EROSION STUDIES ON LITHOMARGIC CLAYS AND SLOPE STABILITY STUDIES OF EXCAVATED SLOPES IN LATERITIC FORMATIONS

Thesis

Submitted in partial fulfilment of the requirement for the degree of

DOCTOR OF PHILOSOPHY

By

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D E C L A R A T I O N

by the Ph.D. Research Scholar

I hereby *declare* that the Thesis entitled "**Erosion Studies on Lithomargic Clays and Slope Stability Studies of Excavated Slopes in Lateritic Formations"** which is being submitted to the **National Institute of Technology Karnataka, Surathkal** in partial fulfilment of the requirements for the award of the Degree of **Doctor of Philosophy** in **Civil Engineering**, is a *bonafide report of the research work carried out by me.* The material contained in this Research Thesis has not been submitted to any University or Institution for the award of any degree.

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Date: 21/01/2020

C E R T I F I C A T E

This is to *certify* that the Research Thesis entitled **"Erosion Studies on Lithomargic Clays and Slope Stability Studies of Excavated Slopes in Lateritic Formations"** submitted by **Ms**. **BIJI CHINNAMMA THOMAS**, (Register Number: **138003CV13F04**) as the record of the research work carried out by her, is *accepted as the Thesis submission* in Partial fulfillment of the requirements for the award of degree of Doctor of Philosophy.

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DEDICATED TO

MY FAMILY

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ABSTRACT

This study is done in three parts. In the first part, a brief study of the geology of the area is being made, especially that of laterites and lateritic soils. The study area is coastal Karnataka in India.-This coastal area receives copious amount of rainfall and a lot of developmental activities are taking place. The soil stratification in lateritic areas consists of lithomargic clays, which are products of laterization, sandwiched between the hard and porous weathered laterite crust at the top and the hard parent rock of granite or granitic gneiss underneath. These lithomargic clays which are extensively used in construction purposes behave as dispersive soils and are found to be highly erosive.

In the second part of this research work, erosion characteristics of lithomargic clays are being studied in detail since very few and detailed studies on erosion of lithomargic clays are available in literature. A number of hole erosion tests are conducted on controlled lithomargic clay samples with varying percentage of fines. The influence of degree of compaction, moulding water content, head causing flow, percentage silt content and plasticity index on the erosion rate index and critical shear stress of controlled lithomargic clay samples are being studied. The results of this study indicate that the critical shear stress for soils with higher silt fraction and fine sand content varied from 45 to 125N/m^2 whereas for soils with higher clay fraction and fine sand content the critical shear stress varied from 200 to 400N/m^2. The erosion rate increased with a decrease in percentage compaction in all the samples and critical shear stress is found to be highest at optimum moisture content conditions. It is generally observed that soils with fines whose plasticity indices are high, are less erodible compared to soils with fines whose plasticity indices are low.

Excavated slopes for railway and highway projects in such lateritic formations are posing serious erosion and slope stability problems, especially, due to the presence of these lithomargic clays and seepage pressures from stagnated water at top. In the third part of this study, slope stability analyses of excavated slopes in lateritic formations is being conducted using the software Plaxis 2D. Slope stability analyses is actually a very complex problem which should take into consideration the combined effect of geotechnical [berm position, height and slope of excavated slope, soil properties etc.], hydrological [precipitation, ponding at top and seepage through the slope etc.] and biological [vegetation (trees at various positions and turfing on slope), wind action on trees etc.] factors in addition to erosion (both surface and internal) problems. In this research work, the influences of these various factors on slope stability are being studied separately. Some of these factors have a positive influence by increasing the factor of safety of slopes, whilst others have a negative influence. When trees are provided at the toe of the slope a percentage increase in factor of safety up to 12% and 6% is observed for drained and undrained conditions respectively. When turfing along with trees are considered, a percentage increase of factor of safety up to 15% and 12% is observed for drained and undrained conditions respectively.

Keywords: lithomargic clay, hole erosion test, erosion rate index, critical shear stress, slope stability, vegetation, lateritic formations

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NOMENCLATURE

Abbreviations

HET	Hole Erosion Test
OMC	Optimum Moisture Content
PI	Plasticity Index

Notations

Q	Flow rate
ϕ_t	Hole diameter
ε_t	Rate of erosion or Erosion rate
C_e	Coefficient of soil erosion
τ_t	Hydraulic shear stress
τ_c	Critical shear stress
I_h	Erosion rate index
f_L	Friction factor for laminar flow conditions
f_T	Friction factor for turbulent flow conditions
g	acceleration due to gravity
s	Hydraulic gradient across the hole
ρ_w	Density of the eroding fluid (water)
ρ_d	Dry density of the soil

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Slopes either occur naturally or are built by humans. These built slopes are not engineered most of the time. They are built or cut with very high and steep slopes. Slope stability problems have been faced throughout history when human being or nature have disrupted the delicate balance of natural soil slopes and sometimes forces of nature has also contributed it. Furthermore, the increasing demand for engineered cut and fill slopes on construction projects has only increased the need to understand all factors affecting its stability and various analytical methods, investigative tools, and stabilization methods to solve slope stability problems. Instability of slopes is not caused merely due to geotechnical reasons or deficiencies. Instability of slopes is also caused due to erosion problems, both surface erosion and internal erosion. Internal erosion is caused due to seeping water. Stability of slopes is also affected by presence or absence of vegetation (turfing or trees) on slopes, hydrological factors like intensity of rainfall, surface flow etc. Slope stabilization methods involve specialty construction techniques that must be understood and modelled in realistic ways.

Lateritic formations pose a special type of slope stability problems, especially in excavated slopes. Such formations are geologically unique, in that they have a hard lateritic crust at top 1-3m thick underlain by soft, dispersive and highly erodible lithomargic clays. Laterites although can be intact and quite strong, they are highly porous and underlying lithomargic clays which are products of laterization are dispersive and highly erodible unless confined. These lithomargic clays are typically sandy silts to silty sands with very little or no clay content. Laterites occur in several places all over the world – in African continent, Thailand, India, Australia, South East Asia, South Africa, South American continents. Excavated slopes in lateritic formations are highly susceptible to formation of gullies or caving – in during heavy rainfall. Caving – in of the lithomargic layers in excavated slopes is a common sight along NH 66 and along Konkan rail route, right from Trivandrum in south to Mumbai in north.

In the present study, hole erosion tests have been conducted to study the influence of various parameters; such as degree of compaction, moulding water content, head causing flow, percentage silt content, plasticity index; on the internal erosion characteristics of lithomargic clay. Slope stability analysis is conducted using the software Plaxis 2D considering the effect of vegetation (trees and turfing) on and near slope, precipitation, ponding at top and seepage through the excavated slope with different heights and for varying cut slope angles.

1.2 SCOPE OF THE STUDY

The study area considered here is coastal Karnataka in India, where the soil is mainly comprised of laterites and lateritic soils. The soil stratification in this area mainly consists of lithomargic clay, which are products of laterization, sandwiched between the hard and porous weathered laterite crust at the top and the hard granite or granitic gneiss underneath. These lithomargic clays are locally called as ‘shedi soils’. It is highly erosive and is dispersive in nature. Both the top laterites and the lateritic soils (lithomargic laterites and lateritic lithomarges) are extensively used for construction purposes. The hard laterite layer is removed as bricks for construction purpose, thus exposing the underlying lithomargic clay. Lateritic soils are quite often also used for the construction of (man-made) highway and railway embankments and slopes. There are serious problems of erosion of shedi soils in slopes during heavy rainfall since the shedi soils are highly erosive by nature. Excavated slopes for railway and highway projects in such lateritic formations pose serious erosion and slope stability problems, especially, due to the presence of these shedi soils and seepage pressures from stagnated water at the top.

Lithomargic clays, predominantly comprising of fine sand and silt fraction, are a problematic soil since they lose much of their strength when they come in contact with water and behave similar to dispersive soils. Apart from the loss of strength of lithomargic clays due to saturation, erosion (both internal erosion and external erosion) has also contributed to the instability of slopes in the region. Slope and embankment failures in lithomargic soils, especially during monsoon season have caused enormous economic losses. Very few research studies have been reported on the erosion characteristics of lithomargic clay and its impact on the stability of excavated slopes in lateritic formations.

1.3 OBJECTIVES

The current study is aimed at:

1. To study the engineering geology of laterite in the Mangalore area.
2. To study the erosion characteristics of controlled lithomargic clay samples with higher clay fraction with varying percentage of fines by conducting hole erosion tests.
3. To study the erosion characteristics of controlled lithomargic clay samples with higher silt fraction with varying percentage of fines by conducting hole erosion tests.
4. To study the influence of the degree of compaction, moulding water content, head causing flow, percentage silt content and plasticity index on the erosion rate index and critical shear stress (erosion characteristics) of controlled lithomargic clay samples.
5. To analyse the slope stability problem in Plaxis 2D considering the effect of precipitation, ponding, vegetation and turfing on excavated slopes for varying slope heights and slope angles.
6. To analyse the slope stability problem in Plaxis 2D considering the effect of precipitation, ponding and piping on excavated slopes for varying slope heights and slope angles.

1.4 ORGANISATION OF THE THESIS

The scopes, objectives, organisation of the presentation of the thesis and major contributions from the research work have been discussed in **Chapter 1.**

In **Chapter 2**, a literature review regarding the geology of laterites and lateritic soils in Dakshina Kannada has been discussed. Literature relevant to erosion, hole erosion test and the various studies conducted using hole erosion test is also reviewed. Also, the literature on stabilization techniques on soil using vegetation, i.e. both turfing and trees are reviewed in this chapter. This chapter also reviews the various studies conducted on slope stability analysis using PLAXIS software mainly focusing on the effect of precipitation, seepage, vegetation and piping erosion on the stability of slopes

Chapter 3 discusses the hole erosion studies conducted on lithomargic clays. The geotechnical properties of controlled samples of lithomargic clay containing higher clay content and higher silt content are studied discussed in this chapter. The hole erosion tests were conducted on these soil samples, and the erosion characteristics of these soil samples are studied. The details of the hole erosion test apparatus and the test procedure conducted are discussed in this chapter.

Chapter 4 discusses the slope stability analysis studies done on excavated slopes with lateritic formations. The method of slope stability analysis done in PLAXIS 2D considering the effect of vegetation (trees and turfing) on and near slope, precipitation, ponding at top and seepage through the excavated slope with different heights and for varying cut slope angles are discussed in this chapter.

Chapter 5 deals with the detailed results obtained from both hole erosion tests and slope stability analyses. The effect of moulding water content, degree of compaction, hole diameter, head causing flow, percentage silt content, plasticity index; on the internal erosion characteristics of lithomargic clay are discussed in this chapter. The results from the slope stability analyses conducted in PLAXIS to study the effect of vegetation, excavation, precipitation, seepage and berm position are discussed in this chapter.

The major conclusions of the investigations are summarised in **Chapter 6** of this thesis.

1.5. MAJOR CONTRIBUTIONS OF THIS RESEARCH WORK

The major contributions of this research work are:

1. Hole erosion tests are carried out at various heads on controlled lithomargic clay samples (with higher and lower silt content) to study the erosion characteristics of the soil.
2. The influence of various factors such as moulding water content, degree of compaction, head causing flow on the erosion index and critical shear stress of lithomargic clays have been studied.
3. The influence of the silt content on the erosion index was also studied in this research work.
4. The influence of the plasticity index on the erosion index at critical head and saturation conditions have also been studied. It has been confirmed from experimental results that increase in plasticity index reduces the erosion potential of the soil.
5. The effect of precipitation, excavation, wind action, ponding on lateritic formations has been analyzed by varying the cut slope angles and slope heights.
6. The effects of berm and vegetation, i.e. both trees and turfing have been analyzed on excavated slopes in this study.
7. An attempt to study the effect of piping on the excavated slope has also been carried out in this study.

CHAPTER 2

LITERATURE REVIEW

2.1. INTRODUCTION

Slope stability is one of the most unpredictable and complicated problems in geotechnical engineering projects and the reason of this unpredictability can be returned to uncertainty in several factors such as condition and location of failure, geotechnical properties of soil and history of loading on the slope. This gets even more serious when the slope is on residual soil like laterite, which means that unlike sedimentary soils, the soil strength parameters cannot be obtained trustfully based on stress-strain relationships.

The unique nature of varying strength of laterite and the presence of lithomargic clay extending upto considerable depth, alternate and wetting and drying make the behaviour, properties and analysis of laterite imperative (Daware and Hegde 2010). The heavy rainfall in the recent past has given a wakeup call to all engineers to analyse and prevent damages to cutting, slopes and foundations on lateritic soil.

2.2. LATERITE AND LATERITIC SOILS

The term "laterite" was coined by a Scottish scientist Dr Francis Buchanan in 1807 in India, from a Latin word "later" meaning brick (Makasa,1998). Buchanan suggested the name laterite to describe “ferruginous, vesicular, un-stratified, and porous soil with yellow ochre’s due to high iron content, occurring in Malabar, India”. From a geological point of view, laterite can also be defined as, “a kind of vesicular rock composed essentially of a mixture of hydrated oxides of aluminum and iron with a small percentage of other oxides such as manganese or Titanium” (Gidigasu 1976). It is defined as a soil layer that is rich in iron oxide and derived from a wide variety of rocks that weather under strongly oxidizing and leaching conditions. It forms in subtropical and tropical regions that have a humid climate. Lateritic soils may contain clay minerals; but they tend to be silica-poor, for silica is leached out by waters passing through the soil.

Typical laterite is claylike and porous. The term laterite is often substituted for ferricrete but refers technically to a soil rich in iron oxides and aluminum. In areas where there is extensive leaching, many plant nutrients are lost, leaving quartz and iron hydroxides, aluminium, and manganese. This residue forms a distinctive soil type, called laterite or latosol. Laterite deposits forms on the top mainly due to chemical weathering of rocks laterisation with a high content of hydroxides of iron. After their formation laterites are either denuded by erosion or covered below younger deposits. They may be covered with a (heavy) forest, with the tree roots deeply penetrating into and through the laterite. Their chemical and mineralogical compositions, and their fabric, are largely determined by their parent rocks.

According to Aleva and Creutzberg (1994) laterite (or rather some varieties of it) is formed by a process, by which certain rocks undergo superficial decomposition, with the removal in solution of combined silica, lime, magnesia, soda, potash, and with the residual accumulation, assisted, no doubt, by capillary action, metasomatic replacement, and segregative changes of a hydrated mixture of oxides of iron, aluminium, and titanium, with more rarely, manganese. These oxides and hydroxides of iron, aluminium, titanium, and manganese are designated the lateritic constituents. This residual rock is true laterite, and the presence of any considerable proportion (> 10 percent) of non-lateritic constituents requires expression in the name, as it always indicates a want of completion in the process of laterization. True laterite contains, then, 90 to 100 percent of lateritic constituents. There is often a gradation in composition between true laterite as defined above and lithomarge which is taken as the amorphous compound of composition $2H_2O.Al_2O_3.2SiO_2$,corresponding to the crystalline mineral kaolinite of the same composition. For rocks intermediate between laterite and lithomarge the terms ‘lithomargic laterite’ and ‘lateritic lithomarge’ are available, The former being applied to forms containing 50-90 percent of lateritic constituents, and the latter to forms containing only 25 to 50 percent of lateritic constituents.

Laterite fabrics, as surficial products, laterites are liable to fall prey to erosion, the more so where they occupy topographically high positions in the landscape. This is commonly the case, and this elevated position promotes the necessary high rate of internal drainage. Most present day laterites have survived erosional attacks as a result of their hard to very hard capping with an iron-rich accumulation /one (cuirass), while a few deposits were buried by younger sediments shortly after their formation, and were protected against erosion and subsequent laterite weathering processes of later periods.

The lithomargic clay (shedi soil) which is whitish, pinkish or yellowish silty sand constitutes an important group of residual soils existing under lateritic soils. The composition of these soils is mainly hydrated alumina and kaolinite powder (Achari and Shivashankar 2005). These dispersive soils are the product of tropical or subtropical weathering and are highly susceptible to erosion (Ramesh and Nanda 2007). Engineers have to be extremely careful in handling these types of soil. As long as this soil is dry and confined, there is little or no problem, but on the exposure in cutting or excavated slopes, when it comes into contact with water, it loses its strength drastically. Landslides, slope failures etc., are quite common in this type of soil. Hence the construction works in this type of soil is challenging.

2.2.1. Laterization

Laterization takes place in tropical climatic regions having dry and wet periods. Parent rocks undergo upper facial decomposition, resulting in removal in solution of combined silica, lime, magnesia, potash and with residual accumulation assisted by capillary changes of hydrated mixture of oxides of iron, alumina and manganese (Daware and Hegde 2010). Three stages are required for the completion of laterization.

(a) The first stage (decomposition) includes physico-chemical break down of primary minerals and the release of constituent elements (SiO_2, Al_2O_3, Fe_2O_3, CaO, MgO, K_2O, Na_2O etc.)

(b) The second stage (laterization) involves the breaching under appropriate of combined silica and bases and the relative accumulation or enrichment from outside sources of oxides and hydroxides of sesquioxides (mainly Al_2O_3, Fe_2O_3 and TiO_2)

(c) The third state (dehydration O_2 desiccation) involves partial or complete dehydration (sometimes involving hardening) of the sesquioxide rich materials and secondary minerals.

2.2.2. Types of Laterite

According to the stratigraphy of the area & the composition of the parent rock, the laterites are mainly categorized in the following three types:

(a) Type I

This occurs extensively over the Deccan trap. Lava flows & shows evidence of in situ development. The typical profile of the laterite developed over Deccan traps shows –

(i) Coarse saprolite horizon

(ii) Lithomarge horizon.

(iii) Duricrust

Coarse saprolite is coarse to medium grained with a spheroidal shell of light pink colour. The course saprolite horizon, which is a transition zone is about 10 cm to 1.5 m thick. Coarse saprolite has regular & sharp texture. Lithomarge is fine saprolite occurring as unconsolidated slightly ferruginous formation constituting of kaolinite, sometimes with quartz & accessory goethite & hematite. The lithomarge thickness varies between 10 cm to 30 cm & this horizon is developed & preserved at or below the level of the ground water table. On top of the lithomarge layer, relatively thick reddish brown to the black red indurate horizon that is known as ferruginous cuirrase or the duricrust is present, duricrust composed of considerably accumulation of iron as Fe_2O_3. The thickness deposits range from 0.2cm to 1.5m. Depending on the presence of solid parts of voids, it can be of solid, nodular or pisolite in texture.

(b) Type – II

This laterite has formed in situ over the Miocene sediments. Such laterites are younger to Miocene & may be lower Pliocene. There shows the typical components laterite profile, including the formation of occasional bauxite. On careful examination, topmost duricrust of these laterites shows well rounded laterite & quartzite pebbles within a lateritic matrix.

(c) *Type-III*

This laterite shows the same profile as of Type I laterite in the sense that these are also developed over the Deccan traps. However, within the upper nearly 1 m of the profile they also show the presence of laterite pebbles. This type of laterite is only observed in the coastal regions.

2.3. EXCAVATED SLOPES IN LATERITIC FORMATIONS

Laterite formations are widely distributed throughout the world, but especially in the inter-tropical regions of South Africa, Australia, India, South East Asia and South America. Laterites have been long known in India, where they occupy large areas of Deccan Peninsula. High level laterites cap the summit of hills and plateau on the highlands of central and Western India. Low level laterite is found in long bands along both coasts of Deccan Peninsula. The laterites in this region are essentially the vesicular soils which are the products of tropical or sub-tropical weathering, rich in secondary oxides of iron, aluminium or both, and also a large amount of quartz and kaolinite.

Lateritic soils are abundantly available in the Konkan belt in the western coast of peninsular India, in the four southern states namely - Kerala, Karnataka, Goa and Maharashtra. Along with heavy rainfall (annual rainfall of 2000mm - 4000mm), the region is characterised by high humidity and little variation in temperatures. The typical stratification in lateritic areas consists of soft to the hard lateritic crust at the top – about 3m thick, underlain by a layer of lithomargic clay (8 to 10m thick) underlain by parent rock, which is granitic gneiss. In the districts of Dakshina Kannada, and Udupi, laterite soil can be found to occur above underlying lithomargic clay (or fine silty soil).

A large number of cuttings in laterite soils are encountered while during the implementation of large projects like Konkan Railway Project, Varahi canal Project, Highway projects etc. The depth of the cuttings varied from 1 to 20 m and various geotechnical methods have been adopted to stabilize the excavated slopes. Laterite formations in this area consist of a top hardened vesicular layer that is highly porous, followed by lithomargic clay 1ayer over the weathered residual soil and parent rock. Various techniques of stabilization were used on the exposed surface of lithomargic clay in these areas. Below the top hard laterite, the existed soft clay called 'Lithomargic clay' which is formed due to the leaching of soils. These layers vary in the thickness from a few centimetres to several meters, followed by hard rock.

During the execution of the Konkan railway project that connects two major cities, Mangalore and Bombay, a large number of cuttings are encountered. Of the 482 cuttings, about 70% of the slopes have failed. Two of these major failure of excavated slopes have been studied by Sabhahit and Rao (2004) namely Kulai and Chelar sites and it was found out that improper stabilization techniques led to the erosion of lithomargic clay thus leading to slope failure during the rainy season. The Konkan belt falls on the windward side of the southwest monsoon and, thus, witnesses' heavy rainfall, which often leads to boulder falls and soil slips in the region. Due to the porous nature of laterites present, water seeps into the underlying clay soil. This increases the density of the rocks and decreases their hold in the soil base, which becomes soft due to water absorption, resulting in boulder falling and soil slippage from slopes, cuttings, and tunnels. Various stabilization measures like rock bolting, micro-piling, vetiver plantation, flattening of slopes etc. have been now adopted to decrease the landslide occurrences in these areas.

Lithomargic clays, predominantly comprising of fine sand and silt fraction, are a problematic soil since they lose much of their strength when they come in contact with water and behave similar to dispersive soils. Apart from the loss of strength of lithomargic clays due to saturation, erosion (both internal erosion and external erosion) has also contributed to the instability of slopes in the region. Slope and embankment failures in lithomargic soils, especially during monsoon season have caused enormous economic losses. Various stabilization techniques using cement, quarry dust, GBFS, lime, sand and coir (Nayak and Sarvade 2012; Sekhar et al. 2017; Ravi Shankar et al. .2012; Shivashankar et al. 2015) have been adopted with lithomargic clays to enhance their strength behaviour. SEM and XRD studies (Darshan and Sitaram 2017), electrical resistivity studies (Nimi et al. 2017) have also been conducted on stabilized lithomargic clays. Very few research studies have been reported on the erosion characteristics of lithomargic clay and its impact on the stability of excavated slopes in lateritic formations.

2.4. EROSION IN EXCAVATED SLOPES

Surface and internal erosion, including piping, are the two major problems faced by engineers around the world. If the slopes of dams, channels, highway embankments and cut slopes contain highly erodible soils, then severe surface erosion is inevitable. Around 0.5% (1 in 200) failures of embankment dams and 1.5% (1 in 60) of piping incidents were caused by internal erosion and piping in the past years. Of these failures and accidents, about half are in the embankment, 40% in the foundations, and 10% from the embankment to foundation (Fell et al. 2003).

2.4.1. Factors Affecting Erodibility

The erodibility of a soil can be quantified in terms of the rate of erosion when a given hydraulic shear stress is applied to the soil and the ease of initiating erosion in the soil. This can be expressed in Equation 2.1.

$$\dot{\varepsilon}_t = C_e(\tau_t - \tau_c) \tag{2.1}$$

where $\dot{\varepsilon}_t$ is the rate of erosion per unit surface area of the slot/hole at time t [kg/s/m^2]; C_e is a constant known as the Coefficient of Soil Erosion [s/m]; τ_t is the hydraulic shear stress along the slot/hole at time t [N/m^2]; τ_c is the minimum hydraulic shear stress for initiation of erosion, also referred to as the Critical Shear Stress [N/m^2].

Depending on its type, several factors influence the erodibility of soils. In order to define them, it is necessary to know the erosion parameters, hydraulic shear stress, critical shear stress, and erosion rate.

a) Erosion rate

Many previous researchers have explained the term erosion rate as the amount of soil which eroded during a unit time over unit surface area. Erosion rate indicates how fast a soil erodes under certain hydraulic shear stress.

b) Hydraulic shear stress and critical shear stress

The hydraulic shear stress is the stress applied on the soil surface by flowing water. Critical shear stress is the minimum hydraulic shear stress necessary to initiate erosion, and it depends on factors such as the type of eroding fluid, soil type, degree of compaction, and water content. Most previous researchers defined the critical shear stress as the threshold hydraulic shear stress below which no erosion was observed, as shown in Fig.2.1.(Wan and Fell 2004)

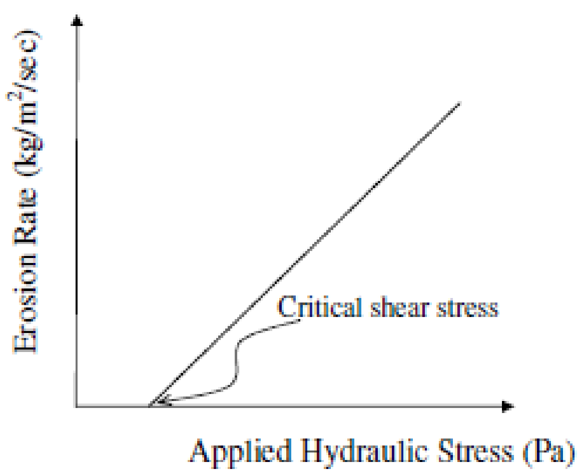


Figure 2.1. Variation of erosion rate with hydraulic shear stress

The erodibility of cohesive soils mainly depends on factors such as hydraulic shear stress, eroding fluid and pore fluid properties, dry density, moisture content, and common soil properties such as the plasticity index and shear strength.

a) Hydraulic shear stress

As shown in Fig.2.1, the erosion rate is direcly proportional to the hydraulic shear stress. It demonstrates that the hydraulic shear stress directly influences the soil's erodibility. The velocity of the flow, friction factor and density of the eroding fluid affect the magnitude of the hydraulic shear stress.

b) Characteristics of eroding and pore fluid

The percentage of salt and sodium concentration of the pore fluid, and the concentration of salt in eroding fluid are the main factors controlling the erosion characteristics of cohesive soils. Many previous studies were carried out to investigate how these parameters affected the critical shear stress (Sargunan 1977; Shaikh et al. 1988). An increase in the Sodium Absorption Ratio at a certain salt concentration in the pore fluid led to a decrease in the critical shear stress when the saturated soil was eroded with distilled water. Further, the critical shear stress increased with an increase in the concentration of salt in the pore fluid for a given sodium absorption ratio. Higher the concentration of salt in an eroding fluid, the lower the erosion rate, but the higher the critical shear stress for a given pore fluid properties.

c) Dry Density and Moisture Content

Water content plays a crucial role on the erodibility of compacted unsaturated soils. An increase in the critical shear stress of a soil compacted at a certain dry density was observed with an increase in water content.

