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THEME

Etude En Laboratoire Le Comportement D'un Sol Renforce Par Des Fibres Organiques

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...نَرْفَعُ دَرَجَاتٍ مِّن نَّشَاء وَفَوْقَ كُلِّ ذِي عِلْمٍ عَلِيمٌ .

{ يوسف -الآية 76-}

ABSTRACT

The behavior of soils reinforced by fibers has been studied by several investigators over the last two decades. Fiber-reinforced soil is becoming a viable soil improvement method for geotechnical engineering problems.

The aim of this research work is to presents an experimental study on the behavior of silty soil reinforced by organic fibers, conducted in triaxial compression tests.

This thesis is organized into five chapters.

Chapter One presents the different types of rheological behavior of soils and the influential parameters.

Chapter Two laid out background information for the study carried out in the current thesis. The background information described the relation between the tree roots and soil stabilisation. Also show the important of tree in improving soil stabilisation.

Chapter Three presents introduction of researches conducted, and provides a literature review of basic methods of soil stabilisation analysis in field studies.it describes the different methods of soil stabilization (slopes, river bank, embankment...) by reinforcements.

Chapter Four describes the methodology used in this study and presents the triaxial apparatus developed for the study of soil behavior. This device can perform triaxial tests following various stress paths (isotropic drained and undrained, monotonic, proportional to deformation, etc...). After describing the procedure followed for the tests, we exposed the different arrangement of fibers in the soil.

Chapter Five includes a presentation of the results of drained and undrained triaxial compression tests performed on the chlef soil from the region of Chlef. It first presents the drained triaxial compression tests performed on samples of plain soil, and the soil reinforced by roots of acacia pycnantha under different confining pressures ranging from 100 to 400 kPa. This chapter also concludes with a determination; based on testing, soil characteristics within the study area (shear strength, secant modulus, and internal friction angle). These results will be compared to those found in literature in particular soil reinforced by tree roots.

Keywords: Compression Triaxial Test, Fiber, Reinforcement, soil, undrained, drained, Root.

الملخـــــــــص

هذه المذكـــرة عبارة عن عمل مخبري لدراســـة سلوك التربة مدعمة بالألياف العضوية تحت تأثير الاحمال المستقرة . في هذا البحث تم دراسة التأثيرات النفعية لتسليح طبقة التربة باستخدام شعيرات ذات أطوال صغيرة (الفيبرات) موزعة عشوائيا اضافة الى استعمال جذور شجرة الاكاسيا. وتشمل اربعة فصول :

* **الفصـــــــــــــــــل الاول** يختص بمراجعة وتحليل البحوث السابقة حول الموضوع.

* **الفصـــــــــــــــــل الثاني** يتطرق الى معرفة طرق تثبيت التربة (المنحدرات ,الركام ..) بالألياف والجذور الخاصة بالأشجار .

* **الفصـــــــــــــــــل الثالث** يحتوي على شرح العمل والاجهزة المستعملة في التجارب بالإضافة الى طريقة قياس دقة النتائج المحصلة من هذه التجارب.

* **الفصـــــــــــــــــل الرابع** يشمل عرض النتائج بواسطة الاحمال الثابتة ومناقشة مدى تأثير الالياف والجذور على سلوك التربة

اظهرت النتائج ان ارتفاع نسبة الالياف تقوي مقاومة التربة الى حد ما ,اما سلوك التربة المدعمة بالجذور بيكون حسب وضعية الجذور اذا كان عموديا او افقيا .

الكلمــــــــــات الدالـــــــة : ضغط ثلاثي المحاور ,دعامة , تربة, الالياف ,التصريف ,الجذور

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Dedication

This work is dedicated to my parents who always encourage, help, think and pray for me.

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Who accompanied me to this work since March 11, for his continuous help, his encouragement throughout the period of preparation of this thesis and for his support and incessant help all

Negadi Kheira

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❖ NOTATION

- B = Skempton coefficient
- c = Cohesion, kN/m^2
- σ_n = the normal stress acting on the rupture plane.
- CSA= cross-section area
- c′ = the effective cohesion
- ϕ' = the effective angle of internal friction of the soil
- u = the pore pressure generated during shearing.
- ϕ = Angle of internal friction, deg.
- Tp= Peak strength
- τr =Residual strength
- T_f =shear strength.
- σ = normal stress applied
- uw = pore water pressure
- $\sigma c'$ = the highest past overburden stress for a soil
- σ' = the current overburden stress for a soil.
- OCR= overconsolidation ratio
- T =reinforcement tensile strength.
- ΔH =Vertical spacing of reinforcement.
- σl, σ 2,σ 3 = Principal stresses, kN/m2
- $\sigma_r 1$ = higher stress level
- $\sigma 1$, $\Delta\sigma 1$ =Vertical stress and increment vertical stress.
- $\sigma 3$, $\Delta\sigma 3$ =Confining stress and increase in confining stress.
- σv =Vertical stress acting on reinforcement.
- θ = angle of shear rotation
- P =Vertical load.
- q =Average footing pressure.
- K_0 = Coefficient of Earth pressure at rest
- Kp = the coefficient of passive earth pressure
- p = Mean normal stress, kN/m2
- q = Deviatoric stress, kN/m2
- ε = Strain.

INTRODUCTION AND OVERVIEW

Chapter One

I. INTRODUCTION

Slope instability is one of the serious geological hazards to most environmentally regions. Significance numbers of failure are reported on residual soil slope and more than 2/3 of slopes movements are shallow sliding with less than 1m depths. Earth slope could be stabilized using reinforcement techniques and bioengineering techniques seem suitable for preventing shallow slope failures. Vegetation plays important roles for slope stability by providing immediate shear strength enhancement and modifying the saturated soil water regime.

The evolution of slope stability analyses in geotechnical engineering has followed closely the development in soil mechanics as a whole .slopes either occurs naturally or is engineered by humans. Slope stability problems have been faced throughout history when the human or nature has disrupted the delicate balance of natural soil slopes. Furthermore, the increasing demand for engineering cut and fill slopes on construction project has only increased the need to understand analytical methods, investigative tools, and stabilisation methods to solve slope stability Problems. Slope stabilization methods involve specialty construction techniques that must be understand and model in realistic ways.

During the last decade there has been a pronounced increase in the number of catastrophique events including shallow landslides and erosion processes after heavy rainstorms, particularly in mountains regions. Slope in thus a major concern for all those responsible for the protection of human lives and infrastructure against natural hazards.

Our country knows this problem of slopes and bank instability, the coverage of this natural risk is an integral part concern of public authorities in town and country planning. Landslides know last decades a large increase, the landslide corresponds to a loss of due resistance mainly a surgénération of the pressure of water in the ground.

Currently trees and plantations are usually in the soil to repair the collapse or the sliding of slopes, the improvement of the stability of breast walls and the elevations. However, the strengthening and the improvement of grounds by the roots of plants and fibers recently acquired the attention in a lot of application in civil engineering. The investigations of the role of the strengthening by roots in the prevention or the reduction of the instability of banks and elevations are largely reviewed and inspired by the works on the strengthening of ground by roots begun(undertaken) by Wu (1976), Waldron (1977), Waldron and Darkessian (1981) and the innovative work of Endo and Tsurat (1969).

Recent experimental investigation on fibre reinforcement in sand yielded controversial findings, depending on the method applied. Using shear tests, Yetimoglu and Salbas (2003) found no improvement in the shear strength of the composite compared to pure sand, and Operstein and Frydman (2000) reported an essentially constant angle of internal friction of soil reinforced by roots, but an increase in the apparent cohesion.

However, analyses based on triaxial tests compression revealed and increase in the angle of internal friction of composite (fibre reinforced sand) compared to the untreated granular matrix (Consoli et al. 2002). The addition fibres to cohesionless pure sand yielded an increase in the angle of internal friction without any change in the cohesion but when added to cemented sand (with cohesion), the increase in the angle of internal friction went along with a decrease in cohesion (Consoli et al. 2002). Furthermore, it was found that the reinforced effect generally correlates positively with the fibre aspect ratio, and if the aspect ratio and concentration of fibres are kept constant, the composite strength is positively correlated with the length of the fibres (Michalowski and Cermak 2003).

The present study undertaken to examine the influence of root fibres and roots of acacia pycnantha on the shear strength of Chlef silty soil. The focus is on triaxial testing with a programme including reinforced and unreinforced soil with root fibres and roots. Consolidated drained and undrained triaxial testing was performed at different confining pressures to assess the effects of root fibers and roots on soil stability.

I.2 PRINCIPAL RHEOLOGICAL CONCEPTS OF GRANULAR MATERIALS

I.2.1 Shear Characteristics

As a result of excessive external stresses, soils suffer irreversible deformation manifested by the sliding of grains on each other. At the rupture, the shear stress of the soil (in fact the skeleton) is called ultimate strength or shear failure.

Coulomb (1776) was the first to define an expression of resistance to shearing of granular materials, based on links between the grains constituting the skeleton (due to links due to capillary tension water content interstitial and adsorbed) and the skeletal structure (shape and arrangement of grains). This expression is given by the following equation:

$$\tau = c + \sigma_n \operatorname{tg}\varphi \qquad \text{(I-1)}$$

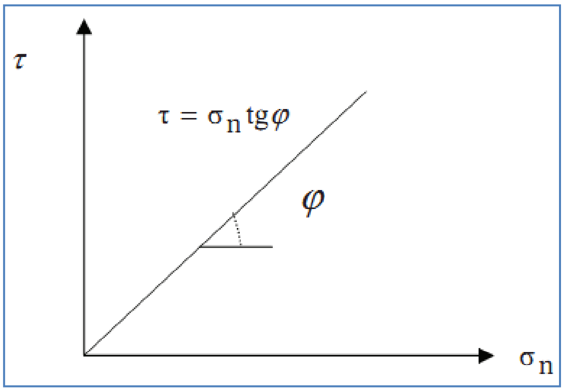
Where: **c** is cohesion and ϕ is the internal friction angle of soil (shear parameters). $\boldsymbol{\sigma_n}$ is the normal stress acting on the rupture plane.

The Cohesion and the internal friction angle are intrinsic properties of the material and depend on the mineralogy, grain size and its geological history. These two parameters take specific values when the material is purely rubbing ($c = 0$) or purely cohesive ($\varphi = 0$). Between these two extremes, they find materials that have properties intermediate as in the case of most natural soils (Figure I.1).

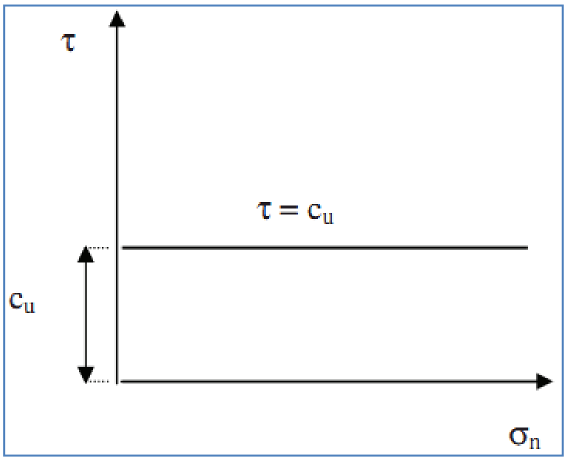
Given the principle of effective stress, Terzaghi (1923) has modified the previous expression and proposed in its place the following formula:

$$\tau = c' + (\sigma_n - u)\operatorname{tg}\varphi' \qquad \text{(II-2)}$$

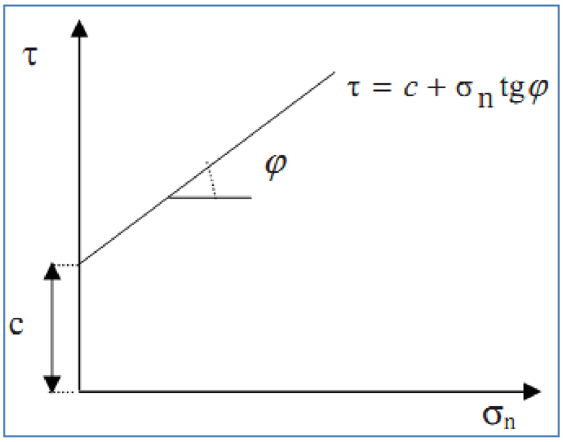
Where: **c′ is** the effective cohesion and **ϕ ′ is** the effective angle of internal friction of the soil and **u is** the pore pressure generated during shearing. This relationship indicates that the shear strength under constant total stress, or some variable increases when the excess pore pressure decrease. In other hand, the shear behavior of soils for long-term, at the end of consolidation, has a shear strength greater than that in the short term corresponding to the beginning of the consolidation.



a / granular soil or purely rubbing (eg sand).



b/Sol purely coherent (eg stiff clay).



c/ cohesive soil or intermediate (eg soft clay).

Figure I.1. Laws of Coulomb and intrinsic curves for different soil types

The Soil type and condition of consolidation in which it is located and conditions drainage determine the shear behavior of soil, but also the intensity of the forces exerted and how these efforts are applied. They distinguish between drained and undrained characteristics described below.

I.2.2 Undrained Characteristics

In undrained tests, the loading is quite fast. In the absence of drainage and volume change, the normal components of stresses induced in the soil by application of the force are transmitted almost entirely in the liquid phase. Figure (I.2) shows the undrained stress paths whose final state is on a straight critical state (curve intrinsic ground state normally consolidated) similar to the curve of isotropic consolidation.

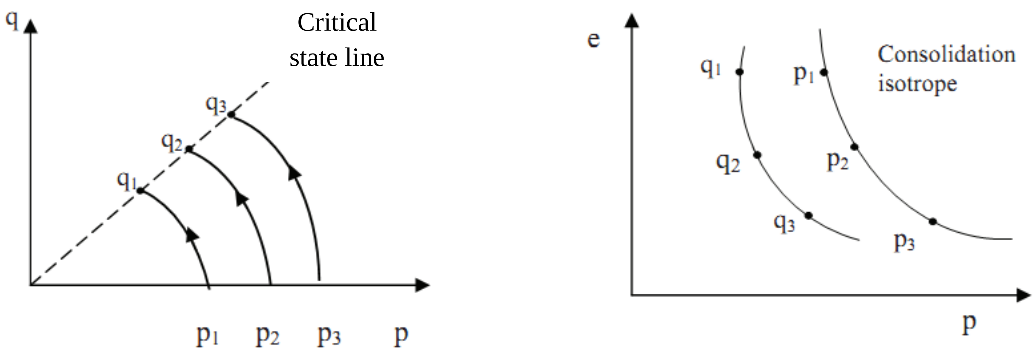


Figure I.2. Undrained stress paths in the plans (p-q) and (e-p)

Under these conditions, the shear is accompanied by the appearance of strong excess pore pressures, a significant reduction in the effective stress and frictional resistance of the particles. The critical shear strength is the maximum stress that can be mobilized during shear.

The shear characteristics corresponding are called undrained characteristics. They reflect the overall behavior of both solid and liquid phases and are determined experimentally from the results of the shear undrained triaxial, so that c_u is the undrained cohesion and $\varphi_u \approx 0$.

I.2.3 Drained Characteristics

In drained tests, considering the soil permeability and the length of drainage ligne, application of the load is slow enough to induce any time of excess pore pressure in the soil (null or negligible). The applied forces are transmitted to the skeleton of the soil and induced stresses are the effective stresses. Figure I.3 shows the stress paths drained whose final states are on the same line of critical condition as that obtained for undrained stress paths.

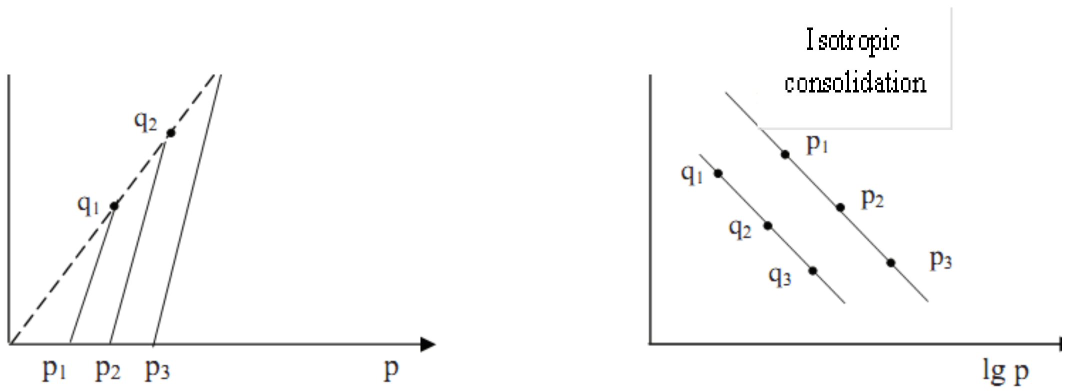


Figure I.3. Drained stress paths in the plans (p-q) and (e-lg p)

Under these conditions, application of the effort is accompanied by a change in volume more or less depending on the applied stresses. This decrease in volume results in a tighter grain which causes an increase in its real cohesion. The effective normal stress and resistance to internal friction increased. And consequently, the shear strength can grow beyond the critical resistance.

The shear characteristics corresponding are called drained characteristics and to be determine from the results of shear drained triaxial, and **c '** is the effective cohesion and **φ'** is the angle of internal friction effect.

I.2.4 CONTRACTANCY AND DILATANCY CONCEPTS

The concept of contractancy -dilatancy is directly related to the granular structure. The Contracting is a densification of the material under a shear loading condition drained. The phenomenon is more pronounced than the sand is loose initially. In the other hand, dilatancy is the opposite phenomenon, observed in dense sands. By analogy to the behavior drained the contractancy (respectively the dilatancy), in an undrained shear test, and is characterized by a positive generation (resp. negative) pore pressure (u).

I.2.5 CRITICAL STATE

Casagrande (1936) was the first to introduce the concept of critical index, and corresponds to the state where the soil deforms continuously under a constant drained shear stress. The critical-state framework originally developed from plasticity theory for saturated soils (Roscoe et al., 1958) offers a theoretical basis for predicting not only the volume change behaviour but also the shear deformation taking place during triaxial compression (Figure 1.4). A more advanced understanding of the behaviour of soil undergoing shearing lead to the development of the critical state theory of soil mechanics (Roscoe et al (1958) In critical state soil mechanics, distinct shear strength is identified where the soil undergoing shear does so at a constant volume, also called the 'critical state'. Thus there are three commonly identified shear strengths for a soil undergoing shear:

Peak strength **τp**
Critical state or constant volume strength **τcv**
Residual strength **τr**

The peak strength may occur before or at critical state, depending on the initial state of the soil particles being sheared:

A loose soil will contract in volume on shearing, and may not develop any peak strength above critical state. In this case 'peak' strength will coincide with the critical state shear strength, once the soil has ceased contracting in volume. It may be stated that such soils do not exhibit a distinct 'peak strength'.

A dense soil may contract slightly before granular interlock prevents further contraction (granular interlock is dependent on the shape of the grains and their initial packing arrangement) . In order to continue shearing once granular interlock has occurred, the soil must dilate (expand in volume). As additional shear force is required to dilate the soil, ‘peak’ strength occurs. Once this peak strength caused by dilation has been overcome through continued shearing, the resistance provided by the soil to the applied shear stress reduces (termed "strain softening"). Strain softening will continue until no further changes in volume of the soil occur on continued shearing. Peak strengths are also observed in overconsolidated

clays where the natural fabric of the soil must be destroyed prior to reaching constant volume shearing. Other effects that result in peak strengths include cementation and bonding of particles.
The constant volume (or critical state) shear strength is said to be intrinsic to the soil, and independent of the initial density or packing arrangement of the soil grains. In this state the grains being sheared are said to be 'tumbling' over one another, with no significant granular interlock or sliding plane development affecting the resistance to shearing. At this point, no inherited fabric or bonding of the soil grains affects the soil strength.

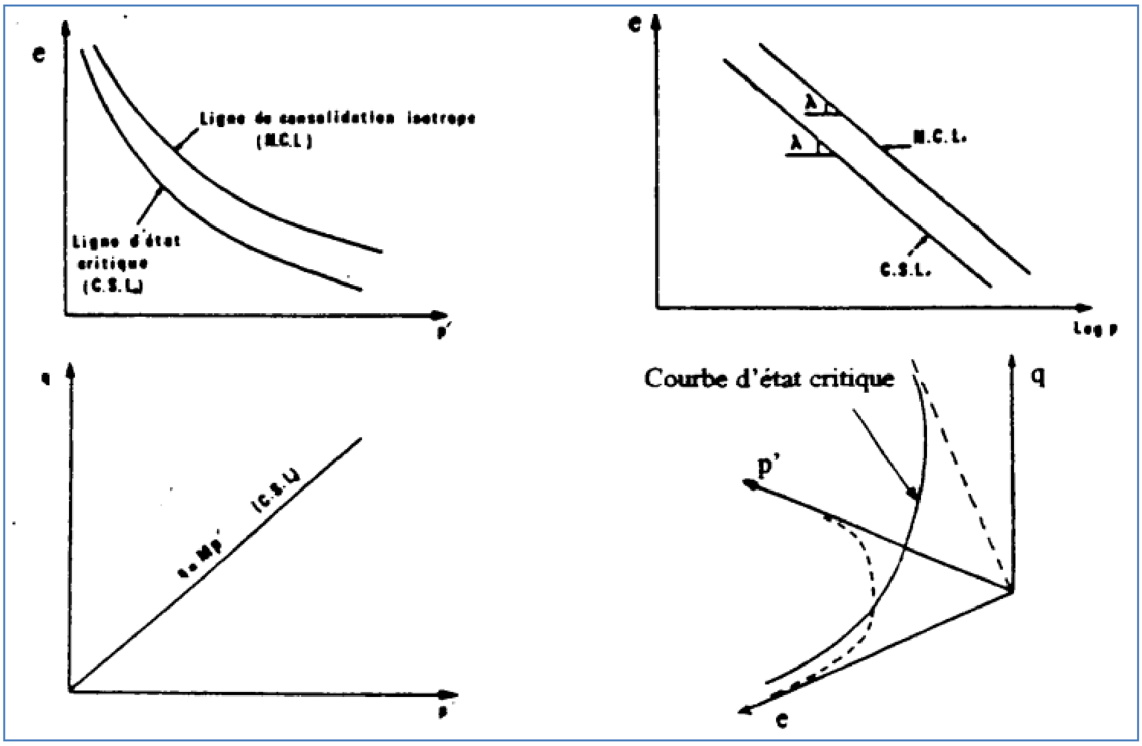


Figure I.4. Representation of the critical state (Roscoe and al.1958)

I.2.6 LIMIT STATE AND CRITICAL STATE CONCEPTS

The behavior of natural soils is defined by a limit pressure, the pressure preconsolidation which is a critical constraint to consolidation, where the compressibility of the soil increases, and their internal structure is changed from a strong state structure, where the volumetric strain and shear are small and a reversible unstructured condition weaker characterized by the appearance of volumetric strain, important shear and largely irreversible.

Roscoe et al (1958) based on theoretical and experimental studies on clay samples reconstituted in the laboratory, they have proposed the concepts of limit state and critical state as ground rules for the study of behavior of clays:

Limit state is defined by a surface charge, called limit state surface, which separates the space of principal stresses, the domain of small deformations (reversible deformation) from that corresponding to the large deformation (irreversible deformation). This limit is formed by all points of limit state (points corresponding to the values of ultimate shear strength) of the stress paths simulated in the laboratory shear tests in triaxial apparatus, using state of consolidation of the overconsolidated domain. The critical state is defined as the condition from occurring in the soil as deviatoric plastic deformations. In this state, the soil behaves like a fluid rubbing, distorts and flows at constant volume (purely deviatoric shear).

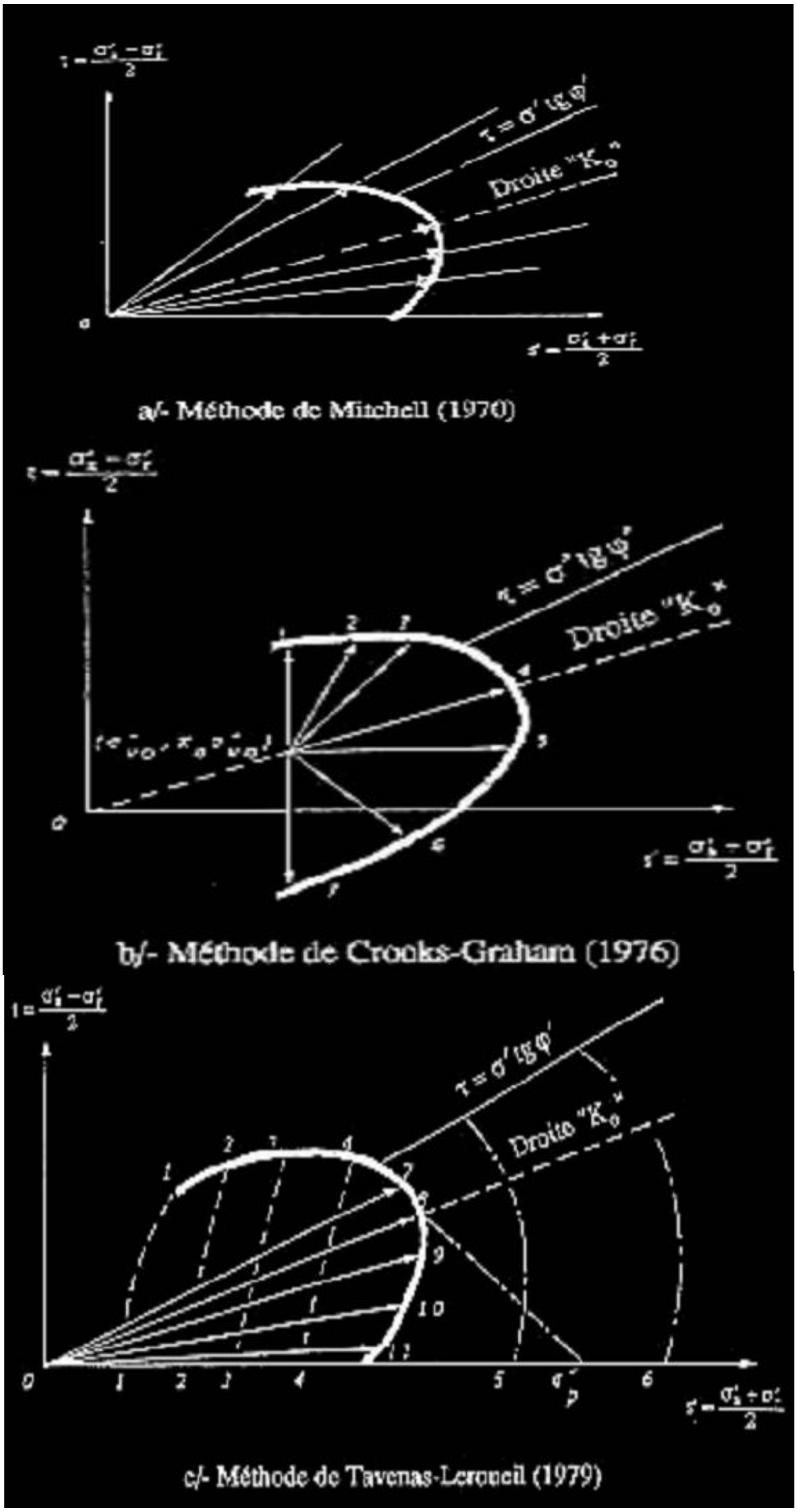
This condition is associated with the existence of a "critical void ratio" reached at the time, which develops shear plastic deformation without volume change, and stress.

Experimental studies have established the existence a limit state curve for each soil studied and show that the concepts of limit state and critical state were applicable. Among these studies, the work of Tavenas and Leroueil (1979) on the clay of St. Alban (Quebec) have confirmed the applicability of the concepts above mentioned and also presented a method for determining the limit state curve of natural soft clays from the results of conventional triaxial tests and oedometer tests conventional loading levels. In addition, Mitchell (1970) and Crooks and Graham (1976) presented methods that also call for routine testing laboratory. These three methods are described briefly as follows (Figure I.5).

The method of Mitchell (1970) is to follow to test the stress paths to constant effective stress ratio $=\sigma'_a/\sigma'_r$. The Limit state points are determined on the curves representing the evolution of volumetric strain according to the actual average applied stress. They are defined by the state of stress at which plastic deformation begins to develop.

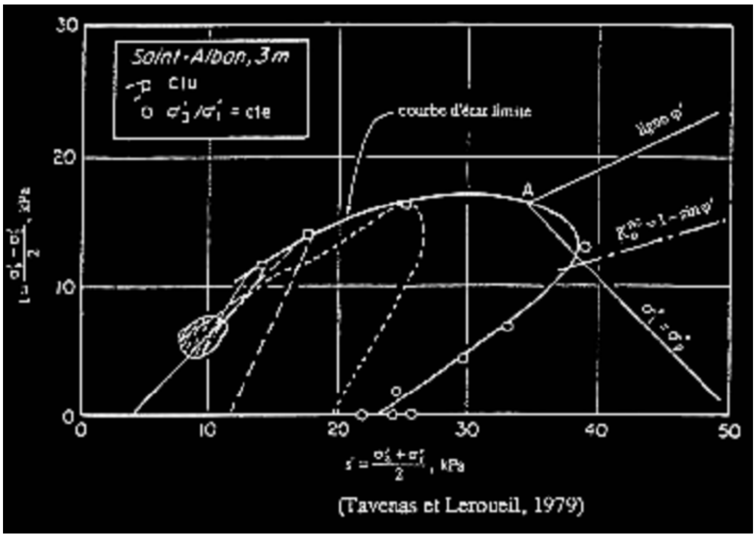
The method of Crooks and Graham (1976) resembles the preceding one; except that must start with reconsolidate the specimens to the effective stress (condition K_0) and then subjecting them to radial path (drained triaxial tests) from this state.

The method of Tavenas-Leroueil (1979) is to perform the compressibility oedometer tests, to determine the preconsolidation pressure of the soil, anisotropic consolidation tests at constant effective stress ratio at which they measured changes in the volume of the specimens according to the constraints applied to them and undrained shear tests after consolidation of these constraints in order 0.1 to 1.5 times the preconsolidation pressure of the soil. For each test corresponding to low shear stress of consolidation in the overconsolidated domain, the peak of the curve corresponds to a shear limit state; while the tests corresponding to strong constraints to determine the angle of internal friction of the normally consolidated state of the soil and therefore the line to critical state.

