

**Investigation of Influence of Compaction  
On the Stability of Earthfill Dams of Tropical Soils**

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**By  
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**Addis Ababa University  
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❖ Mum, If Only You were there...

## DECLARATION

I, the undersigned, declare that this thesis is my work and that all sources of materials used have been duly acknowledged.

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## Abbreviations and Symbols

Designation		Unit
A.A	Sample from Addis Ababa (Semen Gebeya)	-
C	Unit cohesion	kN/m <sup>2</sup>
Dana	Blanket(shell) material of Dana microearth dam	-
D.D	Core Material for Dire Dam	-
e	void ratio	-
Eq.	Equation	-
G.G	Fill material for the cofferdam of Gilgel Gibe Hydroelectric Project	-
G <sub>s</sub>	Specific gravity	-
L.L	Liquid limit	%
p	Mean total stress	kN/m <sup>2</sup>
P.I	Plasticity Index	%
P.L	Plastic limit	%
q	Deviatoric stress	kN/m <sup>2</sup>
s	Degree of saturation	-
USCS	Unified Soil Classification System	-
Φ	angle of internal friction	Degree
γ <sub>d</sub>	Dry unit weight	g/cc
γ <sub>w</sub>	Unit weight of water	“
σ	normal stress	kN/m <sup>2</sup>
σ <sub>1</sub>	Major Principal stress	“
σ <sub>3</sub>	Minor Principal stress	“
τ	Shear stress	“
ω	Moisture content	%

## Abstract

Dam construction is one of the major civil engineering activities, which enhance development. If the construction of such infrastructures can be achieved within sound technical and affordable financial capacity, then it is possible to realize the dream for development and well-to-do life.

One of the merits of earth fill dams is that they can be constructed from locally available natural material (soil) within sound technical and affordable financial capacity. For soils of a given locality to be used safely and economically, appropriate methods of determining the geotechnical properties and shear strength parameters have to be investigated and developed. In addition to this, the appropriate methods of compacting locally available tropical soils have to be developed.

Accordingly, in this paper the influence of compaction on the stability of small earth fill dams of local tropical soils has been investigated. In addition to this, some peculiar geotechnical and geo-chemical characteristics of local tropical soils that have been used in construction of earth fill dams have been studied. Accordingly, the fill material of the cofferdam of Gilgel Gibe Hydroelectric Project has been obtained to be true laterite, while the core material of Dire Dam and the red clay soil of Addis Ababa (Semen Gebeya Area) have been obtained to be lateritic.

The soil samples were compacted at optimum moisture content, at drier of the optimum moisture content and at wetter of the optimum moisture contents. Then, triaxial tests were done on the soil samples to investigate the variation in shear strength properties of these soils, which ultimately affect the stability of the earth fill dams. Compaction moisture has been obtained to influence the stability of earth fill dams. Samples compacted on the drier side of optimum moistures registered high shear strengths. But these strengths were observed to be lost upon further increasing the compaction moisture and also upon sample saturation.

Variation of permeability of compacted soils with respect to compaction moistures and also the effect of saturation settlements were investigated. Accordingly, Permeability has been obtained to decrease towards the optimum moisture content. On the drier and wetter sides of the optimum, higher values of coefficients of permeability have been obtained.

The problem of saturation settlement has been observed to occur when the soil is compacted on the drier side of the optimum moisture.

## **Unit 1. Introduction**

The construction of dams is one of the earliest and most fundamental types of civil engineering activities. The main objective for which dams are constructed is to provide for the safe retention and storage of water. All great civilizations have been achieved with the capability to construct water storage structures that are appropriate to the needs such as irrigation and power generation purposes.

Dam construction represents a major investment in basic infrastructure within all nations. The annual completion rate for dams of all sizes continue to expand steadily in many countries, including some of the more industrialized nations [12].

Dams are designed in such a way that they are of specific solution to site circumstances. The design thus represents an optimum balance of local technical and economic considerations at the time of construction. As a result of this, dams are of numerous types, and their classification is sometimes less clearly defined. In terms of construction material, they can be broadly grouped into two generic groups.

1. Embankment dams: - dams constructed of earth fill and/or rock fill. In these dams, the upstream and down stream slopes are in most of the cases similar and of moderate angle, giving a wide section and a high construction volume relative to height.
2. Concrete dams: - these are dams constructed from mass concrete. Usually they have dissimilar face slopes.

There are also some older dams constructed of masonry.

The embankment dam can be defined as a dam constructed from natural materials excavated or obtained nearby. The materials available are utilized to the best advantage in relation to their characteristics as an engineered bulk fill. The natural fill materials are placed and compacted, in most of the cases, without the addition of any binding agent, using high capacity mechanical plant. Embankment dams are numerically dominant for technical and economical reasons, and account for about 90% of all dams built [12]. They are simple in structural concept (even when compared to the masonry dams), and have also proved to be adaptable to a wide range of site circumstances.

An embankment dam may be categorized as an earth fill dam if compacted soil accounts for over 50% of the placed volume of material. An earth-fill dam is constructed primarily of selected soils compacted uniformly and intensively in relatively thin layers at controlled moisture content. Earth fill dams are further classified as diaphragm, homogeneous and zoned type.

One of the merits of earth fill dams is that they can be constructed of locally available natural materials (soil) that fulfill some criteria of performance as a geotechnical construction material. These geotechnical characteristics of soils have been extensively investigated for temperate climate soils. Also, a great deal of investigations have been carried out to evaluate the influence of compaction on the geotechnical properties of the temperate zone soils with respect to earth fill dam construction and guide lines have been developed based on the investigation results. In contrast, fewer corresponding investigations have been done on tropical residual soils and there are no well-established compaction guidelines that consider the peculiar characteristics of these soils.

On the other hand, the guidelines developed for the design and construction of earth dams of temperate climate soils are commonly used for the tropical residual soils. However, such practices are not appropriate since tropical residual soils exhibit characteristics that are different from that of temperate climate soils.

One typical example to be mentioned here is the case of the fill material of the Tjipanundjang Dam (first described by Terzaghi and later on by L.D Wesley) [18]. The Tjipanundjang Dam is a homogeneous earth dam of 34m height constructed between 1928 & 1939 in West Java, Indonesia. Wesley, based on test results, described the fill material of this rather interesting dam as a yellowish brown andosol soil with unusual properties. The soil is formed through the weathering of volcanic material, mainly ash layers. The liquid limit of this soil has been obtained to be above 100%, which was attributed to its allophane and halloysite mineralogy. The Atterberg limit test results lay below the A – line on the plasticity chart. The grain size distribution of the material indicated that the clay fraction was above 70 %, tested following the procedure for wet sample preparation. However, the shear strength test resulted in surprisingly high values of  $\Phi'$  ( $35^{\circ}$  & above) even though the sample contained about 70% clay.

In the borrow area for the Tjipanundjang Dam cut slopes of over 21m height with side slopes of almost 45<sup>0</sup> have been reported to have remained intact for over 40 years indicating the remarkable stability (strength) of the fill material from which the dam has been constructed.

The points raised above show that the empirical relationships that are based on characteristics of sedimentary clays may not be applicable to tropical soils. In particular, it has been clearly indicated that the general trend towards lower shear strength with decreasing particle size is not applicable. In addition to this, the prediction of lower shear strength with increasing moisture content has been found to be not applicable [18].

Despite the unusual index properties of the soil from which the Tjipanundjang Dam was constructed, the dam was standing (performing) safely as a result of the peculiar performance characteristic of some tropical soils, which was not identified through the geotechnical tests developed for the temperate climate soils.

Moreover, lack of compaction quality control and compaction procedures that may not be applicable to tropical soils have resulted in failures of numerous earth dams. Studies made on failure cases of earth fill dams constructed in Tanzania indicated that lack of compaction quality control was one major cause of the failures [8].

Construction during the dry season often faced severe shortage of water for compaction, and the inadequately available water was generally hauled from long distances. Facilities for compaction control were often unavailable to most dam sites or inadequate for the few that received them. It was concluded that the state of compaction was characteristically poor and the soundness of the completed embankment dams was essentially unknown.

In particular, the case of three dams, which were investigated after failure, is as follows. Wiyenzele Dam (completed in 1963) failed during the first filling, before the water reached the spillway level. It is mentioned that the central portion of the dam was completely washed away along the course of the original valley. Ten years later the dam was reconstructed only to fail again in a similar manner. It was reported that an acute shortage of water for fill compaction was experienced during construction.

Ngerengere Dam (completed in 1974) is mentioned to have failed as a result of d/s slope slip and internal erosion of the dam embankment. During the 1977/78 rainy season several newly constructed earth dams failed in Northeastern Tanzania, the biggest of which (15m) was the newly completed Monduli Juu Dam [8]. Studies indicated that poor compaction led to the dam failure and that compaction water had to be fetched far away from the dam site during construction. It is also mentioned that site witnesses reported of their hearing an explosive sound followed by a cloud of dust during the dam failure [8].

To see the status of some of the small earth dams constructed in our country, paper study has been done and site visit has been paid to some of the micro earth fill dams constructed by Co-SAERAR (Commission for Sustained Agricultural and Environmental Rehabilitation for Amhara Region).

Zana micro earth dam is a modified homogeneous earth fill dam of height 20m constructed in North Gondar (Belesa). It is normally functioning without any visible problem at the time. Another earth dam constructed in North Gondar is Atelkayna micro earth dam. It is of 23 m height, modified homogeneous type. It has been observed that this dam has faced critical seepage (leakage) problem.

Mahbere Genet is a micro earth dam constructed in North Wollo (Sekota). It is a zoned type with the core material having a property of expansiveness. Although functioning at the time of visit, some problems have been observed to exist with this dam, too. The level of water in the dam is far below the design level. Along the crest of the dam (parallel to the dam axis), spot cracks of about 20 cm are visible.

Mylomy is another earth dam constructed at a location very nearby to Mahbere Genet (only about 5 km apart). Mylomy is also a zoned type with an expansive core material. It had a severe seepage (leakage) problem. Spot cracks of similar nature to that of Mahbere Genet are also visible.

Another earth dam constructed in North Wollo (nearby to Woldiya) is Dana micro earth dam. It is a zoned type with a height of 20 m. The core material is of expansive type excavated from the reservoir area, while the blanket (shell) material is reddish brown silty sand. Small spot cracks have been observed on the surface of the crest parallel to the dam

axis. The level of water is low, and it is mentioned by the local people that there has been no increase in the level of water even during the summer, even though this may be because of the insufficient rainfall that prevailed during the summer season of the site visit. Divers mention that they can easily touch the sediment in the reservoir, which is indicative of the existence of siltation problem.

It was also observed that there was scarcity of compaction water nearby to some of the dam sites. As a result, the fill could have been compacted with inadequate water. If efforts were made to attain the maximum dry densities by increasing the number of pass of the compaction equipments, this condition might result in over stressing of the fill material and ultimately formation of cracks. Actually, spot cracks were observed during the field visit paid to some of the dams. Overstressing of the fill material during compaction could have caused these cracks. The effect of saturation settlement of the dry compacted fill material could have also contributed to formation of the cracks.

The above-indicated problems are in one way or another connected with peculiar characteristics of tropical soils and also improper compaction of the soil. From the points raised so far it is clearly seen that if the guidelines established for temperate climate soils are to be inadvertently applied to tropical soils, the consequences will be either failure or uneconomical construction. Therefore, there is still a definite need to carry out investigations on the behavior of local tropical soils with respect to dam construction.

Accordingly, the objective of this research is to investigate the influence of compaction on the stability of earth fill dams of local tropical soils.

Some peculiar geotechnical and geo-chemical characteristics of local tropical soils are investigated with samples taken from sites used as potential sources for fill materials in earth dam construction. The soil samples are compacted at optimum moisture content, at drier of the optimum moisture content and at wetter of the optimum moisture contents. Then, triaxial tests are done on the soil samples to investigate the variation in shear strength properties of these soils, which ultimately affect the stability of the earth fill dams. Small earth dams are considered in the stability analysis. The effect of compaction moisture on permeability and saturation settlement is also investigated.

## **Unit 2. Literature Review**

### **2.1 Characteristics of Tropical Residual Soils**

#### **2.1.1 General**

Blight defines a residual soil as a soil material derived from the in situ weathering and decomposition of rock that has not been transported from its original location [2]. This means that it is not exposed to the sorting process of agents such as water and wind. A tropical residual soil is a residual soil that has been formed within tropics under the influence of humid or semi-humid and warm climate in most of the cases. Residual soils can have characteristics that are quite distinctively different from those of transported soils. In line with this, the conventional concept of a soil grain or particle size is said not to be applicable to many residual soils. This, of course, may be attributed to the fact that the soil is not exposed to the transporting and sorting physical agents like wind and running water. Particles of residual soil often consist of aggregates or crystals of weathered mineral matter that breakdown and become progressively finer if the soil is manipulated. That is, residual soils show a property of changing in physical aggregation upon manipulation, as a result of the change in their macro and micro- fabric structure.

#### **2.1.2 Origin, Formation and Identification**

Tropical residual soils are most commonly formed from igneous or metamorphic parent rocks, but it is not uncommon to see residual soils that are formed from sedimentary rocks. The three major agencies of weathering (soil formation) are physical chemical and biological processes.

Chemical processes tend to dominate the weathering of igneous rocks, where as physical processes dominate the weathering of sedimentary & metamorphic rocks. However, it is also well known that physical and chemical processes are so closely interrelated that one process never proceeds with out some contribution of the other. Occasionally, residual soils may form by the in situ weathering of unconsolidated sediments. Loess or collapsing sand formed by the weathering of feldspars in deposits of windblown sand is the commonest example.

In the weathering process, the parent rock and rock forming minerals break down, releasing internal energy and forming substances having lower internal energy which are, therefore, more stable [2]. Physical processes like stress release by erosion, differential thermal strain, etc disintegrate the rock, expose fresh surfaces to chemical attack and increase the percolation of chemical reaction fluids. Chemical processes, chiefly hydrolysis, cation exchange and oxidation alter the original rock minerals to form more

stable clay minerals. Biological weathering includes physical action (splitting by root wedging) and chemical action (bacteriological oxidation), chelation and reduction of iron and sulphur compounds.

Hydrolysis is considered to be the most important of the chemical weathering processes. It occurs when a salt combines with water to form an acid and a base. In rock weathering, the salt is usually a silicate and the product of the reaction is a clay mineral. Oxidation follows after hydrolysis and affects rocks containing iron sulphates, carbonates and silicates.

Ionic transfer between percolating solutions and the original mineral can facilitate the break down of one clay mineral to form another. Cations such as sodium and calcium are the most readily exchangeable [2]. Cation exchange does not alter the basic structure of the clay mineral, but the crystal interlayer spacing may change and consequently, converts an illite to a montmorillonite, for example.

Bacteria may play the role of catalysts in certain chemical reactions that are responsible for residual soil formation. The presence of the bacteria called thio-bacillus thiooxidans, for example, may enormously accelerate the oxidation of sulphide minerals [2]. Chelation is a process by which lichens growing on rock surfaces promote the rate of hydrolysis.

Pedologists have agreed that the products of rock weathering are determined by certain physical conditions designated as the soil-forming factors [9]. These include climate (precipitation and temperature), topography (drainage), vegetation, parent rock and time. They are of importance because they bear on the possibility of classifying the soils on a rational genetic basis.

The fact that climate exerts a considerable influence on the rate of weathering is well understood; physical weathering is more predominant in dry climates while the extent and rate of chemical weathering is largely controlled by the availability of moisture and temperature. Studies indicate that, other factors being equal, the rate of a chemical reaction approximately doubles for every 10<sup>0</sup>c increase in average temperature [2].

Climate is mentioned to have a further possible effect on the properties of the tropical residual soils. As a result of climatic factor water tables are said to be often deeper than 5m to 10m (even in sub humid tropical or subtropical areas) and the effects of saturation, desiccation and seasonal/long term re-wetting are recommended to be taken into account in geotechnical design.

Topographic relief has also an important role in soil formation. For a deep residual soil to develop, the rate at which weathering advances into the earth's crust must exceed the rate of removal of the products of weathering by erosion. Topography controls the rate of weathering by partly determining the amount of available water and the rate at which it moves through the zone of weathering. In addition to this, it also controls the effective age of the profile by controlling the rate of erosion of weathered material from the surface. Thus, deeper residual profiles will generally be found in valleys and gentle slopes rather than on high ground or steep slopes [2].

According to a study made by Van der Merwe on formation of residual soils from basic igneous rocks, topography and drainage plays a significant role [2]. According to the study made upon samples taken from three different sites, samples taken from a site high upon a slope with good run off showed kaolinite and vermiculite to be the dominant clay minerals. A flatter site indicated chlorite, vermiculite, montmorillonite and kaolinite in weathering sequence. Samples taken from a flat site with impeded drainage, contained montmorillonite as the predominant mineral. This study indicated that good internal drainage and high rainfall are favorable to the development of kaolinite whilst flatter slopes and poor drainage favour the formation of montmorillonite.

The parent rock determines which clay minerals form initially [9]. It is mentioned that kaolinite cannot form, unless alumina is present. The parent rock may be a dominant factor or only of transitory significance. Weathering in the warm humid climate, which is common in the tropics, is rapid and the influence of the parent materials is usually short lived. However, it is clear that the parent material must provide the constituent elements of which the clay minerals are formed.

The process of formation of a residual soil profile is obviously extremely complex, difficult to understand and difficult to generalize. Blight explains that apart from a few

valid generalizations, it is difficult to relate the properties of a residual soil directly to its parent rock. It is mentioned that each situation requires individual considerations even if the underlying hard rock geology in the two areas is similar. As an example, Blight mentions that the weathered granite soils of the warm, humid Malaysian peninsular have very different properties to those of the cooler, semi-arid Transvaal Highveld in South Africa [2].

Wesley concluded that the dark colored andosols and red latosols found in Java originate from much the same volcanic parent material but occur in profiles of different ages [2]. All these indicate the significance of studying/analyzing the nature of a given soil relative to the specific conditions under which it is formed.

In any weathering process that converts rock into soil there will be a gradual transition without a clearly defined point at which the material changes from rock to soil. In connection with this, the residual soils can be described as saprolytic or lateritic.

The term saprolite is used to describe residual soils with clear structural features inherited from parent rock. Saprolites are residual materials that have soil-like strength or consistency, but retain recognizable relics of the physical features or fabric of the parent rock. For example, it is mentioned that a saprolite derived from the weathering of lava may retain the flow structure of the parent rock. One derived from shale often retains the bedding and jointing patterns of its parent rock. Saprolites may occur in profiles of almost any depth [2].

Lateritic soils are residual soils as they are products of in situ weathering. Alexander and Cady have described laterites as products of intense tropical sub aerial weathering of materials rich in ferro-alumino silicates. They are rich in sesquioxides (secondary oxides of iron, aluminum or both) and low in bases and primary silicates but may contain appreciable amounts of quartz and kaolinite [2].

It is mentioned that the term laterite was first coined by Buchanan in 1807 for a certain material he saw in India, which was capable of hardening on exposure [8]. The hardening property of laterites referred to is the result of loss of kaolinite, enrichment of oxides, dehydration and crystallization of amorphous iron oxide minerals. However, it is often

mentioned that other residual soils which are rich in sesquioxides and kaolinite and low in bases and primary silicates, but which are neither hard nor capable of hardening, can also be referred to as lateritic soils. Due to the presence of iron oxides lateritic soils are red in colour ranging from light through bright to brown shades.

Laterites are believed to always correspond to climates in which the wet season is warm. This applies to semi-humid tropical and equatorial climates. But subtropical climates in which the wet season is cold do not seem to favour laterization [9]. The sub aerial weathering process is known to cause morphological changes with depth and this is reflected in the division of soil in to horizons, each of which consists of materials of different composition and structure. As a result of this, to identify soil in a deposit, the vertical morphological variation in the profile should be considered, as soils formed under similar weathering conditions are known to have similar morphological, mineralogical and geotechnical characteristics.

Lateritic soils are observed to have unique significant geotechnical properties as a result of genetic leaching effects and concretion of the sesquioxides of iron and aluminum. The significance of leaching is associated with the development of high in situ porosity, which leads to high permeability and low in situ density. The significance of the sesquioxide concentration is manifested in coating and aggregating of the soil particles as well as the formation of lateritic rock. One mechanism of aggregation is when the positively charged sesquioxides are absorbed in to the surface of the negatively charged clay minerals; the clay behaviour will be suppressed. Smaller particles are cemented together to form bigger aggregates, which are known to have better geotechnical properties. It is thus, seen that the higher the state of laterization the better the geotechnical properties. With increase in accumulation of the amount of iron oxide, the specific gravity of the soil grains increases and together with this, the density of the soil, mechanical strength and durability increase but plasticity and water absorption decrease.