Wan and Fell (2004) analysed and plotted the combined effects of compacted density and water content on the erosion characteristics of soil and is shown in Fig 2.2. Contours were drawn to represent the erosion rate index. According to their classification, the higher the erosion rate index the less the erodibility. They summarised that compacting a soil at high dry density (97% of the maximum dry density) and at the optimum or wet of optimum would significantly reduce erodibility.

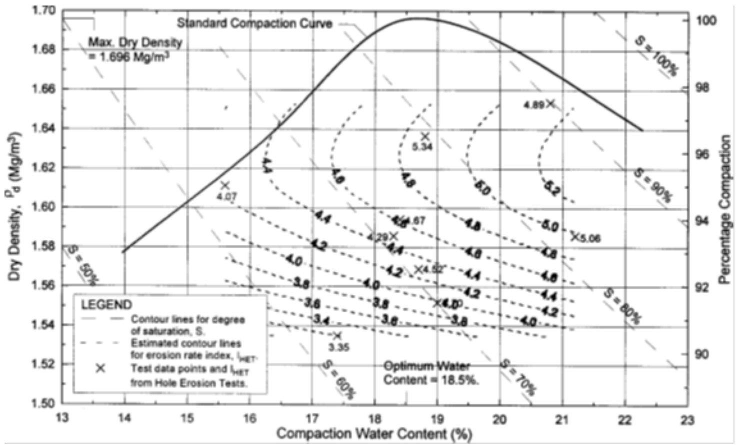


Figure 2.2.Variation of the erosion rate index with the compacted density and the moisture content (Wan and Fell 2004)

2.4.2. Measurement of Erodibility of Soils

Erosion, over a period of time, leads to geometric changes of the slope with subsequent major distress finally causing failure. Erosion can be categorized into surface erosion caused by running water on slope surface and internal erosion occurring within the structure. Internal erosion is the result of a coupled process of surface erosion from the interior of the soils and the subsequent fate of eroded particles in the pores (Reddi et al. 2000). Various qualitative and quantitative studies have been conducted to measure the erosion characteristics of soils. Sherard et al. (1976) developed the pinhole test to directly measure the dispersibility of compacted fine grained soils in which water is allowed to flow through a small hole of 1mm diameter in a soil specimen to stimulate water flow through a crack or other concentrated leakage path in the impervious core of a dam or other structure. Lewis and Schmidt (1977) conducted an investigation to determine the influence of dry density and initial water on the erosion of compacted dispersive clay using the pinhole test. Wan and Fell (2000) extended pin hole test (ASTM D4647/D4647M-13 2013) into a quantitative method for measuring internal erosion, called the hole erosion test (HET), by measuring changes in flow rate with time to back-calculate changes in the pipe diameter and thus the internal erosion (Wilson et al. 2013).

2.4.2.1. Hole Erosion Test (HET)

The hole erosion test (HET) developed by Wan and Fell (2002, 2004a, b) is one of several available methods for quantifying the erosion characteristics of soils. The eroding fluid is passed through a hole of 6mm diameter predrilled along the longitudinal axis of the soil sample, simulating the piping erosion occurring in embankment dams. Wan and Fell (2004a,b) conducted hole erosion test to study the internal erosion and piping of of both cohesive to non-cohesive soils at an unsaturated state. A hydraulic gradient was applied across a 6mm diameter hole in the soil, and then the flow rate through it was measured at regular time intervals. The hydraulic gradient was calculated using pressure heads measured using standpipes located at the inlet and outlet of the hole. Schematic diagram of hole erosion test assembly by Wan and Fell (2002) is shown in Fig 2.3.

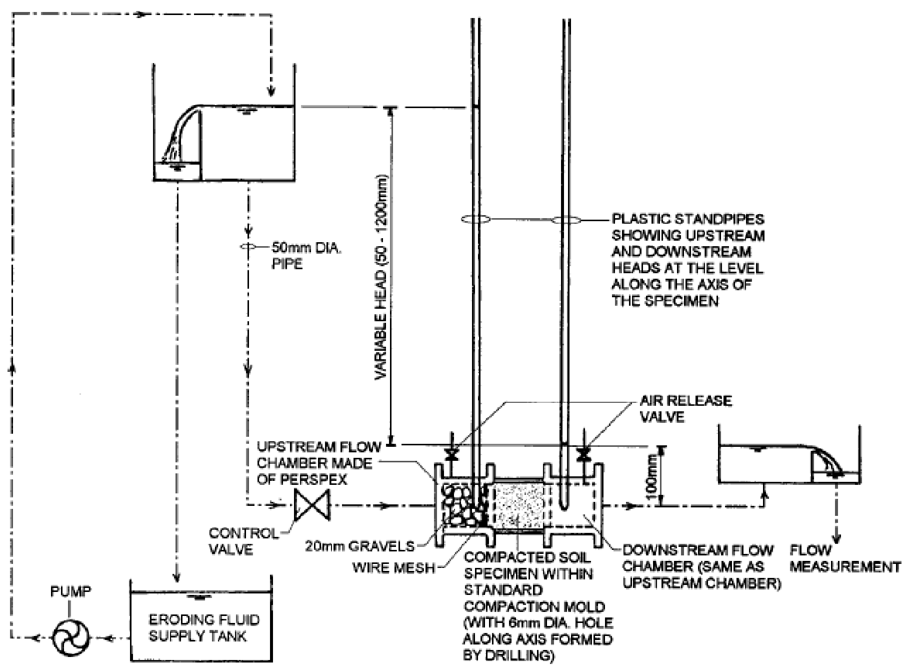


Figure 2.3. Schematic diagram of hole erosion test assembly (Wan and Fell, 2002)

2.4.2.2. Studies Performed Using Hole Erosion Test

Hole erosion test is one of the convenient and efficient erosion tests which are normally carried out to study the piping erosion in soils with fines. The erosion characteristics are defined by the erosion rate index, which measures the critical shear stress and erosion rate. Critical shear stress represents the minimum shear stress required to start progressive erosion. According to Wan and Fell (2004a), the rate of erosion is dependent on the content of soil fines and clay size, plasticity, and dispersivity; compaction, water content, density and degree of saturation; and clay mineralogy, and also the presence of cementing materials such as iron oxides. Lim (2006) conducted hole erosion tests to study the erosion behaviour of clay soils. Dispersive erosion which is indicated by murky or cloudy outflow is characterized by instantaneous erosion and quick enlargement of the hole, inferring substantial erosion at small shear stress. The erosion is mainly induced by the dispersive nature or by slaking in unsaturated cohesive soils. Slaking is mainly due to excess air pressure in the capillaries because of the surface tension in partially saturated soil. The air entrapped in the pores exerts pressure and breaks loose small bits of soil on the surface (Umesh et al. (2011)). Coarse-grained, non-cohesive soils, in general, erode more rapidly and have lower critical shear stresses than fine-grained soils. Koohpeyma et al. (2013) conducted erosion tests on lignosulfonate stabilised clayey sand, and a drastic reduction in the coefficient of soil erosion was noticed.

Xiao et al. (2006) conducted an experimental study by on compost soil as rainfall erosion control material on roadside embankments. A rainfall simulator and soil box were used in the experiments and results showed that soil loss was within tolerable limits. The rainfall erosion tests showed that the compost could be used on roadside embankments to protect base soil and reduce soil erosion under heavy rainfalls. Rainfall erosion tests were conducted on three types of compost, i.e. green compost, manure compost, and co-compost made of bio solid and green compost and it were found that both manure compost and co-compost retained slope stability and reduced soil erosion (Xiao et al. 2008,Xiao and Gomez 2009). Adams and Xiao (2011, 2013) performed hole-erosion tests to quantify the erosion of silty sand, the co-compost, and various ratios of sand-co-compost mixtures. The addition of non-erodible peat to highly erodible mineral soil resulted in a drastic reduction in piping erosion.

Taking HET as the reference test a new apparatus named as Process Simulation Apparatus for Internal Crack Erosion (PSAICE) was designed (Fig. 2.4) and built at University of Wollongong, Australia by Indraratna et al. (2008) for predicting the erosion characteristics of the soil. The effluent turbidity is also measured at a different time interval, and it was incorporated in calculating the erosion rate. The studies conducted using PSAICE is mentioned in Table 2.1.

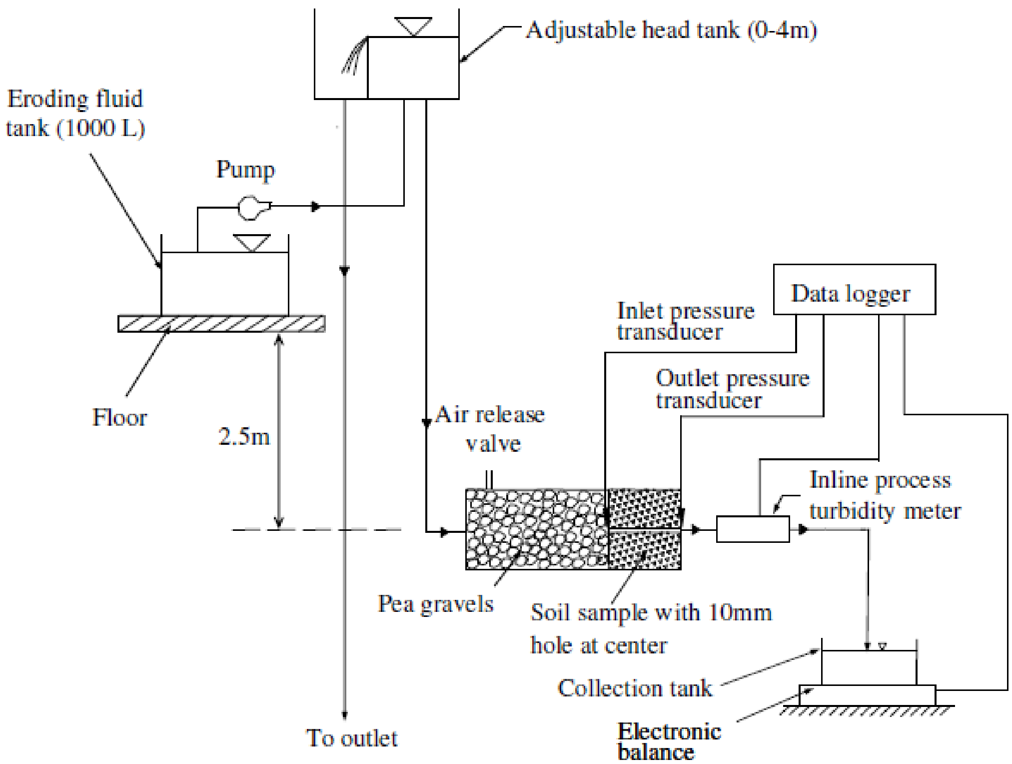


Figure 2.4. Schematic diagram of PSAICE

Table 2.1 Studies conducted using PSAICE

Researchers	Work Done
Muttuvel(2008)	Worked on erosion rate of chemically stabilised soils including tensile stress deformation characteristics of the soil
Indraratna et al.(2013)	A theoretical model to study the rate of erosion of silty sand based on the principle of conservation of energy is explained and is validated with a series of laboratory erosion tests using the PSAICE
Indraratna et al. (2008)	Two chemical stabilisers, lignosulfonate and Portland cement were tested on silty sand, and dispersive clay and effectiveness of stabilisation on increasing erosion resistance was studied using PSAICE
Vinod et al. (2010)	Internal erosional behaviour of lignosulfonate treated dispersive clay has been studied using PSAICE

An improvement of the classical HET by adding a turbidimeter and the interpretation method by analyzing the turbidity signal was presented by Haghighi et al. (2013). Several remoulded kaolinite-sand mixtures were tested as reference soil textures, and the results were analyzed with the proposed and existing interpretation methods. The use of the turbidity signal helped significantly in improving the real-time estimation of instantaneous dimensions of the hole and the eroded mass of soil.

A modified test, termed the HET-P, was carried out (Luthi 2011; Luthi et al. 2012) with 6mm,12mm,and 24mm diameter hole in which a pitot-static tube was incorporated to measure the total energy head and the velocity head of the jet as it emerges from the axial hole in the test specimen, thus providing a direct measurement of the total energy head loss across the specimen.

Benahmed and Bonelli (2012a, b) developed a new experimental device to carry out hole erosion tests in the laboratory under constant flow rate in order to quantify the critical shear stress (τ_c) and the coefficient of piping erosion (C_e) of soils. The main parameters considered in this study were the effects of moisture content, compaction energy and percentage of fines. The experimental results showed that these parameters play a key role in the erosion characteristics (τ_c and C_e). Al-Samarrai (2014) studied the internal erosion behaviour of sand and clay (stabilized by adding cement and fiber-cement mixture) using HET. Mehenni et al. (2016) conducted hole erosion tests on compacted silt treated using clay, lime and cement. The results indicated a reduction in the coefficient of soil erosion when treated with clay. An increase in critical shear stress was observed in cases where the silt was treated with lime and cement. The impact of the curing time on the erosion characteristics was not relevant for the lime-treated silt, whereas, in the case of cement-treated silt, it depended on the amount of cement added to the soil.

The erosion rate index obtained from HET was used to evaluate the effects of fluid properties such as viscosity, pH and ionic strength on the piping of sandy soil (Y. Ma et al. 2020). It was observed that a higher pH caused higher erosive capacity, whereas higher ionic strength caused lower erosive capacity of the fluid.

2.4.2.3 Models for Piping Erosion Applicable to HET

Bonelli et al. (2006a, b) proposed a simplified one-dimensional model for the piping erosion, applicable to the HET test. According to the model, the change in the hole radius is an exponential function of the time of internal erosion process and the initial and critical shear stress. It was established that the product of the coefficient of erosion and the flow velocity is a significant dimensionless number and when this number is small, the kinetics of erosion is low, and the particle concentration does not have any effect on the flow. Theoretical and experimental evidence presented showed that the evolution of the pipe radius during erosion with a constant pressure drop obeyed a scaling exponential law.(Bonelli and Brivois 2008)

Wahl et al. (2009) noticed two significant advantages that the model offers, one is concerned with the diameter, and the other is the minimization of the impact of short-term anomalies in erosion behaviour during a test using the curve-fitting procedure.

Bezzazi et al. (2010) have derived simplified analytical modelling of the Hole Erosion Test by assuming formal analogy between the erosive shear stress and the friction shear that develops at a cylindrical piping wall under an axial viscous flow. A uniform flow was assumed along the tube. Comparisons of the theoretical predictions were with experimental results were analysed and the simplified model was found to predict accurately the increase of flow rate that results from piping erosion. Benaissa et al. (2012) analysed the effects on the wall shear stress resulting from varying water clay content and applied hydraulic gradient. This two-dimensional modelling allowed in understanding the irregular eroded hole wall shape as observed experimentally after performing the standard hole erosion test.

Boukhemacha et al. (2013) used the procedure by Wan and Fell (2004) to estimate critical shear stress and coefficient of soil erosion and two new procedures were also proposed. It is found that the erodibility properties depend on the hydraulic charge and that for one soil sample, it is possible to find more than one pair of solutions (critical shear stress, coefficient of soil erosion) which explains the non-observation of the significant relationship between these two properties and other soil properties.

The Hole Erosion Test which is suitable for soils containing plastic and non-plastic fines (clayey and silty soils) which will sustain an opening when wetted during the test is thus a good 'model' for most circumstances in which concentrated leaks may occur. The Hole Erosion Test simulates the erosion phenomenon in defects such as cracks and micro-fissures induced by settlement or hydraulic fracture, open contact between two different types of soil, roots and burrows, etc. It helps in the determination of the critical shear stress beyond which the erosion is initiated and the erosion coefficient, which represents the kinetics of the erosion **(**ICOLD 2016). The results of Hole Erosion Tests were used to estimate the rate at which the fractures through the plugs would enlarge and release water through the burrow causing erosion of the walls and eventual breach of the levee (ICOLD 2016).

Savage et al. (2019) conducted hole erosion tests and swell tests on five soils used in the core of embankment dams to study the concentrated leak erosion in transverse cracks in dams. This data was used for modelling the rate of erosion and variation of crack width with time.

Various studies done with Hole erosion test when compared with other erosion tests are shown in Table 2.2.

Table 2.2. Studies done Using HET

Researchers	Work Done
Wahl(2010)	HET and Jet erosion tests are compared, and it was found that HET is desirable for application specifically to piping erosion situations
Lim(2006)	HET and rotating cylinder test was conducted on variably saturated clay soil
Wahl et al.(2009)	Two methods of soil erodibility testing, the hole erosion test (HET) and submerged jet erosion test (JET), were evaluated for potential application to the modeling of embankment dam erosion and breach processes
Chevalier et al. (2010)	Comparative tests with Hole Erosion Test and Mobile Jets Erosion Test apparatus were conducted

2.5. SOIL STABILISATION TECHNIQUES

2.5.1. Vegetation

One of the most cost effective erosion control and slope protection methods is to consider the soil-bioengineering or ecological slope engineering (Gray and Sotir 1996).The relative effectiveness of vegetation in any specific locale mainly depends on the quality of vegetation, topography, slope, hydrology, geology, and soils.

Vegetation affects both surficial and mass stability of slopes in significant and important ways. The stabilizing and protective benefits of vegetation depend both on the vegetation type and type of process of slope degradation. In the case of mass stability, the protective benefits of woody vegetation range from mechanical reinforcement and restraint by the roots and stems to modification of slope hydrology as a result of soil moisture extraction via evapo-transpiration. (Gray and Sotir 1996). The loss or removal of vegetation on slopes can result in either increased rates of erosion or higher frequencies of slope failure.

Vegetation is used for stabilizing cut slopes in soil, soil embankments, soil heaps and terraced slopes. It is less likely to be of value in dams where engineering stability is critical, and vegetation could also affect the permeability of soil. Ground bioengineering methods are commonly used on artificial and terraced slopes, as this fast and effective solution can be considered during slope construction and remediation (Norris 2008). The key effects of herbaceous, and to a lesser extent, woody vegetation in minimizing surface erosion of soils includes:

a) Interception- foliage and plant residues absorb rainfall energy and prevent soil compaction.

b) Restraint - root systems physically restrains the soil particles while above-ground residues filter sediment out of the run-off.

c) Retardation - above-ground residues increases surface roughness and slows the velocity of run-off.

d) Infiltration - roots and plant residues help in maintaining the soil porosity and permeability.

e) Transpiration - reduction of soil moisture by plants delays the onset of saturation and run-off.

According to Greenway (1987), roots reinforce the soil and increase the shear strength of soil, roots bind soil particles at the ground surface, minimizing their erosion susceptibility, and roots extract moisture from the soil leading to lower pore-water pressures.

Vegetation can affect the stability of a slope mainly by the increase in soil shear strength due to root reinforcement, although other effects such as surcharge of trees: wind loading on large trees: soil buttressing and arching effects caused by large tree roots: and modification of soil moisture content through the processes of rainfall interception and evapo-transpiration may also provide minor effects on slope stability (Gray and Leiser 1982; Coppin et al. 2007).

	Process	*Type*	*Effect on stability*
1.	Roots increase permeability, increase infiltration, and thereby increase pore pressure	Hydrologic	Negative
2.	Vegetation increases interception and evapotranspiration, and thereby reduces pore pressure	Hydrologic	Positive
3.	Vegetation increases weight or surcharge and thereby increases load on slope	Mechanical	Negative
4.	Vegetation increases wind resistance, and thereby increases load on slope	Mechanical	Negative
5.	Roots reinforce soil and thereby increase strength	Mechanical	Positive

Figure 2.5. Effects of vegetation on slope stability (Morgan 1995)

Much research has been conducted, in the past 30 years or so to quantify the effects of root reinforcement on slope stability. The majority of is research has focused on field or laboratory studies of root-filled soils and modification of limit equilibrium and finite element methods to incorporate the effect of root reinforcement. Quinton et al. (1997) conducted a series of rainfall simulation experiments on small plots to study the impact of different vegetation species and plant properties on the surface runoff and soil erosion. Six vegetation types were studied, at different stages of maturity, giving a total of nine vegetation treatments and two bare soil treatments and concluded that future efforts should be directed at developing ecological successions and re-vegetation methods which promote a substantial and sustainable canopy cover. Dupuy et al. (2005) investigated the influence of root morphology and soil type on the mechanical behaviour of tree anchorage through numerical modelling. The overturning resistance of the four schematic root patterns was determined in four different idealistic soil types which were based on Mohr–Coulomb plasticity models. Chok (2008) used FEM method for modeling the effect of soil variability and the effect of vegetation on natural slopes. Osman et al. (2011) conducted a field shear box test on three plant species assessed in term of their soil-root shear strength properties. Mu'azu et al. (2012) combined the mechanical and hydrological effects of vegetation with slope stability by exploring and incorporating the matric suction generated by tree root water-uptake driven by transpiration. Three positions of the tree on a slope have been analyzed for the factor of safety and concluded that the tree contributes stability to sloping ground both hydrologically and mechanically and best would probably be achieved only when the tree is located at the toe of the slope. Cazzuffi et al. (2014) conducted several tensile strength tests on roots and direct shear tests and pull out tests on root soil system. He also developed a theoretical model to determine the increase in shear strength of the soil, due to the presence of roots, as an increase in soil cohesion.

2.5.1.1. Vetiver Grass

The utilization of vetiver grass (Truong (1990), Dalton et al. (1995)) in bio-engineering for protection of slopes and control of soil erosion was accepted widely from 1990's following the promotion of the Vetiver System by the World Bank (Hengchaovanich (2003), D'Souza et al. (2019)). Nilaweera and Hengchaovanich (1996) conducted tensile strength tests on vetiver grassroots to study its effect on slope stabilization and erosion. The results indicated that when compared to hardwood, the smaller average root diameter of vetiver shows very high mean tensile strength (75 MPa), inferring that vetiver grass is more effective in root reinforcement in soil slopes. Due to its long (2-3.5 m) and massive root networks which are also very fast-growing (within 4-6 months), it is better than many types of trees, and also due to its deep thick root system which spreads vertically rather than horizontally, vetiver grass can endure harsh climatic conditions.

(Hengchaovanich, 2003). Mickovski and van Beek (2009) investigated root system morphology of vetiver (*Vetiveria zizanioides*) in a small plantation growing on abandoned marl terraces in southern Spain. In situ shear test on blocks of soil permeated with vetiver roots were also carried out and showed a greater shear strength resistance than the samples of non-vegetated soil. From the root reinforcement model analysed it was also concluded that the stability of a modelled terraced slope planted with vetiver was marginally greater than the one of a non-vegetated slope. Maneecharoen et al. (2013) conducted model tests using water hyacinth limited life geotextiles (LLGs) as well as using Vetiver and Ruzi grasses for erosion control. Lateritic soil and sandy soil were investigated separately with LLGs and vegetation covers. Jotisankasa et al. (2014) conducted numerical modelling on slopes to study the influence of vetiver grass (*Chrysopogon zizanioides*) on infiltration behaviour and slope stability. It was found that for natural soil slope with gradient of 26 degree, vetiver grass roots (2m deep) appeared to enhance the stability of the soil by reinforcement of the roots. In the coastal areas of Karnataka and Kerala in peninsular India, vetiver grass (*Vetiveria zizanioides*) is used very effectively for erosion control (Anaswara and Shivashankar, 2015).

2.5.1.2 Neem Tree

Neem (*Azadirachta indica*) commonly called ‘Indian Lilac’ or ‘Margosa’, belongs to the family Meliaceae, subfamily Meloideae and tribe Melieae. Neem is the most versatile, multifarious trees of tropics, with immense potential. The Neem tree is duly valued worldwide for its hardiness, medicinal properties, and nutritional value. It is native to Southeast Asian countries but grows well in a variety of tropical environment. Neem is fast-growing trees that can reach a height of 15-20m, rarely to 35-40m.The branches are widespread and fairly dense crown is roundish or oval and may reach the diameter of 4.5 -6 m in old, free-standing specimens. The trunk is relatively short, straight. The root system consists of strong taproot and well-developed lateral roots.

The means of trees standing erect against wind loads and gravity loads involve a complex set of soil structural interactions. Successful tree anchorage depends upon both the size of wind and gravity loads placed on a tree and the tree / site structural resistance to these loads. Resistance to wind and gravity loading is distributed throughout a tree and associated soil. There are a few large diameter roots and a host of small roots, all positioned close to the soil surface. The shape, size, length, taper, and depth of tree roots are optimized to provide both anchorage and soil resource gathering and control (Kalliokoski et al. 2008). The presence of roots highly influences the stability of slopes. The strength properties of the roots and the vastness of the root network gives an understanding of the degree of mechanical stabilization. Research has expanded significantly regarding this topic in the last thirty years due to the attention gathered from deforestation in mountainous areas leading to landslides and slope failures (Nilaweera & Nutalaya, 1999). Mechanical stabilization of slopes by roots is mainly provided through its tensile strength, frictional properties and bending stiffness. Both fine and coarse roots are preferred for an ideal slope, as both offer different advantages. Fine roots are effective at stabilizing the upper soil layers and have higher tensile strengths, while coarse roots extend into greater depths of the soil and help in anchoring large volumes of soil. The growth of roots, among other factors, can create continuous macropores known as soil pipes. Soil pipes improve drainage, which helps to dissipate pore water pressure and is especially important in slopes experiencing large volumes of rainfall. However, when soil pipes become eroded, and the cavities become blocked, water can build up and cause slope failure.