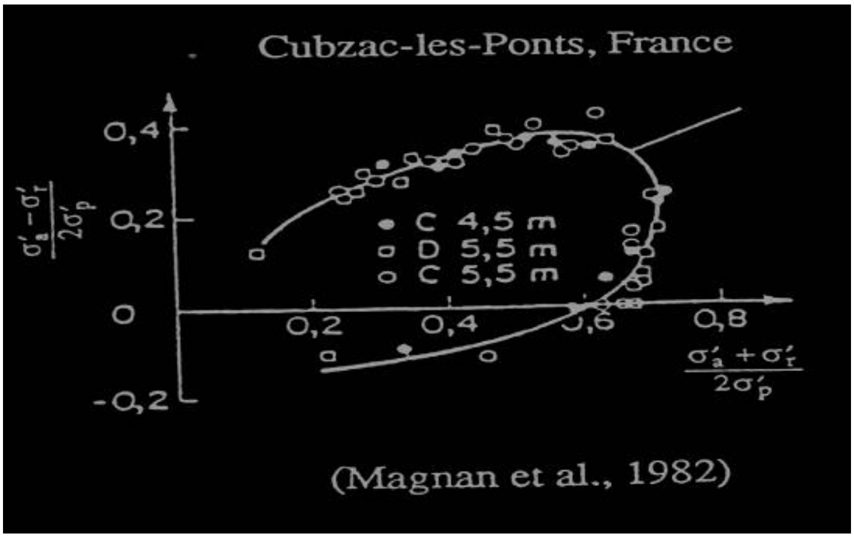


FigureI.5 Experimental techniques for determining the limit state curves of clayey soil

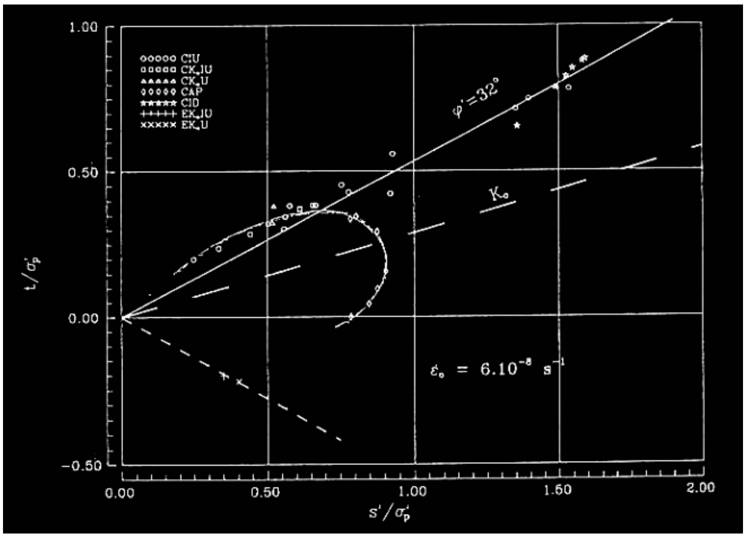
In one or other of these three methods, the limit state curve is obtained by connecting different points of the limit state considered clay. Figure (I.6.a, b, c) shows the limit state curves of some natural clayey soil.



a / Clay St. Alban (Tavenas and Leroueil, 1979)



b / Clay of Cubzac-les-Ponts (Magnan et al., 1982).



c / Clay de Guiche (Khemissa et al., 1993)

Figure I.6. Limit state Curves of some natural clayey soil

I.3 LITERATURE REVIEW

I.3.1 Shear Strength of Soil

Das (2002) defines the shear strength of soil as "the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it." There are two important shear strength parameters for soils, the angle of internal friction (φ) and cohesion (c). The φ angle indicates the degree of friction and interlocking among the soil particles, and the cohesion represents the ionic attraction and chemical cementation between the soil particles. Both of these parameters can be determined in a geotechnical laboratory by performing shear strength tests. Also, there are a few test methods that can be performed in the field to estimate shear strength properties of in-situ soils.

Shear strength of soil is a function of the normal stress applied, the angle of internal friction, and the cohesion. The angle of internal friction describes the interparticle friction and the degree of the particles' mechanical interlocking. This characteristic depends on soil particle gradation and shape and the void ratio. Cohesion describes soil particle bonding caused by electrostatic attractions. So, with normal stress, the angle of internal friction, and cohesion, the following equation, known as the Mohr-Coulomb theory, can be used to find the shear strength of soil under a certain condition:

$$\tau = c + \sigma (\tan \varphi) \qquad \text{(I.3)}$$

Where σ = normal stress applied. This equation can be plotted on an x-y graph with shear stress on the ordinate and normal stress on the abscissa. This is known as a failure envelope and is shown in (Figure I.7).

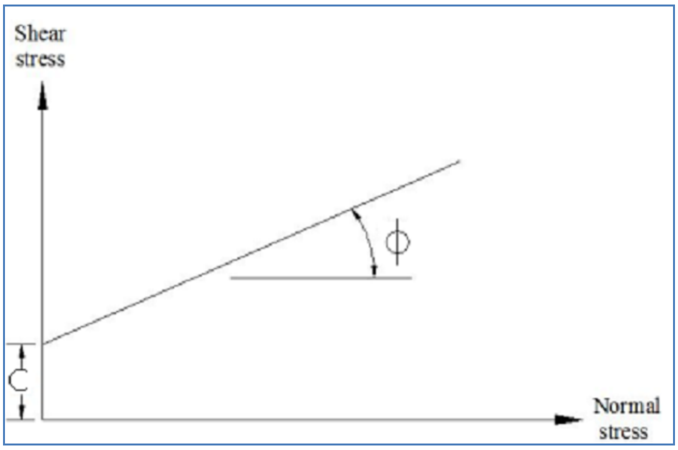


Figure I.7: Failure Envelope for Shear Strength of Soil

In reality, however, the failure envelope is rarely a linear relationship. The degree of electrostatic attraction and cementation of cohesive particles in the soil can cause a slight concave downward curve to form instead.

I.3.2 PORE WATER PRESSURE OF SOIL

Saturated soils have water filling all of their void spaces. This leads to the concept of effective and normal stress. When a column of saturated soil is subjected to load, the total stress is carried by both the soil particles and the water filling the voids. The equation given below describes this:

$$\sigma = \sigma' + u \qquad \text{(I.4)}$$

Where σ' = effective stress; and u = pore water pressure.
The effective stress is the soil particle acting as a skeleton to support the load. Therefore, the effective stress is often directly proportional to the total stress. Also, the shear failure envelope formula, Equation (I.4), can be addressed in terms of effective stresses for saturated soils as:

$$\tau' = c' + \sigma'(\tan \varphi') \qquad \text{(I.5)}$$

where c' = the effective cohesion; and φ' = the effective angle of internal friction.

Many times in the field, however, soil may not be fully saturated. Bishop et al. (1960) gave the following equation to describe the shear strength of unsaturated soils:

$$\sigma' = \sigma - ua - \chi\,(ua - uw) \qquad \text{(I.6)}$$

Where: u_a = pore air pressure; χ = degree of saturation; and uw = pore water pressure.

Going back to Equation (I.6) and adding new variables, the shear strength at failure for unsaturated soil can be written as:

$$\tau_f = c' + [\sigma - ua + \chi\,(ua - uw)]\,(\tan \varphi') \qquad \text{(I.7)}$$

For soil that is completely dry ($\chi = 0$), soil that is 50% saturated, and soil that is 100% saturated, the following three equations result, respectively:

$$\tau f = c' + (\sigma - ua)(\tan \varphi') \qquad (I.8)$$

$$\tau f = c' + (\sigma - 0.5u_a - 0.5uw)(\tan \varphi') \qquad (I.9)$$

$$\tau f = c' + (\sigma - u_w)(\tan \varphi') \qquad (I.10)$$

Typically, u_a is less than 0 and u_w is greater than 0. Experiments done by Casagrande & Hirschfeld (1960) revealed that unsaturated soil has greater shear strength than the same soil in a saturated condition. In some cases the unsaturated state may be temporary, and the soil may become eventually saturated due to surface precipitation and subsurface drainage events. Therefore, it is conservative to design highway embankments using the shear strength of saturated soils.

I.3.3 CONSOLIDATION

When loads are applied to clay that has low hydraulic conductivity, the pore pressure will increase greatly. Gradually, the pore water pressure and the effective stress will increase, resulting in a volume reduction. This can happen over a period of days, months, or years, depending on the type of soil and the corresponding drainage paths (Das 2002).

This leads to a discussion on the overconsolidation ratio (OCR) for soils. The equation for OCR is given below:

$$OCR = \frac{\sigma_c'}{\sigma'}$$

where σ_c' = the highest past overburden stress for a soil; and σ' = the current overburden stress for a soil.

Essentially, if the current overburden stress for a soil is the highest stress it has ever been subjected to, then the OCR will be 1. Soils under this condition are referred to as normally consolidate. Soils with an OCR above 1 are overconsolidated. This means they have been subjected to greater stresses than the current overburden one (Das 2002). The consolidation of soils and their past stress histories are important for triaxial compression testing.

BACKGROUND

Chapter Two

II.1 GENERAL

Slope instability is one of the major problems in geotechnical engineering where disasters, like loss of life and property, do occur. The majority of these slope failures are of vegetated or forested natural slopes. A natural slope is different from an embankment or a man-made slope in that the effects of vegetation and soil variability play an important role in their stability.

The effects of vegetation on the stability of slopes are well recognised.Vegetation affects slope stability through modification of the soil water regime, which in turn causes a variation in soil suction or pore pressure. Vegetation can also enhance the stability of a slope by root reinforcement .Wu et al. (1979) investigated the stability of slopes before and after removal of forest cover and concluded that the shear strength contributed by tree roots is important to the stability of slopes. The study indicated that vegetation could contribute shear strength to the slopes through root reinforcement .Wu et al. (1979) showed that slope failure would have occurred if the effects of vegetation were not taken into account in slope stability analyses.

Vegetation can influence the stability of slope when the roots act as reinforcement to the soil. Their contribution is dependent on the plant material used, the method of installation and their properties. In soil-bioengineering, vegetation is installed artificially to improve stability and a wide range of vegetation is utilized for the stabilization of slope. The geometry of the installed vegetation and its root system is often determined by the type of plants being used as well as the method of installation.

Generally, the properties of roots which are needed for the computation of soil-root interaction include the geometry of root and the strength properties. However, while data are available for a number of species, these are limited to the sites from which the data were obtained. Hence, extrapolation of the data from one site to another involves uncertainties and is only sufficient for approximate calculations in a number of cases and should therefore be verified by in situ tests, whenever possible (Mafian et al., 2009).

II.2. DEFINITION OF SLOPE

II.2.1 Natural slope

Natural slopes formed over long periods, geological and geomorphological processes. These slopes are only stable if the mass of soil has sufficient cohesion to withstand the forces of gravity. Changes in the pore water pressure, the geometry of the slope or work can lead to ruptures on these slopes. In the case of a small valley or valley traversed by a river, the vegetation can help prevent erosion at the toe of slopes, where soil is eroded by the action of waves in the watercourse. Stabilizing the foot of the slope vegetation may be sufficient to maintain the stability of the slope as a whole.

II.2.2 Artificial slope:

Slopes or artificial embankments are formed from natural rocks or brought materials to form dikes or dams. Vegetation can be used to stabilize the soil embankments, but is less used in dams where stability is more assured mainly by works of art (Escostab, 2001).

II.3 ROLE OF VEGETATION IN PREVENTING LANDSLIDES

Vegetative cover can contribute to improving the stability of steep slopes by reducing erosion, reducing direct infiltration from rainfall, and increasing the strength of the near-surface soil. Dense vegetation intercepts direct rainfall before raindrops impact the soil surface, thereby reducing or eliminating rains plash erosion. With dense vegetative cover and thick forest litter, the overland flow is also reduced in intensity and speed, lessening surface erosion. Thick vegetation, forest litter, and organic soils retain moisture from direct precipitation, and evaporate the water back to the atmosphere. Root systems can increase the strength of the soil they penetrate, reducing the likelihood of shallow landslides; and the deeper the roots, the better the protection in this respect. Native vegetation is best because it can be maintained without irrigation during the dry season. However, certain types of vegetation can have an adverse effect on slope stability, e.g. unstable trees can initiate a landslide if they are toppled during high wind conditions.

II.4 THE SLOPE INSTABILITY

Slope instability, also commonly referred to in the plural as slope failures or landslides, is a serious geologic hazard common to many regions of the world. Globally, landslides cause billions of dollars in property damage and fatalities and injuries running into the thousands each year.

Slope instability problems can be subdivided into two broad categories, namely, problems associated with the failure of natural slopes and failures associated with man-made slope (i.e., excavations or fills). There are a number of possible factors that can lead to the instability of a soil slope. However, in general, earthen slopes remain stable unless there are changes in the pore-water pressures in the soil comprising the slope. Changes in pore-water pressures are generally the result of water infiltration related the climatic conditions. Often it is the reduction in negative pore-water pressures in the upper 1 to 6 m that triggers slope instability (Zhang et al., 2004). Slope instability problems become a “hazard” that needs to be managed through the application of sound engineering principles.

The consequences of slope instability can be costly and even result in the loss of many lives (Fell, 1994). Any slope failure can result in substantial costs for remediation while in regions of dense population or areas prone to high velocity landslide, the loss of life can be considerable. Therefore, governments and private agencies are increasingly asked to manage the “hazard” of slope instability.

Natural slopes are subjected to inherent variability both in the soil and the vegetation. It is unlikely that the underlying soil profiles of natural slopes are completely uniform or homogenous. Even within a homogenous soil layer, soil properties tend to vary from point to point (Vanmarcke, 1977). The growth of vegetation is sensitive to environmental conditions and changes. Typically different types of vegetation grow on a natural slope, such as a mixture of grasses, herbs, scrubs and trees. Their differences in size and physical properties will affect the slope stability in different ways. Therefore, the use of a single input value for the vegetation dependent parameters in analyses is best viewed as a first approximation of the field conditions.

II.5 BENEFITS OF VEGETATION IN SLOPE STABILISATION

An enormous body of research concerned with vegetation and slope stability exist. Most of the literature supports the contention that , in the vast majority of case ,vegetation helps to stabilize a slope(Macdonald and witek1994).as Gray and Sotir 1982 remarked ,'the neglect of the role of woody vegetation(and some instances its outright dismissal)in stabilizing slopes and reinforcing soils is surprising'. Their summary of beneficial influences of woody vegetation follows:

- Root reinforcement – roots mechanically reinforce a soil by transfer of shear stresses in the soil to tensile resistance in the roots

- Soil moisture modification –evapotranspiration and interception in the foliage limit buildup of soil moisture stress. Vegetation also affects the rate of snowmelt, which in turn affects soil moisture regime.

- Buttressing and arching –anchored and embedded stems can act as buttress piles or arch abutments in a slope ,counteracting shear stresses .Gray and Sotir (1996) added a fourth beneficial effect.(the earlier work listed it as potentially negative)

- Surcharge –weight of vegetation can in certain instances, increase stability via increased confining (normal) stress on the failure surface.

Greenway (1987) concurred with the work above and notes that as vegetation is removed from a watershed, the water yield increases and water table levels rise in response to logging. These occurrences would tend to increase soil saturation and run-off. Zeimer (1981) states that "root decay after timber cutting can lead to slope failure.in situ measurement of soil with tree roots showed that soil strength increased linearly as root biomass increased".

Zeimer (1981) reports that live brush roots were twice as strong as conifer roots of the same size. (Woods, 1938; Menashe, 1993; Gray and Sotir, 1996) provide information on the effectiveness and use of herbaceous and woody vegetation in slope stabilization.

II.6 STUDIES ON IMPROVEMENT OF EARTH SLOPES BY REINFORCEMENT

The use of reinforcement to improve the behaviour of weak soil is not new. Early civilizations have utilized soil reinforcement in the form of straw, bamboo rods, reeds or similar alternate materials to reinforce mud bricks and walls of primitive houses. In spite of its long history, modern development of reinforced soil was first pioneered by Vidal(1969). Vidal developed the idea of reinforced earth where flat metal strips are laid horizontally in a frictional soil to provide the means of reinforcement. Due to its rapid success, reinforced earth is now used in the construction and repair of embankments and side slopes, roads, retaining walls and erosion control.

In most of the recent investigations, the effect of the reinforcement on the behaviour of the soil mass was studied through full scale models(Brown and Poulos, 1984),laboratory models (Duncan et al .,1970) pull- out tests (Chang et al, 1977b) direct shear tests (Jewell et al.1987)or by an equivalent homogeneous or a discrete finite element method (Mandal and Char, 1985).

The concepts of strengthening the embankment, stabilizing or improving the earth slopes by adding rods, fibres or using stabilizer materials are not new. Lee et al. (1973) and Ingold (1980) reported a brief review of this concept, such as the use of tree trunks and branches, sticks, reeds, straws and other reinforcing materials to strengthen or stabilize soil for projects such as culverts, river banks, dikes and other special uses.

Gray (1974) analysed and summarized available studies on the effect of woody vegetation removal on deep seated stability of slopes. Mckittrick and Darbin (1979) suggested a solution to the problem of needing large quantities of materials for embankment construction by eliminating the embankment side slopes and supporting the roadway platform on a reinforced earth structure.

Madhavi Latha et al. (1999) studied the advantages of geocell reinforcement on the performance of earth embankments constructed over soft foundation soils through laboratory model tests. The influence of various parameters like tensile stiffness of geocell material, aspect ratio of cells, length of geocell layer and type of fill material inside the cells on the load-deformation behavior of the embankment was studied. Geocell reinforcement was found to be advantageous in improving the load bearing capacity and reducing the deformations of the embankments. Slope stability analysis was conducted on all the experimental configurations of geocell supported embankments using a general-purpose slope stability program.

Ingold (1980) mentioned to the work of Long et al. (1972). They published the results of a series of triaxial tests carried out in the hope of defining the mechanism of reinforced earth. They studied the effect of reinforcement spacing (ΔH) as well as the effect of reinforcement tensile strength (T). The results showed that above a certain value of applied confining pressure, there was a constant increase '$\Delta\sigma_1$' in applied vertical stress at failure in samples with reinforcement at a given tensile strength (T) and spacing (ΔH) as shown in Figure.(II.1). It was concluded that the failure of both the reinforcement and unreinforced sand are parallel, and therefore, exhibit the same angle of internal shearing resistance. The additional strength transmitted by the reinforcement could be represented by an apparent cohesion c', as shown in Figure (II.2).

Ingold (1982) believed that, if the soil under the action of confining stress is (σ_3), then the same soil will have confining pressure of ($\sigma_3+\Delta\sigma_3$) when using the reinforcement with it, see Figure.(II.3). In this case, the failure will occur at much higher stress level of (σ_1) r.

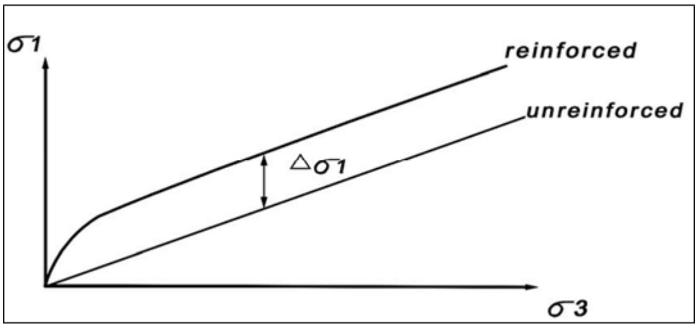


Figure.(II.1): Reinforcement induced cohesion (after Long et. al. 1972)

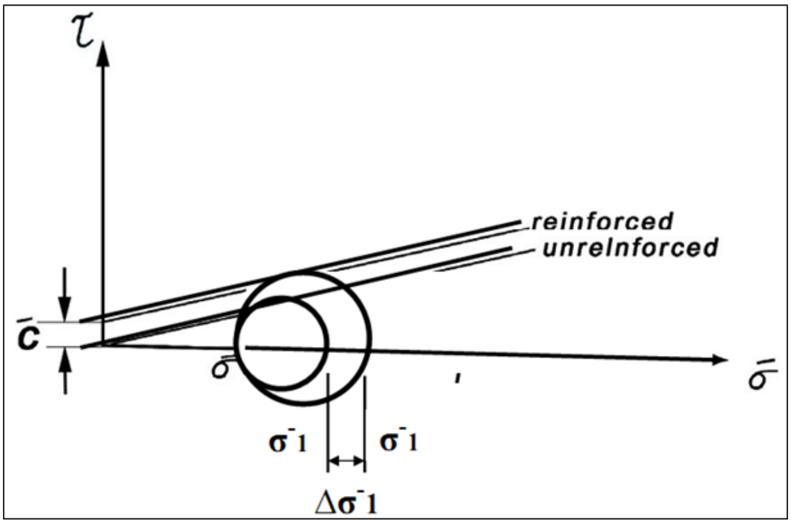


Figure.(II.2): Mohr circles for reinforced and unreinforced sand(after Long et. al., 1972)

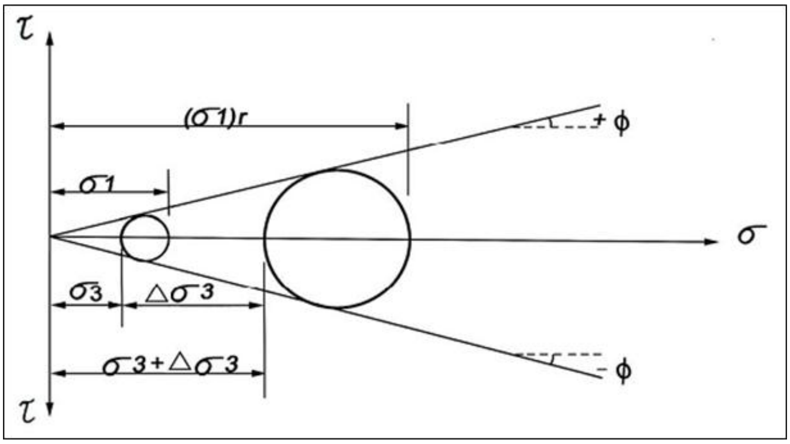


Figure.(II.3): Improvement of strength of soil when using reinforcement (After Ingold, 1982).

II.7 TREE ROOTS AND REINFORCED EARTH

Woody and herbaceous vegetation is commonly used to prevent surficial soil erosion (Coppin and Richards, 1990). Its influence on the processes of mass stability is less well appreciated although it is generally accepted that vegetation affects slope stability through six primary mechanisms (Gray and Leiser, 1982). These are:

1. Root reinforcement of the soil
2. Soil moisture modification
3. Buttressing and soil-arching
4. Surcharge weight of trees
5. Root wedging
6. Wind-throw

It is likely that the first four factors listed here generally aid in the stabilisation of a slope although the surcharge weight of a tree may have either a beneficial or adverse effect depending on such characteristics as its position on a slope, and the geometry and angle of the slope (see Styczen & Morgan, 1995). Both Abernethy & Rutherfurd (2000b) and Hubble (2001) modelled the effect of surcharge weight on riverbank stability and found that generally it had minimal effect. Root wedging and wind-throw will potentially have a negative effect on slope stability however their significance is largely unstudied and therefore unknown. Brown & Sheu (1975) developed a theoretical framework for assessing the effect of wind on trees and asserted that forces could be transmitted to the soil via the roots, thus increasing the likelihood of failure.

The factors listed above have been the subject of comprehensive reviews 5Gray and Leiser, 1982; Greenway,1987; Coppin & Richards,1990; Styczen & Morgan,1995; Wu, 1995) with a general consensus that the positive effects on slope stability far outweigh the negative. As root reinforcement and soil moisture modification directly impact upon soil strength it is suspected that they will have the greatest effect on slope stability. This research focuses only on root reinforcement of the soil as it has not been possible to assess both mechanisms within the constraints of a research programme of this nature.

II.8 HYDROLOGICAL ROLE OF ROOTS

Shallow slope failures can occur when the pore water pressure is increased and effective stress is decreased due to large rainfall events. Site-specific factors, such as “preferential hydrological flowpaths, slope steepness, soil thickness, and material properties” can also contribute to slope failure (Roering .2003). Roots are responsible for creating macropores and cavities in the soil thereby improving infiltration. However, an increasing rate of infiltration also leads to a higher water table, thus increasing seepage pressures (Ruebens.2007). The contiguous chain of macropores beneath the forest floor that transports subsurface water is known as pipeflow. Pipeflow plays a role in slope stability and landslide initiations “since the spatial variation in hydrologic response is attributed to the influence of pipeflow” (Uchida et al 2001).

Researchers have discovered that 50-90% of landslide scars contained soil pipes at the headscarps or origin of the slide. During intense rainfall events, closed ended soil pipes can cause slope instability by preventing the dissipation of water. This causes the pore water pressure to increase, thus lowering the effective stress in the soil mass. A majority of the slope failures in unsaturated conditions result from large rainfall and infiltration events. As negative pore water pressure is reduced, the shear strength of the soil decreases below the critical value along the potential slip surface, causing failure. When soils drain rapidly, suction occurs and creates negative pressure. The soil has no real strength. And will fail. Decreasing the degree of saturation would decrease the permeability of the soil (Budhu 2007). Increasing the degree of saturation in a soil mass causes an increase in permeability because the existing water film on the soil particles result in a lower frictional resistance to flow. If the soil is not completely saturated, the rate of flow would decrease as the inflow of water works to saturate the soil by filling the voids and forming thin films of water around the dry soil particles (Budhu .2007).

Material properties such as the type of soil and their grain size play a major role in determining the permeability of a soil mass and ultimately, slope failure. Permeability is important because it relates directly to pore water pressure. Fine-grained soils such as clay have much greater surface areas and thus absorb large amounts of water and cause swelling and undrained conditions, while coarse-grained soils are looser packed and have large void ratios. Permeability is indirectly controlled by particle size (Budhu .2007). Since void ratio is a function of particle size, fine particles that exist within the sand would interfere with water flowing through the relatively large pores between the coarse - grained particles. As the fine particles migrate and accumulate in the soil sample, the blockage of water flowing will increase and the result will be a decreased permeability.

II.9 MECHANICAL ROLE OF ROOTS

Roots provide mechanical support to a soil mass through its tensile strength, adhesive and frictional properties (Ruebens.2007). Roots growing perpendicular to the soil surface provide resistance to shearing forces acting on the soil. Roots extending parallel to the soil reinforce the tensile strength of the soil zone. A soil mass is reinforced not only by these two strengthening aspects but also in terms of the spatial distribution it occupies. Fine roots (1-2 mm in diameter) are a tertiary root system and represent less than 5% of a tree's biomass but provide more than 90% of the water and nutrient uptake of all roots (Schwarz et al. 2009). Coarse roots are greater than 2 mm in diameter and consist of 15-25% of a tree's biomass. They can be broken down into four classes: taproot, lateral roots, basal roots and adventitious roots (Schwarz et al. 2009). These classes can be subdivided to primary and secondary roots, with secondary roots stemming from primary roots that originate from the root system. There is documentation proving a positive correlation between fine roots and soil reinforcement but the same cannot be said of coarse roots as its data is unproven. The effectiveness of coarse roots highly depends upon its depth and spatial density. If the spatial density is not sufficient, the strengthening effect of the roots is negligible as the soil can easily move around the roots. In general, fine roots are more effective at soil reinforcement but for shallow slope stability, the advantage of fine roots is less obvious. The major factors that govern shallow slope stability are: number, size, tensile strength and bending stiffness of roots penetrating the failure planes (Ruebens 2007). A greater quantity of fine roots is more effective at reinforcing the soil than a smaller number of coarse roots since tensile strength increases as root diameter decreases. Furthermore, during a slope failure, fine roots tend to break off but remain fixed within the soil, while coarse roots can simply slip out. However, only coarse roots can penetrate great depths and firmly anchor the soil mass.

The effectiveness of mechanical slope stabilization depends on the depth of the weakest soil zone, the likely failure mechanism and the steepness of the slope (Ruebens.2007). The environment surrounding the soil plays a large role in determining the effectiveness of root fixation. Factors that hinder the growth of roots, including but not limited to rocks and a water table, reduce the significance it has on a slope.

The soil type also plays a significant role in determining the effectiveness of roots for the texture of the soil can influence the resistance of uprooting while the soil's nutrient level may dictate the spatial density and distribution of roots.

II.10. ROOT SYSTEM ARCHITECTURE

In order to assess the contribution of a plant's roots to a particular slope's stability it is necessary to know the morphology of the root system present. Despite the well-recognized importance of this fact (see Wu, 1995) the systemic morphology of tree roots is one of the least understood aspects of arboriculture (Helliwell, 1986). This is due mainly to observational difficulties and variation, not only from region to region, but to a lesser extent from tree to tree. Kozlowski (1971) observed that root structure as well as depth and rate of root growth are chiefly controlled by the rooting environment. Local soil and site conditions such as moisture availability, soil aeration, temperature, nutrient availability, and mechanical impedance, all affect the development of a plant's root system.

The major components of a tree's root system are illustrated in Figure (II.4) Comprehensive descriptions of root system morphology have been provided by Kozlowski (1971). The lateral roots are mostly found close to the soil surface while tap roots and sinker roots are to a large extent located close to the zone directly below the tree stem. Trees tend to have most of their roots in the upper layers of soil where the mass of laterals are located in what is often referred to as the 'root mat'. Although the lateral root system may play a role in binding the soil into a single mass, the main resistance to shear failure in slopes is provided by vertical roots which are more likely to intercept potential failure planes (Gray and Leiser, 1982). The depth to which vertical roots extend is therefore important and varies considerably between: a) species and b) rooting environment. Many tree species have the inherent capability to develop deep and far-reaching roots in the absence of restrictive soil or substrate characteristics (Stone and Kalisz, 1991).

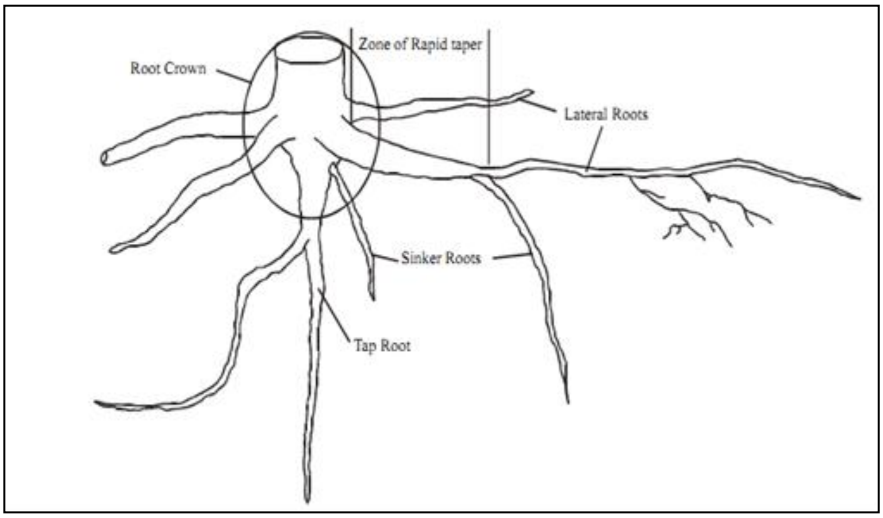


Figure II.4- Representation of the main root system parts (From Wu, 1995).

II.11 CAUSES OF SLOPE FAILURES

Slope failures are initiated by a variety of causes including: natural forces, human misjudgement and activities, and burrowing animals (Budhu. 2007). A slope failure in steep, mountainous landscapes can result in shallow landsliding (Roering, J.J. 2003). This is the common erosional process in these environments and is often comprised of colluvial sediments (Roering, J.J. 2003). As the debris flow accumulates along its long path downwards, it deposits sediment and scours the slope along the way. Shallow landslides can have large implications when it occurs near human values. Water quality and fish habitat are at risk, and in areas where unstable slopes border human activity, infrastructure and human welfare are at stake as well. The following are some of the common human and natural induced activities that compromise the stability of a slope.

