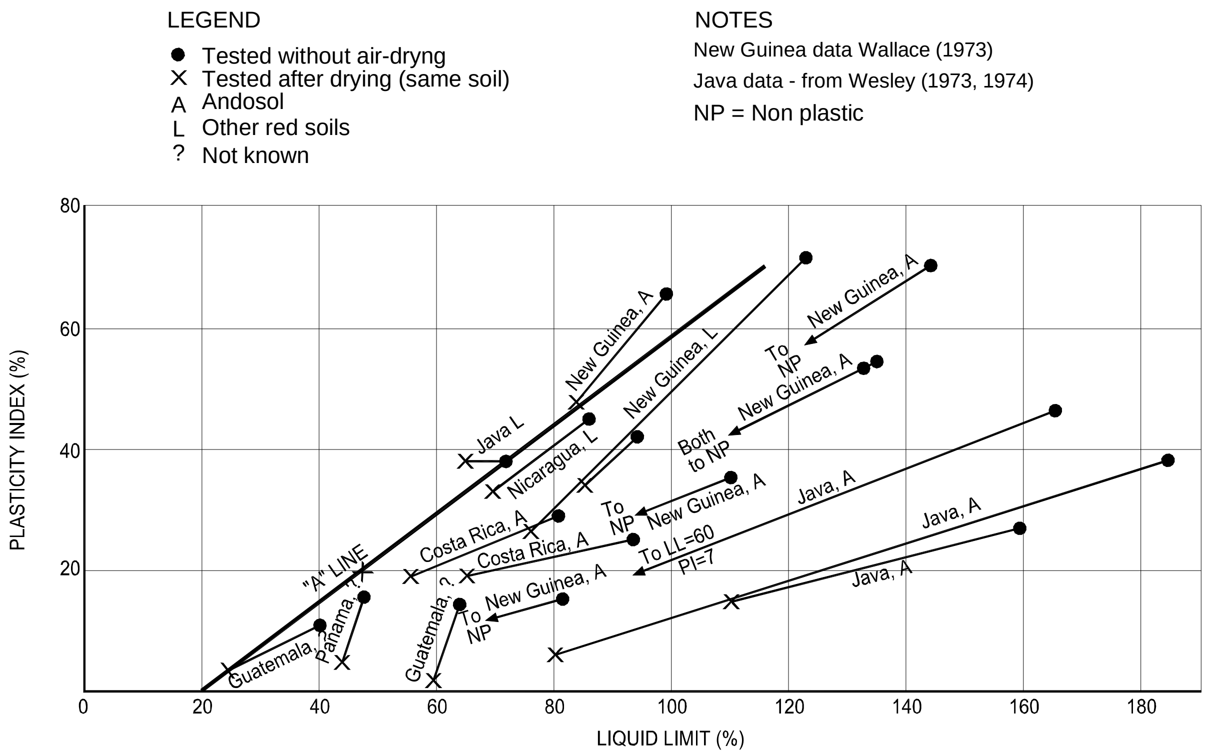

Figure No. 5.1 - Effect of Drying on the Plasticity of Volcanic Soils in Different Parts of the World (Morin & Todor, 1975)

The effects of drying on the Atterberg limits are also showed in Table 5.1 for soils around the world with different mineralogy (Fookes et al, 1997). Values are given for air-dried and oven dried samples. Notice that the effect is minor for soils not containing allophane and halloysite.

Soil location and type	**Atterberg limits**		
	natural W_L:W_P	**air dried W_L:W_P**	**oven dried W_L:W_P**
Costa Rica:			
Laterite	81:29		56:19
Andosol	92:67		66.47
Dominica:			
Allophane	101:69	56:43	
Latosolic	93:56	71:43	
Smectoid	68:25	47:21	
Hawaii:			
Humic latosol	164:162	93:89	

Hydrol latosol	206:192	61:NP	
Java:			
Andosol	184:146		80:74
Kenya:			
Red clay, Sasumua	101:70	77: 61	65:47
Malaysia:			
Weathered shale	56:24	48:24	47:23
Weathered granite	77:42	71:42	68:37
Weathered basalt	115:50	91:49	69:49
New Guinea:			
Andosol	145:75		NP
Vanuatu:			
Volcanic ash, Pentecost	261:184	192:121	NP

Note: NP indicates non-plastic

Table 5.1 - Effects of Drying on the Atterberg Limits of Residual Soils in Different Parts of the World (Fookes et al, 1997)

One of the limitations using conventional Atterberg limits tests to classify residual soils and correlate them with engineering properties of soils with similar values is the destruction of the cementing bonds between the particles.

Pre-drying and mixing in standard tests, has been shown to have important influence on results (Townsend –1985, Fookes et al- 1997, Fourie 1997). Some recommendations proposed to diminish these effects are:

- Control temperature, and avoid sensible changes from in-situ conditions.

- Mixing time should be limited to a maximum of 5 minutes, using fresh samples for each of the test points, this practice avoids an increment of liquid limit and plasticity index in some residual soils due to the breaking down of particles after prolonged periods of mixing.

- The sample should not be dried for seeming; the soil should be broken down using distilled water. If distilled water produces property changes caused by soluble salts, a soil water solution should be used.

Mineralogy has also a strong effect on the Atterberg limits. Residual soils containing particular prevailing minerals tend to behave as a group with regards to Atterberg limits, exhibiting values in a relative close range. When minerals like montmorillonite, allophane and halloysite are present, the aforementioned behavior is applicable. Wesley et al (1997) plotted residual soils containing these minerals in the plasticity chart, where typical regions for soils containing different minerals can be identified (Figure 5.2).

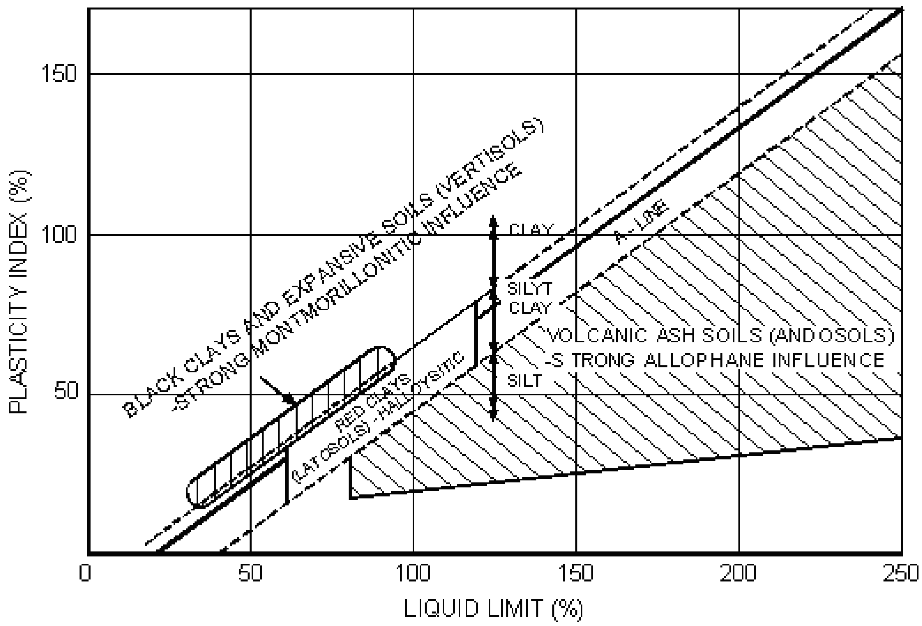

Figure 5.2 - Influence of Mineralogy on the Position on the Plasticity Chart for Residual Soils (Wesley et al, 1997)

In the classification system proposed by Vargas (1982), it was suggested that the plasticity chart be plotted aside from the activity for residual soils. See Figure 5.3. The idea behind this suggestion is to identify the residual soils that are affected by manipulation (Melfi et al, 1985). It should be noted that silty micaceous and / or kaolinitic saprolitic soil exhibit high activity, despite having inactive minerals of the kaolinitic group in their clay fraction. This behavior has been attributed to micaceous or kaolinitic silt, which increases plasticity (Melfi et al, 1985).

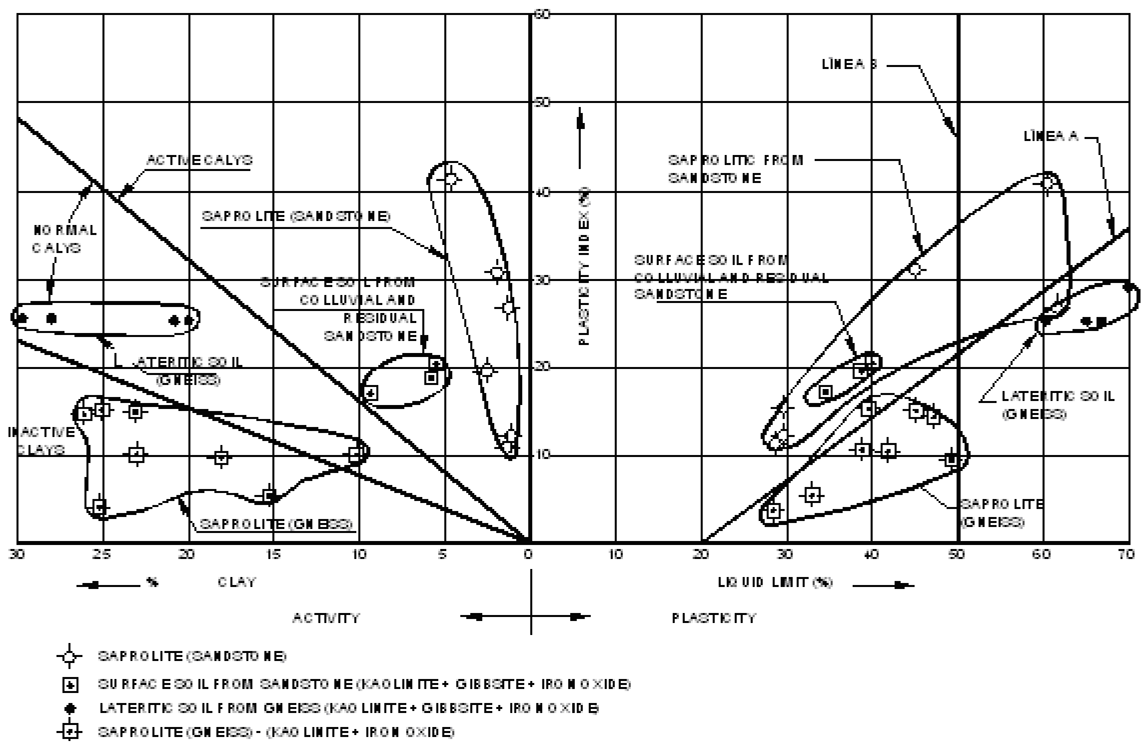

Figure No. 5.3 - Plasticity Chart Associated with Activity for Residual Soils (Vargas, 1992)

The dams built on the "Antioqueño" Batholith (Colombia) residual soils (Piedras Blancas, Quebradona, Troneras, Miraflores, Santa Rita I and II, Punchiná and San Lorenzo) presented liquid limits (w_L) between 40 to 50% and plasticity indexes between 7-12%. The main index properties of these dams are summarized in Table 5.2. All testing for the dams built in the "Antioqueño" Batholith was standard testing.

Dam	LL (%)	IP (%)	% finer than		Water content	
			No. 200 Sieve	2μ	w_n (%)	w_{opt} (%) (Proctor Std.)
Piedras Blancas	47	12	80	10		
Quebradona	35	4	30	6	26	23
Troneras	43	9	79	10	30	25.5
Miraflores	37	6	46	5	22	19.5
Santa Rita I	38	6	54	7	21.5	16.5
La Fe	40	10	68	7	25	19
Santa Rita II	38	8	56	7	26	20.5
Punchiná	36	10	43	6	24.2	20.1
San Lorenzo	45	14	42	7	24.3	20
Las Playas	34	7	45	5	27	19.9
Riogrande	28	6	41	5	26.5	22.3
Average	**38.3**	**8.4**	**53.1**	**6.8**	**25.3**	**20.6**

Table 5.2 Main Index Properties of Antioqueño Batholith Dams.

Table 5.3 presents liquid limit (w_L), plasticity index (IP) and other index properties values for clayey lateritic soils and saprolitic soils found at some Brazilian Dam sites.

Group of soils	Dam Site	w_L (%)	IP (%)	γd_{max} (gr/cm^3)	Gs	%Clay <2microns	% Sand
Lateritic clayey soils	Itaparica	57	36	1.61		40	40
	Três Irmãos	73	42	1.45	2.84	62	21
	Juquía I	66	31	1.46	2.81	44	24
	Juquía II	44	21	1.70		18	38
	Poços de Caldas	48	18	1.38	2.83	50	5
	Itaipu	59	33	1.49	2.86	78	6
Saprolitic Soils	Itaparica	n.p	n.p	1.98	2.7	2	95
	Nova Avanhandava	86	31	1.28	3.02	50	9
	Euclides da Cunha	48	19	1.71	2.71	14	36
	Passauna	50	17	1.54		21	
	Emborcação	46	19	1.61	2.67	10	50

Table 5.3 Main Index Properties of lateritic soils and saprolitic soils found at some Brazilian Dam sites.

Site investigations at the Omai Gold Mine Tailings Dam indicated an average liquid limit of 48% and an average plasticity index of 12%. Natural water contents typically ranged from 20% to 45%. The Atterberg limits data for the saprolite samples of this project is presented in Figure 5.4 (Bedell et al, 2002).

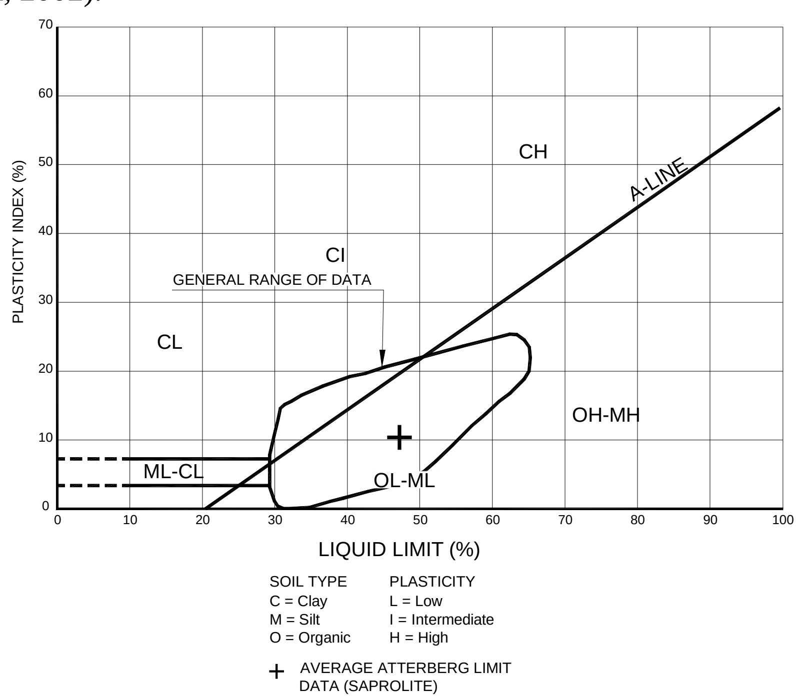

Figure No. 5.4 – Plasticity Chart Average Plotting of the Residual Soils Present at the Omai Gold Mine Tailings Dam Site (Bedell et al, 2002).

At the Goro Nickel Project in New Caledonia the residual soils found at the tailings dam site are identified as limonites, a low to high plasticity sandy clayey silt (ML-MH). The material typically comprises less than 20% sand size particles, with the majority of the material being silt seized. Figure 5.5 shows typical plotting of the limonite in the plasticity chart. Figure 5.6 presents the variation with depth of the water content and Atterberg Limits for the same soil. These parameters increase rapidly with depth over the upper 10 m and then tend to be constant at high values below this depth.

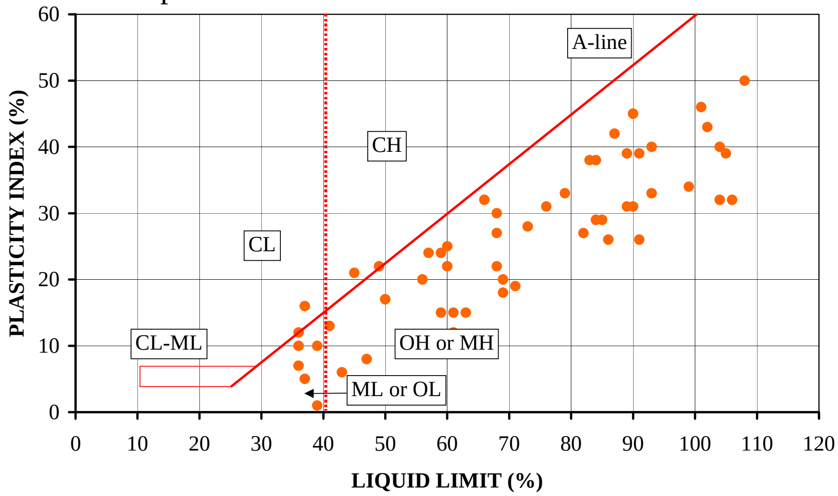

Figure No. 5.5 – Plasticity Chart Average Plot of the Residual Soils (Limonite) found at the Goro Nickel Mine Tailings Dam

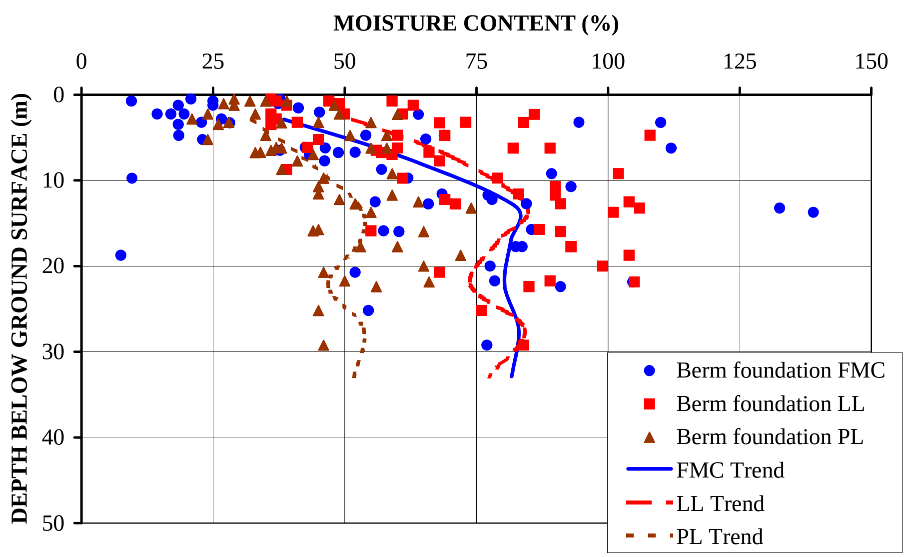

Figure No. 5.6 – Natural Water Content and Atterberg Limits with Depth for the Limonite found at the foundation of a Tailings Dam in an island in the Pacific Rim,

Figure 5.7 shows the plasticity chart plotting of the residual soils found at Tjipanundjang Dam (Indonesia). These soils belong to the pedological group called andosols, a yellowish brown clay derived from the weathering of volcanic materials, mainly ash (Wesley, 1974). It can be noticed that the areas where these soils plot in the plasticity chart, coincide with the aforementioned examples.

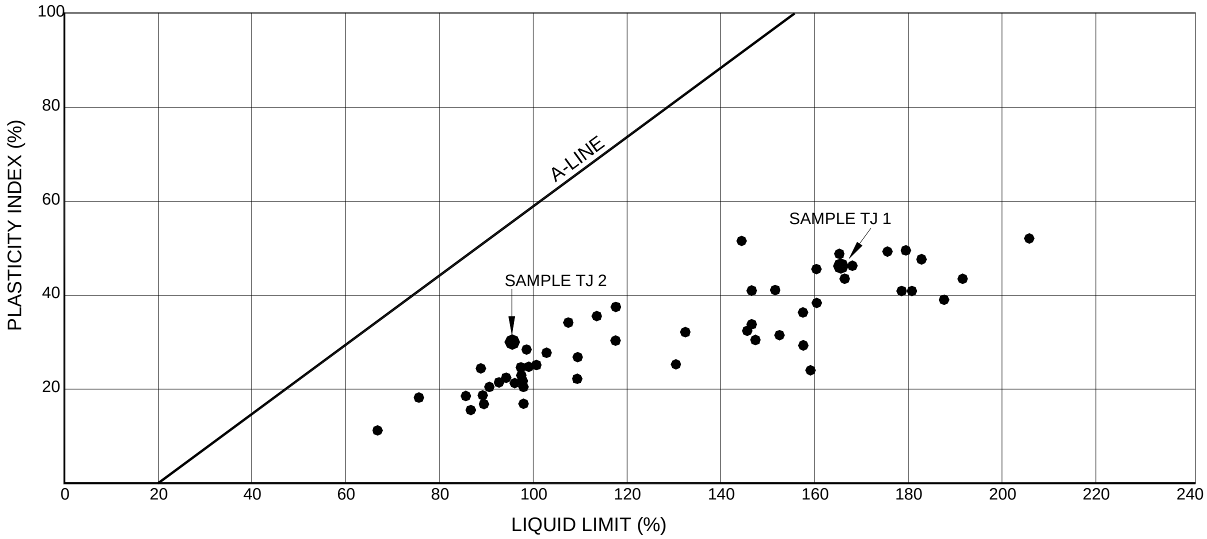

Figure No. 5.7 – Plasticity Chart Average Plotting of the Andosols Found at Tjipanundjang Dam (Wesley, 1974).

Figure 5.8 presents further characteristics of the residuals soils found at Tjipanundjang Dam. There is a steady decline in the water content and Atterberg Limits from the surface down to a depth of about 15 m, where these parameters tend to stabilize. The water content is about 150% at the surface and drops below 100% after a depth of 15 m. Water contents of this magnitude are not rare in volcanic ash soils, and usually suggest the presence of the amorphous clay mineral allophane (Wesley, 1974). In some testing run at Tjipanundjang Dam it was found that in some locations the natural water content of the material was above the Liquid Limit. When this happened high sensitivity values of the materials were to be expected.

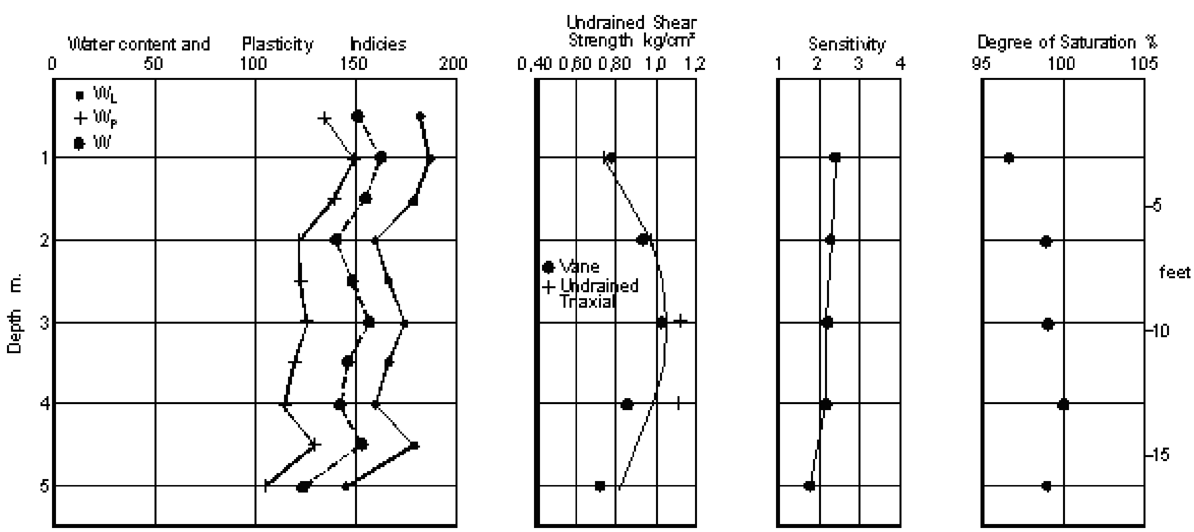

Figure No. 5.8 – Characteristics of the Andosols Found at Tjipanundjang Dam (Wesley, 1974).

The abovementioned problems of predrying residual soils to determine their Atterberg limits and other properties was strongly experienced at Tjipanundjang Dam. Table 5.4 shows the difference in several index properties of these soils if predrying was performed. The drastic changes in the material properties after predrying enforce the argument of testing residual soils in their natural state.

	Atterberg Limits		Specific Gravity	Particle Size		Standard Compaction Test	
Test Condition	**LL**	**PL**		**(%) Passing No. 200 sieve**	**Clay Fraction**	w_{opt}	$\square_{dmax}$
Natural	165	119	2.8	96	65	120	0.61
Air Dried (w=30%)	60	53	2.78	37	11	55	0.96
Oven Dried	44	42	254	16	4		1.19

Table No. 5.4– Effect of Predrying Andosols to Determine some of their Index Properties at Tjipanundjang Dam (Wesley, 1974).

Atterberg Limits Tests on undried samples were performed on the clay fill composed of residual soils found at Sasumua Dam site. All tests, with the exception of three, plotted below the A-Line in the Plasticity Chart (See Figure 5.9). Terzaghi attributed the low dry density at optimum water content to this abnormal behavior.