Vegetation thus improves the slope stability both in terms of its mechanical and hydrological processes. Due to the mechanical reinforcement from the root system of trees, vegetation influences the balance of stresses in a slope. It also impacts the slope stability by altering the hydrologic regime of the soil. Since plants and grass absorb different amounts of water depending on the type of soil in which they grow, there are several criteria for selecting the most suitable species. A general rule is to use local plants and grass, which can be adapted to the local climate (Coder Kim 2010).

Much of coastal Karnataka and Kerala areas have sloping terrains, and the soils (lithomargic clay/lateritic lithomarges) that are abundantly available here are of a dispersive and highly erosive nature. A lot of developmental activities are taking place in this coastal area. So many embankment slopes, as well as slopes in excavation, are created, whose stability is a concern to the engineers. As a general rule, a single species of vegetation should not be planted in isolation. On typical slopes, vegetation cover should not consist only of grass but should include trees and shrubs. Consideration should also be given to appropriate vegetation management techniques to assist the natural succession process. In this study, vetiver grass and Neem tree are considered for numerical analyses.

2.6. SLOPE STABILITY ANALYSIS

Finite element method being a very powerful computational tool in engineering gains its power from the ability to simulate physical behaviours using computational tools without the need to simplify the problem. Therefore, complex engineering problems need finite element methods to obtain more reliable and accurate results.

The finite element method is used to study the slope stability using a failure definition similar to that in the limit equilibrium method. Limit equilibrium methods first define a proposed slip surface then the slip surface is examined to obtain the factor of safety, which is defined as the ratio between the available and the mobilized shear strength along the surface. Numerous methods for the analysis of slope stability using finite elements have been proposed during the last two decades. Among those methods, the most widely used methods are the gravity increase method and strength reduction method. Gravity forces are gradually increased in the gravity increase method until the slope fails. The factor of safety is then defined as the ratio between gravitational acceleration at failure (gf) and actual gravitational acceleration (g). In the strength reduction method, parameters of the soil strength are decreased until the slope becomes unstable. The factor of safety is then defined as the ratio between the initial strength parameter and critical strength parameter. Therefore, the strength reduction method has exactly the same definition as the limit equilibrium methods. The gravity increase method is used to study the stability of embankments during construction as it gives more accurate results while the strength reduction method is used to study the stability of existing slopes (Matthews et al. (2014)).

Slope stability analysis is done in PLAXIS 2D using finite element method. To analyse slopes, the strength reduction method is applied. This method is based on the reduction of the cohesion (c) and the tangent of the friction angle (tanφ) of the soil. The parameters are reduced in steps until the soil mass fails. The total multiplier ΣMsf is defined as the ratio of the strength parameters entered as input values over the reduced ones. ΣMsf is set to 1 at the start of a calculation to set all material strengths to their unreduced values. The strength parameters of soil are thereby reduced automatically step by step with an increment equal to 0.1 until failure.

Factors of safety obtained from stability analysis methods that satisfy all limit equilibrium conditions are within 6% of each other (Duncan 1996). A comparative study of the finite element method and limit equilibrium method in slope stability analysis was done by Alkasawneh et al. (2008). Berilgen (2007) used the FEM method in studying the stability of slopes under drawdown conditions. Finite element analysis was used by Gasmo et al. (2000) to find out the infiltration effects on slope stability of residual soil slope. Hamdhan and Schweiger (2011) evaluated the effects of hydraulic characteristics of soils(hydraulic conductivity and initial degree of saturation) and rainfall in slope stability calculations performed with the finite element method, simultaneously computing deformation and groundwater flow with time-dependent boundary conditions (fully coupled flow-deformation analysis).The factor of safety was calculated by means of the shear strength reduction technique. In the study conducted by Chanmee et al. (2016), the factor of safety of erosion protection on slopes was simulated by PLAXIS FEM 2D and Limit Equilibrium Slide Software. It could be observed that the result from PLAXIS FEM 2D had higher accuracy when compared to the result from Slide Limit Equilibrium Software. Taccari and van der Meij (2016) conducted a case study on the influence of animal burrowing on the failure of the levee of San Matteo along the Secchia River. Plaxis 2D was used in this study for assessing the influence of animal burrowing. Saliba et al. (2019) used Plaxis 2D to model one of the case study of sustained earth dams in Laqlouq area in Lebanon to show the piping evolution using an iterative approach within the body of the structure. Merat et al.(2019) conducted slope stability analysis in PLAXIS 2D to study the effect of rainfall on slope stability and concluded that slope with highest cohesion and angle of internal friction showed the highest stability whereas slope with combination of factors such as high slope angle and slope height failed easily.

With the evolution of computers and their application, more advanced ways should be considered for analysing slope stability in the geotechnical analysis. There are significant opportunities in using the more comprehensive finite element analysis. The criteria for the selection of method by the user should be the complexity of the problem, which is to be modelled. For example problems with complex geometries or that require analysis of seepage, consolidation and other coupled hydrological and mechanical behaviour (pore water pressure induced shrink-swell cycles for example) along with those problems with more complex mechanical soil responses may be better tackled using FE analysis. Conversely, simpler problem geometries or where complex material responses are not expected, or those problems where data is limited, or it is necessary to make an initial stability estimate before undertaking more complex analysis may better be undertaken in limiting equilibrium software.

In the current study, slope stability analysis is done in PLAXIS 2D using finite element method. To analyse slopes, the strength reduction method is applied. This method is based on the reduction of the cohesion (c) and the tangent of the friction angle (tanφ) of the soil. The parameters are reduced in steps until the soil mass fails. The total multiplier ΣMsf is defined as the ratio of the strength parameters entered as input values over the reduced ones. ΣMsf is set to 1 at the start of a calculation to set all material strengths to their unreduced values. The strength parameters of soil are thereby reduced automatically step by step with an increment equal to 0.1 until failure. The most important feature of the current version of PLAXIS 2D is its ability to take into account various boundary conditions for flow such as seepage, head, prescribed boundary flux and infiltration/precipitation. In the current analysis, some of these features have been exploited to arrive at better stimulations associated with flow that could affect the stability of excavated lateritic slopes.

CHAPTER 3

EROSION STUDIES ON LITHOMARGIC CLAYS

3.1. GENERAL

The resistance of cohesive soils to internal erosion can be quantified by conducting Hole Erosion Test (HET), the one developed by Wan and Fell (2004 a, b). Hole erosion tests have been conducted to study the erosion characteristics of the controlled samples of lithomargic clay and its erosion potential. This chapter discusses the influence of moulding water content, degree of compaction, head causing flow, percentage silt content and plasticity index on the erosion rate index and critical shear stress of controlled lithomargic clay samples.

3.2. MATERIALS USED

3.2.1. Lithomargic Clay (Shedi soil)

In the present study, the soil samples were collected from a site near Padapanambur, Dakshina Kannada District of Karnataka state (Fig.3.1a). The soil profile from Panjimooger site is shown in Fig. 3.1 (b). Two lithomargic clay samples were procured from two nearby sites from depths of 2-3m below ground level, below the laterite layers. These lithomargic clay samples had particle sizes finer than 150μ sieve size. The first procured sample (designated as C0 sample) had higher percentage of clay fraction (smaller than 2μ size) {55.3%} and second procured sample (designated as M0 sample) had higher percentage of silt fraction (2μ to 75μ size) {79.9%}. The soil was kept for air drying for 24 hrs. After air drying, samples were kept in oven for 24 hrs. This oven dried soil was used for various tests.

3.2.2. River sand

For all the tests, the soil was mixed with river sand to vary gradation of the soil. River sand was collected from Kulur river site.

3.3. LABORATORY TESTS

3.3.1. Samples Tested

Both C0 and M0 samples were blended in the laboratory with different percentages (10%, 20%, 30% and 40%) of river sand (passing 1.18mm sieve) to prepare controlled samples. These samples are designated as C10, C20, C30, C40 (for C0 blended samples) and as M10, M20, M30, M40 (for M0 blended samples) respectively. C samples are soils with higher percentage of clay fraction (or high compressibility soils with liquid limit in excess of 50%) and M samples are soils with higher percentage of silt fraction (or liquid limit less than 50%). Controlled soil samples thus prepared were then studied for both geotechnical and erosion properties. A series of hole erosion tests are carried out on all the ten samples at various water contents (namely at 50% of OMC, at OMC and at full saturation), hole diameters (6mm and 8mm) and compaction (100%, 90% and 80%) with the suitable range of head (within the laboratory constraints). The basic geotechnical properties of all the C and M soil samples are listed in Table 3.1 and Table 3.2 respectively.

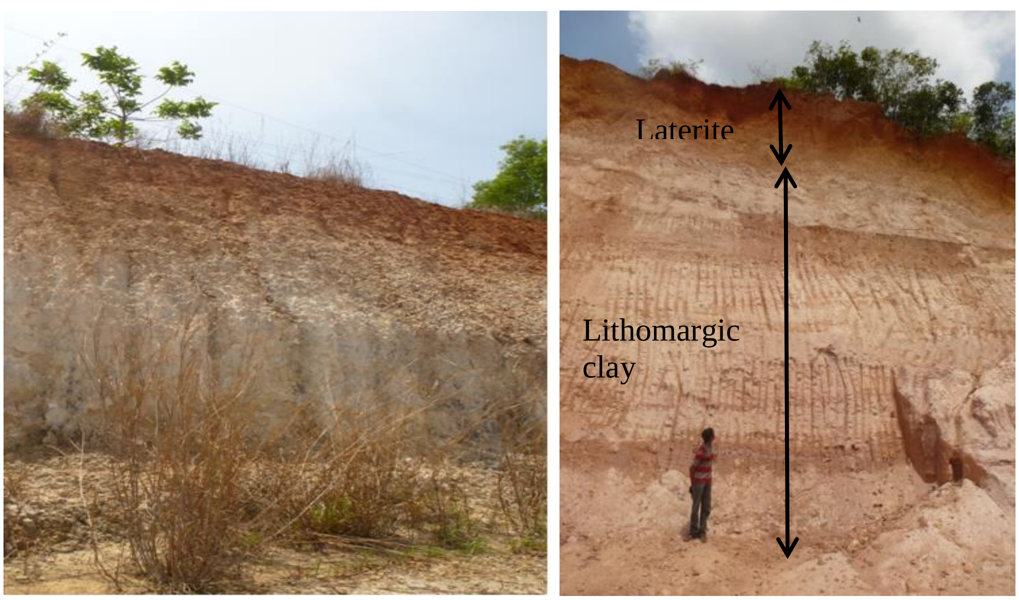


Figure 3.1. a) Padupanambur site from where the shedi soil was brought for investigation b) Panjimooger site with soil profile

Table 3.1. Geotechnical properties of C samples

Parameter		C0	C10	C20	C30	C40
Specific gravity (G)		2.56	2.57	2.61	2.63	2.65
Maximum Dry Density (γ_{dmax}) (kN/m^3)		14.22	14.32	14.81	15.70	15.99
Optimum Moisture Content (OMC) (%)		27.0	26.4	24.3	23.0	19.6
Void ratio(at γ_{dmax})		0.77	0.76	0.73	0.64	0.63
Plastic Limit (%)		30.0	29.0	26.0	25.0	24.0
Liquid Limit (%)		53.0	50.0	45.0	41.0	39.0
Plasticity Index (PI)		23.0	21.0	19.0	16.0	15.0
Fine fraction (Dusty fraction)	Clay fraction(%) (< 2µ)	55.3	49.8	44.2	38.7	33.2
	Silt fraction (%) (2µ to 75µ)	42.7	38.4	34.2	30.7	26.3
Coarse fraction	Sand fraction (%) (75µ to 4.75mm)	2.0	11.8	21.6	30.6	40.5
	Gravel fraction (%) (> 4.75mm)	0.0	0.0	0.0	0.0	0.0
Unified Soil Classification		MH	MI-MH (Boundary Classification)	CI	CI	CI

Table 3.2. Geotechnical properties of M samples

Parameter		M0	M10	M20	M30	M40
Specific gravity (G)		2.49	2.50	2.52	2.54	2.58
Maximum Dry Density (γ_{dmax}) (kN/m^3)		13.24	14.03	15.21	15.40	16.58
Optimum Moisture Content (OMC) (%)		27.2	24.6	20.2	18.8	16.4
Void ratio(at γ_{dmax})		0.81	0.76	0.63	0.61	0.53
Plastic Limit (%)		36.0	31.0	25.0	23.0	22.0
Liquid Limit (%)		48.0	46.0	41.0	36.0	31.0
Plasticity Index (PI)		12.0	15.0	16.0	13.0	9.0
Fine fraction (Dusty fraction)	Clay fraction (%) (< 2μ)	16.6	13.3	13.2	12.3	7.4
	Silt fraction (%) (2μ to 75μ)	79.9	74.5	64.1	55.1	51.0
Coarse fraction	Sand fraction (%) (75μ to4.75mm)	3.5	12.2	22.7	32.6	41.6
	Gravel fraction (%) (> 4.75mm)	0.0	0.0	0.0	0.0	0.0
Unified Soil Classification		MI	MI	CI	CI	CL

3.4. HOLE EROSION TEST

3.4.1. Experimental Setup

The schematic diagram of the experimental setup used for the hole erosion tests is shown in Fig.3.2. It consists of a constant head tank, an inlet chamber, mould along with the specimen, an overflow container, a collecting tank and a weighing balance. The constant head upstream tank is provided with a continuous water supply. The inlet chamber is connected to the constant head tank. It is filled with 20mm coarse aggregates to reduce the impact of water on the soil specimen. An air valve is provided in the inlet chamber to remove air bubbles (if any) during the experiment. The plate in between the inlet chamber and the specimen is provided with a hole of diameter 5cm to ensure uninterrupted flow of water through the specimen. A wire mesh is fixed on the plate to avoid the coarse aggregates from disturbing the specimen. The hole erosion test apparatus is shown in Fig.3.3. The weight of the overflowing water is continuously measured. The discharge at various time intervals was calculated from the weight and the head was obtained by measuring the vertical distance between the water level in the overhead tank and the free water surface at the downstream end. The test is terminated (by closing the downstream valve) upon observing one of the following conditions i) several minutes of accelerating flow, no significant erosion in one hour at maximum test head, ii) extreme erosion with hole enlargement, reaching the walls of the mould (Luthi 2011).

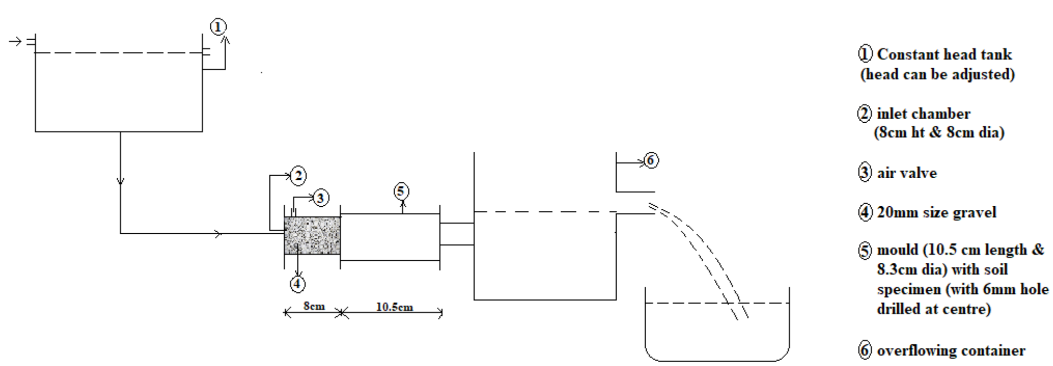


Figure 3.2. Schematic diagram of the hole erosion test setup

Figure 3.3. Hole Erosion Test apparatus (without constant head tank) a) Front view b) Top view

3.4.2 Specimen preparation

The test specimens were prepared at the required dry density and water content. The soils were compacted to a thickness of 105mm in a mould of 83mm diameter corresponding to the desired dry density. The samples were then kept in a desiccator for 1 day. This was to attain uniform moisture content throughout the sample. The preparation of sample is shown in Fig.3.4.

Figure 3.4. Preparation of sample using a hydraulic extruder

3.4.3 Test procedure

A hole of 6mm diameter was drilled along the central longitudinal axis of the specimen. The two circular surfaces of the specimen were coated with paraffin excluding an inner circular area of 3cm around the hole. This is done to avoid the dispersion of the soil surface. The inlet chamber is filled with 20mm gravels in order to regulate the flow of water on the upstream side of the sample and also to reduce the impact of water on the specimen surface. The soil sample is placed between the inlet chamber and overflow container. It is ensured that a constant head is maintained throughout the test. When the inlet valves are opened, the air bubbles are eliminated by opening the air valve provided at the inlet chamber. The flow rate at the downstream side of the apparatus was measured at different time intervals during the test. The specimen of eroded soil was then retrieved out of the device and melted paraffin is poured into the eroded hole. After the paraffin solidifies, the specimen is cut out and the wax is prudently extracted. This represents the shape of the final eroded hole. The final average hole diameter is noted after the test.

3.4.4. Equations Governing Hole Erosion Test

Wan and Fell (2002, 2004 a,b) developed the hole erosion test to measure the erosion properties of soils. The rate of erosion per unit surface area of the hole is expressed in Equation 3.1.

$$\varepsilon_t = C_e(\tau_t - \tau_c) \qquad (3.1)$$

where ε_t is the rate of erosion per unit surface area of the hole at time t [kg/s/m^2]; C_e is a constant known as the Coefficient of Soil Erosion [s/m]; τ_t is the hydraulic shear stress along the hole at time t [N/m^2]; τ_c is the critical shear stress i.e. the minimum shear stress required to initiate progressive erosion [N/m^2].

The hole diameter at any time during erosion is calculated by Equations 3.2 and 3.3 for laminar and turbulent flow respectively.

$$\emptyset_t = \left(\frac{16Qf_L}{\pi\rho_w gs}\right)^{\frac{1}{3}} \text{ , for laminar flow conditions} \qquad (3.2)$$

$$\emptyset_t = \left(\frac{16Q^2 f_T}{\pi^2\rho_w gs}\right)^{\frac{1}{5}} \text{ , for turbulent flow conditions} \qquad (3.3)$$

where, ϕ_t (m) is the hole diameter at time t; Q (m^3/s) is the flow rate through the hole at time t; f_L and f_T are the friction factors for laminar and turbulent flow conditions; g (m/s^2) is the acceleration due to gravity; s is the hydraulic gradient across the hole at time t; and ρ_w (kg/m^3) is the density of the eroding fluid. The erosion rate and hydraulic shear stress were then calculated using the Equations 3.4 and 3.5, respectively.

$$\varepsilon_t = \frac{\rho_d}{2}\left(\frac{d\phi_t}{dt}\right) \tag{3.4}$$

$$\tau_t = \frac{\rho_w g s \phi_t}{4} \tag{3.5}$$

where, ε_t(kg/s/m^2) is the erosion rate; ρ_d (kg/m^3) is the dry density of the soil. According to Wan and Fell (2002, 2004 a,b), the erosion rate changed linearly with the hydraulic shear stress. The coefficient of soil erosion C_e is the slope of the straight line obtained from plotting ε against τ_t. The critical shear stress, τ_c can be obtained graphically by extrapolating the plot of ε_t versus τ_t to zero.

The rate of erosion of a soil can be represented by an 'erosion rate index' "I_h" which can be derived from the results of the hole erosion test from Equation 3.6.

$$I_h = -\log(C_e) \tag{3.6}$$

The rate of progression of erosion is classified as per Table 3.3. Soils that erode rapidly have lower I_h values than soils that erode slowly.

Table 3.3 Qualitative relation of representative erosion rate index and progression of internal erosion as recommended by Wan and Fell (2002, 2004 a,b)

Group number	**Erosion Rate Index (I_h = -log(C_e))**	**Progression of internal erosion**
1	Less than 2	Extremely rapid
2	2-3	Very rapid
3	3-4	Moderately rapid
4	4-5	Moderately slow
5	5-6	Very slow
6	Greater than 6	Extremely slow

CHAPTER 4

SLOPE STABILITY ANALYSES ON EXCAVATED SLOPES WITH LATERITIC FORMATIONS

4.1. INTRODUCTION

Slope stability analysis is a complex problem. It does not involve just geotechnical factors, but also very much influenced by hydrological factors (precipitation, ponding, seepage), biological factors (vegetation and turfing) and erosion (surface and internal erosion). However, it is difficult (in present status) to study the influence of all the factors together. An attempt is made to study the influence of these various factors separately.

Experimental analysis was conducted on shedi soil to find the undrained and drained shear strength parameter by using large scale triaxial test. Shedi soil was taken from the site, Padupanambur in Dakshina Kannada district from depths below 3m from the ground level below the laterite layer to measure the in-situ dry density and water content. Numerical analysis was then done in the FEM software, Plaxis 2D targeted at solving the slope stability problem where vegetation and various boundary conditions for flow due to the effect of precipitation, ponding and seepage were considered.

4.2. BASIC GEOTECHNICAL PROPERTIES OF THE SOIL SAMPLE

The basic geotechnical properties of the soil sample are shown in Table 4.1.

Table 4.1. Basic Geotechnical properties of the soil sample

Parameter	Value
Unit weight (kN/m^3)	18
Water content (%)	28.67
Dry unit weight of in situ soil (kN/m^3)	14

4.2.1. Large scale Triaxial test

The large scale triaxial test (samples tested were large size of diameter 10cm and height 20cm) was conducted to calculate the shear strength parameters of shedi soil in both drained and undrained conditions at field density using standard equipment and standard procedure mentioned in the IS code (IS:2720-11). Shear strength of shedi soil at different depths are shown in Table 4.2.

Table 4.2 Shear strength properties of shedi soil

	Drained	Undrained
C at 3.5 m depth	20	35
C at 3.5 m depth	23	0
ϕ at 3.5 m depth	-	50
C at 6.5 m depth	-	0
ϕ at 6.5 m depth	-	57
C at 7 m depth	-	0
ϕ at 7 m depth		

4.3. SLOPE STABILITY ANALYSIS USING PLAXIS 2D

Slope stability analysis is conducted considering the effect of wind action on trees, turfing on excavated slopes, precipitation, ponding at top and seepage through the excavated slope by varying the slope heights and slope angles.

4.3.1. Undrained Case

4.3.1.1. Defining the Problem

The analysis is carried out to simulate a situation that occurs during the months of heavy rainfall in which the ground water table rises to ground level in a vegetated and excavated slope. The slope considered consists of laterite for the top 3m underlain by lithomargic clay and hard rock (granitic gneiss) below it. Water gets accumulated on the top hard laterite near to the crest (ponding) for a considerable width and for varying depths (i.e. 1m, 2m and 3m respectively in different cases) for a period of 3 days was considered. Tree is provided at the toe, mid and crest (top) position of the slope. A berm of 3m width was provided at 4m and 5.5m height of the slope. Keeping the geotechnical properties constant, the cut slope angles for the excavated slope were varied as 30, 40, 45 and 60 degrees with the ponding situation is separately analysed for slopes of height 11m and 8m. A berm of 3m width was provided at 4m and 5.5m height for 8m high slope and at 5.5m height in the case of 11m height slopes. Effect of turf vegetation is also included as a layer of soil of 3m thickness, with 30% increased cohesion than the original soil layer (Shivashankar et al. 2014).

4.3.1.2. Undrained (B)

For undrained soil layers with a known undrained shear strength profile, PLAXIS offers the possibility of undrained effective stress analysis. In this, the input parameter is directly given, i.e. setting the friction angle to zero and cohesion equal to undrained shear strength. Also, in this case, a distinction made between the pore pressure and effective stress path may not be fully correct, the resulting the undrained shear strength is not affected, since it is directly specified as an input parameter. Most of the soil shows increasing shear strength with depth, and it is possible to specify increase per unit depth in PLAXIS in the advance subtree in the parameter tab sheet of the soil window.

4.3.1.3. Effectiveness of Plaxis 2D in solving the problems associated with flow

The current Plaxis version has been successful in simulating various boundary conditions associated with flow. As the physical process is time-dependent, it leads to mixed equations of displacements and pore pressure, called coupled hydro-mechanical approach in which these equations are solved simultaneously. This kind of calculation is termed as fully coupled flow-deformation analysis. When conducting this type of calculation, the full interaction between deformations, consolidation and groundwater flow simultaneously in the same phase is solved. In this calculation, it uses combined staged construction and transient groundwater flow.

4.3.1.4. Simulating Precipitation in Plaxis 2D

The precipitation option can be used to specify a general vertical recharge or infiltration (q) due to weather conditions. This condition is applied at all boundaries that represent the ground surface. This option can be selected in the model conditions subtree in the model explorer (Fig.4.1).

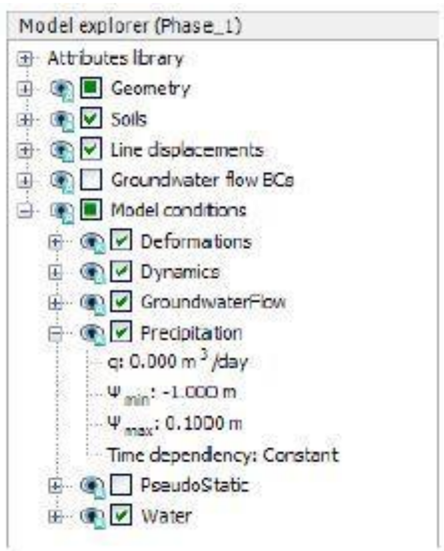


Figure 4.1. The expanded precipitation subtree in the model explorer

4.3.2. Drained Case

4.3.2.1. Defining the Problem

The analysis is conducted to simulate a situation that occurs during the months other than heavy rainfall in which the ground water table is below the lithomargic clay layer. The slope consists of laterite for the top 3m, thereafter shedi soil and hard rock (probably gneiss) below it. The top soil (i.e. laterite) is excavated (i.e. 1m, 2m and 3m respectively in different cases) over a period. Keeping the geotechnical properties constant, the cut slope angles for the excavated slope were varied as 30, 40, 45 and 60 degrees with the removal of top lateritic soil and with trees at different position over the slope was separately analysed for slopes of height 5m, 8m and 11m. The effect of turfing is also analysed.