✓ II.11.1 Erosion

The weathering and transportation of solids on natural slopes is a continuous process. Erosion alters a slope's geometry where it may lead to slope failure. In a forestry example, erosion is commonly seen when the soil is heavily compacted after harvesting. Forest harvesting exacerbates erosion by exposing mineral soil and removing the forest floor. The forest floor protects the underlying soil from the impact of rain drops and helps absorb water. Roads also lead to increased rates of erosion by changing the natural drainage pattern.

✓ II.11.2 Earthquakes

Earthquakes apply seismic loading that reduces the shear strength in soils. These shear Forces cause the grains in the soil to compact closer together, reducing the soil pores. Water then quickly fills the spaces between the soil grains. This occurs so quickly that even coarse-grained soils cannot dissipate the excess pore water pressures. This phenomenon is known as liquefaction. Sometimes the dynamic forces are so great that the pore water pressure is increased to values near total vertical stress, resulting in the total effective stress to approach zero.

✓ II.11.3 Rainfall

A slope experiencing prolonged periods of rainfall may be susceptible to failure. Rain saturates, softens and erodes soil by entering cracks in the soil and weakening soil layers due to increasing seepage forces. Failure in these cases can lead to mud slides.

✓ II.11.4 External loading

Loads placed on top of a slope add to the gravitational load and may cause a slope to fail. Conversely, loads placed at the toe of the slope, also known as a berm, increase the stability of the slope. Piling rocks, for example at the berm of a slope can help stabilize weak slopes.

✓ II.11.5 Tension cracks

Although tension cracks may not always be a significant factor in slope failures, they are worth mentioning because they are quite common. Firstly, a tension crack modifies the slip surface. When a tension crack is present, the slip surface intersects the base of the tension crack and not the base surface of the road (Budhu. 2007). Secondly, when a tension crack is filled with water, there is a hydrostatic pressure applied along the depth of the crack (Budhu. 2007). The result is a decrease in the factor of safety due to an increasing moment of force. Lastly, the tension crack provides an opening for water to seep through the slope and into underlying soil layers. This can induce seepage forces, which compromise the slope.

Figure II.5 - Tension crack on road surface

II.12 Conclusions

In this chapter we have seen that the use of vegetation in restoring the stability of slopes becomes highly demanded especially to solve the problem of shallow slope failure in both natural and man-made slopes. With variation in plant species that may be established on severe slopes condition, variation reinforcing trend can be observed. Several key factors have been identified that determine slope stability.

The role of roots in assessing slope stability can be narrowed to two factors: mechanical and hydrological. From a mechanical perspective, roots help to stabilize the soil through their tensile strength, adhesive and frictional properties. The strength and spatial distribution of roots within the soil are major variables to consider when assessing the degree of soil reinforcement influenced by roots (Nilaweera & Nutalaya, 1999). In terms of the hydrologic effects of roots, they aid in reducing the soil moisture and effectively dissipating the pore water pressure through evapotranspiration and water absorption through the fine roots. The role of roots in slope stability is an extremely important topic, especially when human lives and infrastructure are at risk.

LITERATURE REVIEW

Chapter Three

III.1 INTRODUCTION AND HISTORY

During the last decades there has been a pronounced increase in the number of catastrophic events including shallow landslides and erosion processes after heavy rainstorms, particularly in mountainous regions, which has raised public awareness of the hazard (Bezzola and Hegg, 2007).Slope instability is a serious geologic hazard common to many regions of the world. Globally, landslides cause billions of dollars in property damage and human fatalities and injuries annually. In tropical regions numerous landslides have been reported during the past four decades. It is therefore imperative that the slope problems need to be addressed urgently. The solution required to be comprehensive. An increased adoption of the bioengineering (vegetation) approach to the design of slope covers, taking advantage of the benefits of grass and woody on slopes with respect to erosion and stabilization, optimized slope drainage and improved slope maintenance appears to be the optimal way forward.

The public has recently become much more aware of natural hazards in general, and demand for security has increased. Thus, precise information about the effects of plants on slope stability is needed, and the development of methods to provide evidence for vegetation effects is an urgent objective. In particular, we need to find ways of including the influence of plants in conventional models for estimating the stability of natural slopes and embankments (Janbu,1954), which are generally based on the Mohr-Coulomb failure criterion ($\tau_f = c' + \sigma' \tan\varphi'$, in Lang et al. (1996). For this purpose, the vegetation effects need to be assigned to shear strength, i.e. to the angle of internal friction Φ' or to the cohesion c'.

To calculate and model vegetation effects on soil stability, suitable measurement techniques are necessary to properly address the relevant parameters. During the early history of soil mechanics, the direct shear test was the most popular approach, but it has some considerable disadvantages. To overcome some of its most serious limitations, the triaxial compression apparatus was developed in the 1930 (Casagrande, 1936). The triaxial compression test is more demanding and time consuming than the direct shear test, but it is also much more versatile. Several improvements have made it the appropriate choice, today, for experimental investigations of complex stress paths Lang et al. 1996).

Wu (1984) proposed, probably for methodical reasons, implementing the effects of plant roots on soil stability as an additive constant of the cohesion c' in the Mohr-Coulomb failure criterion (Mohr-Coulomb: $\tau_f = c' + \sigma'\tan\Phi'$; Wu: $sr = c' + \sigma'\tan\Phi' + c_r'$; with sr as the shear strength τ_f and r for root). This approach has the advantages that, as a plain strength value, the additional cohesion c_r' mobilised by roots, may be measured relatively simply by direct shear tests. Nevertheless, the soil stability conditions and, correspondingly, the effects of roots in the near-surface zone of the soil are not satisfactorily described by the cohesion c'. The stress-dependent term $\sigma'\tan\Phi'$ represents the proper characteristics much better. During a simple shear test in which the only stresses measured are the normal and shear stresses on horizontal planes. The results of laboratory tests on Cowden Till and on Blue London Clay (Atkinson et al. (1991) showed that the strength measured in simple direct shear tests differed from those measured in triaxial compression tests. The conventional interpretation of direct shear tests leads to a false cohesion intercept with friction angles smaller than those measured in triaxial compression tests (Atkinson et al. (1991).

Recent experimental investigations on fibre reinforcement in sand yielded controversial findings, depending on the method applied. Using direct shear tests, (Yetimoglu and Salbas,2003) found no improvement in the shear strength of the composite compared to pure sand, and (Operstein and Frydman ,2000) reported an essentially constant angle of internal friction of soil reinforced by roots, but an increase in the apparent cohesion with increasing cross-sectional area and tensile strength of the roots. Accordingly, they interpreted the general increase in shear strength of the composite as the result of an increase in cohesion.

However, analyses based on triaxial compression tests revealed an increase in the angle of internal friction of a composite (fibre reinforced sand) compared to the untreated granular matrix (Consoli et al. 2002). The addition of fibres to cohesion less pure sand yielded an increase in the angle of internal friction without any change in the cohesion but when added to cemented sand (with cohesion), the increase in the angle of internal friction went along with a decrease in cohesion (Consoli et al. 2002). Furthermore, it was found that the reinforcement effect generally correlates positively with the fibre aspect ratio, and, if the aspect ratio and concentration of fibres are kept constant, the composite strength is positively correlated with the length of the fibres (Michalowski and Cermák, 2003).

The triaxial compression test is better suited than the direct shear test at representing processes and characteristics of the superficial soil layers reasonably well. There is no rotation of the principal stresses, and, although stress concentrations still exist, they are significantly less. Normal stress is applied in three dimensions, and the area of shearing does not change during the test procedure. Furthermore, the failure plane can occur anywhere, and the stress paths can be controlled reasonably well. This means that complex stress paths in the field can be more effectively modelled in the laboratory. In particular, if undrained shear strength and the effective stress parameters of low-permeability material are needed, the triaxial compression test (consolidated undrained, with pore water pressure measurements) is by far more adequate (Holtz and Kovacs, 1981).

Various investigations on peaty soil have been performed using triaxial compression tests, showing that the behaviour of the peaty soil is essentially frictional, with high angles of internal friction (Φ') and relatively small cohesion (c') intercepts (Yamaguchi et al.1985).The high angles of internal friction are due to not entirely decomposed fibres intersecting the failure plane. This indicates that shearing resistance depends on the mutual orientation of fibres and failure plane. Undrained triaxial compression tests conducted by Yamaguchi et al. (1985) revealed significant differences in the angle of internal friction of samples taken vertically (Φ' = 52°) and horizontally (Φ' = 35°).

According to Wu et al. (1988), for roots to reinforce soil most effectively the vertical and horizontal growth should take up the stress applied.

Reinforced soils have been widely used in geotechnical engineering. The use of plants and their roots to protect slopes from erosion and shallow landslides is a useful and well-known natural bioengineering method that has been applied extensively worldwide. The most conspicuous vegetation source that enhances the stability of slopes is root reinforcement [Gray and Sotir, (1996). Gray and Sotir (1996)] and Reuben et al. (2007) showed that the shear strength increment provided by plant roots in the soil relied not only on the properties of the roots (root strength and soil–root interface properties) but also on the concentration, branching characteristics, and spatial distribution of the root system in the soil. It has been widely recognized and accepted that plant roots can improve soil shear strength (e.g. Waldron and Dakessian, 1981; Abe and Ziemer, 1991; Zhou et al., 1997; Operstein and Frydman, 2000) and stabilize slopes of surface soil (Ekanayake et al., 1997; Nilaweera and Nutalaya, 1999). Such functions turn vegetation into an effective ecological engineering tool to achieve harmony between humans and nature.

Several studies have shown that vegetation can positively affect slope stability, by influencing both hydro-geological processes and the mechanical structure of the soil (Wu and Watson, 1998; Schmidt et al., 2001; Roering et al., 2003; Bischetti, 2003; Normaniza and Barakban, 2006; Pollen, 2007).

There are two main mechanical effects of roots: the small size flexible roots utilize their tensile strength through soil–root friction increasing the strength of the compound matrix (soil-fibre) (Gray and Leiser, 1982) whereas the large size roots that intersect the shear plane act as individual anchors (Coppin and Richards,1990) and can tend to slip through the soil matrix without breaking, utilizing only a small portion of their tensile strength (Burroughs and Thomas, 1977; O'Loughlin and Watson, 1979; Schmidt et al., 2001;Pollen, 2007).

Wu et al. (1979) incorporated the effects of vegetation in slope stability analysis by using conventional limit equilibrium method. In limit equilibrium methods, the shear strength of the soil along a potential slip surface is assumed to be fully mobilized at the point of failure. The Mohr-Coulomb equation is used to describe the shear strength of the soil:

$$\tau = c' + (\sigma - u) \tan \varphi' \qquad \text{(III-1)}$$

By incorporating the effect of root reinforcement, Equation (III-2) becomes:

$$\tau = (c' + c_R) + (\sigma - u) \tan \varphi' \qquad \text{(III-2)}$$

Wu et al. (1979) incorporated the apparent root cohesion (c_R) in their infinite slope analysis and found an increase in the factor of safety (FOS) for some slopes. The results indicated that tree roots improved the stability of forested slopes.

Among the various approaches, the simplified models based on the equilibrium-limit of the strengths have been validated by in situ and laboratory studies (Wu, 1976; Waldron, 1977; Gray and Ohashi, 1983). These models can be used both in the evaluation of natural slope stability and in the area of works which will use plants covers. This method is based on the hypothesis that the root is cylindrical, linearly elastic, perpendicular through the critical slip surface and that the shear resistance angle of the soil is not influenced by the roots (Figure. III.1)

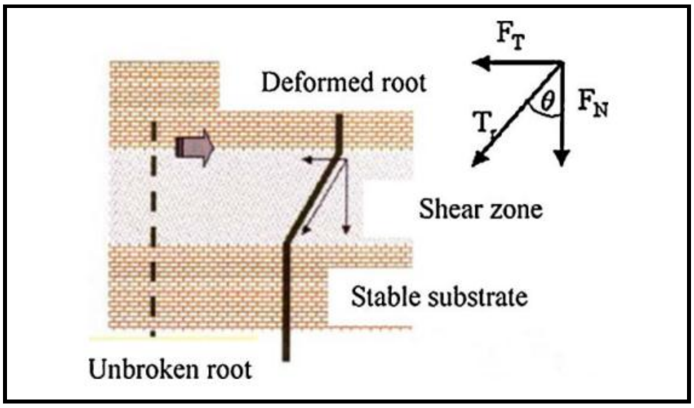


Figure .III. 1.Model of reinforcement with roots perpendicular to the shear area (Bischetti, 2003)

The shear strength of the roots is divided in F_T, a tangential factor (opposed directly to the shear stress) and in F_N a perpendicular factor that increase the normal stress:

$$F_T = T_R \sin\theta \qquad \text{(III-3)}$$

$$F_N = T_R \cos\theta \qquad \text{(III-4)}$$

Where θ = distortion angle of the root (that is variable) caused by the shear stress; T_R = tensile strength activated by the root.T_R is a passive strength that is mobilized with the displacement of the soil.

Comino and Druetta (2009) have shown that grass roots increase the shear strength of Alpine soil and delay the phenomenon of soil slipping. This outcome increases proportionally to the number of roots that cross the shear plane as well as their diameters. it was demonstrated that the soil should be fine enough to enable the roots to adhere strongly to the soil particles and to allow tensile stresses within the roots to be dissipated in the body of the soil.

The mechanism by which vegetation and especially trees, stabilize sloping soil has been studied in detail over the last 30 years (e.g. Wu, 1976; Waldron, 1977; Greenway, 1987; Gray and Sotir, 1996; Reuben's et al., 2007; Danjon et al., 2008; Stokes et al., 2007b, 2008b). Trees reinforce the soil matrix through their root system, by either increasing soil shear strength (Anderson and Richards, 1987; Coppin and Richards, 1990; Operstein and Frydman, 2000), providing structural support, or lowering the pore water pressures in the soil (Coppin and Richards, 1990; Gray and Sotir, 1996). The presence of plant roots results in an increase in apparent cohesion via root fibre reinforcement, which usually augments superficial slope stability (Schmidt et al., 2001). The most important parameters of the root system governing soil fixation are root density, depth and tensile strength (Wu, 1976, 2007; Nilaweera and Nutalaya, 1999; Operstein and Frydman, 2000; Roering et al.2003; Reuben's et al., 2007). These characteristics of root systems are determined by genetics, environmental and edaphic conditions. Root strength and density are influenced by species and site factor, e.g. local climate, soil characteristics, land use management and type of stand (Operstein and Frydman, 2000; Schmidt et al., 2001; Schmid, 2002). However, the influence of spatial and temporal variations in root parameters with regard to soil fixation on slopes is still lacking (Abernethy and Rutherfurd, 2001; Danjon et al., 2008), largely because root systems are difficult to sample (Böhm, 1979).

In conventional methods of reinforced soil construction, the inclusion of strips, fabrics, bars, grids etc are normally oriented in a preferred direction and are introduced sequentially in alternating layers. The development of these materials has been accompanied by an increase in the applications for which they are being used. The discrete fibres are simply added and mixed randomly with soil, much the same way as cement, lime or other additives. Fibre reinforced soil exhibits greater extensibility and small losses of peak strength i.e. greater ductility in the composite material as compared to unreinforced soil or soil reinforced with high modulus inclusions. Therefore, fibre-reinforced soil can be used as a soil-reinforcement technique with respect to embankment, subgrade, subbase, and other such problems. However, the data concerning the impact due to the addition of random discrete fibres on the characteristics of compacted native or virgin soils are limited, Maher and Ho. (1993).

Past research has demonstrated that inclusion of fibres significantly improves the engineering response of soils. Maher and Ho. (1994) studied the mechanics of fibre reinforcement in cohesion less soils and showed that inclusion of fibres increased peak shear strength and ductility of soils under static loads.

Gray and Ohashi. (1983) conducted an investigation on the influence of fiber reinforcement in sand. Direct shear tests were performed on dry sand with various types of natural and synthetic fibers. The concentration of fibers was given in a ratio of total cross-sectional area of fibers to shear cross-sectional area of soil. The average strength increase in the sand was shown to be approximately the same for sand in loose and dense soil conditions.

Laboratory triaxial compression tests were performed to determine the static stress–strain response of the compacted sandy soil reinforced with randomly distributed polypropylene fibers by (Consoli et al. 2003). it can be observed that the fiber reinforced specimens showed ultimate strengths that were significantly increased for all confining pressures.They also showed a marked hardening behavior up to the end of the tests at axial strains of more than 20%, whereas for the non-reinforced specimens, an almost perfectly plastic behavior was observed at large strains(figure III.2 a and b). On the other hand, the volumetric response was hardly affected by fiber inclusion, showing for both materials a slight expansion for low confining pressures, changing into an increasingly compressive behavior for higher confining stresses. They also concluded that the deviatoric stress of reinforced soil specimen increased with increase in fibre length, fibre aspect ratio and fibre content, whilst it decreased with increase in fibre diameter alone.

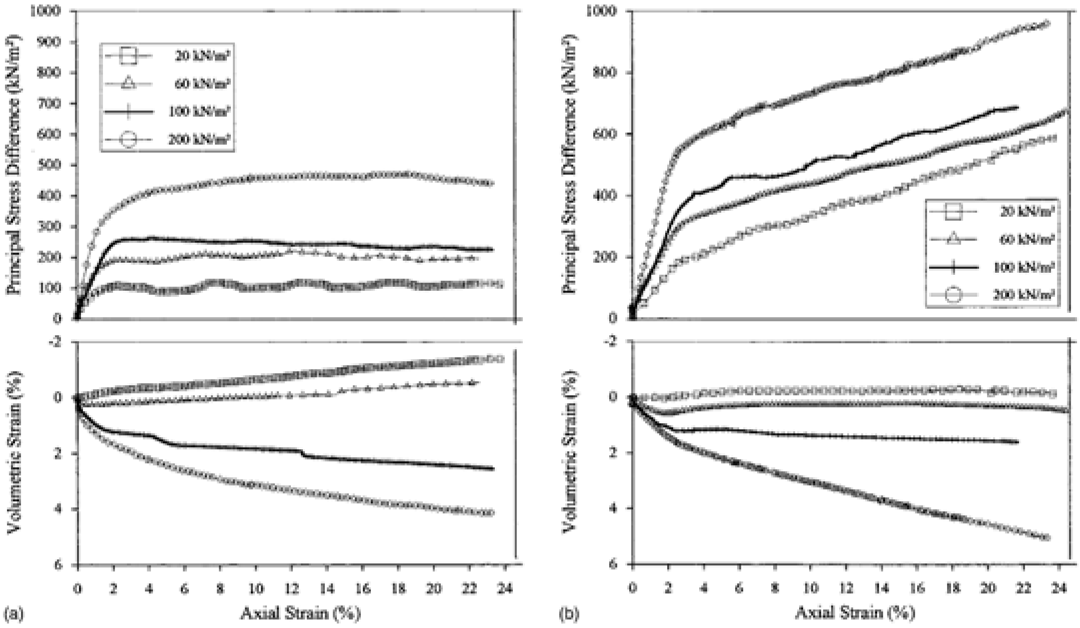


Fig.III.2. (a) Drained triaxial tests: compacted sandy soil and (b) drained triaxial tests: compacted fiber-reinforced sandy soil

This improvement of soil behavior due to fiber addition suggests the potential application of fiber reinforcement in shallow foundations, embankments over soft soils, and other earthworks that may suffer excessive deformation. See (Consoli et al 2003).

Gray and Al-Refeai (1986) described testing completed on fabric-reinforced and fiber-reinforced soil. The fibers used included reed fibers and glass synthetic fibers and the fabrics included commercially available geotextiles. The results showed that the strength increase in the soil was generally proportional to the amount of reinforcement, but the strength increase eventually reached a limiting value.

The use of random discrete flexible fibres mimics the behaviour of plant roots and gives the possibility of improving the strength and the stability of near surface soil layers. Diambra et al. (2009) have used a series of conventional triaxial tests in compression and extension were performed on reinforced and unreinforced sand with discrete crimped polypropylene fibres. They have observed that in triaxial compression the net deviatoric strength increase for 0.6% fiber content reaches 180% to 200%, but only 8% to 10% net strength increase is recorded for the extension tests (figure III.3-4).

In both compression and extension, the volumetric response for reinforced sand, as for the unreinforced sand, showed an initial compression followed by dilation. In triaxial compression, the addition of fibres resulted in a decrease to the amount of compression followed by a tendency to exhibit more dilation; the volumetric behaviour is clearly affected by the addition of fibres: dilation increases with the fibre content (figureIII-4).

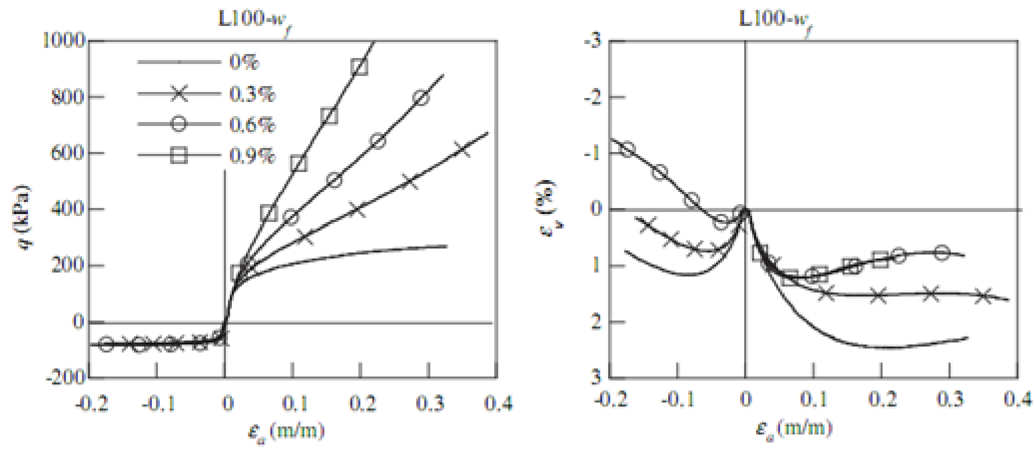
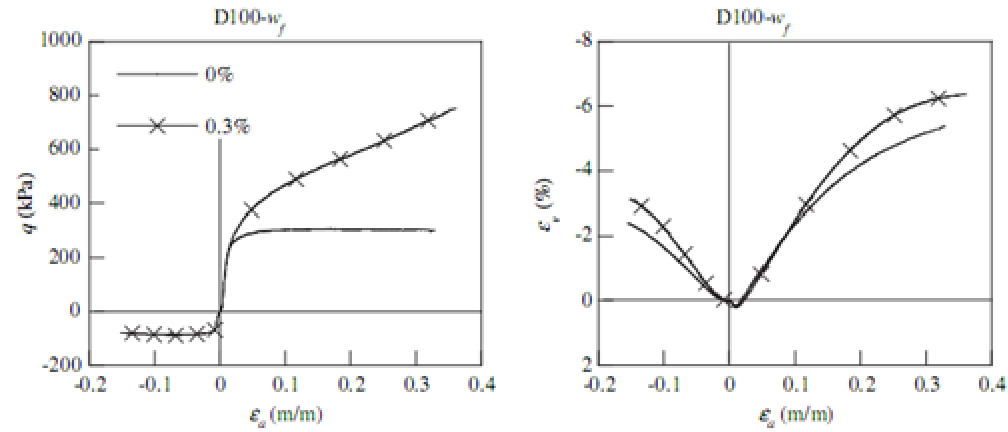


Figure .III. 3- Deviator stress–axial strain and volumetric behavior for drained (L) tests series confined at 100 kPa cell pressure (legend gives the fiber content used).



Figurre. III.4 - Deviator stress–axial strain and volumetric behavior for drained (D) tests series confined at 100 kPa cell pressure (legend gives the fiber content used).

They have indicated that dense specimens tend to dilate more than loose ones, inducing a greater desire for radial strain and therefore greater potential tensile stresses in the fibres which create an increased confinement on the sand in the dense specimens and hence a much larger increase in strength than observed for loose specimens.

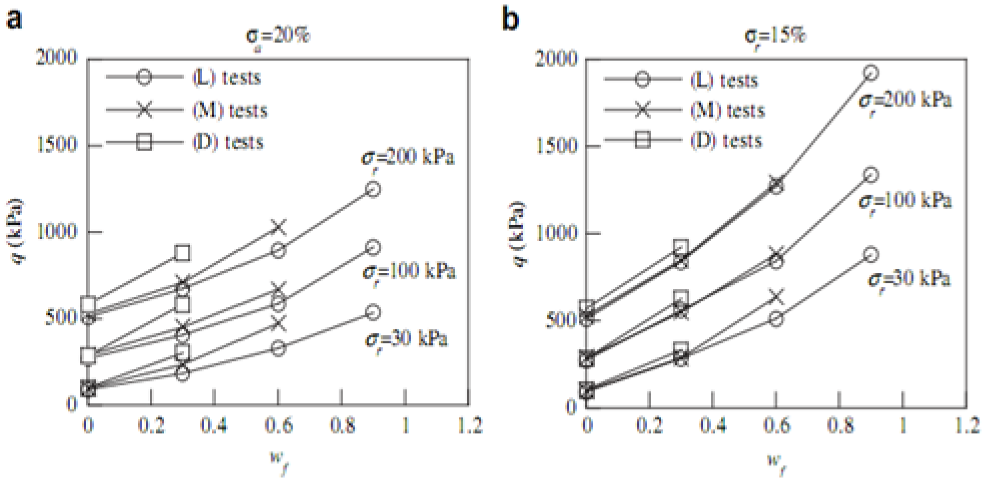


Figure III.5 - Deviatoric strength at 20% axial strain (a) and at 15% radial strain (b) for ensemble of specimen densities and three different confining stresses, 30, 100, and 200 kPa

Ingold (1979) used a triaxial apparatus to conduct research on reinforced cohesive soils. Ingold and Miller (1983) reported the results of undrained triaxial tests on Kaolin clay reinforced by aluminum plates and permeable plastic. Fabian and Fourie (1986) defined the effect of the permeability of the reinforcing material on the undrained strength of reinforced clay by conducting UU triaxial test on clay reinforced by materials with different values of permeability..

Ibraim et al (2009) have conducted a series of laboratory experiments the prospect of altering the undrained monotonic response of a loose clean sand to reduce its liquefaction potential by mixing the sand with short flexible fibres. Reinforcing sand with flexible discrete fibres does not represent a new technique in geotechnical engineering. However, no study has been reported concerning the undrained monotonic behavior of fiber reinforced sands or on the effect of fiber inclusions on the static liquefaction response of sand.

Monotonic loading in shear box tests, consolidated and unconfined drained triaxial compression tests have shown that shear strength is increased and post-peak strength loss is reduced when discrete fibres are mixed with the soil (Gray and Ohashi, 1983; Maher and Ho, 1994; Yetimoglu and Salbas, 2003; Ibraim and Fourmont, 2007, among others). The presence of fibres appears to prevent the formation of shear bands and loss of fabric in the directions of tensile strain (Ibraim et al, 2006).

The effectiveness of the reinforcement is influenced by fiber properties: type, volume fraction, length, aspect ratio, modulus of elasticity, together with orientation and also soil characteristics including particle size, shape, and gradation, as well as stress level and soil (matrix) density. At high confining stresses, the compressive strength of the reinforced sand appears to increase linearly with the concentration of fibres (the fibre concentration is conveniently expressed in terms of weight fraction of dry sand); for low values of the confining stress, this increase approaches an asymptotic upper limit (Gray and Al-Refeai, 1986; Al Refeai, 1991; Ranjan et al., 1996). Also, for a given fibre concentration, strength, as expressed by the major principal stress at failure, increases linearly with fibre aspect ratio (fibre length over fibre diameter). It has also been noted that for a given confining stress, the strength of the reinforced sand increases with reducing average grain size D50 (Gray and Al-Refeai, 1986; Maher and Gray, 1990). Also, an increase in coefficient of uniformity (Cu = D60/D10) results in higher contribution of fibres to strength.

Diambra et al. (2007a) found that the most common procedure for preparing reinforced specimens, moist tamping, leads in fact to preferred sub- horizontal orientation of fibres. The same conclusion is found for specimens prepared with vibration (Diambra et al., (2008a)). Ibraim et al. (2009) have used specimens for triaxial testing with diameter 70 mm and height 70 mm were prepared in three layers of equal height. The optimum moisture content of 10% was used for the fiber/sand mixing process, for all the specimens presented in this study, the quantity of sand, Ws, was kept unchanged when different proportions of fibres were added. For the specimens tested in compression, three different concentrations of fibres – 0.3%, 0.6% and 0.9% – were used whereas only two, 0.3% and 0.6%, were used for those specimens tested in extension.

Ibraim et al. (2009) have observed that while the volumetric responses for unreinforced sand, in both compression and extension, show initial contraction and only limited eventual dilation at large strains ,which is a typical pattern of a low density sand, the volumetric behavior of the reinforced sand approaches the characteristic response of a dense granular soil .After an initial reduction in volume, less significant than for the unreinforced sand, there is volumetric dilation with the dilatancy increasing with the fiber content. For a given fiber content, the dilatancy is higher in extension than in compression (see figure III.6a and b).

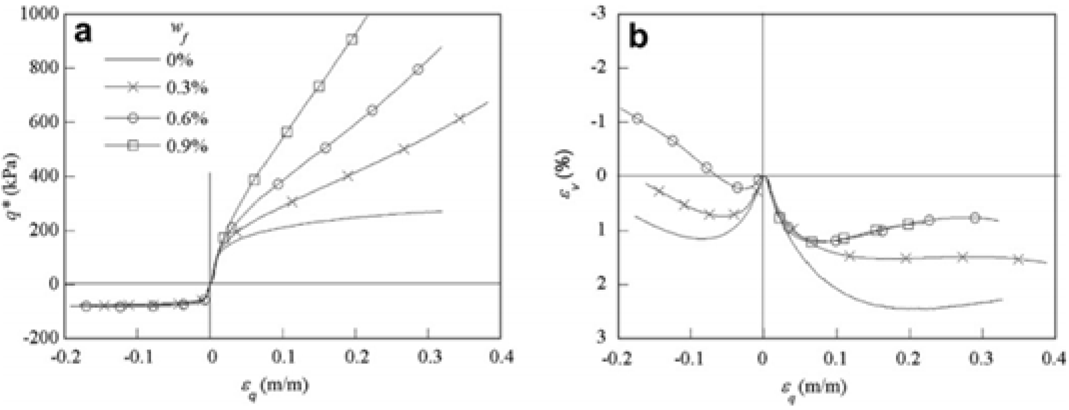


Figure III.6 . Deviator stress–shear strain and volumetric behavior for drained compression and extension triaxial tests on isotropically consolidated specimens at 100 kPa consolidation pressure (wf represents the fiber content).

The results showed that the strength increase contributed by the presence of fibres is highly anisotropic. Qualitative awareness of this actual orientation is needed to appreciate the difference in response in compression and extension. These results clearly suggest that the volumetric response of the composite could be a consequence of an apparent densification of the sand matrix resulting from the presence of the fibres in the voids: the fibres appear to steal some of the voids from the sand.