As far as the chemical characteristic is concerned, lateritic soils exist in various stages of weathering (laterization, desiccation and decomposition) which is known to have profound influence on their geotechnical properties. Based on the chemical test results, the lateritic soils can be assessed by the Winterkorn/ Chandrashekharam silica/sesquioxide ratio. Silica

has got a chemical formula of  $\text{SiO}_2$ . Sesquioxide is a combination of aluminium oxide ( $\text{Al}_2\text{O}_3$ ) and iron oxide ( $\text{Fe}_2\text{O}_3$ ), designated as  $\text{R}_2\text{O}_3$  [2].

Accordingly, if the ratio, R, of silica/ sesquioxide is less than or equal to 1.33, this indicates true laterites. If  $1.33 < \text{R} < 2.00$  then the soil is said to be lateritic. Non-laterites have a ratio of  $\text{R} > 2.00$  [2]. The Gidigas chemical characteristic diagram is another method of evaluating the degree of laterization. Gidigas found out that lateritic soils formed in dry tropical climates under savannah vegetation with annual rainfall  $< 1200$  mm and evaporation exceeding precipitation are ferruginous soils. Those formed under the annual rain fall of  $> 1200$  mm are feralitic. Studies show that the former have liquid limit  $< 50\%$  and the plasticity indices  $< 30\%$ . The latter have higher corresponding values [8].

### **2.1.3 Classification and Index Property Tests**

#### **2.1.3.1 Classification**

##### **What is the need for special classification?**

L.D. Wesley and T.Y Irfan have mentioned the following specific features of residual soils that are supposed not to be adequately covered by conventional methods of soil classification such as the unified soil classification system [2].

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

Wesley proposed a practical system for classifying all residual soils, based on mineralogical composition and micro and macro-structure [2]. His classification system is intended to provide an orderly division of residual soils into groups which belong together because of common factors in their formation and/or composition which can be expected to give them similar engineering properties. The system is supposed to be based on a grouping framework designed to enable engineers to place any particular residual soil into a specific category on the basis of common engineering properties. It is reported that the system is not intended to provide an all embracing method for the detailed systematic description of residual soils as a replacement to any particular method of classification in use at present. It is to be used in conjunction with the standard methods such as the Unified Soil Classification System or any other method proposed specifically for residual soils, by supplementing the existing systems at the points where their applicability is questioned [2].

It has already been mentioned that composition and structure are the two bases for the Wesley classification scheme. Composition refers to the material of which the soil is made, and includes the particle size, shape and especially the mineralogical composition of the fine fraction. It is, in fact, divided into physical and mineralogical composition. Structure refers to the specific characteristics of the soil in its undisturbed in situ state and can be subdivided as follows.

**Macrostructure:** – this includes all features visible to the naked eye such as layering, discontinuities, and presence of unweathered or partially weathered rock and other relict structures inherited from the parent rock.

**Microstructure:** - this include fabric, inter particle bonding or cementation, aggregation of particles, pore sizes & shapes, etc.

The first step in the grouping residual soils according to Wesley is to divide the soils in to groups on the basis of mineralogical composition alone. The following three main groups are suggested:

**Group A** - Soils without a strong mineralogical influence

**Group B** – Soils with a strong mineralogical influence deriving from clay minerals also found in transported soils.

**Group C** – Soils with a strong mineralogical influence deriving from clay minerals found only in residual soils.

The fact that mineralogical composition is the starting point has been mentioned as the disadvantage of Wesley Classification System. But it has been shown that the occurrence and mineralogy of local soils could be available from pedological studies and geological surveys, too. Wesley's schematic table for the classification is given below.

### **2.1.3.2 Index property Tests**

It is well known that oven drying, sun drying and even air-drying affects the properties of soils, although this effect is usually small for transported soils. Residual soils are in particular prone to changes in properties as a result of drying and exposure to air. This is because residual soils are formed in situ decomposing in a largely anaerobic environment. Drying can cause partial or complete dehydration of the clay minerals and can change their property irreversibly [2]. Although the severity of the effect may be somewhat different, even air drying at ambient temperature can cause changes that cannot be reversed through re-wetting and maturing for long periods.

Consequently, all the classification and index property tests that involve drying and wetting are deemed to affect the residual soils and hence are not expected to give realistic results as far as the in situ performance of the soil is concerned. In addition to this, method of mixing and time duration of mixing is known to affect the index property of residual soils. As a result of this, wet sample testing (at natural moisture) and controlled drying temperature, usually below 50<sup>0</sup>c, are recommended as remedial measures. G.E Blight recommends that Atterberg limit tests should be done without any form of drying prior to carrying out the test [2]. This is, of course, with the appropriate care in selecting the method of mixing and predefined duration of mixing.

## **2.2 Peculiar Characteristics of Tropical Soils wrt to Dam Construction**

Tropical residual soils are widely used as construction materials, mainly as fill for embankment dams and road embankments and also as selected layers in airfield construction. Some residual soils, such as those containing smectite or halloysite clays may be unsuitable for these uses, because of either inadequate strength, or excessive change of volume with varying water content, or because of loss of strength on wetting. However, it is often reported that smectite and halloysite materials have been used to form impervious layers in embankment dams. Sasumua Dam (known to be described by Terzaghi) [2], the Arenal Dam (described by Rodda et al) & the Tjipanundjang Dam (described by Terzaghi & later on by Laurence D. Wesley) are some of the examples [18].

Some special characteristics of tropical residual soils must be understood if the compaction process is to be understood and the effort and cost of compaction is to be optimized [2]. The following points are reported to be some of the characteristics anticipated with special considerations for efficient compaction of residual soils. Residual soils especially those of volcanic and igneous origin are reported to have [2]:

- high in situ moisture minerals
- meta-stable clay minerals
- soil structures that are lightly cemented
- weathered soil particles that breakdown under compactive effort
- sesquioxide minerals that are affected by wetting and drying.

Obviously, the starting point for many engineering use of soil is the in situ, borrow pit. Many tropical and subtropical environments are characterized by frequent or seasonal heavy rainfall. Handling of soils under these conditions are known to be difficult and the characteristics of the soils may add further complexity to the problem of effective compaction.

Drying of the soil from its in situ water content may change both its index and compaction properties. Because of this soil samples have to be treated and tested with greatest care if the results of compaction tests are to be at all meaningful.

Compaction often results in progressive breakdown of particles in a residual soil. Gidigas & Dogbey illustrated the progressive breakdown of particle size under compaction of quartzitic gravel and lateritic gravel which are both residual soils formed from the weathering of granite [2]. In cases like this, it is required to use a fresh soil sample to establish each point on the compaction curve. Otherwise, the compaction characteristics of the soil will change progressively as the test proceeds, and the test result may be meaningless.

## Unit 3. Laboratory Tests and Analysis

### 3.1 Index Property Tests

Soil is a more complex material to characterize as compared to other engineering materials. Basically, the complexity is attributed to the fact that soil is a combination of materials of different phases, namely solid, liquid and gases [13]. This situation is further complicated because of the fact that the relative quantities of the solid, liquid and gases in a given soil is changing under the influence of loading, seasonal variation and change in temperature. In addition to this, the mineralogical composition is of different type, and as a result of this, it is possible to say that soil exists in almost innumerable varieties.

Tests for determination of the physical properties of soils are given priority as the physical characteristics are of first hand importance to engineers. To understand the textural behavior of soils, the composition and the relative amount of each component must be known. The textural composition (particle size distribution) of coarse grained soils is obtained by screening a known weight of the soil through a stack of sieves of progressively finer mesh size, where as the particle size distribution of fine grained soils is commonly determined from a hydrometer test [4].

Atterberg limits are done to determine the consistency of fine-grained soils. The physical properties of soils which serve mainly for identification and classification are commonly known as index properties. Grain size analysis, Atterberg limits, free swell & specific gravity tests of soils could be considered as index property tests. The procedures followed in the laboratory tests are that of ASTM.

#### 3.1.1 Grain size Analysis

##### Dry preparation of sample

Soil sample brought from field (test pits) was first dried (ASTM D421 –21) and then pulverized before it was screened through the nest of sieves. During the test procedure, the sample was to be divided in to two portions. The portion containing particles coarser than no 10 (2.00 mm) sieve was to be screened through mechanical shaking before any further treatment. The other portion passing the no 10 sieve was to be subjected to a hydrometer test. As the soils from the different sites were proved to be finer than the no 10 sieve, the grain size analysis was done through hydrometer and sieve analysis of the portion passing the no 10 sieve (according to the procedure detailed in ASTM D 422 – 63).

In reference to the locations from which the samples were taken, the investigated soils are designated as follows.

A.A .....Sample taken from Addis Ababa (Semen Gebeya)

D.D .....Sample taken from where some of the core material for Dire Dam was taken (about 35 km away north east of Addis Ababa).

Dana ..... Blanket (shell) material for Dana micro earth dam  
(North Wollo)

G.G ..... Material for the earth-fill cofferdam of the Gilgel Gibe  
Hydroelectric Project (obtained from nearby to the  
dam site).

The Percentage amounts of the grain sizes obtained are indicated in table 3.1. 1/1

Table 3.1.1/1 Percentage Amount of the Grain Sizes, Dry Sample

Location of the sample	Test No	Depth of sampling (m)	Percentage Amount of the grain sizes		
			Sand	Silt	Clay
Dana	1	0.6	75	22	3
D.D	1	1.0	18	54	28
G.G	1	1.0	16	34	50
A.A	1	1.0	13	33	54

The grain size distribution curves for dry sample preparation are indicated below in fig.  
3.1.1/1 a, b, c & d



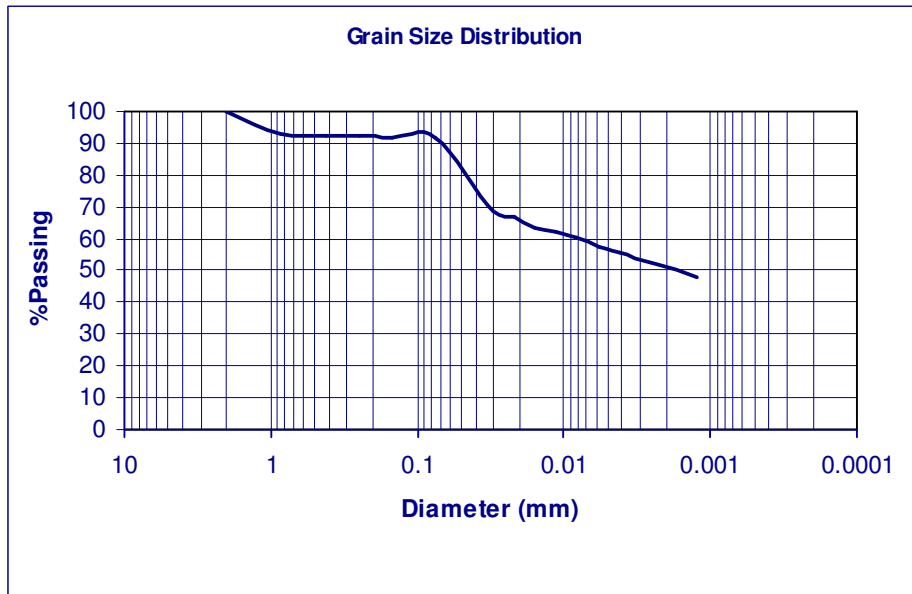


Fig.3.1.1/1c) Sample G.G

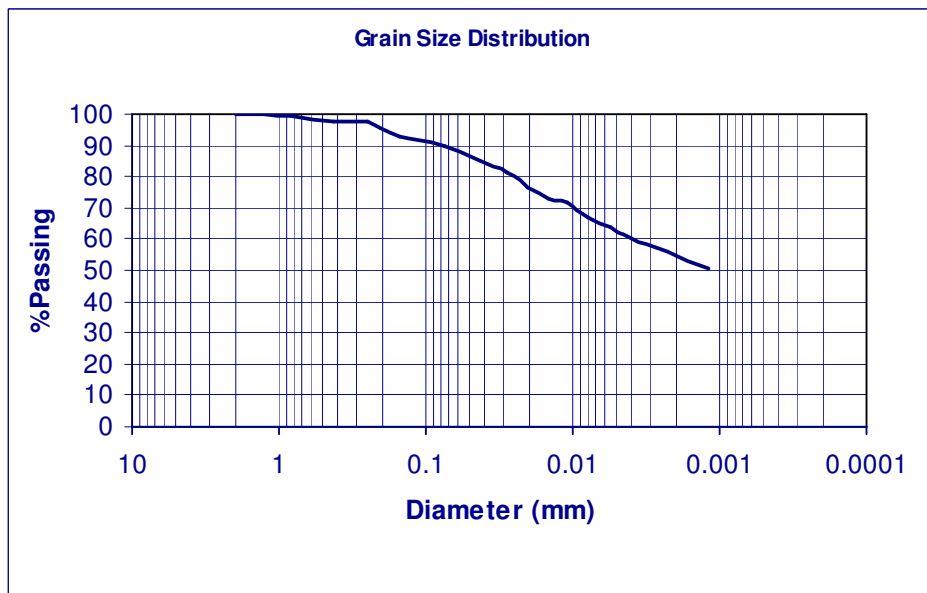


Fig.3.1.1/1d) Sample A.A

Fig.3.1.1/1 Grain Size Distribution Curves- Dry Sample Preparation

## Wet Preparation of Sample

in addition to the dry preparation of sample, index property tests were done following the procedure for wet preparation of sample, also (as detailed in ASTM D 2217-85) and [2]. It was tried to determine the grain size distribution, liquid and plastic limits for the sample obtained from Addis Ababa (Semen Gebeya). This was done to determine the true natural gradation and plasticity characteristics of the soil without affecting the natural in situ characteristics of the soil through drying. (The sample used was at moisture content of 20%). The results obtained are indicated in table 3.1.1/2 & fig. 3.1.1/2 a and b.

Table 3.1.1/2 Percentage Amount of the Grain Sizes, Wet Sample

Location of the sample	Test No	Depth of sampling (m)	Percentage Amount of the grain sizes		
			Sand	Silt	Clay
A.A	1*	1.2	16	32	52
	2	1.2	15	31	54

The Sample indicated under 1\* is not worked by hand, whereas the sample indicated under test no 2 is worked by hand.

The grain size distribution curves for wet sample preparation are indicated below in fig. 3.1.1/2 a & b

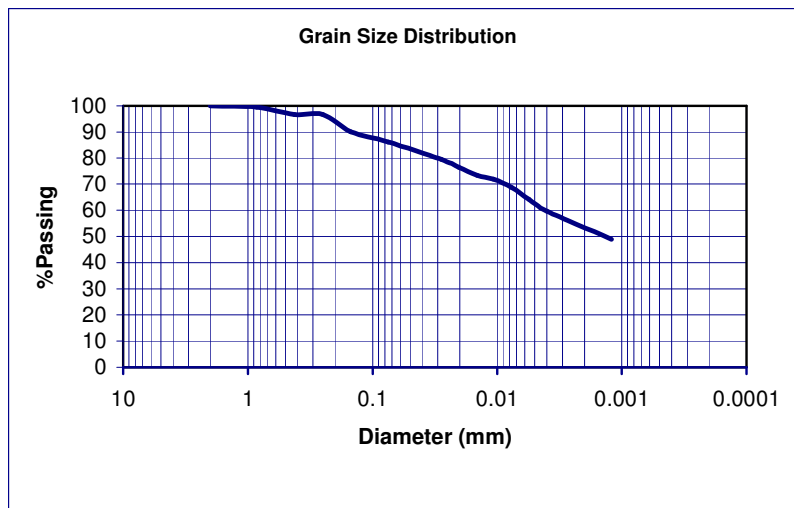


Fig. 3.1.1/2a) Sample A.A –1\*

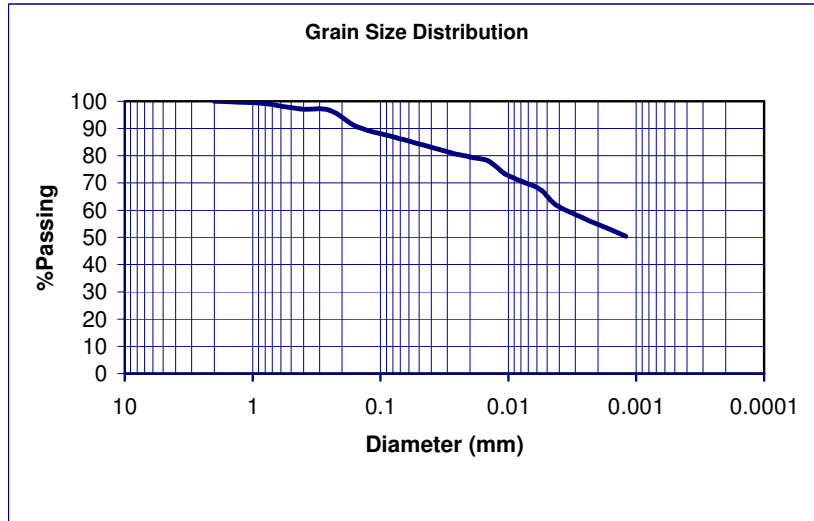


Fig. 3.1.1/2b) Sample A.A -2

Fig.3.1.1/2 Grain Size Distribution Curves - Wet Sample Preparation

### 3.1.2 Atterberg Limits

For the determination of values of Atterberg limits the soil samples were tested following the procedures given in ASTM D4318 – 84, and the results obtained are given in tables 3.1.2/1 and 3.1.2/2 for the dry and wet preparation of samples respectively.

**Table 3.1.2/1 Atterberg Limits, Dry Sample**

Location of the sample	Test No	Depth of sampling (m)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)
Dana	1	0.6	35	20	15
D.D	1	1.0	66	36	30
G.G	1	1.0	69	36	33
A.A	1	1.0	65	32	33

**Table 3.1.2/2 Atterberg Limits, Wet Sample**

<b>Location of the sample</b>	<b>Test No</b>	<b>Depth of sampling (m)</b>	<b>Liquid limit (%)</b>	<b>Plastic limit (%)</b>	<b>Plasticity index (%)</b>
A.A	1	1.2	64	31	33

Wet sample preparation was followed to investigate the effect of sample drying condition on grain size distribution and plasticity characteristics of the soil samples under investigation. The grain size distribution and plasticity characteristics of some soils change as a result of hydration reaction when being dried. The test result for wet preparation of sample is expected to result in increased percentage of the finer fraction (clay), and consequently increased plasticity indices. But the test results of both the grain size analysis and Atterberg limits of the sample taken from Addis Ababa (Semen Gebeya) resulted in negligible difference in percentage composition and plasticity property for the samples tested following the procedures for dry sample preparation and wet sample preparation to investigate the effect of sample drying.

Plasticity index (P.I) shows the range over which the soil is in plastic state. A high numerical value of plasticity index is known to be an indicator of the presence of high percentage of clay in the soil sample [13]. As far as conventional classification system such as USCS is concerned, information regarding the type of fines in the sample may be obtained by considering the plasticity index in relation to the liquid limit. Experimental results of soils tested from different parts of the world were plotted on a graph of plasticity (ordinate) versus liquid limit (abscissa), and it was found out that clays, silts and organic soils lie in distinct regions of the graph [4].

A line defined by the equation

$$P.I = 0.73 (L.L - 20) \dots\dots\dots \text{Eq. (3.1.2/1)}$$

(P.I and L.L in %), called the “A-line”, delineates the boundary between clays (above the line) and silts & organic soils (below the line).

A second line, called the “U-line” expressed as

$$P.I = 0.9 (L.L - 8) \dots\dots\dots \text{Eq (3.1.2/2)}$$

defines the upper limit of the correlation between plasticity index (P.I in %) and the liquid limit (L.L in %). If the results of the test fall above the U-line then the results obtained are doubtful and hence the tests are recommended to be repeated [4]. In this case, all the test results are found to lie below the U- Line, and hence considered acceptable.