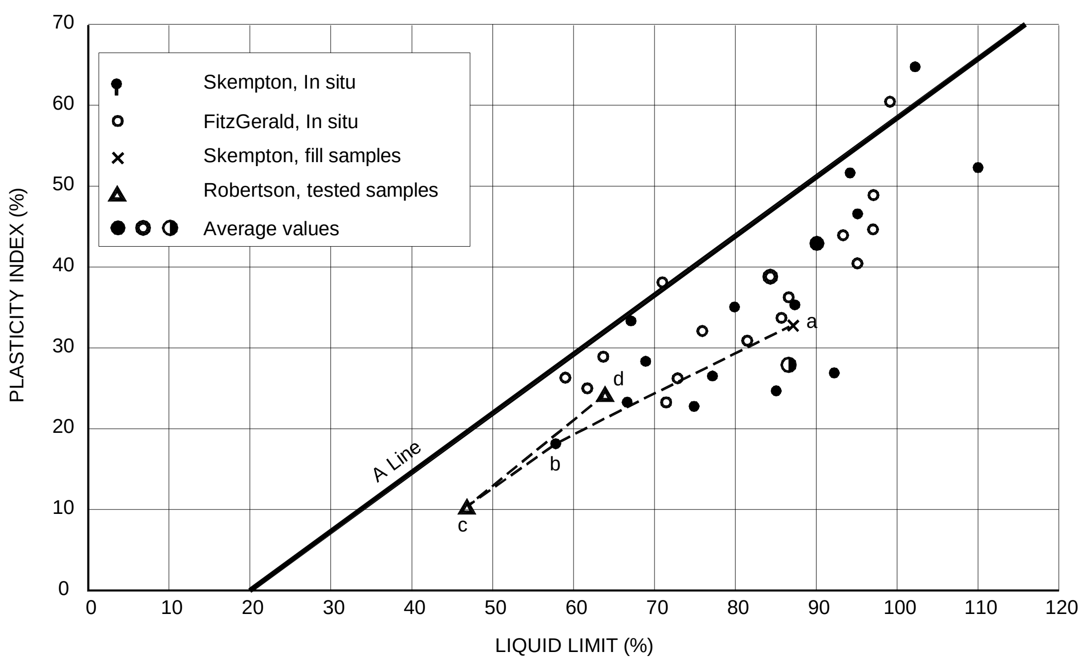

Figure No. 5.9 – Plasticity Chart of Samples from Sasumua Dam (Terzaghi, 1958).

The residual soils found at the Sasumua dam site were also susceptible to testing procedures due to their halloysite mineral content. Different treatments of samples before testing resulted in a large scatter (Terzaghi, 1958). Table 5.5 presents Atterberg Limit results for Sasumua Dam residual soils tests.

Treatment	Average Liquid Limit	Average Plastic Limit	Average Plasticity Index
a. Natural, before drying.	87	54	33
b. Dried at 105 ° C and powdered in mortar.	58	39	19
c. As b, but treated with tetrasodium pyrophosphate.	47	37	10
d. Dried, powdered and rehydrated 1 month	63	39	24

Table No. 5.5 – Effect of Predrying on Residual Soils Found at Sasumua Dam Site (Terzaghi, 1958).

5.4.2.1 Specific Gravity

Specific gravity is controlled by mineralogical contents of the soil. This parameter exhibits wide variability in residual soils and its value is related to the type of parent rock from which the soils were formed.

Residual soils can exhibit unusual low specific gravity value, due to their mineralogy (i.e. allophane). The soil should not be dried before performing this test, and the specific gravity should be established at the soils natural moisture content. (Fookes et al, 1997, Fourie - 1997). Figure No. 5.13 shows typical Specific Gravity values for clayey lateritic soils and saprolitic soils for Brazilian Dam sites.

Specific Gravity values of lateritic and saprolitic soils found at Brazilian Dam sites are presented in Table 5.3 (Texeira et al, 1985). Values range from 2.67 to 3.02.

Limonite found at Goro Nickel project presented high specific gravity values due to the high iron content in this residual soil. The reported values are on average 3.8.

Skempton reported average specific gravity values for the residual soils particles of Sasumua's clay fill around 2.83.

5.4.2.2 Particle size distribution

For determining the particle size distribution of soil sample, drying should also be avoided, and the soil should we weighted at its natural moisture content. Instead of drying the sample, a dispersant solution can be utilized to sieve the sample in a wet state. Chemical treatment should not be used, but dispersants are recommended prior to sedimentation. (Fookes et al, 1997).

Figure No. 5.10 and 5.11 present the percentage of sand and fine material for clayey lateritic soils and saprolitic soils for Brazilian Dam sites.

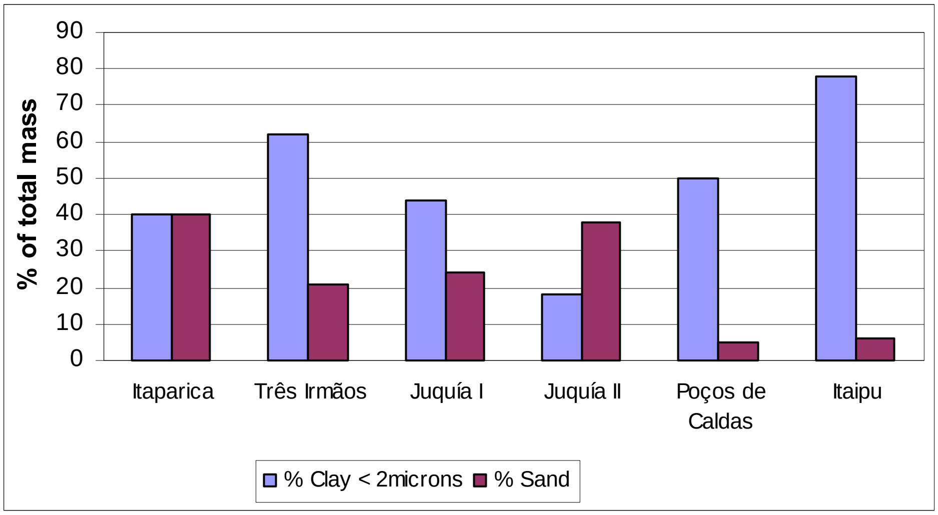

Figure No. 5.10 - Percentage (%) of sand and clay of lateritic soils for Brazilian Dam sites (Texeira et al, 1985).

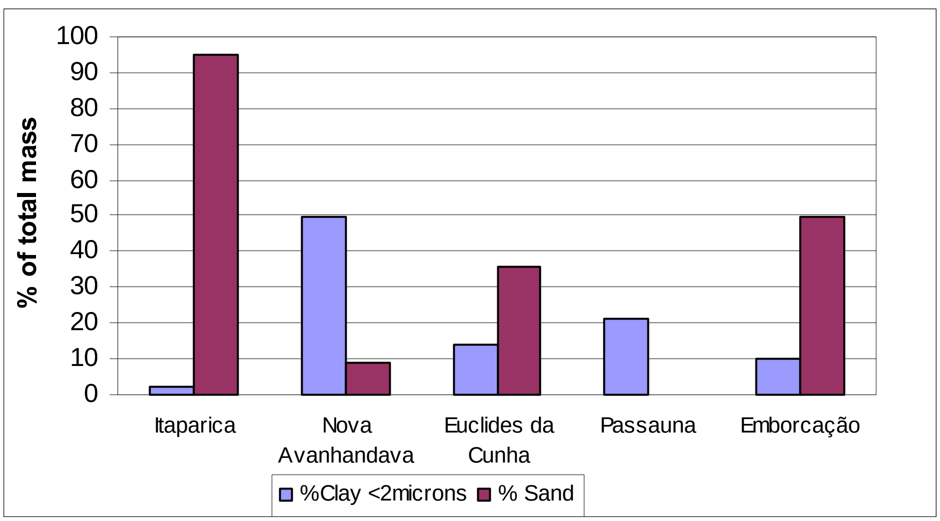

Figure No. 5.11 - Percentage (%) of sand and clay of saprolitic soils for Brazilian Dam sites (Texeira et al, 1985)

Figure 5.12 shows the particle size distribution for the residual soils found at Guri Dam in Venezuela (Prusza et al, 1983). The percentage of clay tends to decrease with depth (especially when saprolite is encountered), while the percentage of coarse particles (i.e. sands and gravels) increases.

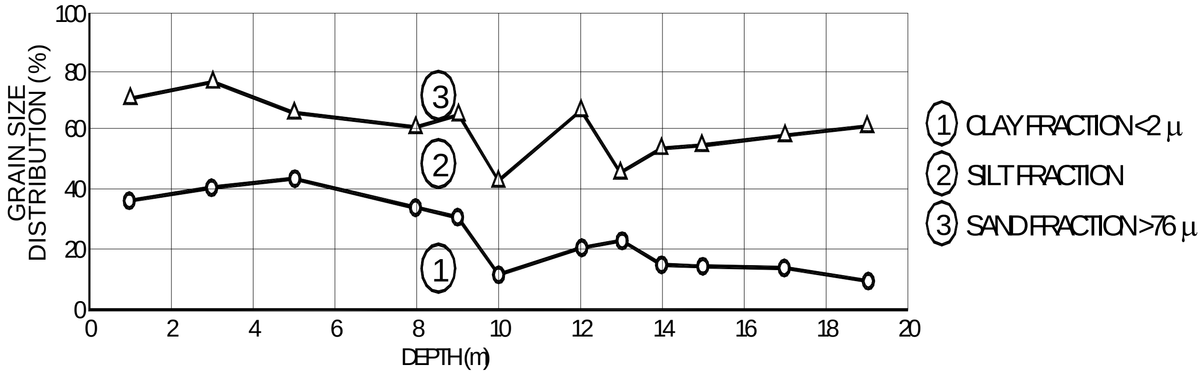

Figure No. 5.12 – Particle size distribution vs. depth for residual soils found at Guri Dam Site (Prusza et al, 1985)

Figures 5.13 and 5.14 show the particle size distribution for the Antioqueño Batholith Dams. Figure 5.18 presents the data for Quebradona, Troneras and Miraflores dams, while Figure 5.19 shows the results for Santa Rita, La Fe, Punchiná and San Lorenzo dams. The data presented was determined using conventional testing methods.

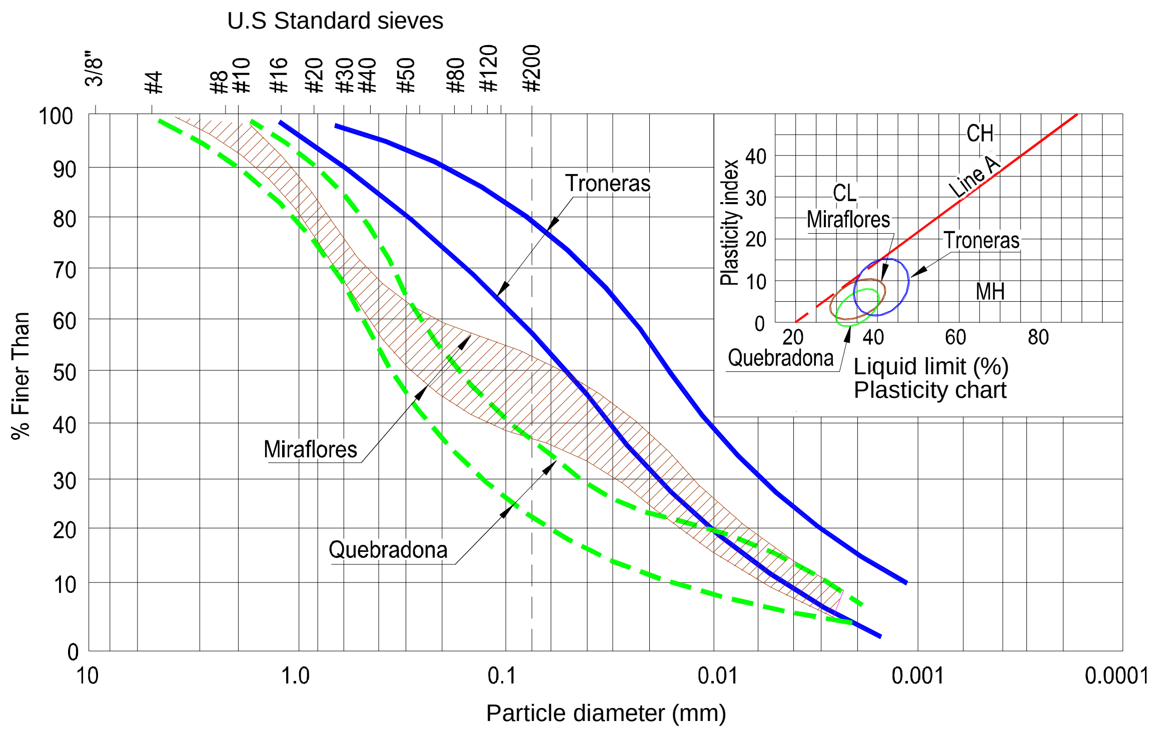

Figure No. 5.13– Particle Size Distribution for Quebradona, Troneras and Miraflores Dams

Figure 5.15 shows particle size distributions for residual soils (i.e. andosols) found at Tjipanundjang Dam, with and without predrying. It can be noticed that the effect of predrying on the particle size distribution is very different for the two samples presented. This can attributed to the quantity of allophane contained in the individual samples. The clay fraction is between 65 and 76% for the two samples without predrying. It should be stated that the measurement of particle size for soils containing large amounts of allophane can be very difficult, due to a tendency of the particles to flocculate after standing some time in the hydrometer test cylinder (Wesley, 1974). Hence, a large scatter on the particle size distribution results can be exhibited by this type of soil.

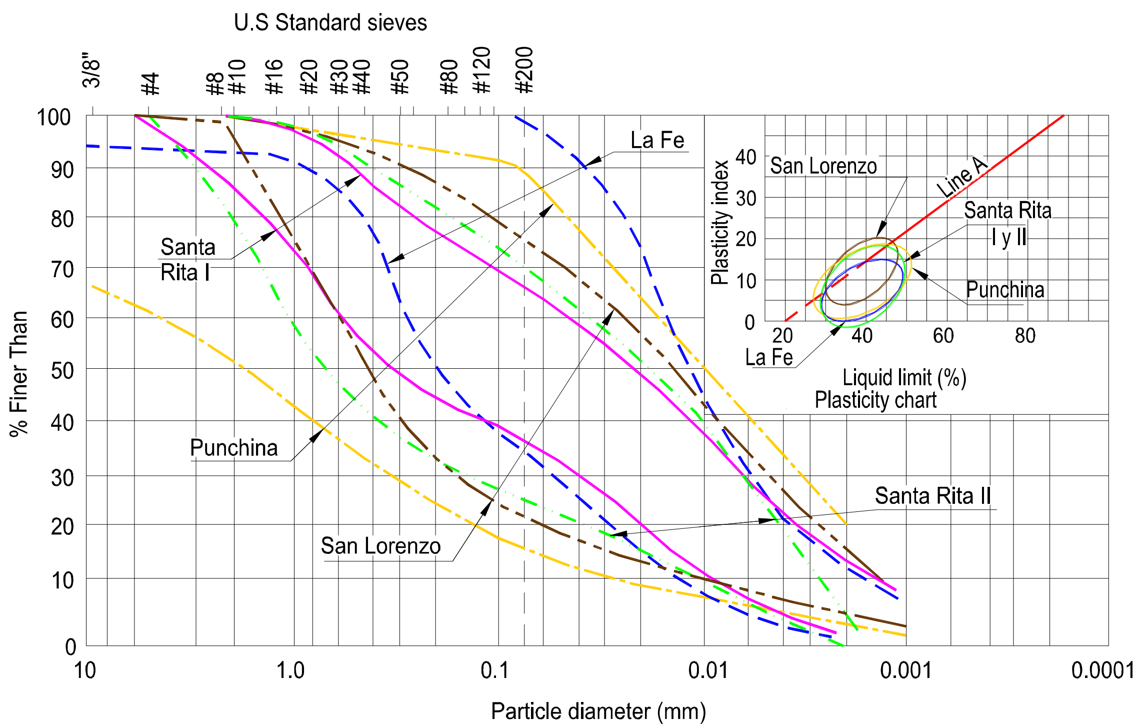

Figure No. 5.14 – Particle Size Distribution for Santa Rita, La Fe, Punchiná and San Lorenzo Dams

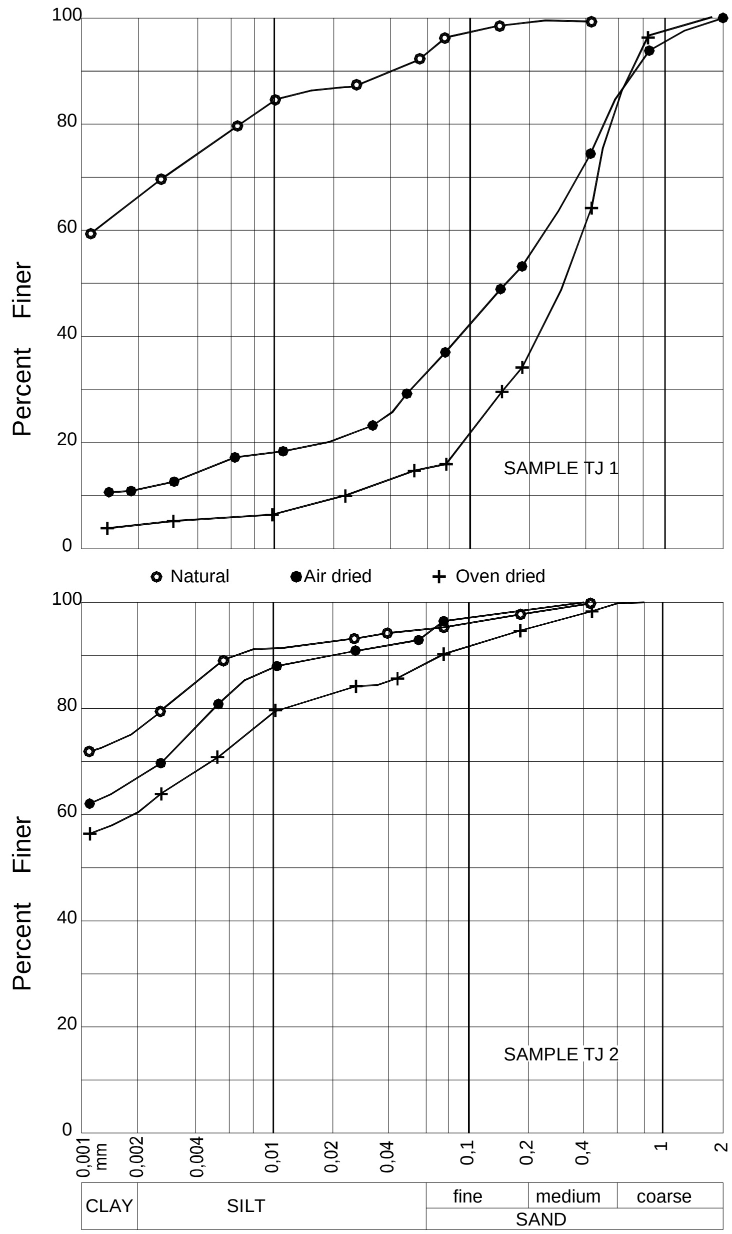

Figure No. 5.20 – Particle Size Distribution for Residual Soils Found Tjipanundjang Dam

6 ENGINEERING PROPERTIES

Since 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. Therefore, the data presented herein are intended more to indicate some of the trends and suggest ideas for approach to evaluation of the engineering properties of these soils rather than to provide definite values.

6.1 PERMEABILITY

As mentioned before, a general characteristic of residual soils is the large variability of their engineering properties both vertically and horizontally in the weathering profile. Permeability is not an exception to this general rule.

More important than in transported soils, it is essential to differentiate between the permeability coefficients of residual soils in-situ in their natural state and as fill materials, after being remolded and compacted. The macrostructure of residual soils generally controls the permeability. In saprolites, relic structures (joints, bedding planes, quartz veins) are predominant paths of flow for water seepage (Garga, 1997). Another factor affecting permeability of tropical residual soils is their cemented structure that creates big particles and voids. These features generate greater permeabilities than those typical of transported soils with similar grain size distribution. Consequently, it is evident that using common correlations for coefficients of permeability based on grading (i.e. Hazen) can be misleading.

Generally, more mature residual soils, close to the surface (i.e. laterites) tend to have relative low permeabilities, while young residual soils or saprolites exhibit relative permeabilities from medium to high. Leakage in the high permeability zones (IC and IIA) should be given special attention due to the potential of inducing piping in these materials.

It can be evidenced, that permeability commonly increases with depth in the weathering profile, until sound rock is found, where low permeabilities are encountered. Superficially, residual soils allow fractures to remain open resulting in easier penetration of water and greater permeability (Fookes et al, 1997). Laterally, variations of permeability can also be important, and therefore the use of “typical” values of permeability can be problematic and misleading (Garga et al, 1997).

Understanding that macro-scale characteristics dominate flow, the question of representativity of permeability testing arises. Small laboratory tests do not model these types of conditions, due to the limitations of scale. Therefore, permeability field tests are more appropriate to model the flow properties of residual soils. Since residual soils are mostly found in places where the water table is relatively deep, pumping tests are sometimes not applicable, and infiltration tests have to be used (Garga et al, 1997). The specific choice between the two tests depends on specific site conditions. During the exploratory stage of Guri “In situ” field tests performed in 38 cm. diameter bucket auger holes indicated a permeability of $1x10^{-4}$ cm/s. Tests conducted in the laboratory indicated that permeability decreased with increasing confining pressure and consolidation, ranging from 40 to 579 kPa respectively. Permeability tests conducted in triaxial cells before and after consolidation, but prior to shearing, indicated that at the end of the consolidation test, permeability decreased by at least ten times ($1x10^{-4}$ to $1x10^{-5}$ cm/s). This confirms the hypothesis that upon saturation under loads exceeding about 393 kPa the soil structure loses grain contact bonding, causing settlement and increased imperviousness.

Insects, mainly termites, have generated in some deposits large diameter holes, which can act as preferential flow paths. Besides leading to high seepage flow, this condition can lead to piping under a dam foundation. This feature has been evidence in the Amazon Region of Brazil, where this specific type of features have been named “Canaliculi”. The tubes or galleries have been found to range from a few millimeters to 20 centimeters. De Mello et al (1988) described problems with termite channels in the foundations of a 30 m high earth dam. “Canaliculi” or biologically worked soils have been encountered at the following dam sites: Tucuruí, Vereda Grande, Balbina, Samuel and Kararao (De Morais et al, 1985).

Villegas (1990) reported coefficients of permeability after compaction in the range of 10^{-7} to 10^{-8} m/s for the fill materials for the Troneras and Miraflores Dams. These materials are mainly silts, sandy silts and silty sands, derived from the weathering of quartzdiorites and granodiorites (Antioqueño Batholith, Colombia). The horizon IB (according to Deere and Patton’s Classification) was reported with a permeability of 10^{-7} m/s, and horizon IC with 10^{-6} m/s. According to Villegas, the permeability of the saprolite was not much higher than the lateritic horizon, because the spaces between the boulders of rock where completely filled with a matrix of weathered material, composed of sand and some silts that inhibited the flow of water.

Costa Filho et al (1985) presented typical permeability values for residual soils of Brazilian dams foundations:

Rock Type	Residual Soil	Permeability (m/s)	Type of test
Basalt	Mature residual and saprolitic	$9{,}510^{-5} – 3\ x10^{-6}$	Variable head in piezometers and infiltration in boreholes
Basalt	Mature residual and saprolitic	$9{,}5x10^{-7} – 9{,}510^{-10}$	Infiltration in boreholes and pits.
Gneiss	Mature residual (porous clay) and saprolitic	$4{,}8x10^{-6} – 9{,}5x10^{-7}$	Infiltration and pumping in boreholes
Gneiss	Mature residual (porous clay) and saprolitic	$4{,}8x10^{-5}$	Infiltration and pumping in boreholes
Gneiss	Mature residual (porous clay) and saprolitic	$2{,}2x10^{-6}$	Infiltration in pit.
Gneiss	Saprolitic	$9{,}5x10^{-7}$	Infiltration in pit and pumping
Gneiss	Saprolitic	$9{,}5x10^{-7}$	Variable head in lab. permeameter
Migmatite	Saprolitic	$3{,}2x10^{-5}– 9{,}5x10^{-8}$	Infiltration in boreholes

Table No. 6.1 - Typical Permeability Values for Residual Soils of Brazilian Dams Foundations (Costa Filho et al, 1985)

In general terms, saprolitic soils derived from granite can exhibit permeability values ranging from 4 x 10^{-3} to 5 x 10^{-9} m/s, while saprolitic soils derived from gneiss have permeability values ranging from 5x 10^{-6} to 1 x 10^{-7} m/s. Mature residual soils originated from the same parent materials have permeability coefficients from 4x 10^{-6} to 5 x 10^{-9} and 5x 10^{-5} to 1 x 10^{-6} m/s respectively (Costa Filho et al, 1985).

The collapsible behavior exhibited by some residual soils (i.e. laterites) due to their fragile natural structure, can have significant effects on their permeability. When these materials are wetted, under loading, major collapse can occur. After collapse takes place, the structure is destroyed, reducing the void ratio considerably. Consequently, the permeability is also reduced. Itaipú and Itumbiara dams experienced collapse in their foundation while being impounded (De Morais et al, 1985). Guri Dam was also founded on soils classified as collapsible soils (Prusza. 1983). The collapsibility of a soil is also a function of the applied pressure. For Guri Dam, laboratory tests indicated permeability decreased from 2x 10^{-5} to 5 x 10^{-7} m/s at 50 kPa to 600 kPa respectively (Prusza et al, 1983). Although the foundation soils at Guri were characterized as "collapsible", foundation settlements at Guri (more than 2000 mm) took place gradually during construction, reservoir filling and project operation. No rapid settlement within the foundation was observed (Prusza et al, 1983).