Ibraim et al. (2009) have concluded that the practical application of the use of flexible fibres to improve the liquefaction resistance of real soils will evidently require consideration of large scale methods of preparation of the sand–fiber mixtures and the possible costs of any compaction procedures used to produce particular initial densities.

Gray and Al-Refeai (1986) reported that the shear strength of reinforced earth could be determined by the tensile strength of reinforcing materials in his research on shear strength of fiber-reinforced soil using a triaxial compression test.

In order to evaluate influences of roots on soil shear strength Zhang et al (2009) was carried out a triaxial compression test to study the shear strength of plain soil samples and composites comprised of roots of Robinia pseucdoacacia and soil from the Loess Plateau in Northwest China .Roots were distributed in soil in three forms: vertical, horizontal, and vertical–horizontal (cross). All samples were tested under two different soil water contents.

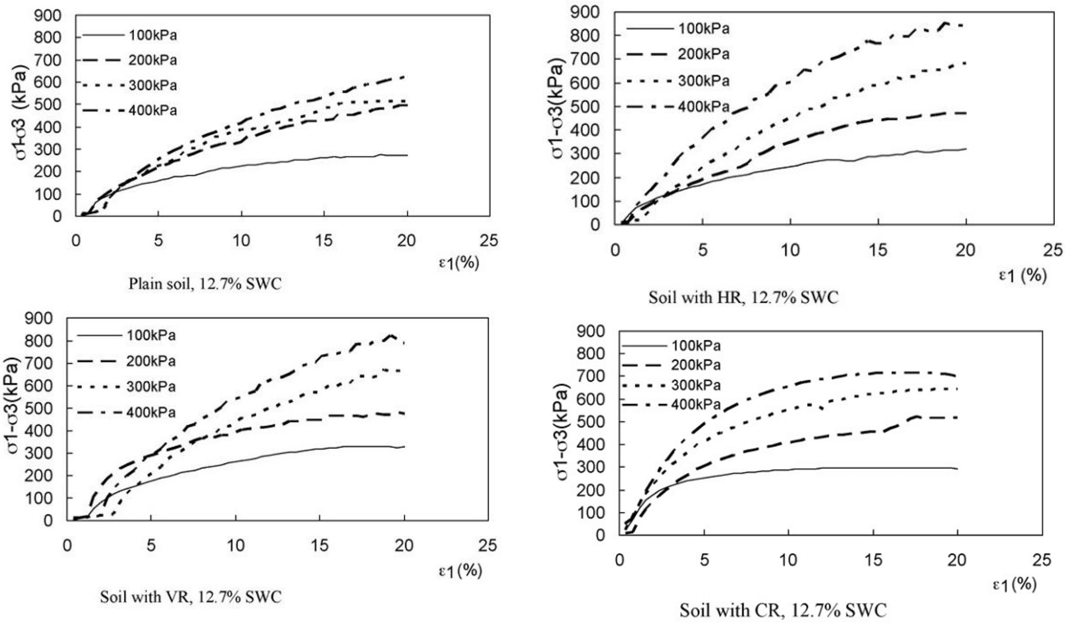


Figure III.7. Curves of principal stress difference versus. Axial strain

When the direct shear test is used to study the shear strength of soil–root composites, it is usually assumed that the internal friction angle is not affected by roots (Waldron, 1977). Results of the triaxial compression test demonstrate that roots are able to increase soil shear strength mainly through the increase in cohesion of the composite. The internal friction angle of the soil–root composite may increase or decrease in comparison to that of plain soil, but the final effect is an increase in shear strength (Zhang et al, 2009). They have observed among three forms of roots (HR, VR, and CR), effect of the CR on soil shear strength is the most considerable, since the CR can comprehensively enhance soil from both horizontal and vertical directions. Plants with roots of complex distribution pattern are expected to be more efficient in reinforcing soil and more adaptive to stabilizing slope.

In other hand they have concluded that soil water content (SWC) has strong influence on shear strength of soil. It has been found that an increase in SWC results in decrease in cohesion of soil, then reduction of the soil shear strength. Table 1.

- **Table 1**

Ultimate principal stress difference, (σ_1-σ_3)(kPa). (See Chao-Bo Zhang et al, 2009)

Confining pressure (kPa)	12.7% soil water content				20.0% soil water content			
	Plain Soil	Soil with HR	Soil with VR	Soil with CR	Plain Soil	Soil with HR	Soil with VR	Soil with CR
100	262	290	321	298	177	230	193	270
200	431	444	449	457	280	360	323	408
300	486	592	580	625	426	522	496	523
400	540	770	735	714	537	616	611	621

The change in SWC also has similar influences on shear strength indexes for soil–root composites as that for plain soil. The increase in SWC affects cohesion of composites more than friction angle (Table 2). The cohesion (C) of soil decreased noticeably with increasing SWC, while the friction angle (Ø) changes relatively small. The main reason is that the increase in water content significantly reduces cohesion between soil particles.

- **Table2**

Indexes of shear strength

Indexes of shear strength	12.7% soil water content				20.0% soil water content			
	Plain Soil	Soil with HR	Soil with VR	Soil with CR	Plain Soil	Soil with HR	Soil with VR	Soil with CR
C (kPa)	29	40	64	74	18	22	23	26
C_r (kPa)	–	11	35	45	–	4	5	8
ϕ (°)	27	26.6	23	23	22.3	25.3	23.7	26.2

Pollen also reported in 2007 that root cohesion is low when SWC is high and soil shear strength is low. Change in SWC affects not only mechanical properties of roots, but also soil shear strength.

The role of vegetation in the stability of slopes has gained increasing recognition in the functions of mechanical and hydrological mechanisms (Greenway, 1987).Vegetation has been known as a natural and helpful bioengineering method to protect slopes from erosion and shallow landslides and has also been used in practice throughout the world. The most conspicuous source that vegetation enhances the stability of slopes is via root reinforcement (Gray and Sotir, 1996). (Wu, 1976; Wu et al., 1979; Gray and Lieser, 1982) developed simple force equilibrium models for evaluating the additional shear strength that roots can provide in soils and can provide useful insights into the mechanism of soil–root interactions.

Rainfall is considered the major cause for most of the landslides. Shallow landslides are commonly seen on steep residual slopes during or after intense rainfall event. Normaniza et al. (2008) indicated that vegetation and the selection of plant species are important in stabilizing slopes and protecting against soil erosion in terms of its capacity of the root reinforcement and water absorption capacity. Precipitation, however, is known as the most important factor for triggering shallow landslides. Infiltration of rainfall in slopes lead to an increase in soil moisture content, especially in the near-surface. Normaniza and Barakbah (2006) suggested that both the soil moisture content and the root length density (RLD) could be used as indicators of slope stability.

Additionally, roots have little influence on the friction angle of root-reinforced soils with respect to that of root-free soils (Gray and Ohashi, (1983), and of the shear strength increase of root-reinforced soils with respect to root-free soils is equated to the increase in apparent cohesion (Waldron, 1977;Operstein and Frydman, 2000).

Gray et al. (1980) studied the combined vegetative-structural slope stabilization. Firstly, they mentioned the role of vegetation and their uses to reduce the side slope erosion or sliding. Herbaceous plants (grass and forbs or herbs) prevent erosion by direct interception of rain, by binding soil particles, by filtering soil from runoff, by dissipating the energy of runoff, and by maintaining good infiltration. Woody plants (trees and shrubs) likewise prevent surgical erosion but in addition they limit sliding or mass-movement by root reinforcement. They reported the combined approach, at which the slopes required a combination of vegetation and structural treatments as conducted by the Department of Environmental Horticulture, University of California. A simple illustration of a combined approach is the use of a low breast wall at the base of a slope to buttress and protect the toe and decrease the slope angle.

Gray and Sotir (1992) reported other slope stabilization schemes. These are a drained rock buttress, earthen brush-layer inclusions and a composite drained rock buttress and earthen brush-layer fill.

A number of researchers have investigated the effects of vegetation on slope stability. Plant roots, particularly of woody vegetation, are known to contribute to the shear strength of a soil-root system, and when located on a slope, such roots are believed to improve the stability of the slope .Gray and Leiser (1982) categorize the possible ways vegetation affects the balance of forces in slope as follows:

1) Root reinforcement –Roots penetrate downward and mechanically anchor the upper soils to the lower subsoil and bedrock.

2) Soil moisture modification – Evapotranspiration removes moisture from the soil, reducing the buildup of soil moisture.

3) Buttressing and arching –anchored and embedded stems act as buttress piles or arch abutments in a slope, resisting shear stresses.

4) Surcharge – Weight of vegetation in a slope induces a downslope stress, which reduces stability, and a stress normal to the slope, which increases the slopes resistance to movement.

5) Root wedging – Roots penetrate cracks and fissures in the underlying soil or rock, increasing instability through wedging or prying.

6) Wind throwing – Wind exerts force on a tree. This force produces a moment at the base of the tress. This moment acts to overturn the tree and detracts.

The importance of plant root systems to the stability of slopes has received considerable attention in recent years particularly on engineered cut slopes. Roots can influence slope stability through hydrological and mechanical factors .Several studies have recognized that root systems contribute to soil strength by providing additional cohesion (ΔC) and that they have negligible influence on the frictional component of strength [Endo and Tsurata(1969),Waldron(1977), Gray and Megahan (1981) ; Waldron and Dakessian(1981)].

Roots increase soil strength by providing direct resistance to shearing and by mobilising shear strength over a wider area through the transmission of tangential shear forces between roots and soil [Collison and Anderson (1996)]. The cohesion and friction component are increased by the presence of roots [Waldron and Dakessian (1981)]. The artificial cohesion caused by Alnus glutinosa with an average stem diameter of 16 mm increased in proportion to the fresh weight of roots per unit volume of soil [Endo and Tsurata (1969)]. Their data show that the presence of roots raised the soil strength between 5 and 10 kPa for root content ranging from 4 to 12 kg/m3 (fresh weight). A study through a modelling approach on soil-root system indicated that root strength (ΔC) contributed about 5.9 kPa to the shear strength of the soil [Wu et al., 1979].

Plant roots reinforcement in soil affecting both mechanical and hydrological properties. The mechanical reinforcement effect (increase of soil shear strength) is studied by modelling roots as fibers inclusion within the soil matrix. The mechanical properties of the root–soil system are regulated by a combination of soil strength, single root strength, the interface strength between soil and roots [Waldron and Dakessian, (1981); Waldron, (1977)] and the morphological characteristics of the root systems (Dupuy et al., 2005). The hydrological effects are studied investigating the relationship between soil water and root profile [Coppin and Richards, (1990); Gray and Sotir, (1996); Normaniza and Barakban, (2006)]. And the root water uptake [Fatahi et al., (2010); Mu'azu and Ali (2011)].

Several studies have documented the relationship between mechanical and hydrological factors: Pollen (2007) studied the effect of soil moisture content on the shear strength of root-reinforced soils; Normaniza et al. (2008) showed the connections between root reinforcement and water absorption capacity; Normaniza and Barakban (2006) suggested that both the soil moisture content and the root length density (RLD) could be used as indicators of slope stability.

Soil susceptibility or soil erodibility can be evaluated through laboratory tests on small soil samples, which are easy to implement and far less expensive and time-consuming than field experiments [Barthes and Roose, (2002]). Soil stability index is an important parameter that measures the rate of dispersion of soil aggregates in water and its magnitude can reflect the capacity of soil stability [Wang et al., (1994). Past researches have indicated that plant roots' extension, entanglement and adhesion in soil masses can enhance the stability of soil aggregate and improve the soil stability index [Wu et al., (1997); Reubens et al., (2007)]. Root length density is the total length of all roots within a unit soil volume [DeBaets et al.,(2006)], and it reflects plant root extension and entanglement in the soil body and promotes soil adhesion [Reubens et al., (2007)]. It provides an estimate of the total number of roots and is not skewed by the presence of large roots, as compared with other root structural parameters including root mass, root volume, or root area [Böhm, (1979)]. If vegetated soils are viewed as a fiber-reinforced composite material, the root length density represents the number of fibers in the sample.

Vegetation, through a living root network, has the potential to increase beach stability by decreasing the erosion rate on beaches exposed to fluvial forces by retarding the flow and increasing sediment shear strength through binding and buttressing of the tree roots [Rutherfurd, (2007)]. Roots growing in the soil surface reinforce the soil by increasing the in-plane tensile strength of the rooted soil zone and weakening the surface erosion processes. Deep roots, especially tree roots, extend to soil reinforce the soil by increasing shear-strength of the rooted soil mass on the sheared surface, thus reducing tidal current erosion and avoiding mass failure [Reubens et al., (2007)].

Wu (1976) and Wu et al. (1979) pioneered a model that has been applied in numerous studies for the assessment of how roots contribute to soil shear reinforcement. The impact of root reinforcement on soil is generally expressed as an increase in soil cohesion [Burroughs and Thomas, (1977); Wu et al., (1979); Wu (1995; Abernethy and Rutherford, (2001); Stokes et al., (2007b.2008b]). A number of factors influence the tensile strength test: species, season, age, soil compaction, deformation of roots, soil and root moisture, root preservation, field or lab test, type and size of testing equipment, procedure for clamping the root, test speed, and rate of elongation [Rienstenberg, (1994]).

The mechanism f the soil–root interaction and the contribution of plant roots to the shear strength of the soil have been studied both analytically [Waldron, (1977); Wu et al., (1979, 1988); Waldron and Dakessian, (1981)] and experimentally [Operstein and Frydman, (2000); Docker and Hubble,(2008)]. A simple root reinforcement model based on the force equilibrium principle has also been developed to evaluate the shear strength increment that can be provided by roots. This model has been applied to both vertical roots [Waldron, (1977); Wu et al., (1979)] and inclined roots [Gray and Leiser, (1982)]. However, experimental studies showed that the shear strength increment provided by plant roots was considerably less than that estimated using the simple root reinforcement model [Operstein and Frydman, (2000); Docker and Hubble, (2008)]. The mechanism by which plant roots contribute to shearing resistance inherently involves an underground 3-D soil–root interaction. The architecture or branching characteristics of the root system play an important role inmobilizing the increase in shearing resistance and in protecting the soil mass from erosion or shearing failure. The root system geometry and root topology determine the force transmission in the entire root system, which in turn affects the extent of soil reinforcement.

Docker and Hubble (2008) used in situ shear tests and reported RAR-based estimates of the increased shear resistance of soils due to the presence of four common Australian riparian tree species-Casuarina glauca, Eucalyptus amplifolia, Eucalyptus elata, and Acacia floribunda. At equivalent RAR values, the roots of A. floribunda were the greatest contributors of shear strength increment to soil blocks of these four plant species. However, these results may be influenced by the difference in both the geometry of the root system and the tensile strength of the roots across different plant species. Docker and Hubble (2009) further linked the RAR based shear strength increment provided by plant roots to the root architecture system of these four plant species. The RAR values of the root systems were measured in terms of the spatial distribution (vertical and lateral extent) below the ground surface. E.elata exhibited the highest RARs in soil zones beneath it, while E. amplifolia reinforced a greater volume of soil than the three other species. When the spatial distribution of RARs in the root system was taken into account, E. elata showed the highest values of increased soil shear strength followed by A. floribunda, E. amplifolia, and C. glauca.

Stokes et al. (2009) discussed how plant root traits affect the protection of slopes from shallow mass movement. This research indicated that root architecture (branching pattern) can significantly change the distribution of stresses and plastic strains within the soil medium, and affecting the resistance to pull-out. However, the aspects of the root architecture are not yet taken into account in the root reinforcement model. Thomas and Pollen (2010) demonstrated that root reinforcement in soils may vary considerably across plant species that differed in terms of root architecture and growing location (sloping versus horizontal surfaces).

Biogrout is a new soil improvement method based on microbial induced carbonate precipitation Whiffin et al (2007. Bacteria and reactants are flushed through the soil, resulting in calcium carbonate precipitation, causing an increase in strength and stiffness of the soil.

Biogrout can be applied to a wide variety of situations, in which it is desirable to change the properties of the subsoil (DeJong et al. 2009).

Biomediated soil improvement is an exiting new opportunity to improve the physical characteristics of soils and sediments. In the subsoil most processes will be occurring based on microbial activity especially polymer production or microbial induced precipitation as discussed by DeJong et al. (2009).

Soil reinforcement by roots is studied by considering the contribution of the tensile force in a root segment that intersects a potential slip surface in a root–soil system, where the roots mechanically reinforce the soil by transferring shear stresses in the soil to tensile resistance in the roots. Different types of root systems of plants can provide different strengthening effects on the stability of the slope via fibre reinforcement near the slope surface and deeper-binding soil structure effect through tap or lateral root networks. The anchorage of the roots and the improvement in slope stability depend on the properties of the root systems such as root distribution and tensile strength (Normaniza and Barakbah .2006) as well as soil conditions.

When the soil is permeated by fibres (as in the case of roots), the displacement of soil, as a consequence of shear tension, generates friction between soil grains and fibre surfaces, causing the fibres to deform and to mobilize their tensile strengths. In this way, some of the shear tension can be transferred from soil to fibres, producing a reinforcement of the soil matrix itself (Khalil nejad et al 2011).

On the other hand, vegetation can protect soil from erosion via foliage; also, they can draw water from soil via respiration and transpiration and consequently cause an increase in the soil suction by reducing the soil moisture, which will help increase the shear strength in soil, as discussed by Faisal et al. (1999).

III.1.1 THE USE OF GEOSYNTHETICS IN REINFORCEMENT

The use of various reinforcements to improve the tensile capacity of soils has been widely used in many soil structures, especially in the construction of reinforced earth walls, reinforced slopes, embankments on soft soils, vertical landfills and foundation soils. The interface friction between the soil and geosynthetics is a very important factor for design of these structures. The use of reinforcements will provide additional shear stress in the soil mass through the tensile force in the reinforcement, which will increase the strength of soil-reinforcement mass, and hence reduce the horizontal deformations, and thereby increasing the overall stability of the structure. Geosynthetics were first introduced as reinforcement material for reinforced soil structures in the 1970s (Holtz et al., 1977).

The main limitation to soil structure stability is the low strength of many cohesive soils. By reinforcing the soil with geosynthetics this problem is somewhat overcome. One of the most common geosynthetics materials used to reinforced soil is geotextiles. Several laboratorial and theoretical investigations have been conducted in this field, most of which are related to granular soils reinforced with geotextile, while limited studies have been made concerning cohesive soils reinforced with geotextiles.

Al-Omari et al. (1989) performed CU and CD triaxial tests in order to study the behavior of clay reinforced with geomesh. The mechanical and stress–strain behavior of cohesive soils reinforced with geotextile from a different perspective has been evaluated by Noorzad and

Mirmoradi (2010). It is also evident that the geotextile increases the axial strain at failure and also the residual strength ratio; meaning that the geotextile causes a decrease in the strength loss after the peak strength. It is important to notice that, non-woven geotextiles have a high axial strain at failure, and therefore it is nearly impossible for geotextiles to rupture during a traditional triaxial test. This point was confirmed by checking the geotextiles at the end of experiments. They have observed that the reinforced samples have higher peak strength in comparison the unreinforced soil, and as the number of geotextiles increases, the strength increases further (Figure .III.8). They have found that the Stress–strain behavior of soil improved with an increase in the number of geotextile layers, and the effect of geotextile type illustrated that the first type geotextile has a greater influence on the sample strength (Figure. III.9). The reason may be due to the difference in permeability of the two types of geotextiles.

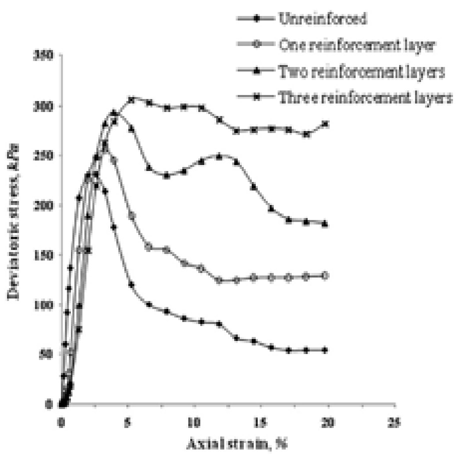


Figure. III.8 Stress–Strain curves for unreinforced and reinforced clay of type II with several layers of first type geotextile for the moisture content 22% and the relative compaction of 90%.

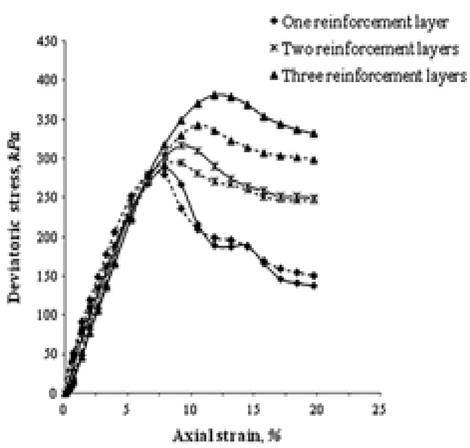


Figure. III.9. Stress–Strain curves for type I clay with relative compaction of 100% and moisture content of 20%: first type geotextile– second type geotextile.

Noorzad and Mirmoradi (2010) have concluded that Reinforcing improves the mechanical properties of soil, which means the existence of geotextiles increases the peak strength, axial strain at failure and decreases strength loss after the peak strength. Also, the reinforced samples are less stiff than the unreinforced ones. The improvement of mechanical properties increases as the number of geotextile layers increases. They also concluded that the comparison of samples reinforced by two different types of geotextiles provides evidence that the permeability of the geotextile may have an important role on the strength of the sample. The more permeable the geotextile, the higher the peak strength of the clay soil.

III.1.2 ROOT REINFORCEMENT TESTING

The behavior of fibre-reinforced soils has been studied by several investigators over the last two decades. Fibre-reinforced soil is becoming a viable soil improvement method for geotechnical engineering problems. Fibre-reinforced soils are currently being used or considered for applications that include stabilization of shallow slope failures (Gregory and

Chill 1998), construction of new embankments with marginal soils, reduction of shrinkage cracking in compacted clay liners (Rifai 2000).

A number of investigators have determined the influence of root or fibre reinforcement on the shear strength of soils by performing direct shear test on laboratory and field samples .Kassif and Kopelovitz (1968) tested a non-cohesive soil that contained synthetic fibres and compared the shear strength of the reinforced soil with the shear strength of the non-reinforced soil .they concluded that the fibre reinforcement increased the cohesion of the soil but had little effect on the internal friction angle .cohesion increased with increases in the surface area .bulk density ,and fixity of the fibres. Kassif and Kopelovitz suggested deformation and failure of a reinforced soil occurs in a number of stages .the first stage is the elastic deformation of both the soil and reinforcement. Next, the soil undergoes plastic deformation while the reinforcement continues to deform elastically.

Manbeian (1973) sheared soil columns reinforced with barley sunflower , and alfalfa roots .the plants were grown in large diameter containers of homogeneously packed silty clay loam.He then sheared both fallow and root reinforced samples in a direct shear machine .Manbeian reported that peak and residual shear strengths of the root reinforced samples were 2 to 4 times greater than the shear strength of the root free samples .these srength increases were ettributed entirely to the mechanical reinforcement provided by the roots since soil suction was eliminated by saturating samples prior to testing.

Wang (1997) used recycled carpet waste fibres for reinforcing soil as well as concrete. He concluded that waste carpet fibres increase the compression strength of soil and its ductility. He reported that fibre reinforced specimens exhibited significant increases in peak stress up to 303%.

Gray and Al-Refeai (1986) compared the effect of both continuous, oriented fabric layers and randomly distributed fibres on stress-strain behaviour of dry sand. The research outcomes demonstrated that, both fabric reinforced and fibre reinforced specimens show an increase in peak shear strength, axial strain at failure and in most cases limited reduction in post peak shear strength with increase in amount of reinforcement.

Kumar et al (2006) investigated the effect of adding polyester fibres in to soft clay soil by means of unconfined compression tests (UCS). They reported that there was a significant increase in unconfined compressive strength of highly compressive clay due to addition of polyester fibres and also the rate of increase in UCS value of soil increased with increase in length of fibres.

Fibre-reinforced soil is a mixture of soil and synthetic fibres. Synthetic fibres can be made of different materials, shapes and lengths. Polypropylene and polyester are the most common materials used to manufacture fibres. Fibres can be flat or round and continuous or discrete. Discrete fibres are manufactured in several lengths, ranging from 13-mm to 76-mm, and are available in different types such as monofilament, fibrillated, tape, and mesh.

Significant research has been performed over the last few decades to evaluate basic shear strength properties and deformation characteristics of fibre-reinforced soils. Previous work has shown that an increase in fibre content generally increases the shear strength of soil

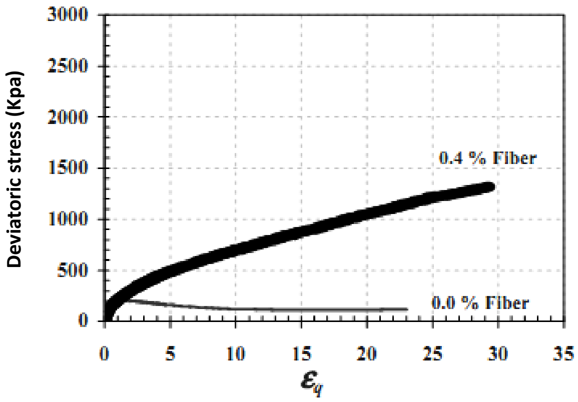
Most investigators have found that shear strength increases in direct proportion to fibre content or area ratio (Gray and Ohashi 1983; Gray and Al-Refeai 1986; 1989) observed that increase in strength was not proportional to the reinforcement concentration.

Some previous research has shown that inclusion of fibres increases both the cohesion intercept and angle of internal friction values as compared to values for unreinforced soil (Kumar et al. 1999; Gregory and Chill 1998). However, Gray and Ohashi (1983), Gray and Al-Refeai 1986), and Ranjan et al. (1996) found that inclusion of fibres did not significantly affect the angle of internal friction, but rather that fibre-reinforced specimens exhibited bi-linear failure envelopes as a result of the existence of a critical confining stress below which the fibres tended to slip or pull-out. Consoli et al. (1998) observed an increase in the angle of internal friction but a decrease in the cohesion intercept. Consoli et al. (2003) found the friction angle to be barely affected by fibre inclusion whereas the cohesion intercept increased with increasing fibre content

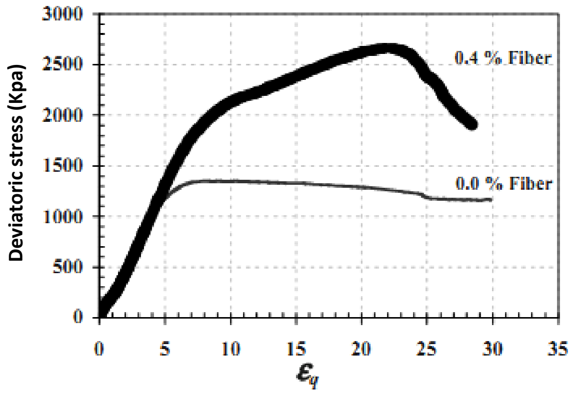
Al-Refeai (1991) studied the effect of three different fibres on fine and medium sands. He concluded that, fibres increase the peak principal stress of sand and this increase is proportional to the length of fibres. Ranjan et al (1996) investigated the effect of synthetic and natural randomly distributed fibres on sandy soils. Their test results indicated that there is a critical confining pressure bellow which fibres tend to slip. The critical confining pressure is a function of fibre aspect ratio. They also concluded that the shear strength of the reinforced soil increases with increase in fibre inclusion.

A series of consolidated-undrained and consolidated-drained type triaxial compression tests were performed on comparable unreinforced and fibre-reinforced specimens of Ottawa sand to evaluate the effective stress-strain-pore pressure and effective stress-strain-volume change behavior of fibre-reinforced sands (see Chen and Loehr (2008). The fibres utilized in these tests are polypropylene fibres.

The results of the triaxial tests performed on loose and medium-dense Ottawa sand have been found by Chen and Loehr (2008) show that inclusion of fibres can improve the strength of soils under undrained and drained loading conditions. (Figures III.10 and 11). It was shown that fibre reinforced specimens must deform before developing and increase in shear strength due to the inclusion of fibres.

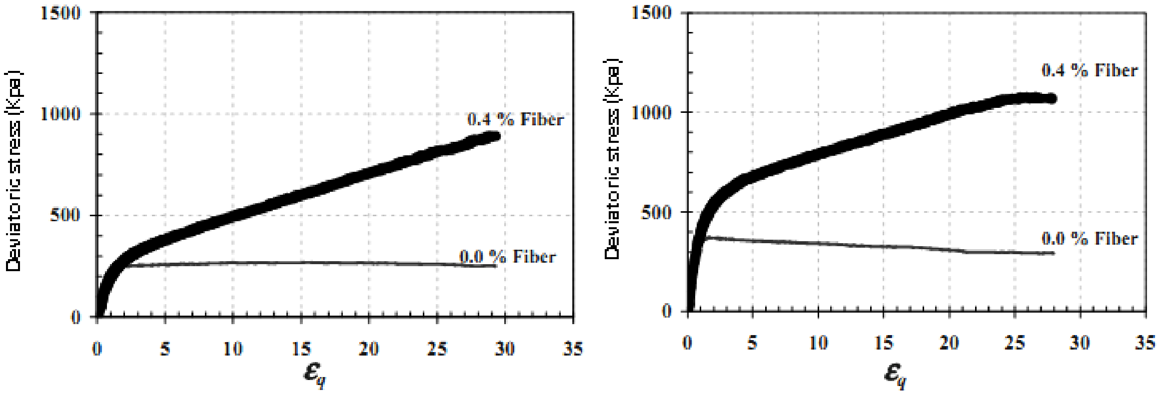


(a) Loose state (D_r≈10%)



(b) Medium-dense state (D_r≈55%)

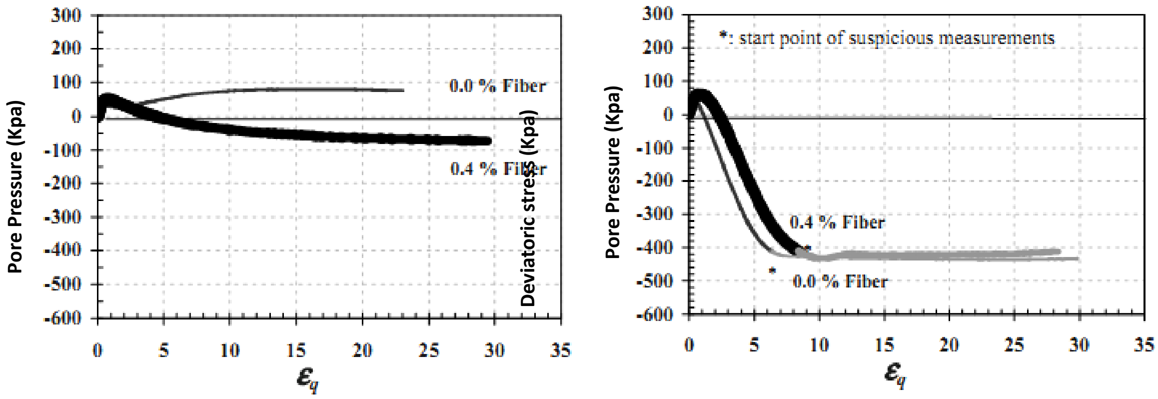
Figure. III.10. Deviatoric stress (q) versus triaxial shear strain (εq) curves from CU tests for specimens consolidated to 140-kPa effective stress and prepared at: a) loose state, and b) medium-dense state



(a) Loose state (D_r≈10%) (b) Medium-dense state (D_r≈55%)

Figure. III.11. Deviatoric stress (q) versus triaxial shear strain (εq) curves from CD tests for specimens consolidated to 140-kPa effective stress and prepared at: a) loose state, and b) medium-dense state

Chen and Loehr (2008) also have found In CU tests, loose reinforced specimens exhibited lower pore pressures than comparable unreinforced specimens. It is noted the reinforcing fibres alter the pore pressure response of specimens tested under undrained loading conditions and the volume change response of specimens tested under drained loading condition.