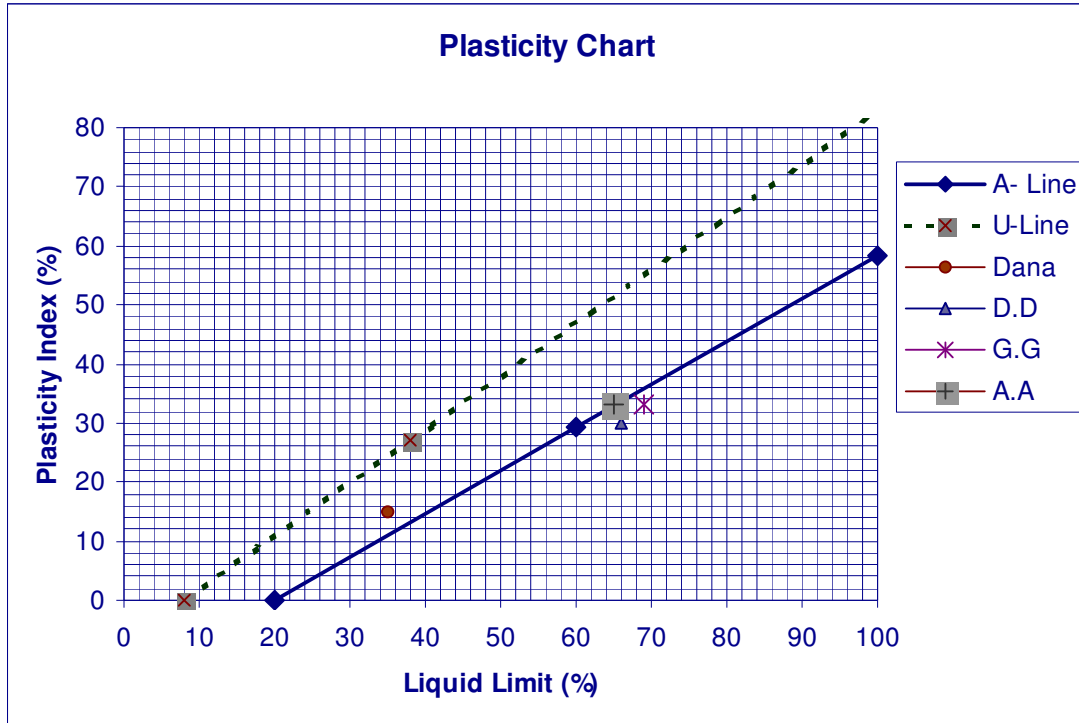


Fig.3.1.2/1 Plasticity Chart

### 3.1.3 Free Swell

nount of swelling and the magnitude of swelling pressure are known to be dependent on the clay minerals present in the soil, the soil mineralogy and structure, fabric and several physico-chemical aspects of the soil. Among the clay minerals, montormorillonites are known to influence the magnitude of swelling more than illites and kaolinites [6].

f the simple tests developed to study the swelling property of soils is the free swell test. The free swell test is a good indicator of degree of expansiveness. The test is performed by slowly pouring 10cm<sup>3</sup> of dry soil which has passed the no 40 sieve into a 100 cm<sup>3</sup> graduated cylinder filled with water. The difference in volume is observed, the final volume being the volume of the soil in the suspension after 24 hrs.

$$\text{Free Swell} = \frac{\text{Final volume of soil} - \text{Initial volume of soil}}{\text{Initial volume of soil}} \times 100 \%$$

Free swell test results are summarized in table 3.1.3/1

Table 3.1.3/1 Free Swell Test Result

Location of the sample	Test No	Depth of sampling (m)	Free swell (%)
D.D	1	1.0	60
G.G	1	1.0	35
A.A	1	1.0	40

According to the practice, which is based on suggestion of Holtz, soils with free swell value of less than 50% are considered to be nonexpansive, while those with free swell value of 50% to 100% are considered to be of intermediate degree of expansiveness. Free swell value of greater than 100% is supposed to indicate that the soil is expansive. According to the free swell test result, the core material of Dire Dam is of intermediate degree of expansiveness while that of the cofferdam of the Gilgel Gibe Hydro electric Dam and the sample from Addis Ababa (Semen Gebeya) are non- expansive.

#### 3.1.4 Specific Gravity

The specific gravity is an important factor which is used in computing other soil properties such as void ratio, the unit weights, soil particle size distribution by means of the hydrometer, degree of saturation, etc. The specific gravities of the soil samples were determined following the procedure given in ASTM D854 – 58 and the results obtained are as follows.

Table 3.1.4 /1 Values of specific gravity

Location of sample	Test No	Depth of sampling (m)	Specific gravity
Dana	1	0.6	2.65
D.D	1	1.0	2.75
G.G	1	1.0	2.70

A.A	1	1.0	2.74
-----	---	-----	------

### 3.1.5 Classification of the Soil

Making use of the Grain size distribution and the Atterberg Limit values obtained, the soils have been classified according to the USCS. Accordingly, the core material for Dire Dam has been obtained to lie below the A- Line, in the area for the soils that are classified as inorganic silts. The grain size composition is in agreement to this situation with about 54% silt content and about 28% clay content. The soil has already been classified as clayey silt during the investigation of fill material for Dire Dam. The values obtained here are in agreement with the established ones.

The soil that has been used as fill material for the cofferdam of Gilgel Gibe Hydroelectric Project has been obtained to lie a little bit below the A –Line which makes this soil be classified as inorganic silt based on the USCS. But the grain size composition of this soil indicates that about 50 % of the soil is clay. The remaining 34% and 16% are silt and sand respectively. Thus, the clay material dominates from grain size point of view. Much of the properties of this soil are dependent on the conditions under which the tests are carried out and hence it is difficult to characterize this soil based on the method of USCS only.

The soil from Addis Ababa (Semen Gebeya) lay above the A –Line and hence has been classified as inorganic clay of high plasticity according to the USCS. The sieve and hydrometer analysis indicated that the clay content of this soil is about 53 % on the average.

Table 3.1.5/1 Soil Classification According to the USCS

Location of the sample	Liquid Limit (%)	Plastic Limit (%)	Percentage Amount of the grain sizes			Classification According to USCS
			Sand	Silt	Clay	
Dana	48	24	75	22	3	Silty sand
D.D	74	36	18	54	28	MH (Inorganic Silts)
G.G	69	36	16	34	50	MH (Inorganic Silts)
A.A	64	31.4	13	33	54	CH (Inorganic Clays)

### 3.2 Geochemical & Mineralogical Tests

Geochemical (oxide) and Mineralogical composition of a soil is essential to understand the fundamental behavior of the soil. Mineralogy is the primary factor controlling the size, shape physical and chemical properties of soil particles. The solid phase of a soil sample is known to contain various amounts of crystalline clay material, oxides, organic matter and precipitated salts [11]. It has been understood that the property of tropical residual soil is quite dependent on Geochemical & Mineralogical composition of the soil. It has also been mentioned that soil classification and performance ratings, which are based on mineralogical composition and structure has been proposed by Wesley and Irfan [2].

#### 3.2.1 Geochemical Test

Geochemical Test furnishes the oxide composition of the soil material. The existence of oxides and hydroxides of aluminium, iron, and silicon are of greatest interest since they are the ones most frequently encountered. These materials coat mineral particles, or

cement soil particles together. They may also occur as distinct crystalline units, such as hematite and magnetite.

Geochemical test was conducted at the Geological Survey of Ethiopia Geochemical Laboratory. Atomic Absorption Spectrometry and Colorometer Analysis method was followed to obtain the percentage oxide composition indicated in table 3.2.1/1

Table 3.2.1/1 Percentage Oxide Composition

Sample Location	SiO <sub>2</sub>	Al <sub>2</sub> O <sub>3</sub>	Fe <sub>2</sub> O <sub>3</sub>	CaO	MgO	Na <sub>2</sub> O	K <sub>2</sub> O	MnO	H <sub>2</sub> O	LoI	TiO <sub>2</sub>	P <sub>2</sub> O <sub>5</sub>
D.D	48.90	16.93	13.73	1.46	1.50	0.68	1.02	0.16	1.50	9.40	3.12	0.30
G.G	36.57	24.90	18.30	0.18	0.72	0.01	0.36	0.11	5.52	12.24	2.45	0.09
A.A	50.42	18.18	8.93	0.17	0.61	0.28	1.35	0.13	9.71	7.51	1.25	0.09

The degree of laterization of the soil samples were evaluated based on the Winterkorn/Chandrashekaram ratio of silica/sesquioxide as detailed in section 2.2. As previously indicated, silica is given by the chemical formula SiO<sub>2</sub>, where as sesquioxide is the combination of aluminium oxide (Al<sub>2</sub>O<sub>3</sub>) and Iron oxide (Fe<sub>2</sub>O<sub>3</sub>); sesquioxide is designated as R<sub>2</sub>O<sub>3</sub>.

The sample from Gilgel Gibe has resulted in Silica/Sesquioxide ratio of 0.85, which indicates that the soil is a true laterite. For the core material of Dire Dam the Silica/Sesquioxide ratio has been obtained to be 1.6. Thus, it is not a true laterite but has undergone a considerable degree of laterization and can be called lateritic soil.

The sample from Addis Ababa (Semen Gebeya) has got Silica/Sesquioxide ratio of 1.9. According to the value obtained from this test, the soil can be classified as lateritic, but as the value obtained is on extreme side, it is clear that the degree of laterization of this soil is very small as compared to the other soils. Some papers mention that even though the physical and geotechnical properties of the Addis Ababa red clay soil is much similar to that of lateritic soils, but the soil fails to fulfill the criteria for lateritic soils from chemical composition point of view [6].

The locations from which the samples were taken had experienced climatic conditions in which the wet season is warm (semi- humid tropical and equatorial climates). This is especially true with the sample from Gilgel Gibe (Jimma) in which case the soil has been obtained to be true laterite. It is also worth mentioning that the climate of Addis Ababa and the surrounding is modulated by relatively high altitudes.

### 3.2.2 Mineralogy

Mineralogy controls the sizes, shape & surface characteristics of the particles in a soil. These features along with interactions with the fluid phase, determine plasticity, swelling, compression strength and hydraulic conductivity behavior. Thus, mineralogical composition (together with structure) is an important factor that is fundamental to the understanding of geotechnical properties [11].

Soil mineralogy can be assessed in various ways. Very specialized techniques like X-ray diffraction, thermo- gravimetry, scanning or transmission electron microscopy, etc have been developed [2]. Mineralogical identification using these techniques requires specialized training and procedures. Often, combinations of techniques are necessary in order to make definite identifications.

The X-ray diffraction (XRD) technique is by far the most widely used, but is only appropriate for minerals with discrete crystallography.

Mineralogical test was also conducted at the Geological Survey of Ethiopia Mineralogy and Petrography Laboratory. The X-ray diffraction method was used following the powder method of sample preparation. Clay separation was not carried out, and thus it resulted in the mineralogical composition indicated in table 3.2.2/1 with unusually high percentage of silica.

Table 3.2.2/1 Percentage Mineralogical Composition

Sample Location	Minerals identified
D.D	Quartz = 48.6% Magnetite = 16.7% Goethite = 16% Anorthoclase = 13.5%

	Hematite= 5.2%
G.G	Quartz = 51.4% Kaolinite = 25.6% Hematite = 23%

According to Wesley's Mineralogical Classification, the soil samples from Gilgel Gibe and Dire Dam fit into group C subgroup c, that is, residual soil with a strong mineralogical influence deriving from special clay minerals only found in residual soils, namely sesquioxides. The aluminium oxide and iron oxide content of these soils is of a considerable magnitude and hence can influence the property of these spoils.

### 3.3 Dispersion (Erodibility) Test

Dispersive soils are soils in which the clay content has a high percentage of sodium. The clay fraction in dispersive soil readily breaks down to form a suspension in water.

Dispersive soils are known to form strong, tough impermeable clods. The compactive effort needed to break down such clods and appropriately compact the soil is very high.

If dispersive soils are used to form an embankment without compacting properly, air voids will result within the embankment. When water seeps in and leaches out the clay fraction, small pipes are formed and then can easily develop into tunnels, resulting in a breached embankment.

#### 3.3.1 Test Procedure

Dispersive test was conducted for the soil from Gilgel Gibe. To determine the dispersibility of this soil a simple test, which can be easily conducted, was done as follows:

100 ml demineralized water was added to a beaker of internal diameter 6.5cm. Then, five air-dried soil crumbles, each about 5mm diameter, were placed in the beaker and allowed to stand for one hour. Then, turbidity of the soil was observed.

#### 3.3.2 Results Obtained

According to the established methods of comparison, the soil was obtained to be slightly dispersive. During the site visit to Gilgel Gibe Hydroelectric Project, it was observed that the cofferdam was constructed from the soil material under investigation. It was also explained by the site engineers that one of the reasons why the main dam was not constructed from the same material was because of the dispersibility of the soil.

A number of irrigation dams that have been constructed with dispersive soils are functioning successfully. The important factor is employing the correct compaction procedure. Compaction equipments that can apply very huge compaction pressures (sheepsfoot roller) that can destroy the clod structure of the soil and produce a uniform density throughout the embankment are advised.

### 3.4 Compaction Tests

#### 3.4.1 General

Compaction is defined as the process of densifying soil and reducing air voids by applying mechanical energy. Compaction increases the strength, lowers the compressibility and reduces the permeability of a soil by rearranging its fabric [4]. The soil fabric is forced into a denser configuration by the mechanical effort used in compaction. Compaction is known to be the most popular technique of improving soils.

The basic concept of the densification process can be explained by the following equation

$$\gamma_d = \left( \frac{G_s}{1+e} \right) \gamma_w = \left( \frac{G_s}{1+wG_s/s} \right) \gamma_w \dots\dots\dots \text{Eq. (3.4.1/1)}$$

How can one increase the dry unit weight?

Examination of Eq. (3.4.1/1) reveals that one has to reduce the void ratio; that is, w/s must be reduced. The theoretical maximum dry unit weight is obtained when s = 1 because when s=1 the theoretical minimum void ratio is attained ( e<sub>min</sub> = wG<sub>s</sub>). But the soil remains invariably unsaturated at the maximum dry density, i.e., s<1 as it is impossible to remove void air all in all.

Free water (water that is not chemically bound by the clay minerals) has got a controlling influence on the ease with which a given degree of compaction is achieved. When the water content is low, the soil is stiff and hence difficult to compress thus having low dry density and high air contents. With increased moisture contents, the water acts as lubricants, making the soil more workable and resulting in closer packing (higher dry

density) of the soil grains and lower air contents upon adding more water to the soil. However, a stage is reached when water tends to keep the particles apart (because of pore pressure set up). At this stage there will be no appreciable decrease in air content. The total voids continue to increase with moisture content and hence the dry density of the soil falls. The moisture content at which the maximum dry density is achieved is called optimum moisture content. Eq. (3.4.1/1) can be used to plot series of theoretical curves of dry density versus water content for different degrees of saturation. The curve corresponding to  $s=1$  (100% saturation) is called the zero air voids curve (saturation line).

Although there are basic scientific principles involved in the compaction process, the means of achieving the desired degree of compaction involves a combination of technology & judgment, and the recognition that soil materials are inherently variable [2]. Efficient compaction is an art that is dependent on engineering skill and judgment. The field compaction product requires compromises between energy and cost expended. Engineering design must recognize the reality of what can be achieved in the field. The best field compaction conditions are mentioned to be determined by a test embankment using the selected construction materials [8]. If the bearing capacity of the material to be compacted is too low for the heavy compaction equipment, stage compaction is carried out starting with lighter equipment [14]. That means overstressing of the soil to be compacted should be avoided lest it should result in shear cracking of the embankment.

### 3.4.2 Test procedure

The Standard Proctor Test was used for the compaction. Soil specimens, some sun dried and others air-dried, passing the no 4 (4.75mm) sieve were soaked overnight. Then, they were compacted in a cylindrical mold of 944-cm<sup>3</sup> volume (Standard Proctor mold) by repeated blows from the mass of a hammer 2.5 kg falling freely from a height of 30.5 cm. The soils were compacted in three layers, each layer being subjected to 25 blows. Fresh specimens were used for each compaction moisture. Likewise, the soil from Gilgel Gibe was compacted at modified Proctor compaction also for further investigation. [For the detail procedures refer ASTM D698-91&D1557-91].

### 3.4.3 Results Obtained

To determine the maximum dry densities and the corresponding optimum moisture contents, the compaction test data summarized in table 3.3.3/1 has been used.

Table 3.4.3/1 Compaction Test Data (Standard Proctor)

Location of sample	Sample sun dried		Sample air dried	
	Moisture content (%)	Dry density (g/cc)	Moisture content (%)	Dry density (g/cc)

D.D	24	1.15	28.4	1.24
	25.6	1.2	32.2	1.32
	25.8	1.2	34.9	1.34
	26.0	1.21	35.8	1.34
	33.7	1.35	42.8	1.21
	38.7	1.28		
G.G	26.1	1.21	28.4	1.19
	29.3	1.28	33.7	1.31
	31.4	1.30	35.5	1.32
	35.6	1.34	35.6	1.32
	47.7	1.16	42.9	1.23
A.A	16.9	1.24	22.5	1.28
	20.2	1.27	26.2	1.35
	26.3	1.38	32.3	1.41
	28.0	1.4	34	1.39
	35.3	1.39	35.7	1.36
	44.1	1.22	40.7	1.26

The value of maximum dry density versus optimum moisture contents for the soils exposed to different drying conditions has been summarized in table 3.4.3/2.

Table 3.4.3/2 Summary of Compaction Test (Standard Proctor)

Location of sample	Sample sun dried		Sample air dried	
	Optimum moisture content (%)	Max dry density g/cc	Optimum moisture content (%)	Maximum dry density (g/cc)
D.D	34.4	1.350	35.5	1.340
G.G	34.4	1.344	35.4	1.320
A.A	32	1.415	32.5	1.41

Plots of dry density versus moisture contents, with the zero air void curves included are indicated in figure 3.4.3/1 for standard Proctor compaction.

The difference between dry densities of samples sun dried and air-dried obtained here is small. But if oven dried samples had been used for compaction, large differences could have been obtained at least for the highly laterized soil from Gilgel Gibe. Still for the same moisture content, the samples sun dried attained somewhat higher values of dry densities as compared to air-dried samples. This condition is a bit clearly seen for the soil from Gilgel Gibe.

The compaction curves for standard Proctor are indicated below in fig. 3.4.3/1a, b & c