One of the consequences of the relatively high permeability displayed by residual soils is that loading and unloading at typical engineering rates rarely results in undrained behavior. This condition excludes soils that have collapsed and have experienced a void ratio reduction, and as a consequence can generate excess pore pressures (Fookes et al, 1997).

6.2 COMPRESSIBILITY AND SETTLEMENT

Like other engineering properties, compressibility is also affected by the considerable heterogeneity of residual soils along the weathering profile. Compression indexes vary significantly in the different horizons of the weathering profile. For the purpose of design settlement estimates, the representability of values used has to be analyzed with utmost care. An evident differentiation between properties of dissimilar layers has to be made.

6.2.1 Collapsibility

A clear understanding of the structure and inter-particle bonding of residual soils is deemed fundamental to comprehend the compressible behavior of residual soils. The structure of residual soil is a product of the weathering processes by which they were formed (Fookes et al, 1997). As an effect of leaching and loss of material, residual soils turn into silty or clayey sand, with high void ratios and an unstable collapsible particle structure (Barksdale et al, 1997).

One of the consequences of residual soils' structure is that they exhibit a yield stress. When a certain state of stress is exceeded they present a discontinuity in their stress-strain behavior, decreasing in stiffness. In certain types of residual soils after the yield stress is exceeded, the structure collapses (Fookes et al, 1997). The collapsibility of residual soils is analogous to a preconsolidated behavior in transported soils. That is why the yield stress is sometimes referred as the “quasi-pre-consolidation” stress. After the yield stress has been exceeded, a true consolidation stress can be determined, and the compressibility of residual soils can be analyzed in the same matter as for transported soils. The equivalent preconsolidation pressure can be a measure of the inter-particle or inter-mineral bonds after weathering (Barksdale et al, 1997). After it is exceeded the bonds are gradually shattered. The collapse usually occurs when the loaded soils are wetted, eliminating the suction in the soil, thus reducing resistance. The collapsible behavior is most important in laterites, since they have been more strongly weathered. Saprolites do not show this characteristic as often.

Since the settlement suffered by in-situ residual soils is small if the yield stress is not exceeded, determining this threshold value is important. Yield can only be evidenced if the soil exhibits a discontinuity in a linear stress - strain plot (Fookes et al, 1997). The use of log-log plots of the same variables can facilitate the identification of the yield stress (Vaughan, 1985). Typical void ratio or strain vs. the logarithm of stress curves can inhibit recognizing the yield point easily (Vaughan, 1985). Fookes et al (1997) summarized the oedometer data gathered by Vargas (1973) illustrating the discontinuity in stress-strain behavior after the yield stress is exceeded.

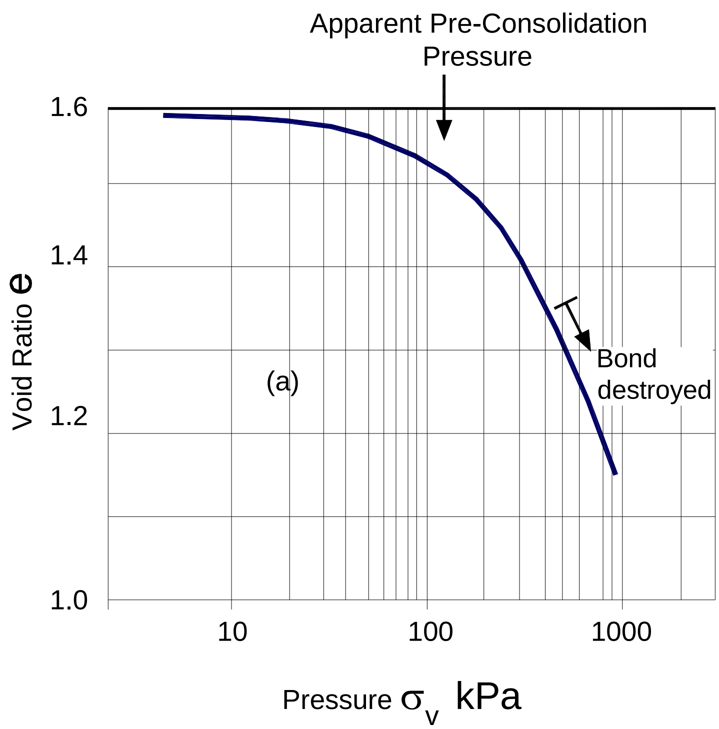

Figure No. 6.1 - Yield Stress for Residual Soils (Vargas, 1973)

As mentioned in the previous chapter, Itaipú and Itumbiara dams experienced collapse in their foundations while being impounded (De Morais et al, 1985). In Guri Dam, the foundation was prewetted, by excavating contoured canals at different elevations and filling them with water. This practice has been followed in places where the water table was found deep, precluding the collapse of the soils (Prusza. 1983). The capillary tension seemed to be the fundamental cause of collapse of Guri´s foundation soils, where the adjacent aggregates were connected together by high negative pore pressure. By increasing the degree of saturation, a reduction of the negative pore pressure occurred, resulting in a micro shear failure which divided the soil structure into individual particles. Considering the effects of permeability, raising the dam in stages, topography and ground water levels, those parts of the foundation which had low degrees of saturation, were prewetted by means of 2700 m of contoured canals at various elevations and filling them with water. A series of 0,5 m. diameter holes were drilled at 20 m. centers from the bottom of the canal and filled with gravel to enable quicker lateral movement of water in the foundation. Each canal was operated during a period of two to four months, depending on the rate of the foundation wetting. During this stage it was possible to determine the large scale permeability ($2x10^{-4}$ to $4x\ 10^{-4}$ cm/s) of the foundation materials.

Besides the aforementioned method of pre-wetting the residual soils to induce collapse, compaction has also been effective. The difficulty with compaction has been determining the depth of potentially collapsible soil. Among the different type of densification techniques that have been used, the following can be listed: vibrating smooth wheeled rollers and impact rollers (with or without prior watering) and dynamic compaction (Barksdale et al, 1997).

Laboratory and field tests can be used to determine the yield stress. In both cases, sample disturbance and stress path followed during the test influence the result. Among the reported tests commonly used are: plate load test, screw plate test, pressuremeter tests, oedometer and triaxial tests (Barksdale et al, 1997).

For the design of Guri Dam one-dimensional consolidation, triaxial consolidation and field plate load test were performed to evaluate the magnitude of compressibility of the residual soils foundation (Prusza et al, 1983). Tests were conducted saturating the specimen prior to loading and saturating the specimen at various intermediate loadings. Total settlement measured by one-dimensional consolidation tests ranged between 10 and 20% under a 980 kPa pressure, whereas that measured in triaxial consolidation and plate tests ranged between 4 to 7% (one third of the measured values using one-dimensional consolidation tests). A structure collapse was detected at loads greater than 400 kPa as shown in Figure 6.2.

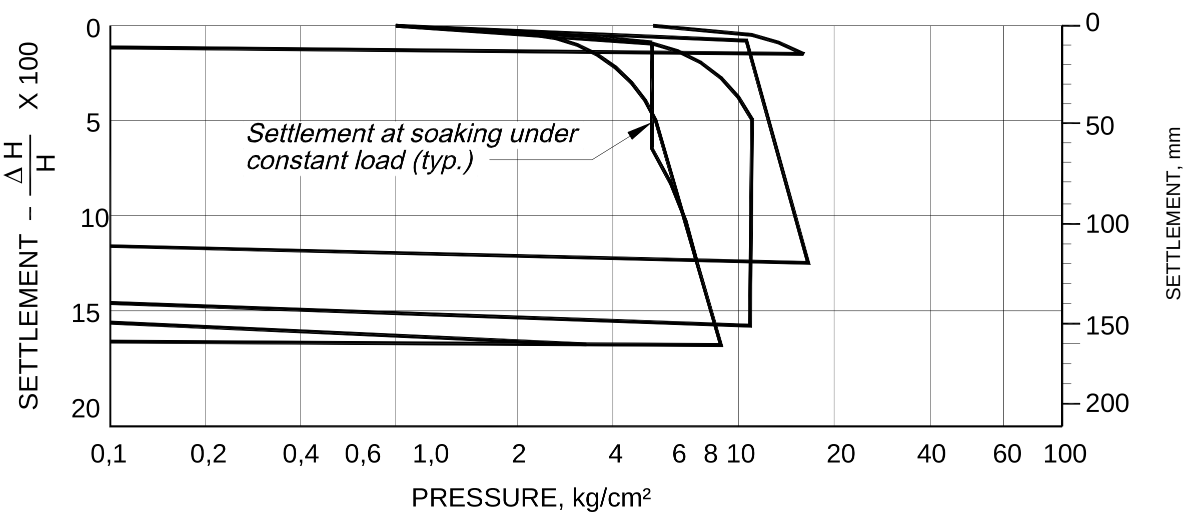

Figure No. 6.2 – Summary of plate load test results performed for the design of Guri Dam (Prusza, 1983).

6.2.2 Compression indexes

As mentioned earlier, residual soils not subjected to disturbance tend to behave as if overconsolidated. Despite this fact, the at rest coefficient K_o has been found to be in some non-expansive materials less than unity (approximately 0,5). This condition has been attributed to the leaching of materials (Barksdale et al, 1997).

The compression index (the slope of the void ratio vs. the logarithm of stress curve) for residual soils, unlike for transported soils, is a function of the yield stress and the initial void ratio. Fookes et al (1997) gathered data from several authors correlating the compression index Ccs and the initial void ratio. The data is presented in Figure No. 6.3.

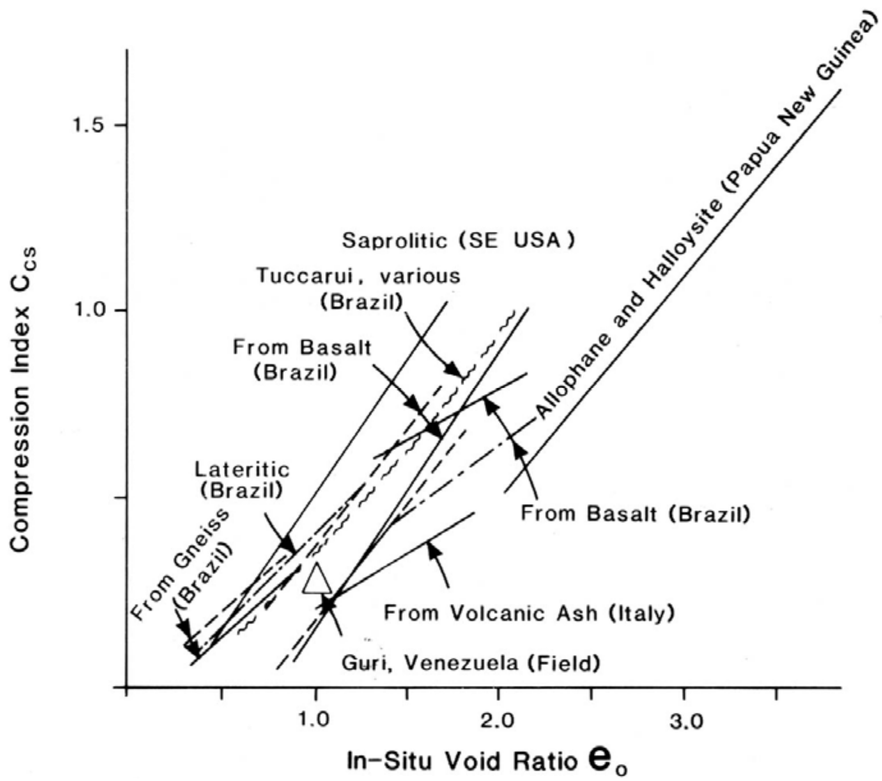

Figure No. 6.3 - Relationship between Compression Index and Initial Void Ratio for Residual Soils (After Fookes et al, 1997).

Blight & Brummer (1980) performed tests correlating compression indices with different variables. The results suggest a direct correlation between initial void ratio (e_o) and compression and rebound indices. Figure No. 6.4 shows the relationship for residual weathered andesite lava.

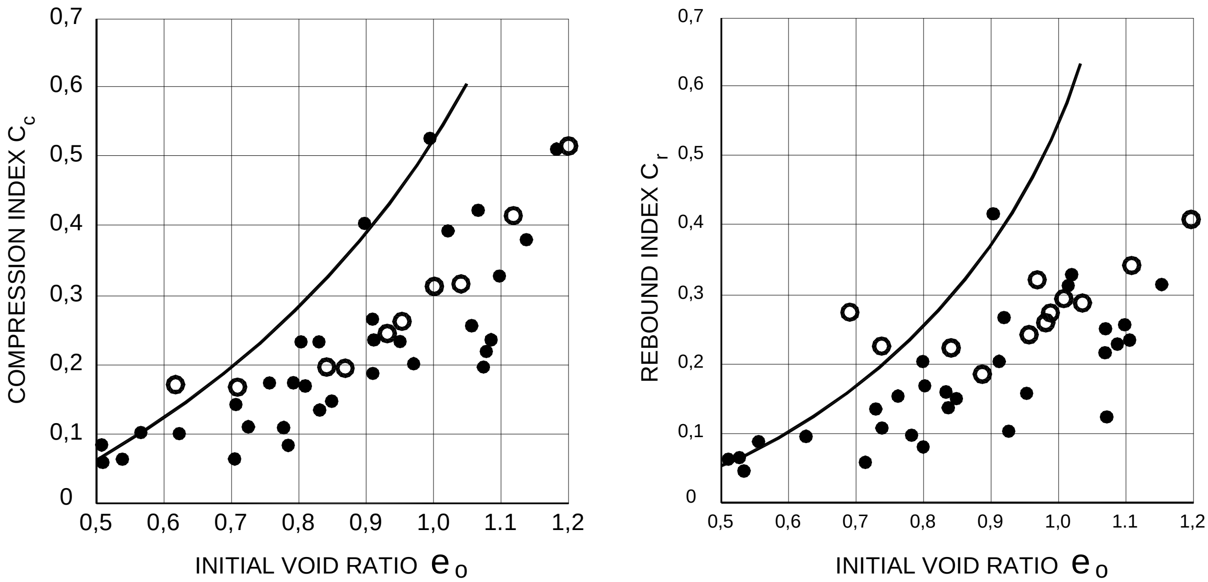

Figure No. 6.4- Correlation between Initial Void Ratio (e_o) and Compression and Rebound Indices (After Blight & Brummer, 1980)

The difficulty of predicting settlements in residual soils can be illustrated with the experience at Guri Dam. Oedometer tests performed in the laboratory predicted settlements between 4 to14 %, while the actual field measured settlements under a load of about 0,3 MPa were in the range of 0.4 to 1.2%. Table No. 6.2 illustrates the considerable difference between predicted and actual settlement values at Guri Dam (Prusza, 1989).

	% Settlement at 700 kPa load	
Data From	**Foundation**	**Fill**
Oedometer tests	4,0 - 14	5,0 - 12
Triaxial Consolidation (Ko) test	1,5 – 4,0	-
Triaxial Anisotropic Consolidation (Ko = 0,5) test	1, 5 - 7,0	1,5 - 3,0
Field Instrumentation	1,5 – 3,0	1,2 – 2,5

Table No. 6.2 - Comparison between predicted and actual settlement values at Guri Dam (After Prusza, 1989)

Consolidation tests for residual soils found at the Sasumua Dam site were performed on a specimen with initial water content equal to the liquid limit (slurry sample) and on several others with an initial water content close to the optimum, to model the compacted fill. The compression index Cc of the slurry sample (Ll= 75, Pl= 47 and IP=28) was estimated around 0.32 and the swelling index equal to 0.0083. The compression index of “normal” clay with the same liquid limit would be about 0.5. Therefore, the compressibility of the Sasumua clay is considered unusually low and close to that of a normal clay compacted at optimum water content (Terzaghi, 1958).

6.2.3 Heave

Heaving of residual soils usually takes place in semi-arid regions, triggered by seasonal changes of moisture in the soil. This phenomenon is mostly present in soils having smectite minerals (montmorillonite), which tend to swell with moisture increases. Residual soils derived from weathered shales, mudrocks and basic igneous rocks are more prone to swell (Barksdale et al, 1997).

The reason for heaving due to increase in water content can be explained with a reduction of suction forces in the soil mass. Usually, the consequences of heaving tend to be more notorious in light structures (i.e. houses). Dams, being massive structures have fewer propensities to be affected.

6.2.4 Compressibility Design considerations

The heterogeneity of residual soils, leads to dissimilar compressibility characteristics at a dam site. This circumstance can cause differential settlements in the dam structure. As a result the dam has to be able to support this type of movements. Experience has shown that the types of dams most suitable for this behavior are the earth core rockfill, homogeneous and concrete face rockfills.

If a rockfill with central core dam is chosen, it is important to guarantee that the core settles less than the adjoining shells, avoiding a core-hangup that can generate hydraulic fractures (De Mello et al, 1985).

Residual soils are compressible and the majority of deformation (more than 95%) usually occurs during the construction the dam fill and the next six months. This type of behavior was observed in the Antioqueño Batholith Dams (Colombia).

6.3 COMPACTION PROPERTIES: MAXIMUM DRY DENSITY AND OPTIMUM WATER CONTENT

Residual soils have been extensively used as fill materials. They can be very appropriate for a central core, due to their low permeability after compaction. Residual soils also have been commonly utilized in homogeneous dams, providing adequate shear strength for the slopes of the dam body and a watertight barrier.

The high variability of tropical residual soils complicates the control of the compaction properties of a fill. Obtaining representative samples along a weathering profile can be misleading. Also, controlling water content in tropical residual soils during placement can prove to be a difficult task, since dam sites located in these areas present high precipitation. This issue is further explained later in this chapter with some case histories of Colombian dams. Mineralogy of residual soils also presents additional challenges, since soils containing halloysite and smectites can present inadequate strength for compaction or can exhibit intolerable volume changes after hydrating (Simmons et al, 1997). Despite this limitation, soils containing the aforementioned mineralogy have been used as impervious layers in Sasumua and Arenal Dams (Simmons et al, 1997).

The compaction characteristics of residual soils should be determined without prior drying, especially when drying sensitive minerals are present (i.e. allophane and halloysite). The optimum water content is not the only variable affected by drying; the maximum dry density is also disturbed. Simmons et al (1997) show considerable effects on these two variables in a lateritic soil in Ghana (Figure No. 6.5).

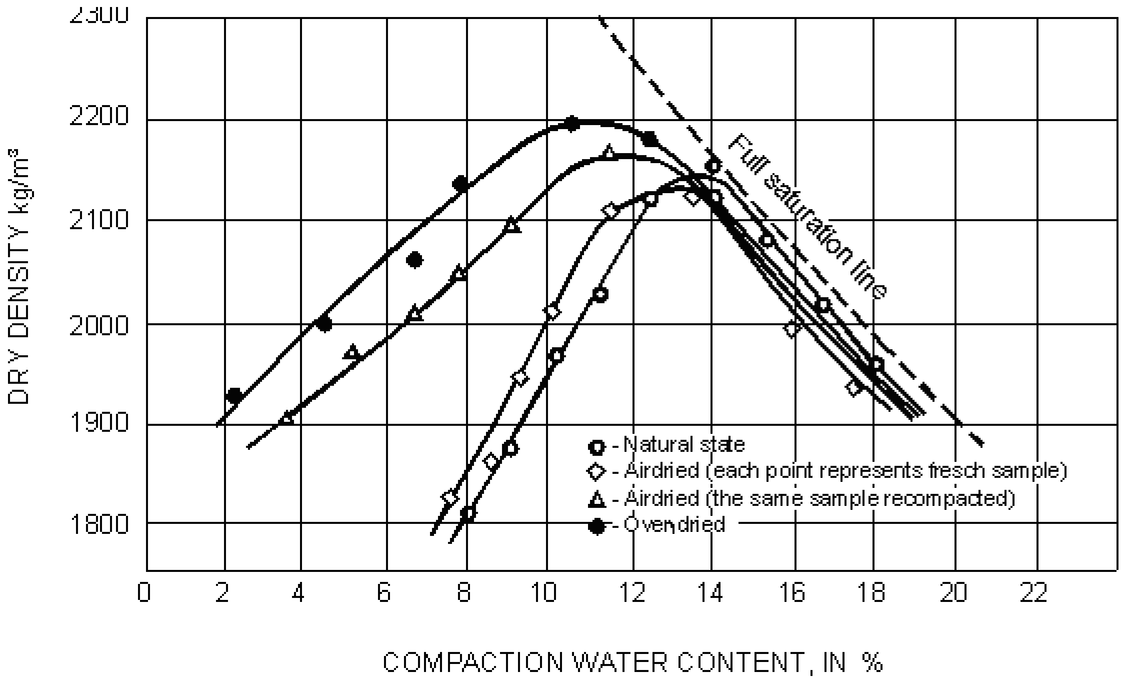

Figure No. 6.5 – Sample Preparation and Drying Effects on Compaction Characteristics of a Lateritic Soils in Ghana (After Simmons et al, 1997)

Compaction energy can also influence the compaction characteristics of residual soils. For instance, this behavior can take place in lateritic soils presenting concretions. The compaction effort can break down the bonds and cementation between particles, increasing the fine fraction of the soil. To diminish this problem, it is recommended to utilize fresh samples for every point to determine the compaction curve (Simmons et al, 1997).

In order to achieve a better representativity of the anticipated compaction characteristics of the soil, test embankments are recommended to establish the compaction properties of the fill materials in the field. At Guri Dam trials at the test embankments showed that the sheepfoot rollers were most appropriate. It was also evidenced that the residual soils derived from gneiss found at the site should be compacted in layers 0,2 m thick at the most with six passes of the roller to achieve 96% of the maximum dry density, measured with the Hilf rapid compaction control method. In general terms, for this case, the Hilf method gave a higher compaction percentage compared with the Standard Proctor Test (Choudry et al, 1979). The difference in values has been attributed to the effect of drying in the Standard Proctor Test that dehydrates the halloysite and sesquioxides present in the soil. If the sample is not dried before executing the Standard Proctor Test results tend to be similar for both methods.

Pad-foot rollers for compaction of the fill at Guri Dam were very effective, always reaching a compaction percentage higher than 96% and on average higher than 98%. This was accomplished if the water content was kept within 2% of the optimum (Choudry et al, 1979).

At Guri, where an appreciable amount of water was required to be added to borrow materials (6-8%), a significant amount of equipment would have been required to haul, spread, add water and compact the material to meet the monthly peak productions requirements of about $2x10^6 m^3$. Prewetting of the borrow areas was successfully performed using sprinkling techniques. The results indicated that prewetting up to 15 m depth could be achieved with 15 to 39 days of sprinkling depending on the rate and average permeability of the area being prewetted. Sprinkling showed to be the most economical method for prewetting the borrow materials, compared to other methods that involve either premixing in staging areas or addition of moisture during fill placement.

At Punchiná Dam (Colombia) Villegas (1982) reports the following material average characteristics after compaction:

Characteristics	Average value
Maximum dry density (t/m^3)	1,65
Optimum water content (%)	20,1
Field dry density as % of max. γd (t/m^3)	95,4

Table No. 6.3 - Punchiná Dam, compaction characteristics (Villegas, 1982)

The soils used for construction of the fill material at Punchiná were mainly sandy silts classified as SM in the USCS (Villegas, 1982). The placement of the fill at Punchiná Dam was simplified because the natural and optimum water contents differed only 4.1% on average. The high precipitations of the area allowed only for a short construction season and consequently the contractor was allowed to place the material with the same water content as exploited (Villegas, 1982).

6.4 STRENGTH CHARACTERISTICIS

The analysis of shear strength behavior of tropical residual soils demands taking some unique factors into consideration that are not as relevant for transported soils. Furthermore, the shear strength characteristic of saprolitic soils is very different compared to the behavior of laterites. In the first case, the shear strength characteristics and controlling factors for saprolites resemble that of rocks because of the presence of the structure and texture of the parent rock.

The most relevant factors influencing shear strength of residual soils are the following (Brenner et al-1997, Fookes et al, 1997):

- Stress history.
- Bonding between particles.
- Mineralogy.
- Relic structures and discontinuities.
- Anisotropy.
- Widely variable void ratio.
- Partial saturation.

6.4.1 Stress history

Unlike transported soils, stress history does not have an important effect in the shear strength of residual soils. Weathering progressively modifies the state of stresses that is followed by and adjustment of the weathered material to reach a state of equilibrium with the prevailing conditions (Brenner et al, 1997).

6.4.2 Bonding between particles

Leaching and solution of cementing agents generate bonding between the soil mass particles. The bonds tend to be fragile causing a brittle shear strength behavior. Moreover, inadequate sampling methods destroy the bonds, and as a consequence shear strength is underestimated. The bonds tend to have a greater influence on the behavior of a large mass in situ and decrease with a progressive weathering (Brenner et al, 1997). Accordingly, small laboratory tests do not properly reflect the benefit of bonding in shear strength. Brenner et al (1997) report that in partly weathered rock bonds provide high strength (greater than 200 kPa), while in mature residual soils the bonds tend to much weaker (lower than 100 kPa).

As mentioned in chapter 6.2 (Compressibility and Settlement) interparticle bonds are very sensitive to saturation, and can be destroyed if the soil is stressed during saturation (Brenner et al, 1997).

6.4.3 Drained vs. Undrained shear strength

The adequate planning of a laboratory testing program to estimate shear strength requires anticipating what type of shearing behavior the soil will present: drained or undrained. The shearing behavior of residual soil depends if the material is in its natural state or compacted, and the loading case being considered.