(a) Loose state (D_r≈10%) (b) Medium-dense state (D_r≈55%)

Figure III.12- Change in pore pressure (Δu) versus triaxial shear strain (εq) curves from CU tests for specimens consolidated to 140-kPa effective stress and prepared at: a) loose state, and b) medium-dense state

Significant fundamental research has been performed over the last few decades to evaluate basic shear strength properties and deformation characteristics of fibre-reinforced soils. Previous work has clearly shown that an increase in fibre content increased the shear strength of the soils. Most investigators found that shear strength increased in direct proportion to fibre content or area ratio (Ranjan et al 1996); Maher and Gray 1990; Gray and Al-Refeai 1986). However, (Shewbridge and Sitar .1989) observed that increase in strength was not proportional to the reinforcement concentration.

Inclusion of fibres was generally found to increase the peak and post-peak strength, as well as the strain at failure. Furthermore, inclusion of fibres has been found to not noticeably affect the initial stiffness of the unreinforced specimens. However, some investigators have reported an increase in the initial stiffness of specimens with increasing fibre content [Nataraj and McManis .1997), whereas others have shown a decrease in initial stiffness with increasing fibre content (Consoli et al .1998; Michalowski and Cermak .2003).

III.1.3 FIBRE REINFORCED SOIL

The use of vegetation by civil engineers when dealing with unstable slopes has become increasingly popular over the last 20 years (Coppin and Richards, 1990; Gray and Sotir, 1996; Greenway, 1987; Roering et al., 2003) . In particular, trees and woody shrubs have been studied with regards to the soil reinforcing properties that their root systems convey to slopes subject to erosion or slippage problems (Schmidt et al.,2001; Wu, 2006). If the root system characteristics, which govern soil stabilization, could be better identified, screening of suitable species for use on unstable slopes would be more efficient.

Vegetation has been recognized as a factor useful for increasing the shear resistance of soil on an unstable slope (Anderson and Richards, 1987; Coppin and Richards, 1990; Operstein and Frydman, 2000). The major factors which influence the shear resistance of root-permeated soil are the quantity and directional distribution of roots as well as their tensile strength, soil shear strength and soil–root interaction. Strength is the maximum force per unit area required to cause a material to break (Niklas, 1992). Tensile strength is considered one of the most important factors governing soil stabilization and fixation, and has therefore been studied in great detail (Burroughs and Thomas, 1977; Nilaweera and Nutalaya, 1999; Operstein and Frydman, 2000).

Wide variations in root tensile strength have been reported in the literature, and appear to depend on species and site factors such as the local environment, season, root diameter, and orientation (Gray and Sotir, (1996). Root resistance to failure in tension can be influenced by the mode of planting e.g. naturally regenerated Scots pine (Pinus sylvestris L.) had stronger roots than those of planted pines (Lindstrom and Rune (1999). The time of year has also been found to affect tensile strength, roots being stronger in winter than in summer, due to the decrease in water content (Turmanina, 1965). Tensile strength usually decreases with increasing root size (Burroughs and Thomas, 1977; O’Loughlin and Watson, 1979; Operstein and Frydman 2000; Turmanina, 1965; Wu, 1976) and this phenomenon has been attributed to differences in root structure, with smaller roots possessing more cellulose per dry mass than larger roots (Turmanina, 1965).

Generally, the more known about a plant and its root system and the conditions that limit growth, the better the estimate will be of its contribution to soil reinforcement. Knowing growth performance parameters of individual plants allows interspecies comparisons to be made and predictive models to be developed and used to optimise species mixes to meet specific revegetation goals. However, complete information is not always possible because collection of root data is time - consuming and expensive. What are needed are easily obtainable measures or surrogates for determining stabilisation effectiveness. Work in New Zealand hill country has previously suggested a number of vegetation parameters that govern plant performance for stabilising land prone to landslides (Phillips et al. (2000a, b), (2001). Such parameters include canopy occupancy, root occupancy, root depth, root biomass, and root cross-sectional area per shear area (sometimes called root area ratio (RAR);(e.g. Stokes et al. 2009), and root tensile strength. In terms of roots and soil reinforcement, there is a well-established literature on the benefits of trees, shrubs and grasses for reinforcing soils to control or reduce both surficial and mass-movement erosion ([e.g.Wu et al. 1979; Greenway 1987; Gray and Sotir 1996; Stokes et al. 2008.b) and for stabilising riverbanks (Abernethy and Rutherfurd .2001; Pollen .2007; Docker and Hubble .2009; Pollen and Simon 2010). Vegetation does this by both mechanical and hydrological processes (e.g. Greenway 1987; Pollen and Simon (2010). While the relative importance of hydrologic and hydraulic processes vs.

mechanical root-reinforcement on stream banks has been difficult to determine and varies with importance at different times of the year, the effect of mechanical root reinforcement on soil stability can still be considerable(Pollen and Simon (2010). This mechanical contribution to soil strength is by way of an additional apparent cohesion cR (Stokes et al. (2008b). However, while the contribution of root reinforcement enhances stability at shallow depths, in deeper soils the effect of root cohesion diminishes, usually because the number of roots declines with depth (Gray and Sotir (1996). The ability of a tree to reinforce soil also depends on the morphological characteristics of its root system and the strength of the combined root-soil system (Stokes et al. 2008) and is limited by a range of factors including individual root strength, the soil-root bond, and the distribution of roots about the stem (Greenway.1987; Coppin and Richards 1990). Further, the spatial distribution of a root system also varies enormously between and within species and in response to environmental conditions (Roering et al. 2003). In shallow soils, roots may penetrate the entire soil mantle and anchor the soil into more stable substrate (e.g. Wu et al. 1979). Where plants form a dense membrane of lateral roots, the upper soil horizons can be stabilised (e.g. Schmidt et al.(2001). Deep roots (>2 m) while not common in many hill slope situations, can occur on river banks and have been known to occur as deep as 20 m or more in Australia as a result of species evolution to seek permanent summer water tables (Hubble et al. 2010).

III.1.4 RIVERBANK STABILITY

III.1.4.1 Introduction

It is generally accepted that tree roots can reinforce soil and improve the stability of vegetated slopes. Tree root reinforcement is also recognised in riverbanks although the contribution that the roots make to bank stability has rarely been assessed due to the reluctance of geomorphologists to examine riverbank stability by geomechanical methods that allow for the inclusion of quantified biotechnical parameters.

Previous studies have demonstrated that the strengthening of riverbank soils can increase the resistance of the channel to morphological change both in terms of hydraulic geometry (Smith, 1976) and lateral stability (Hubble (2001) who used simple and generalised geomechanical models to demonstrate that the removal of vegetative root reinforcement was a major contributor to the widespread bank collapse between 1947 and 1992.previous analyses [Abernethy & Rutherfurd, (2000a, b; (2001); Hubble, (2001)] were dependent on conservative estimates of vegetative earth reinforcement inferred from studies mostly undertaken on exotic species growing in different environments, rather than on direct measurements of species extant within the study area. Given the large variation in earth reinforcement values measured between different environments (see Wu, 1995) there are clear concerns regarding the utility and accuracy of these previous stability analyses.

III.1.4.2 Stability Analysis of Riverbanks

Riverbanks are essentially a class of slope and so many of the principles of traditional slope stability analysis are applicable to them (see Thorne & Osman, 1988a). Riverbanks are however characterised by very different hydrological processes than hill-slopes and due to their mostly smaller length to height ratio and more varied profile, are influenced to a greater extent by the spatial variability of vegetative effects (Abernethy & Rutherfurd, 2000a). Therefore despite the general theoretical agreement concerning the stabilising influence of riparian vegetation on riverbanks (Hubble & Hull, 1996; Abernethy & Rutherfurd, 2000a) the lack of knowledge concerning the variation in total volume and spatial distribution of tree roots within the bank material is a serious limitation on their assessment by geomechanical means.

The incorporation of biotechnical factors in riverbank stability analysis has rarely been attempted and it has been uncommon even for a geotechnical approach (see Thorne & Osman, 1988a) to riverine morphological change to be pursued. Only three major studies of this type undertaken on Australian rivers have been identified (Hubble & Hull, 1996; Abernethy & Rutherfurd, 2000a; Hubble, 2001). They focused on root reinforcement as the most important vegetative factor influencing riverbank stability. All three reports significantly increased factors of safety under vegetated conditions, though only Abernethy & Rutherfurd (2001) measured the actual amount and distribution of increased shear strength within the riverbank.

III.2 REVIEW OF ROOT REINFORCEMENT THEORY

III.2.1 Curent Soil Stabilization Technologies

Current Soil Stabilization Technologies the shear strength of a soil influences the stability of the structures it supports. The shear strength, τ (tau), of a soil is the internal resistance per unit area that a soil mass can provide to resist failure and sliding along its plane. Most geotechnical failures involve a shear-type failure which is determined by the nature of the soil. Soil is composed of individual particles that slide when the soil is loaded. The characteristics of the different types of soil particles and their proportions in the soil establish the amount of cohesion and friction between particles. Mohr-Coulomb's equation describes the relationship between shear strength and normal stress, angle of friction and cohesion (Day, 1999):

$$\tau = c' + \sigma' . \tan \theta' \qquad \text{(III-5)}$$

As Equation (III-5) presents, the shear strength of a soil has a direct relationship to the soil cohesion, c', and angle of friction, θ' . When the maximum shear resistance of a soil is reached, the soil is regarded as having failed. The total stress on any plane can be determined by the normal stress, σ', which acts perpendicular to the surface and the shearing stress which acts along the surface. The shear strength of a soil depends on its moisture content and its compaction level.

A stable slope can be defined as a slope where the forces available to resist movement within the soil are greater than the forces driving movement. Slope stability encompasses the analysis of static and dynamic stability of embankments and natural slopes. In order to establish the stability condition of a slope, the slope's factor of safety is calculated. The factor of safety is the ratio of the forces resisting movement to the forces driving movement. If the factor of safety (FS) is equal or greater than 1, then the slope is stable, and if the factor of safety is less than 1, then the slope is unstable. The planes along which the factor must be calculated in slope stability analysis are usually irregular, which makes the process very complex. A method for determining the factor of safety in a slope is Bishop's Method (Coduto, 1998).

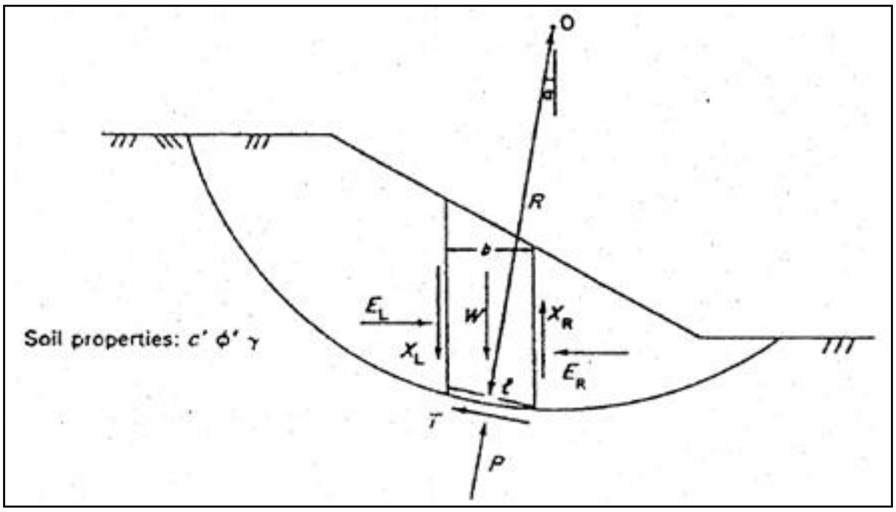


Figure III.13- Bishop's Method. Copyright Tsushida, 2002.

The Simplified Bishop's Method is a method for calculating the stability of slopes. The method can produce factor of safety values within a few percent of the correct values. The Simplified Bishop's Method is as follows:

$$F = \frac{\sum[\frac{c'+((W/b)-u)\tan\phi'}{\psi}]}{\sum[(W/b)\sin\alpha]} \qquad \text{Where} \qquad \psi = \cos\alpha + \frac{\sin\alpha\tan\phi}{F} \qquad \text{(III-6)}$$

C': is the effective cohesion

Φ : is the effective internal angle of internal friction

b : is the width of each slice

W; is the weight of each slice

u :is the water pressure at the base of each slice

α: is the slope angle

Equation (III-6) must be solved iteratively because it contains F on both sides of the equation. Since the process to reach convergence can be long and tedious, several simple programs exist to model Bishop's method in different circumstances. According to this method the failure occurs along a cylindrical slip surface generated by the rotation of a block of soil around a center point O. The method obtains the factor of safety of the slip surface by evaluating the whole system moment equilibrium about O. This is a simplified method because all the inter-slice forces are assumed horizontal.

Currently there are several methods to improve slope stability. We searched for a method that could be implemented in our study area. This required that we focused on methods that were relatively inexpensive and that required minimum machinery. The following methods align the best with these requirements:

✓ Steel wire reinforcement: This method consists of dividing the soil in compacted layers and then reinforcing each layer with steel wire mesh. The forces that the mesh induces into the soil depend on the mesh geometry, frictional characteristics, vertical soil pressure on the strip, and strength and stiffness characteristics of the strip. The mesh should be designed to include a layer of steel that will corrode during the expected life of the mesh preventing loss of critical mesh cross-area. The durability of this soil reinforcement relies on the ability of the mesh to retain a pre-established level of tensile strength. The following comparison illustrates how the reinforcement works: Figure III.14-A illustrates that if a vertical stress is applied on unreinforced soil it deforms both laterally and vertically until it reaches a new equilibrium. Figure III.14-B illustrates that if a vertical stress is applied to a mass of soil reinforced with metal sheets on plane perpendicular to the normal stress, the soil deformations are constrained due to the interaction between the soil and the mesh.

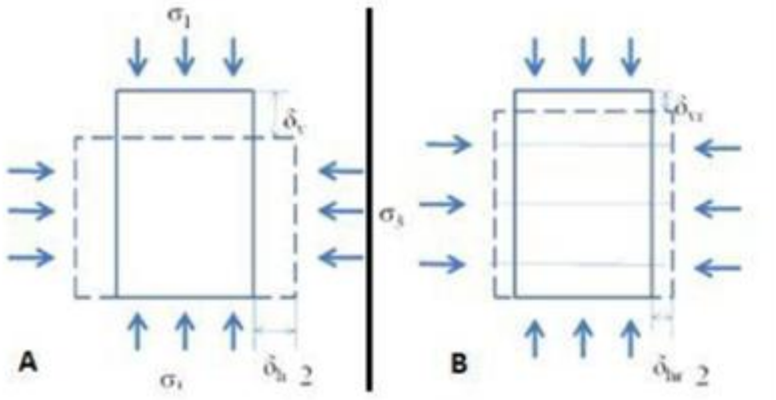


Figure III.14- Distribution of stresses in SWR

One of the advantages of using this method is that the construction materials are light, easy to transport and quick to construct. Other advantages are the only machinery required is a backhoe and a compactor, and it’s not extremely expensive. Disadvantages of this method are that it cannot be implemented in soils with a high content of silt and clay, and it is very difficult to apply it to extensive sloped areas. Another problem is that it has detrimental impacts to the environment at the end of its useful life because the corroded steel is toxic to the environment (Pereira, 1994).

✓ Geo-synthetic reinforcement: This method consists of dividing the soil in compacted layers and reinforcing each layer with geo-synthetics. The synthetics are used in two ways during slope reinforcement. The first approach is to provide increased lateral confinement at the slope face by placing narrow strips at the edge of the slope. This prevents sloughing and reduces erosion. In cohesive soils special geo-textiles with great drainage capabilities allow for rapid pore pressure dissipation. The second approach is to insert strips of the synthetic perpendicularly to the normal stress plane. The tensile capacity and orientation of the layers that intersect the slip surface increase the resisting moment occurring here. Advantages of this method are that the material allows for good filtration and drainage, it is very flexible, and its manmade properties gives the synthetic a long durability. Its durability has been calculated between 500 and 5000 years, although its strength characteristics have to be adjusted periodically. These properties allow for this method to be applied in all types of soil. However, the materials are not readily available to poor communities, plants cannot grow through them, the implementation has average costs and its implementation in large sloped areas is complex (Holtz, (2001).

✓ Adding lime to the soil: This method consists of mixing lime with the soil to increase the load bearing capacity of the soil. The most improvement caused by this method occurs in clay soils of moderate to high plasticity. The increase in strength occurs because the calcium cations in the hydrated lime replace the cations present in the clay mineral. This alteration in clay reduces its plasticity, the moisture-holding capacity and swell. Advantages of this method are that it is easily and rapidly implemented and it works well with our focus soil. Disadvantages of this method are that it is a short term stabilization method and it is toxic for plans and human health (The National Lime Association, 2003).

✓ Randomly mix fibres into the soil: This method consists of randomly mixing fibres into the soil to increase its shear strength. The fibres increase the cohesion among the soil particles. In addition the interaction of the fibres among themselves and the fibber's flexibility makes them behave as a structural mesh that holds the soil together increasing the soil structural integrity. Advantages of this method are that there are several different materials that can be used to reinforce the soil, the machinery required is minimal, the fibres can be inexpensive and environmentally friendly, and it can be implemented in all types of soils. Disadvantages of this method are that some of the fibre only last short periods of time and can only be implemented in shallow depths. However, this characteristic of the reinforcement method allows it to be easily implemented in large areas (Babu and Vasudevan,2008).

III.2.2 ROOT REINFORCEMENT THEORY

Root reinforcement theory has basically been developed along two avenues. The first method originated with the efforts to quantify the effects of deforestation and precipitation on the stability of slope, and entailed a description of root soil interaction within a shear band through force equilibrium. The formulations were proposed by Waldron (1977) and Wu et al. (1979). Subsequent advances to these approaches mainly comprised refinements (for instance) in the form of explicit definitions of reinforcement element orientation (Gray & Ohashi, 1983), improved description of load transfer from soil to reinforcement elements (Juran et al., 1988) and the effect of sand granulometery (Maher & Gray, 1990). These advances were, however, increasingly based on fibre reinforced soil behavior with root reinforcement.

The second avenue, along which root reinforcement theory was developed, owed its origin to the description of the behavior of composite materials. This method considers the macroscopic properties of composites, with the distinct characteristics of fibres and matrix having been homogenised or averaged (Michalowski & Zhao, 1996). Within this context of fibre reinforcement, root reinforcement is clearly identified as a specific case. A very limited number of attempts still exist at description of fibre reinforced soil using this method. Among other, De Buhan et al. (1989) and Michalowski & Zhao (1993) addressed uni-axially reinforced soil, while Michalowski & Zhao (1996) attempted to describe continuous filament and isotropic fibre reinforced soil.

Soil is strong in compression but weak in tension and roots are weak in compression but strong in tension. Therefore when soil and roots are combined the resultant soil-root matrix produces a mass which is much stronger than either the soil or the roots on their own. The roots act by transferring the shear stresses developing in the soil to the tensile resistance in the roots, and also by distributing stresses through the soil, so avoiding local stress build-ups and progressive failures (Docker 2003).

The theory of reinforced earth was first developed by Vidal (1969). As a vertical principal stress is applied to an unconfined element of soil the element will strain laterally as it compresses axially (Figure. III.15). If reinforcement is added to the soil in the form of horizontal strips, the lateral movement induced in the soil generates a frictional force between the soil and the reinforcement. As a tensile force develops within the reinforcement a corresponding compressive lateral confining stress is generated within the soil.

This lateral confining stress is analogous to an externally applied confining pressure and is proportional to the applied normal confining stress up to a limit defined as the 'critical confining stress' (Long et al., 1972, Ingold, 1982).

The action of reinforcement in soil is therefore not one of carrying developed tensile stresses but of the anisotropic reduction or suppression of an applied normal strain rate. This suppressive mechanism led to the concept of anisotropic cohesion.

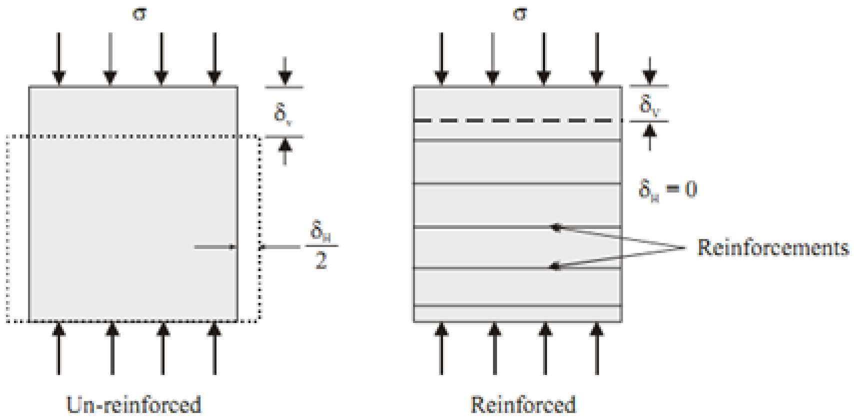


Figure III.15: The action of reinforcements on a cohesionless soil element (after Gray & Leiser, 1982).

The reinforced element resists lateral expansion through the mobilisation of a frictional force between the soil and the reinforcement.

Observations by Long et al. (1972) of the critical confining stress and failure modes of fibre reinforced sand samples indicated that above this critical stress value the reinforcement tended to fail in tension rather than slip or pull-out of the soil, as was the case below. It was also shown that above this point the 'equivalent confining stress' ceases to increase, but instead a constant increase in shear resistance occurs (provided the applied confining stress remains above this point). As a result the failure envelopes of both the reinforced and unreinforced sand are parallel (Figure.III-16) for tensile reinforcement failure, indicating the same angle of internal shearing resistance. They therefore concluded then that the additional strength imparted by the reinforcement could be represented by an apparent anisotropic cohesion. Schlosser & Long (1973) supported these observations with an expression for the anisotropic cohesion obtained by theoretical analysis.

$$c' = \frac{T\sqrt{K_p}}{2h} \qquad \text{(III-7)}$$

Where c' is the anisotropic cohesion; T is the tensile strength of the reinforcement; h is the vertical reinforcement spacing; and K_p is the coefficient of passive earth pressure. Below the critical confining stress failure occurs by disruption of the soil-reinforcement bond whereby the reinforcement slips or pulls-out of the soil. As stated above, for this kind of failure it is assumed that friction along the reinforcement is proportional to the normal confining stress. The resultant effect is for an increased friction angle of the earth reinforced sample (Figure. III.16). the increased friction angle is determined by (Hausmann, 1976):

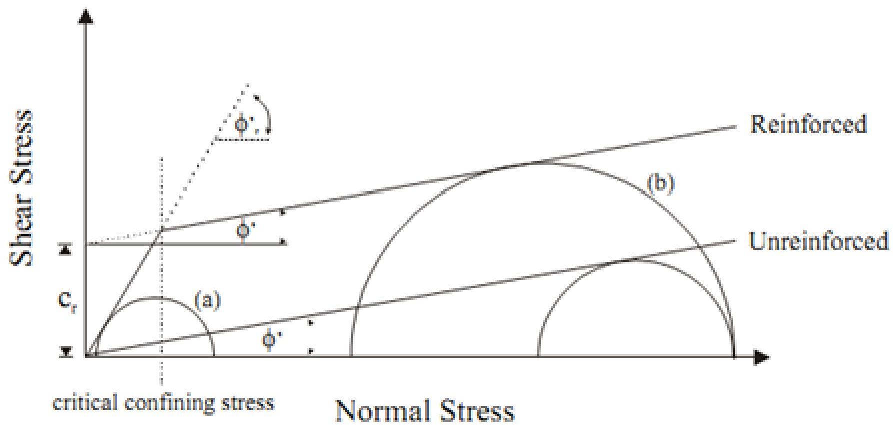


Figure III.16- Mohr-Coulomb envelopes for reinforced and unreinforced soils with circles describing failure by (a) slippage and (b) reinforcement rupture (after Hausmann, 1976).

The critical confining stress varies for different soil-fibre systems and is a function of such properties as tensile strength and modulus of the fibres, length/diameter ratio of the fibres, and frictional characteristics of the fibres and soil (Gray & Ohashi, 1983).

Investigators of root reinforcement in soil have generally found that roots have failed in tension and therefore posit that root systems have a negligible influence on the frictional component of soil strength (Waldron, 1977; Gray & Megahan, 1981; Waldron & Dakessian, 1981; Riestenberg & Sovonick-Dunford, 1983; Abernethy & Rutherfurd, 2001). The shear zone must also be wide enough to allow roots crossing it to deflect, elongate, and develop their maximum tensile strength, rather than failing in shear, as would be the case with a thin shear zone (a few millimetres wide) where the roots are held rigidly by the soil on either side (Burroughs & Thomas, 1977). These observations have been used to demonstrate that root reinforcement of soil is best approximated by an increase in apparent soil cohesion that varies in proportion to the concentration of roots within the soil.

Some studies indicate that the increase in apparent soil cohesion is limited to roots up to about 2 cm in diameter (Coppin & Richards, 1990). The justification for this limit is not completely clear as field studies often cited as supporting it (e.g. Burroughs & Thomas, 1977; O’Loughlin & Watson, 1979), although demonstrating the importance of small roots to increased soil shear strength, do not actually measure the effect of larger roots. Burroughs & Thomas (1977) measured roots up to 1 cm in diameter, and O’Loughlin & Watson (1979) up to 3 cm. An extensive literature search was unable to locate any study that assessed the reinforcing actions of roots of different sizes.

III.2.3 ROOT REINFORCEMENT MEASUREMENTS

Studies that have measured the direct contribution of roots to soil shear strength include (Endo & Tsurata. 1969; Wu et al. 1988a, and Wu & Watson. 1998) by in situ tests; and Waldron .1977, Waldron & Dakessian .1981, Terwilliger & Waldron. 1991) by laboratory tests. It igenerally accepted from these studies that the increase in soil strength is a measure of increased apparent cohesion and that this increases as root quantity across the shear zone increases. The actual values of additional strength vary considerably from study to study as environmental conditions, soils and tree characteristics differ (Table 3).

The relationship between increased shear resistance and root quantity has been found to be both exponential (Tengbeh, 1989, cited in Styczen & Morgan, 1995) and linear (Endo & Tsurata, 1969; Waldron, 1977) therefore the exact nature of the relationship remains elusive. Jewell & Wroth (1987) and Shewbridge & Sitar (1989) also argue that the strength increase in reinforced soil may not be linear. All of these studies show however that even at low root densities, root reinforcement can have a significant effect on soil strength.

Table 3: Typical values of root shear strength obtained in previous investigations (modified after Wu, 1995). (Docker 2003)

Investigators	Soil/Vegetation	Study Method	S_r/A_r or $[C_r]$ (kPa)
Endo & Tsurata (1969)	Loam/European Alder (Hokkaido)	*In-situ* Shear	0.05 % 10^4
Swanston (1970)	Till, Colluvium/Conifers (Alaska)	Slope Failure	[3.4-4.4]
O'Loughlin (1974b)	Till, Colluvium/Conifers (British Columbia)	Slope Failure	[1.0-3.0]
Waldron (1977)	Loam/Barley	Laboratory Shear	3 % 10^4
Burroughs & Thomas (1977)	Till/Conifers (West Oregon & Idaho)	Tensile Strengths	[3.0-17.5]
Wu *et al.* (1979)	Till, Colluvium/Conifers (Alaska)	Slope Failure	[5.9]
Ziemer (1981)	Sand/*Pinus Contorta* (California)	*In-situ* Shear	0.1 % 10^4
Gray & Megahan (1981)	Sandy Loam/Conifers (Idaho)	Excavation	[10.3]
Waldron & Dakessian (1981)	Clay Loam/Pine Seedlings	Laboratory Shear	[~5.0]
Riestenberg & Sovonick-Dunford (1983)	Colluvium, Silty Clay Loam/Sugar Maple (Cincinnati, Ohio)	Slope Failure/Tensile strengths	2.8 % 10^4
Wu (1984)	Till, Colluvium/Conifers (Alaska)	Slope Failure/Tensile strengths	1.4 % 10^4
Terwilliger & Waldron (1991)	Loams/Chaparral	Laboratory Shear	[0.4-0.8]
Wu & Watson (1998)	Silty Sand/*Pinus radiata* (New Zealand)	*In-situ* Shear	[2.5-4.5]
Abernethy & Rutherfurd (2001)	Silty Loam/River Red Gum/Swamp Paperbark (Latrobe Valley, Vic)	Pull-out tests	[10-120]
Schmidt *et al.* (2001)	Colluvium/Mixed forest species (Oregon)	Tensile strengths	[6.8-94.3]

The tensile strength of roots varies enormously not only between species but also within species growing at different locations (Greenway, 1987). It generally reduces with increasing root diameter, leading to claims that the finest roots have the potential to contribute most to soil reinforcement (Burroughs and Thomas, 1977; O'Loughlin and Watson, 1979). This is also probably due to the fact that smaller roots are more likely to be located at the margins of a root system where instability is more likely to occur; and because they are the first to decay upon death of the tree, resulting in a bigger influence on slope stability after clear-cutting. The strength of small roots is much easier to measure than for larger roots, which is the most probable reason that no studies can be identified that measure the influence of large roots (> 4 cm) on soil shear resistance. (Docker.2003) .

Larger roots however, require a greater load to pull them from the soil or to cause failure in tension (Nilaweera and Nutalaya, 1999) and therefore the amount of increased shear strength they provide should be larger than that supplied by small roots. This is supported by the observation that roots larger than 2 cm are rarely found in landslip scarps (Wu et al., 1979).

While most root reinforcement investigations have focused on an increase in soil shear strength, Zhou et al. (1997) studied the traction effect of lateral roots of Pinus yunnanensis by direct in-situ test in the Hutiaoxia Gorge, Southwest China. In contrast to the effect of vertically-extending roots, the traction effect reinforces the soil not by increasing shear strength, but by enhancing the tensile strength of the rooted soil zone. It was found that the traction effect of the roots increased the tensile strength of the shallow rooted soil by 4.2~5.6 kPa. The results of this study indicate that together with the pine's vertical roots, which may potentially anchor the shallow rooted soil zone to a more stable substrate, the lateral roots through a traction effect, are able to mitigate against shallow instability in forested slopes.