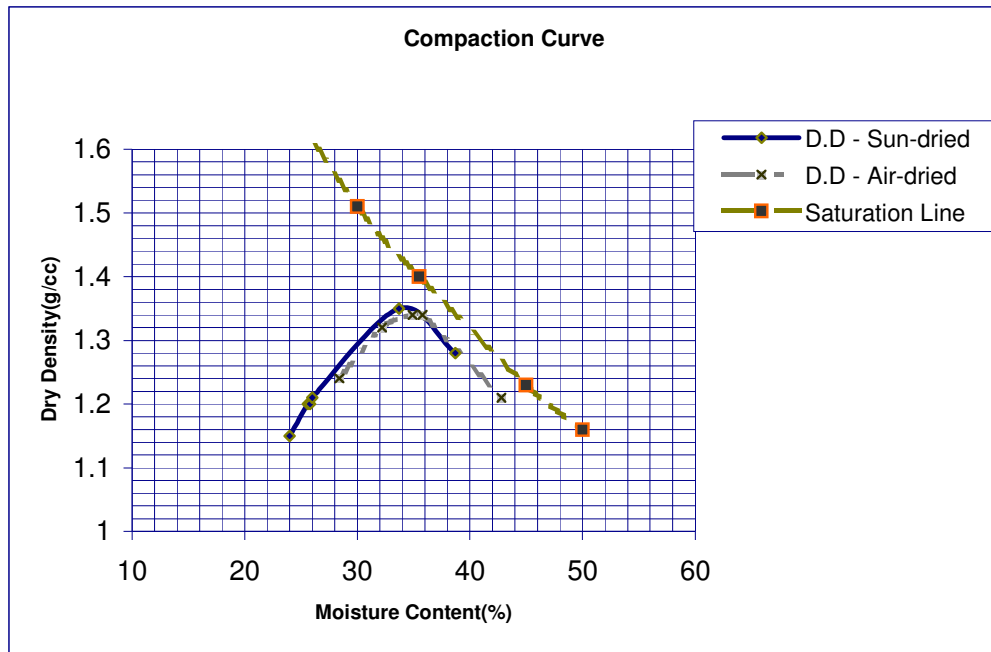


Fig.3.4.3/1a) Sample D.D

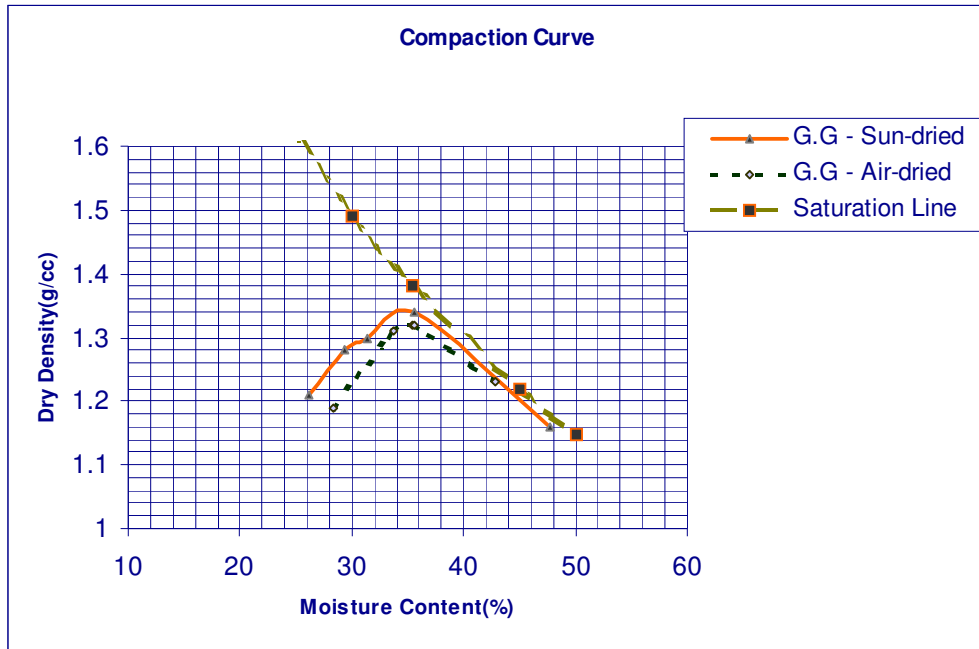


Fig.3.4.3/1b) Sample G.G

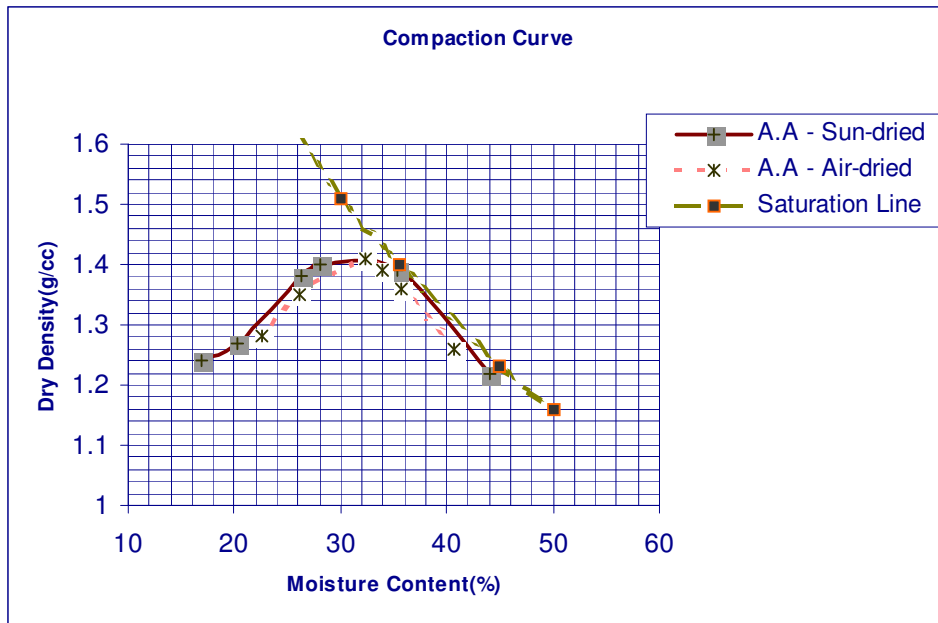


Fig.3.4.3/1c) Sample A.A

Fig.3.4.3/1 Compaction Curves (Standard Proctor)

The test data for the modified Proctor Compaction of the soil from Gilgel Gibe is given in table 3.4.3/3 and the combined compaction curves of standard Proctor & modified Proctor are indicated in fig.3.4.3/2. The maximum dry density in the case of

modified Proctor Compaction is 1.468 g/cc with the corresponding optimum moisture content of 30%.

Table 3.4.3/3 Compaction Test Data (Modified Proctor)

Location of Sample	Moisture Content (%)	Dry density (g/cc)
G.G	23	1.367
	26.6	1.422
	30	1.468
	33.8	1.378
	37.6	1.312

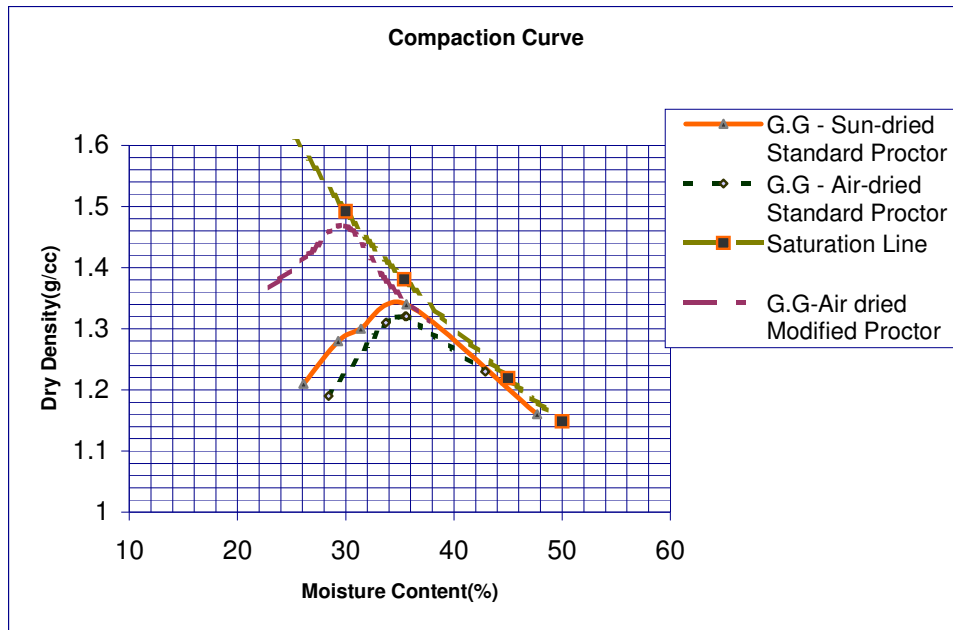


Fig.3.4.3/2 Sample G.G-Combined Compaction Curve (Stand. &Mod. Proctor)

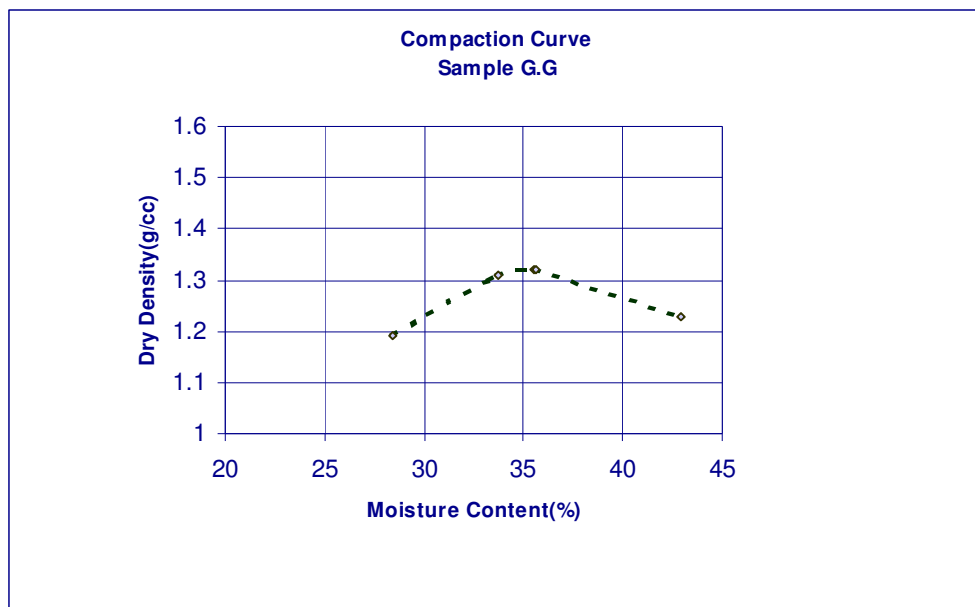
### 3.5 Permeability of Compacted Soil

Permeability test has been done for the soil from Gilgel Gibe after compacting the soil (standard Proctor) at varying moisture contents. The test result has indicated that the soil compacted on the drier and wetter sides of the optimum moisture contents are more

permeable than the soil compacted at optimum moisture content, the test being carried out after saturating the samples. The permeability coefficient for wet compaction has been obtained to be slightly lower than that for dry compaction. Similar results have been obtained by R.O. Lucas [8].

Table 3.5/1 Coefficients of Permeability

Moisture Content (%)	Average Coeff.of Permeability. (cm/sec)
32.4	$1.07 \times 10^{-6}$
35.4	$1.67 \times 10^{-7}$
39.4	$9.00 \times 10^{-7}$



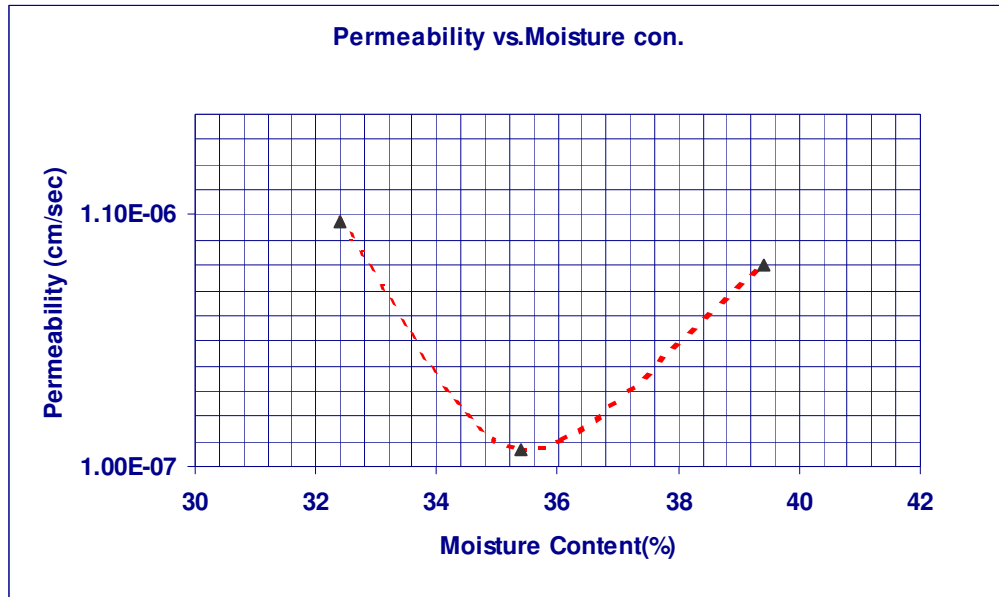


Fig. 3.5/1 Variation of Coefficient of Permeability wrt Compaction moisture

### 3.6 Shear Strength of Compacted Soils

#### 3.6.1 General

The shear strength characteristics were investigated for the soils at as compacted moisture contents and with the compacted sample soaked (saturated) with backpressure in the triaxial cell. The shear strength tests were carried out at the optimum moisture content, at moisture contents on the drier and wetter sides of the optimum moisture content. The investigation was carried out using the unconsolidated undrained triaxial test.

This investigation was carried out in order to determine the characteristics and variation of the shear strength parameters with respect to the compaction moisture content, soil type and state of compaction. The shear strength parameters thus obtained are to be used in the stability analysis for earth fill dams.

#### 3.6.2 Test Procedure

Test specimens were prepared by compacting soil samples in the Proctor mold as per the Standard and Modified (in the case of the soil from Gilgel Gibe) Proctor tests with samples soaked to attain the moisture contents at which the shear strength tests were to be carried out. After compacting the soil in the Proctor mold, specimens were prepared for the triaxial shear tests by hydraulically pressing the samplers into the compacted soils, which were in the Proctor molds. Specimens of 76 mm height and 38 mm diameter were used in the triaxial tests.

The triaxial shear test was strain- controlled. For the values of the failure axial stresses, either the peak axial stress or the axial stress corresponding to the twenty percent axial strain was taken, depending on which ever occurred first.

### 3.6.3 Test Results and Analysis

The computed values of the undrained shear strength parameters,  $C_u$  &  $\Phi$ , are tabulated in table 3.6.3/1. The stress – strain curves and plots of stress paths are also indicated.

Table 3.6.3/1 Shear Strength Test Result

Location of sample	Dry density (g/cc)	Moisture Content (%)	Degree of Saturation (%)	$C_u$ (kPa)	$\phi$ (Degree)
<b>D.D</b>	<b>1.32</b>	<b>32.5</b>	<b>82.5</b>	<b>102</b>	<b>7.9</b>
	<b>1.34</b>	<b>35.5</b>	<b>92.8</b>	<b>72</b>	<b>3.3</b>
	<b>1.28</b>	<b>39.5</b>	<b>94.6</b>	<b>69.8</b>	<b>0</b>
<b>G.G</b> Standard Proctor	<b>1.28</b>	<b>32.4</b>	<b>78.9</b>	<b>190.6</b>	<b>15.9</b>
	<b>1.32</b>	<b>35.4</b>	<b>91.4</b>	<b>174</b>	<b>8.3</b>
	<b>1.27</b>	<b>39.4</b>	<b>94.5</b>	<b>127.1</b>	<b>0</b>
<b>G.G</b> Standard Proctor (Saturated)	<b>1.28</b>	<b>32.4*</b>	95.8	88	8.1
<b>G.G</b> Modified Proctor	<b>1.43</b>	<b>27</b>	<b>82.1</b>	<b>475</b>	<b>18.9</b>
	<b>1.468</b>	30	<b>96.5</b>	256	16.5
	<b>1.42</b>	33	<b>98.8</b>	168	7.2

A.A	1.36	30	81	110.5	13.6
	1.41	33	95.8	86.4	6
	1.34	37	97.1	81.5	0

32.4\* = as compacted moisture content

The values obtained from the triaxial tests were analyzed with respect to the effects of compaction moisture content, sample saturation, soil type and state of compaction.

### 3.6.3.1 Effects of Moisture content, Soil Type & State of Compaction

In general, it has been observed that the shear strength parameters,  $c$  &  $\phi$  have decreased when the compaction moisture content has been increased. The decrease in cohesion ranged from 26% to 65%. The smallest change in cohesion occurred to the soil from Addis Ababa (Semen Gebeya) whereas the largest occurred to the soil from Gilgel Gibe. For the triaxial  $uu$  –tests carried out using samples compacted at standard Proctor, all soils lost the angle of internal friction on the wetter side of the optimum moisture.

At 3% drier moisture content (32.4%) the soil from Gilgel Gibe was tested at both as compacted and saturated moisture contents. The soil lost much of its shear strength when tested after sample saturation. The decrease in cohesion was about 54% and that of angle of internal friction was about 49%. Loss of suction upon saturation and the effect of saturation softening of the sesquioxides may be factors contributing to the decrease.

The soil from Gilgel Gibe registered larger shear strength as compared to the other soils, the comparison being made with soils compacted under the same state of compaction (compactive effort). The increased stiffness in this soil may be attributed to the bracing (cementing) effect between the quartz grains and the sesquioxides. The presence of goethite or haematite as discrete particles in the soil matrix can enhance the formation of cemented particles and thereby increasing the strength of the soil.

The sample from Gilgel Gibe was compacted at modified Proctor also and triaxial tests were carried out to see the effect of state of compaction (change in compactive efforts). The shear strength parameters,  $c$  &  $\phi$ , increased remarkably as compared to the corresponding values at standard Proctor compaction.

### 3.6.3.2 Axial Stress vs. Axial Strain relationships

In most of the cases, for specimens tested at moisture contents drier of the optimum moisture contents the soils attained large deviatoric stress values at small strain values as compared to the corresponding values obtained for wetter samples as indicated in fig.3.6.3.2/1. As the moisture content increased further, the peak stresses were lost and strains occurred at almost constant (and smaller) stresses. Thus, at drier moisture contents, maximum stresses determined the failure values (points) where as at wetter moisture contents maximum strains determined the failure values.

The comparison of the three-soil sample showed that the soil from Gilgel Gibe (specimen G.G.) attained very large axial stress as compared to the other soils. As explained previously in section 3.6.3.1 the increased stiffness in this soil can be attributed to the bracing (cementing) effect between the quartz grains and the sesquioxides. The presence of goethite / haematite as discrete particles in the soil matrix can enhance the formation of cemented particles and thereby increasing the strength of the soil. Besides, the soil from Gilgel Gibe exhibited the property of brittle failure when compacted at higher compactive effort, especially for dry compaction.

Another point observed from plots of stress vs. strain curves is the effect of confining pressure. Given specimens of the same soil tested at the same moisture content, the one exposed to higher confining pressure ( $\sigma_3$ ) is plotted above the one exposed to smaller confining pressure.

The Deviatoric stress vs. axial strain curves are indicated in fig. 3.6.3.2/1 a - m