Generally, compacted residual soils present an undrained shearing behavior, due to their low permeabilities that inhibit dissipation of pore pressures (even for loads relatively slowly applied). Compaction of residual soils destroys the relic features that promote drained behavior. Furthermore, the amount of fines inhibits drainage. Several case histories of dams experiencing high pore pressures in their residual soil fills have been reported. The compacted material normally presents negative pore water pressures and thus shear strength is reduced during filling. For these circumstances short-term conditions are the most critical. Hence, slope stability problems can occur during construction and have to be closely monitored. This situation was experienced with the dams built on the “Antioqueño” Batholith residual soils in Colombia (Piedras Blancas, Quebradona, Troneras, Miraflores, Santa Rita I and II, Punchiná and San Lorenzo). The region where these dams were constructed presents high precipitations (2100-5300 mm per year) with only a short dry season of 3 months during the year. The only easily available materials are residual soils with water content 2 to 10% above optimum (Villegas, 1984). Under these difficult circumstances, almost all the available materials were placed at their natural water content (without prior treatment) at a high rate to take advantage of the short construction season. This procedure leads to high pore pressures and excessive strains (Villegas, 1984). The monitoring of these pressures with piezometers was fundamental to avoid possible failures and the construction sequence was adapted according to the observed behavior. Long term monitoring of the pore water pressures during reservoir operation was difficult, because large settlements damaged a high percentage of the instruments.

On the other hand, the shearing behavior of residual soils in their natural state is controlled by the degree of weathering. The weathering of residual soils generates leaching of the material, bio-channels and large void ratios, promoting a drained behavior in saprolitic soils. Inherited geological structures, such as fractures and bedding planes also encourage dissipation of pore pressures in this type of soils. In more mature residual soils (i.e. laterites) the formation of clay minerals reduces their permeability and generates pore pressures. In this case the shearing behavior is undrained.

6.4.4 Influence of Mineralogy on shear strength

The general characteristics of minerals present in residual soils were described in Chapter 4. Regarding shear strength, soils containing smectites are the most problematic. These platey clay minerals exhibit low angles of friction. Additionally, displacements orient these minerals reducing the shear strength to a residual value. This condition can be identified by the presence of slickensided surfaces.

During the initial studies of Agua Vermelha Dam slickensided surfaces were recognized in the first observations in pits, forcing to adapt the design to lower shear strength values. The volume of material involved was very large and its removal was not feasible. The chosen approach was to perform a thorough study of the geotechnical properties and instrument the foundation (De Morais Leme et al, 1985). Additionally, a stabilizing berm was built to provide a factor of safety of 1 for residual strength. A secondary berm was foreseen in case unacceptable movements occurred. After detailed observation, installed inclinometers registered displacements as high as 25 mm, thus forcing the construction of the second berm to avoid a progressive failure (De Morais Leme et al, 1985).

Fookes et al (1997) reported typical angles of shearing resistance for residual soils containing residual clay minerals. The values are valid for soils in which the clay minerals are the prevailing shearing mechanism (Fookes et al, 1997).

Clay Mineral	ϕ'_{CV}	ϕ'_{R}
Smectites	15-20°	5-11°
Kaolinites	22-30°	12-18°
Allophane	30-40°	30-40°
Halloysite	25-35°	25-35°

Table 6.4 - Shearing Resistance Angles According to Clay Mineralogy (Fookes et al, 1997).

6.4.5 Relic structures and discontinuities

As mentioned earlier, the shear strength of saprolites resembles that of rock, because of the relic structures and discontinuities they inherit from the parent rock. These features are difficult to identify by borings (Brenner et al, 1997). They are easier to detect using test pits. Usually, saprolites fail under shear along these planes of weakness (Brenner et al, 1997). Several authors have reported that the overall strength of the soils mass is controlled by the prevalent "orientation and frequency of structural features in relation to the direction of stress application, and to the strength characteristics of these features" (Brenner et al, 1997).

Relic structures are partly responsible for the anisotropic behavior of residual soils. Bedding planes, schistosities, fractures and joints inherited from the parent rock are accountable for different responses to shear, depending on the shear stress application in relation to the structural features (Brenner et al, 1997). Wolle et al (1985) report shear strength parameters for residual soils tested perpendicular and parallel to different types of structural features. Table 6.5 summarizes these results. Notice the large differences depending on the direction of testing.

Material	*Macro-structure*	*Parallel*	*Perpendicular*	*Remarks*
Saprolitic soil from ferrifereous quartzite	Laminated (Silty sand)	c= 20 kPa ϕ=37°	c= 20 kPa ϕ=37°	Partially saturated
Saprolitic soil from micaceous quartzite	Schistous (Sandy silt)	c= 40 kPa ϕ=22°	c= 45 kPa ϕ=27°	Partially saturated
Saprolitic soil from migmatitic geniss	Banded	c= 40 kPa ϕ=20°	c= 52 kPa ϕ=23°	Partially saturated
Saprolitic soil from migmatitic geniss	Banded	c= 30 kPa ϕ=21°	c= 49 kPa ϕ=22°	Submerged

Table 6.5 - Shear Strength Parameters for Residual Soils Tested Perpendicular and Parallel to Structural Features (Wolle et al, 1985).

Another consequence of the macro-structure of saprolites is that the stresses distribute in an exceptionally heterogeneous manner within the soil mass. Great stresses concentrate on the framework and remaining nucleuses (Wolle et al, 1985).

6.4.6 Partial Saturation

It was mentioned before that tropical regions where residual soils are commonly present often present depressed groundwater tables. Hence, tropical residual soils are commonly not saturated. Pore pressures tend to be negative, due to capillary effects within the pores of the soil (Brenner et al, 1997). As a result, the suction in the soil increases the effective stress and consequently the shear strength. However, for design purposes this additional component of strength should not be accounted, because it can easily disappear after saturation. This condition takes place despite high precipitations common in tropical regions where these soils are present, due to the impermeability of the upper layers.

6.4.7 Determination of shear strength parameters

Typical tests used for determining shear strength of transported soils can be utilized for residual soils. The main differences lay not on the apparatus, but on the testing procedures and interpretation. The major disadvantages with laboratory testing are the scale factor and representativity. Commonly, the scale of relic structures exceeds that of the sample. Hence, weak planes that frequently dominate shearing resistance in residual soils are not reflected in the determined parameters. For laboratory tests the common direct shear tests and triaxial tests are routinely carried out.

Field testing has the benefit of reflecting the in-situ features that may dominate the shearing behavior. Field direct shear test, vane shear test, pressuremeter test, and Cone Penetration Test have been frequently used. Large shearbox tests are appropriate and give reasonable results, but have the disadvantage of being costly and time consuming.

Back-calculating parameters from slope failures in the area can be a good approximation, because several aspects that are difficult to take into account in tests, are incorporated in the analysis.

Correlations with index properties can be misleading and present large scatter. De Mello (1985) warns against the errors and danger of using SPT (Standard Penetration Test) and dry density indices to estimate foundation competence; particularly in soils exhibiting high anisotropy (i.e. relic structures). For Itumbiara Dam (Brazil) SPTs were performed. Results showed a large scatter.

Shear strength parameters determined from the compacted material for Chinese Dams using laterites as fill material are presented in Table 6.6.

Name of Dam	Year completed	Performance	Type of Dam	Degree of Compaction relative to maximum density	Angle of friction	Cohesion (kPa).
Luyinhe	1961	Normal	Homogeneous	0.89	11	36
Guodihe	1959	Normal	Homogeneous	1	24	67
Luomatang	1981	Damaged	Homogeneous	.88	11.3	34
Yaoshang	1966	Damaged	Homogeneous	0.87-0.9	12	43
Xiaoqiaohe	1981	Damaged	Sloping core	0.88-0.9	14	27
Wangguan	1958	Normal	Homogeneous	0.89-0.91	16.2	38

Table 6.6 - Shear strength parameters from compacted laterite fill exhibited on Chinese Dams (Liusheng, 1985).

Figure 6.6 shows triaxial tests results, consolidated drained and undrained, for residual soils (i.e. andosols) found at Tjipanundjang Dam. Results for both type of tests and for two different sites are very similar and present little scatter. Shear strength parameters, ϕ and c', were found to be around 35º and 24 KN/m^3 respectively. These values were used for stability analysis and were considered conservative (Wesley, 1974). However, these values are quite high for a soil containing a clay content around 70%. Terzaghi attributed the high shear strength both at Sasumua and Tjipanundjang dams to the fact that particles are cemented together and form clusters. Latter tests performed at Sasumua indicate that the high shear strength is more related to individual particles and not to the formation of clusters (Wesley, 1974).

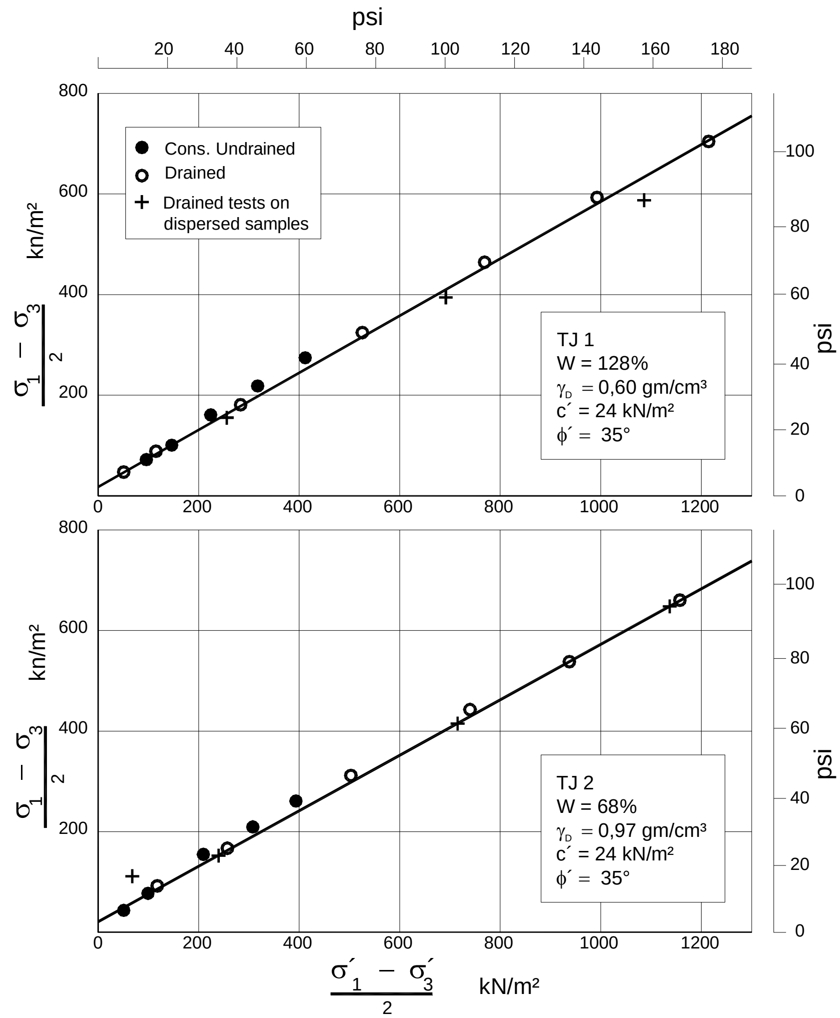

Figure 6.6 – Triaxial tests results from residual soils found at Tjipanundjang Dam (Wesley, 1974).

The initial design stability analysis for Punchiná Dam was performed based on the average triaxial tests results of the material at its natural water content. The following effective strength parameters were reported for the fill material and foundation:

Material	c' (kPa)	ϕ' (°)	γ_t (t/ m^3)
Residual Earthfill	15	30	1.9
Foundation-Horizon IC (SM)	12	34	1.9

Table 6.7 – Shear strength parameters used for Punchiná Dam during design.

Effective shear strength parameters of the in-situ residual soils found at Punchiná Dam and other Antioqueño Batholith Dams were high despite their low density. The cohesion and friction angle were around 10 kPa and 30° respectively. Undrained resistance was also found to be high, with average values close to 60 kPa.

Test samples compacted in the laboratory were used to determine shear strength parameters for Corumbá I Dam. Table 6.8 presents the average values for these tests (Mori, 1996).

Test	Parameters	Saprolitic Soil Upper Layer	Saprolitic Soil Lower Layer
Triaxial UU (non saturated)	c'(kPa) ϕ' (°)	44 29	32 31
Triaxial CU and CD (saturated)	c'(kPa) ϕ' (°)	41 30	5 33

Table 6.8 – Shear Strength Parameters for Saprolitic Soils Found at Corumbá I Dam.

Drained and undrained triaxial tests with pore pressure measurement were performed on placed fill at Sasumua Dam. Both type of tests indicated that the angle of friction was practically independent of the water content. The results ranged between 30° and 36°, with an average of 34°. On the other hand, an empirical relationship was established to estimate the cohesion in terms of the water content. The cohesion for the dowstream portion of the dam was estimated around 26 kPa and for upstream portion 30 kPa (Terzaghi, 1958).

For the design of Guri Dam in Venezuela isotropically consolidated undrained (CU) triaxial tests with pore pressure measurement were performed (Prusza et al, 1983). They showed a substantial increase in pore pressure as the sample was sheared, due to a progressive collapse of the soil structure. The deviator stress reached a peak at relatively low strain and decreased moderately with increased strain. Generally the tests performed for this project showed that the drained strength is at least 10 to 15% higher than the undrained strength. A design strength of ϕ'21.5° and a c'= 20.6 KPa was selected for the decomposed granitic gneiss foundation materials based on a strength equivalent to 10% strain in CU tests.

Plenty of test results performed on lateritic and saprolitic soils, related to the construction of more than 50 Brazilian Dam have been reported. Based on these data correlations between shear strength properties and index properties have been developed. Cruz & Maiolino (1983) proposed correlation between plasticity index and φ' presented in Figure 6.7.

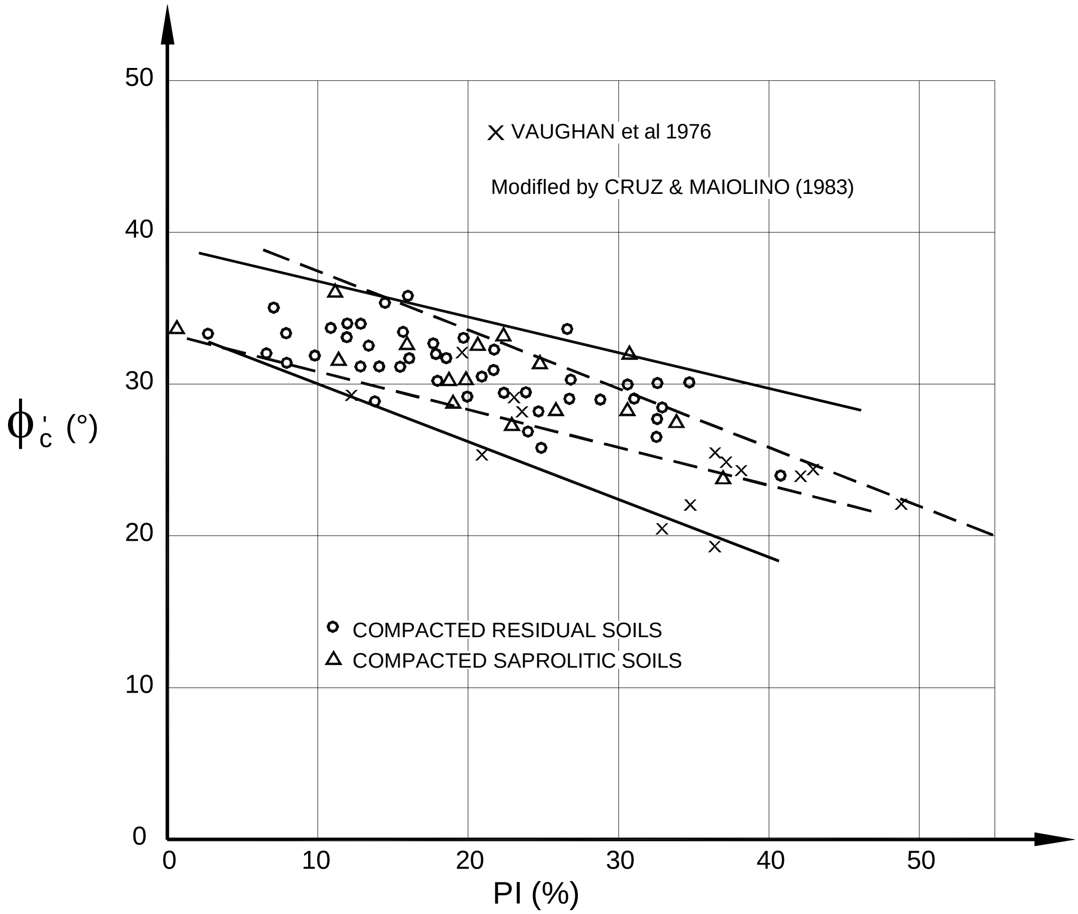

Figure 6.7 – Characteristic φ' Vs. Plasticity Index for residual soils found in Brazilian Dam Sites (Cruz & Maiolino, 1983).

7 ADDITIONAL DESIGN CONSIDERATIONS

7.1 TROPICAL RESIDUAL SOILS AS DAM FOUNDATION MATERIAL

7.1.1 FOUNDATION COMPETENCE

The competence of the foundation materials should be evaluated, among other variables, in terms of their compressibility. The Modulus of Elasticity of the foundation material should be compatible with the fill material deformation characteristics.

Attempts have been made to predefine excavation levels based on Deere and Patton's (1971) weathering profile. This approach is inconvenient. Deere and Patton's classification was developed to unify criteria with regards to weathering profiles and to apply precedent for stability evaluation of slopes. Their description was not intended to cover adequacy of foundations. A more reasonable method is to estimate the material modulus of elasticity E, based on geophysical methods, plate tests or laboratory tests. Foundation limits can then be established based on minimum seismic velocity values that correlate with the deformation modulus of the rock mass. The minimum adequate value depends on the particular type of structure contemplated for the site.

7.1.2 FOUNDATION TREATMENT

Tropical residual soils are typically composed of silty sands. Silty sands are particularly susceptible to piping and internal erosion. Hence, foundation treatment should be aimed at controlling piping and internal erosion. Generally, extensive filters and drains along the foundation (especially along the downstream shell) have proven to be effective to inhibit the occurrence of piping for this type of dams. These measures have been successfully implemented for the Antioqueño Batholith Dams, Corumbá I and Guri. De Mello et al (1985) recommended the use of two elements to deal with foundation seepages in residual soils: the use of a cut-off across the pervious horizon and the increase of the seepage path by utilizing an internal impervious blanket. On the other hand, they discredit the use of external blankets. Additionally, they recommend a strict control of the rate of rise during the first filling, in order to detect foundation seepage and treat it properly. These practices diminish the hydraulic gradients that can lead to piping and erosion. During the testing program of Guri, water soluble salt analysis showed that some samples plotted in the dispersive Zone A, of the percent sodium vs. total dissolved salt (meq/liter) plot. However, more than 300 pinhole and crumb tests indicated that the decomposed residual soil was basically non dispersive. Double hydrometer tests with and without dispersants also showed the same results.

The effectiveness of grouting treatments for residual soils is somewhat dubious. At Salvajina concrete face rockfill dam in Colombia (Sierra et al.1985), throughout the entire foundation for the toe slab (i.e. plinth) consolidation grouting was carried out, except for an area at the right abutment were residual soils were left in place due to the considerable thickness and consequent cost of removing them. It was deemed that this type of treatment would not be of any benefit in this type of material because of its low permeability. For the Antioqueño Batholith dams it was also considered that grouting treatment of saprolite and residual soil foundations was ineffective to control seepage.

Figure 7.1- Residual Soils Found at the Right Abutment (Far Left) at Salvajina Dam that mandated special treatments at the Plinth and Foundation.

7.1.2.1 Cut-off Walls

To avoid erosion of residual soils, as well as the sandstones and siltstones altered by hydrothermal activity at Salvajina Dam (Sierra et al.1985) a positive cut-off was utilized. The cut-off was 1m wide, extending to a maximum depth of 3m, filled with concrete and a water stop was provided. At the area of the foundation where residual soils were present the characteristics of the cut-off wall were modified to avoid cracking at the wall due to expected settlements. At this sector the depth of the wall was limited to 0.6m and it was backfilled with asphalt –impregnated sand (Sierra et al.1985). The location of the cut-off trench is illustrated in Figure 7.2.

At Corumbá I Dam (Mori. 1996), located in the South-Central region Brazil, the earth core was partly founded on top a natural saprolitic soil at the left abutment. To control seepage through the core foundation and avoid erosion of this material, a concrete slab (0.1 m thick, reinforced by a 0.1 X 0.1 m and ϕ 3/16'' steel mesh) isolates the core from the foundation. This way potential concentrated flow through old fractures, some with clayey infill, will not endanger core stability by eroding it. Additionally, a filtering blanket covering all the foundation area of the left downstream rockfill was placed to avoid internal erosion (Mori. 1996).

Observations in deep cuts for the Antioqueño Batholith dams (Colombia) have shown that even the saprolite and weathered rock (IC and IIA) exhibit low permeabilities, with little open joints due to lixiviation of the material from the upper layers. Additionally, in situ permeability tests showed that the permeability of the silt and weathered rock only differed in one order of magnitude. Therefore it was deemed that expensive cut-off walls or extensive impermeable layers were not required to guarantee the integrity of the soil formation close the fill. Alternatively, the continuity of the silty layers downstream of the dam was assured, the capacity of the drainage system at the downstream shell was increased and drainage collectors at the foundation level were provided.

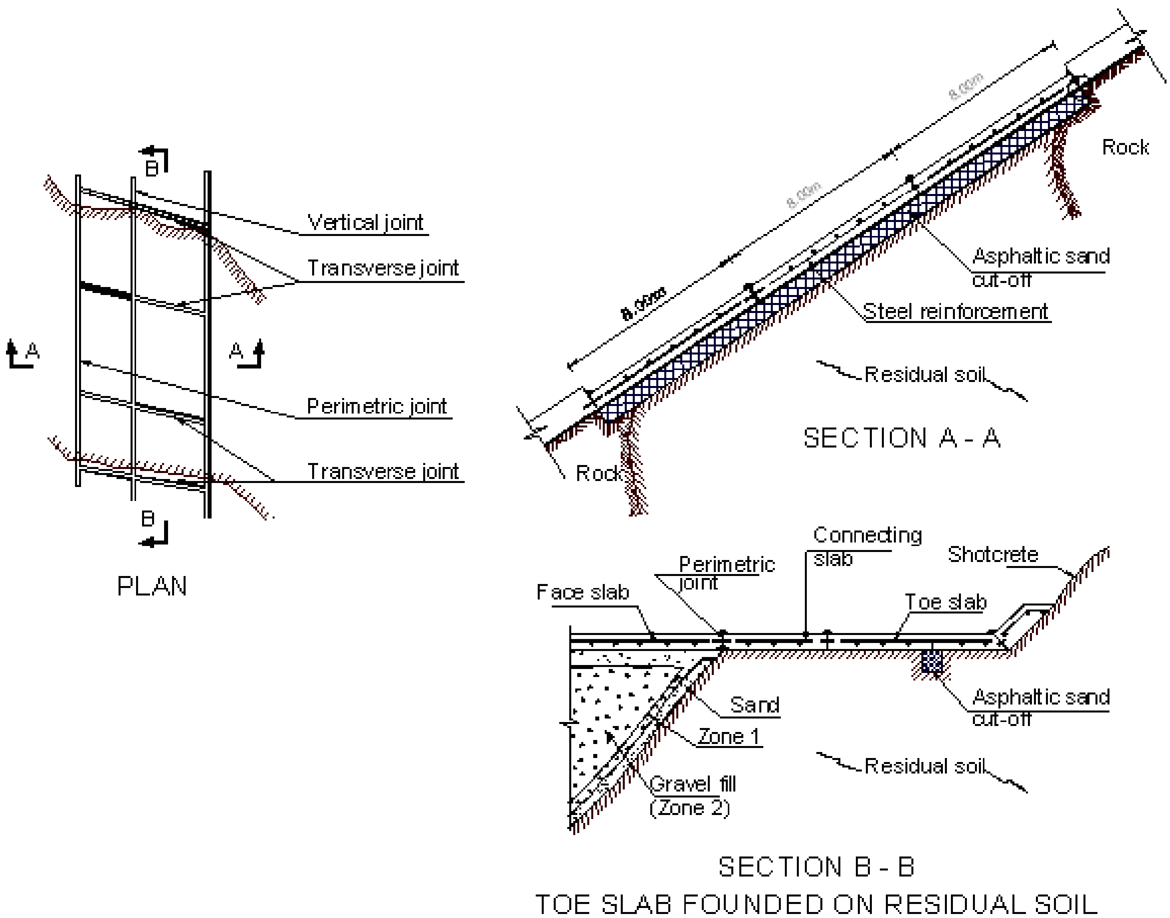

Figure 7.2- Plinth and Slab Design at Salvajina Dam Founded on Residual Soils (Sierra et al.1985)

Figure 7.3- Corumbá I Earth Core Rockfill Dam, Brasil.

At Guri, a rockfill dam (Figure 7.4) with an impervious central core, with a maximum height of 90 m, the foundation is traversed by 60 m wide and 100 m deep inactive fault, filled with a saprolitic soil derived from gneiss (Medina et al, 1985). To deal with this challenging foundation, treatment was carried as follows: (a) the material in the fault was excavated to a practical depth (30 m under the core), (b) the excavated fault upstream of the core was covered with an impervious blanket that was separated from the rockfill by two layers of filters, (c) the excavated fault downstream of the core was covered by two layers of filters each 2 m thick (Medina et al, 1985).