Clearly then there are different models and interpretations of the mechanism of soil reinforcement by roots. All published models agree however that the presence of tree roots increases the resistance to shear of a mass of soil that forms a slope. The main difference between the resultant effects of each model, whether it is by increasing the apparent cohesion of the soil, anchoring the soil to a more stable substrate, or buttressing and arching, will be the magnitude of the increased shear resistance and the manner in which it is calculated. The magnitude of increased shear resistance will obviously have a big influence on the relative stability of a slope and so it is essential to realise a good understanding of the reinforcement and subsequent failure mechanism of the roots in the particular environment being assessed.

III.4.7 MODEL OF ROOT REINFORCEMENT

Most modelling approaches that have been proposed so far have concentrated on the prediction of the contribution of fibres to shear strength increase. The various approaches to describe the shear strength increase are based on force equilibrium (Gray and Ohashi, 1983; Maher and Gray, 1990) and energy dissipation (Micha1owski, 2008; Michalowski and Zhao, 1996).More recently Zornberg (2002) proposed a framework to predict failure of different reinforced soil types based on the superposition of the sand and fiber effects.

The presence of plant roots in the soil matrix results in an increase in soil cohesion c_s through a reinforcing effect which usually augments superficial slope stability (Schmidt et al., 2001). The root–soil reinforcement model developed by Wu (1976), and elaborated upon by Waldron (1977), is widely used to estimate the additional cohesion c_r taking into account the presence of roots in the soil (Gray and Sotir, 1996; Roering et al., 2003).This model states that the shear strength of soil reinforced by roots τ_{sr} is calculated by the Mohr–Coulomb equation as follows:

$$\tau_{sr} = c_s + c_r + \sigma \tan \phi \qquad \text{(III-8)}$$

Where c_s is soil cohesion, c_r is additional cohesion due to the presence of roots, s is the normal stress on the shear plane and f is the soil apparent friction angle. Shear forces developed in the soil when the soil layer moves are translated into tensile forces in the roots. The mobilization of this tensile force in the roots can then be split into tangential and normal components. Assuming that roots are elastic initially oriented perpendicularly to the slip plane, fully mobilized in tension and that f is unaffected by root reinforcement (Waldron, 1977; Greenway, 1987), c_r can be defined as:

(III-9)

$$c_r = t_r(\sin\delta + \cos\delta\tan\phi)$$

Where δ is the angle of deformed roots with regard to the shear surface and t_r is the average mobilized tensile strength of roots per unit area of soil. t_r can be expressed as the product of t_r, the average tensile strength of roots and A_r/A the fraction of soil occupied by roots called the root area ratio (RAR). The values of (sin d +-cos δ tan ϕ) can be approximated as 1.2 (Wu et al., 1979) and so Eq. (III-10) can be rewritten as:

$$c_r = 1.2T_r\left(\frac{A_r}{A}\right) \qquad \text{(III-10)}$$

Both RAR and Tr are influenced by species and site factors, e.g. local climate, soil type, land use management, season, root type and size, as well as orientation of roots in the soil (Turmanina, 1965; Gray and Sotir, 1996; Operstein and Frydman, 2000).

The presence of plant roots crossing the potential shear surface results in an increase in soil cohesion through a reinforcing effect which usually augments superficial slope stability. The root – soil reinforcement model developed by Wu (1976), and elaborated upon by Waldron (1977), is widely used to estimate the additional cohesion taking into account the presence of roots in the soil (Gray and Sotir, 1996). This model states that the additional cohesion due to the presence of roots can be estimated as follows:

$$C_r = R_f * T_r * RAR \qquad \text{(III-11)}$$

Where T_r is the average tensile strength of roots and RAR is the Root Area Ratio, i.e. the total root cross-section area (CSA) per unit of surface at the potential shear surface. R_f is the root orientation factor. It depends on the friction angle of the soil and on the angle of the root at rupture, relative to the failure plane (Thomas and Pollen, 2009). Moreover,

$$T_r = F_r / CSA \qquad \text{(III-12)}$$

with F_r the maximum load that the root can support before it breaks. In the literature, it is often reported that F_r increases when root diameter increases (Schmidt et al., 2001). An analysis of stability may be used to evaluate an existing condition or to determine whether a proposed condition meets the requirement of safety. This procedure is commonly based on the limit equilibrium method whereby a mass of soil in place on a slope is considered to be on the verge of failure, and the shear strength of the soil is fully developed along a potential slip surface. The stability of the slope is generally expressed as a factor of safety, which is the ratio of Restoring to Disturbing forces present at incipient failure:

$$\text{FoS} = \frac{\text{Restoring Forces}}{\text{Disturbing Forces}} \qquad \text{(III-13)}$$

A factor of safety ≥1.0 means the slope will resist failure, while a factor of safety < 1.0 will be calculated for an unstable slope and one that should fail in shear. In reality a factor of safety of 1.0 does not necessarily indicate that failure of a slope is imminent (De Mello, 1977) as the real factor of safety will be strongly influenced by minor geological details, stress-strain characteristics of the soil, actual pore-pressure distribution, initial stresses, progressive failure, and numerous other factors (Nash, 1987).

The method of slices is a well-established limit equilibrium approach for assessing the stability of slopes. In its most basic form the infinite slope method describes the condition where a single vertical slice is representative of the entire slope (Figure III-17) (Docker, 2003) This method is only suitable for slopes that exhibit a large length to depth ratio but it is an effective and quick first approximation calculation that can be used to demonstrate the essential behaviour of a given slope (Mostyn & Small, 1987). It is expressed in the following form to include the effects of vegetation (after Wu et al., 1979):

$$\mathrm{FoS} = \frac{(c + S_r)l + [(W + S_w)\cos\beta - u]l\tan\phi}{(W + S_w)\sin\beta} \qquad \text{(III-14)}$$

where **c** is the soil cohesion; $\mathbf{S_r}$ is increased shear strength due to roots; W is the weight of soil; $\mathbf{S_w}$ is the surcharge weight of vegetation; **ß** is the slope angle; **u** is the pore water pressure which is $\gamma_w h_w \cos^2\alpha$; l is the length of shear surface; and ø is the internal friction angle of the soil. For simplification the effects of wind-throw, soil suction, and root anchorage have been removed.

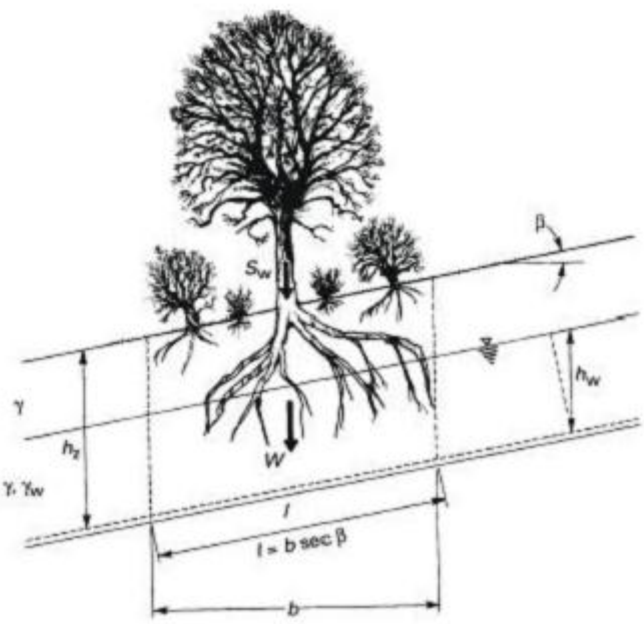


Figure III-17: Diagrammatic representation of the infinite slope model with the addition of forces through the surcharge weight of vegetation. The soil mass is only partly saturated and under conditions of steady-state seepage (after Bache &MacAskill, 1984).

An alternative to the limit equilibrium method for stability analysis of vegetated slopes has been proposed by Ekanayake & Phillips (1999). It concerns an assessment of the energy consumed in the shearing process as well as the ability of the soil-root system to withstand larger shear displacements and therefore larger shear strains than fallow soils. These authors suggest that the limit equilibrium method may underestimate the additional shear resistance of soils containing roots by only considering the increased peak shear resistance of the soil-root system and not the additional shear resistance provided during large displacements of the roots, prior to failure. Application of this method is limited at the present time to slopes that can be approximated by the simplified infinite slope model. There are also practical concerns about collecting sufficient data to deal with slopes exhibiting a variety of failure types and sizes (Ekanayake & Phillips, 1999).

Generally the increased shear resistance of tree roots is modelled as an increase in apparent cohesion that increases with increasing concentration of roots on a potential shear plane (see Gray, 1978; Greenway, 1987; Coppin & Richards, 1990). Clearly the distribution of root concentration in the soil beneath a tree will be a critically important parameter for input to the slope stability model. Analysis on hill-slopes with fairly uniform tree cover, often assume fairly uniform root concentrations at any given depth over the entire slope (see Greenway, 1987), which is a reasonable estimation for the average increased shear strength over a large area of a single species forest. Riestenberg (1994) for instance found that when modelled with a uniform distribution of root anchors, small white ash trees may be spaced as much as seven metres apart and still stabilise a 30 degree hill-slope with colluvium thickness of 43 cm. Variations in root distributions between multiple species however have led to measurements of large variation in increased shear strengths over relatively small areas (Terwilliger & Waldron, 1991; Schmidt et al., 2001) resulting in adjacent zones of varying landslide susceptibility and potential scarp size.

The variability of increased shear resistance has been recognised not just between different vegetation types but also at different locations within the soil mass below a single tree (Abernethy & Rutherfurd, 2001). Accounting for these differences in slope stability modelling, results in different locations of the critical failure plane and different calculated factors of safety for different species and different tree locations on the hill-slope (Collison et al., 1995), or riverbank (Abernethy & Rutherfurd, 2000a).

For the past three decades, research has focused on utilizing plant root reinforcement to stabilise slopes. The ability of plant roots to strengthen a soil mass is well known. The inclusion of plant roots with high tensile strength increases the confining stress in the soil mass by its closely spaced root matrix system. The soil mass is bound together by the plant roots and the shear strength is increased by this effect. The contribution of root reinforcement to shear strength is considered to have the characteristics of cohesion (Wu et al., 1979).

They proposed a simplified perpendicular root model to quantify the increased shear strength of soil due to root reinforcement. The increase in shear strength of the soil, Sr, was expressed by the following relationship:

$$Sr = t_R\,(\cos\theta \tan\varphi' + \sin\theta) \qquad \text{(III-15)}$$

Where Sr = shear strength increase from root reinforcement, t_R = average tensile strength of root per unit area of soil, θ = angle of shear rotation, and φ' = friction angle. Since the mechanical effect of plant roots is to increase the cohesiveness of the soil mass, Sr can be considered as equivalent to an apparent cohesion of the soil, known as apparent root cohesion (c_R). Typical values of apparent root cohesion (c_R) range from 1kPa to 17.5kPa (Coppin and Richards, 1990). These values were obtained from the studies of several investigators using different techniques including back analysis, direct shear tests, root density information combined with vertical root model equations, and back analysis combined with root density information. The values of apparent root cohesion (c_R) are dependent on the type of vegetation and in-situ soil conditions.

In order to evaluate the contribution of tree roots to soil shear strength (i.e. to determine Sr) a simple model was developed independently by Waldron (1977) and Wu et al. (1979). The model was designed to simulate the idealised situation of a tree's vertical roots extending across a potential sliding surface in a slope. It consists of a flexible, elastic root extending vertically across a horizontal shear zone of thickness z (Figure III-18).

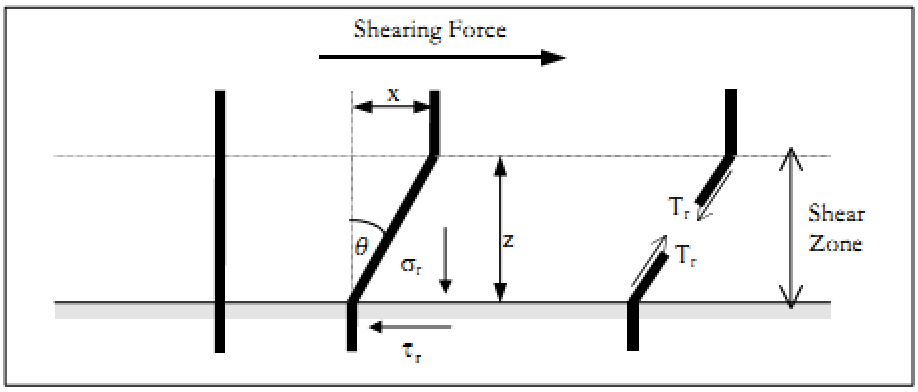


Figure III-18 Model of a flexible, elastic root extending vertically across a horizontal shear zone

As the soil is sheared a tensile force Tr develops in the roots. As shown in Figure III-18 this force is resolved into a tangential component (tr) which resists shear and a normal component (sr) which increases the confining stress on the shear plane

It was suggested by Waldron & Dakessian (1981) that the strength of the soil-root bond was the most important unmeasured model parameter. Its value rather than root strength, limited root reinforcement in a saturated clay loam permeated with barley and pine roots, and led to the failure of different roots at different displacements. As such, they went on to suggest that the assumption that all roots fail in tension simultaneously may lead to large overestimates of the increased shear strength of the soil-root system.

The above models assume that the roots are initially orientated perpendicular to the shear surface. In nature plant roots may be inclined at many different angles to a sliding or failure surface and so to take this effect into account Gray & Ohashi (1983) developed a model for a long elastic fibre orientated either perpendicular to the shear surface or at some arbitrary angle. It was found that the maximum values of increased shear-strength correspond to fibre inclinations close to (45 + f/2)º, however for fibres inclined between 30 and 90 degrees to the shear plane, both the theory and experiment indicate little difference in reinforcement (Gray & Leiser, 1982). For investigators of root-reinforced soil the perpendicular root model provides a useful and the most widely applied interpretation of the situation.

The effect of roots on soil fixation has been reported by several authors, but quantifying the gain in soil shear strength is difficult to achieve. Pioneering modelling work by Wu (1976) and Waldron (1977) have introduced the root mechanical contribution as additive soil cohesion in the Coulomb's failure criteria using a simple mechanistic model. The additional cohesion at the slip surface was defined by two variables: the average root tensile strength and root area ratio (RAR, or the fraction of a plane of soil occupied by roots). It was assumed that roots are initially perpendicular to the slip surface and bend according to the relative displacement of soil on both sides of the shear zone. The tangential component of root tensile force thus directly contributes to the increase in soil shear strength, whilst the normal component augments the confining pressure. These models of soil reinforcement have been shown to overestimate the additional cohesion due to roots in tension, as all roots are assumed to break at the same time (Bischetti et al. 2009a).

A significant improvement of this approach has been proposed by Pollen and Simon (2005) and Pollen and Simon (2009) who applied a Fibre Bundle Model to rooted soils. This model considers that the root network breaks progressively from the weakest to the strongest roots, and that stresses of broken roots are redistributed to the remaining elements. More recent papers also pointed out the limitation of assuming that roots are initially oriented perpendicular to the slip surface, and propose using root architectural models as an improvement in slope stability analyses (Reubens et al. 2007; Danjon et al.2008).

Long et al. (1996) studied stability analysis of reinforced and unreinforced embankments on soft ground. General solutions were developed for the rotational stability analysis of an embankment with and without reinforcement and constructed on soft ground that has an undrained shear strength varying with depth. The solutions were presented in the form of simple equations. The relationship between the critical slip circles through the embankment with and without reinforcement was expressed explicitly. The analysis procedures and supporting graphs allow the user to obtain the solutions using hand computations.

In most models, roots are considered as very flexible elements, thus limiting their application to the finest roots. However, from a mechanical point of view, a distinction must be made between fine and thin roots, which behave like cable elements, i.e. with a very low bending stiffness, and structural roots that are similar to beams, developing longitudinal shear stresses (Reubens et al. 2007).

III-5 CONCLUSIONS

Significant research has been performed over the last few decades to evaluate basic shear strength properties and deformation characteristics of fiber-reinforced soils. Previous work has shown that presence of plants significantly improves the engineering response of soils. By focusing more research and efforts into understanding the mechanics of roots in slope stability, we can take advantage of utilizing trees and other vegetative options to stabilize slopes, as it is an inexpensive and environmentally friendly alternative to other methods. For that we will study in the next chapter in detail using the triaxial apparatus the ability of a tree of acacia pycnantha to reinforce the soil. This ability is limited by the spatial distribution of its root system and the strength that the roots impart to the soil during shear. The use of tree roots and their fibres to protect slopes is a useful and well-known natural bioengineering method that has been applied extensively worldwide.

RESEARCH METHODOLOGY

Chapter Four

IV-1 INTRODUCTION

To investigate the effects of root of acacia pycnantha mimosas on the behavior of unreinforced and reinforced silty soil,a total of 37 unconfined and 28 triaxial compression tests were performed. Moreover, during the experiments, some of the tests were repeated to determine the accuracy of the results. The experiments were all conducted on a sample of 70 mm in diameter and 70 mm in height. The procedures for specimen preparation and testing were standardized to achieve repeatability in the test results. All the initial tests were repeated until consistent results were obtained.

IV-2 PRINCIPLES OF THE TRIAXIAL COMPRESSION TEST

The triaxial compression test is used to measure the shear strength of a soil under Controlled drainage conditions. In the basic triaxial test, a cylindrical specimen of soil encased in a rubber membrane is placed in a triaxial compression chamber, subjected to a confining fluid pressure, and then loaded axially to failure. Connections at the ends of the specimen permit controlled drainage of pore water from the specimen. The procedures presented herein apply only to the basic test conducted with limited drainage conditions, and do not include special types or variants of this, test. In general, a minimum of three specimens, each under a different confining pressure, are tested to establish the relation between shear strength and normal stress. . The test is called "**triaxial**" because the three principal stresses are known and controlled. Prior to shear, the three principal stresses are equal to the chamber fluid pressure. During shear, the major principal stress, $\boldsymbol{\sigma_1}$ is equal to the applied axial stress (P/A) plus the chamber pressure, $\boldsymbol{\sigma_3}$ (see FigureIV-1). The applied axial stress, $\boldsymbol{\sigma_1 - \sigma_3}$, is termed the "deviator stress". Intermediate principal stress, $\boldsymbol{\sigma_2}$, and the minor principal stress, $\boldsymbol{\sigma_3}$ are identical in the test, and are equal to the confining or chamber pressure hereafter referred to as $\boldsymbol{\sigma_3}$.

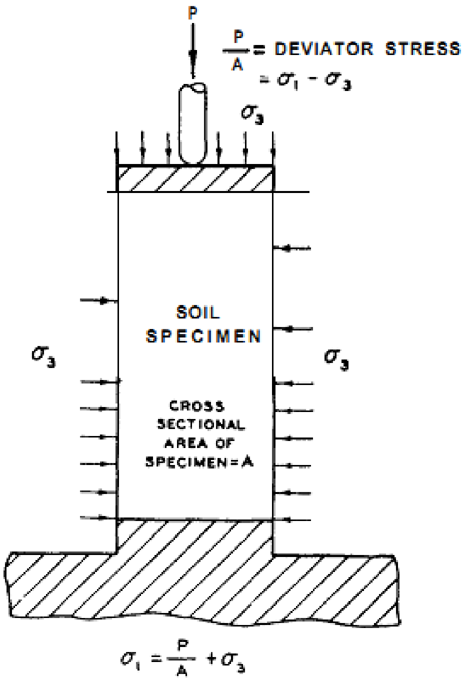


***Figure IV-1**: Diagram showing stresses during triaxial compression test*

IV-3 TRIAXIAL CELL BISHOP-WESLEY

All triaxial tests were performed with a triaxial cell Bishop - Wesley (1975) double pressure chamber which can control the vertical stress σ'_v and σ'_h containment independently. This device allows performing tests in undrained triaxial compression and extension, and also drained tests with radial (i.e the ratio $\frac{\Delta p'}{\Delta q} = const$) stress paths theoretically effective as desired. This is a major advantage compared to a conventional triaxial (when $\boldsymbol{\sigma'_v}$ is applied by imposed displacement) that allows for only one path constraint mode drained ($\Delta p'/\Delta q=\pm 1/3$) .

The principle of the device is shown in FigureIV-2. The sample is mounted on the lower base of the cell based on a piston. The vertical force can be applied to the sample through the piston which is pushed by fluid pressure in the lower chamber. Membranes "Bellofram" are used to maintain the fluid in the lower and upper chambers. The movement of the piston is guided by linear bearing system that ensures the movement with negligible levels of friction. In practice, the vertical motion or force applied to the sample is controlled by a controller pressure / volume GDS outside.

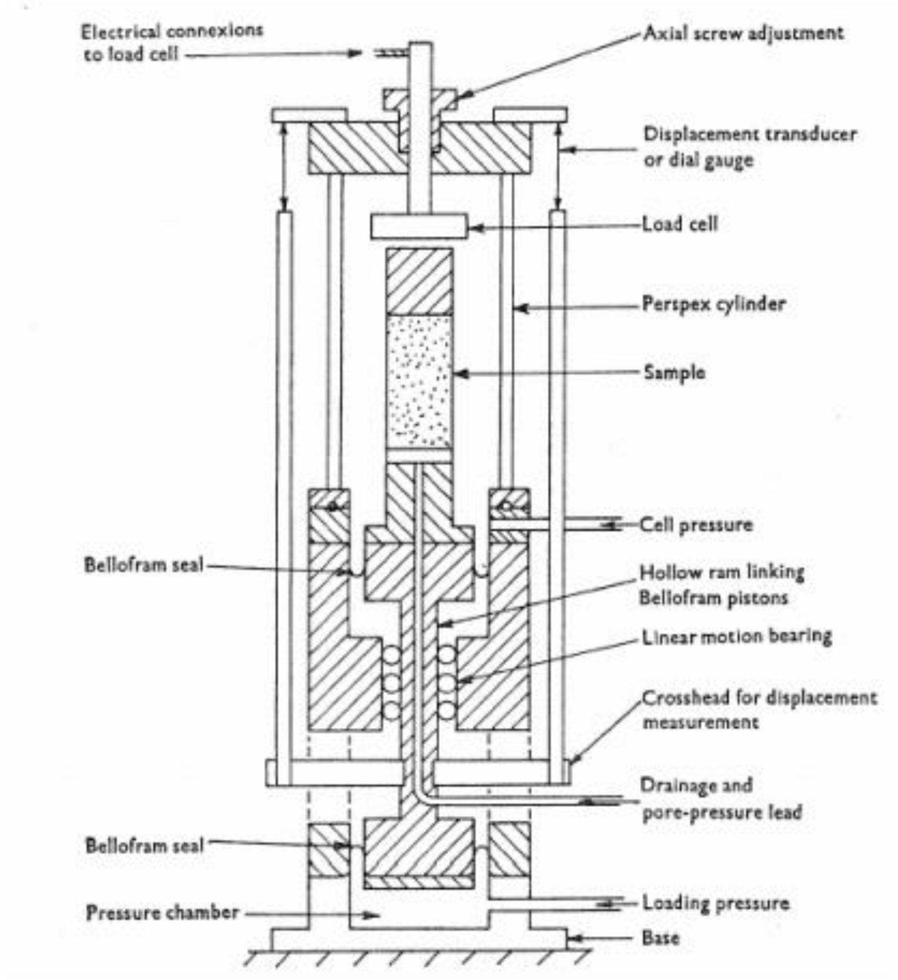


Figure IV-2: Diagrammatic representation of the Bishop &Wesley Stress path cell. (Bishop and Wesley, 1975)

The radial pressure of confinement ($\sigma_2 = \sigma_3$) is imposed by a second pump GDS. The pore water pressure (pressure cons) of the sample is applied via a buffer cell water / air and controlled w it the regulator which allows an accuracy of ± 0.3 kPa. Both pumps are driven by GDS Labview program, and control parameters such as pressure and volume injected. Using a GDS pump to control the movement of the piston can perform phase shear strain or stress imposed (by the speed of movement of the piston).This system is able to perform any type of test soils: isotropic compression, test K_0 , test radial path conventional triaxial tests (undrained compression and extension confinement constant)($\Delta p'/\Delta q$=cste).

IV-4 THE MOLD MANUFACTURING SAMPLES

The samples are made using a mold consisting of two semi cylindrical shells (Figure IV-3). The two shells can be assembled or repelled from each other easily with a clamp. In order to maintain the latex cuff along the walls of the mold, four suction ducts are drilled in the shells. These conduits communicate with the interior of the mold by rows of small holes (1mm diameter). They are connected to hoses which are assembled in a single tube. The latter can be connected to a vacuum pump.

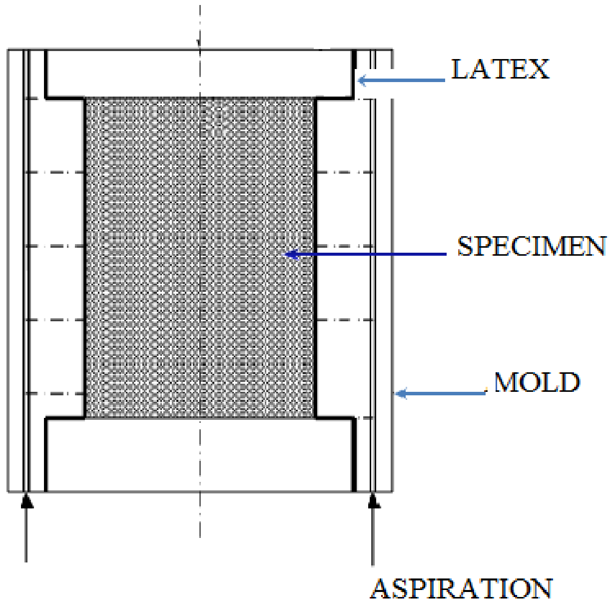


Figure IV-3: mould manufacturing samples

IV -5 MEASUREMENT SYSTEMS

The device used to measure or control with an acquisition system the following quantities:

- The variation in the height of the sample (ΔH).
- The change in sample volume (ΔV).
- interstitial pressure(u),
- Confining pressure in the cell (σ_c).

From these measures and sample characteristics (height and volume V_0, H_0), we can calculate the following quantities:

- **Axial strain**: $\varepsilon = \Delta H/H_0$

Where: **AH** = change in height of specimen during test, cm,
H_o = initial height of specimen, cm, (Where a significant decrease in specimen volume occurs upon application of the chamber pressure, as in partially saturated soils, the height of the specimen after application of the chamber pressure should be used rather than the initial height.)

- **volumetric strain** : $\varepsilon_v = \Delta v/v_0$
- **Deviator stress** : q = ΔF/S

The development of Bishop & Wesley's stress path cell coincided with the advent of so-called programmable calculators which were ancestors of the present day PC. Linking these desk-top computers to the hydraulically actuated stress path cell required the development of some kind of pump with a computer interface. Menzies et. al. (1977) developed a microprocessor controlled flow pump (or screw pump) for this purpose .As electronic technology improved, so the system evolved from analogue process control to fully digital control(Menzies, 1988; Menzies and Hooker, 1992). Figure IV-4 shows the principle of operation of the digital pressure/volume controller (as the modern version of the flow pump is now called).

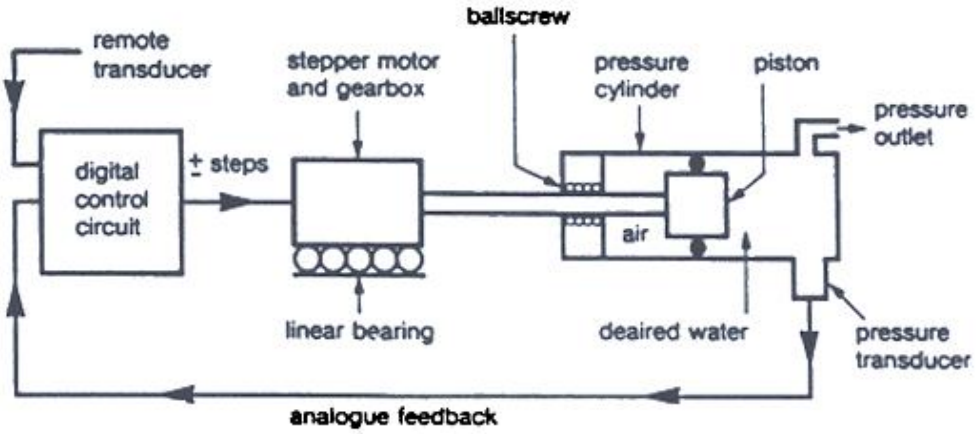


Figure IV-4 Diagrammatic layout of the GDS digital pressure/ volume controller

Deaerated water in a cylinder is pressurised and displaced by a piston moving in the cylinder. The piston is actuated by a ball screw turned in a captive ball nut by a stepping motor and gear box that move rectilinearly on a ball slide. A photograph of the controller is shown in Figure 3-5

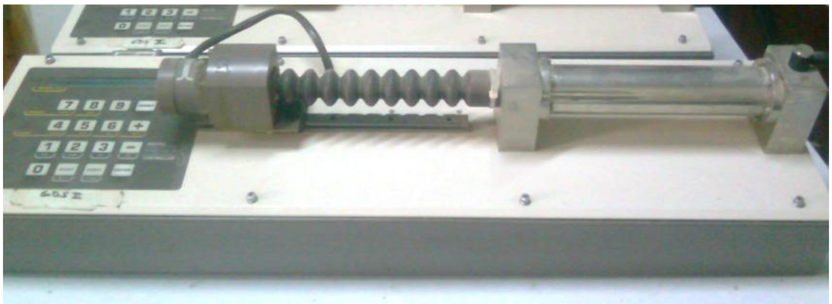


Figure IV- 5. Photograph of the GDS 2OOcc/2MPa digital pressure/volume controller.

Pressure is detected by means of an integral on-board solid state pressure transducer. There is also a user installed field upgrade which enables feedback control from a remote transducer such as load cell or Hall Effect local strain transducer. Control algorithms are built into the programmable memory to cause the controller to seek to a target pressure or step to a target volume change. Volume change is measured by counting the steps of the stepping motor. Knowing the number of steps per revolution of the motor, the gearbox ratio and the pitch of the ball screw, the bore of the pressure cylinder may be found such that one step of the motor equals 1 mm^3 in the 2MPa range controller (Menzies, 1988).

Three digital pressure/volume controllers (usually abbreviated to "digital controllers") are put under computer control and regulate axial stress, cell pressure and back pressure. The computer also logs data from various transducers including, internal submersible axial load cell, Hall Effect local strain transducers (one radial, two axial), axial digital indicator, and mid-plane and base pedestal pore pressure transducers. Under PC control, both conventional and advanced tests can be carried out. These include the conventional U-U, C-U, C-D tests, as well as the advanced tests of k-zero and stress paths.