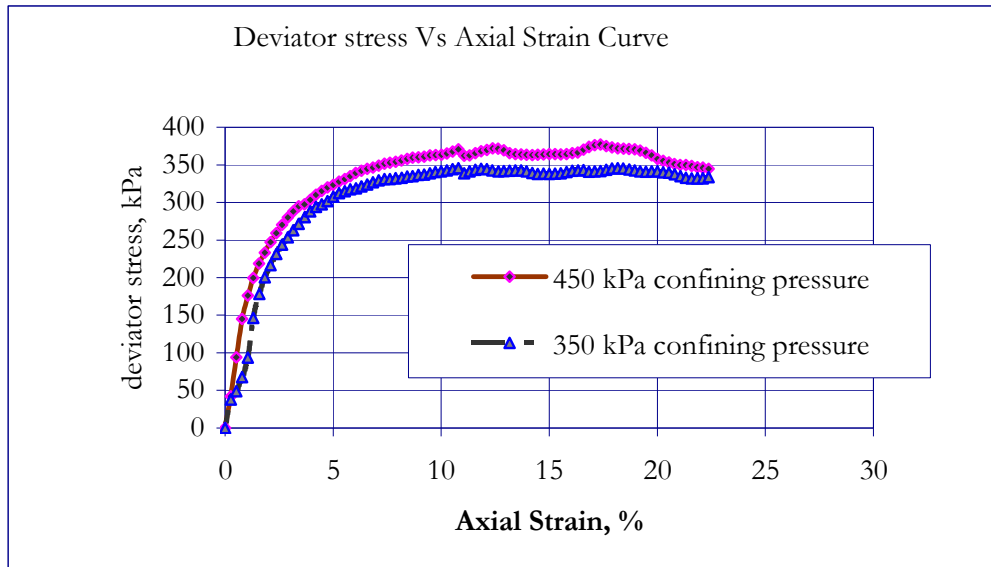


Fig.3.6.3.2/1a Sample D.D –at 32.5% moisture content

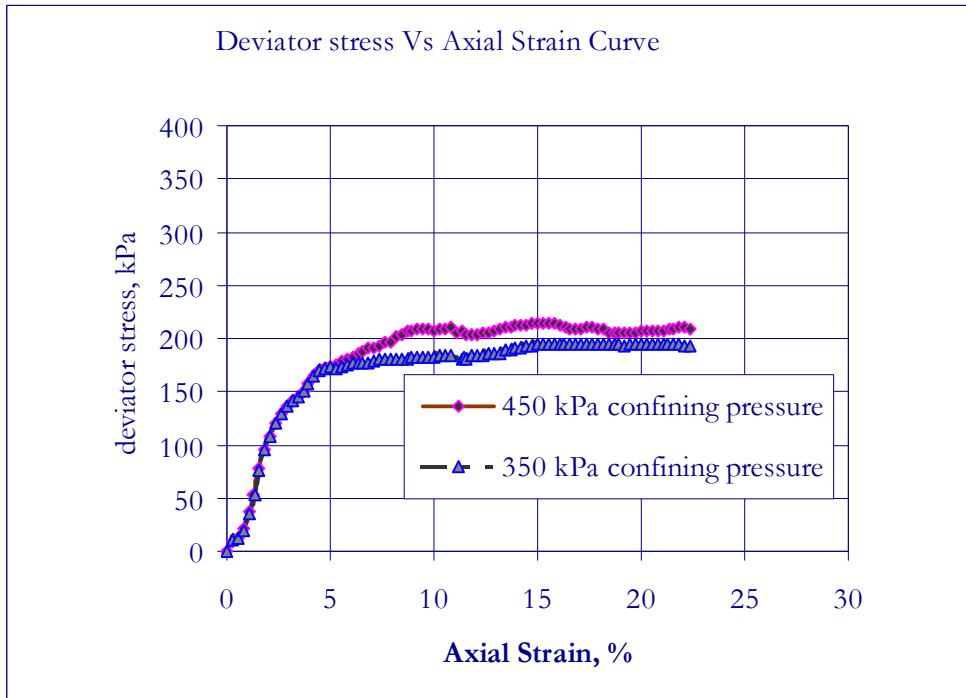


Fig.3.6.3.2/1 b Sample D.D –at 35.5% moisture content

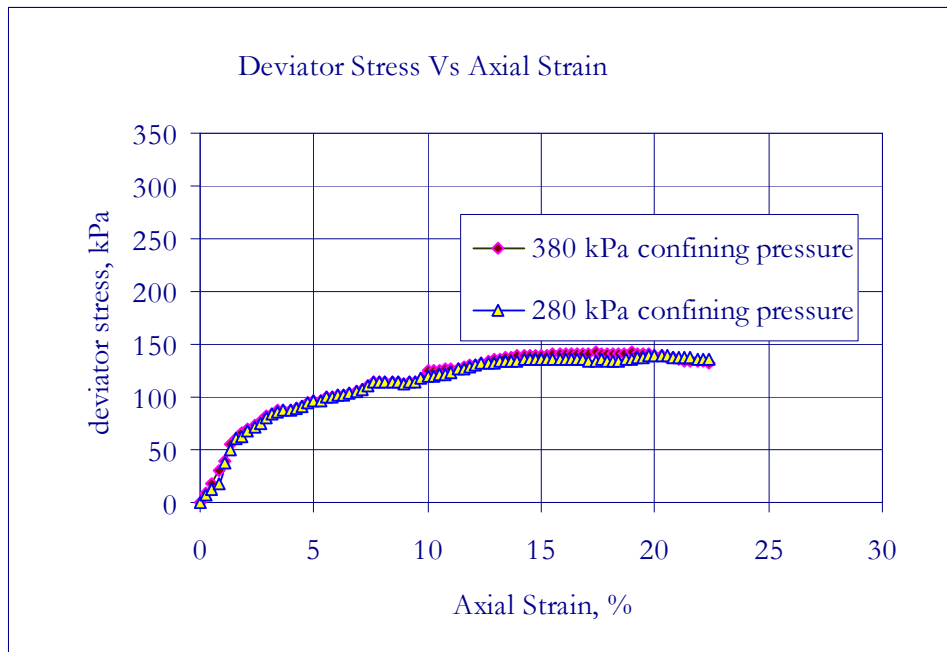


Fig.3.6.3.2/1c Sample D.D –at 39.5% moisture content

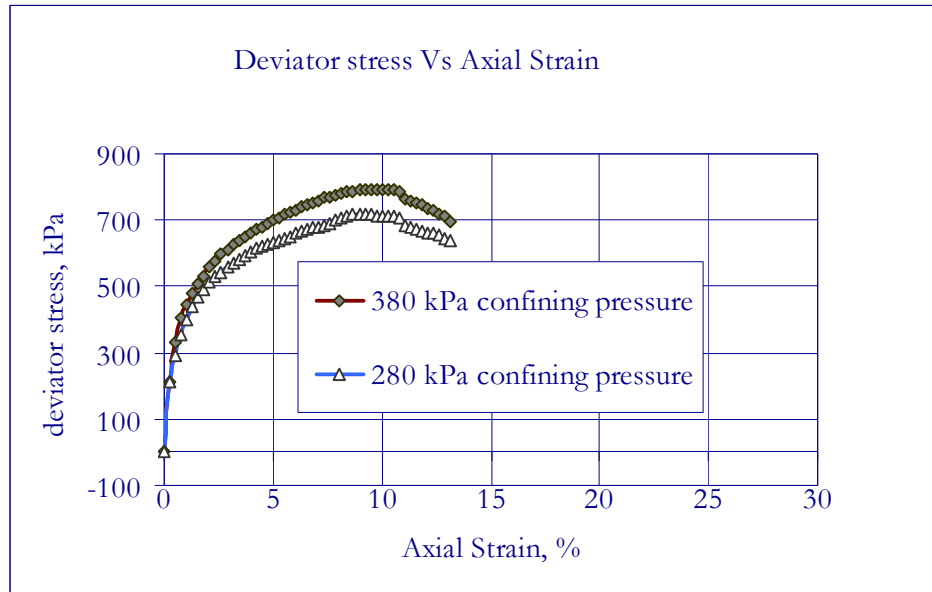


Fig.3.6.3.2/1d Sample G.G –at 32.4% moisture content

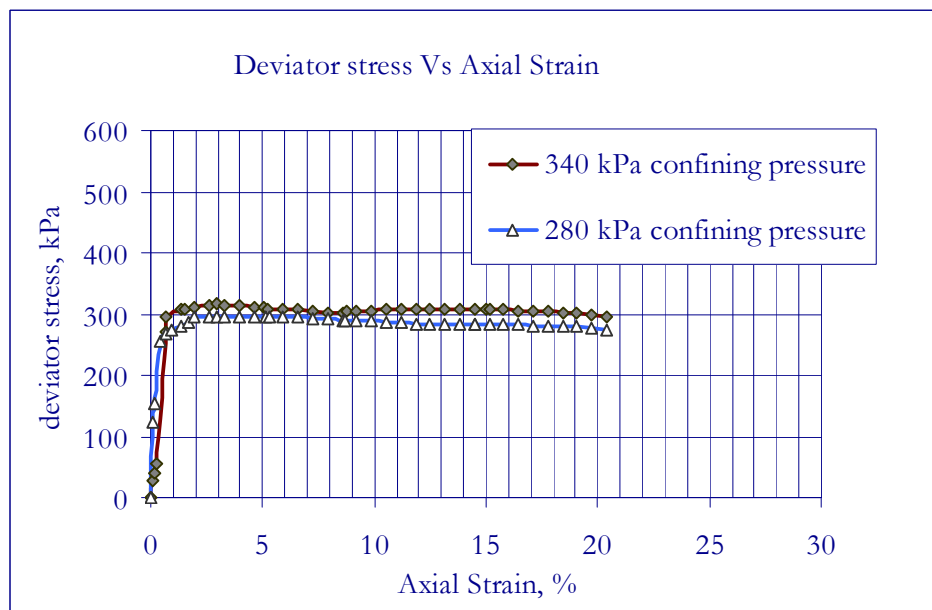


Fig.3.6.3.2/1e Sample G.G –at 32.4% moisture content (Sample Saturated)

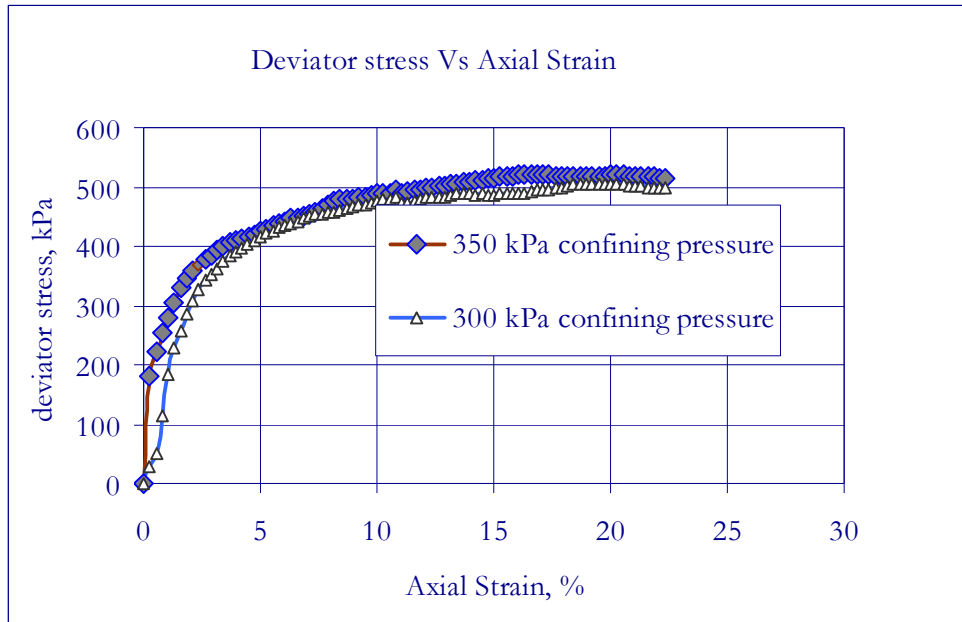


Fig.3.6.3.2/1f Sample G.G –at 35.4% moisture content

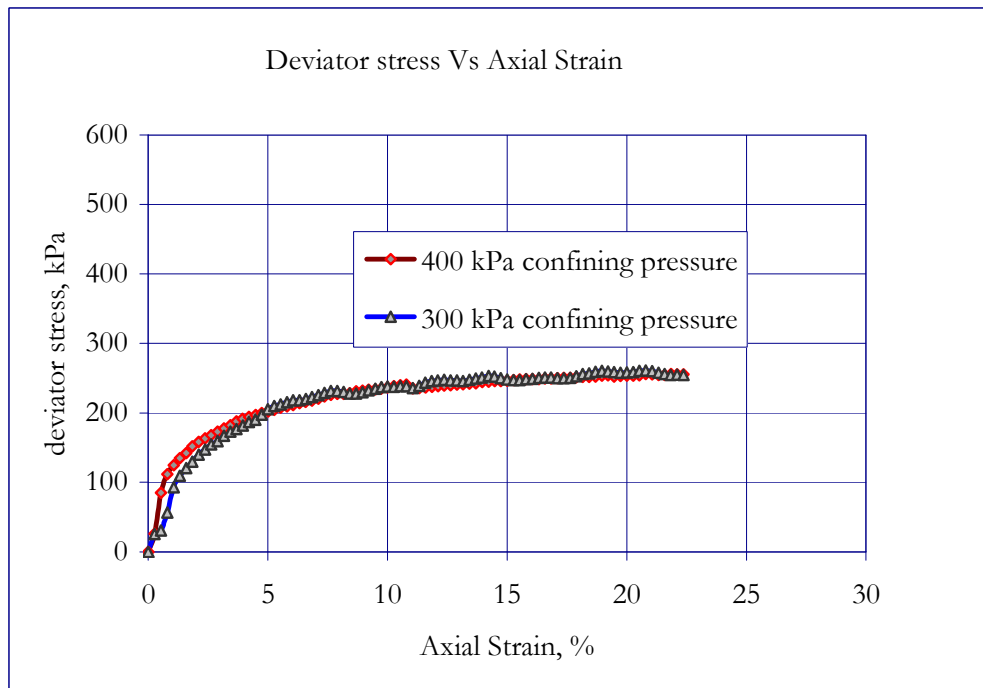


Fig.3.6.3.2/1g Sample G.G –at 39.4% moisture content

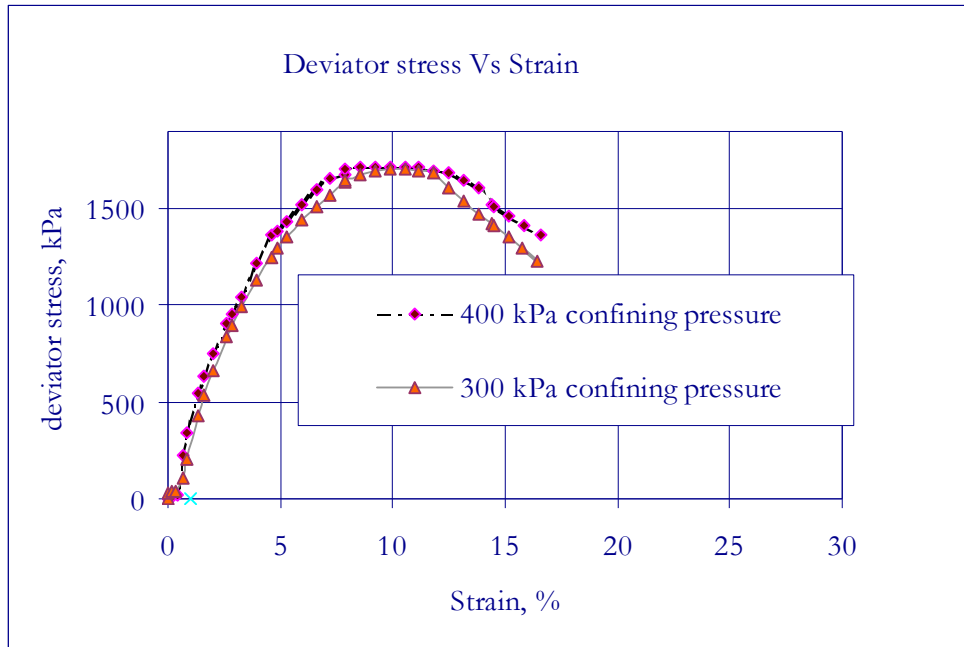


Fig.3.6.3.2/1h Sample G.G – at 27% moisture content (Modified Proctor)

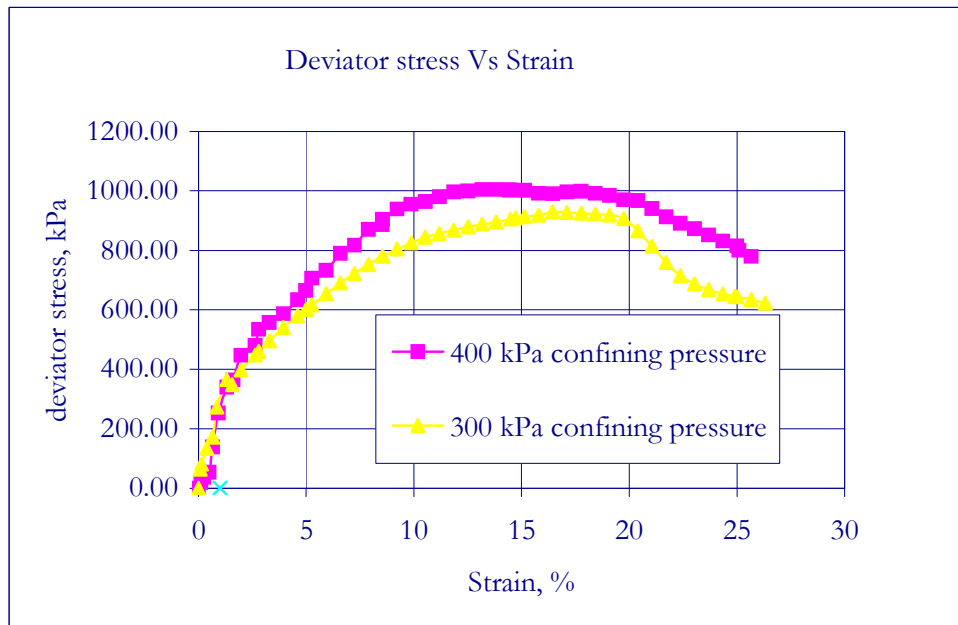


Fig.3.6.3.2/1i Sample G.G – at 30 % moisture content (Modified Proctor)

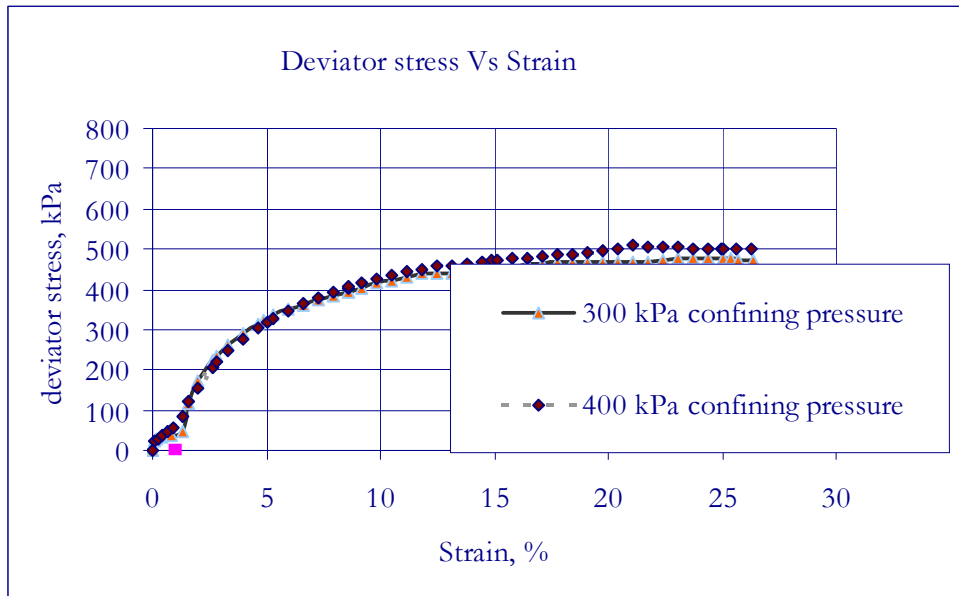


Fig.3.6.3.2/1j Sample G.G – at 33 % moisture content (Modified Proctor)

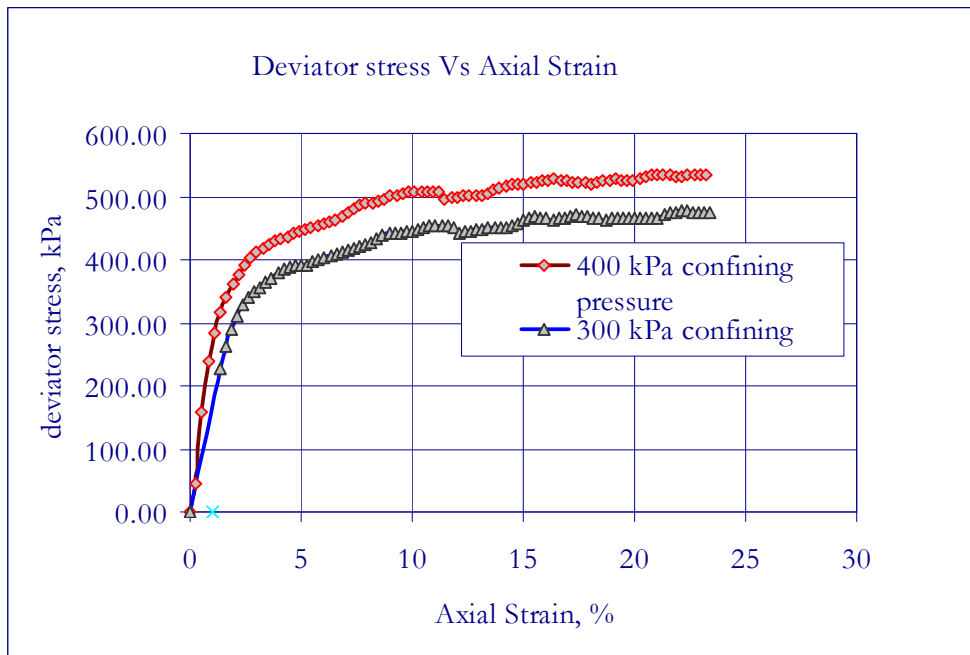


Fig.3.6.3.2/1k Sample A.A –at 30 % moisture content

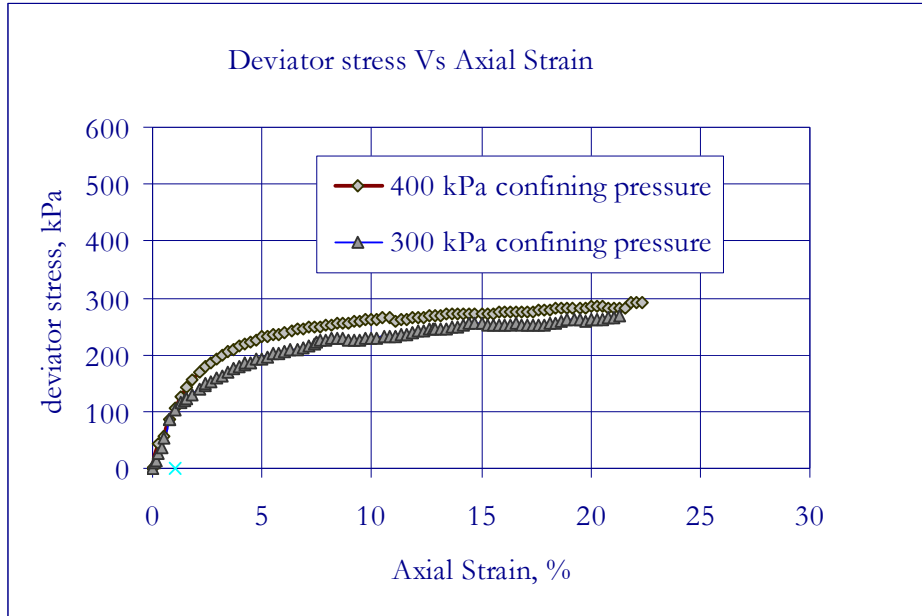


Fig.3.6.3.2/1 l Sample A.A –at 33 % moisture content

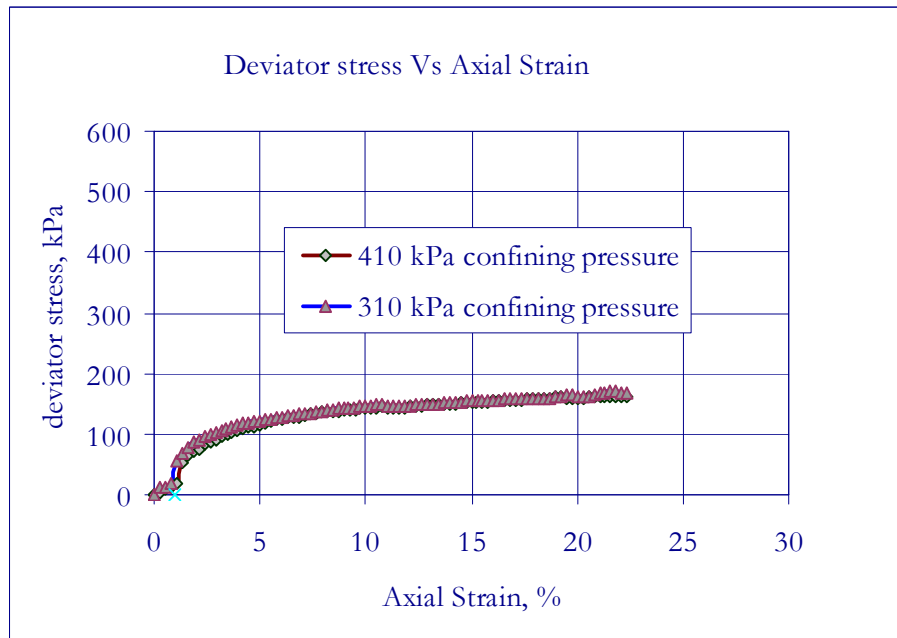


Fig.3.6.3.2/1 m Sample A.A –at 36 % moisture content  
 Fig. 3.4.3.2/1 Deviatoric Stress Vs. Axial Strain Curves

### 3.6.3.3 Stress Paths

The stress path, graphical representation of stresses in stress space, is one of the methods through which triaxial test results can be depicted. One convenient and most frequently used stress path is the plot of deviatoric stress ( $q$ ) versus mean effective stress ( $P'$ ) or mean total stress ( $p$ ).

$$\text{Deviatoric stress, } q = \sigma_1 - \sigma_3$$

$$\text{Mean total stress, } p = (\sigma_1 + \sigma_2 + \sigma_3) / 3$$

$$\text{For axisymmetric stress condition, } \sigma_2 = \sigma_3$$

Thus,

$$P = (\sigma_1 + 2\sigma_3) / 3$$

For unconsolidated undrained tests, plots of  $q$  vs.  $p$  (total stress path) results in 3:1 ratio of  $\Delta q / \Delta p$  as discussed below.