Figure 7.4- Guri Dam, Venezuela

7.1.2.2 Adjustments to the Plinth of CFRDs to Support the Slab on Residual Soils

Very little precedent exists of a CFRD dam supported on residual soils. One of the few cases is Salvajina Dam in Colombia. At this dam a settlement analysis, based on laboratory tests and plate tests of the residual soils present at the right abutment, showed that upon water loading the plinth resting on these materials would experience significant differential settlements on the order of 0.125 m, with respect to the adjacent plinth resting on rock. Cracking of the slab with subsequent seepage of water was particularly undesirable, because of the highly erodible nature of the residual soils (USCS classification MH-ML; 92% passing No. 200). To cope with this situation the design of the plinth was adjusted for this sector. Four transverse cold joints with PVC waterstops and mastic protection were provided; two of them at the boundaries between the residual soil and the adjacent rock. To increase flexibility, the thickness of the toe slab (i.e. plinth) was reduced to the same thickness of the adjacent connecting slab and the concrete cut-off was replaced by a cut-off made of asphalt impregnated sand (Sierra et al.1985). The details of the plinth design of Salvajina are presented in Figure 7.2.

Due to the heterogeneity of residual soils, excavations to clean the foundation can be difficult to estimate and execute. Therefore, there is a possibility that foundation treatment is not enough or impractical along narrow fractures and erosion of the fracture infilling is something to be considered. To preclude this type of erosion a filter should cover the foundation for a certain distance behind the plinth. An approximate criterion of a distance equal to 1/3 of the maximum water pressure can be implemented.

7.2 TROPICAL RESIDUAL SOILS AS DAM FILL MATERIAL

Tropical residual soils can become the most attractive alternative dam construction material under certain conditions. Weathering profiles, commonly 30 m deep, imply important amounts of waste material that cannot be used as aggregates for concrete. Additionally, the variability of the extent of weathering in short distances hinders acceptable estimation of quarry volumes for aggregate production. This situation can lead to possible claims by the contractor. Consequently, the production of concrete under these circumstances can be costly. The compressibility of this material is not compatible with the stiffness of concrete. With this in mind, mature residual soils and saprolites have been used as filling material in several dams around the world.

7.3 Construction Methods

7.3.1 COMPACTION REQUIREMENTS

Typical compaction design requirements for fill materials have been modified for dams using residual soils. Usually, maximum dried density and optimum water contents are determined for fill materials, based on laboratory tests (Proctor or Modified Proctor) and it is specified that the Contractor has to achieve a certain degree of compaction. However, conditions often encountered in regions where residual soils are present complicate the application of these criteria. High precipitation in tropical areas (Table 7.1), lead to available materials having water contents above optimum. Moreover, short dry periods and intense rain forced to shorten placement cycles of fill material. In order to satisfy this reduced construction programs, selection or drying of materials is prohibited.

Dam	Height (m)	Annual Precipitation average (mm)	Average Fill Increase (m/day)	Maximum fill increase during placement season (m) (between 15 and 105 days)
Piedras Blancas	25	2300		
Quebradona	34	2500	0.31	21
Troneras	37	3250	0.37	32
Miraflores	63	3050	0.40	40
Santa Rita I	30	5350	0.5	25
La Fe	40	2320	0.2	14
Santa Rita II	60	5350	0.5	10
Punchiná	75	3100	0.26	27
San Lorenzo	55	3400		
Las Playas	65	3300		
Riogrande	65	2100		

Table 7.1- Height, Annual Precipitation and Construction Rates for Antioqueño Batholith Dams

Under these circumstances, if the materials cannot be processed or modified adequately, the construction methods have to be adjusted. In the case of the Colombian Dams built on the Antioqueño Batholith, available materials exhibited water contents between 2 and 8% above optimum (Villegas, 1984); see Figure 7.2. To be able to use these materials without prior treatment, compaction energy was adjusted according to water content. As shown in Figure 7.3, roller weight and number of passes (compaction energy) was adjusted for the range of available materials. This way, waste material was reduced and fill placement productivity increased.

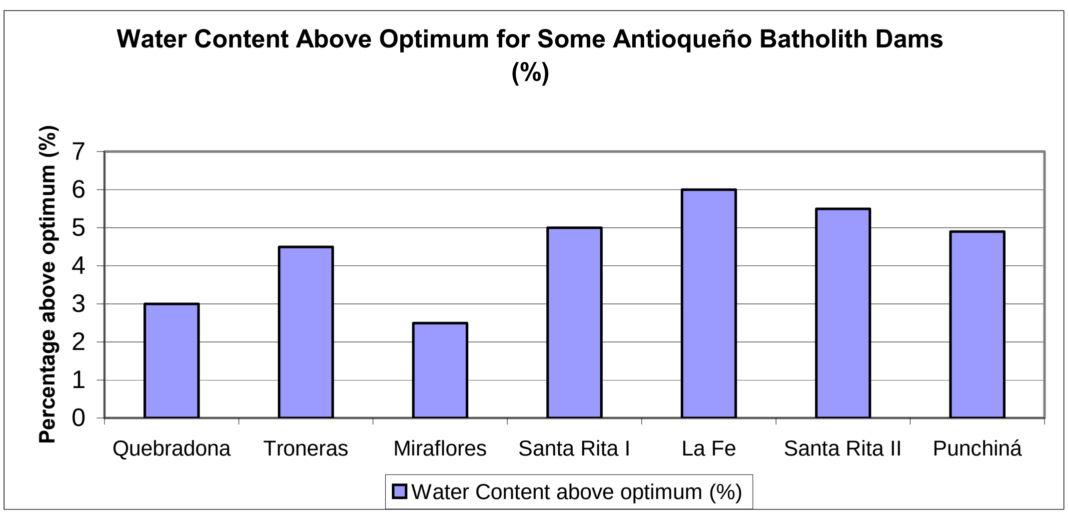

Figure 7.2- Water Content above Optimum for Several Antioqueño Batholith Dams

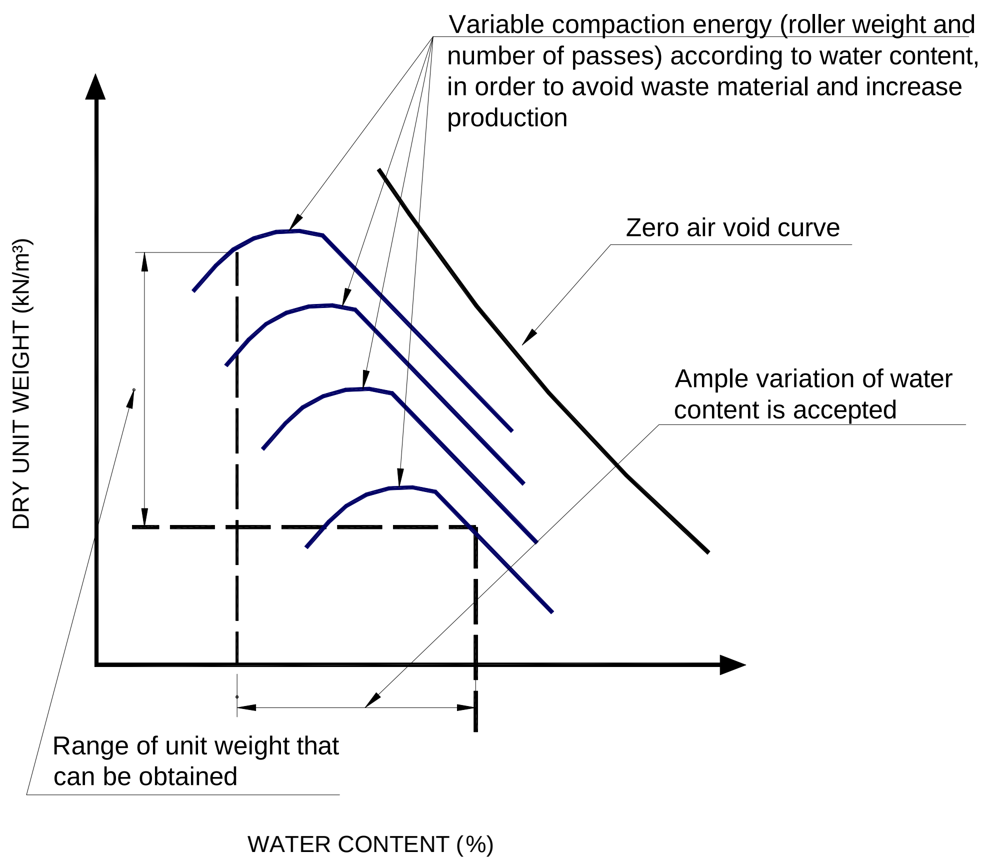

Figure 7.3- Compaction Control Criteria for Antioqueño Batholith Dams

At the Omai Gold Mine in Guyana, available materials to build tailing embankments with a maximum height of 55m and overall length of about 4000 m, also presented water contents of the saprolitic materials above optimum water contents for compaction. Placement water contents ranged from 12 to 45% (averaging 31%) or up to 17% above optimum water contents in some cases (Bedell et all, 2002). For this project an alternative compaction control criterion that took into consideration variable water contents was implemented. For water contents drier of the optimum, 95% of the Standard Proctor maximum dry density was specified. For higher placement water contents, 98% of the Standard Proctor dry density at that water content was demanded (Bedell et all, 2002). The abovementioned compaction control criterion is presented in Figure 7.4.

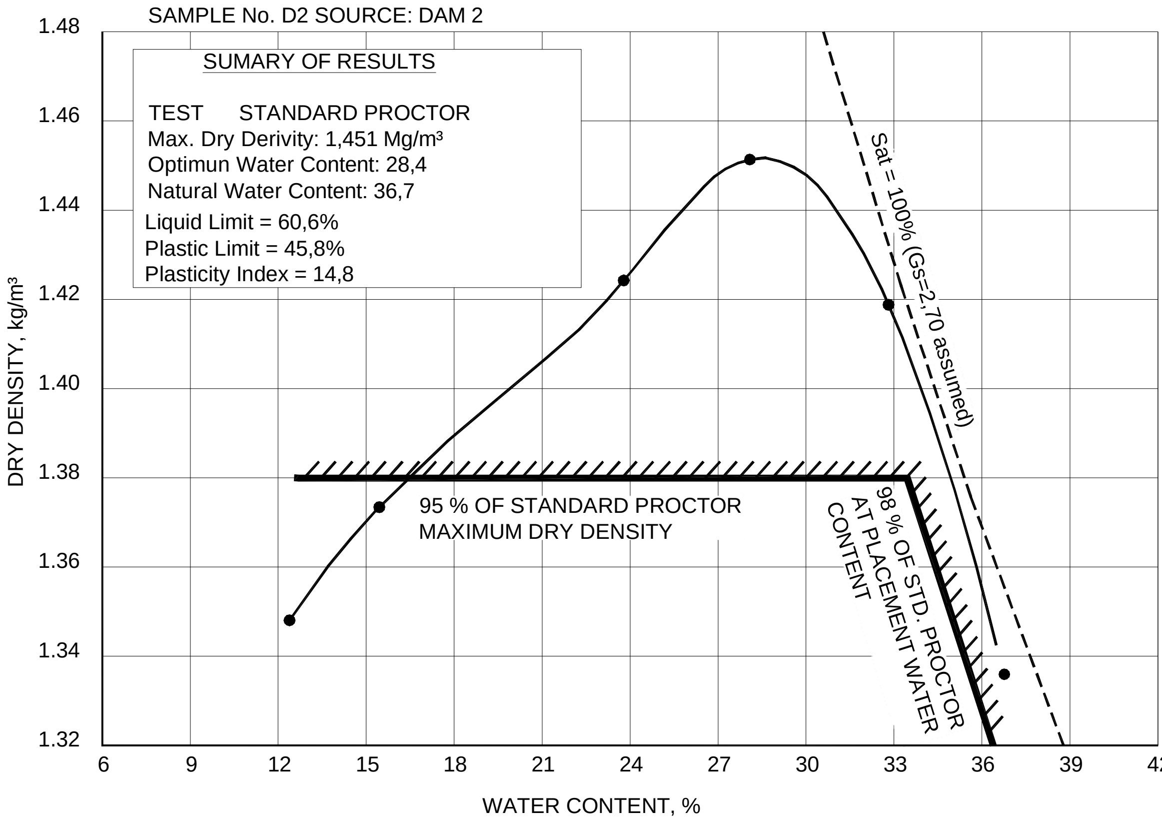

Figure 7.4- Compaction Control Criteria for the Omai Gold Mine Tailings Dam (Bedell et all, 2002)

It is advisable to change working fronts frequently, to avoid excessive concentration of pore pressures and maintain a uniform increase of fill height for stability purposes. This method was successful at the Antioqueño Batholith dams.

Another important question to be addressed when considering compaction criteria is whether the fill should be compacted dry or wet of the optimum water content. The variability of the compaction water content can affect the permeability, strength, pore pressures and compressibility of fills material composed of tropical residual soils. Strength is higher dry of the optimum, because of lower pore pressures and greater intimate interparticle contact, promoted by lower dispersion of the material. However, permeability is greater dry of the optimum. Compressibility is lower and the rate of settlement is higher dry of the optimum, since pore pressures that need to be dissipated are smaller. Nevertheless, settlements for water contents that are wet of the optimum tend to produce lesser cracking, because of a more ductile behavior of the material. In light of this comparison, it can be noticed neither of the two conditions present the best engineering properties for all important aspects. With this in mind, the design has to asses what engineering properties are the most relevant for a specific dam site.

Liusheng reported that the main reason for faulty behavior in some laterite fill dams (Xiaoqiaohe-46.2 m, Luomatang 33 m, Fly ash retaining dam for Guiyang Power Plant-33 m) built in the southern provinces of China, is related to poor compaction. Deficient compaction on these dams resulted in the formation of sinkholes and piping. Liusheng recommends the following alternative compaction control criterion:

Size of dam	Fill	Degree of Compaction
Large and Medium	Homogeneous	0.95
	Central or sloping core	0.97
Small	Homogeneous	0.92
	Central or sloping core	0.95

Table 7.2- Compaction Control Criteria for Laterite Fills Recommended by Liusheng, Based on Experience in Chinese Dams (Liusheng, 1985)

7.3.1.1 Equipment, lift thickness and number of passes

A wide range of compaction equipment has resulted in acceptable compaction conditions in different projects. Therefore, compaction equipment specification for tropical residual soils cannot be restricted beforehand.

At Punchiná, roller compactors with pads (CAT 824 B, 825 B and 825 C) with variable roller weights were implemented in order to modify compaction energy in accordance with the compaction criterion presented in Figure 7.3. The specification required 8 passes of this equipment for 0.18 m layers.

For the Omai Gold Mine Tailings Dams, available equipment from the open pit mines was used to construct the embankments. The saprolite was excavated by a P&H 1550 excavator and hauled with Caterpillar 777B mine haul trucks. The borrow materials was placed in 0,5 m thick loose layers by D-9 bulldozers and compacted using the loaded haul trucks. A Caterpillar 1.5 m diameter vibratory peg and smooth rollers were later used to provide additional compaction effort. Between lifts, the surface was scarified with a CAT 16 G grader, fitted with a scarifier rake (Bedell et al, 2002). The use of the haul trucks precluded the placement of very wet material due to the increased difficulties for traversing the fill, but guaranteed adequate undrained shear strength of the placed materials. The shear strength required to support the loaded 777B haul trucks without excessive rutting was about 240 kPa (Bedell et al, 2002).

At Guri Dam trials at test embankments showed that the pad-foot rollers were most appropriate. Based on the same tests, its was decided to compact the residual soils in layers 0,2 m thick at the most with six passes of the roller to achieve 96% of the maximum dry density (Choudry et al, 1979).

Trial fills were implemented at Corumbá I dam (Brazil) to establish the most adequate compaction procedure for the saprolitic soil, derived from chlorite schist, used for core material. The soils were compacted with four different rollers: a smooth vibratory roller, two different tamping rollers and a pad-foot roller. A smooth drum vibratory roller 10 t static weight (CAT 25) roller provided the best results for the silty soils. After 12 passes of this equipment the compaction degree was 98.5% with a water content between 3% and 5% above optimum moisture of Standard Proctor. One of the tamping rollers also presented acceptable results, but all others showed poor compaction degrees. Residual soils for the core material were routinely compacted in 0.15 m layers (Mori et al, 1996).

The limonite, a low plasticity sandy clayey silt (ML-MH) residual soil, used for the embankments at the Goro Nickel Project (New Caledonia), was compacted with a moisture content between 40 and 50% using a fully loaded six-wheel drive CAT 740 truck, having a gross mass in excess of 60 t. After the truck compaction, the surface was readily sealed by a 10t smooth roller. However, compaction of the material could not be achieved using the drum roller alone, as the drum picked up material and the wheels lost traction (Smith, 2002).

Terzaghi reported difficulties encountered in compacting the andosols (yellowish brown clay) of Tipanundjang Dam (Indonesia) with steel rollers and he pointed out that at this dam no control was implemented over water content or density. Check outs were made excavating pits to ensure no cavities were formed in the soil mass.

At Piedras Blancas Dam in Colombia compaction with pad-foot roller could not achieve the specified density of the original design. As explained before, the criteria of using different compaction energy according to the water content was adopted and the fill was successfully compacted with light equipment.

7.3.2 INSTRUMENTATION

A comprehensive instrumentation program is fundamental to monitor the performance of dams involving residual soils, not only in the long term, but also during construction. Usually, possible failures of these types of structures are preceded by warning signs like pore pressure build-up, large deformations, cracking, increase or sudden disappearance of seepage and discontinuities in stresses.

As explained in the stability chapter, pore pressures should be carefully monitored during construction by means of vibrating wire piezometers. By implementing the Observational Method, stability analysis should be verified based on the results of piezometers. Pore pressures at the foundation level should also be carefully monitored. In this case open well piezometers (i.e. Casagrande piezometers) can be more adequate. Information from piezometers should always be analyzed together with reservoir level, precipitation and fill level, since these variables can have a significant effect on the measured values.

Settlements should be measured by settlement plates or cells, always calibrated with the least precise, but more reliable topographic control points. During construction, it is important to analyze the movements of the structure with the height of the fill to be able to properly understand behavior.

Seepage should be monitored by means of a calibrated wire at the foundation level. Not only flow rate should be measured, but water color should be inspected, in order to verify that no material from the dam is being washed. Again, results have to be evaluated together with reservoir level and precipitation.

The use of pressure cells is widely debated among the dam engineers community. Some favored it, some don't. Pressure cells are intended to measure the state of stress at a particular point of the dam. They also allow determining the actual earth pressure coefficient in the dam (Ko). They have also been used to estimate the range of resistance of the material to cyclic stresses under actual field conditions. If pressure cells are properly installed and protected they can provide information for performance evaluation that is worth measuring.

Good quality instruments and proper installation are fundamental for a good monitoring program, but there results are meaningless if they are not evaluated in a timely fashion by an experienced engineer with sound engineering judgment that can make decisions and take actions based on the measured values.

In the case of Guri dam, a total of 1800 measuring devices, including pneumatic, vibrating wire and Casagrande Piezometers, inclinometers, settlement plates, double fluid settlement devices, hydraulic gages and others were installed to verify the design hypothesis, as well as to monitor its performance during and after construction, design stage reservoir impounding and operation.

8 PERFORMANCE

8.1.1 STABILITY PROBLEMS

As mentioned above, short dry seasons common in tropical areas, demand accelerated fill placement programs. This can lead to high pore pressure generation during construction, often making the construction stage the most critical stability condition.

Careful cofferdam stability monitoring has been important for securing the behavior of the larger dam structure. Trial berms have also been used for this purpose. Initial stability design analyses had to be verified with actual pore pressures generated in the fill, by evaluating the true pore pressure distribution with piezometers. Also, shear strength of fill material should be confirmed on samples taken from compacted material either in a trial berm. Figure 8.1 shows the pore pressures generated after construction at Troneras Dam (Villegas, 1984)

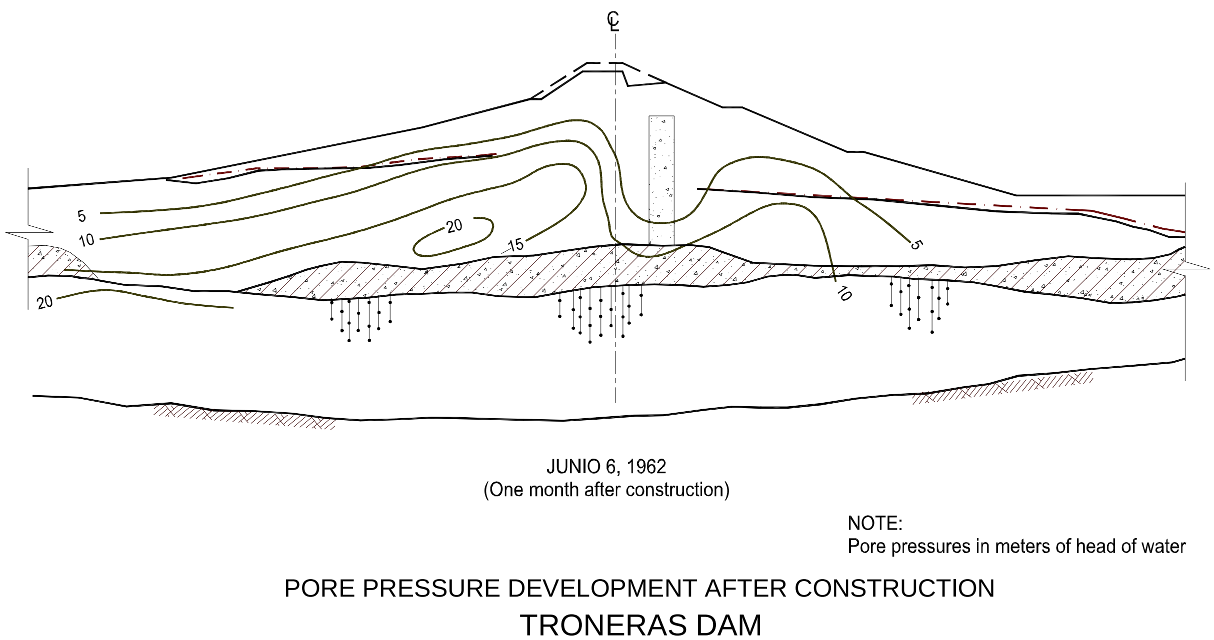

Figure 8.1- Pore Pressures Developed at Troneras Dam One Month after Construction.

At Punchiná Dam a delay in the deviation works and foundation preparation and the proximity of the rainy season, obliged the Contractor to increase the rate of placement of fill material for the cofferdam from 7500 to 12000 m^3/d. This situation caused an increase in pore pressures that induced an incipient failure of the cofferdam. In light of these circumstances, pore pressures were monitored closely and construction procedures were modified to guarantee the stability of the structure. The experience in the construction of the cofferdam and the corrective measures taken to guarantee its stability were fundamental for improving the security of the larger structure.

Figure 8.2- General Overview of Punchiná Dam

Other measures taken in the Antioqueño Batholith dams to improve stability conditions include:

- Installation of horizontal drains in the downstream face, both for the cofferdam and main dam, to control pore pressures during construction (See Case Histories).

- Utilization of the more sandy soils of the saprolitic horizon for the downstream shell to improve its stability during construction and rapid drawdown.

- Provision of buttresses of more competent material compared with the main fill residual material to increase stability and reduce fill volume.

- Thick chimney filters composed of well graded material to dissipate pore pressures and control seepage through the fill.

Seismic design for dams built with tropical residual soils demand particular considerations. Among them, the following should be considered: conservative free board to compensate for seismically induced motions (for Punchiná Dam 12 m were contemplated), provision for upstream buttresses to improve stability and avoid liquefaction, wide crest to improve stability, thick inclined filters to rapidly seal any cracks that may form in the dam body after an earthquake, horizontal drains in the downstream shell to reduce seismically induced excess pore pressures and generous drains and collectors to collect any seepage through cracks.

Stability problems of residual soils should also be evaluated in the reservoir's influence area. These types of soils are particularly susceptible to rapid drawdowns. In the Punchiná Dam (San Carlos Hydroelectric Project) in Colombia, this matter has been a major concern during its operation. Due to electricity generation requirements the reservoir's level oscillates permanently (up to 16 m in three days). This condition has strongly affected the reservoir's slopes stability (INGETEC S.A., 2002). Most of the reported slides have been translational, relatively shallow and constituted by mature residual soils. A finite element model and static slope stability analysis confirmed the following behavioral geotechnical model (INGETEC S.A., 2002):

The lowering of the reservoir's level eliminates the confining effect of water that contributes to stability, but due to the relatively rapid rate of drawdown pore pressures develop in the more impervious materials. Saprolites (level IC) tend to dissipate pore pressures rapidly due to their relatively higher permeability generated by relic structures. On the other hand, the superficial mature residual soils, due to their lower permeability exhibit an undrained behavior with a buildup of excess pore pressures. The pore pressures that dissipate in the IC level encounter the relatively impermeable IB level and build up pressure behind the "crust" of this material. If the magnitude of this pressure is high enough instabilities occur. For moderate slopes, the slides are limited to level IB of Deere and Patton's weathering profile, but for steeper ones displacements involve level IC. The initial instability problem turns into an erosion problem, when the saprolitic materials, with little cohesion are exposed to the action of the reservoir. This in time can promote larger instabilities.