In the case of 5m slope, only lithomargic clay layer of 5m height is considered and the excavation, ponding effect is not considered. The 11m slope has berm at 5.5m, and for 8m slope, a berm of 3m width is provided at 4m and 5.5m for 30, 40, 45 and 60 degree slopes.

4.3.2.2. Effectiveness of Plaxis 2D in solving the problem

The current Plaxis version has been successful in simulating plastic condition. A plastic calculation is used to carry out an elastic-plastic deformation analysis in which it is not necessary to take the change of pore pressure with time into account.

4.3.2.3. Safety analysis in Plaxis 2D

A safety analysis in Plaxis can be executed by reducing the strength parameters of the soil. This process is called phi-c reduction. In this approach, the strength parameters tan φ and c of the soil are successively reduced until the collapse of the slope occurs. The total multiplier ΣMsf is defined as the ratio of the strength parameters entered as input values over the reduced ones. ΣMsf is set to 1 at the start of a calculation to set all material strengths to their unreduced values. The strength parameters of soil are thereby reduced automatically step by step with an increment equal to 0.1 until the maximum number of steps (default value is 100 but can be increased up to 1000) already set is reached.

4.3.3. Detailed Procedure Adopted in Plaxis 2D

4.3.3.1. Creating model geometry, assigning soil properties and meshing

Plane strain model is used as the finite element model. The slope is analyzed as a plane strain model. Displacements and strains in the z-direction are assumed to be zero. However, normal stresses in the z-direction are fully taken into account.15-node triangular elements are selected for modelling soil layers and volume clusters. The 15-node triangle is very accurate and can produce high-quality stress results for difficult problems. The units for length, force and time used are m, kN and day respectively. The first step in every analysis is to set the basic parameters of the finite element model. These settings include the description of the problem, the type of analysis, the basic type of elements, the basic units and the size of the drawing area. The standard fixities are used to define the boundary conditions, or this is considered as the default boundary condition in new PLAXIS version.

In PLAXIS, soil properties are stored in the material data set. Mohr-Columb model was selected as the material model and the properties assigned for laterite and shedi soil is given in Table 4.3.

The excavated slope geometry was created in the structures mode using soil polygon tool as it was relatively easy to create geometric entities using this tool. The tree is simulated by plate element and roots by embedded beam row element. Hence soil properties, tree and root properties were assigned to the respective soil clusters, plate and roots using the material data set in which the properties were already stored. Fine meshes were generated in the mesh mode prior to the staged construction phase. Here in this model, the water level was fixed at the top of the slope.

Table 4.3. Input properties of the Laterite and shedi soil for both drained and undrained case

Properties	Laterite	Shedi	Shedi	Laterite New*	Shedi New*	Shedi New*
Material type	Drained	Undrained (B)	Drained	Drained	Undrained (B)	Drained
Young's Modulus (E) kN/m^2	4000	2000	2000	4000	2000	2000
Cohesion (c) kN/m^2	35	35 at 3m depth	20	45	45 at 3m depth	26
Angle of internal friction(ϕ)	30	0	23	30	0	23
Hydraulic Conductivity (k) m/day	0.1	0.035	0.035	0.1	0.035	0.035
Incremental shear strength	0	5kN/m depth	0	0	5kN/m depth	0

*Shedi new, Laterite new- Properties of soil in which effect of turfing (30% increased cohesion) is considered.

The properties of plate element (tree) and embedded beam row element (roots) are shown in Table.4.4 and 4.5, respectively.

Table 4.4. Properties of tree (Plate Element)

Properties	Tree
Material type	Elastic
EA(kN/m)	491000
Flexural rigidity,EI (kNm^2/m)	2766
Poisson's ratio	0.35

Table 4.5. Properties of roots (Embedded Beam Row Element)

Properties	Tap Roots	Branch Roots
E (kN/m^2)	6957000	6957000
Self-Weight (kN/m^3)	10	10
Diameter (m)	0.3	0.2

4.3.3.2. Calculation of Wind Load

Spacing between each tree is considered to be 3m and height of the tree is taken as 10m. Height of crown is 5m, wind velocity is around 15 kmph, and the drag coefficient is 0.2 to 0.5 for moderate wind. The wind force can be calculated (Coder Kim 2010) using Equation 4.1. The wind force developed on the treetop (Coder Kim 2010) per metre length was obtained as 7kN/m.

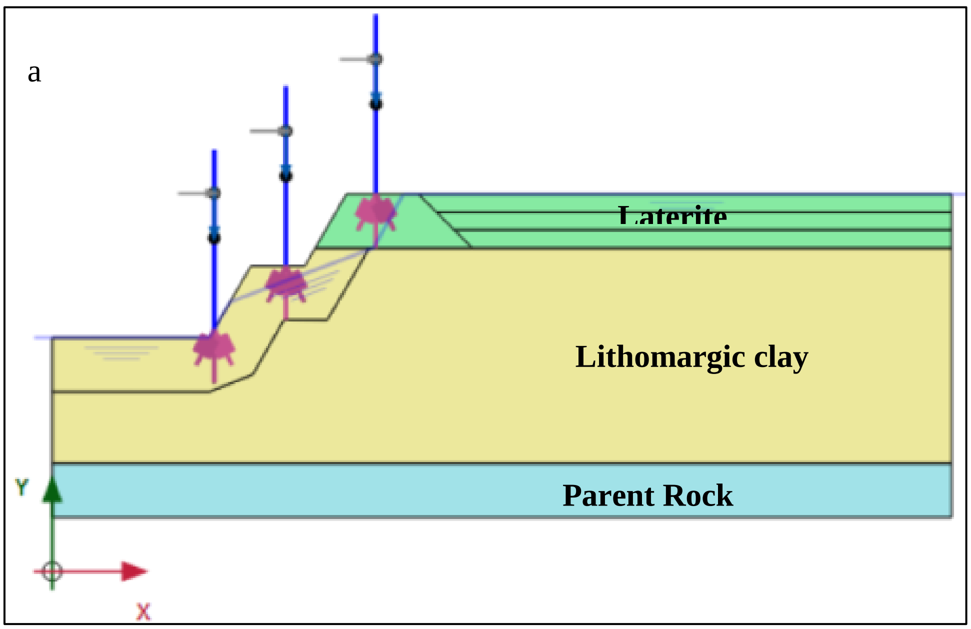
Wind force = 0.5*(wind velocity)^2*(air density)*[(Drag coefficient*Crown length*height)/2*(Height to crown base)*(Crown length/3)] (4.1)

4.3.3.3. Defining the Calculation phases for undrained and drained cases

The calculation phase/staged construction phase has been divided into 4 phases to simulate the situation for a particular geometry with a fixed slope angle and height. Trees at different position over the slope and effect of turfing along with wind load and precipitation are studied.

***(a) Initial phase* -** The initial phase is defined with the excavated slope geometry and water table at ground level assuming the slopes are almost fully saturated during the months of June and July in the study area. Initial stresses are generated in this phase using the gravity loading type of calculation that is recommended where there is a non-horizontal surface (sloping part) as the K_0 procedure does not ensure the full equilibrium of the stress field. Poisson's ratio of 0.3 was considered that resulted in the value of K_0 to be 0.5 (assuming one-dimensional elastic compression). The trees and turfing were introduced in this phase.

***(b) First phase (precipitation phase)* -** In this phase, precipitation is brought into picture on a fully saturated slope. To simulate this, infiltration rate was applied at each of the surface boundaries. It is known that in case of saturated soils, the infiltration rate almost reaches the saturated hydraulic conductivity instantaneously and becomes nearly constant. This point is used but has to be understood that an ideal case of assumption is considered here is the rainfall rate not greater than the potential infiltration rate (Ks) or no surface ponding occurs to arrive at the condition that all rainfall will infiltrate into the soil without runoff and thus the actual infiltration rate is rainfall rate (independent of time). Hence even though the infiltration rate could be slightly less or equal to the saturated hydraulic conductivity, worse case of infiltration rate equal to saturated hydraulic conductivity is adopted which means the results which imply safety can be relied upon but the vice versa is not true always. The effect of rainfall for a period of 3 days was given with calculation type as fully coupled flow deformation analysis. On the sloping surface, precipitation is modeled perpendicular to the surface and a value $q\cos\theta$ is applied where θ is the angle of slope and q the infiltration rate on a horizontal surface (i.e θ=0) for the same soil. However, internal deep ponding is a major problem in such slopes that will be applied in the next phase. Ponding effects are assumed unrealistic to occur on the sloping surface and hence the values of θmin and θmax are respectively specified as -0.001m and 0.001m which means water can only flow on the surface with a maximum height of 1mm. Safety analysis was also conducted after this to determine the safety factor (short term safety) that indicated the extent of instability caused to the slope under continuous rainfall that lasts for days. The boundary condition here is not time-dependent or is made constant i.e. discharge or head variations with time is not accounted in the absence of suitable and sufficient data. Wind load on trees are activated from this phase. The geometric profile of the slope for phase 1 for slopes with and without turfing is shown in Fig.4.2.



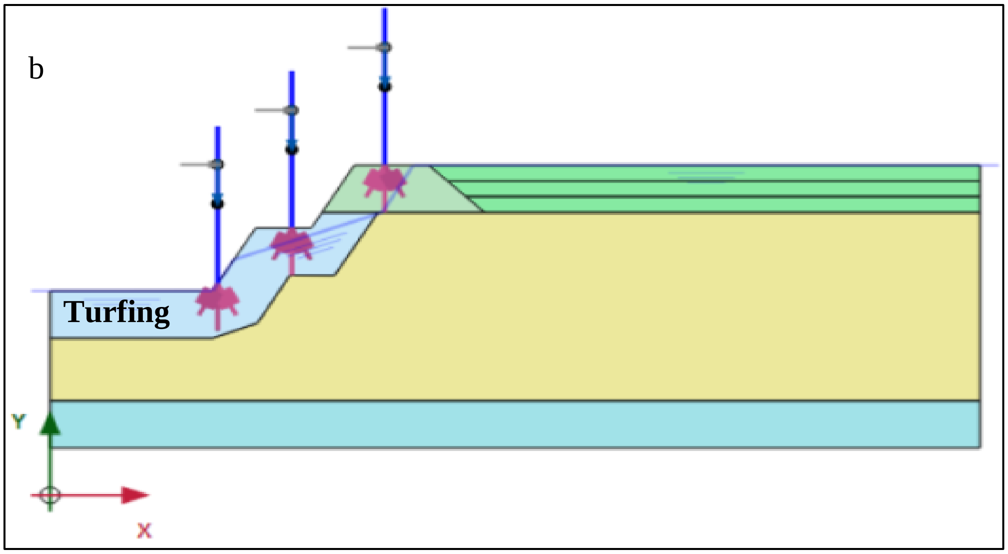


Figure 4.2. Geometric profile of the slope for first phase in undrained condition (a) without turfing (b) with turfing

***(c) Second phase* -** In the second phase, the effect of ponding is also introduced along with the precipitation and wind load. Ponding was provided for a depth of 1m at a distance of 4m from the crest for a period of 3 days. Such a soil cluster was already made in the geometry and now it is only required to deactivate the cluster i.e. removing the soil properties but keeping the water conditions as it is which means the hydrostatic pressure prevails. The additional feature of seepage behavior has to be mentioned to relevant boundaries in the ground water flow boundary conditions under the model explorer menu. Hence the additional change in the boundary conditions is done and is allowed to start from the first phase to carry forward the stresses from this phase. The calculation is still fully coupled flow deformation analysis that cares for the simultaneous development of pore pressures and displacements with time-dependent changes in the boundary condition. Here the calculations end after a period of 3 days as mentioned in the time period for the analysis between which the active pore pressures are changing continuously over time. The effect of ponding on the stability of the slope was investigated by running a safety analysis after this phase. The geometric profile of the slope for phase 2 for slopes with and without turfing is shown in Fig.4.3.

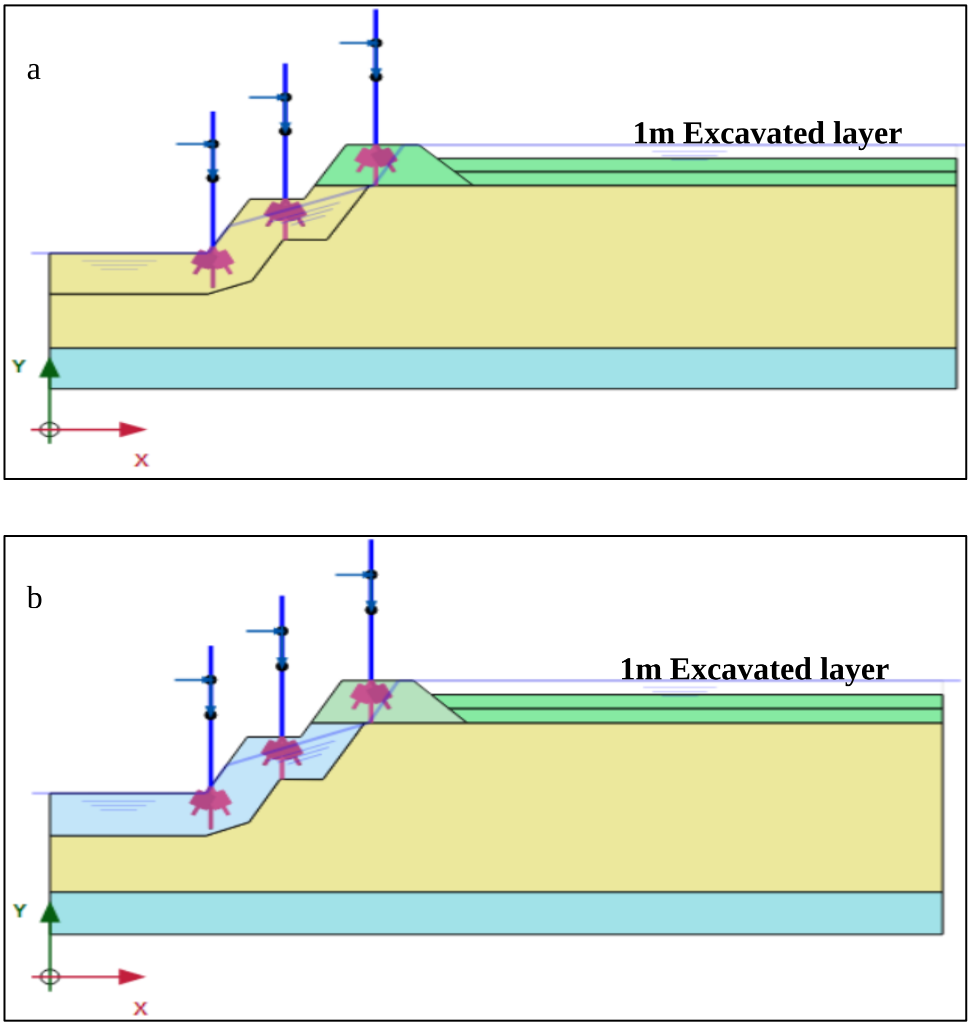


Figure 4.3. Geometric profile of the slope for second phase in undrained condition (a) without turfing (b) with turfing

(d) Third phase - This phase starts from the second phase. The procedure was almost repeated with ponding depth increased to 2m and additional boundary condition of seepage behavior been added in the groundwater flow boundary condition. Hence two clusters are in deactivated state now and the analysis is carried out for 3 days itself as for a comparison. The geometric profile of the slope for phase 3 for slopes with and without turfing is shown in Fig.4.4. Safety analysis is also carried out.

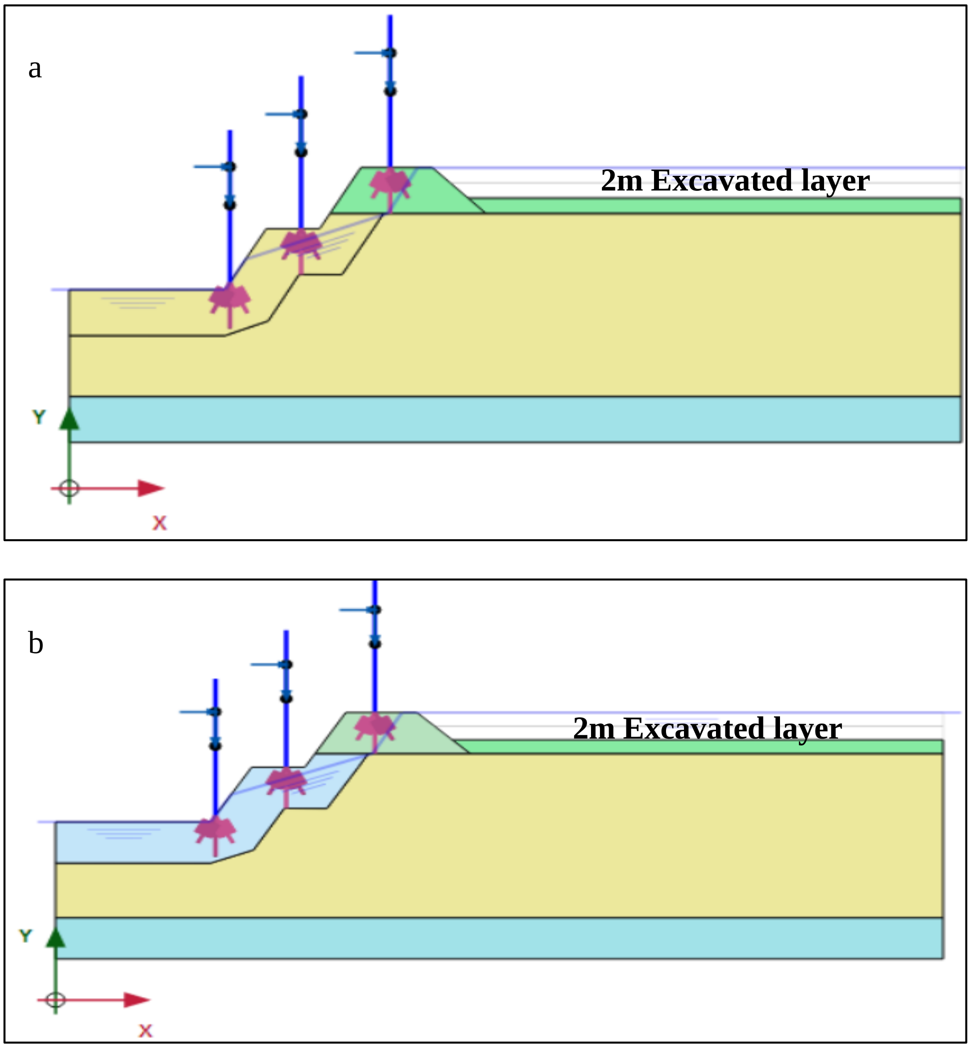


Figure 4.4. Geometric profile of the slope for third phase in undrained condition (a) without turfing (b) with turfing

(e) Fourth phase **-** This is the last and final phase in which the ponding depth almost covers whole of the top lateritic layer i.e. 3m and the effect on the slope stability was carried out as done in other phases after deactivating the last cluster and changing the hydraulic boundary behaviour to seepage at the required boundaries. It is to be remembered that all these phases including this final phase has got the precipitation and wind effect. The geometric profile of the slope for phase 4 for slopes with and without turfing is shown in Fig.4.5.

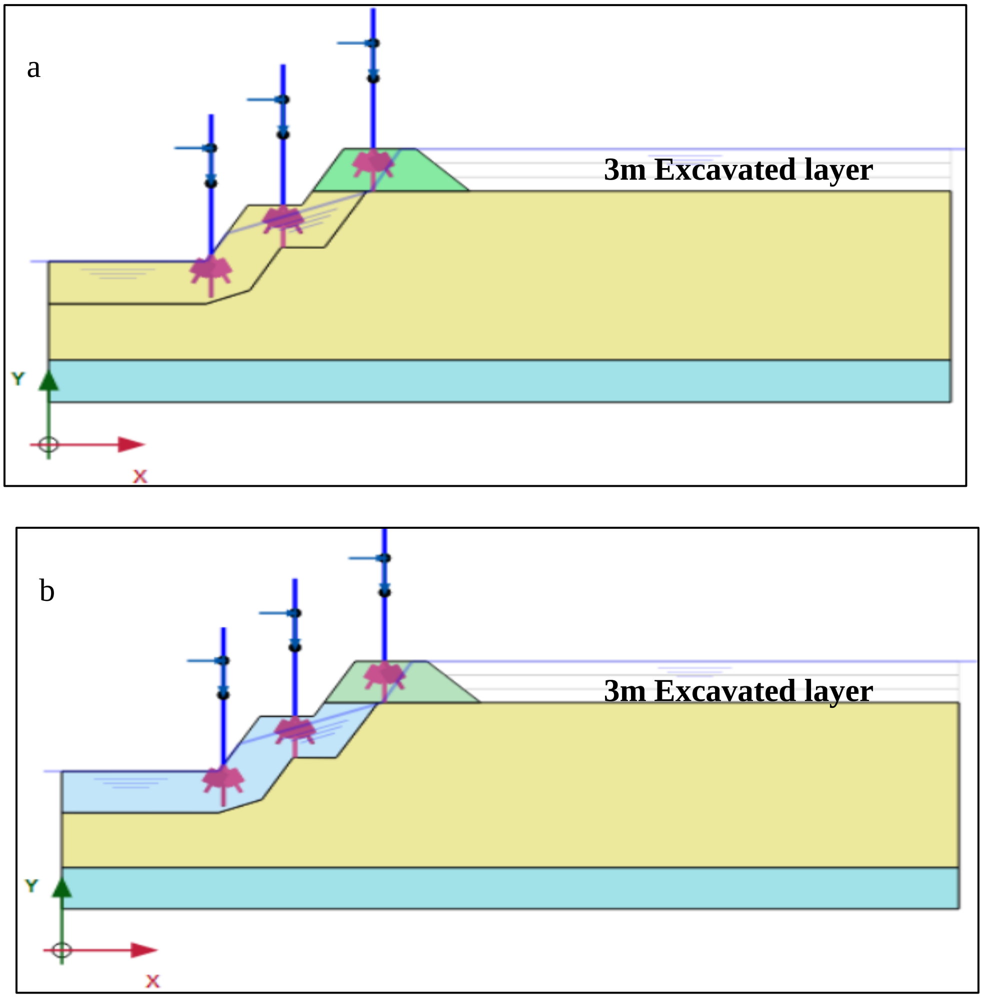


Figure 4.5. Geometric profile of the slope for fourth phase in undrained condition (a) without turfing (b) with turfing

In drained case, same procedure is used but in the absence of ground water table and precipitation (Fig.4.6). In the case of 5m slope only 2 phases are present. Initial phase consists of slope with tree and in phase 1 wind load is activated.

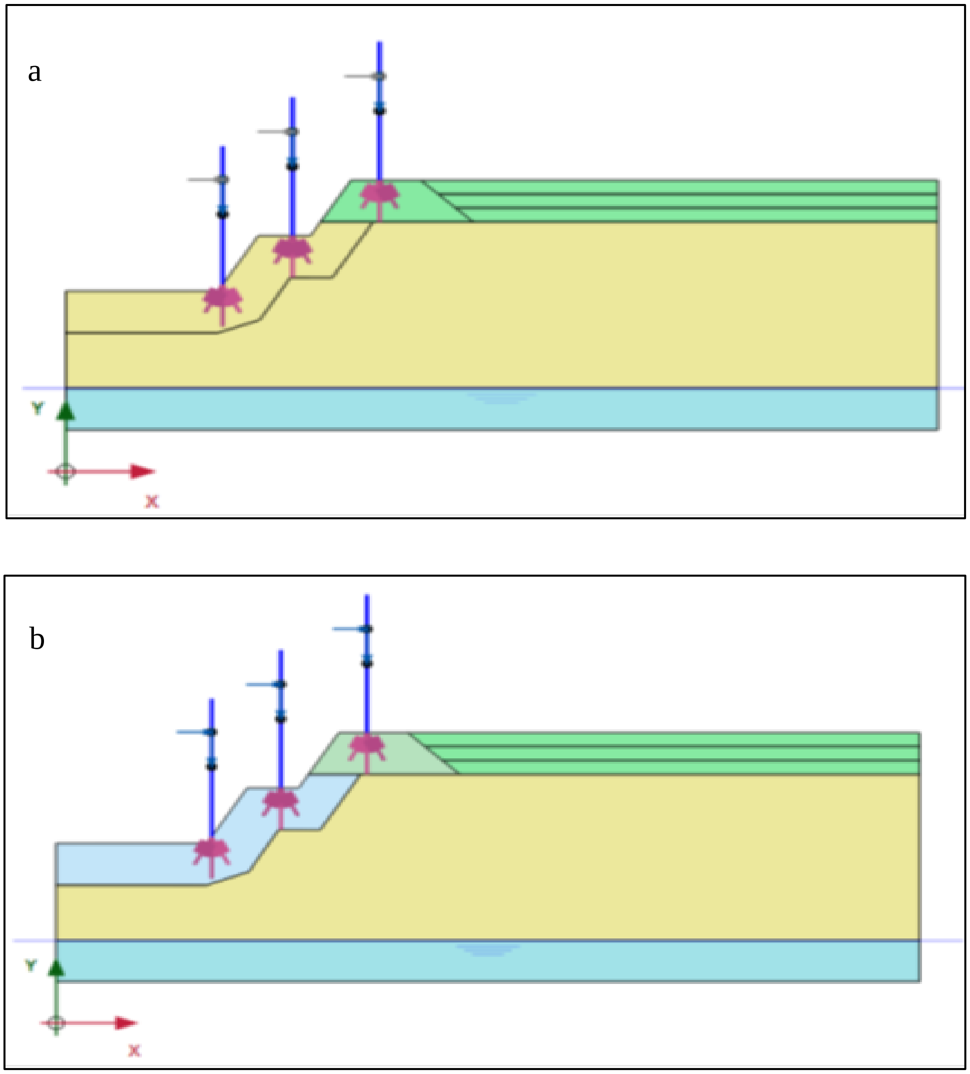


Figure 4.6. Geometric profile of the slope for first phase in drained condition (a) without turfing (b) with turfing

CHAPTER 5

RESULTS AND DISCUSSIONS

5.1. HOLE EROSION TEST RESULTS

5.1.1. Determination of critical head

Test results indicated slaking for C samples (clay fraction 33.2% to 55.3%, silt fraction 26.3% to 42.7%, PI 15% to 23%) when tested at various heads ranging from 50cm to 100cm. Slaking by definition is the "disintegration of unconfined soil after exposure to air and subsequent immersion in water; no external confining pressure is assumed to act over the soil prior to immersion" (Moriwaki and Mitchell 1977). However, all the C samples underwent progressive erosion from a head of 110cm. Tests were conducted on C specimens at heads of 110cm, 125cm, 140cm and 155cm. In contrast, all the M samples (clay fraction 7.4% to 16.6%, silt fraction 51.0% to 79.9%, PI 9% to 16%) underwent progressive erosion from a head of 30cm. Tests were conducted on M specimens at heads of 30cm, 40cm, 60cm and 70cm. At higher heads the specimens were washed off. The soil samples at different stages or conditions of HET are shown in Figs.5.1 and 5.2.