IV-6 MATERIALS AND METHODS

IV-6-1- Triaxial Compression Test

The triaxial compression test is the most widely used technique to determine the shear strength of soils. The apparatus is shown diagrammatically in the (figureIV-6). The sample, which is cylindrical, is tested inside a perspex cylinder filled with water under pressure. The sample under test is enclosed in a thin rubber membrane to seal it from the surrounding water. The pressure in the cell is raised to the desired value, and the sample is then brought to failure by applying an additional vertical stress.

One of the major advantages of the triaxial apparatus is the control provided over drainage from the sample. When no drainage is required (i.e. in undrained tests), solid end caps are used. When drainage is required, the end caps are provided with porous plates and drainage channels. It is also possible to monitor pore-water pressures during a test.

The triaxial compression test is a useful method for obtaining shear strength parameters from undisturbed soil specimens. Currently, there are two types of tests used. They all use the same equipment but vary in procedure and effectiveness.

IV-6-2 Triaxial Test Equipment

The triaxial compression test system housed in the Laboratory of Materials Sciences and Environment (University of Chlef) comprised of many equipment. The important system components are listed below:

- An autonomous triaxial cell type Bishop and Wesley: This cylinder shaped cell held the soil test specimen and pressurized water around it. The top plate allowed a loading piston to penetrate into the cell. The bottom assembly connected pressure transducers and drainage/saturation lines to the soil specimen or chamber water
- Pressure controllers /GDS (Triaxial Testing System) volume (2MPa).
- Vacuum Pump : This was used to pull air out of the soil specimen and deair water.
- Water Tank : This cylinder shaped tank was used to hold deaired water.
- A computer: A standard IBM-compatible PC ran special software prepared by the manufacturer of the triaxial test system; so that the sensor readings acquisition and test management will be automatic once the soil specimen is conditioned in the test cell.

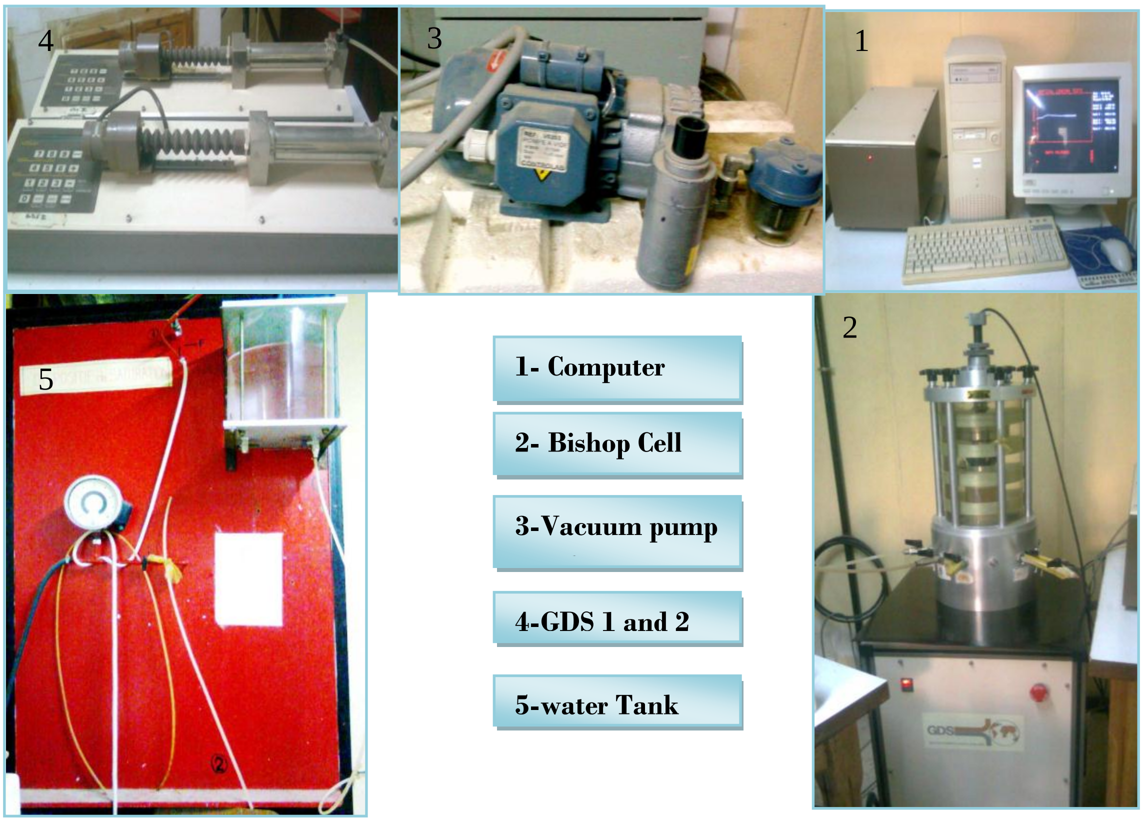


Figure IV-6: Laboratory Testing Devices

IV-6-3- Materials

IV-6-3-1 Fibre

The plant specie namely Acacia pycnantha (see figure IV-7) was chosen based on their prominent and physiological characteristic. These species are known to be resistant towards the poor and eroded condition of the slope.

A root fibre of Acacia pycnantha was used in this study with variations in fibre contents ranging from 0 to 8 percent. This root fibre was mixed with Chlef soil to get a composite samples comprised of root fibres (figureIV-8)

Figure IV-7 : Acacia Pycnantha

Figure IV-8. Root Fibres of Acacia pycnantha (Golden Wattle)

IV-6-3-2 Root

A compression drained (CD) and undrained (UD) triaxial test is applied in the laboratory to study the behavior of soil reinforced with roots of Acacia pycnantha (Golden Wattle). In addition to pure soil samples, soil–root composites were also prepared and two root distribution forms were used in composites to study root reinforcing effects of different root distribution forms (figureIV-9). One, three and four segments of root, 67mm long, were distributed vertically (vertical root or VR) in the sample sides ; and three, seven segments of roots, each 30mm long, were distributed (horizontal root or HR) horizontally in the sample sides.

Figure IV-9 .Roots system of Acacia pycnantha (Golden Wattle)

IV-6-3-3 Soil

The soil samples used in the present study were obtained from the region of Chlef in northern Algeria. The Atterberg limits of the portion passing No. 40 sieve are: liquid limit 33.5% and plastic limit 21.07%. The particles have a mean diameter (**D50**) of 0.06 mm, a minimum void ratio ($\mathbf{e_{min}}$) of 0.70, a maximum void ratio ($\mathbf{e_{max}}$) of 1.12, a uniformity coefficient (Cu) of 37.5, and a specific gravity of 2.65.According to the Unified of Soil Classification System, the soil is classified as low plasticity silty soil. Figure IV-12 shows the Particle Size Distribution curve of the soil used in this study.

IV-6-4 Grain Size Distribution

IV-6-4-1 Sieve Analysis test

Grain size analysis of chlef soil includes three steps: wet sieve analysis (XP P 94-041), the dry sieve analysis (NF P 94-056) and the hydrometer test (NF P 94-057).

Sieve analysis is conducted by taking a measured amount of dry, well pulverized soil. The soil is passed through a stack of sieves with a pan at the bottom. The amount of soil retained on each sieve is measured, and the cumulative percentage of soil passing through each sieve is determined. This is generally referred to as percent finer. Soil particles are generally separated into particle- size ranges using a series of sieves: 80 µm ;100 µm ;200 µm ;400 µm ;1mm ;2mm ;5mm; 10mm;20mm.the size of particles less than 0.08 millimeter(fine fraction) is generally determined by a sedimentation process, using a hydrometer to secure the necessary data.

The main outcome of this tests was the grain size distribution curve, which provided percent gravel, percent sand, percent fines (silt + clay), and key particle sizes (D60, D30, and D10).

The amount of soil retained on each sieve for dry sieving is a mechanical sieving performed with the device shown in Figure3-10.

Figure IV-10 : Sieve analysis device

IV-6-4-2 Hydrometer Analysis test

Hydrometer analysis is conducted on the principle of sedimentation of soil particles in water. In the test 40 grams of dry pulverised soil. A deflocculating agent is always added to the soil. The soil is allowed to soak for at least 16 hours in the solution of Sodium Hexametaphosphate. After the soaking period, distilled water as added, and the soil-deflocculating agent mixture is thoroughly agitated. The sample is then transferred to a 1000-ml glass cylinder. More distilled water is added to the cylinder to fill it up to the 1000-ml mark, and then the mixture is again thoroughly agitated .a hydrometer is placed in the cylinder to measure usually over 24 hour period.

Figure IV-11-: Hydrometer analysis test

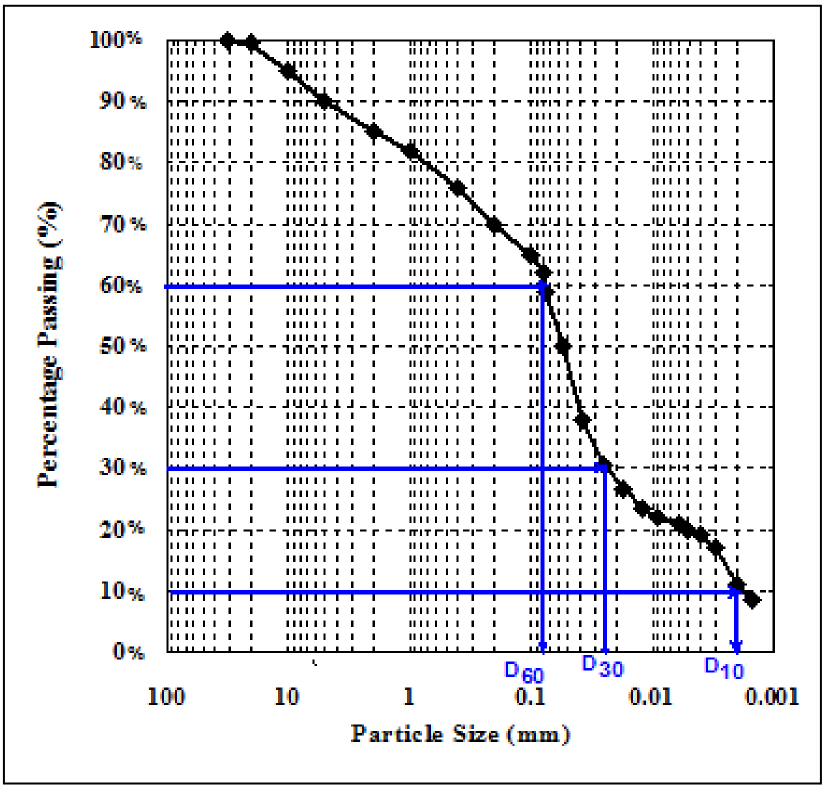


Figure IV-12. Grain size distribution curve of tested soil

IV-6-5 Sample Preparation

The sambles were 70 mm in diameter and 70 mm in height with smooth lubricated end-plates. First we put on filter paper pads (drainage hole) to protect them, and then we put a layer of silicone (KS63G) on two bases. After the specimen has been formed, the specimen cap is placed and sealed with O-rings; it was mounted on the base of the triaxial cell. The base

platen was lightly coated with a film of thin grease prior to attaching the membrane. The membrane was then sealed to the top loading cap and the bottom platen with O-ring seals. To ensure a good homogeneity of stress and strain in the sample and reduce friction between the sample and the upper and lower bases .Saturation was performed by purging the dry specimen with carbon dioxide for approximately 15 min. De-aired water was then introduced into the specimen from the bottom drain line. Water was allowed to flow through the specimen until an amount equal to the void volume of the specimen was collected in a beaker through the specimen upper drain line .Therefore, to maintain contact between the top loading cap and the load cell a nominal deviatoric stress of about 2 KPa was applied to the samble.
To quantify the important influence of plant roots on shear strength of chlef soil, we performed consolidated-drained and undrained triaxial compression tests with different confining pressures (σ_3' = 50, 100, 200, 300, 400 kPa).all samples were prepared on medium dense state (Dr=50%). Two different types of samples were tested: pure soil samples and composites samples comprised of roots of Acacia pycnantha (Golden Wattle). Two root distribution forms were used in composites to study root reinforcing effects of different root distribution forms. One, three and four segments of root, 67mm long, were distributed vertically (vertical root or VR) in the sample sides(see figure IV-13); and three, seven segments of roots, each 30mm long, and 0.7mm diameter were distributed horizontally (horizontal root or HR) in the sample sides.

Figure IV-13. View of type of soil–root composite (root distribution form with four roots)

IV-6-6 Back Pressure Saturation

In a triaxial compression test, saturation of the specimen is done by back pressure of water through the drainage lines. As the specimen is surrounded by a rubber membrane on its sides and plastic pieces at the top and bottom, water is pushed in to fill the void spaces. Saturation can be checked by finding the specimen's b-value. This is found by closing the drainage valves and increasing the confining pressure and recording the corresponding increase in pore pressure. This ratio is known as the b-value:

$$B = \Delta u/\Delta\sigma_3$$

Where: Δu = change in pore pressure
$\Delta\sigma_3$ = change in cell pressure

Saturation is considered complete when B has exceeded 0.95. If this value is over 0.95, then it can be assumed that the specimen has reached full saturation.

IV-6-7 Consolidated-Drained (C-D) Test

In this test, the specimen is extracted, saturated, and then put through a consolidation process. Consolidation is done by opening drainage lines and removing any back pressure. Then, a confining pressure acts on the specimen, causing all of the pore pressures to be removed. After this, an axial stress slowly compresses the specimen with drainage valves open. Bishop et al. (1960) point out that this prevents any excess pore pressures from developing, which is important, since this test looks at the long term stability of soil when dissipation has already occurred. These tests do take a long time to carry out, however, which is why they are not used very frequently.

IV-6-8 Consolidated-Undrained (C-U) Test

The C-U compression test differs from the C-D test in a few ways. First, during consolidation, there is a back pressure being applied to the specimen through the drainage lines. This is typically done for a 24 hour period. Also, because there is back pressure applied, the pore pressure in the specimen will not reduce to zero. So, after consolidation is completed, the drainage lines are closed off and an axial stress is applied to the specimen. The axial stress is applied by a strain rate that is determined from consolidation data. This type of test typically lasts for a few hours.

IV-7 CONCLUSIONS

In this chapter we present the apparatus used for studying the behavior of reinforced and unreinforced soil purposes under various Confining Pressure. This device is based on the cell with Bishop Improvements to the system for attaching the base and top. The procedure followed was based on recommendations developed by various researchers. It can make homogeneous samples and perform tests of good quality. In addition to the tests conducted on soil such as the sieve analysis and Hydrometer analysis, also the roots and fibres of acacia pycnantha used in this investigation. Preparing samples, especially for triaxial tests, is difficult and time consuming. Triaxial test was selected for this study because of its accuracy. The actual tests were performed on a computerized triaxial testing system, which provided accurate results. It presents more reliable values of soil parameters and stress-strain data.

RESULTS AND DISCUSSIONS

Chapter Five

V-1 INTRODUCTION

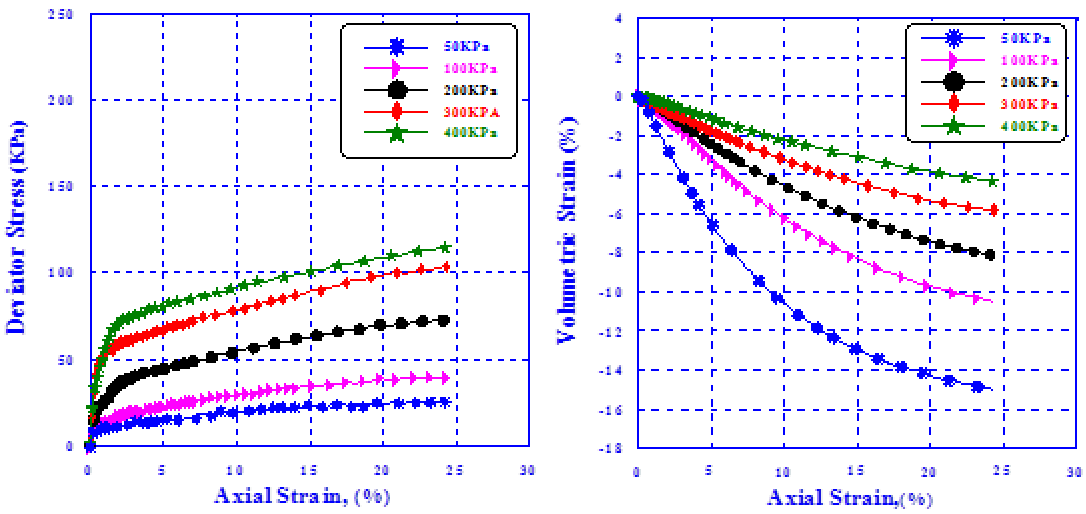
Generally speaking, advantages of the triaxial compression test overweigh its disadvantages, and test results appear reasonable and credible. It is considerably adaptive to measure shear strength of soil–root composite. This work treated roots as a new reinforcement material to study shear strength of unreinforced soil and composite comprised of roots of acacia pycnantha. Reasonable results show that the triaxial compression test is a valid method for further study on shear strength of roots reinforced soil. (Zhang et al, 2009). A series of undrained (CU) and drained (CD) type triaxial compression tests were performed on comparable unreinforced and fibre-reinforced specimens of Chlef soil to evaluate the effective stress-strain-pore pressure and effective stress-strain-volume change behavior of soil and reinforced soil.

V-2 MONOTONIC COMPRESSION TRIAXIAL TESTS

V-2-1 DRAINED TRIAXIAL COMPRESSION TESTS

V-2-1-1 unreinforced soil

The results of drained compression tests under monotonic loading conditions are shown in Figure (V-1) .Five confining pressures of 50, 100, 200, 300 and 400 kPa were applied to each sample. It can be observed that as the confining pressure increases, the shear strength increases further. In contrast, there is not a peak deviatoric stress despite large confining pressure (fig V-1(a)). The increase results from the role played by the fines in increasing the contractancy phase of the unreinforced soil. The Figure V-1(b) shows that the volumetric strains remain in the contractancy phase in spite of the increase in confining pressures which is also in conformity with Consoli et al.’s (2003) observations. for fine soils (silt or clay) the volumetric strains decrease with the increase in confinement, while sandy soils the volumetric strains increase with increase in the confinement (Chen and Loehr.(2008).

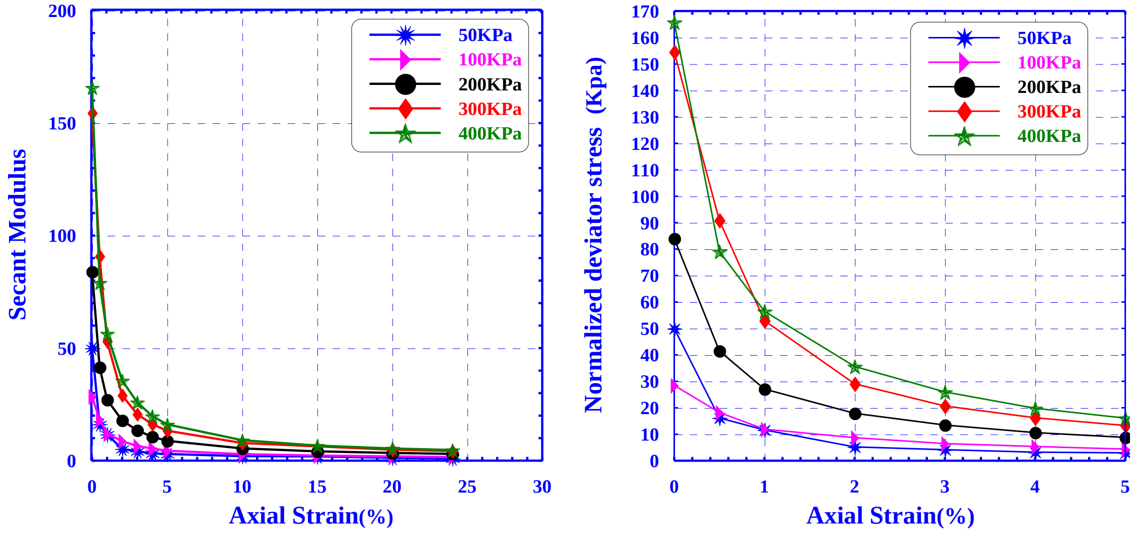


(a) (b)

Figure V- 1. Curves of Drained Triaxial Tests: (a) Deviator Stress (q). (b) Volumetric Strain versus Axial Strain (εp) with different confining pressure

Figure V- 2 (a) and (b) illustrates the evolution of secant modulus and normalised stress with respect to axial strain with different confining pressure. It can be seen in figure V- 2(a) that the deformation modulus decreases sharply with increasing axial strain up to 5% and then stabilizes for all confining pressures. This increase in modulus resulting increase in soil stiffness for large confinements, the same results can be found in Chen et al (2008).

The drained triaxial compression tests clearly indicate an increase in the normalised stress of the unreinforced soil investigated (Chlef soil) with the increase in confining pressure as shown in Figure V- 2(b).



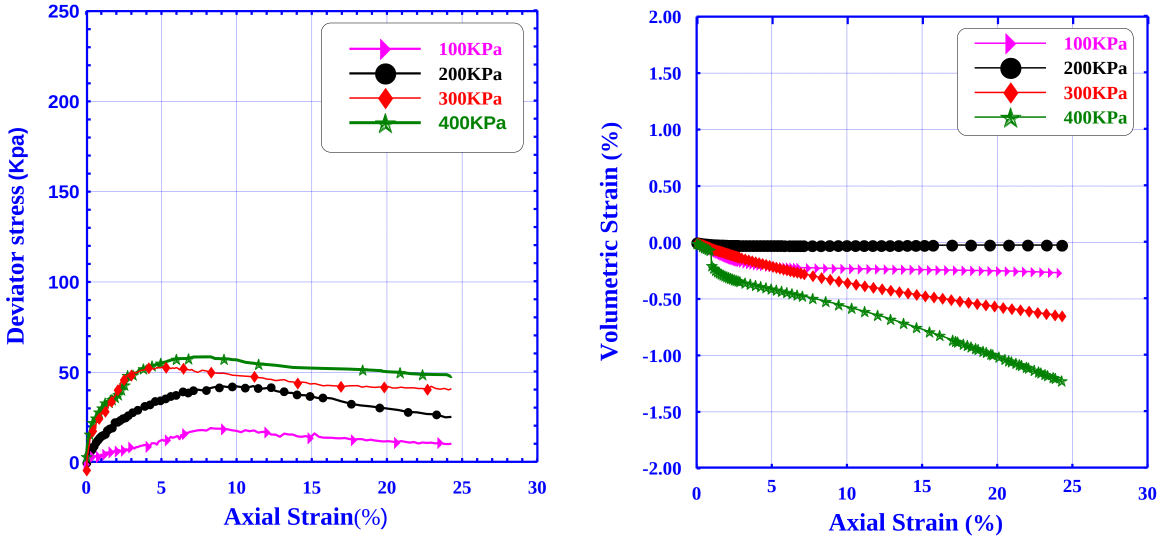
(a) (b)

Figure V- 2: Influence of the Confining Pressure: (a) Secant Modulus. (b) Normalized deviator Stress versus Axial Strain

V-2-1-2 Soil reinforced with vertical root

The results of drained monotonic tests showed that the shear strength of reinforced soil with vertical root (root diameter =12mm) increased steadily with increasing axial strain and gradually fell down when the maximum strength achieved (see Figure V-3a). Moreover, it was observed that the samples studied increased in the value of shear strength as the confining pressure increasing. It is found that the shear strength can be drained influenced by the presence of roots, after a peak deviator stress decreases until the end of shearing.

On the other hand, the volumetric response was hardly affected by roots showing for soil reinforced a slight expansion for low confining pressures, changing into an increasingly compressive behavior for higher confining stresses, Figure V- 3b. This increase is reflected in the fact that the vertical roots do not prevent the volumetric strain.



(a) (b)

Figure V- 3: drained Response of soil reinforced with vertical root (diameter = 1.2mm) a- deviator stress, b- volumetric strain versus axial strain

The variation of the secant modulus versus of the axial strain is illustrated in figure V- 4.it is found that the secant module decreases for all confining pressures up to 10% axial strain, beyond that it stabilized.

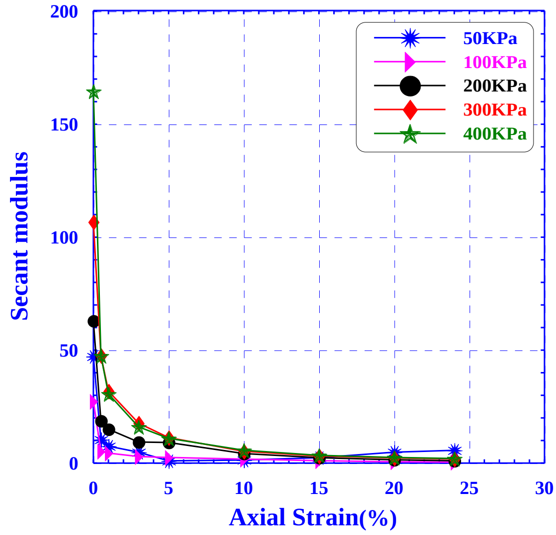
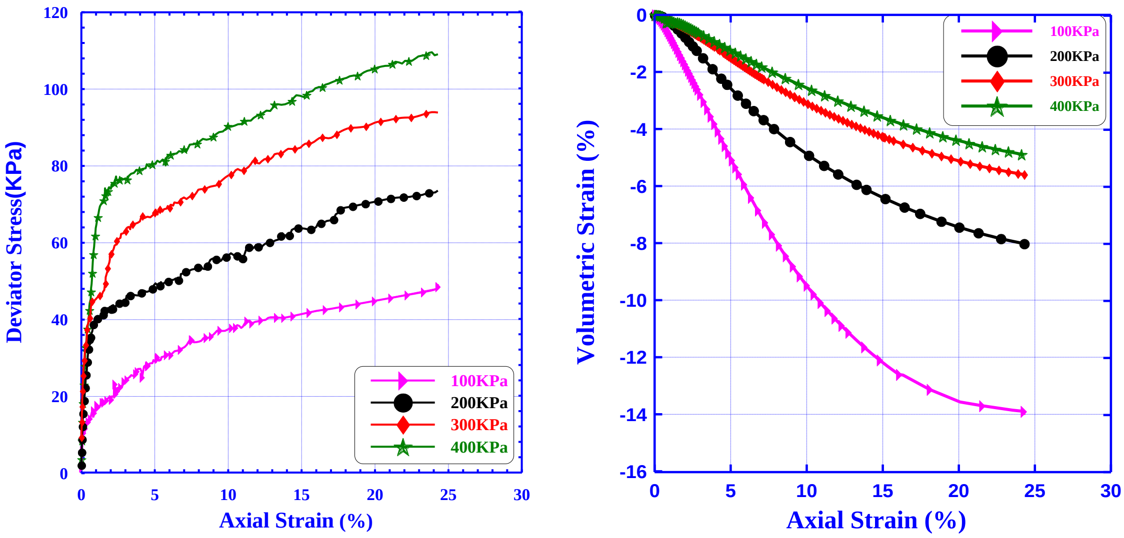


Figure V- 4: variation of the secant modulus versus axial strain (root diameter = 1.2mm).

V-2-1-3 Soil reinforced with three vertical roots

Figure V- 5(a) illustrates the change a of the shear strength characterized by the deviatoric stress versus the axial Strain of the soil reinforced with tree root with 3.5 mm in diameters and 30mm in length. It is found that the shear strength gradually from 50 kPa, 75, 95 to 110 kPa for consolidated reinforced soil under an isotropic stress 100, 200, 300 and 400 kPa respectively. The deviator stresses (q) in good approximation indicate a positive correlation with the confining pressure.

Figure V- 5(b) shows the evolution of volumetric strains versus the axial strain, we note that the presence of root in the soil reduces the volumetric strains and therefore contractancy.



(a) (b)

Figure V- 5: Response drained of reinforced soil (root diameter = 3.5mm)

a- Evolution of deviatoric stress, b-Evolution of volumetric strain

Figure V- 6 shows experimental results of the monotonic triaxial tests of the deviatoric stress versus confining pressure, it was observed that the shear strength is almost linearly proportional to the confining pressure (σ_n). We assumed that the drained shear strength increases with the increase of the confining pressure according to the following expressions (1),(2) and (3). The increase in shear strength was due to the increase of the number of the vertical root of Acacia pycnantha compared to the unreinforced soil. Previous studies indicated that vertical root helps in plant establishment on slopes as it increases the pull-out resistance where surface movement are frequent and it also anchored the soil to improve the resistance (Anisuzzaman et al. 2002; Schroeder 1985).

Unreinforced Soil $\quad Y = 0.267431 * X + 14.734762 \quad$ (1)

Soil with one root $\quad Y = 0.099986 * X + 7.847816 \quad$ (2)

Soil with three root $\quad Y = 0.199848 * X + 31.470345 \quad$ (3)

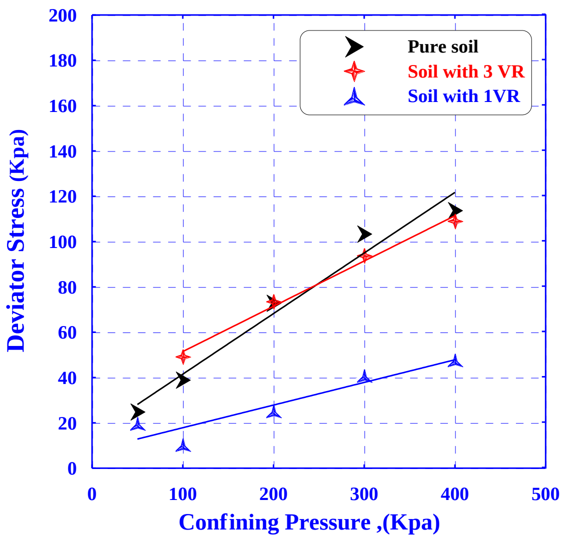


Figure V- 6: Deviatoric stress (q) versus confining pressure curve from CD tests for reinforced and unreinforced samples.