Figure 8.3- Reservoir's Slope Instabilities Experienced at Punchiná Dam Due to Rapid Drawdown (INGETEC S.A., 2002)

These types of instabilities can reduce the lifespan of a project by reducing the useful volume of the reservoir. With the aforementioned problems in mind, designers and operators ought to evaluate the maximum reservoir fluctuations justified economically, having in mind that in a long term they can shorten they life of the project.

8.1.2 SEEPAGE

It is highly recommended to incorporate extensive drainage systems for the fill and foundations to handle seepage that may generate a failure of the structure. As mentioned before, these drains also allow for pore pressure dissipation. This design philosophy was widely applied at the Antioqueño Batholith dams with good results. At Tipanundjang Dam (Indonesia), great emphasis was placed on installing adequate drains in the dam body to ensure the seepage line remained a satisfactory distance below the downstream slope (Wesley, 1974).

8.1.3 LONG TERM BEHAVIOR

Dams involving tropical residual soils (either at their foundation or fill material) are particularly vulnerable in the long term. Filter or drainage plugging can have serious consequences in the behavior, due to pore pressure build up or piping. Therefore, a well-planned monitoring program is fundamental for this type of structures. Not only an increase in seepage is alarming, but also a halt, that might indicate plugging of the drainage system. Seeping water has to be inspected, to verify no fines are being carried or suspended in the flow. The aforementioned phenomena can develop rapidly and have catastrophic consequences on the structure. For that reason, a course of action has to be defined in advanced, anticipating these problems. A decision matrix based on the monitoring results and their implication in the dam security is recommended. Predefined warning values for instrumentation should be implemented (green, yellow and red) to alert operators of possible problems.

9 CONCLUSIONS AND RECOMMENDATIONS

The following are the main conclusions and recommendations that dam designers should consider when dealing with dams with residual soils as their foundation of fill material.

- Residual soils are characterized by their heterogeneity, both vertically and horizontally. This condition forces designers to be conservative in the estimation of their engineering properties.

- Among the problems to be faced when dealing with soft “undisturbed” residual soils is the sampling procedure. In saprolitic soils, block samples have shown a lower strength than core samples, indicating that common procedures of core sampling and/or sample extracting affect the soil structure. Because there is a clear influence of drying on soil properties, the classification, specific gravity and compaction tests should be performed with samples in its natural condition or air dried. Oven dried samples may affect the results

- Due to their particular behavior, promoted by their mineralogy, careful engineering judgment should be applied to extrapolate knowledge developed from transported soils.

- The engineering design of dams to be founded on residual soils or where residual soils will be used as fill materials, should give particular consideration to preclude the possibility of piping. Extensive drains and filters have proven to be effective do deal with this potentially catastrophic event.

- During construction, pore pressure monitoring should be implemented. The construction stage can be the most critical stage for the stability of the fills composed of residual soils.

- The design of dams involving residual soils should be characterized by its flexibility. Foundation excavation volumes are difficult to estimate accurately and their engineering properties present large scatter. Therefore designers should aim for adaptable specifications, both technically and contractually, to cope with the sometimes unexpected circumstances.

- Several dams presenting residual soils in their foundation or body have presented problems in their long term behavior. Thus, a thorough monitoring system should be implemented to review their performance. Predefined course of actions should be considered in case the dam instrumentation indicates abnormal behavior.

10 CASE HISTORIES

In order to illustrate design concepts and characteristics of dams involving tropical residual soils as dam foundation or fill materials case histories were prepared from available literature. Technical aspects such as type of structure, foundation materials and treatment, instrumentation, construction and operational issues are briefly discussed given information is available. Also, references are provided to allow the reader to further research a particular dam.

The following case histories are included in Appendix A:

Dam Name	**Location (Country)**
Tucurui	Brazil
Passauna	Brazil
Balbina	Brazil
Ilha Solteira	Brazil
Piedras Blancas	Colombia
Quebradona	Colombia
Troneras	Colombia
Miraflores	Colombia
Santa Rita I	Colombia
Santa Rita I	Colombia
Santa Rita II	Colombia
Punchiná	Colombia
San Lorenzo	Colombia

Dam Name	Location (Country)
Salvajina	Colombia
Guri	Venezuela
Ambuklao	Phillipines
Binga	Phillipines
Gangapur	India
Gyobyu	Burma
Samson Brook	Australia
Sasumua	Kenya

Table 10.1- Dam case histories involving residual soils as dam or foundation materials

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Classification of Residual Soils (after Wesley,1997)

Grouping system		Common pedological names used for group	Descriptive information on in situ state	
Major Division	Sub-group		Parent Rock	Information on structure
Group A (Soils without a strong mineralogical influence)	(a) Strong macro-structure influence	Give names if appropriate	Give details of type of rock from which the soil has been derived.	Describe nature of structure: -stratification, reflecting parent rock -fractures, fissures, faults, etc. -presence of partially weathered rock (state % and physical form, e.g. 50% corestones)
	(b) Strong micro-structure influence	Give names if appropriate	.	Describe nature of microstructure or evidence of it: -effect of remolding, sensitivity -liquidity index or similar index
	(c) Little or no structure influence	Give names if appropriate	.	Describe evidence for little or no structural influence
Group B (Soils strongly influenced by commonly occurring materials)	(a) Montmorillonite (Smectite group)	Black cotton soils, Black clays, Tropical black earths, Grumusols, Vertisols	.	Describe any structural effects, which may be present, or other aspects relevant to engineering properties. Evidence of swelling behavior, extent of surface cracking in dry weather, slickensides below surface etc.
	(b) Other minerals	.	.	.
Group C (Soils strongly influenced by clay minerals essentially found only in residual soils)	(a) Allophane sub group	Volcanic ash soils, Andosols, Andepts	.	Give basis for inclusion in this group. Describe any structural influences, either macro-structure or micro-structure.
	(b) Halloysite sub group	Tropical red clays, Latosols, Oxisols, Ferralsols	.	
	(c) Sesquioxide sub group (gibbsite, geo-thite,haematite)	Lateritic soils, Laterites, Ferralitic soils, Duricrust	.	Give basis for inclusion in this group. Describe structural influences - Especially cementation effects of the sesquioxides.

Characteristics of residual soils groups (after Wesley,1997)

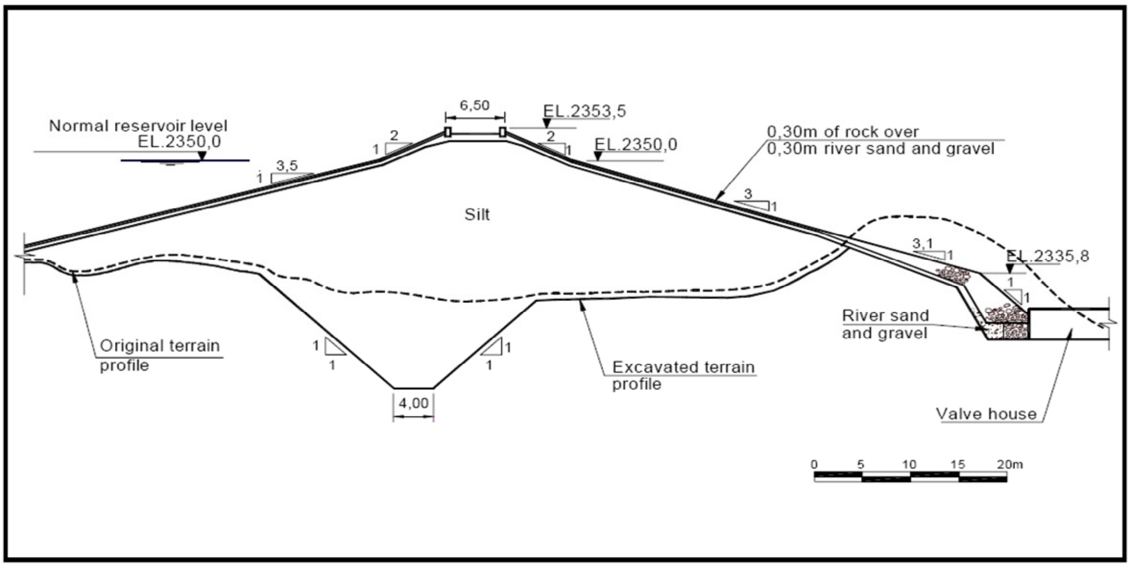
Group		**Examples**	**Means of identification**	**Comments on likely engineering properties and behavior**
Major Group	**Sub-group**			
Group A (Soils without a strong mineralogical influence)	(a) Strong macro- structure influence	High weathered rocks from acidic or intermediate igneous rocks, and sedimentary rocks.	Visual inspection	This is a very large group of soils (including the 'saprolites') where behaviors (especially in slopes) is dominated by the influence of discontinuities, fissures , etc.
	(b) Strong micro- structure influence	Completely weathered rocks formed from igneous and sedimentary rocks.	Visual inspection, and evaluation of sensitivity, liquidity index, etc.	These soils are essentially homogeneous and form a tidy group much more amenable to systematic evaluation and analysis than group (a) above. Identification of nature and role of bonding (from relict primary bonds to weak secondary bonds) important to understanding behavior.
	(c) Little structural influence	Soils formed from very homogeneous rocks	Little or no sensitivity, uniform appearance	This is a relatively minor sub-group. Likely to behave similarly to moderately overconsolidated soils.
Group B (Soils strongly influenced by commonly occurring minerals)	(a) Smectite (Montmorillonite group)	Black cotton soils, Many soils formed in tropical areas in poorly drained conditions	Dark color (gray to black) and high plasticity suggest soils of this group	These are normally problem soils found in flat or low lying areas, of low strength, high compressibility, and high swelling and shrinkage characteristics.
	(b) Other minerals			This is likely to be a minor sub-group.

Group C (Soils strongly influenced by clay minerals essentially found only in residual soils)	(a) Allophane group	Soils weathered from volcanic ash in the wet tropics and in temperate climates.	Very high natural water contents, and irreversible changes on drying.	These are characterized by very high natural water contents, and high liquid and plastic limits. Engineering properties are generally good, though in some cases high sensitivity could make handling and compaction difficult.
	(b) Halloysite group	Soils largely derived from older volcanic rocks; especially tropical red clays	Reddish color, well-drained topography and volcanic parent rock are useful indicators.	These are generally very fine-grained soils, of low to medium plasticity, but low activity. Engineering properties are generally good. (Note that there is often some overlap between allophane and halloysitic soils).
	(c) Sesquioxide group	This soils group loosely re-ferred to as 'lateritic', or laterite	Granular, or nodular appearence	This is a very wide group, ranging from silty clay to coarse sand and gravel. Behavior may range from low plasticity to non- plastic gravel.

APPENDIX A

CASE STUDY No. 1 Piedras Blancas

Section



General Data

Location	Antioquia, Colombia.
Construction Period	1947 - 1952.
Purpose	River regulation and hydroelectric power generation, water supply.

Technical Aspects

Type of structure	25 m high earth dam. Embankment materials: residual soils with w_L=47%, PI=12%, 80 % passing #200 sieve.
Geology	"Antioqueño Batholith": Lower Cretaceous intrusion with an approximate extension of 8000 km^2 The predominant rock varies from granodiorite to quartz diorite, fine to coarse grained, constituted by plagioclase, quartz biolite and hornblende.
Foundation materials	Low permeability residual soils. Average thickness: 10 – 20 m of low compressibility silty sands (ML). Brownish red color, low to medium plasticity (w_L= 40 - 50%, PI= 7 - 12%), medium to high compressibility, high water content (25 - 40%), low unit weight (1.5 - 1.6 g/cm^3) and internal friction angle 30° in natural state.
Foundation treatment	Filter at the toe of the downstream slope. Cut-off trench.
Construction issues	The desired compaction level could not be achieved using sheepsfoot rollers as initially specified. The designer had to make adjustments to the dam slopes taking into account the observed water content of the soils during construction. The desired soil compaction was successfully achieved using lightweight compaction equipment.
Operational issues	During operation (1961) a higher than expected water table was observed. The higher water table was caused by fines washed to the main drain. In 1967 seepage was observed at the embankment-left abutment contact. Apparently, the seepage came from the upstream borrow zone. The remedial works (impermeable mantles) performed at the dam-natural terrain contact did not stop the flow.

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Villegas, Fabio. "Experiences in Earth Dam Construction in Antioquia". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.
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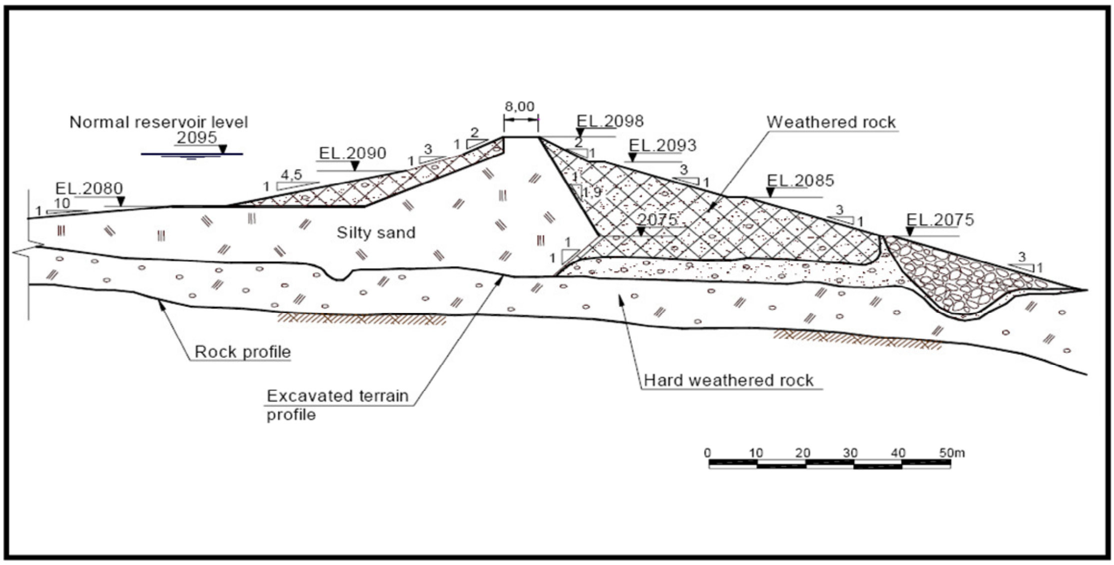
CASE STUDY No. 2 Quebradona

Scheme



General Data

Location	Antioquia, Colombia
Construction Period	1956 - 1958
Purpose	River regulation and hydroelectric power generation.

Technical Aspects

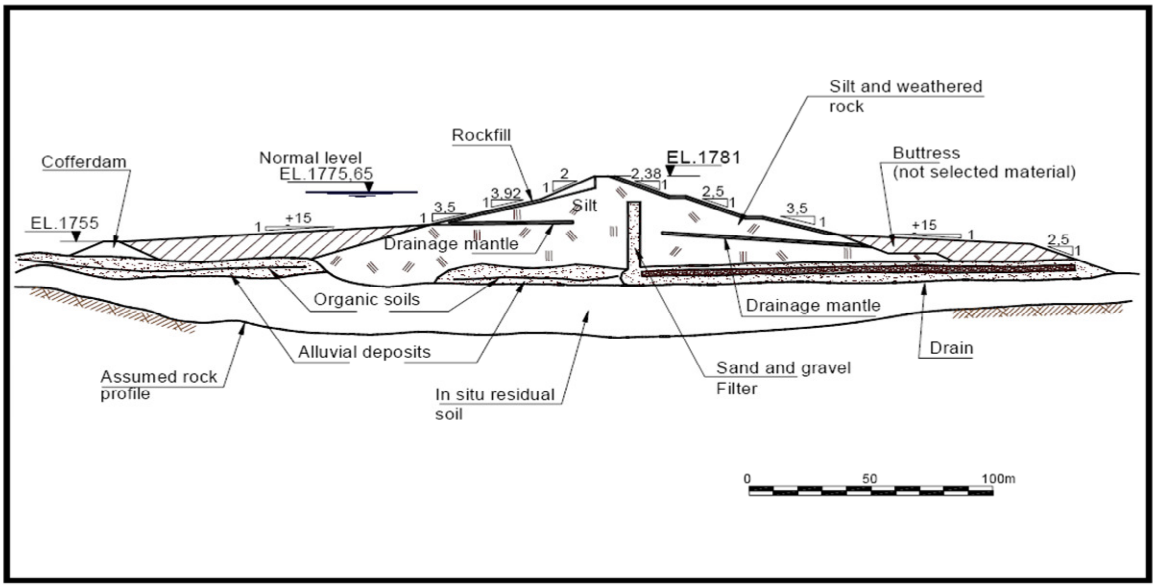
Type of structure	34 m high earth dam with a fill volume of 331.000 m^3. Embankment materials: decomposed rock to non-plastic sand soil with water content between 15 and 28% and natural unit weight of 1.7 g/cm^3. Silts with an average PI of 6%, natural water contents between 27 to 35% and average natural unit weight of 1.53 g/cm^3. Embankment material average properties: LL=35%, PI=4%, 30% passing #200 sieve, wnat=26%, wopt=23%.
Geology	"Antioqueño Batholith": Lower Cretaceous intrusion with an approximate extension of 8000 km2. The predominant rock varies from granodiorite to quartz diorite, fine to coarse grained, constituted by plagioclase, quartz biolite and hornblende.
Foundation materials	Low permeability residual soils. Average thickness: 10 – 20 m of low compressibility silty sands (ML). Brownish red color, low to medium plasticity (w_L= 40 - 50%, PI= 7 - 12%), medium to high compressibility, high water content (25 - 40%), low unit weight (1.5 - 1.6 g/cm^3) and internal friction angle 30° in natural state.
Foundation treatment	Horizontal filter at downstream fundation contact.
Instrumentation	Installation of piezometers and settlement measuring devices.
Construction issues	Designs were based on Piedras Blancas Dam experience (Case Study No. 1). More gentle slopes were designed to avoid failures during construction stages due to excessive pore pressures. However due to the type of material used, the pore pressures generated during construction dissipated faster than predicted causing a rapid consolidation process. It is estimated that about 5% of the total settlement occurred during construction.
Operational issues	The performance of the dam has been satisfactory.

References

Villegas, Fabio. "Experiences in Earth Dam Construction in Antioquia". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.
Villegas, Fabio. "Dynamic Stability Analyses of some Earth Dams in Residual Soils". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.

CASE STUDY No. 3 Troneras

Section



General Data

Location	Antioquia, Colombia
Construction Period	1959 - 1962
Purpose	River regulation and hydroelectric power generation.

Technical Aspects

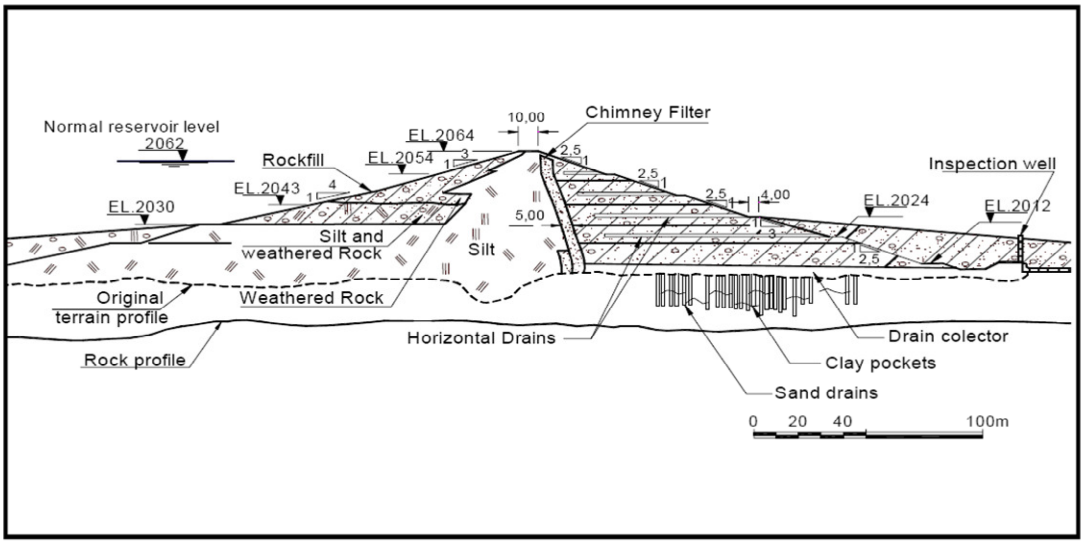
Type of structure	37 m high earth dam with a fill volume of 1.129.000 m3. Embankment material average properties: LL=43%, PI=9%, 79% passing #200 sieve, wnat=30%, wopt=25.5%.w_{opt}=25.5%.
Geology	"Antioqueño Batholith": Lower Cretaceous intrusion with an approximate extension of 8000 km2. The predominant rock varies from granodiorite to quartz diorite, fine to coarse grained, constituted by plagioclase, quartz biolite and hornblende.
Foundation materials	Low permeability residual soils. Average thickness: 10 – 20 m of low compressibility silty sands (ML). Brownish red color, low to medium plasticity (w_L= 40 - 50%, PI= 7 - 12%), medium to high compressibility, high water content (25 - 40%), low unit weight (1.5 - 1.6 g/cm^3) and internal friction angle 30° in natural state.
Foundation treatment	Excavation of organic materials. Installation of horizontal sand and gravel filter at downstram foundation contact.
Construction issues	Counterweights were constructed at the toe of upstream and downstream slopes. A chimney filter was constructed to control seepage. The filters and the upstream and downstream counter forts were crucial for the stability during construction due to high pore pressures generated. During construction the foundation excavations were reduced and the lateral slopes and crest length were modified. Due to delays the embankment was constructed in only three months (dry season) generating high pore pressures that caused excessive movements and the incipient failure of the downstream slope.
Operational issues	The performance of the dam has been satisfactory.

References

Villegas, Fabio. "Experiences in Earth Dam Construction in Antioquia". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.
Villegas, Fabio. "Dynamic Stability Analyses of some Earth Dams in Residual Soils". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.

CASE STUDY No. 4 Miraflores

Scheme



General Data

- **Location** — Antioquia, Colombia.
- **Construction Period** — 1962 - 1965.
- **Purpose** — River regulation and hydroelectric power generation.

Technical Aspects

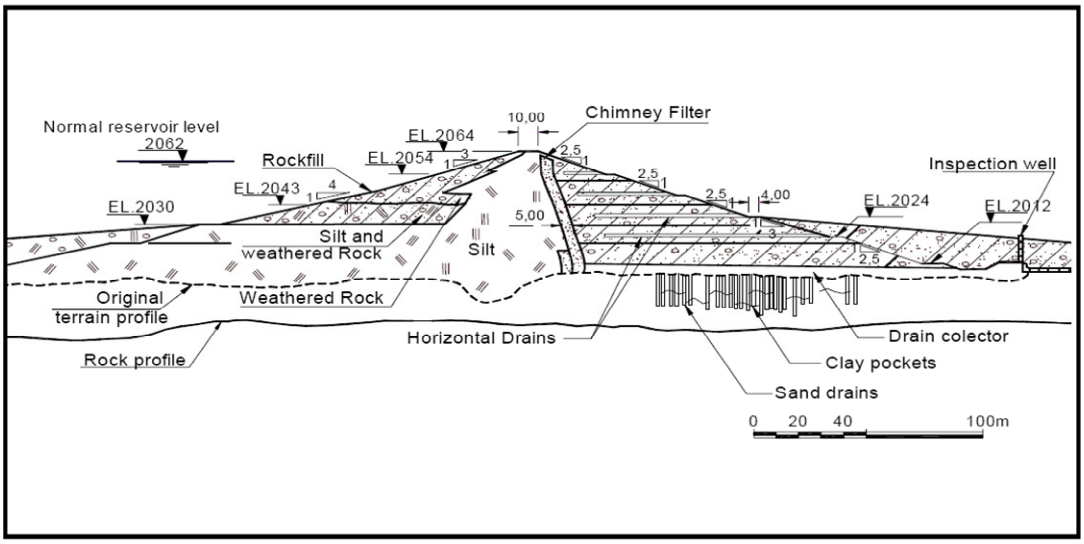
- **Type of structure** — 63 m high earth dam with a fill volume of1.058.000 m^3. Embankment materials with w_L =37% PI=6%, 46% passing #200 sieve, w_{nat}=22%, w_{opt}=19.5%.
- **Geology** — "Antioqueño Batholith": Lower Cretaceous intrusion with an approximate extension of 8000 km2. The predominant rock varies from granodiorite to quartz diorite, fine to coarse grained, constituted by plagioclase, quartz biolite and hornblende
- **Foundation materials** — Low permeability residual soils. Average thickness: 10 – 20 m of low compressibility silty sands (ML). Brownish red color, low to medium plasticity (w_L= 40 - 50%, PI= 7 - 12%), medium to high compressibility, high water content (25 - 40%), low unit weight (1.5 - 1.6 g/cm^3) and internal friction angle 30° in natural state.
- **Foundation treatment** — Horizontal filter at downstream foundation contact. Sand drains to accelerate the consolidation of the foundation.
- **Construction issues** — Higher pore pressures for design were assumed given the recent experience at Troneras Dam (Case Study No. 3). Very gentle slopes were constructed using upstream and downstream counter weight fills. As in Troneras the embankment was constructed during the three month dry season. The pore pressures exceeded the registered at Troneras and caused excessive movements and stability problems. The situation became critical and the construction had to be suspended. To remediate the situation a substantially larger than expected downstream counterweight had to be constructed. On the upstream side a mix of decomposed rock and rock fragments was adopted for the fill material.
- **Operational issues** — Subterranean flow from the abutments maintains the downstream counterweight saturated. Other than that the performance of the dam has been satisfactory.