Figure 5.1. (a) Sample before test with 6mm hole (b) Sample(C10) which showed very less surface erosion after HET when coated with wax on its surface

Figure 5.2. (a) Cross section of a sample after HET (b) Formation of second hole due to slaking

5.1.2. Typical Results (Determination of I_h and C_e)

The discharge (Fig.5.3), hole diameter (Fig.5.4), hydraulic shear stress (Fig.5.5) and erosion rate (Fig.5.6) were plotted with time. The erosion rate was plotted against hydraulic shear stress (Fig.5.7) to know whether progressive erosion has occurred and also to understand how critical the erosion is.

The typical results obtained for progressive erosion are shown in Fig. 5.3 to Fig. 5.7. The variation of discharge with time showed that the discharge increased from 0.0001 to 0.00033m^3/s (Fig.5.3). The evolution of hole diameter over time indicates that hole diameter increased from 6mm to 15mm in 24 minutes (Fig.5.4). It is important to note that the hole diameters represented average diameters as they varied over the length. The hole was enlarged at both ends. This is due to the spalling of soil caused by eddies present in the inlet and outlet (Wahl et al. 2009). In some cases surface had been eroded reducing the length of the eroded hole.

The hydraulic shear stress increased over time from 150Pa to 475Pa over a time of about 24 minutes (Fig.5.5). The rate of increase in hydraulic shear stress increased towards the end of the test. The increase in rate of erosion represents the rate of increase in hole diameter over time. Figure 5.6 indicates that progressive erosion has occurred during the test. A slightly decreasing trend observed initially represents the clean off phase, where the loose soil particles around the hole are removed by water and is called as cleanout erosion. These are formed due to the disturbances caused in the sample while the hole is being drilled. The hole stabilizes itself during the cleanout erosion and when the critical shear stress is attained progressive erosion is initiated.

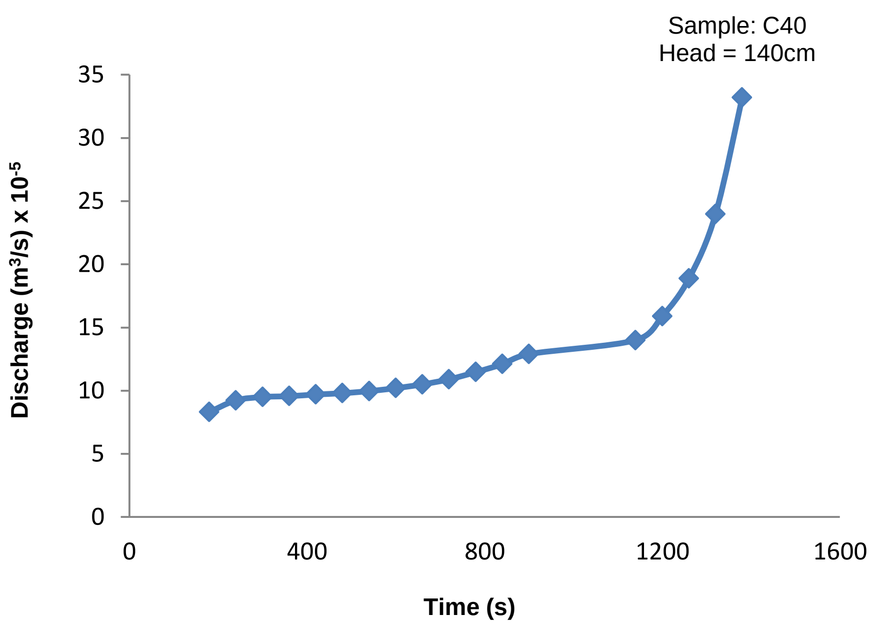


Figure 5.3. Variation of Discharge with time for C40 sample

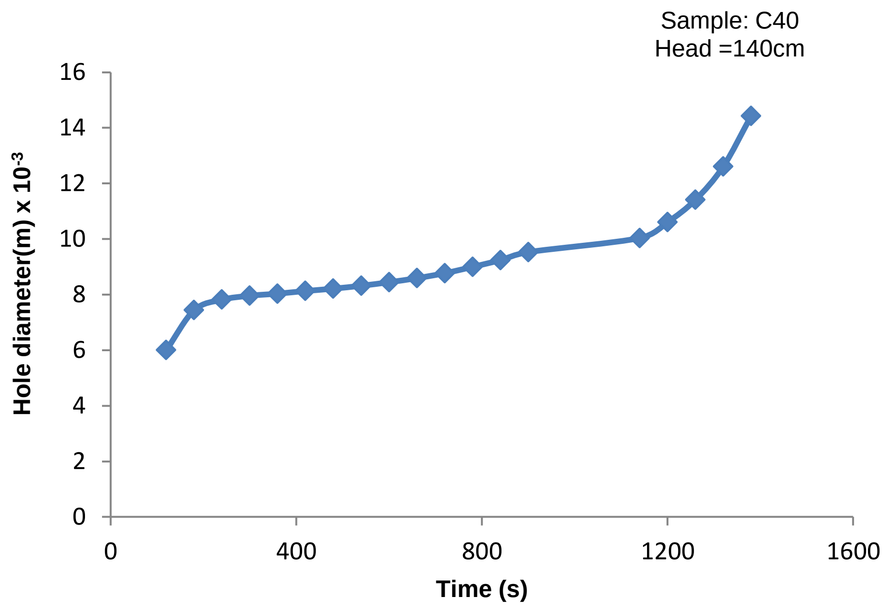


Figure 5.4. Evolution of hole diameter over time for C40 sample

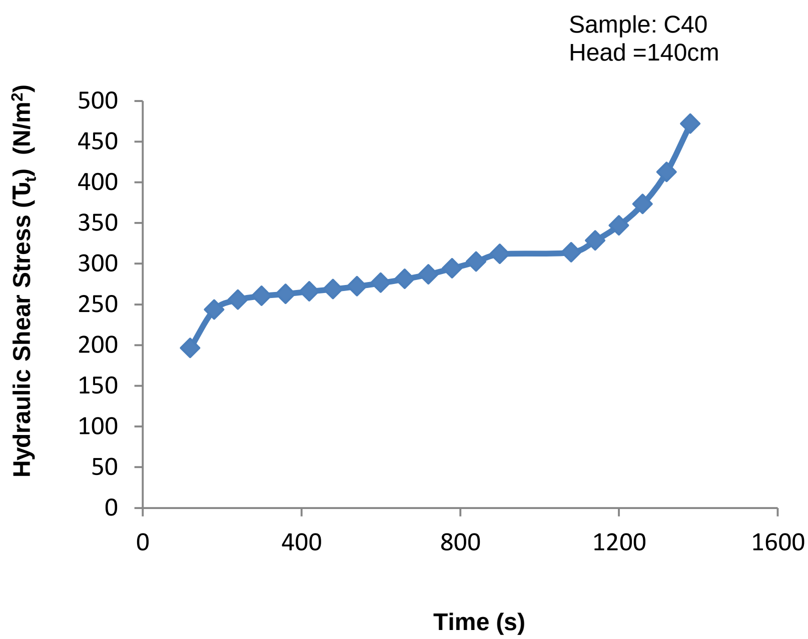


Figure 5.5. Variation of hydraulic shear stress with time for C40 sample under a head of 140cm

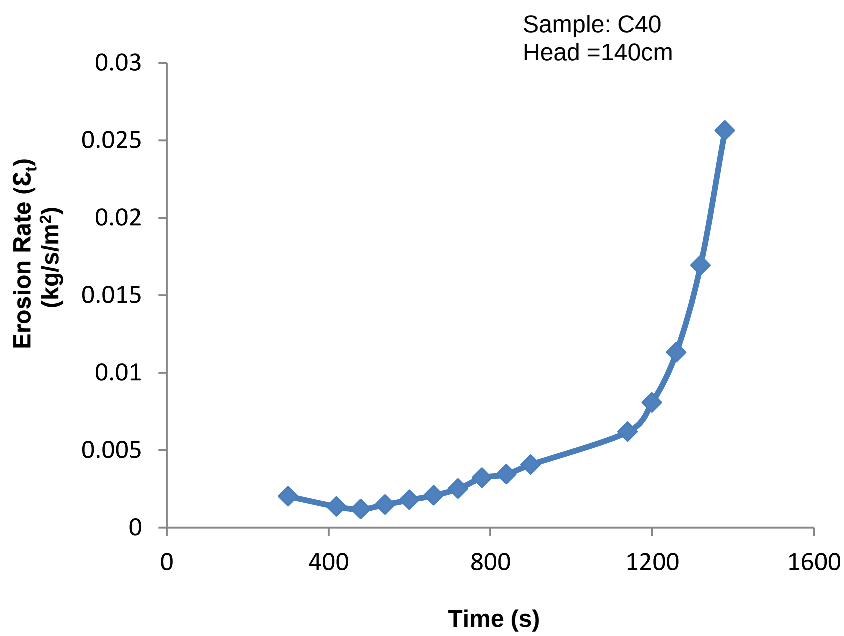


Figure 5.6. Variation of erosion rate with time for C40 sample under a head of 140cm

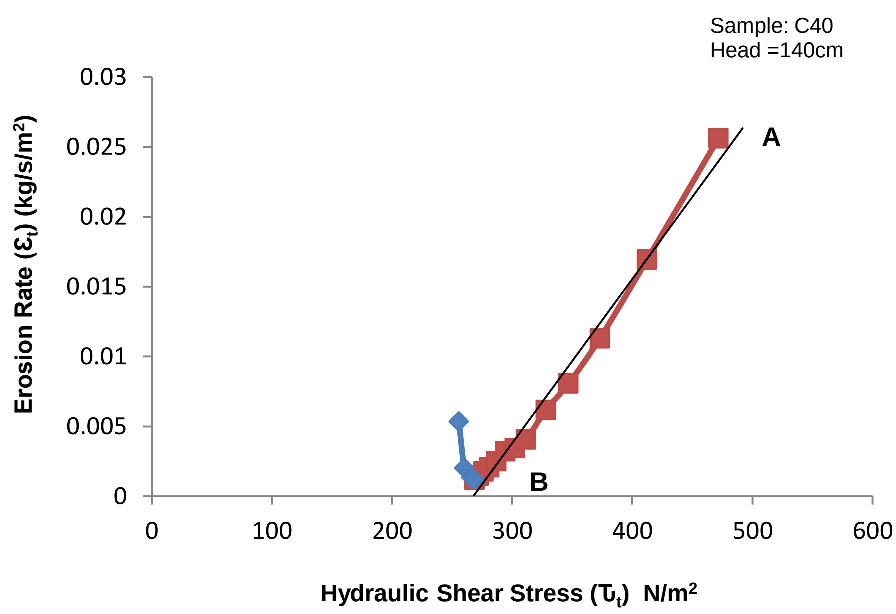


Figure 5.7. Plot of Erosion rate v/s Hydraulic Shear stress

Figure 5.7 shows a plot of erosion rate [ε_t (kg/s/m^2)] versus hydraulic shear stress [τ_t (N/m^2)] for sample C40. Therein AB represents the best fit curve of the increasing portion of erosion rate v/s shear stress relationship. Its slope gives the Coefficient of soil erosion (C_e) and the hole erosion index (I_h) is obtained as 4.52 for the sample C40 under a head of 140cm. As per the soil classification by Wan and Fell [10-12] the erosion rate of soil may be classified as moderately slow (Table 3.3). The intercept of the best fit line with the x- axis gives the critical shear stress and is obtained as 270N/m^2 (Fig.5.7). This value is less than the critical shear stress obtained for other samples containing lesser sand fraction.

5.1.3. Influence of change in moulding water content

In excavated slopes, in lateritic formations, caving - in of lithomargic clay soil can be observed after heavy rainfall. After a first few days of constant and low intensity rainfall, the soil gets fully saturated causing erosion of lithomargic clay. After a heavy intensity rainfall (higher heads due to stagnation) wash off might occur, resulting in caving in or concavities. These conditions were simulated in the laboratory by varying the moulding water content of the soil and by applying varying heads of water. Specimens were prepared with the different fines and sand combinations at varying water contents. The soils were compacted to their maximum dry densities.

5.1.3.1. C samples

Three water contents used were such as the optimum moisture content (OMC) of the corresponding samples, 50% of optimum moisture content and the water content corresponding to 100% saturation (i.e.110% of optimum moisture content). This is done to study the influence of moulding water content on erosion index (I_h) of C samples. The samples were tested at heads of 110cm, 125cm, 140cm and 155cm.

i) Soil Samples ('C' Samples) Prepared at 50% OMC

Table 5.1 Erosion rate index of C Samples prepared at 50% of OMC

HEAD(cm)	Erosion Rate Index (I_h)				
	C0	C10	C20	C30	C40
110	3.95	4.07	4.17	4.20	4.52
125	3.97	4.09	4.25	4.44	4.62
140	3.98	4.18	4.26	4.57	4.70
155	4.12	4.30	4.52	4.62	4.76

Table 5.2 Coefficient of soil erosion (C_e) of C Samples prepared at 50% of OMC

HEAD(cm)	Coefficient of soil erosion, C_e (s/m) x10^{-4}				
	C0	C10	C20	C30	C40
110	1.12	0.85	0.68	0.63	0.30
125	1.07	0.81	0.56	0.36	0.24
140	1.05	0.66	0.55	0.27	0.20
155	0.76	0.50	0.30	0.24	0.17

The soil samples were compacted to maximum dry density at a moisture content corresponding to 50% of OMC. The observations for C samples are shown in Figs.5.8 and 5.9. Table 5.1 and 5.2 shows the erosion rate index and coefficient of soil erosion of C samples compacted at 50% OMC. The erosion rate index of all the soil samples vary from 3.95 to 4.76. Hence the rate of erosion may be classified as moderately slow according to Wan and Fell [10-12] as shown in Table 3.3. An increase in erosion rate index represents a decrease in the rate of erosion. From Fig.5.8, it can be observed that the original lithomargic clay sample C0 has the maximum rate of erosion and sample C40 (in which lithomargic soil C0 was mixed with 40% river sand) is having least erosion rate and therefore is more stable. When the sample is compacted at 50% OMC the eroding water has to seep radially through the sample to wetten the soil and disperse it. Hence the soil containing the highest void ratio allows water to pass more easily through it. C0 has the highest void ratio and hence is the most erodible while C40 has the least void ratio. In addition, the sand fraction in the C40 sample adds stability to the soil structure. The rate of wetting is higher when the velocity of water is the least. Hence a higher rate of erosion is observed at lower heads. To confirm the same, UCC samples were prepared and immersed in water and the time taken by the soil samples to fully disperse were determined. It was found that the C0 sample dispersed more quickly than C40 sample.

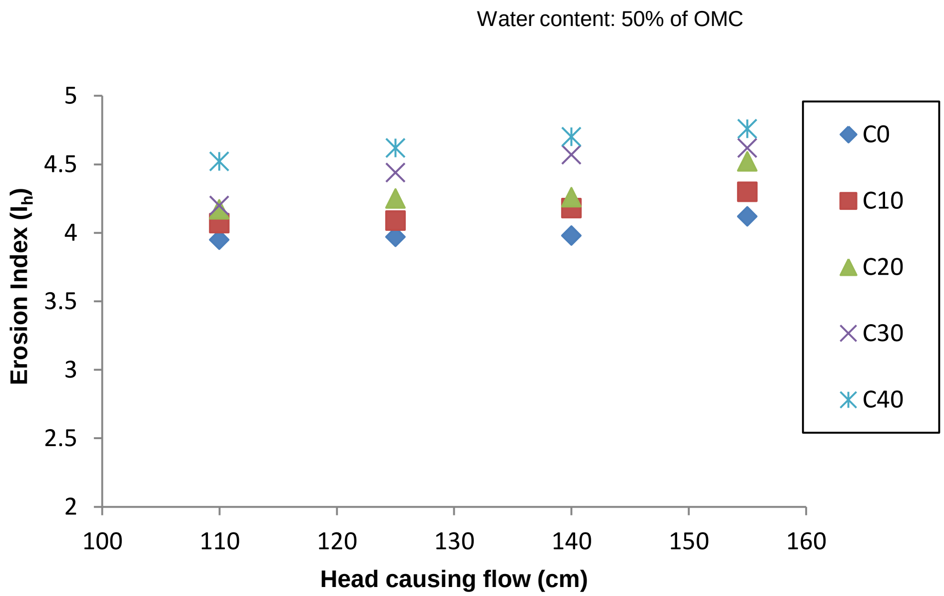


Figure 5.8.Variation of Erosion rate index of C samples compacted at 50% of OMC

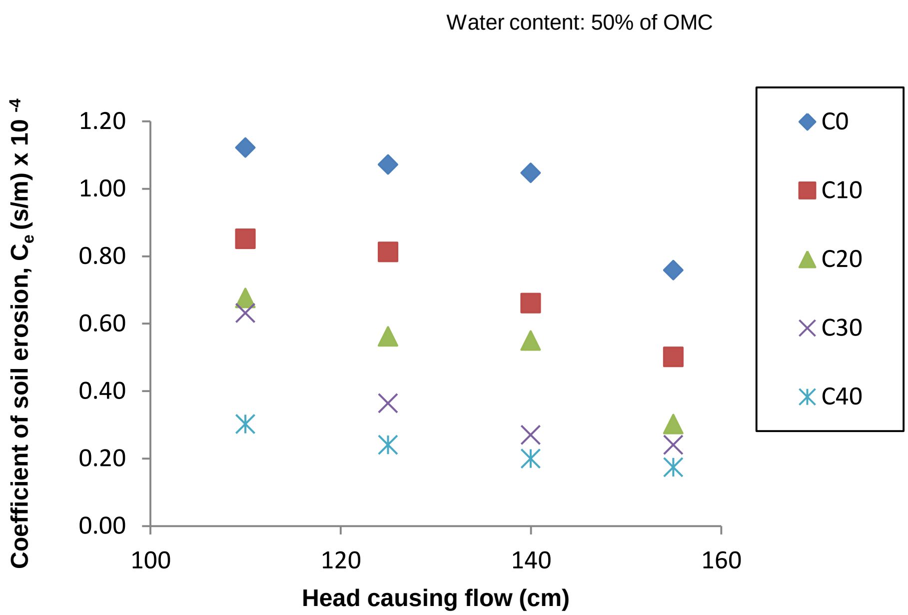


Figure 5.9. Variation of Coefficient of soil erosion, C_e, of C samples compacted at 50% of OMC

ii) Soil Samples ('C' Samples) Prepared at OMC

The soil samples were compacted to maximum dry density with a moisture content corresponding to their OMC. The erosion rate index varied from 3.89 to 4.84 in case of samples moulded at OMC (Fig.5.10). Table 5.3 and 5.4 shows the erosion rate index and coefficient of soil erosion of C samples compacted at OMC. The C40 sample is seen to be more stable than C0 sample. The soil is not fully saturated at OMC and the better soil size distribution of C40 sample makes it more resistant to erosion. The water passing through the hole removes the soil surrounding it. The non-eroded portion of the sample remains undisturbed after the test and no significant change in the water content has been noticed. As the head increases, the stress acting on the soil increases and consequently erosion rate increases. The variation of Coefficient of soil Erosion, C_e, of various samples compacted at OMC for C samples is shown in Fig.5.11.

Table 5.3 Erosion rate index of C Samples prepared at OMC

HEAD(cm)	Erosion Rate Index (I_h)				
	C0	C10	C20	C30	C40
110	4.26	4.35	4.50	4.52	4.84
125	4.03	4.13	4.21	4.27	4.64
140	3.94	4.06	4.16	4.19	4.52
155	3.89	3.97	4.06	4.09	4.46

Table 5.4 Coefficient of soil erosion (C_e) of C Samples prepared at OMC

HEAD(cm)	Coefficient of soil erosion, C_e (s/m) $x10^{-4}$				
	C0	C10	C20	C30	C40
110	0.55	0.45	0.32	0.30	0.14
125	0.93	0.74	0.62	0.54	0.23
140	1.15	0.87	0.69	0.65	0.30
155	1.29	1.07	0.87	0.81	0.35

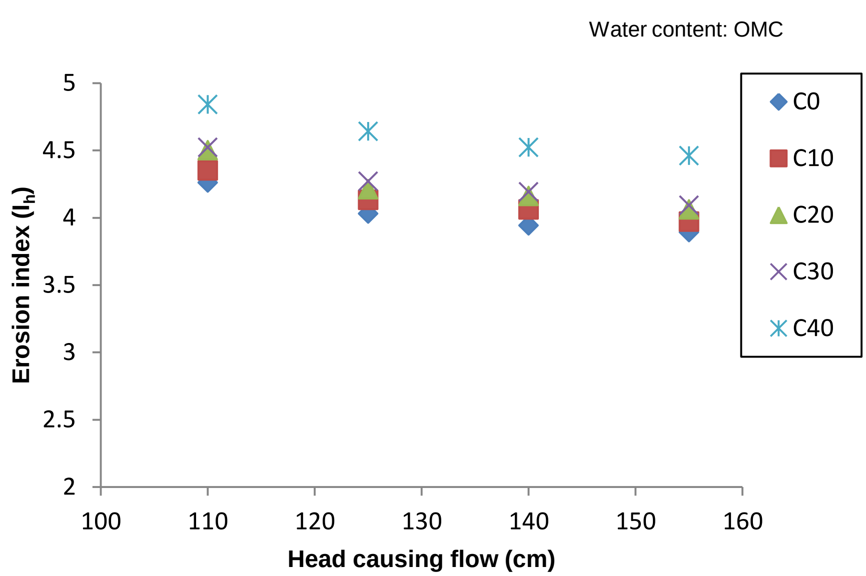


Figure 5.10.Variation of erosion rate index (I_h) of C samples compacted at OMC

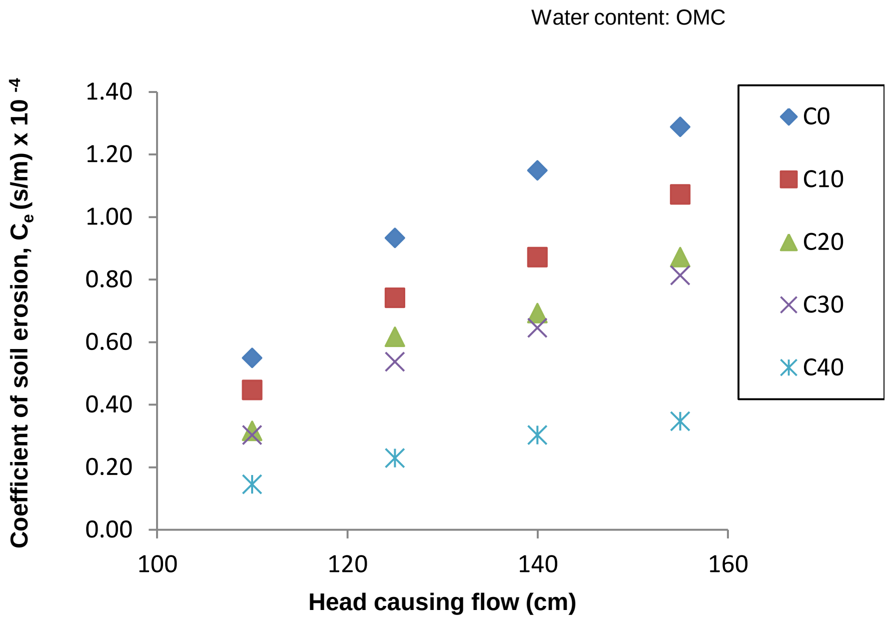


Figure 5.11. Variation of Coefficient of soil Erosion, C_e, of C samples compacted at OMC

iii) Soil Samples ('C' Samples) Prepared at 110% OMC (Fully Saturated Condition)

Table 5.5 and 5.6 shows the erosion rate index and coefficient of soil erosion of C samples compacted at 110% OMC (fully saturated condition).