Isotropic Compression phase:

$$\Delta \delta_1 = \Delta \delta_3, \Delta u \neq 0$$

$$\Delta p = [\Delta \delta_1 + 2\Delta \delta_3] / 3 = (3\Delta \delta_1) / 3 = \Delta \delta_1$$

$$\Delta q = \Delta \delta_1 - \Delta \delta_3 = 0$$

$$\text{Hence, } \Delta q / \Delta p = 0$$

Shearing Phase:

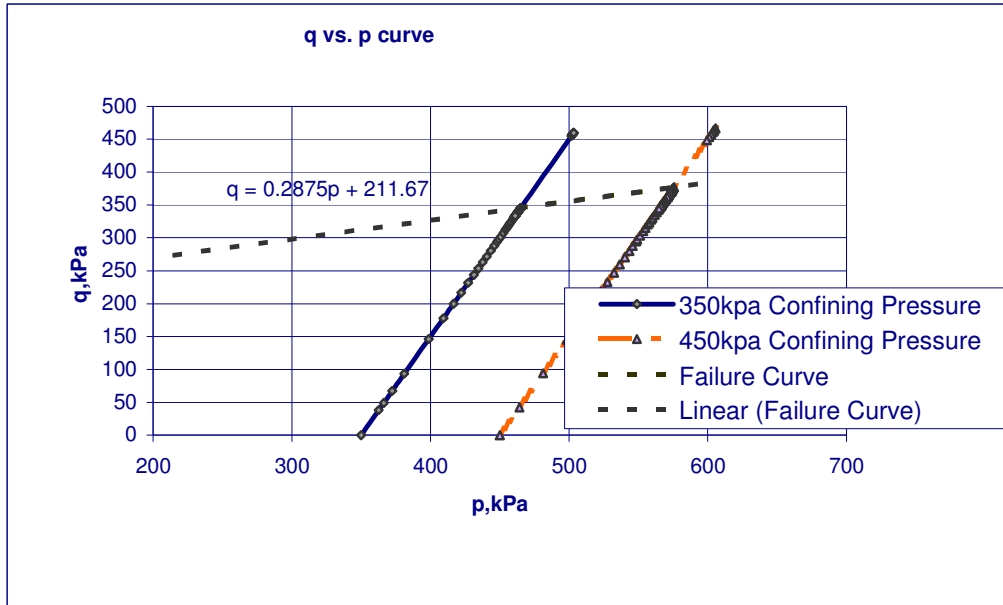
$$\Delta \delta_1 > 0, \Delta \delta_3 = 0$$

$$\Delta p = [\Delta \delta_1 + 2\Delta \delta_3] / 3 = \Delta \delta_1 / 3$$

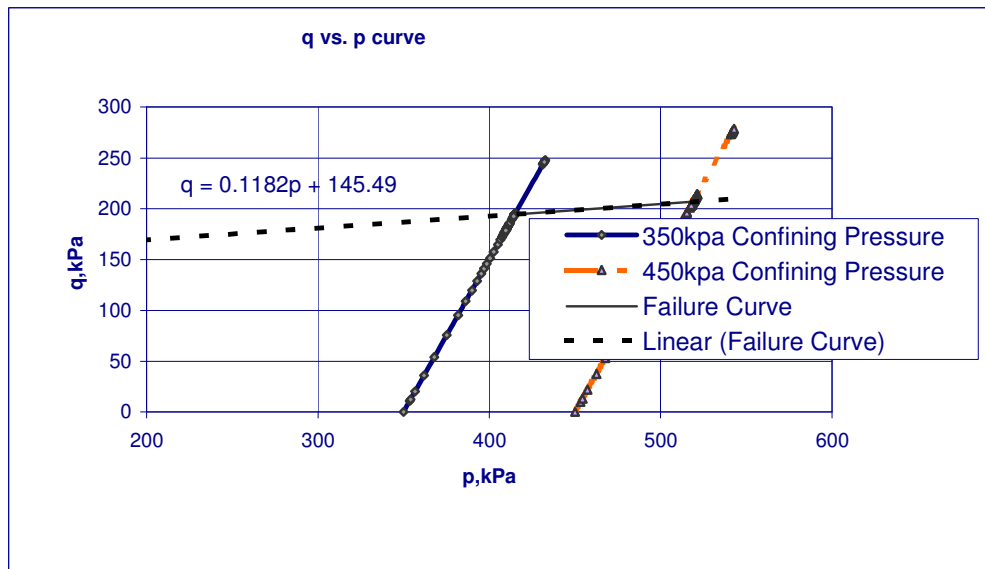
$$\Delta q = \Delta \delta_1 - \Delta \delta_3 = \Delta \delta_1$$

$$\text{Hence, } \Delta q / \Delta p = \Delta \delta_1 / (\Delta \delta_1 / 3) = 3 : 1$$

Plots of  $q$  vs.  $p$  are indicated in fig. 3.6.3.3/1. From the plots it is seen that for all the unconsolidated undrained tests conducted, the ratio of  $\Delta q / \Delta p = 3:1$ , and hence acceptable.

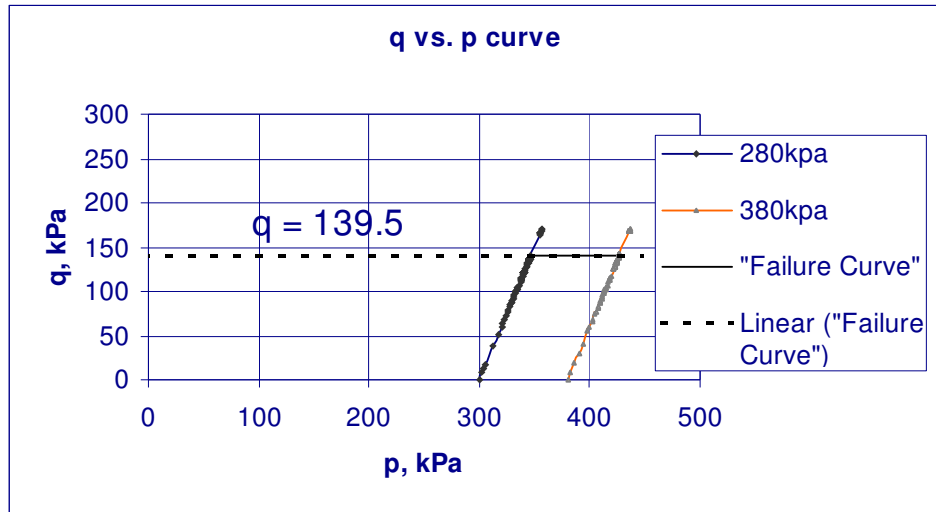


a) Sample D.D –at 32.5 % moisture content

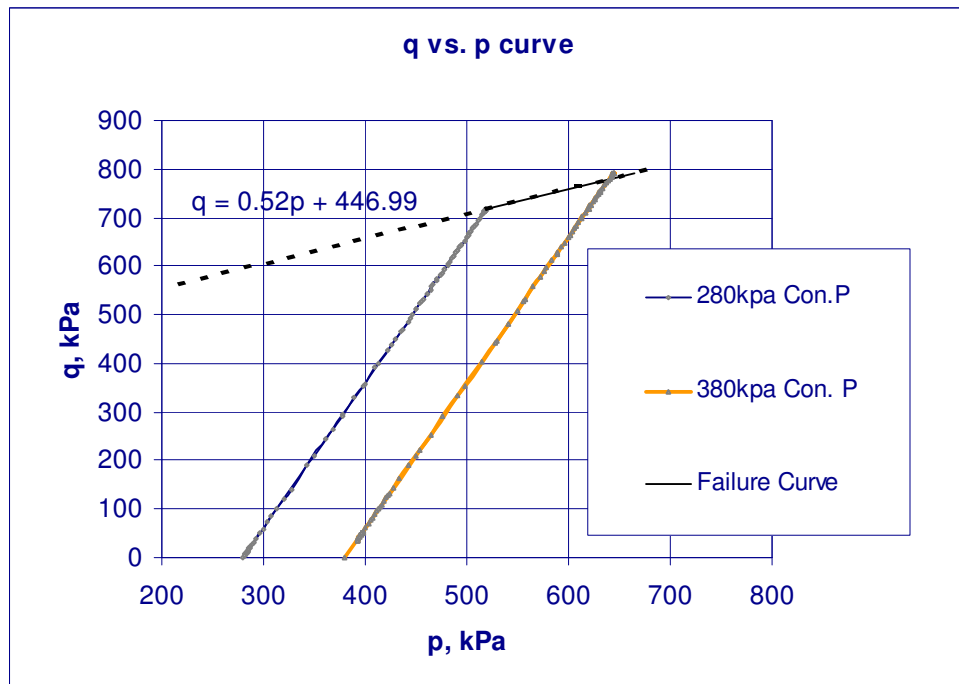


b) Sample D.D –at 35.5 % moisture content

Fig.3.6.3.3/1 Stress Paths

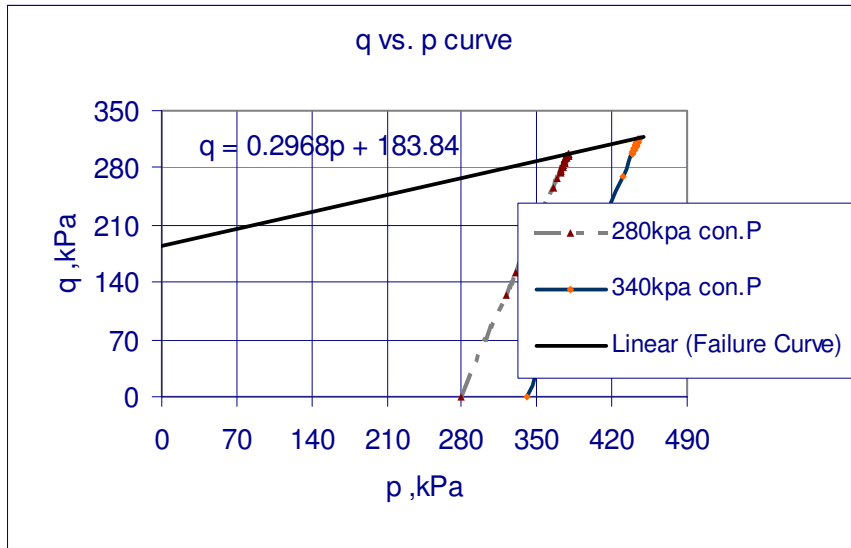


c) Sample D.D –at 39.5 % moisture content

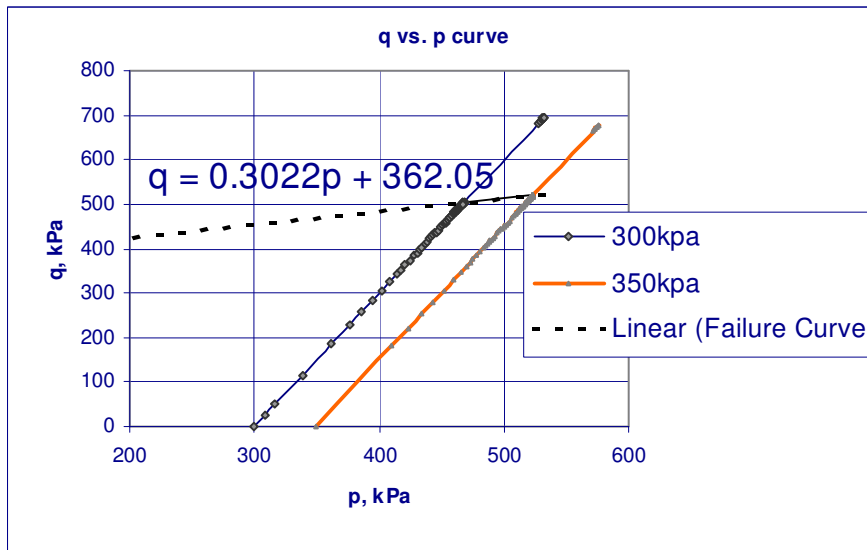


d) Sample G.G –at 32.4 % moisture content

Fig.3.6.3.3/1 Stress Paths .....continued.

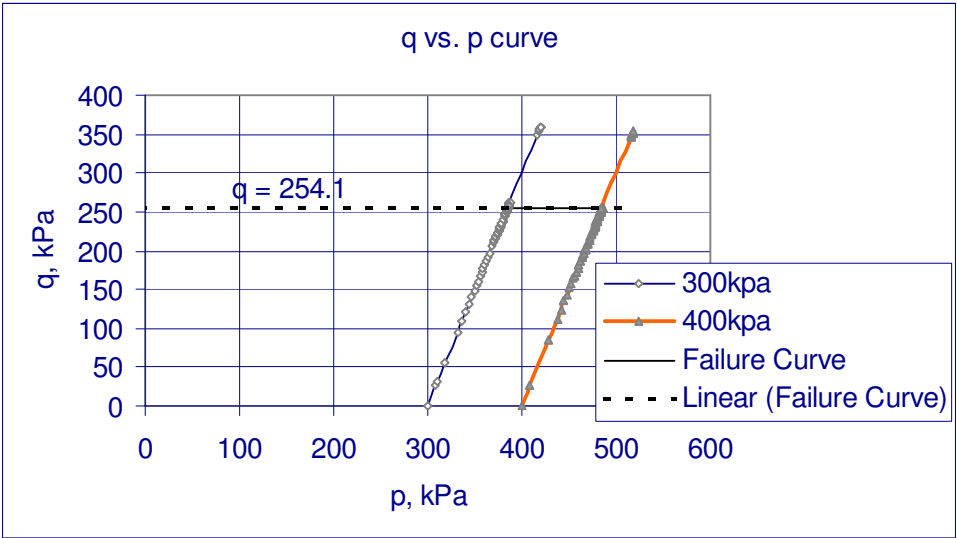


e) Sample G.G –at 32.4 % moisture content (Sample saturated)

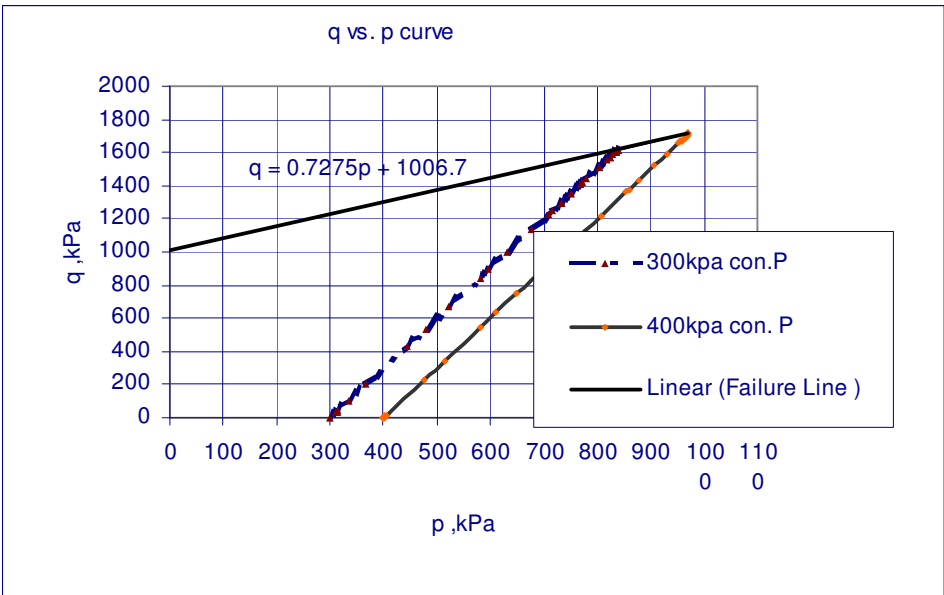


f) Sample G.G –at 35.4 % moisture content

Fig.3.6.3.3/1 Stress Paths .....continued.

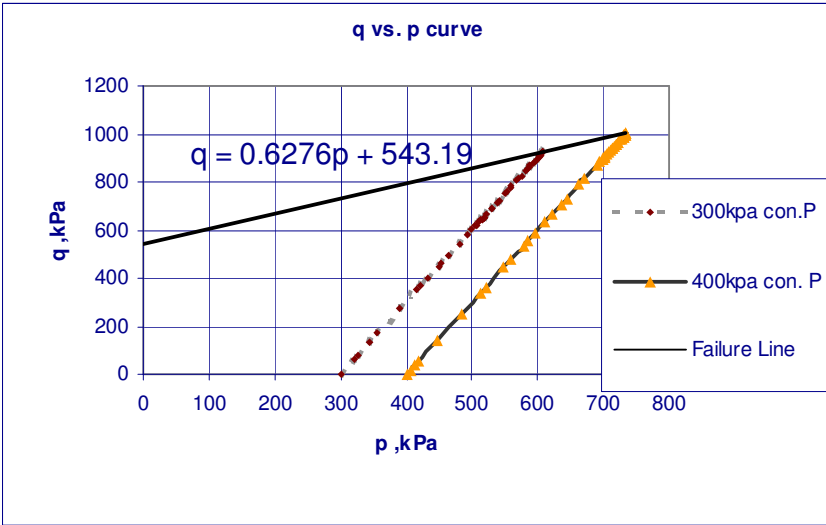


g) Sample G.G –at 39.4 % moisture content

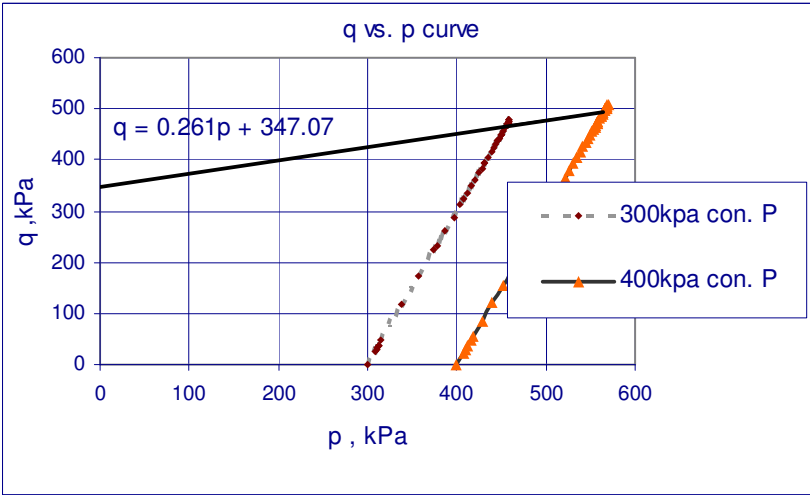


h) Sample G.G –at 27 % moisture content (Modified Proctor)

Fig.3.6.3.3/1 Stress Paths .....continued.