References

Villegas, Fabio. "Experiences in Earth Dam Construction in Antioquia". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.
Villegas, Fabio. "Dynamic Stability Analyses of some Earth Dams in Residual Soils". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.

CASE STUDY No. 4 Miraflores

Scheme



General Data

- **Location** Antioquia, Colombia.
- **Construction Period** 1962 - 1965.
- **Purpose** River regulation and hydroelectric power generation.

Technical Aspects

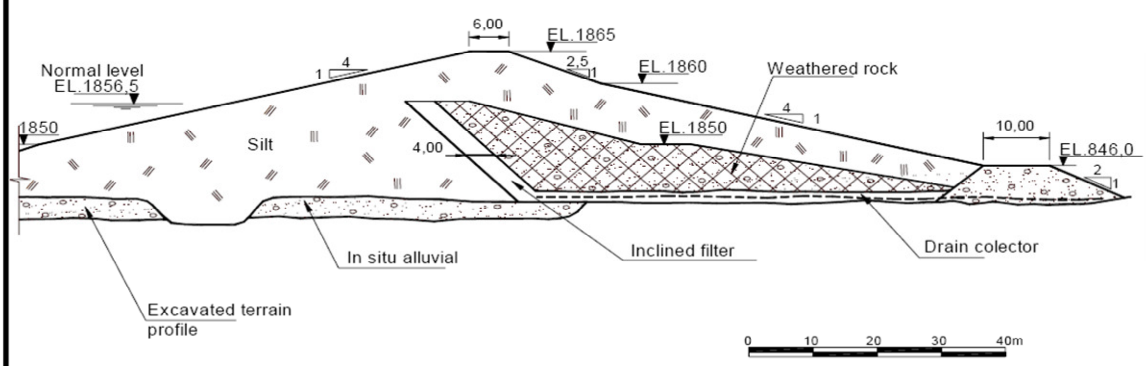
- **Type of structure** 63 m high earth dam with a fill volume of1.058.000 m^3. Embankment materials with w_L =37% PI=6%, 46% passing #200 sieve, w_{nat}=22%, w_{opt}=19.5%.
- **Geology** "Antioqueño Batholith": Lower Cretaceous intrusion with an approximate extension of 8000 km2. The predominant rock varies from granodiorite to quartz diorite, fine to coarse grained, constituted by plagioclase, quartz biolite and hornblende
- **Foundation materials** Low permeability residual soils. Average thickness: 10 – 20 m of low compressibility silty sands (ML). Brownish red color, low to medium plasticity (w_L= 40 - 50%, PI= 7 - 12%), medium to high compressibility, high water content (25 - 40%), low unit weight (1.5 - 1.6 g/cm^3) and internal friction angle 30° in natural state.
- **Foundation treatment** Horizontal filter at downstream foundation contact. Sand drains to accelerate the consolidation of the foundation.
- **Construction issues** Higher pore pressures for design were assumed given the recent experience at Troneras Dam (Case Study No. 3). Very gentle slopes were constructed using upstream and downstream counter weight fills. As in Troneras the embankment was constructed during the three month dry season. The pore pressures exceeded the registered at Troneras and caused excessive movements and stability problems. The situation became critical and the construction had to be suspended. To remediate the situation a substantially larger than expected downstream counterweight had to be constructed. On the upstream side a mix of decomposed rock and rock fragments was adopted for the fill material.
- **Operational issues** Subterranean flow from the abutments maintains the downstream counterweight saturated. Other than that the performance of the dam has been satisfactory.

References

Villegas, Fabio. "Experiences in Earth Dam Construction in Antioquia". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.
Villegas, Fabio. "Dynamic Stability Analyses of some Earth Dams in Residual Soils". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.

CASE STUDY No. 5   Santa Rita I

Section



General Data

- **Location** Antioquia, Colombia.
- **Construction Period** 1966 - 1969.
- **Purpose** River regulation and hydroelectric power generation.

Technical Aspects

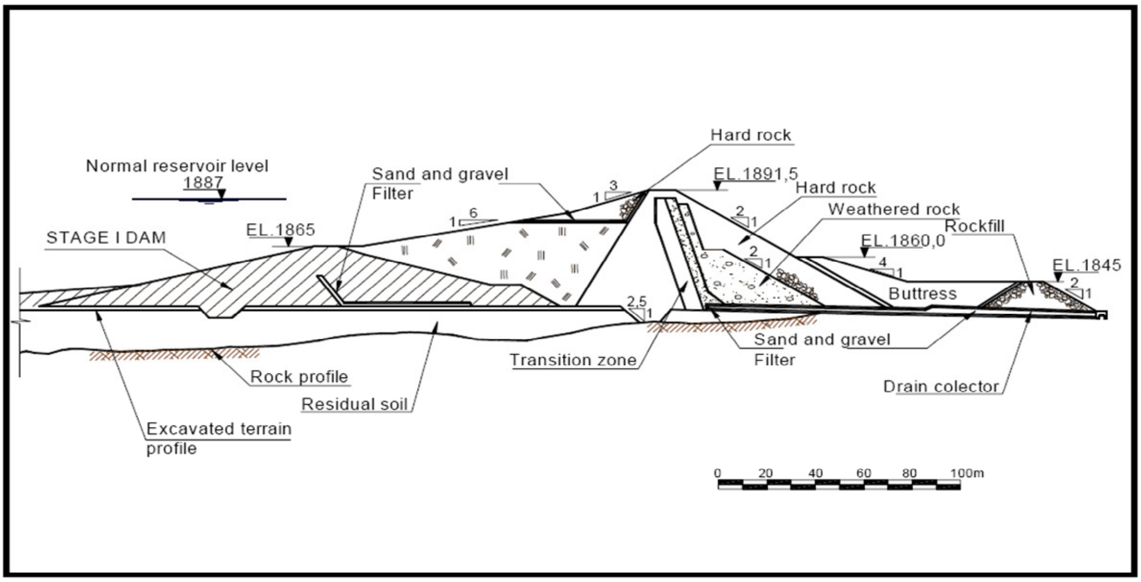
Type of structure	30 m high earth dam with a fill volume of 943.000 m^3. Embankment material average properties: LL=38%, PI= 6%, 54% passing #200 sieve, wnat=21.5%, wopt=16.5%.
Geology	"Antioqueño Batholith": Lower Cretaceous intrusion with an approximate extension of 8000 km2. The predominant rock varies from granodiorite to quartz diorite, fine to coarse grained, constituted by plagioclase, quartz biolite and hornblende.
Foundation materials	Low permeability residual soils. Average thickness between 10 and 20 m. Low compressibility silty sands (ML). Brownish red color, low to medium plasticity (LL= 40 - 50%, PI = 7 - 12%), medium to high compressibility, high water content (25 - 40%), low unit weight (1.5 - 1.6 g/cm3) and internal friction angle of about 30° in natural state
Foundation treatment	Horizontal filter at downstream foundation contact.
Construction issues	Given the high rainfall and short period of time to construct the embankment (short dry season) the foundation excavation was reduced. The reduction was based on a detailed exploration during the early construction stages. A wedge of weathered rock mixed with alluvial gravel was included during construction to allow for a steeper downstream slope and to accelerate the construction. Upstream, selected silt and a counterweight were used to increase the slope stability.
Operational issues	Upstream toe of Santa Rita II

References

Villegas, Fabio. "Experiences in Earth Dam Construction in Antioquia". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.
Villegas, Fabio. "Dynamic Stability Analyses of some Earth Dams in Residual Soils". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.

CASE STUDY No. 6 Santa Rita II

Section



General Data

Location	Antioquia, Colombia.
Construction Period	1972 - 1976.
Purpose	River regulation and hydroelectric power generation.

Technical Aspects

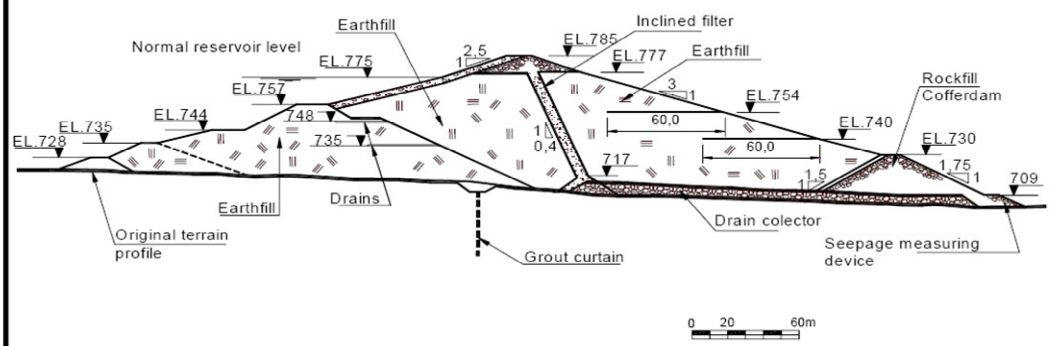
Type of structure	60 m high dam with a fill volume of 3.836.000 m^3. Embankment material average properties: LL=38%, PI=8%, 56% passing #200 sieve, wnat=26%, wopt=20.5%.
Geology	"Antioqueño Batholith": Lower Cretaceous intrusion with an approximate extension of 8000 km2. The predominant rock varies from granodiorite to quartz diorite, fine to coarse grained, constituted by plagioclase, quartz biolite and hornblende.
Foundation materials	Low permeability residual soils. Average thickness between 10 and 20 m. Low compressibility silty sands (ML). Brownish red color, low to medium plasticity (LL= 40 - 50%, PI = 7 - 12%), medium to high compressibility, high water content (25 - 40%), low unit weight (1.5 - 1.6 g/cm 3) and internal friction angle of about 30° in natural state.
Foundation treatment	Horizontal filter at downstream foundation contact.
Construction issues	Flexible approach adopted by the contractor, owner and consultants to cope with the difficult conditions (high rainfall and short dry season) allowed the successful completion of the project. The local soils were used adjusting the compaction effort to the water contents. The work fronts were alternated constantly to avoid developing excessive pore pressures. The good maintenance of the borrow zones and roads were essential to restart construction fast after rain periods.
Operational issues	The performance of the dam has been satisfactory.

References

Villegas, Fabio. "Experiences in Earth Dam Construction in Antioquia". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.
Villegas, Fabio. "Dynamic Stability Analyses of some Earth Dams in Residual Soils". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.

CASE STUDY No. 7 Punchiná

Section



General Data

Location	Antioquia. Colombia
Construction Period	1978 - 1983.
Purpose	Regulation of the Guatape river and hydroelectric power generation at San Carlos Station.

Technical Aspects

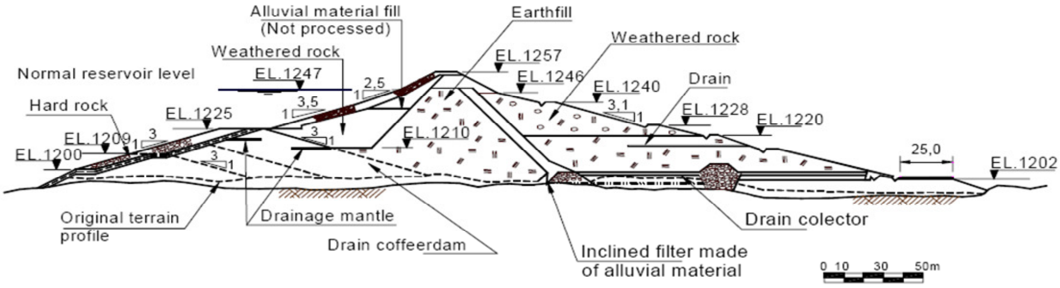
Type of structure	75 m high dam with a fill volume of 6 million m^3. Embankment material average properties: LL=45%, PI=14%, 42% passing #200 sieve, wnat=24%, wopt=20%.
Geology	"Antioqueño Batholith": Lower Cretaceous intrusion with an approximate extension of 8000 km^2 The predominant rock varies from granodiorite to quartz diorite, fine to coarse grained, constituted by plagioclase, quartz biolite and hornblende.
Foundation materials	Low permeability residual soils. Average thickness between 10 and 20 m. Low compressibility silty sands (ML). Brownish red color, low to medium plasticity (LL= 40 - 50%, PI = 7 - 12%), medium to high compressibility, high water content (25 - 40%), low unit weight (1.5 - 1.6 g/cm^3) and internal friction angle of about 30° in natural state.
Foundation treatment	Filter mantle and drainage collectors downstream of the dam axis to control foundation cracking and fine migration during an earthquake. Drainage galleries through the saprolite to capture seepage water and a system to measure seepage through the foundation were constructed. Grout curtains were constructed to reduce flow throuh the foundation.
Instrumentation	88 piezometers, 57 superficial control points, 63 control points along the crest, 1 inclinometer, 1 device to measure horizontal movement, 2 accelographs and 8 devices to measure water inflows.
Construction issues	from 3,5H:1V to 2H:1V to accelerate the construction progress. To compensate for the steeper slope, horizontal drains were installed and the pore pressures where closely monitored to ensure slope stability. Due to the proximity of the rainy season the rate of fill placement was increased causing a dangerous increment in the pore pressures and an incipient failure of the cofferdam. The downstream slope moved 1,5 m horizontally in 11 days. During construction it was decided to install a counterweight fill at the upstream slope toe to increase the stability under seismic loading.
Operational issues	The performance of the dam has been satisfactory.

References

Villegas, Fabio. "Experiences in Earth Dam Construction in Antioquia". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.
Villegas, Fabio. "Dynamic Stability Analyses of some Earth Dams in Residual Soils". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.

CASE STUDY No. 8   San Lorenzo

Section



General Data

Location	Antioquia, Colombia
Construction Period	1980-1984
Purpose	Diversion of Nare River to increase the Guatape River flow for hydroelectric power generation at San Carlos and Las Playas Stations.

Technical Aspects

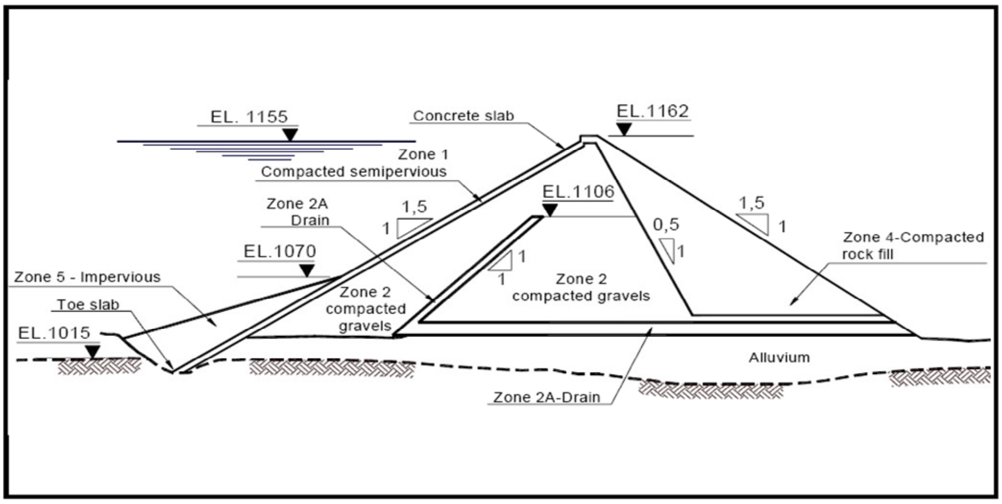
Type of structure	63 m high dam with a fill volume of 5.4 million m^3. Embankment material: silts and sprolites with some rock wedges.
Geology	"Antioqueño Batholith": Lower Cretaceous intrusion with an approximate extension of 8000 km^2. The predominant rock varies from granodiorite to quartz diorite, fine to coarse grained, constituted by plagioclase, quartz biolite and hornblende.
Foundation materials	Low permeability residual soils. Average thickness between 10 and 20 m. Low compressibility silty sands (ML). Brownish red color, low to medium plasticity (LL= 40 - 50%, PI = 7 - 12%), medium to high compressibility, high water content (25 - 40%), low unit weight (1.5 - 1.6 g/cm^3) and internal friction angle of about 30° in natural state. The foundation soils are compose by 60% sand, 27% silt and 13% clay
Foundation treatment	Excavation of unsuitable material (up to about 6 m of excavation at some locations). Filter mantle and drainage collectors downstream of the dam axis to control foundation cracking and fine migration during an earthquake.
Instrumentation	152 piezometers, 102 superficial control points, 58 control points along the crest, 3 inclinometer, 3 extensometers, 4 accelographs and 10 devices to measure water inflows.
Constructional issues	The following considerations were taken into account during design and construction to increase stability under seismic loading: gentle slopes, thick crest, very conservative freeboard, use of a non cohesive material zone in the higher section of the upstream slope to facilitate pore pressure dissipation after an earthquake, thick chimney filter to prevent fine migration if case of cracks.
Operational issues	The performance of the dam has been satisfactory.

References

Villegas, Fabio. "Experiences in Earth Dam Construction in Antioquia". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.
Villegas, Fabio. "Dynamic Stability Analyses of some Earth Dams in Residual Soils". III Colombian Geotechnical Seminar, Bogota, August 6-10, 1984.

CASE STUDY No. 9 Salvajina

Scheme



General Data

Location	Cauca, Colombia
Construction Period	1982 - 1984
Purpose	Hydroelectric power and river flood control

Technical Aspects

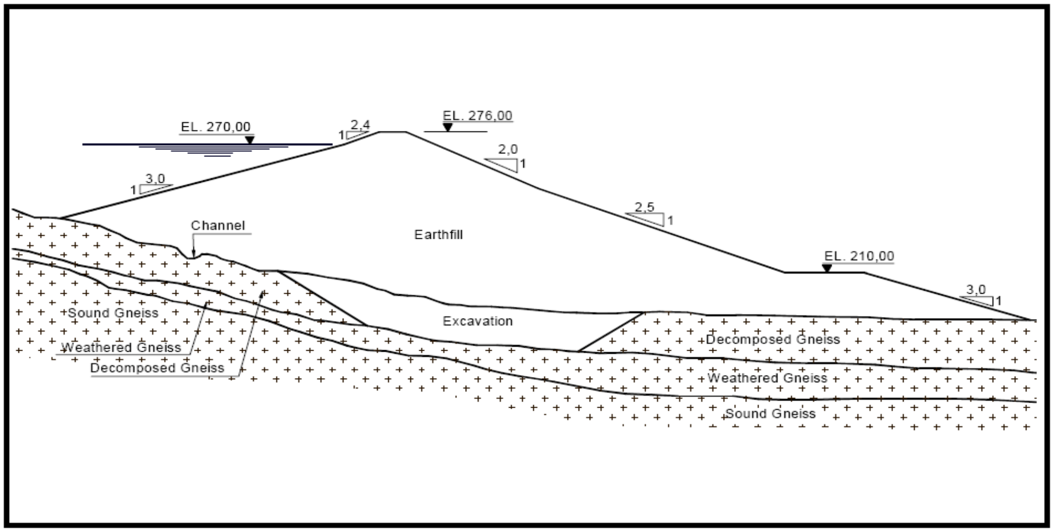
Type of structure	148 m high and 362 m length concrete face rockfill dam with a fill volume of 3.9 million m³.
Geological site conditions	Tertiary sedimentary rocks consisting mainly of siltstones and sandtones. Rather deep deposits of colluvion and residual soils partially covered the rock at the abutments and a tick alluvium, as deep as 30 m, filled the river channel.
Foundation materials	The rocks at the toe slab foundation consisted mainly of siltstones and sandstones. At the right abutment a dike of highly weathered porphyritic diorite was present. The rock weathering intensity varied being more intense toward the upper parts of the abutments where it was completely weathered to soil (residual soil).
Foundation treatment	Removal of any colluvium or residual soils present at the abutments upstream from the dam axis. The colluvium or residual soil downstream from the axis was left in place, since the deformations downstream from the axis have little effect on the concrete slab performance. Consolidation grouting was applied to the entire foundation of the toe slab except where the slab was on residual soils (low permeability). In the residual soil area a cut-off trench was built. A grout curtain was built along the toe slab foundation. A filter was installed where the foundation consisted of weathered rock to prevent migration of fined into the dam fill causing piping. Removal and backfilling with concrete of seams of highly weathered material.
Type of instrumentation	Extensive instrumentation program, consisting of: piezometers, pressure cells, pneumatic and hydraulic settlement devices, joint meters, strain meters and superficial control points. With these devices it was possible to monitor: vertical settlement within the embankment, surface movements along the crest, downstream slope and right abutment toe slab (founded on weathered rock), total pressures within the fill, pore pressures within the alluvium, perimeter joint movements and strains in concrete face.
Construction Issues	The expected excavation volume for the foundation almost doubled due to substantially poorer than anticipated conditions of the abutment foundation material. Because of the poorer conditions the width of the foundation toe slab had to be increased at those locations to reduce the gradient through the weathered rock and residual soils. The concrete face slab was constructed in stages as the fill advanced with no problems.
Operational issues	The performance of the dam has been satisfactory.

References

Hacelas J.E., Ramirez C. A., Regalado G., "Design Features of Salvajina Dam", Symposium on Concrete Face Rockfill Dams - ASCE, Detroit, October, 1985.
Hacelas J.E., Ramirez C. A., Regalado G., "Construction and Performance of Salvajina Dam", Symposium on Concrete Face Rockfill Dams - ASCE, Detroit, October, 1985.

CASE STUDY No. 10 Guri (final stage)

Scheme



General Date

Location	Venezuela
Construction period	1984 (completion year)
Purpose	Hydroelectric power.

Technical Aspects

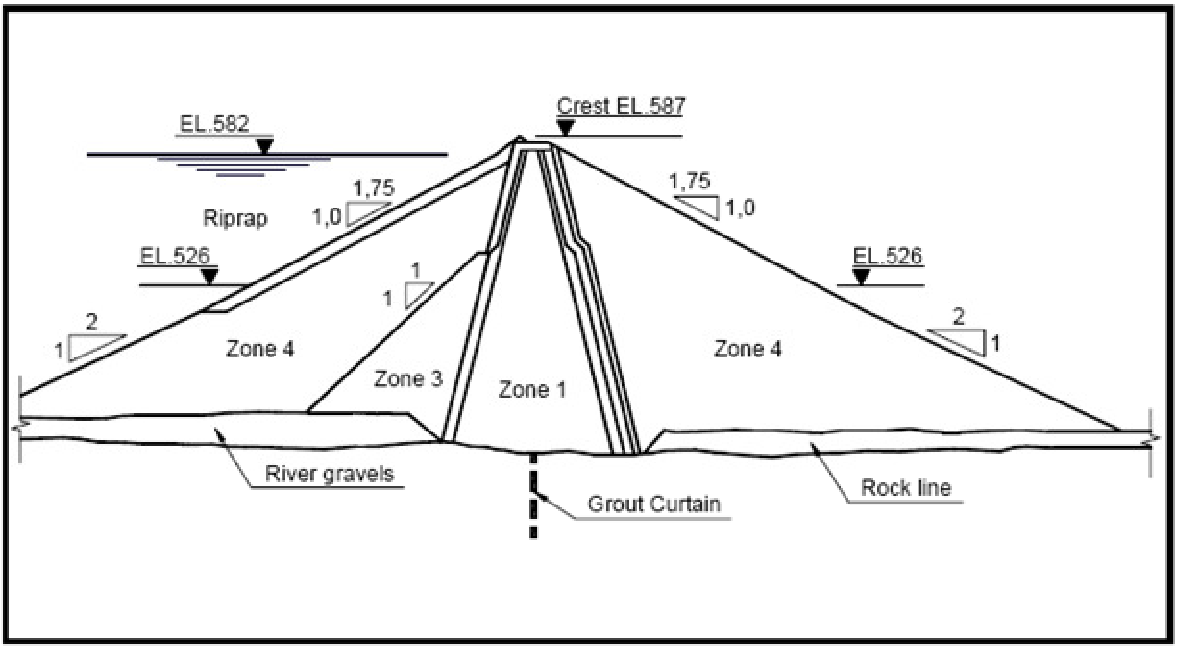
Type of structure	The final stage of the Guri project included the construction of 5500 m of earth-rock embankment dams with an earth-rock embankments 100m high and 2 km long on the left side and 4 km long on the right side with about 20 km long reservoir rim dikes for a total fill volume of about 72 million cubic meters. The embankment dam is divided into the left and right dams. Embankment materials: slope wash and residual soils. Filter and drain materials obtained by crushing the granite gneiss.
Geology	Precambrian rock consisting mainly of granitic gneiss interspersed with bands of ferruginous quartzite ranging in thickness from few meters to over 100 m. The dominant geologic structure is the Bolivar Fault System. Evidence in the field suggest that the faults have not experienced recent activity. There is a fault about 60 m wide and 100 m deep beneath a section of the left embankemt dam.
Foundation materials	The leaching out of soluble elements during weathering produced low density, porous (collapsible) residual soils derived from the decomposition of the granitic gneiss and quartzite and with thickness varying from few meters to over 70 m. The soils exhibit sudden collapse under load and saturation in the in-situ state. When remolded no significant sudden settlements occur when the soils are loaded or saturated.
Foundation treatment	Excavation of porous soils to top of hard weathered rock and replacement with compacted fill. Before the embankment was constructed irrigation channels were used to prewet the residual soils that were not excavated and that were above the water table. It was acknowledge that prewetting in some areas and excavating in others could cause cracking due to differential settlements of the foundation. Thick blanket filters (1,1 m) to prevent fine migration if cracking develops were installed. At the fault location, a rockfill dam with impervious central core was constructed. The material in the fault was excavated to a practical depth (about 30 m beneath the core). Upstream from the core the excavated fault was covered with an impervious blanket with two layers of filters on top of it. Downstream from the core two layers of filters were used (each up to 4 m thick beneath the core).
Instrumentation	Piezometers and inclinometers were installed in the foundation and fill.
Operational issues	The performance of the dam has been satisfactory