Table 5.5 Erosion rate index of C Samples prepared at 110% OMC (Fully Saturated Condition)

HEAD(cm)	Erosion Rate Index (I_h)				
	C0	C10	C20	C30	C40
110	4.64	4.26	4.18	4.10	3.98
125	4.52	4.38	4.23	4.12	3.83
140	4.20	4.04	3.93	3.73	W
155	4.14	W	W	3.67	W

(Note: W – Wash off occurred at higher heads in the case of these samples)

Table 5.6 Coefficient of soil erosion (C_e) of C Samples prepared at 110% OMC (Fully Saturated Condition)

HEAD(cm)	Coefficient of soil erosion, C_e (s/m) x10^{-4}				
	C0	C10	C20	C30	C40
110	0.23	0.55	0.66	0.79	1.05
125	0.30	0.42	0.59	0.76	1.48
140	0.63	0.91	1.17	1.86	W
155	0.72	W	W	2.14	W

(Note: W – Wash off occurred at higher heads in the case of these samples)

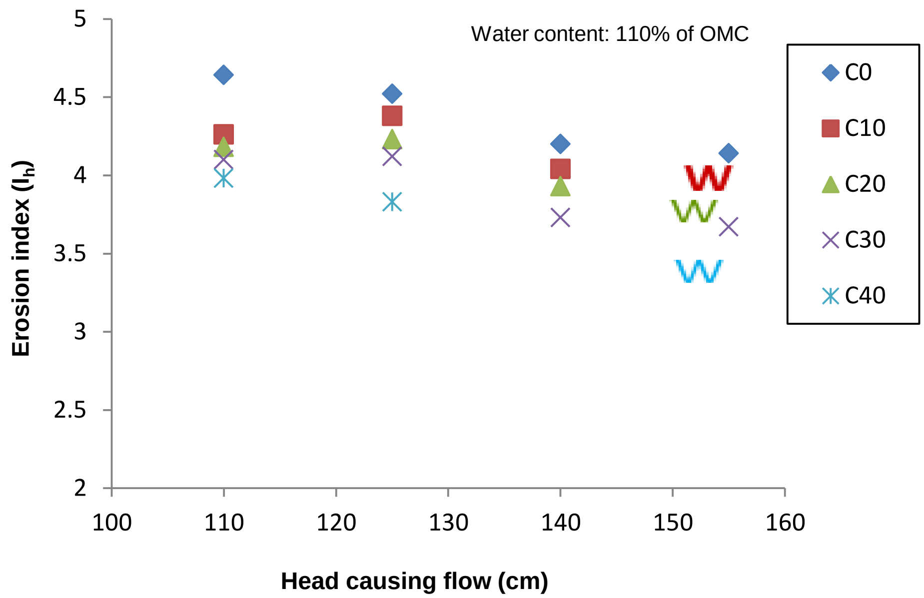


Figure 5.12. Variation of Erosion rate index (I_h) of C samples at fully saturated condition (Note: W denotes wash off occurred beyond this head)

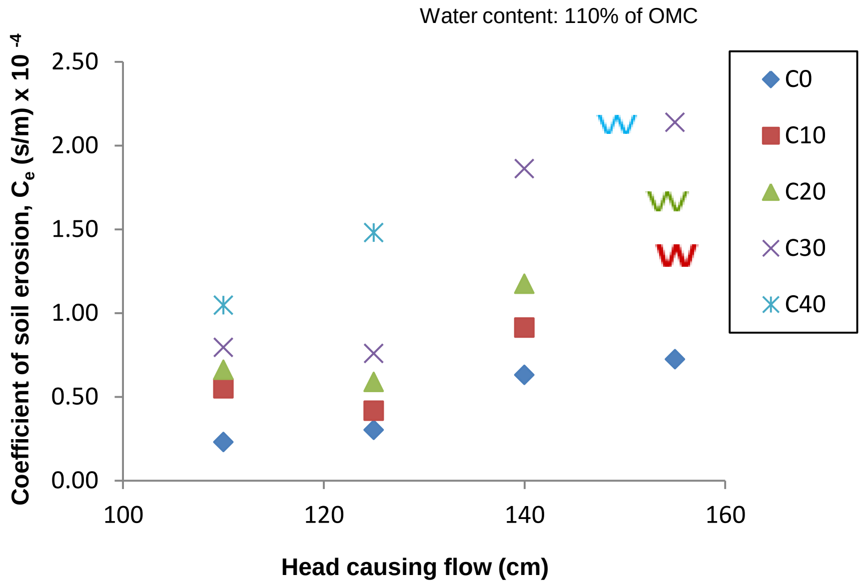


Figure 5.13. Variation of Coefficient of soil Erosion, C_e, of C samples at fully saturated condition (Note: W denotes wash off occurred beyond this head)

The samples were compacted to maximum dry densities at water content corresponding to 110% of OMC (fully saturated condition) and the erosion rate index varied from 3.67 to 4.64. When fully saturated, C0 sample which had more cohesion due to high percentage of fine fraction of soils was more stable (Figs.5.12 and 5.13). In C0 sample, it was observed that the fines eroded as individual particles and not as lumps. At higher heads, samples containing higher sand content were washed off. The sand soil mix was found to be deposited in the overflowing jar as lumps. Also as the head increases the rate of erosion increased due to the increase in hydraulic shear stress.

5.1.3.2. M samples

Three various water contents used were such as the optimum moisture content (OMC) of the corresponding samples, 50% of optimum moisture content and the water content corresponding to 100% saturation (106% of optimum moisture content). This is done to study the influence of moulding water content on erosion index (I_h) of M samples. The samples were tested at heads of 30cm, 40cm, 60cm and 70cm.

i) Soil Samples ('M' Samples) Prepared at 50% OMC

Table 5.7 and 5.8 shows the erosion rate index and coefficient of soil erosion of M samples compacted at 50% OMC.

Table 5.7 Erosion rate index of M Samples prepared at 50% of OMC

	Erosion Rate Index (I_h)				
HEAD(cm)	M0	M10	M20	M30	M40
30	3.70	3.68	3.48	3.42	3.37
40	3.58	3.59	3.39	3.30	3.24
60	3.50	3.32	3.15	3.09	2.77
70	3.48	W	W	W	W

(Note: W – Wash off occurred at higher heads in the case of these samples)

Table 5.8 Coefficient of soil erosion, C_e of M samples prepared at 50% of OMC

	Coefficient of soil Erosion, C_e (s/m) x10^{-4}				
HEAD(cm)	M0	M10	M20	M30	M40
30	2.00	2.09	3.31	3.80	4.27
40	2.63	2.57	4.07	5.01	5.75
60	3.16	4.79	7.08	8.13	16.98
70	3.31	W	W	W	W

(Note: W – Wash off occurred at higher heads in the case of these samples)

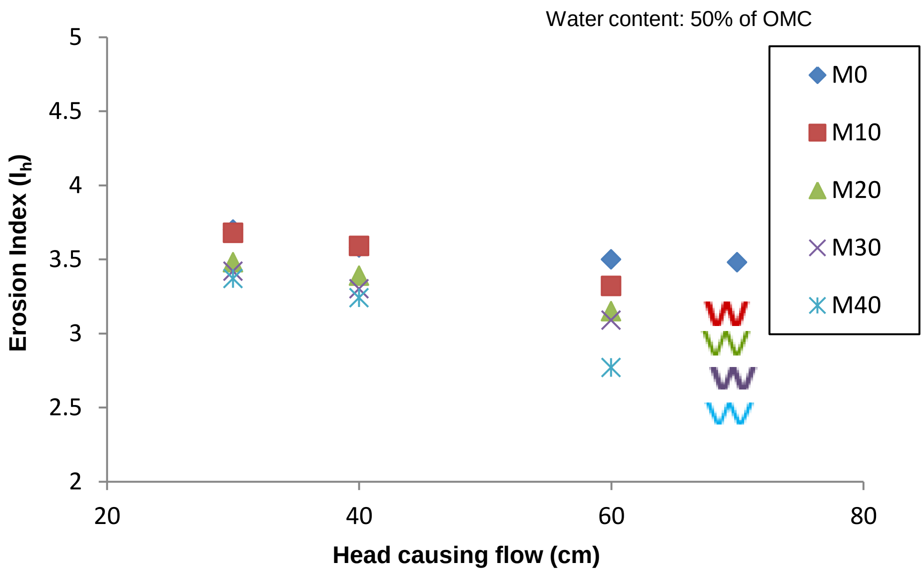


Figure 5.14. Variation of Erosion rate index (I_h) of M samples compacted at 50% of OMC (Note: W denotes wash off occurred beyond this head)

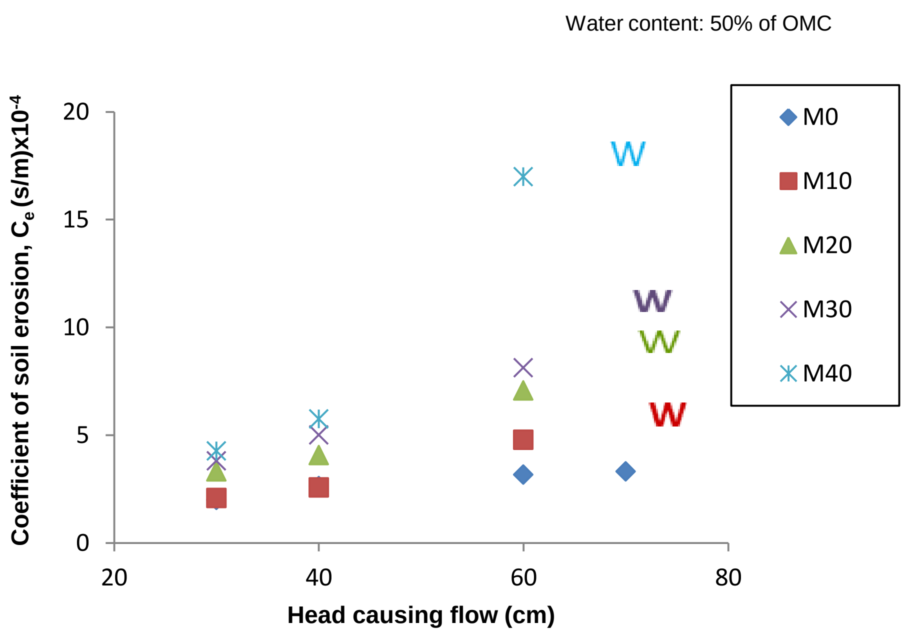


Figure 5.15. Variation of (Coefficient of soil erosion, C_e, of M samples compacted at 50% of OMC (Note: W denotes wash off occurred beyond this head)

The soil samples were prepared at maximum dry density at a moisture content corresponding to 50% of OMC. The erosion rate index varies from 2.77 to 3.7 (Fig.5.14) and can be classified as very rapid to moderately rapid erosion (Table 3.3). It can be observed that the M40 sample (M0 sample + 40% sand) has lower I_h value and thus has the maximum rate of erosion. The original lithomargic soil sample (sample M0) has higher I_h value and so is the most stable. The M0 sample is more stable because of higher percentage of fines (because of which it possesses higher cohesion) and lesser sand fraction. The cohesion in the M0 sample acts as a bond and holds the soil particles together. Due to the higher percentage of sand along with silt content in the M40 sample, the cohesion is less and it gets eroded more and hence is least stable. The erosion also increases with head causing flow. Higher rate of erosion is observed at higher heads. The variation of Coefficient of soil Erosion, C_e, of various samples compacted at 50% OMC is shown in Fig.5.15.

ii) Soil Samples ('M' Samples) Prepared at OMC

Table 5.9 and 5.10 shows the erosion rate index and coefficient of soil erosion of M samples compacted at OMC.

Table 5.9 Erosion rate index of M Samples prepared at OMC

	Erosion Rate Index (I_h)				
HEAD(cm)	M0	M10	M20	M30	M40
30	3.69	3.62	3.60	3.52	3.51
40	3.68	3.56	3.52	3.52	3.47
60	3.26	3.20	3.07	2.99	2.96
70	W	W	W	W	W

(Note: W – Wash off occurred at higher heads in the case of these samples)

Table 5.10 Coefficient of soil erosion, C_e of M samples prepared at OMC

HEAD(cm)	Coefficient of soil Erosion, C_e (s/m) $x10^{-4}$				
	M0	M10	M20	M30	M40
30	2.04	2.40	2.5	3.02	3.09
40	2.09	2.75	3.02	3.02	3.39
60	5.50	6.31	8.51	10.23	10.96
70	W	W	W	W	W

(Note: W – Wash off occurred at higher heads in the case of these samples)

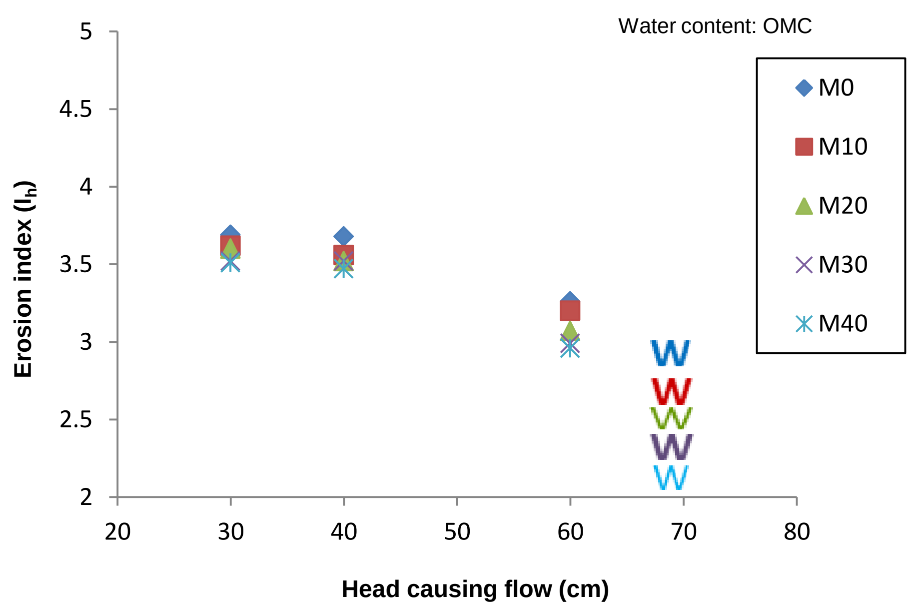


Figure 5.16. Variation of erosion rate index (I_h) of M samples compacted at OMC (Note: W denotes wash off occurred beyond this head)

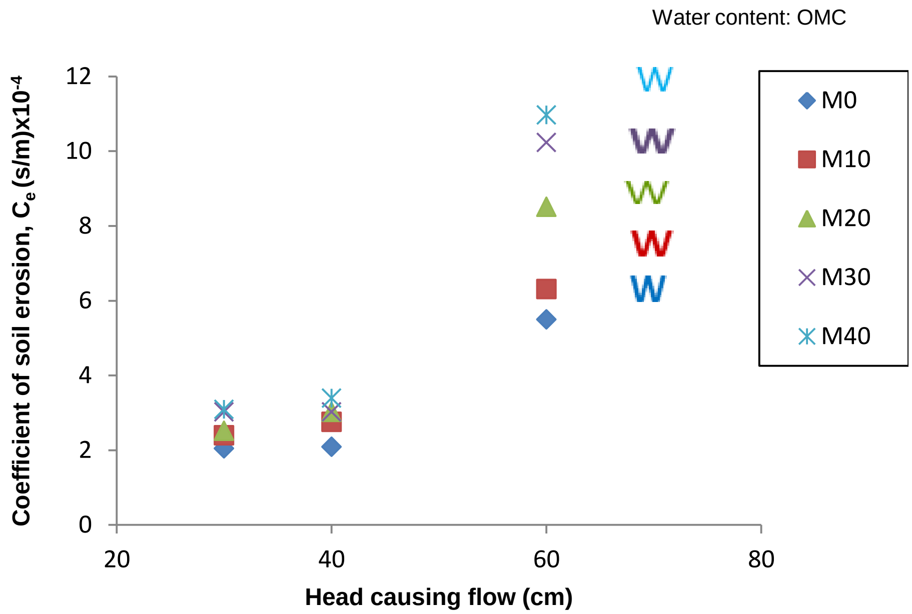


Figure 5.17. Variation of Coefficient of soil Erosion, C_e, of M samples compacted at OMC (Note: W denotes wash off occurred beyond this head)

The soil samples were compacted to maximum dry density with a moisture content corresponding to their OMC. The erosion rate index varies from 2.96 to 3.69 (Fig.5.16) and can be classified as moderately rapid erosion as per Table 3.3. At OMC conditions, M0 sample had maximum cohesion due to the increase in water content and hence was more stable. As the head increases, the stress acting on the soil increases and consequently erosion rate increases. The variation of Coefficient of soil Erosion, C_e, of M samples compacted at OMC is shown in Fig.5.17.

iii) Soil Samples ('M' Samples) Prepared at 106% OMC (Fully saturated condition)

Table 5.11 Erosion rate index of M Samples prepared at 106% OMC (Fully saturated condition)

HEAD(cm)	Erosion Rate Index (I_h)				
	M0	M10	M20	M30	M40
30	3.35	3.18	3.01	2.94	2.72
40	3.32	3.28	3.02	3.02	2.75
60	3.02	2.93	2.83	2.82	2.59
70	2.91	2.78	2.76	W	W

(Note: W – Wash off occurred at higher heads in the case of these samples)

Table 5.12 Coefficient of soil erosion, C_e of M samples prepared at 106% OMC (Fully saturated condition)

HEAD(cm)	Coefficient of soil Erosion, C_e (s/m) x10^{-4}				
	M0	M10	M20	M30	M40
30	4.47	6.61	9.77	11.48	19.05
40	4.79	5.25	9.55	9.55	17.78
60	9.55	11.75	14.79	15.14	25.70
70	12.30	16.6	17.38	W	W

(Note: W – Wash off occurred at higher heads in the case of these samples)

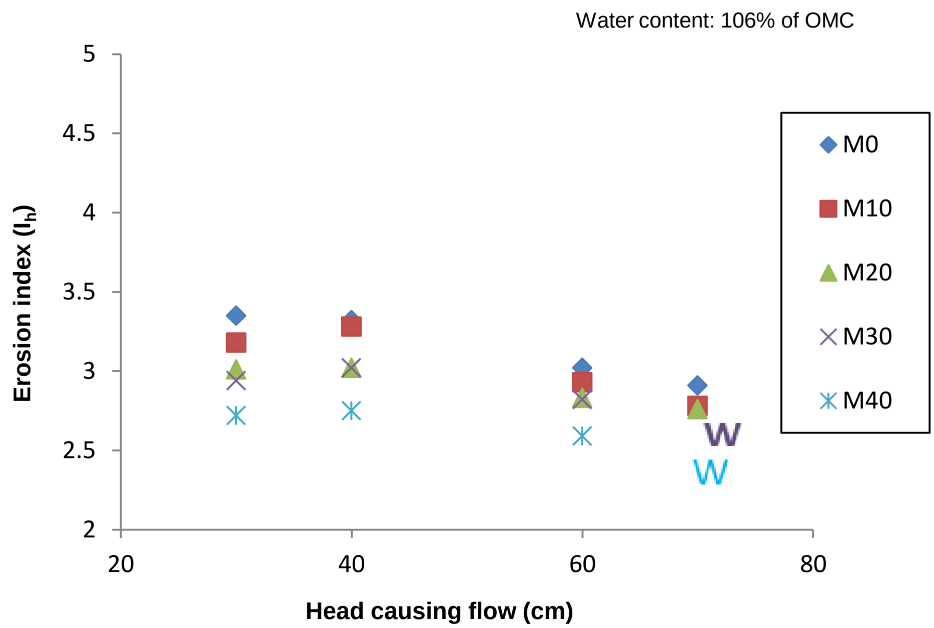


Figure 5.18. Variation of (a) erosion rate index (I_h) of M samples compacted at 106% of OMC (Note: W denotes wash off occurred beyond this head)

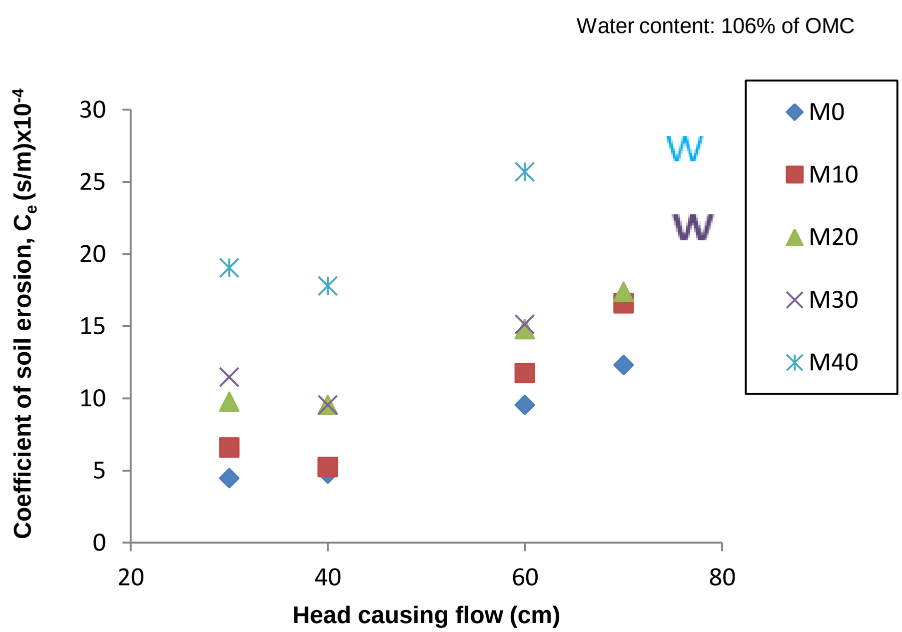


Figure 5.19. Variation of Coefficient of soil Erosion, C_e, of M samples compacted at 106% of OMC (Note: W denotes wash off occurred beyond this head)

The soil samples were prepared at maximum dry density at a moisture content corresponding to 106% of OMC (fully saturated condition). Table 5.11 and 5.12 shows the erosion rate index and coefficient of soil erosion of M samples compacted at 106% OMC. The erosion rate index varies from 2.59 to 3.35 (Fig.5.18). Hence the rate of erosion may be classified as very rapid to moderately rapid as per Table 3.3. A decrease in the erosion rate index represents an increase in the rate of erosion. At 106% of OMC, M0 sample had fullest cohesion due to the highest degree of saturation and hence was more stable. As the head increases, the stress acting on the soil increases and consequently erosion rate increases. The variation of Coefficient of soil erosion, C_e, of M samples compacted at OMC is shown in Fig.5.19.

It can be inferred that erosion is higher at full saturation and lower at partially saturated condition including at OMC. As fine sand fraction increases, erosion also increases.

5.1.4. Influence of degree of compaction

5.1.4.1. C samples

To understand the influence of degree of compaction, hole erosion tests were conducted on C20, C30 and C40 samples. Soil samples were compacted to 80%, 90% and 100% of their maximum dry densities. The water contents were the corresponding moisture content on light compaction curves on the dry side of optimum. Specimens were tested at heads of 125cm, 140cm and 155cm. Table 5.13 shows the erosion rate index of C samples for different degrees of compaction.

Table 5.13 Erosion rate index of C Samples for different degrees of compaction

Degree of compaction	Erosion rate index (I_h)								
	100%			90%			80%		
Head(cm)	C20	C30	C40	C20	C30	C40	C20	C30	C40
30	4.21	4.27	4.64	3.53	3.75	3.80	4.92	4.22	3.41
40	4.16	4.19	4.52	4.16	4.25	4.48	4.22	3.52	3.37
60	4.06	4.09	4.46	4.13	4.60	W	3.58	3.23	W

(Note: W denotes wash off occurred beyond this head)

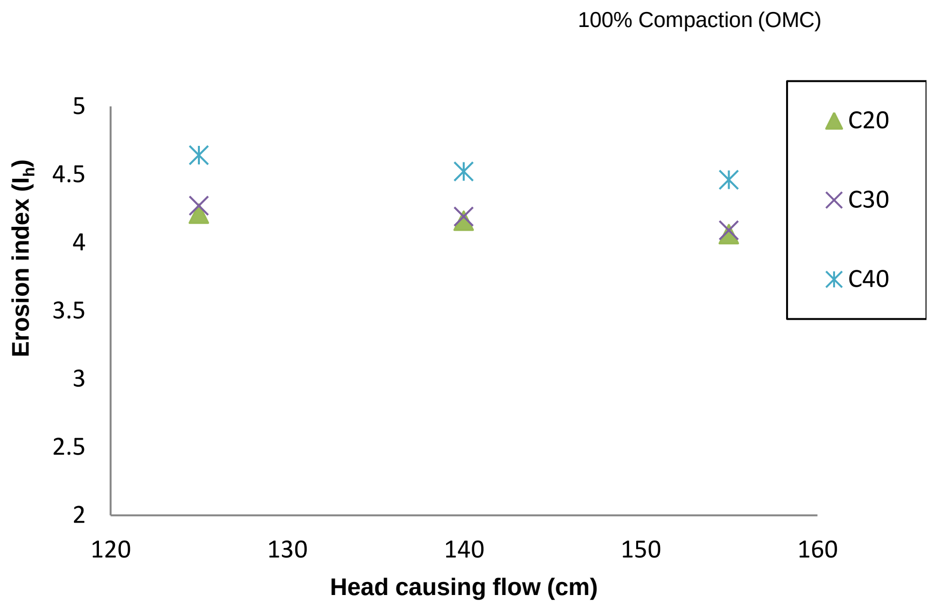


Figure 5.20. Erosion rate of C samples compacted to 100% relative density (Note: W denotes wash off occurred beyond this head)

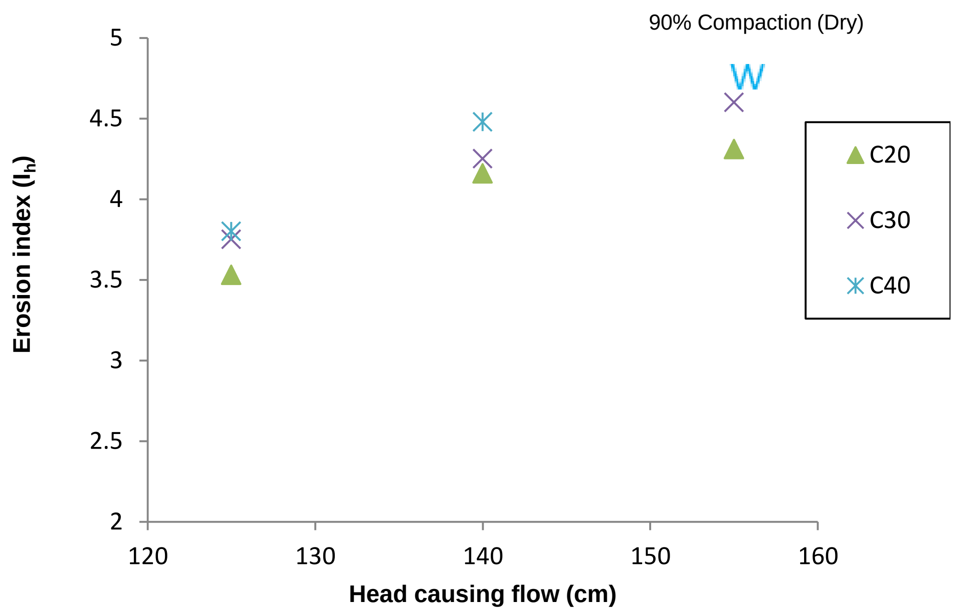


Figure 5.21. Erosion rate of C samples compacted to 90% relative density (Note: W denotes wash off occurred beyond this head)

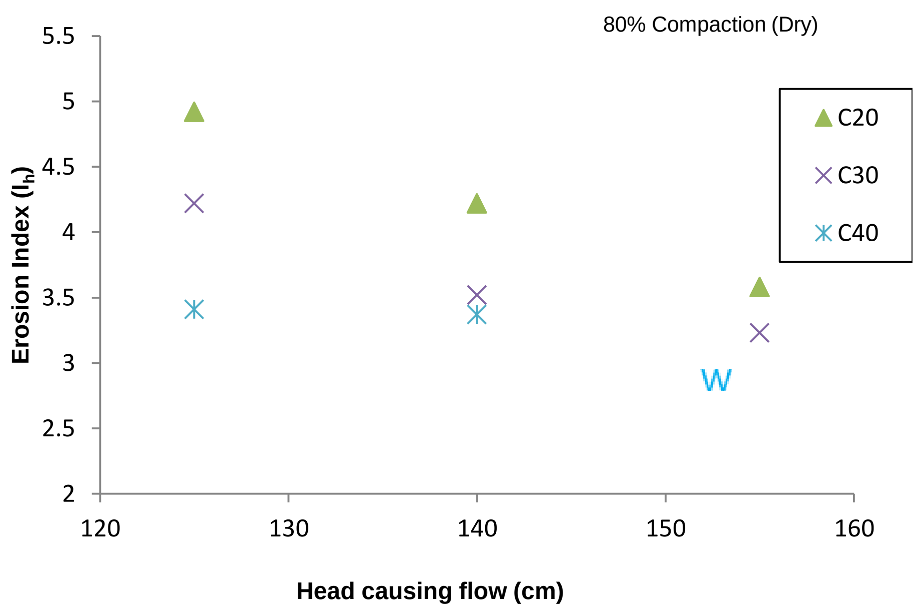


Figure 5.22. Erosion rate of C samples compacted to 80% relative density (Note: W denotes wash off occurred beyond this head)

From Figs.5.20 to 5.22, it can be observed that at 90% and 100% degrees of compaction, C40 sample (with about 60% fines and 40% sand fraction) shows more resistance to erosion. The larger sand particles help in better holding of fines that have some cohesion. When compacted to 80%, the moulding water content is very less and consequently the cohesion is less. At 90% compaction (dry side), the results are similar to the ones observed when the soil is compacted to maximum density at a moisture content corresponding to 50% of OMC. Due to particle dispersion at lower heads, a higher rate of erosion is observed. When the soil is compacted to 100% compaction density at OMC an increase in head causes an increase in erosion rate.