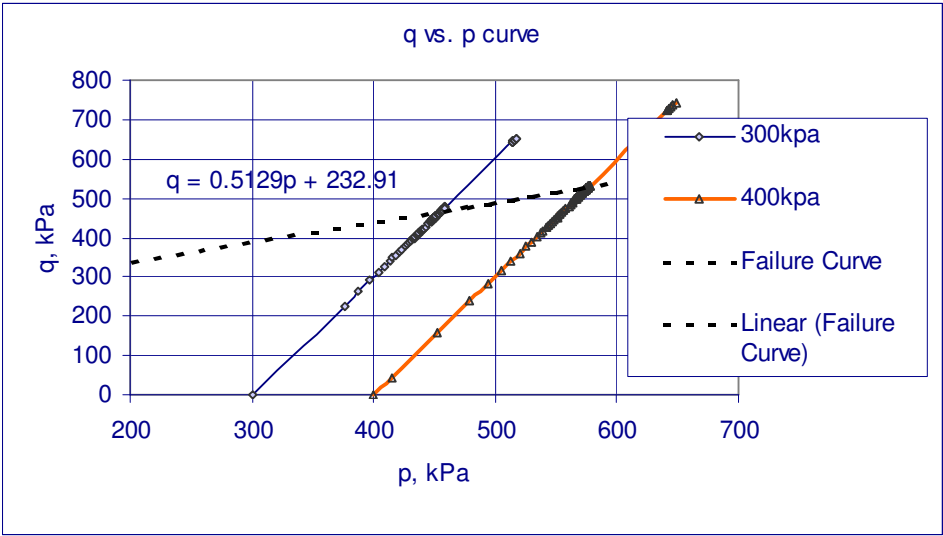


i) Sample G.G –at 30 % moisture content (Modified Proctor)

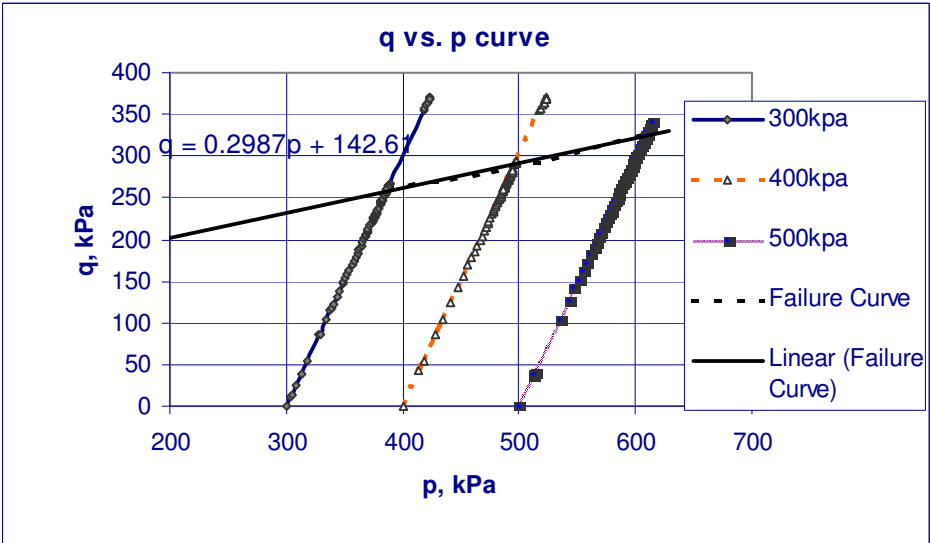


j) Sample G.G –at 33 % moisture content (Modified Proctor)

Fig.3.6.3.3/1 Stress Paths .....continued.

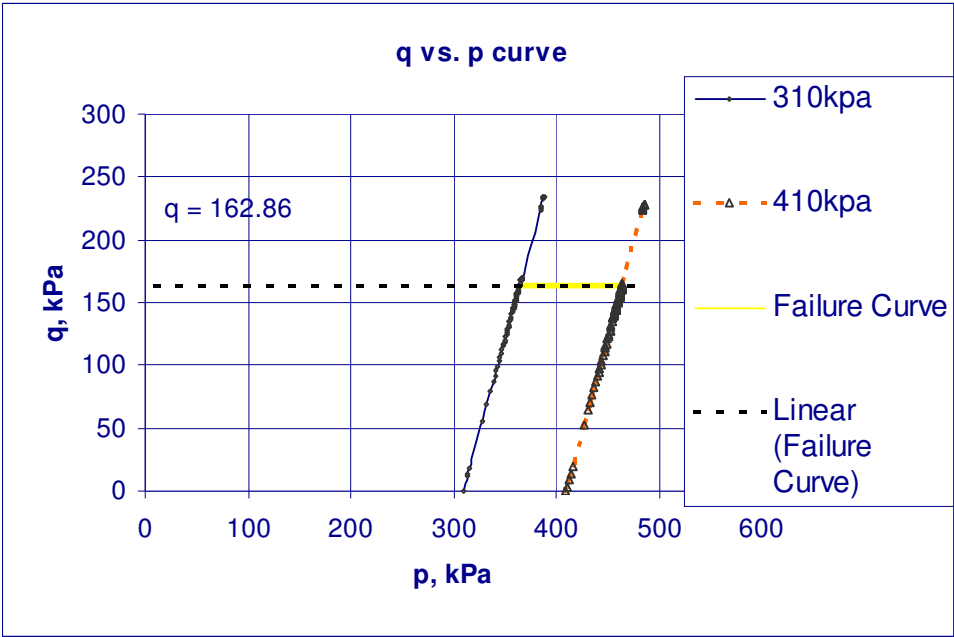


k) Sample A.A –at 30 % moisture content



l) Sample A.A –at 33 % moisture content

Fig.3.6.3.3/1 Stress Paths .....continued.



m) Sample A.A –at 36 % moisture content

Fig.3.6.3.3/1 Stress Paths .....continued.

### 3.7 Saturation Settlement

When a soil is compacted and then saturated with water, it may swell or settle without the influence of external forces [8]. Saturation settlement is known to induce cracks and arching. These effects are known to initiate dam failures. Whether a soil expands or settles depends on the soil type, compaction moisture content, and degree of compaction and overburden pressure.

#### 3.7.1 Test Procedure

The effect of saturation was studied on the soil from Gilgel Gibe by compacting a soil specimen of 20 cm depth and 3.55 cm diameter in the consolidometer mold at selected moisture contents and standard Proctor compactive effort. A consolidating load of 300kPa was placed on the specimens to simulate the surcharge load on a model dam of 15m height. Settlement readings were recorded until equilibrium was maintained. Then, the specimen was flooded with water and the resulting displacements were recorded.

#### 3.7.2 Test Results and Analysis

Summary of the maximum settlements that occurred after flooding the specimens, corresponding to the various compaction moistures, is given in table 3.7.2/1

Table 3.7.2/1 Summary of Saturation Settlement

Compaction Moisture (%)	32.4	35.4	37.3
Saturation Settlement (mm)	0.655	0.007	0.01
Final Moisture (%)	37.5	35.4	37.5

The test result indicated that saturation settlement occurred for the soil that was compacted on the drier side of the optimum moisture. At the optimum moisture and on the wetter side of it very small (negligible) settlement occurred. The saturation settlement obtained for the sample compacted on the drier side of optimum moisture is also rated to be small. It so happened because of large pre- saturation compression or settlement under a large

pressure (as large as  $300\text{kN/m}^2$ ). If the applied pre-saturation pressure were smaller, larger value of saturation settlement would be obtained. Fig. 3.7.2/1 shows variation of saturation settlement with respect to square root of time.

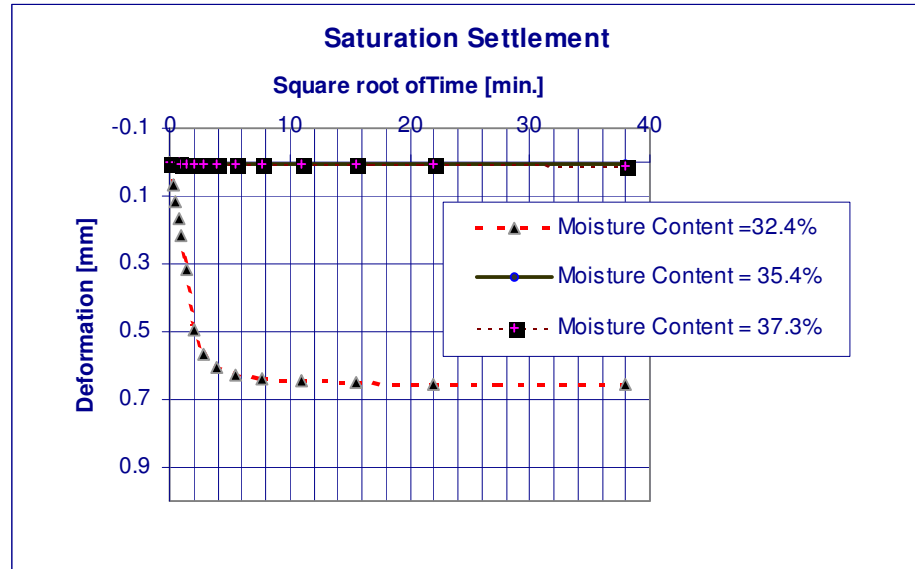


Fig.3.7.2/1 Saturation Settlement at Varying Moistures

## Unit 4. Stability Analysis

### 4.1 General

Geotechnical and shear strength parameters of the soils investigated have been determined as indicated in the previous sections. The shear strength parameters have been determined by varying conditions under which the soils were compacted. Now, making use of these shear strength parameters, the ultimate purpose of the work is to investigate the influence of compaction on the stability of earth fill dams.

Stability analysis is carried out in order to determine the factor of safety of a potential (shear) failure surface. The factor of safety is defined as the ratio of the resisting force or moment to the driving force or moment. The computations for the factor of safety should be based on the most unfavorable condition under which the tests for the determination of the material properties (parameters) are to be carried out. The greatest uncertainties in stability problems arise in the selection of pore pressure and strength parameters.

The selection of minimum factor of safety depends on various factors. The assumptions made concerning soil strength parameters, pore pressures, method of analysis used, the consequences of failures, construction quality control and economics are the governing factors. Different agencies suggest minimum values within the range of 1.2 to 1.5 depending upon the risks involved [8].

Slope Stability problems are usually assumed to be two-dimensional. Computations are made for that part of the length of the dam where a slide would be most likely to develop. The procedure is repeated until the critical potential failure surface is found which gives a minimum factor of safety.

Earth dam stability analysis requires knowledge of appropriate shear strength parameters of the soil comprising the embankment and the foundation. Two basically different approaches to the stability problem are in common use [14]. They are the effective stress analysis (ESA) and the total stress analysis (TSA).

There are three periods in the life of a dam that may be critical from stability point of view

- Construction phase
- Full reservoir condition
- Rapid drawdown condition

At each of these stages, the mechanics of failure is different and slightly different analytic techniques are used.

### **Construction Phase (End of Construction) Stability Condition**

It is often mentioned that slides of rolled earth dams during construction have occurred as frequently as slides during the operation of the reservoir. However, such slides have not resulted in failures that are catastrophic. Nevertheless, the pore pressures that develop in the embankment or foundation during construction may be higher than at any other subsequent time [14]. As a result of this, it is usually advisable to analyze the stability of the embankments for construction phase conditions. The construction phase condition is mentioned to be especially likely to be critical for dams on soft foundations.

The construction phase stability may be analyzed by the total stress method. When this method is used, the results of the undrained shear tests on unsaturated samples are used to determine the shear strength (resistance) of the embankment. The results of the laboratory tests are greatly influenced by the water content at which the samples are compacted. In addition to this, it is also difficult to estimate exactly the average water content that may be obtained in the field during construction. Therefore, several sets of laboratory tests on samples compacted at different water contents should be performed and average as well as conservative estimates of the shear strength should be used in the analysis.

In this work the unconsolidated undrained triaxial tests were conducted, mostly at as compacted moisture contents, and hence shear strength parameters that were pertinent for construction phase stability analysis were established. The shear strength parameters obtained largely depended on compaction moisture, state of compaction and soil type.

In analyzing the stability for construction phase through the effective stress method of analysis, the important factor is estimation of pore water pressure. Even though there are laboratory test procedures, which can be used to predict the pore water pressures that develop during construction, still it is not possible to predict reliably (precisely) in the design stage what these pressures might be in a given dam. It is indicated that the reason

for this is that the construction pore pressures depend on the amount of construction which will be done during the wet and dry weather, the influence of drying in dry weather, and the effectiveness of moisture and density control during construction. As a result, it is customary to take construction pore pressures conservatively based on past experience.

### **Full Reservoir Stability**

Stability failure (shear slide) when the reservoir is full can lead to disastrous effect. As a result, the stability analysis for full reservoir condition should be treated more carefully and conservatively than during construction and rapid dropdown conditions [14]. It is indicated that for the full reservoir state, only the down stream portion of the dam need to be analyzed. The upstream slope slide during full reservoir is assumed to arise only if the strength of the foundation were to be reduced very greatly by wetting.

The pore pressures which exist within an embankment at any given time under the full reservoir condition are generated as a result of two actions which can be considered to be independent for practical purposes [14]. They are gravity seepage flow and changes in pore volume due to changes in the total stress.

It is indicated that the full reservoir stability condition is nearly always analyzed using the effective stress method of analysis and the pore pressures acting are assumed to be those governed by gravity flow through the embankment [14]. The shear strengths used should be those determined from consolidated- undrained tests on saturated samples to which pore pressures are applied simulating those which may exist under the gravity flow in the dam and foundation. It is also mentioned that this approach is conservative for most well-compacted embankments since any shear strain which may be imposed on the embankment after the construction is completed and the reservoir is full, is likely to cause the soil to dilate and to reduce the pore water pressures temporarily. But studies of some failures have indicated that the procedure may not be conservative when the embankment material consists of very fine grained and highly plastic soils. In here the experience recorded by Peterson with failures of several clay dams in Canada had been mentioned as an example [14]. Consequently, for stability analysis where a large portion of the potential failure surface passes through highly plastic clay, which does not contain an appreciable quantity of sand or gravel, it is advised that the embankment stability analysis be carried out using the total stress method.

#### **Reservoir drawdown condition**

The computed factor of safety of the upstream portion of the dam is known to be the lowest following reservoir drawdown. The primary factor for this is the existence of higher pore pressures (drawdown pore pressures) acting in side the upstream slope.

Both the effective stress and the total stress methods of analysis are known to be in common use. The effective stress method of analysis is adopted more widely. On the other hand it is much difficult to estimate the pore pressures that develop following sudden reservoir drawdown. The shear strength parameters used for the case of rapid drawdown are those obtained from consolidated- undrained tests on saturated specimens.

## **4.2 Stability of Model Earth Dams**

Model of small earth dams (dams with heights smaller than or equal to 15m) has been used in order to carry out the stability analysis. For the purpose of comparative analysis,

an idealized prototype dam section of homogenous earth fill with 15m height, 6m crest width and side slopes of 1V: 2H on both the u/s & d/s faces was adopted. The foundation material of the model earth dam is assumed to be bedrock. End of construction stability condition was considered, and the u/s face of the dam was analyzed by the method of total stress analysis. A sketch of the model earth dam is indicated in fig. 4.2/1

The computation was facilitated by a computer program (Slope/W), and safety factors based on Bishop's simplified, ordinary method of slices & Janbu's method were taken. Slope/W is a software product that uses limit equilibrium theory to compute the factors of safety of earth and rock slopes. Slope/W is applied in the analysis and design of geotechnical, & mining engineering projects.

The slope stability analysis was carried out with respect to varying moisture content, soil type and state of compaction. The results of slope stability analyses are given in tables 4.2/1, 4.2/2, 4.2/3 & 4.2/4.

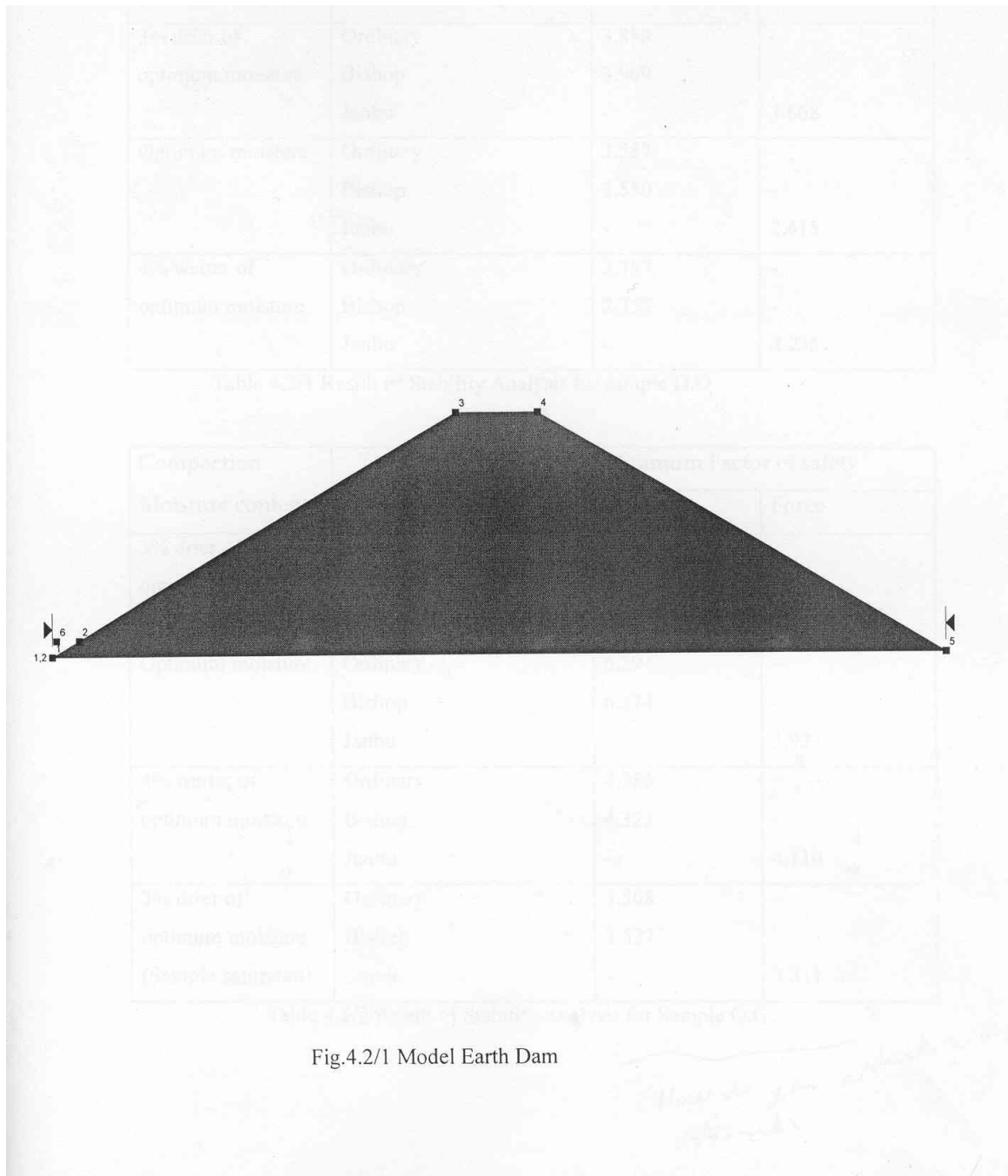


Fig.4.2/1 Model Earth Dam

Table 4.2/1 Result of Stability Analysis for Sample D.D

Compaction Moisture content	Method of Analysis	Minimum Factor of safety	
		Moment	Force
3% drier of optimum moisture	Ordinary	3.888	-
	Bishop	3.909	-

	Janbu	-	3.668
Optimum moisture	Ordinary	2.557	-
	Bishop	2.550	-
	Janbu	-	2.415
4% wetter of optimum moisture	Ordinary	2.387	-
	Bishop	2.352	-
	Janbu	-	2.235

Table 4.2/2 Result of Stability Analysis for Sample G.G

Compaction Moisture content	Method of Analysis	Minimum Factor of safety	
		Moment	Force
3% drier of optimum moisture	Ordinary	7.539	-
	Bishop	7.577	-
	Janbu	-	7.120
Optimum moisture	Ordinary	6.291	-
	Bishop	6.274	-
	Janbu	-	5.923
4% wetter of optimum moisture	Ordinary	4.386	-
	Bishop	4.322	-
	Janbu	-	4.110
3% drier of optimum moisture (Sample saturated)	Ordinary	3.508	-
	Bishop	3.527	-
	Janbu	-	3.311

Table 4.2/3 Result of Stability Analysis for Sample G.G (Modified Proctor)

Compaction Moisture content	Method of Analysis	Minimum Factor of safety	
		Moment	Force
3% drier of optimum moisture	Ordinary	16.872	-
	Bishop	16.714	-
	Janbu	-	15.802

Optimum moisture	Ordinary	8.893	-
	Bishop	8.916	-
	Janbu	-	8.394
3% wetter of optimum moisture	Ordinary	5.808	-
	Bishop	5.789	-
	Janbu	-	5.467

Table 4.2/4 Result of Stability Analysis for Sample A.A

Compaction Moisture content	Method of Analysis	Minimum Factor of safety	
		Moment	Force
3% drier of optimum moisture	Ordinary	4.392	-
	Bishop	4.433	-
	Janbu	-	4.156
Optimum moisture	Ordinary	3.064	-
	Bishop	3.068	-
	Janbu	-	2.887
4% wetter of optimum moisture	Ordinary	2.732	-
	Bishop	2.698	-
	Janbu	-	2.555

From the results given in the tables, one can observe the following:

The maximum values of safety factors against slip failures were obtained for the samples compacted on the drier side of the optimum moisture contents. Towards the optimum moisture contents and on the wetter side the safety factors decreased for all the soils.