References

Prusza Z., Kleiner D.E., Sundaram A.V. "Characteristics of Guri Soils", Presented at ASCE Annual Meeting, Houston, Texas, 1983.
Medina J., Liu Bernard."The Influence of a Collapsible Foundation on the Design of Guri Embankment Dams", Fourteenth International Congress on Large Dams, Rio de Janeiro, Brazil, 3-7 May, 1982.
Medina J., Liu Bernard."Foundation Treatment for Control of Seepage at the Guri Embankment Dams", Fifteenth International Congress on Large Dams, Lausanne, Suisse, 24-28 June, 1985

CASE STUDY No. 11 Ambuklao

Scheme



General Data

Location	Agno River, Island of Luzon, Republic of the Phillipines.
Construction Period	1952 - 1956.
Purpose	Hydroelectric power

Technical Aspects

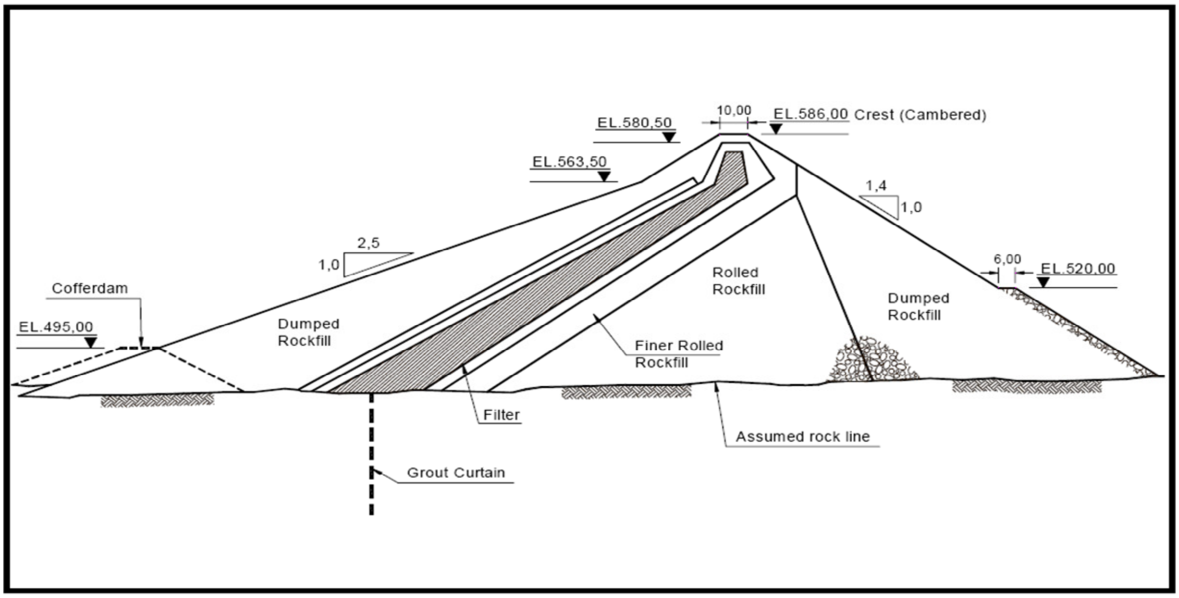
Type of structure	130 m high rockfill dam with 450 m long crest. Dam materials: Zone 1 - Impermeable core compacted in 18 inch layers with rubber tired rollers; well graded sand, silt and clay. Zone 3 - Sand and gravel, maximum size about 2 inches. Zone 4 - Coarser blasted rockfill, sand and gravel size, maximum size about 6 inches.
Foundation materials	Igneous, metamorphosed igneous and sedimentary bedrock. The formation has been subjected to much folding and faulting. The overburden consists of deeply weathered residual soils especially on the left abutment.
Foundation treatment	An impervious blanket was placed on the upstream face of the left abutment. The blanket was continuous along the core of the dam. The foundation of the core was excavated to sound rock. A grout curtain was placed below the core of the dam.
Operational issues	Total vertical settlement has been less than 6 inches and downstream movement has been about 3 inches.

References

Fernandez, Gabriel. Earth Dams Course (CEE 481) Class Notes. University of Illinois at Urbana-Champaing.

CASE STUDY No. 12 Binga

Scheme



General Data

Location	Agno River, Luzon, Republic of the Phillipines.
Construction Period	1959 (year of completion).
Purpose	Hydroelectric power.

Technical Aspects

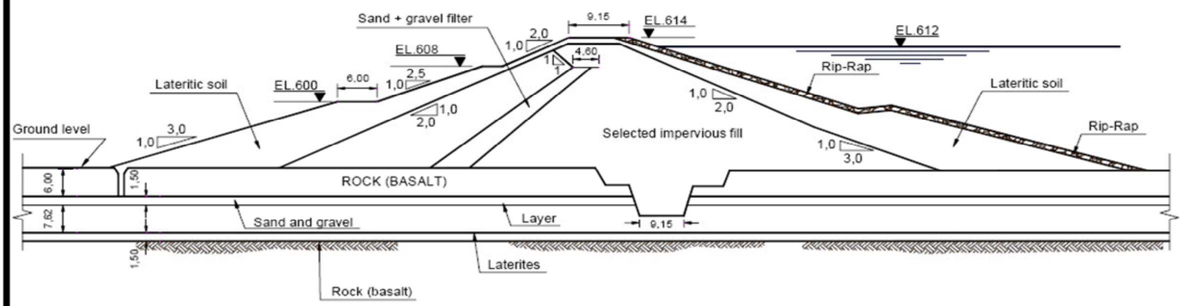
Type of structure	92 m high sloping core rockfill dam
Foundation materials	Metamorphosed igneous and sedimentary bedrock. The formation has been subjected to much folding and faulting. The overburden consists of deeply weathered residual soil on hard, brittle, generally broken unweathered rock.
Foundation treatment	Grout curtain beneath core.

References

Fernandez, Gabriel. Earth Dams Course (CEE 481) Class Notes. University of Illinois at Urbana-Champaing.

CASE STUDY No. 13 Gangapur

Section



General Data

Location	Bombay, India.
Construction Period	1950 - 1953.
Purpose	Hydroelectric power.

Technical Aspects

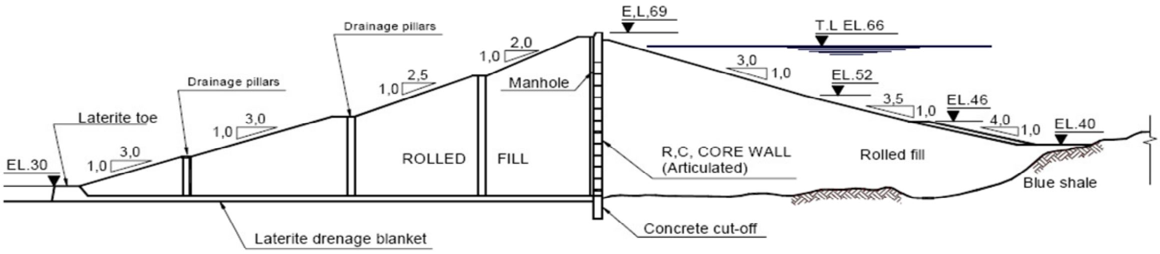
Type of structure	37m high embankment dam.
Foundation materials	Highly fissured plastic clay (black cotton soils).
Foundation treatment	Cut-off trench backfilled with clay. 12" diameter relief wells leading to outfall drains. Filters for the outfall drains.

References

Fernandez, Gabriel. Earth Dams Course (CEE 481) Class Notes. University of Illinois at Urbana-Champaing.

CASE STUDY No. 14 Gyobyu

Scheme



General Data

Location	55 miles north of Rangoon, Burma.
Construction Period	1940 (year of completion)
Purpose	Water supply.

Technical Aspects

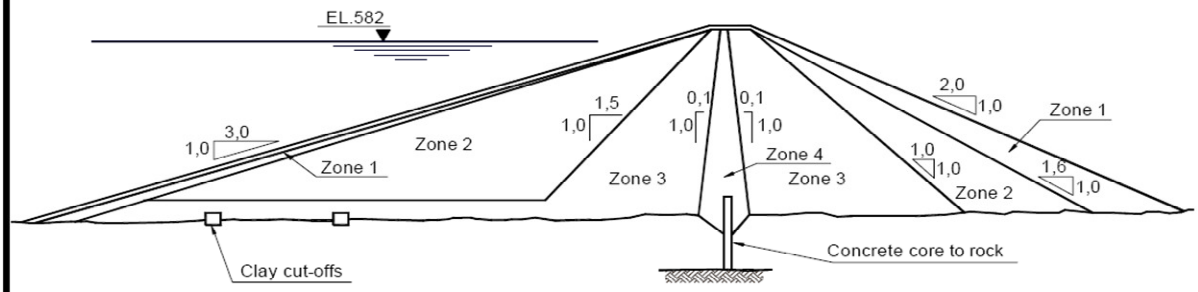
Type of structure	41 m high rolled fill earth dam. Dam materials: light sandy loam compacted in 6" layers. Rolled compacted articulated concrete wall used as core as no clay material was available. The thickness of the wall varies from about 2,5 m at the bottom to abot 1 m at the top of the dam. Manhole installed downstream of the wall to inspect the joints. Downstream laterite vertical drains .
Foundation materials	Blue shale.
Foundation treatment	Laterite horizontal drain at downstream foundation contact and toe. Rolled compacted concrete cut-off wall.
Operational issues	The articulated joints of the concrete wall were designed to withstand at least 5 m of uniform horizontal movement at the crest during an earthquake

References

Fernandez, Gabriel. Earth Dams Course (CEE 481) Class Notes. University of Illinois at Urbana-Champaing

CASE STUDY No. 15 Samson Brooke

Scheme



General Data

Location	8 miles south-eastern of Wakoona, western Australia.
Construction Period	1941 (year of completion)
Purpose	Irrigation

Technical Aspects

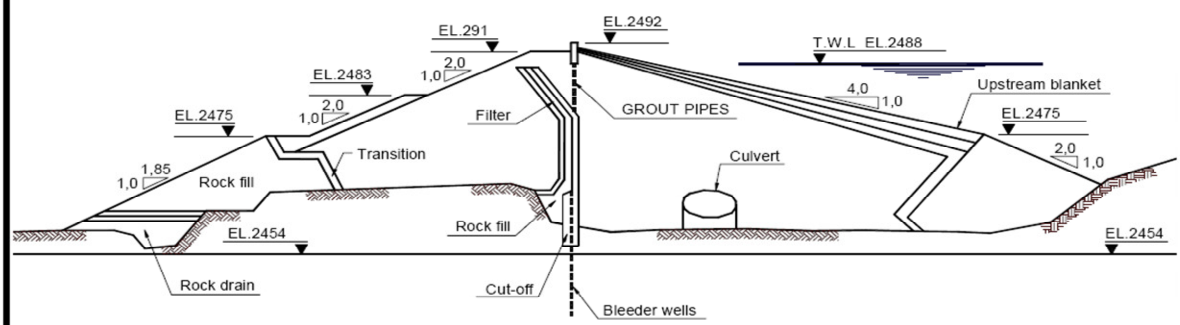
Type of structure	31 m high rolled fill earth dam. Dam materials: Zone 1 – Upstream: 12" of hand-packed ironstone lumps over 6" of ironstone gravel over porous gravelly material varying in thickness from 96" at the toe to 12" at the top. Downstream: Rockfill of ironstone lumps. Zone 2 - Coarser material from the borrow pit with sufficient fines to prevent the occurrence of honeycombing and porous layers. Zone 3 – Low permeability clayey material, free of stony or lumpy fill. Zone 4 - Clay core.
Foundation materials	Deeply weathered Precambrian granitic gneiss-spirodite. Complex structure
Foundation treatment	Impervious blanket beneath the upstream embankment. Concrete cut-off wall to rock beneath core. Clay cut-offs at downstream foundation contact

References

Fernandez, Gabriel. Earth Dams Course (CEE 481) Class Notes. University of Illinois at Urbana-Champaing.

CASE STUDY No. 16 Sasumua

Section



General Data

Location	Nairobi, Kenya
Construction Period	1956 (year of completion)
Purpose	Water supply.

Technical Aspects

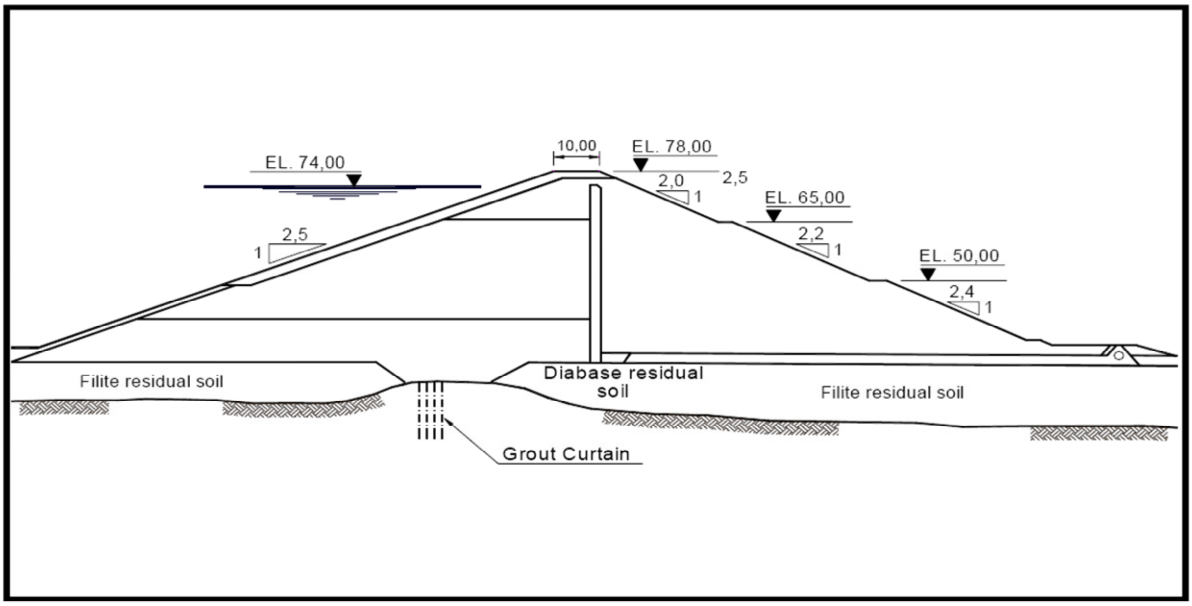
Type of structure	45 m high homogeneous earth fill dam. Dam materials: hydrated halloysite and goethite. LL=90%, PI=33%, wn=63% and wopt=50%.
Foundation materials	Sound lava flows.
Foundation treatment	Concrete cut-off wall underlain by bleeder wells.
Operational issues	Settlement has been less that 2%.

References

Fernandez, Gabriel. Earth Dams Course (CEE 481) Class Notes. University of Illinois at Urbana-Champaing.

CASE STUDY No. 17 Tucurui (phase 1)

Section



General Data

Location	Brazil
Construction Period	1984 (year of completion)
Purpose	Hydroelectric power.

Technical Aspects

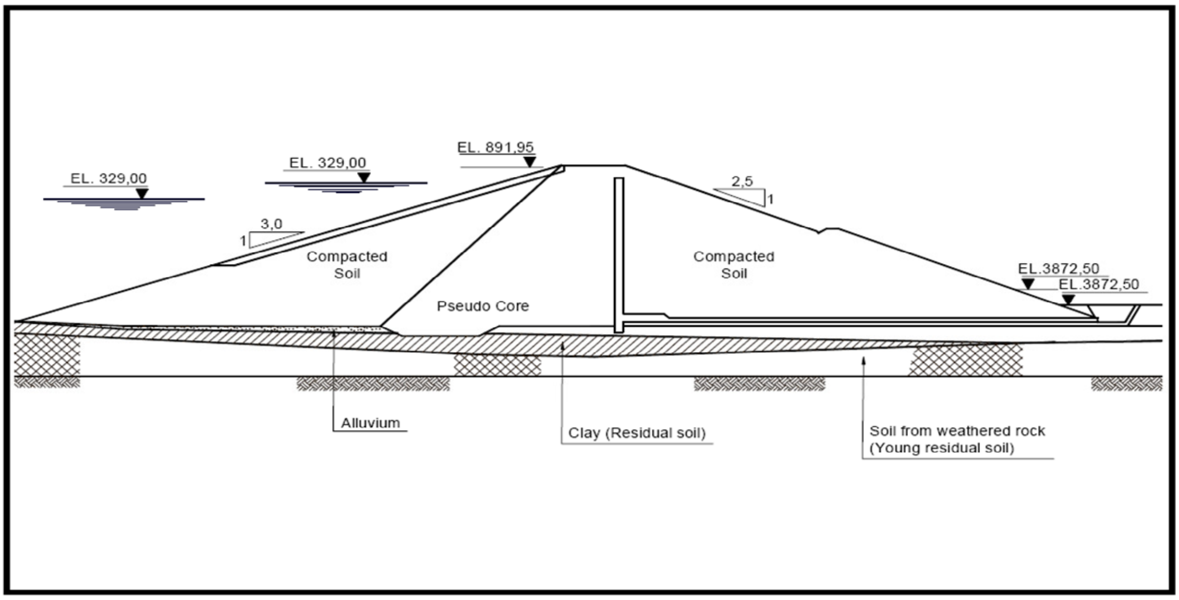
Type of structure	77 m high earth fill and rockfill with impermeable core dam.
Foundation materials	Residual soils and weathered bedrock.
Foundation treatment	Excavation of trenches through the residual soil extending to rock and installation of grout curtains at the earth fill dam locations. Chimney filter-drain connecting to horizontal filter-drain beneath the downstream embankment. At the rockfill with impermeable core sections a concrete blanket was installed at the core-foundation contact along with grout curtains.

References

Cruz, Paulo Teixeira da. "100 Brazilian Dams: Case Histories, Construction Materials, Projects". Sao Paulo: Oficina de Textos, 1996.

CASE STUDY No. 18 Passauna

Section



General Data

Location	Brazil
Construction Period	1987 (year of completion)
Purpose	Hydroelectric power.

Technical Aspects

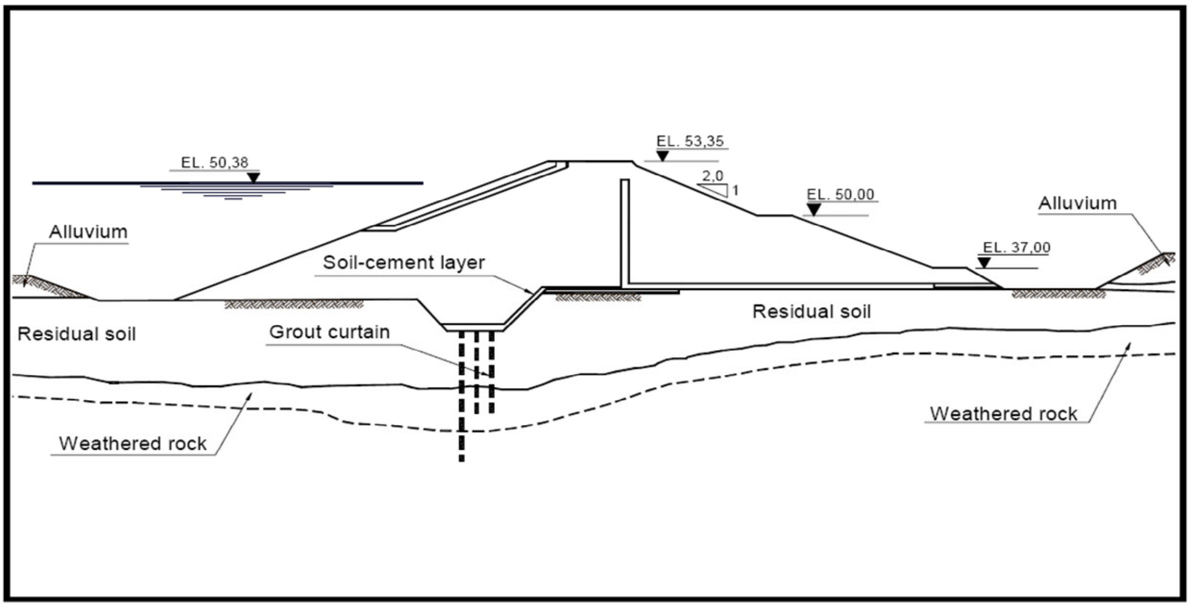
Type of structure	22 m high earth fill with pseudo-core dam.
Foundation materials	Saturated residual soils derived from the weathering of gneiss.
Foundation treatment	A section of the pseudo-core was embedded in the residual soil profile to prevent seepage through a thin alluvium layer. A horizontal drain was constructed beneath the downstream embankment.
Construction issues	During foundation excavation it was noted that the excavators were disturbing the saturated residual soils transforming them into soft clay. To avoid the destruction of the residual soil structure a layer of gravel was placed on top of the foundation material and lighter machines were used. A detailed drainage system, consisting of trenches and pumping wells, was implemented to preserve the foundation residual soils. The system was deactivated when the dam was 8 to 10 m high. Trouble finding borrow zones for the pseudo-core clayey materials caused construction delays.

References

Cruz, Paulo Teixeira da. “100 Brazilian Dams: Case Histories, Construction Materials, Projects”. Sao Paulo: Oficina de Textos, 1996.

CASE STUDY No. 19 Balbina

Section



General Data

Location	Brazil
Construction Period	1989 (year of completion)
Purpose	Hydroelectric power.

Technical Aspects

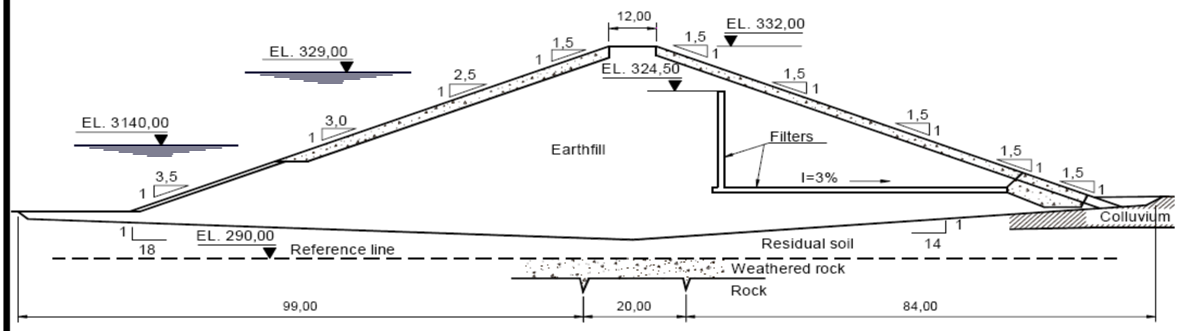
Type of structure	30 m high earth fill dam.
Foundation materials	Colluvial and alluvial deposits and residual soils from volcanic and sedimentary rocks. The sedimentary residual soils occur in the upper level of the abutments and are predominantly of sand nature with some intercalations of clayey layers on the left abutment. The volcanic residual soils are of clayey silty nature with fine sand, low density and low permeability. However, during infiltration tests some zones presented high permeability values that were considered incompatible with the type of material. Later, canalicules were observed in the volcanic residual soils. The diameter ranged from few millimeters in the micro-canalicules to 10 and even 30 mm in the macro-canalicules. Most of the canalicules were not filled giving the soil high permeability. The thickness of the volcanic residual soil profile is variable and can reach up to 25 m.
Foundation treatment	Because of the canalicules in the volcanic residual soils and high permeability of the weathered fractured rock the foundation treatment required the prevention of fine migration and reduction of foundation seepage to acceptable values. A cut-off trench to rock was excavated where the rock level was lower that 5 m. Where the rock was deeper than 5 m a grout curtain was adopted. A pressure high enough to fracture the soil was used during grouting. A protective layer of soilcement was applied tot eh volcanic soil surface at the grouting location.

References

Cruz, Paulo Teixeira da. “100 Brazilian Dams: Case Histories, Construction Materials, Projects”. Sao Paulo: Oficina de Textos, 1996.
Sathler G., Pires de Camargo F."Tubular Cavities, “Canalicules”, in the Residual Soil of the Balbina Earth Dam Foundation", Fifteenth International Congress on Large Dams, Lausanne, Suisse, 24-28 June, 1985.

CASE STUDY No. 20 Ilha Solteira

Section



General Data

Location	Brazil
Construction Period	1973 (year of completion)
Purpose	Hydroelectric power.

Technical Aspects

Type of structure	Earth fill dam with a maximum height of about 74 m.
Foundation materials	Porous colluvial soils, residual soils and bedrock. Due to the different foundation materials differential settlements were of concern.
Foundation treatment	The porous colluvial soils were removed
Operational issues	During first filling, piezometers readings indicated an increasing flow through the foundation. A three meter thick embankment (1 m sand and 2 m of compacted soil) was placed at the upstream and downstream toes. However, seepage continued after the embankments were constructed. Only when relief wells were installed did the piezometric levels decrease to acceptable levels. The perfomance of the dam has been satisfactory.

References

Cruz, Paulo Teixeira da. “100 Brazilian Dams: Case Histories, Construction Materials, Projects”. Sao Paulo: Oficina de Textos, 1996.