It can be inferred that all the samples tested show better stability against erosion when better compacted, especially at smaller heads.

5.1.4.2. M Samples

The influence of compaction on M samples was studied with M20, M30 and M40 samples. Soil samples were compacted to 80%, 90% and 100% of their maximum dry densities. The water contents were taken on the dry side of the corresponding light compaction curves. Specimens were tested at heads of 30cm, 40cm and 60cm. Table 5.14 shows the erosion rate index of M samples for different degrees of compaction.

At 80%, 90% and 100% degrees of compaction M20 sample (with about 80% fines+20% sand) is more erosion resistant (Figs.5.23 to 5.25). This is perhaps because of good gradation and higher cohesion between the soil particles.

Table 5.14.Erosion rate index of M Samples for different degrees of compaction

Degree of compaction	Erosion rate index (I_h)								
	100%			90%			80%		
Head(cm)	M20	M30	M40	M20	M30	M40	M20	M30	M40
30	3.6	3.52	3.51	3.13	3.02	2.99	3.15	2.98	2.85
40	3.52	3.52	3.47	3.1	3	2.96	3.09	2.85	2.81
60	3.07	2.99	2.96	2.77	2.77	2.65	2.79	2.73	2.73

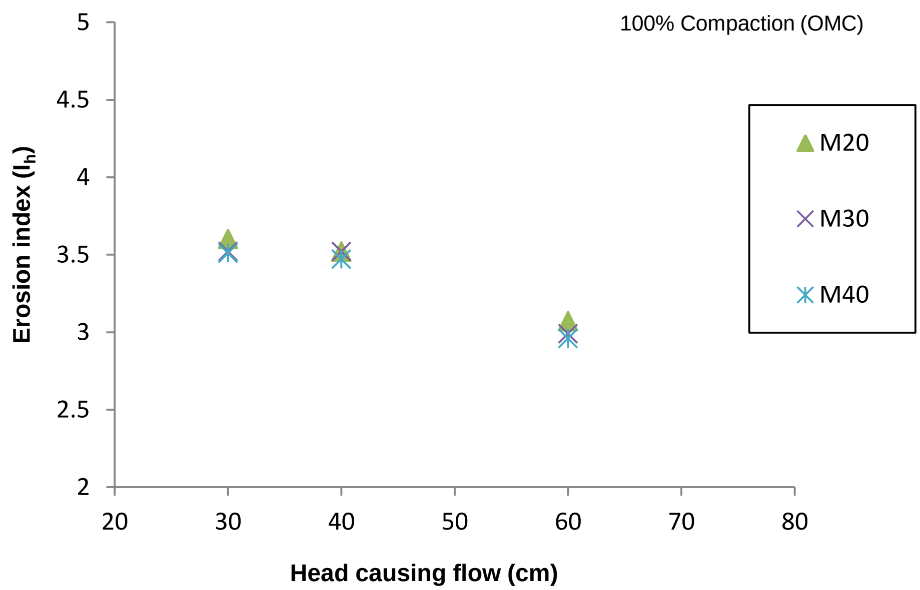


Figure 5.23. Erosion rate of M samples compacted to 100% relative density

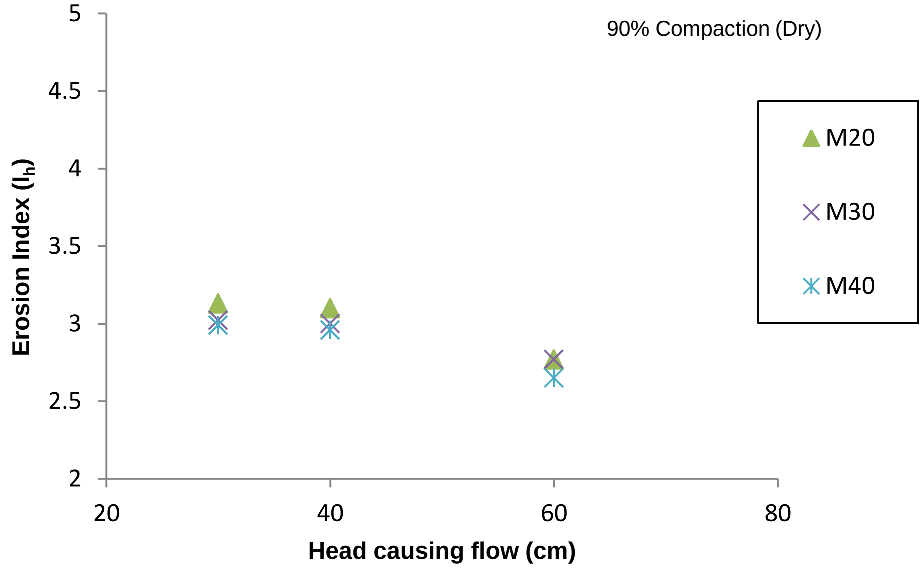


Figure 5.24. Erosion rate of M samples compacted to 90% relative density

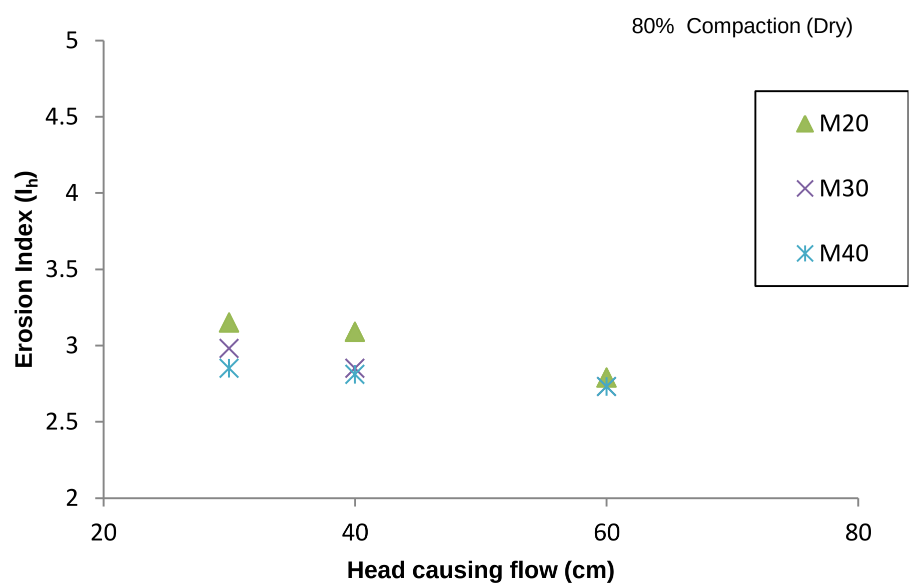


Figure 5.25. Erosion rate of M samples compacted to 80% relative density

Also the M40 samples (with about 60% fines + 40% sand) are least stable as the sand fraction is high, probably reduced cohesion, causing it to erode more. It can be observed that all the samples show maximum stability when compacted to its maximum dry density and have least stability when compacted to 80% of its maximum dry density.

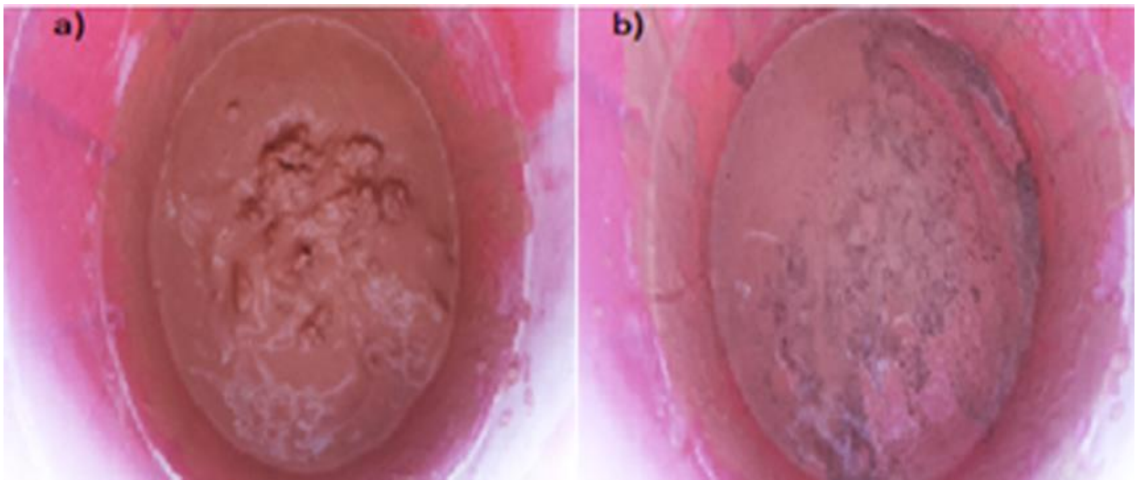


Figure 5.26. Settled soil particles in the overflowing container in case of a) wash off b) erosion

Figures 5.26 (a) and (b) shows the settled soil particles in the overflowing container in case of wash off and erosion respectively. Trials were attempted on samples compacted to 90% and 80% wet of optimum for both C and M samples. But due to the higher water content, the sample collapsed when the hole was drilled and placed horizontally for the experiment (Fig.5.27).

Figure 5.27. Sample which collapsed when compacted to 90% wet of optimum

5.1.5. Influence of change in initial hole diameter

To determine the influence of initial hole diameter in HET, C samples were compacted to their maximum dry densities at their corresponding optimum moisture contents. Holes of 8mm diameter were drilled in the specimen and the tests were conducted.

Table 5.15 Erosion rate index of C samples at OMC and maximum dry density condition conducted with an initial hole diameter of 8mm

Head(cm)	Erosion rate Index (I_h)(values in brackets are for 6mm hole diameter)				
	C0	C10	C20	C30	C40
110	3.78 (4.26)	4.16 (4.35)	4.52 (4.50)	4.67 (4.52)	4.68 (4.84)
125	3.72 (4.03)	3.91 (4.13)	4.26 (4.21)	4.26 (4.27)	4.61 (4.64)
140	3.64 (3.94)	3.83 (4.06)	4.02 (4.16)	4.03 (4.19)	4.56 (4.52)
155	3.59 (3.89)	3.64 (3.97)	3.85 (4.06)	3.91 (4.09)	4.32 (4.46)

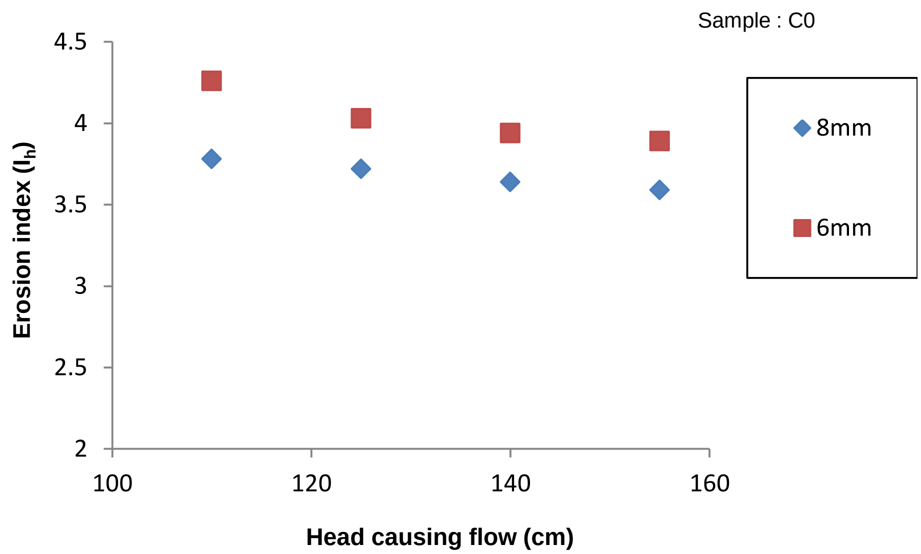


Figure 5.28(a). Comparison of Erosion rate index obtained for C0 sample with initial hole diameter 6mm and 8mm

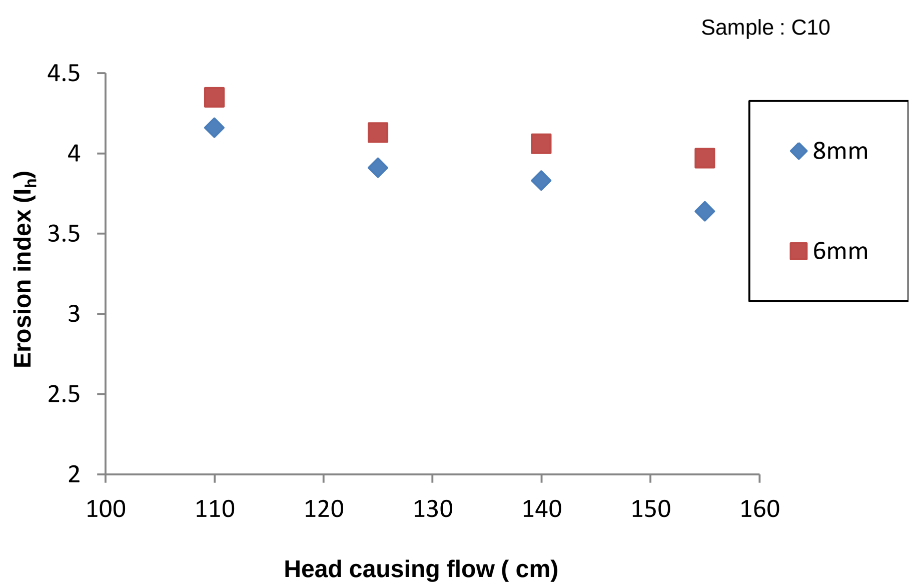


Figure 5.28(b). Comparison of Erosion rate index obtained for C10 sample with initial hole diameter 6mm and 8mm

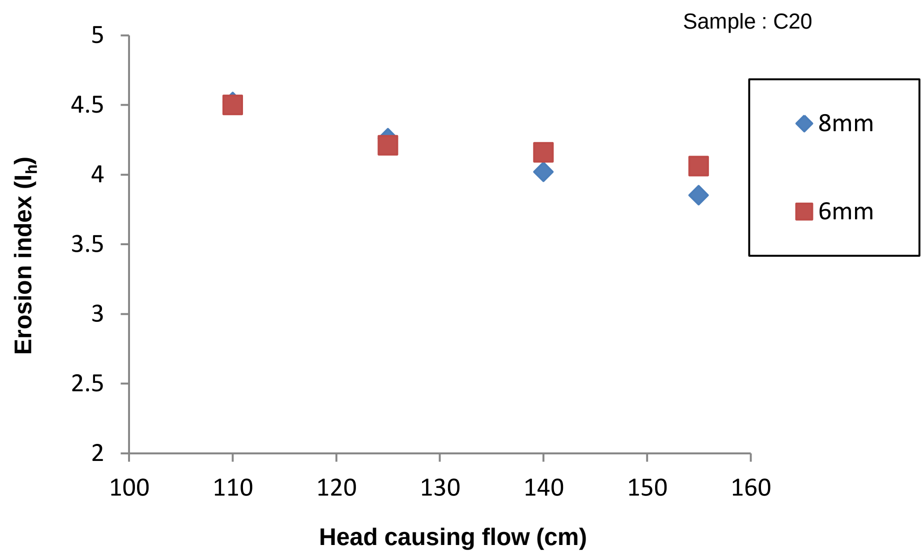


Figure 5.28(c). Comparison of Erosion rate index obtained for C20 sample with initial hole diameter 6mm and 8mm

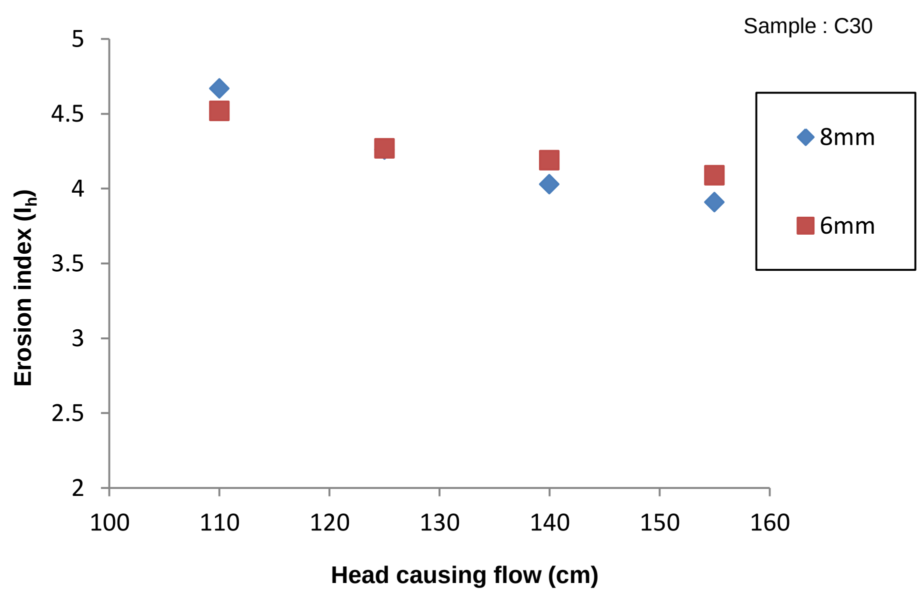


Figure 5.28(d). Comparison of Erosion rate index obtained for C30 sample with initial hole diameter 6mm and 8mm

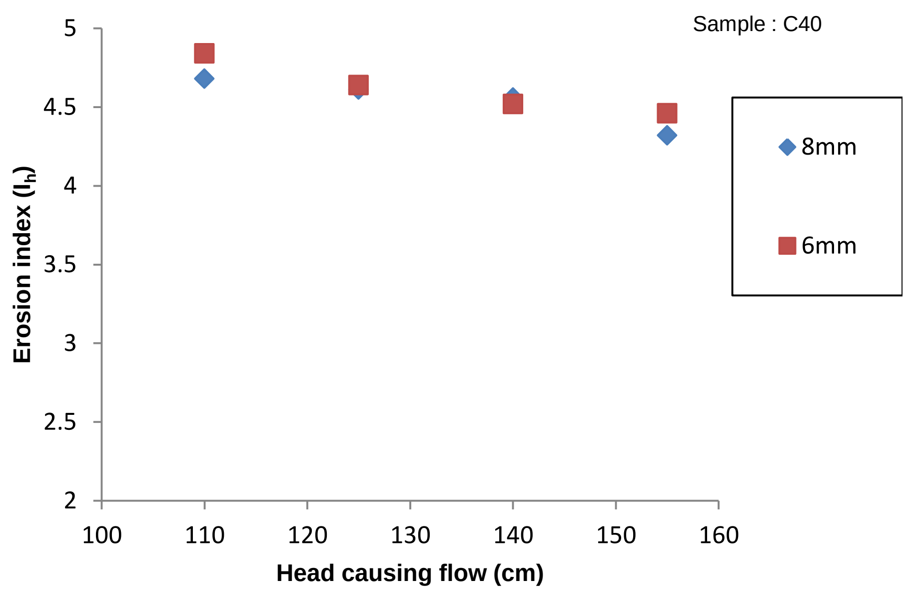


Figure 5.28(e). Comparison of Erosion rate index obtained for C40 sample with initial hole diameter 6mm and 8mm

The erosion rate index varies from 3.59 to 4.68 (Table 5.15) and can be classified under moderately rapid to moderately slow erosion, similar to as that of samples with 6mm diameter. Table 4 also shows values of erosion rate index (I_h) for 6mm diameter holes in brackets for comparison. This comparison is also shown in Figs.5.28 (a) to 5.28 (e).

It is observed that the initial hole diameter does not influence the erosion rate index significantly. Hence the initial diameter is not a very significant parameter affecting the test results and rate of erosion remains fairly constant for a given dry density, head causing the flow and water content, irrespective of hole diameter.

5.1.6. Critical shear stress

Critical shear stress is defined as the minimum stress to be applied on the soil surface to initiate progressive erosion. The average critical shear stresses obtained for different soil samples at various moulding water contents are shown in Figs.5.29 and 5.30. In general it is found that at optimum moisture content condition, the soil is most stable and shows a higher value of critical shear stress.

In the case of C samples, critical shear stress varies from 200N/m^2 to 400 N/m^2. At 110% OMC (i.e. full saturation condition), since C0 sample was most stable it has the highest value of critical shear stress. At OMC and 50% OMC conditions, C40 sample being more stable shows a higher value of critical shear stress.

In the case of M samples, critical shear stress varies from 45N/m^2 to 125N/m^2. At all the initial moulding water content, M0 sample shows a higher value of critical shear stress.

It can be clearly observed that soils with higher silt content (M samples) show a lower critical stress than those of soils with higher clay content (C samples).

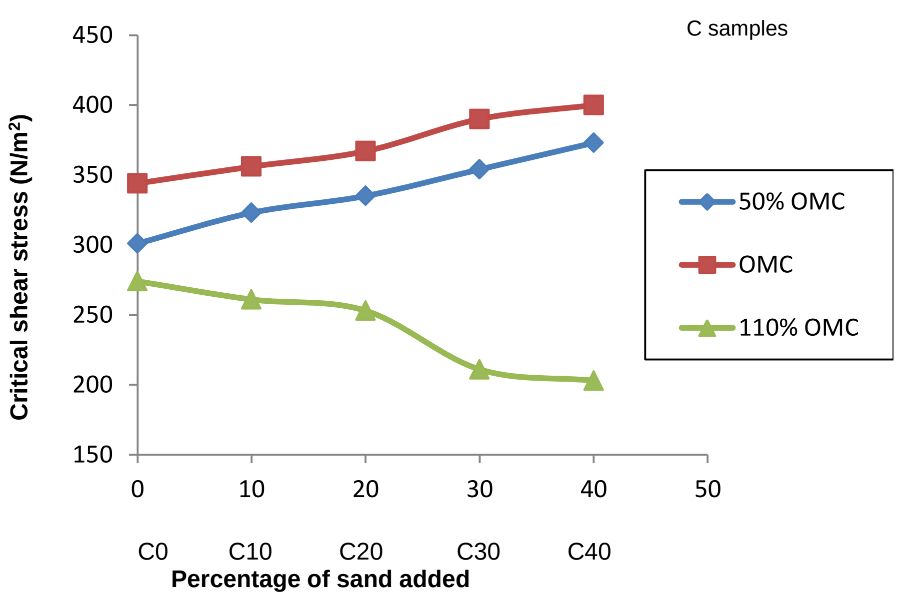


Figure 5.29. Critical shear stress variation of samples at different moulding water contents for C samples

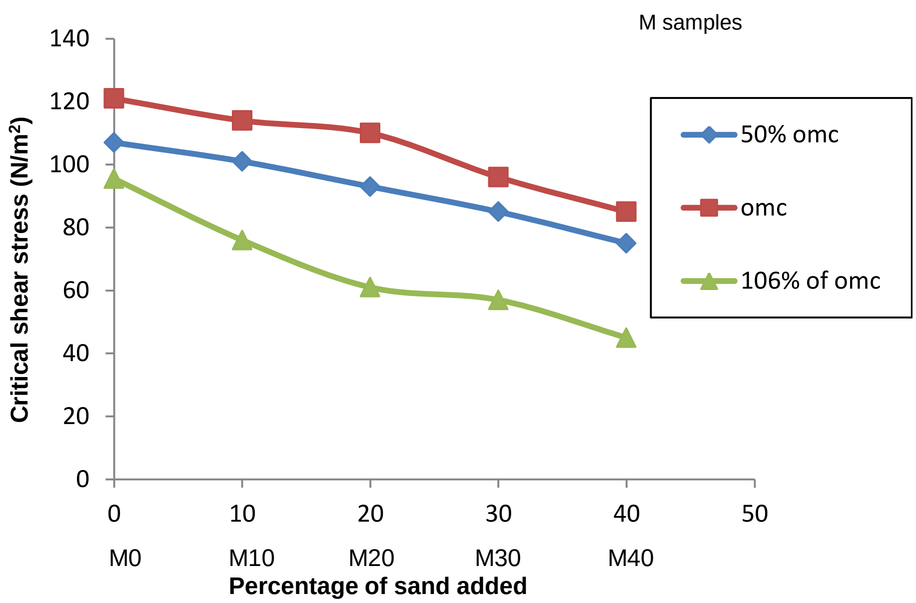


Figure 5.30. Critical shear stress variation of samples at different moulding water contents for M samples

5.1.7. Variation of erosion index with silt fraction

A comparison of erosion rate indices is studied for the M samples (having comparatively higher silt content) and C samples to understand the influence of silt content on erosion index values.