The largest safety factor was obtained for the soil from Gilgel Gibe. The minimum safety factor obtained for the soil from Gilgel Gibe has been obtained to be almost equal to the maximum safety factor for the core material of Dire Dam.

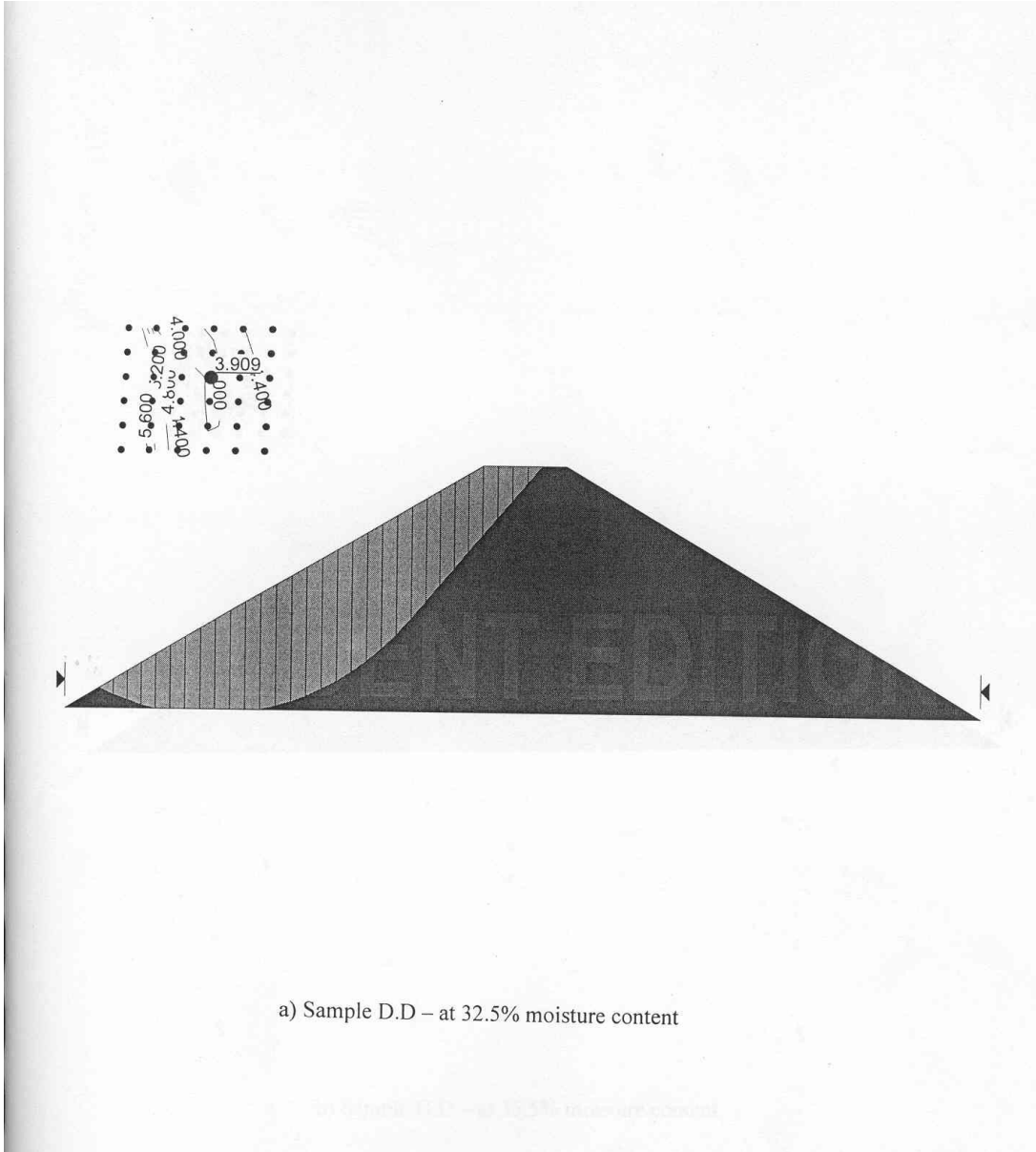
The relatively large safety factors obtained from the stability analysis were on one hand because of the x-sectional dimensions adopted for the analysis; on the other hand they indicated that the investigated materials are good enough to be used as embankment fill

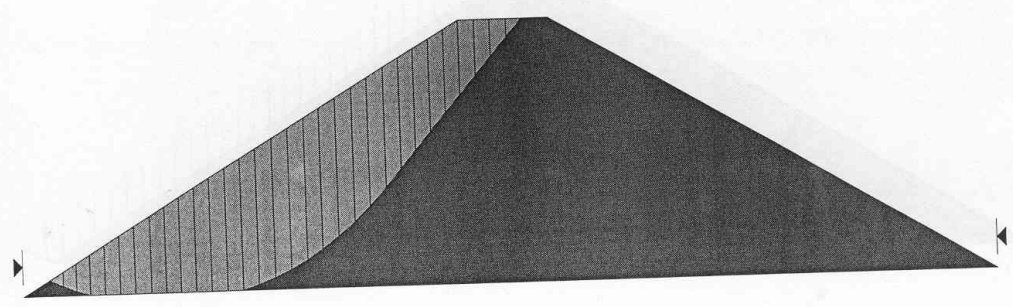
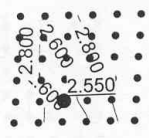
material from stability point of view. In all the cases, safety factors greater than two were obtained. The other point to be noticed is that the stability analysis was done only for the case of end of construction condition. If stability analysis were made for reservoir full and/or rapid drawdown condition smaller safety factors might be obtained.

At the modified Proctor compaction, which was done for the soil from Gilgel Gibe, Pronounced safety factors were obtained.

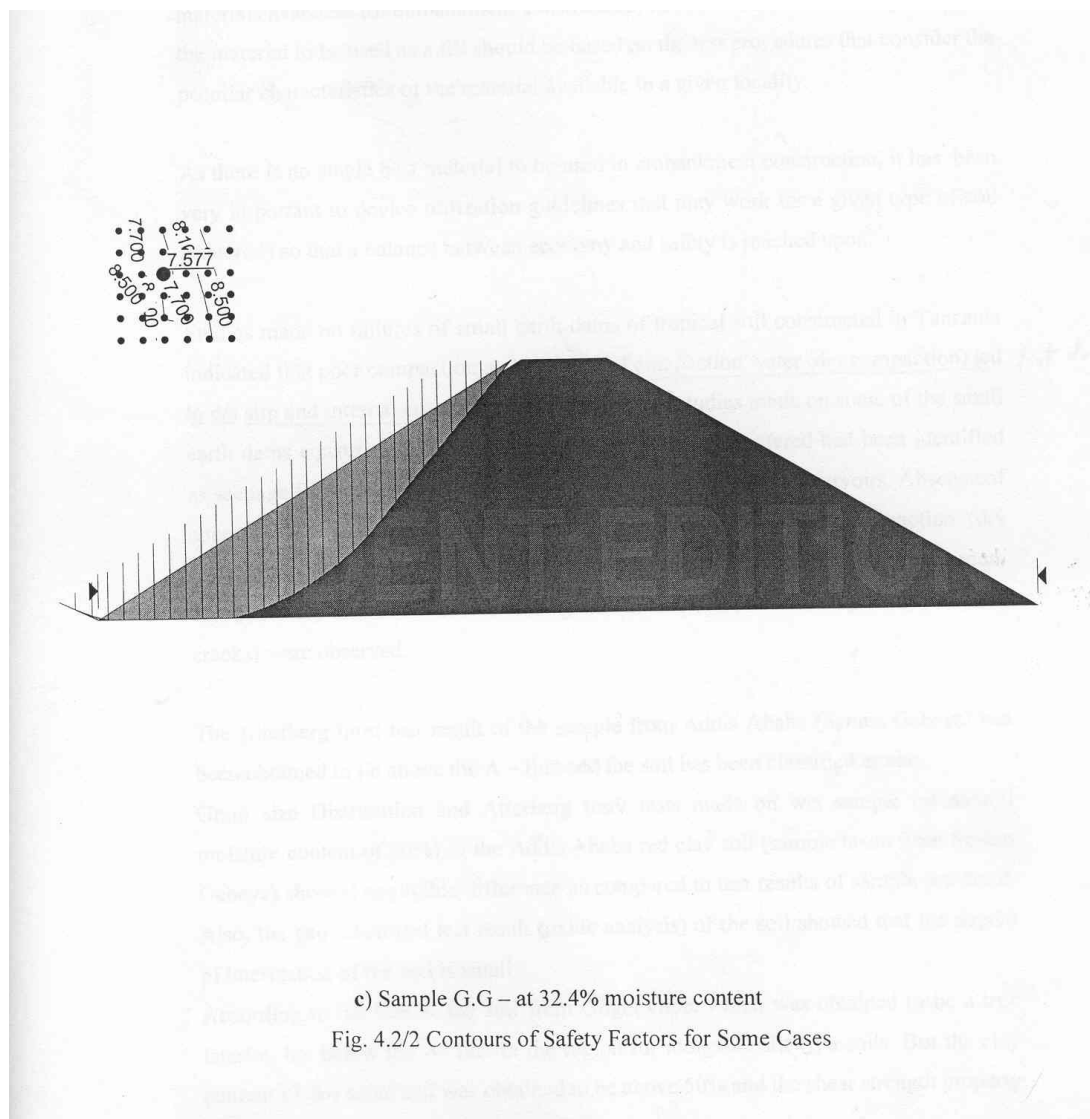
As the shear strength parameters used were those of uu – triaxial tests, change in cohesion influenced the change in safety factors more significantly than the change in angle of internal friction.

Illustrative figures of the contours of safety factors are indicated in fig. 4.2/2 a, b and c





b) Sample D.D – at 35.5% moisture content



## Unit 5. Discussions

The points addressed in the thesis work are reanalyzed and discussed as follows:

From experiences established based on earth dams constructed so far, it has been observed that the design of earth fill dams should reflect the actual site condition and materials available for embankment construction. In line with this, the manipulation of the material to be used as a fill should be based on the test procedures that consider the peculiar characteristics of the material available in a given locality.

As there is no single best material to be used in embankment construction, it has been very important to device utilization guidelines that may work for a given type of soil (material) so that a balance between economy and safety is reached upon.

Studies made on failures of small earth dams of tropical soil constructed in Tanzania indicated that poor compaction and shortage of compaction water (dry compaction) led to d/s slip and internal erosion of earth dams. From studies made on some of the small earth dams constructed in our country, the problems encountered had been identified as seepage flows, surface cracks, siltation and non-filling of the reservoirs. Absence of appropriate geological and geotechnical investigations, improper compaction (dry compaction by increasing merely the number of pass to attain the supposed maximum dry densities, ultimately over stressing the fill material, which may result in shear cracks) were observed.

The Atterberg limit test result of the sample from Addis Ababa (Semen Gebeya) has been obtained to lie above the A – line and the soil has been classified as clay.

Grain size Distribution and Atterberg limit tests made on wet sample (at natural moisture content of 20%) of the Addis Ababa red clay soil (sample taken from Semen Gebeya) showed negligible difference as compared to test results of sample pre-dried. Also, the geo- chemical test result (oxide analysis) of the soil showed that the degree of laterization of the soil is small.

According to the USCS, the soil from Gilgel Gibe, which was obtained to be a true laterite, lay below the A- line in the region for inorganic silt type soils. But the clay content of this same soil was obtained to be above 50% and the shear strength property of this soil was obtained to be much better than the shear strength of the other soils investigated. The Wesley mineralogical classification of the same soil indicated that the soil falls into the group of residual soils which are strongly influenced by the clay

minerals only found in tropical residual soils, namely, the sesquioxides. The sesquioxides cemented the soil particles together to result in stiff soil aggregation and hence soil with better shear strength parameters. It has been observed that the geotechnical property of such soils can be properly described if mineralogical classification is used to support the conventional classification systems. The soil that has been used as a core material of Dire Dam has been classified as clayey silt.

It has been tried to observe the influence of sample pre-drying on dry densities of the compacted soils. Although the gap is small, relatively higher dry densities have been obtained for sun-dried samples than for the air-dried ones. This effect has been noticeable for the drier side of the optimum moisture contents. A noticeable difference between the maximum dry densities has been observed for the soil from Gilgel Gibe for the samples compacted after sun drying and air-drying.

The permeability test carried out on the soil from Gilgel Gibe indicated that the soil is more permeable when compacted on the drier and wetter side of the optimum moisture contents than at the optimum, the tests being carried out after saturating the samples. Also, the soil has been obtained to be slightly dispersive (erodable) when soaked.

In general, the as compacted unconsolidated undrained shear strength of the samples tested on the drier side of the optimum moisture contents were high but decreased upon increasing the compaction moisture contents. The sample from Gilgel Gibe, found to be true laterite, attained the maximum values of the angle of internal friction and cohesion as compared to the remaining soils, comparing the results obtained under the same degree of compaction. But the same soil registered higher percentage decrease in shear strength parameters upon further increasing the compaction moisture content to the wetter side of the optimum. This was because of the effect of saturation softening of the sesquioxides. When compacted at higher compactive effort (modified Proctor) the soil from Gilgel Gibe attained very large values of shear strength.

When the lower portion of a dam embankment is soaked for the first time with the rising reservoir water, the saturated soil may lose its structural strength if it has not been compacted adequately. When this happens saturation settlement will occur either under the influence of the overburden load or without the influence of the overburden

load (collapse). This condition creates an opening for concentrated seepage through the dam body. Saturation settlement tests indicated that the sample compacted on the drier side consolidated as a result of saturation (soaking of the sample) While those at the optimum moisture and on the wetter side remained undeformed.

Slope stability analysis was carried out following the total stress analysis method. In general, the safety factors obtained against slip failure of the model dams analyzed indicated higher values for the samples compacted on the drier side of the optimum moisture. Following the shear strength parameters, the largest safety factor was obtained for the soil from Gilgel Gibe whereas the smallest factor of safety was obtained for the soil that has been used as a core material in Dire Dam, comparing the results obtained under the same degree of compaction.

## **Unit 6. Conclusion & Recommendation**

### **6.1 Conclusions**

Based on the results of the tests carried out & the foregoing discussions the following conclusions are drawn:

1. The red clay soil used in the construction of the cofferdam of Gilgel Gibe Hydroelectric Project is a true laterite soil where as the clayey silt soil used as a core material of Dire Dam is lateritic. Conventional classification methods should be supported by Wesley Mineralogical Classification for such tropical soils.
2. Some tropically weathered (laterized) soils such as the fill material for the cofferdam of Gilgel Gibe hydroelectric Project have very high shear strength regardless of their position on the plasticity chart. Therefore, such materials should not be discarded based on their position on the plasticity chart only.
3. Method of sample preparation during compaction has been obtained to have an influence on the dry densities. Sample preparation for laboratory compaction tests should resemble the conditions prevailing in the field, i.e air- dried or partially wet samples should be used in the laboratory.
4. Compaction moisture has been obtained to influence permeability. Permeability has been obtained to decrease towards the optimum moisture content. On the drier and wetter sides of the optimum, higher values of coefficients of permeability have been obtained with the value on the drier side slightly greater than the one on the wetter side for the sample from Gilgel Gibe. Therefore, dry compaction should be avoided to minimize the effect of internal erosion and seepage flows for such soils.
5. The problem of saturation settlement has been observed to occur when the soil is compacted on the drier side of the optimum moisture. Due to saturation settlement, cracks may be formed and ultimately seepage flows may occur. Therefore, dry compaction should be avoided to prevent such problems.
6. For the soils investigated, compaction moisture has been obtained to greatly influence the stability of embankments. Samples compacted on the drier side of optimum moistures registered high shear strengths. But these strengths were observed to be lost upon further increasing the compaction moisture and also upon sample saturating. Therefore, dry compaction results in unreliable and unsafe shear strength and hence should be avoided.
7. In the soil from Gilgel Gibe, intensive compaction energy has been observed to result in high shear strengths that are obtained at drier moistures and small strain

levels with a reflection of the nature of brittle failure. Embankment construction with prevalence of brittle stress - strain condition may result in formation of local shear cracks that may develop into progressive shear failures. Therefore, overstressing of brittle materials should be avoided during compaction.

8. Although slightly more sensitive to compaction moisture, the material from Gilgel Gibe, true laterite, has got large shear strength and hence is more preferable as a fill for earth dams from stability point of view. But compaction equipments that destroy clods of soil should be used to avoid the dispersion (erodibility) of the fill material.
9. Small changes in moisture contents ( $\pm 3\%$ ) have been observed to result in considerable changes in shear strength and hence the stability of earth dams. Therefore, careful field compaction moisture control should be practiced.

## **6.2 Recommendations**

1. The influence of sample preparation on the index property tests and compaction characteristics of tropical soils has to be investigated.
2. Further investigations have to be carried out to consolidate the results obtained in this work by taking a soil of only one generic type and locality at a time, and by deeply focusing on specific aspect of the problem so that definite conclusions can be made for each type of soil. Afterwards, the results may be combined to establish appropriate guidelines.

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**Table 2.1.3.1/1 Characteristics of residual soil groups\_**

(After Wesley)

Group properties and behavior	Examples	Means of identification	Comments on likely engineering
Major group	sub- group		
<p>Group A                      large group of soils (including the ‘ saprolites’) where (Soils without a strong mineralogical influence)                      homogeneous and form a tidy group                      systematic evaluation and analysis than                      and role of bonding                      secondary bonds) important</p>	<p>(a) Strong macro-structure influence from igneous rocks, and sedimentary rocks                      Completely weathered rocks formed from igneous and Sedimentary rocks</p>	<p>Highly weathered rocks                      Highly weathered rocks                      Visual inspection, and                      evaluation of sensitivity,                      liquidity index, etc.</p>	<p>Visual inspection                      This is a very behavior (especially in slopes) is of the discontinuities, fissures,                      These soils are essentially much more amenable to group (a) above. Identification of nature (from relict primary bonds to weak to understanding behavior.</p>
<p>sub-group. Likely to behave similarly</p>	<p>(c) Little structural influence                      rocks</p>	<p>Soils formed from very homogeneous rocks                      Little or no sensitivity,                      uniform appearance</p>	<p>This is a relatively minor to moderately over consolidated soils.</p>
<p>Group B                      problem soils found in flat or low lying areas</p>	<p>(a) Smectile</p>	<p>Black cotton soils, many soils formed</p>	<p>Dark color (grey to Theses are normally</p>

(Soils strongly influenced by commonly occurring minerals) (montmorillonite group) in tropical areas in poorly drained conditions (black) and high plasticity suggest soils of this group low strength, high shrinkage characteristics.

(b) Other minerals This is likely to be a very minor subgroup

Group C (a) Allophane 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 generally very fine-grained soils, of low to medium activity. Engineering properties generally good. (Soils strongly influenced by clay minerals essentially found only in residual soils) (b) Halloysite Soils largely derived from older volcanic rocks; especially tropical red clays Reddish colour, well drained topography and volcanic parent rock are (Note that there is often some overlap between allophane and halloystic soils).

(c) Sesquioxide This soils group loosely referred to as 'lateritic' or laterite Granular, or nodular appearance and gravel. Behavior may range from low plasticity to non-plastic gravel.