The results of secant modulus versus axial strain of soil reinforced by vertical roots of acacia pycnantha with different confining pressure shows that this module decreases sharply up to 5% axial strain and then stabilized beyond 5%. (See Figure V- 7)

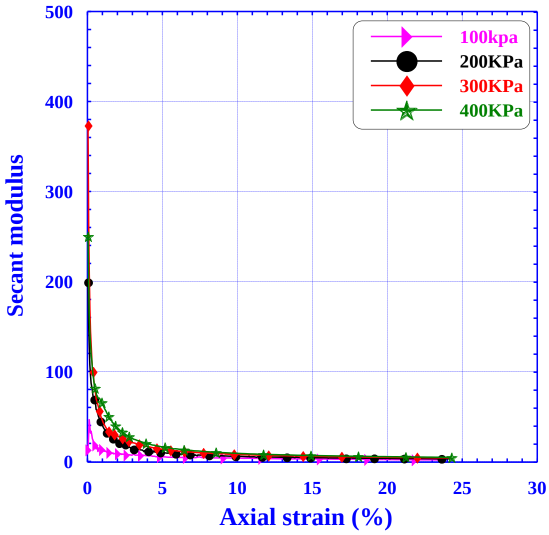
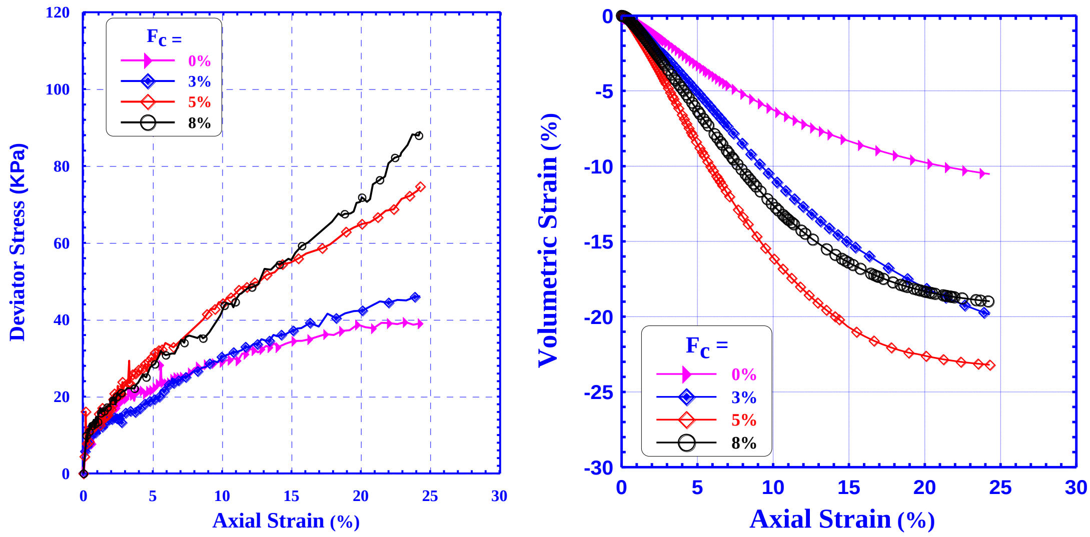


Figure V- 7: Variation of Secant Modulus versus Axial Strain

V-2-1-4 Influence of fibre-reinforced drained silty soil

The drained monotonic compression triaxial tests also have been carried out on specimens reinforced with 3% ,5% and 8% fibre content (Fc) to compare the effect of different fibres on stress-strain behaviour of the soil under an effective confining pressure σ 'c = 100 kPa. The results showed that the shear strength increased steadily with increasing axial strain (see Figure V- 8(a). Moreover, it was observed that the samples studied increased in the value of shear strength as the fibre content increasing. This increase follows a nearly linear trend for the soil with 8% fibres. The deviator mobilized at the end of shearing soil from 40 kPa to 90 kPa when the fibre content (Fc) ranging from 0% to 8%. On the other hand the fibre effects the volumetric strain as it is clears in figure V- 8(b) that the volumetric strains increase with increasing fibre content up to 5% in contrast there is a decrease of deformation for the soil reinforced with 8% fibre, which shows that beyond the value of 5% fibre content (Fc), fibre reinforcement has an adverse effect on strength gaining of the reinforced soil. It can be inferred from the results that the failure strain increase with increase in fibre content(Fc) which results a decrease in contractancy phase of reinforced silty soil and consequently to show a dilative behaviour for soil reinforced with 8% .

Fibres used in this study appear to contribute to strength gaining process. The results suggest that there is a limiting fibre content that beyond that value the fibre reinforcement of soil has an adverse effect rather than improving effect on its strength. The limiting fibre content(Fc) which is called optimum fibre content is unique for each type of fibre. These results are in good agreement with the observations by (Dall’aqua et al.2010; Stefania Lirer et al.2011).



(a) (b)

Figure V- 8: Experimental results of the monotonic drained triaxial tests on specimens reinforced with fibres at confining pressure σ'$_3$= 100Kpa. (a) - Deviator stress, (b)- volumetric strain. Versus axial strain

Figure V- 9 illustrates the variation of secant modulus of deformation versus axial strain with different fibres content (Fc) .we notice that an increase in the fibres content leads to an increase in the secant modulus. The increase results from the increasing fibre content up to 8%, and then we note that there is a little influence on this modulus beyond 3% axial strain when the modulus tends to stabilize.

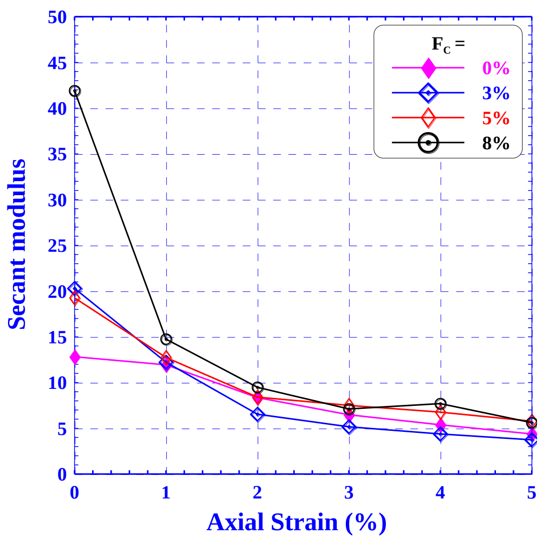


Figure V- 9: variation of Secant modulus versus axial strain with different fibres content.

Figure V- 10 shows the normalized drained deviator stress versus axial strain. We note that the reinforced soil with 3% begin to participate and improve resistance after 10% axial strain. For soil reinforced with 5% and 8% fibre content begin actively to improve soil shear strength after only 2% axial strain. We concluded that the triaxial test results showed that the addition of root fibres significantly improved the behavior of the soil.

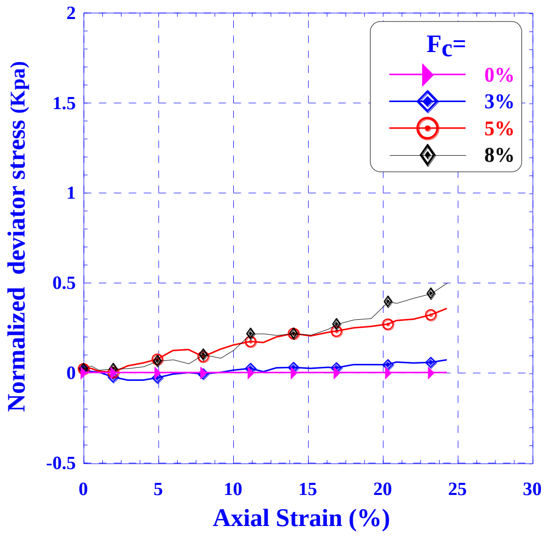


Figure V- 10: drained compression tests (σ 'c = 100 kPa) Variation of normalized deviator stress

The results of drained compression tests for stabilized silty soil samples with root fibres are shown in Figure V- 11. The figure shows the change in the friction angle mobilized versus fibre content. It appears that the friction angle mobilized increases linearly with increasing fibre content (Fc). The results showed the friction angle to be barely affected by fibre content, increasing from 9 to 18°. Additionally, roots have little influence on the friction angle of root-reinforced soils with respect to that of root-unreinforced soils (Gray and Ohashi, 1983).

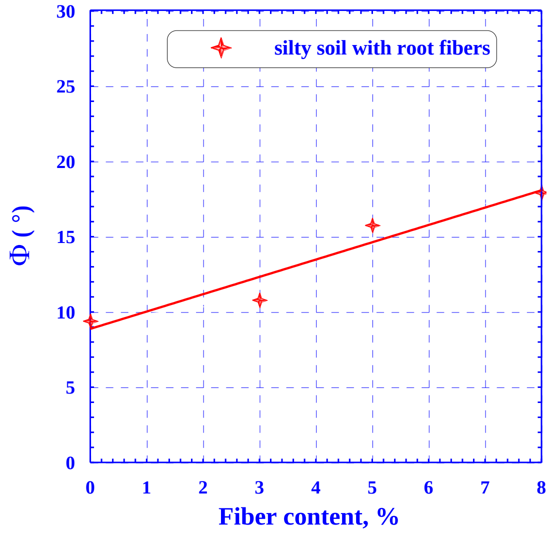
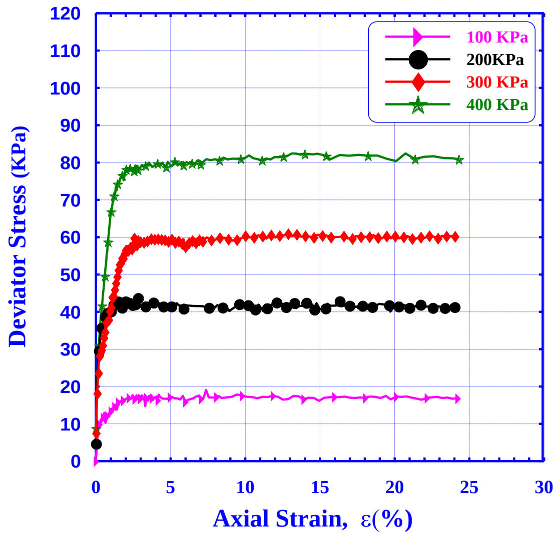


Figure V- 11: variation of friction angle with fibre content for reinforced silty soil

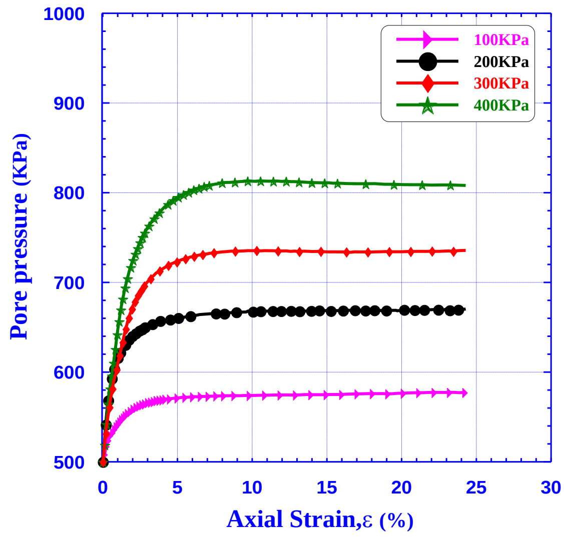
V-2-2 UNDRAINED TRIAXIAL COMPRESSION TESTS

V-2-2-1 Unreinforced Soil

Undrained triaxial compression tests were performed to determine the shearing resistance of the soil samples at 100,200,300 and 400Kpa confining pressure. It is found that the undrained shear strength increases with increasing confining pressure. The deviator stress reached a peak of 18, 44, 60 and 80 kPa and then decreases slightly mobilizing a residual force until the end of shearing (see Fig V-12 (a)). On the other hand the evolution of pore pressure versus axial strain with different confining pressure is illustrated in figure V-12(b), we note that the unreinforced soil quickly generates pore pressure of 575, 660, 725 and 800 kPa until the value of 5% axial strain beyond that a stabilization of the pore pressure until the end of shearing. This increase results from the role of fines in increasing contractancy behavior of soil observed in the drained compression tests.



(a)



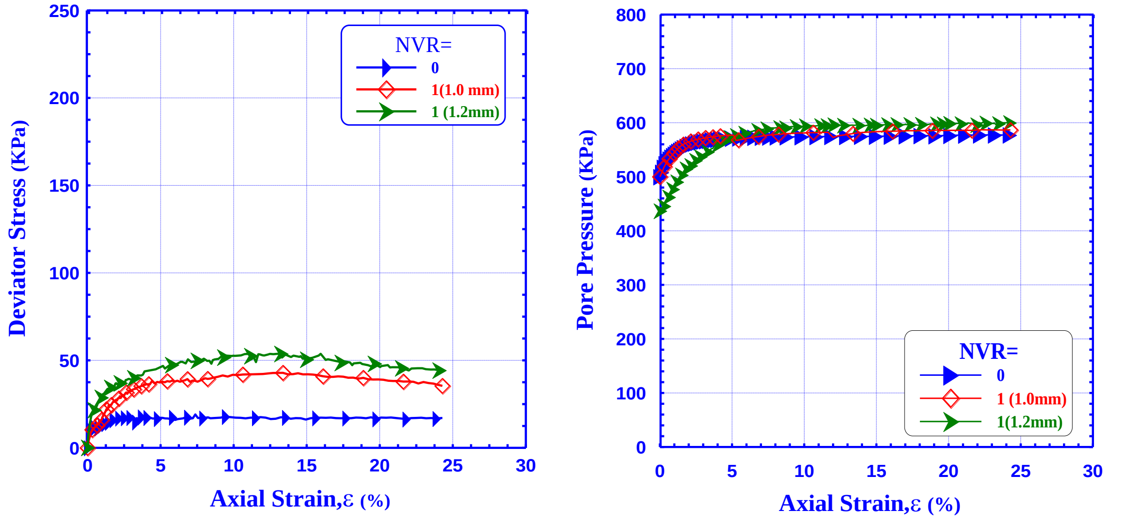
(b)

Figure V-12: Undrained response of unreinforced soil: (a) - deviatoric-stress (b) - pore pressure, versus the axial strain

V-2-2-2 Effect of Root Diameter on Reinforcing Soil with Vertical Root

The results of compression undrained triaxial tests performed on reinforced soil with different vertical roots of diameter ranging from 10 mm and 12 mm are illustrated in Figure V-13 (a). It is clear from this that the undrained shear strength increases with increasing root diameter, the resistance reached a peak deviator stress 40 and 52 kPa for soil reinforced by a root diameter 10mm and 12 mm respectively; here there is a significant improvement in the resistance with large diameters. In current engineering practice and research, root stresses mobilized in root-permeated soils subjected to shear are generally assumed to reach the ultimate conditions for evaluating the shear strength increment provided by roots (Wu et al., 1979; Greenway, 1987; Coppin and Richards, 1990; Abernethy and Rutherfurd, 2001). Furthermore, roots with large diameters tend to act as individual anchors rather than as contributors to the shear strength of the soil (Coppin and Richards, 1990).

Figure V-13(b) shows the evolution of pore pressure versus axial strain. It is clear from this figure that an increase in diameter of the roots leads to an increase in the pore pressure.

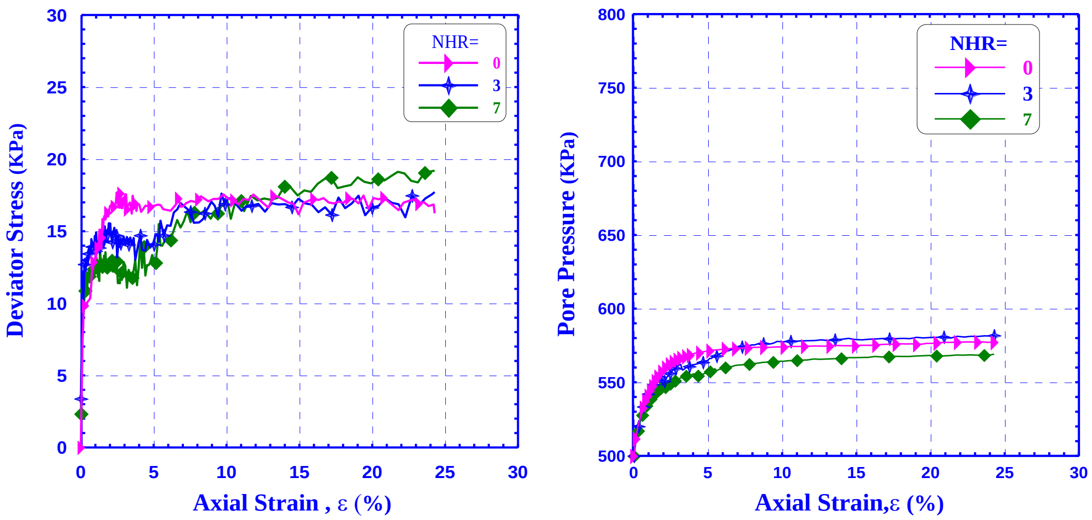


(a) (b)

Figure V-13: undrained response of reinforced soil (root diameter 10 and 12 mm) with number of vertical roots (NVR). a- deviator stress, b- pore pressure, versus axial strain

V-2-2-3 Soil Reinforced with Horizontal Root

The results of undrained triaxial compression tests are illustrated in Figure V-14(a).we note that there is not improved soil resistance despite the presence of the number of roots up to 10% axial strain beyond this value there is a slight improvement in resistance of soil reinforced by seven roots with 0.7mm in diameter (see fig V- 14(a)). Figure V-14(b) shows the evolution of pore pressure versus axial strain. A slight decrease in pore pressure with increasing number of roots, the soil reinforced by seven roots shows its pore pressure stabilized at around 565 kPa, while the soil reinforced by three roots and unreinforced soil the pore pressure stabilized at around 580 and 575 kPa respectively. This decrease results from the role of the fibres to decrease the contractancy phase of the reinforced soil leading to an increase in the dilatancy phase.



(a) (b)

Figure V-14: Undrained response of soil reinforced with different number of horizontal roots (NHR). a- deviator stress, b- pore pressure, versus axial strain

Figure V-15 illustrates the variation of the normalized undrained deviatoric stress versus axial strain, It seems that the normalized undrained deviatoric stress decreases sharply until the value 5% axial strain, beyond that it increases and begin actively to participate in improving the soil shear strength from 15% axial strain; beyond this value, the reinforced soil with seven roots continues to increase in shear strength rather than the reinforced soil with three roots and both of them contribute in improving soil shear strength.

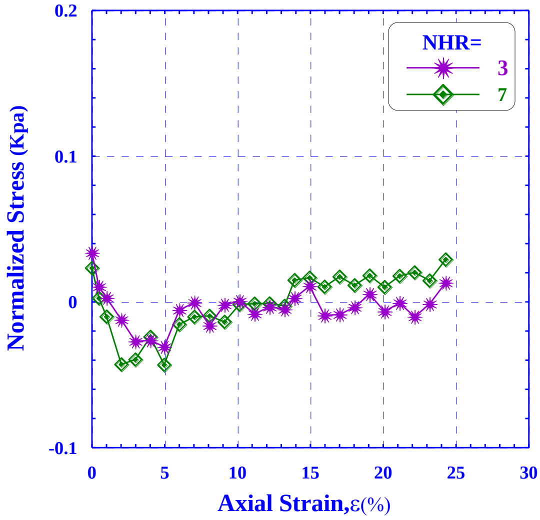


Figure V-15(c): Curve of the normalized stress versus the axial strain

V-2-2-4 Reinforced Soil with Vertical Root

Figure V-16 illustrates the undrained monotonic compression triaxial test results carried out under an initial confining pressure of 100 kPa for different numbers of vertical roots. Figure V-16(a) shows the variation of the deviatoric versus axial strain; we note that the undrained shear strength of root-reinforced soil with 11 mm in diameter increases with increasing organic fibres, this increase is very significant for the soil reinforced with four roots. The value of the deviatoric stress mobilized reached 150 Kpa; 40 kPa for the soil reinforced with three roots and 20 kPa for the unreinforced soil. , it is emphasized that as the numbers of root increases as the soil shear strength increases further. The increase in soil strength was due to the large number of vertical root.

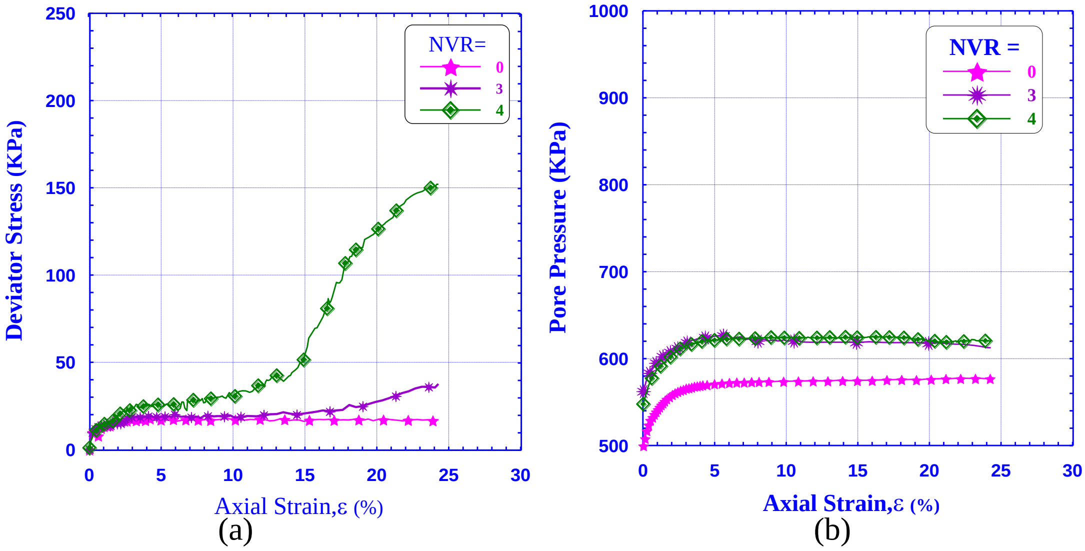


Figure V-16: Experimental results of the monotonic triaxial tests. Deviatoric stress versus axial strain

FigureV-16(b) shows the results of undrained compression tests carried out for different number of vertical roots at 100 kPa confining pressure. We notice in general that an increase in pore pressure up to 5% axial strain, beyond this value it is became to stabilize for all the specimens unreinforced soil and root-reinforced soil.

The effect of the number of root is illustrated in Figure V-17. The evolution of the normalized deviator stress versus axial strain, we note that the sample reinforced by four roots has a greater influence on the strength than the sample reinforced by three roots .the reason may be due to the difference in number of roots. We observe that the soil-root composite (four roots) begins in improving soil strength after 5% of axial strain, when the soil –root composite (three roots) was reached 15% axial strain and beyond this value begins in improving soil shear strength. This figure shows clearly that the shear strength of two composites is higher than that of unreinforced soil under the same confining pressure. It is observed from these results that roots can improve the shear strength of unreinforced soil. Although this study used a different testing method from most previous studies (Day, 1993; Ali and Osman, 2007) for soil–root composites, the same conclusion was obtained that roots can reinforce soil in improving soil shear strength.

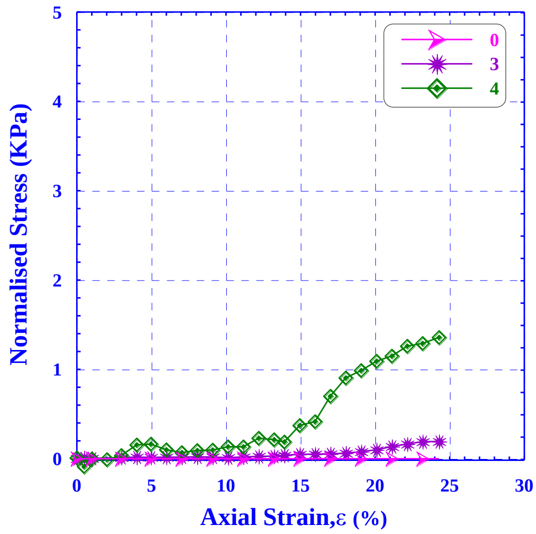


Figure V-17: Curve of normalized deviatoric stress versus axial strain

V-3 EFFECT OF ROOTS ON THE FRICTION ANGLE OF SOIL

The estimated of the angle of internal friction [Φ'] for unreinforced and root-reinforced specimens with different confining pressure is shown in Figure V-18.the results show that the angle of internal friction decreases with respect to increase in confining pressure. It is apparent that change in friction angle of composites, due to presence of roots in soil, the final effect of roots in soil results an increase in the shear strength of soil under different confining pressure. Changes in friction angle have insignificant effects on the shear strength, which is also observed in the study of Liu et al. (2006).Our results are in perfect agreement with those found in the literature (Tatsuoka et al. 1986; Flavigny. 1990; Al Mahmoud. 1997; Arab. 2008).

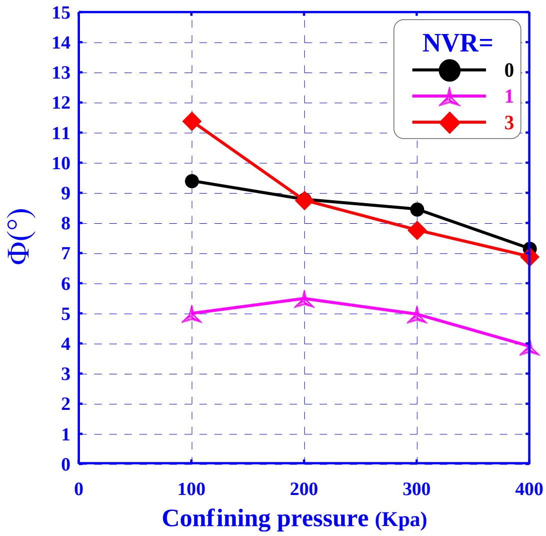


Figure V-18: variation of the friction angle with respect to confining pressure

Figure V-19 shows the variation of the friction angle based on the number of roots. The angle of internal friction decreased significantly with increasing confining pressure. The roots' effect on the unreinforced soil is reflected by an increase of 9° in the angle of internal friction, compared to root-reinforced soil samples at the same confining pressure (100Kpa). Furthermore, the friction angle of the root-reinforced soil samples with three roots (11.80°) was higher than that of unreinforced soil(7°) and the root-reinforced soil samples with one root(5°) at the same confining pressure of 400Kpa, We note that the internal friction angle of composites decreases in one case with increasing confining pressure; and increases in the other case with increased in number of roots and the range of changes in friction angle of the composites(with one root) is much less than that in composites samples with three roots.

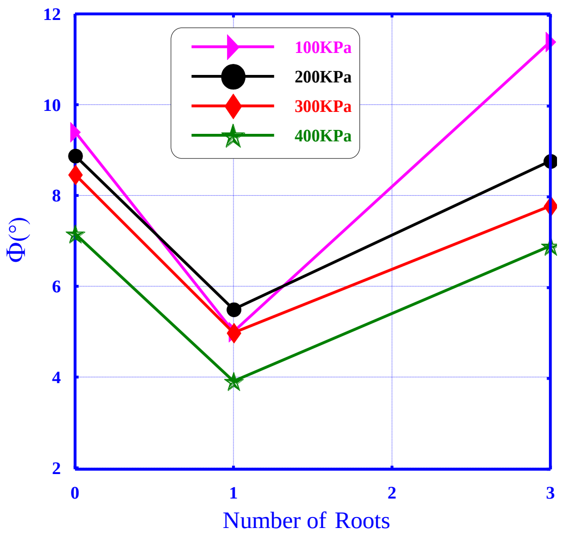


Figure V-19: variation of the friction angle with different confining pressure

V-4 EFFECT OF DEGREE OF SATURATION ON SHEAR STRENGTH OF SOIL

Different experimental methods such as a triaxial test can be used to investigate the effect of saturation on shear strength of reinforced soil. The effect of saturation on reinforced earth has already been studied by Ashaari (1990), and Elias et.al (1983), to evaluate the effect of reinforcement in submerged soil, a triaxial tests were carried out in consolidated-undrained condition at an initial confining pressure of 100 kPa. Figure V-20 shows the stress- strain curves for reinforced and unreinforced samples with different degree of saturation ranging from 1, 49% to 90%. It is clear from the results that the saturation has a significant effect on the shear strength of soil, and saturated reinforced samples exhibit higher shear strength than unreinforced sample under the same confining pressures; but the high strength of fully saturated reinforced sample because of negative pore pressure generated in the soil due to dilation of soil during shearing.

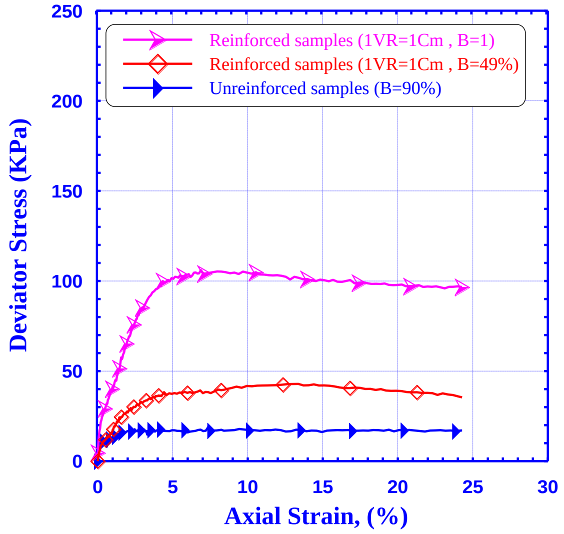


Figure V-20. Stress- Strain Curves for Reinforced and Unreinforced Samples

V- 5 CONCLUSIONS

In this chapter the results of the triaxial tests performed on medium-dense silty soil show that addition of roots can improve the strength of soil under undrained and drained loading conditions. Shear strength parameters of volumetric strain, pore pressure and friction angle increase significantly in the CU and CD tests, due to the addition of fibres. It is noted that the reinforcing fibres alter the pore pressure response of specimens tested under drained loading conditions and the volume change response of specimens tested under drained loading condition.

Among the forms of roots distribution, the vertical root distribution has the most significant effect on reinforcing soil. Vertical roots improve soil shear strength and result in better reinforcing effects. Horizontal roots composites result small increase in soil shear strength. The presence of vertical roots and root fibres in soil substantially increased the soil shear strength as well. Also the number of roots play important role in improving soil shear strength and friction angle. Additionally, root orientation plays an important role in the mobilization of root stresses and the associated shear strength increment because roots within the soil undergo shear deformation. It can be concluded that a saturated reinforced soil exhibits a high strength than that of unreinforced one; and noted that the improvement of shear strength of reinforced soil due to saturation.

CONCLUSIONS

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This work was carried out to study the behavior of reinforced and unreinforced soil by tree roots and root fibers of acacia pycnantha as mentioned in this study. It consists of laboratory tests on different loading paths and in different soil conditions. The following conclusions were drawn from this investigation on reinforced and unreinforced soil subjected to monotonic loading are:

- The position of the roots in the soil plays an important role to improve the shear resistance of the soil. The roots horizontally decrease the volumetric behavior resulting in a considerable increase in shear strength, while the vertical roots rise against the volumetric behavior resulting in an amplification phase contractancy.

- The positive effect of the fiber reinforcement was clearly established during the analysis of the data obtained from the testing. However, the data obtained from the tests is accurate and extensive to establish the specific effect of the root fibers on the soil.

- Drained triaxial tests showed that the addition of root fibers to the soil specimens increased the value of shear strength of the soil. The addition of fibers results in substantial increase in the values of the friction angle and secant modulus until 8% fiber content.

- Undrained monotonic tests showed that the layout and root diameter plays an important role in the undrained shear strength. The soil reinforced by roots arranged horizontally plays no role in improving soil strength, while the roots placed vertically in a manner significantly improve the undrained shear strength. Also, the number of roots plays a role in improving the drained and undrained shear strength.

- The excellent performance of the triaxial compression test implies that it will play an important role in further evaluation of reinforcing effect of roots on soil shear strength.

- The results are a very good data base for development and validation of numerical models. It would be interesting at first to test existing models and to determine parameters for soil types and then use finite element codes to study the behavior of structures in sites with a risk of instability.

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