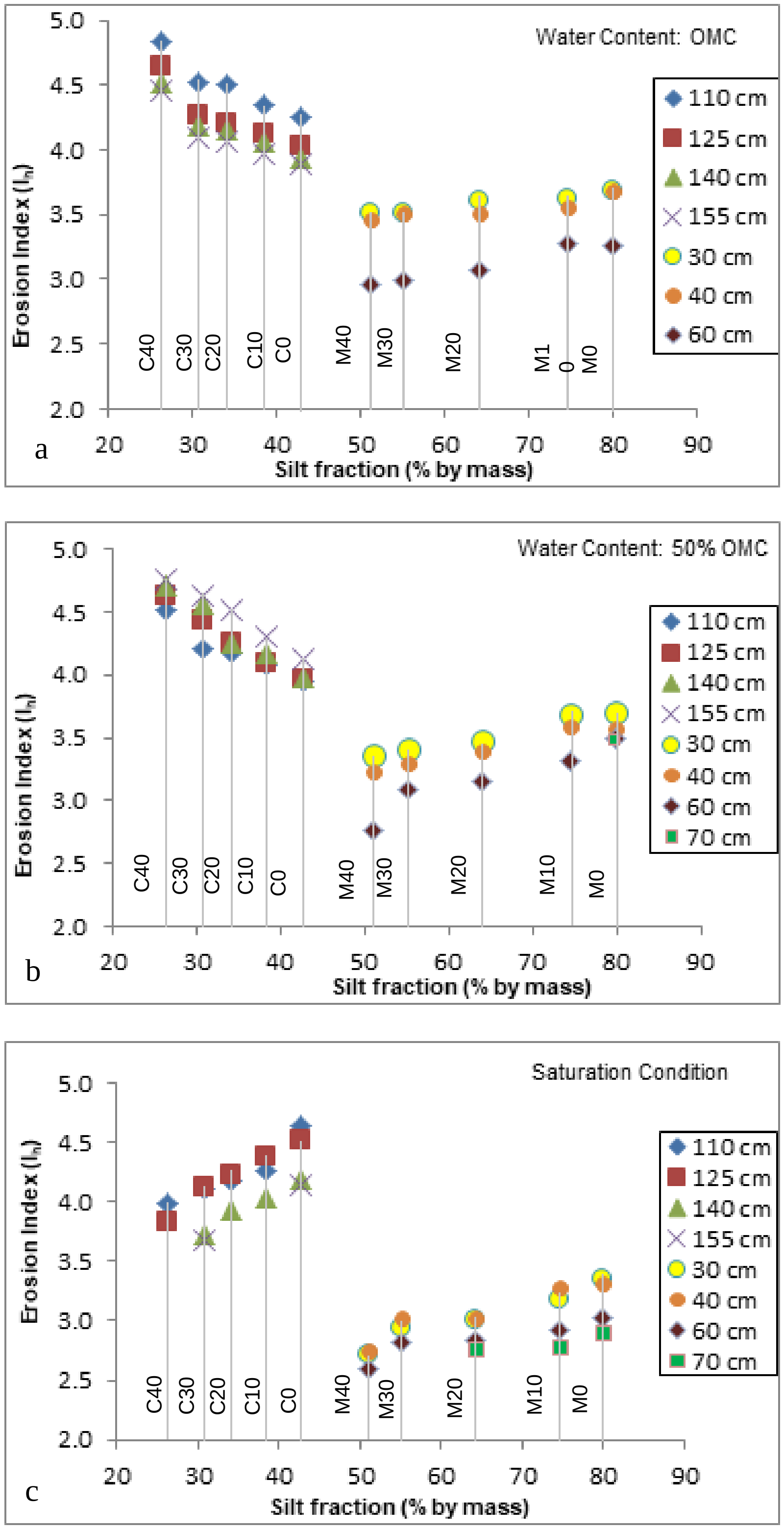
From Table 5.16 it is clear that the M samples have higher percentage of silt fraction (2μ to 75μ size) when compared to the C samples. Figures 5.31 (a) to (e) show the variation of erosion index with silt content for different conditions of water content and compaction.

Table 5.16 Percentage of silt for all the blended samples with varying percentage of sand added

Percentage of sand added to C and M samples*	Percentage of silt fraction (2μ to 75μ size)			
	C samples		M samples	
0#	C0	42.7	M0	79.9
10	C10	38.4	M10	74.5
20	C20	34.2	M20	64.1
30	C30	30.7	M30	55.1
40	C40	26.3	M40	51.0

Note: *C samples procured from source 1and M samples procured from source 2; # Natural procured sample)

It is observed that the C samples have lower erosion rate when compared to the M samples which have higher silt content. Since the M samples have higher silt content (silt fraction), the structure of soil mass is less stable. However, in the C samples, as silt content is less, the structure of soil mass is more stable with the fine clay particles filling the voids of the coarser particles leading to higher inter-particle shearing resistance and higher stability of the C samples.


Water Content: OMC
Erosion Index (I_h)
Silt fraction (% by mass)
110 cm
125 cm
140 cm
155 cm
30 cm
40 cm
60 cm
C40
C30
C20
C10
C0
M40
M30
M20
M10
M0
a
Water Content: 50% OMC
70 cm
b
Saturation Condition
c

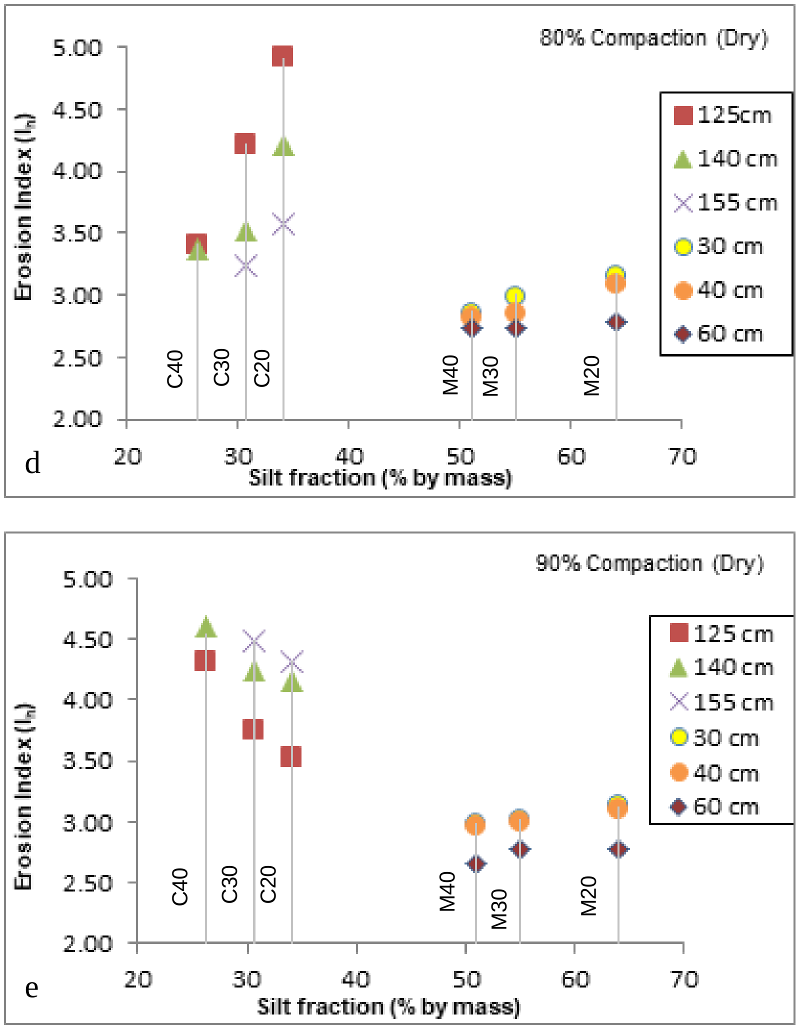


Figure 5.31. Variation of erosion index of samples with silt content for (a) OMC condition (b) 50% of OMC condition (c) Saturation condition (d) 80% compaction (e) 90% compaction

5.1.8. Variation of erosion index (I_h) with plasticity index (PI) at saturation

The variation of erosion index with plasticity index for all the samples prepared at saturation condition and at critical head is analyzed and is shown in Fig.5.32. In the field, after heavy rain, caving in is observed in the slopes in lateritic formations, with exposed lateritic soils (lateritic lithomarge or lithomargic laterites). The soil gets fully saturated and undergoes erosion. This was being simulated in the laboratory by preparing samples at fully saturated condition and then conducting the hole erosion tests. From Fig.5.32 it can be observed that erosion index (I_h) increases with increase in plasticity index (PI). This indicates that higher the plasticity index higher will be the erosion resistance of the soil. Critical head indicates the minimum head (and thereby representing critical shear stress) at which the soil sample in laboratory will undergo progressive erosion in a hole erosion test.

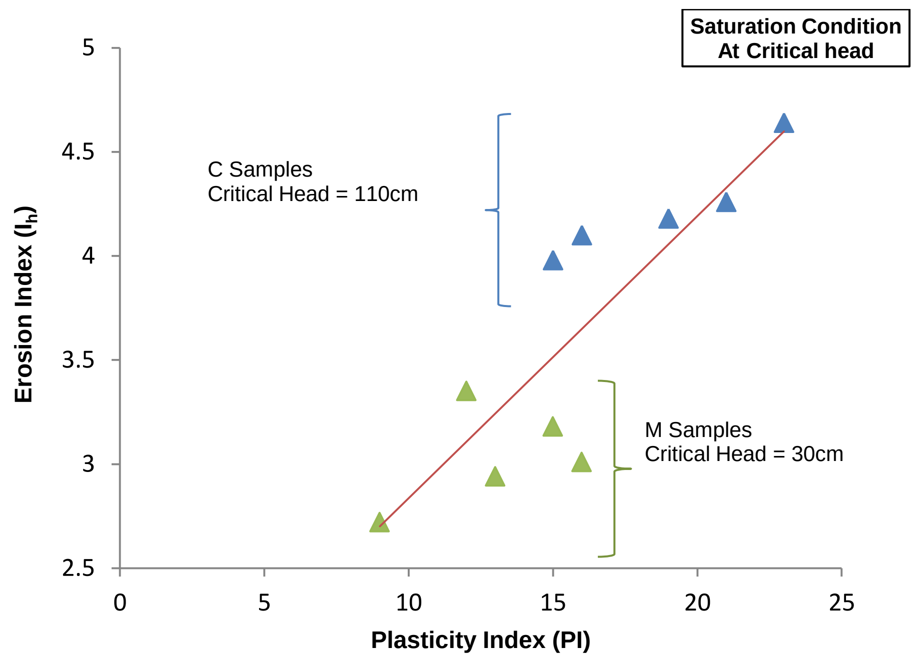


Figure 5.32. Variation of erosion index (I_h) with plasticity index (PI) at critical head (samples prepared at saturation)

5.2. SLOPE STABILITY ANALYSIS RESULTS

5.2.1. General

For slope stability analysis, 3 different heights of slope with different slope angles are considered. We have considered 5m, 8m, 11m slope with 30, 40, 45 and 60 degree slope angle. For 11m height slope berm is provided at 5.5m. In case of 8m slope, two cases are considered, first case berm at 4m, and second berm at 5.5m and then factor of safety (FOS) is compared for all the above cases. In 8m and 11m slope, effect of top soil (i.e.

laterite) excavation, ponding effect is considered and the effect of this over FOS of slope is studied. In the case of 5m slope, only lithomargic clay layer of 5m is considered without the excavation and ponding effect. In all the slopes effect of vegetation and effect of trees, wind load on trees over the slope is analysed. The stability of slope in both dry (drained condition) and saturated condition (undrained condition) are studied.

5.2.2. Drained Condition

5.2.2.1. 5m height slope

In 5m slope, lithomargic clay layer of 5m height is considered without the excavation and ponding effect.

Table 5.17 Factor of safety due to tree position over slope of 5m height

30 Degree 5m slope				
Factor of safety	**Toe***	**Top***	**Toe+Top***	**Without tree**
With WL	3.36	2.95	3.2	3.1
Without WL	3.36	2.88	3.04	3.1

Table 5.18 Factor of safety due to tree position over slope with turfing

30 Degree 5m slope with Turfing and trees				
Factor of safety	**Toe*+Turfing**	**Top*+Turfing**	**(Toe+Top)*+Turfing**	**Turfing only**
With WL	3.65	3.2	3.48	3.45
Without WL	3.65	2.9	3.4	3.45

Table 5.19 Factor of safety due to tree position over slope

40 Degree 5m slope				
Factor of safety	**Toe***	**Top***	**Toe+Top***	**Without tree**
With WL	3	2.52	2.83	2.68
Without WL	3	2.43	2.73	2.68

Table 5.20 Factor of safety due to tree position over slope with turfing

40 Degree 5m slope with Turfing and trees				
Factor of safety	**Toe*+Turfing**	**Top*+Turfing**	**(Toe+Top)*+Turfing**	**Turfing only**
With WL	3.11	2.92	3.22	3.16
Without WL	3.11	2.83	3.15	3.16

Table 5.21 Variation of factor of safety due to tree position over slope

45 Degree 5m slope				
Factor of safety	**Toe***	Top*****	Toe+Top*	**Without** tree
With WL	2.87	2.35	2.63	2.5
Without WL	2.87	2.26	2.55	2.5

Table 5.22 Variation of factor of safety due to tree position over slope with turfing

45 Degree 5m slope with Turfing and trees				
Factor of safety	**Toe*+Turfing**	**Top*+Turfing**	**Toe+Top*+Turfing**	**Turfing only**
With WL	3.22	2.73	3.08	2.93
Without WL	3.22	2.64	3.04	2.93

In this case (Table 5.17 to 5.22), it can be observed that wind load have very little effect on factor of safety (Fig.5.33 (a) to (c)). Tree near the toe have no effect of wind load because the soil is subjected to very less disturbance. If the tree is placed in middle or top position of the slope, the factor of safety decreases and when wind effect is considered the factor of safety still reduces. Due to the effect of wind, for tree position at middle and top, the disturbance in soil is high compared to the tree at toe. Roots reinforce the soil through growing across failure planes, root columns acting as piles, and through limiting surface erosion. Wind-throw is a factor when only one tree is considered alone. However it is of lesser importance when considering general slope stability for a body of trees as the wind forces involved represents a smaller percentage of the potential disturbing forces (0 to 3% decrease in FOS). The trees which are in the centre of the group will be sheltered by those on the outside because the tree is not taking full wind force when compared to the force faced by the first tree.

5m Height, 30 Degree

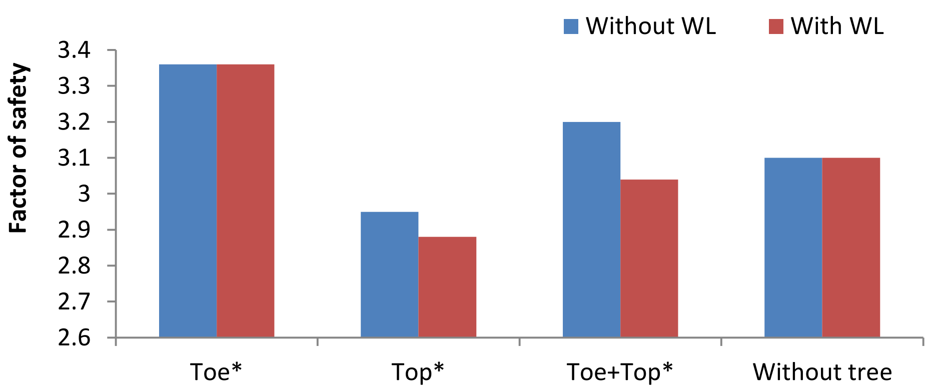


Figure 5.33(a).Variation of factor of safety due to wind load for 5m height 30 degree slope (*- Tree position, WL – wind load)

5m Height, 40 Degree

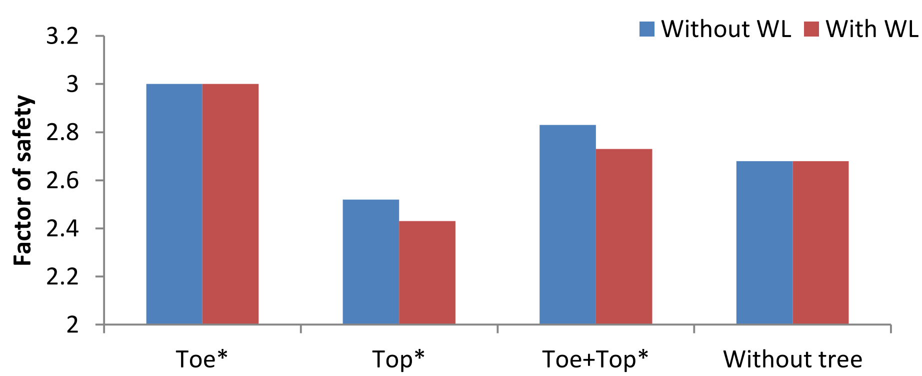


Figure 5.33(b).Variation of factor of safety due to wind load for 5m height 40 degree slope (*- Tree position, WL – wind load)

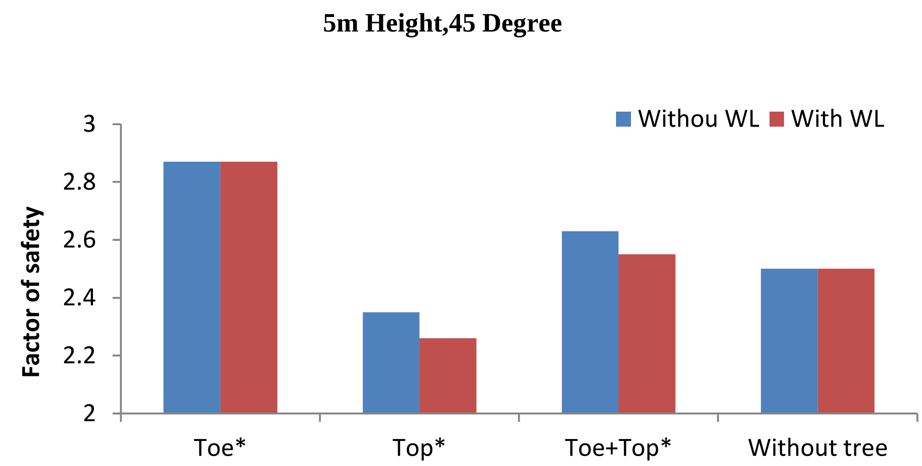


Figure 5.33(c).Variation of factor of safety due to wind load for 5m height 45 degree slope (*- Tree position, WL – wind load)

5.2.2.2. Variation of Factor of Safety With Respect To Slope Angle

In this case (Table 5.23 to 5.28), it can be observed that, as the slope angle increases the effect of tree at the toe of slope is more i.e. percentage increase in FOS when compared with bare slope is increasing. This is because the root penetration in resisting zone is more in case of steep slopes. The variation in factor of safety with respect to slope angle is shown Fig.5.34 (a) to (f).

Table 5.23 Factor of safety with respect to slope angle for 8m slope with berm at 4m

Slope Angle	Toe*	Mid*	Top*	Toe+Mid+Top*	Toe+Mid*	Toe+Top*	Mid+Top*	Without tree
30^o	2.78	2.67	2.6	2.72	2.77	2.72	2.6	2.67
40^o	2.46	2.32	2.25	2.4	2.46	2.4	2.18	2.33
45^o	2.35	2.21	2.18	2.29	2.35	2.29	2.14	2.22
60^o	1.97	1.87	1.84	1.95	1.97	1.93	1.84	1.89

Table 5.24 Factor of safety with respect to slope angle for 8m slope with berm at 4m with turfing

Slope Angle	Toe*+Turfing	Mid*+turfing	Top*+Turfing	Toe+Mid+Top*+Turfing	Toe+Mid*+Turfing	Toe+Top*+Turfing	Mid+Top*+Turfing	Turfing only
30º	2.9	2.76	2.71	2.84	2.9	2.84	2.68	2.79
40º	2.53	2.39	2.33	2.47	2.52	2.49	2.33	2.4
45º	2.44	2.25	2.25	2.37	2.4	2.38	2.26	2.31
60º	2.17	2.02	1.99	2.12	2.17	2.17	1.99	2.03

Table 5.25 Factor of safety with respect to slope angle for 8m slope with berm at 5.5m

Slope Angle	Toe*	Mid*	Top*	Toe+Mid+Top*	Toe+Mid*	Toe+Top*	Mid+Top*	Without tree
30º	2.71	2.63	2.56	2.65	2.71	2.65	2.54	2.65
40º	2.41	2.22	2.2	2.34	2.38	2.35	2.18	2.28
45º	2.27	2.08	2.08	2.19	2.21	2.2	2.07	2.12
60º	1.96	1.72	1.72	1.93	1.96	1.92	1.73	1.74

Table 5.26 Factor of safety with respect to slope angle for 8m slope with berm at 5.5m with turfing

Slope Angle	Toe*+Turfing	Mid*+turfing	Top*+Turfing	Toe+Mid+Top*+Turfing	Toe+Mid*+Turfing	Toe+Top*+Turfing	Mid+Top*+Turfing	Turfing only
30º	2.83	2.73	2.68	2.77	2.82	2.74	2.66	2.71
40º	2.52	2.38	2.32	2.46	2.52	2.49	2.31	2.38
45º	2.35	2.12	2.2	2.26	2.33	2.36	2.18	2.22

60º	2	1.85	1.86	1.96	1.96	1.96	1.8	1.89

Table 5.27 Factor of safety with respect to slope angle for 11m slope with berm at 5.5m

Slope Angle	Toe*	Mid*	Top*	Toe+Mid +Top*	Toe+Mid*	Toe+Top*	Mid+Top*	Without tree
30º	2.28	2.21	2.17	2.24	2.28	2.25	2.18	2.21
40º	1.97	1.86	1.83	1.91	1.92	1.92	1.83	1.87
45º	1.856	1.73	1.71	1.82	1.854	1.83	1.71	1.73
60º	1.92	1.81	1.81	1.89	1.87	1.89	1.79	1.81

Table 5.28 Factor of safety with respect to slope angle for 11m slope with berm at 5.5m with turfing

Slope Angle	Toe* +Turfing	Mid* +turfing	Top* +Turfing	Toe+Mid +Top*+Turfing	Toe+Mid*+Turfing	Toe+Top*+Turfing	Mid+Top*+Turfing	Turfing only
30º	2.33	2.26	2.24	2.33	2.36	2.33	2.24	2.26
40º	2.05	1.94	1.91	1.95	1.96	2.03	1.91	1.93
45º	1.93	1.77	1.76	1.9	1.91	1.9	1.77	1.79
60º	2.08	1.97	1.94	2.05	2.07	2.06	1.94	1.97

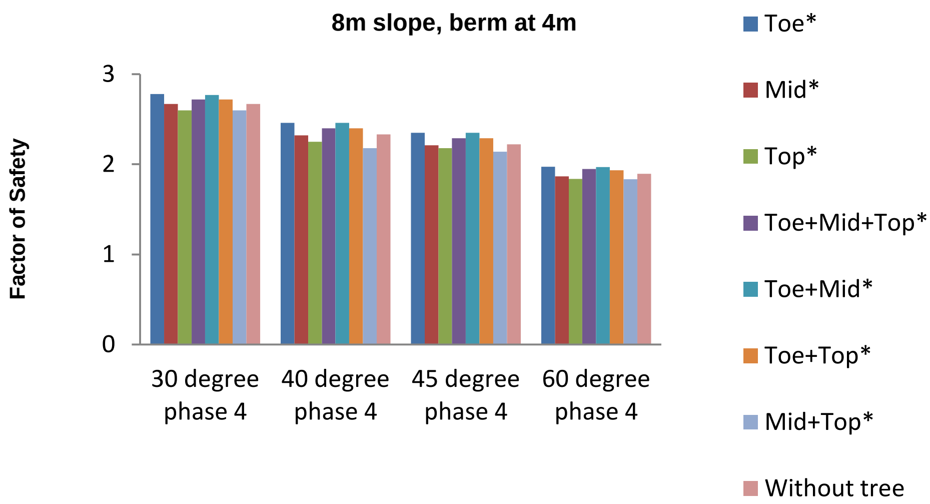


Figure 5.34(a). Variation of factor of safety with respect to slope angle for 8m slope with berm at 4m (*Tree position)

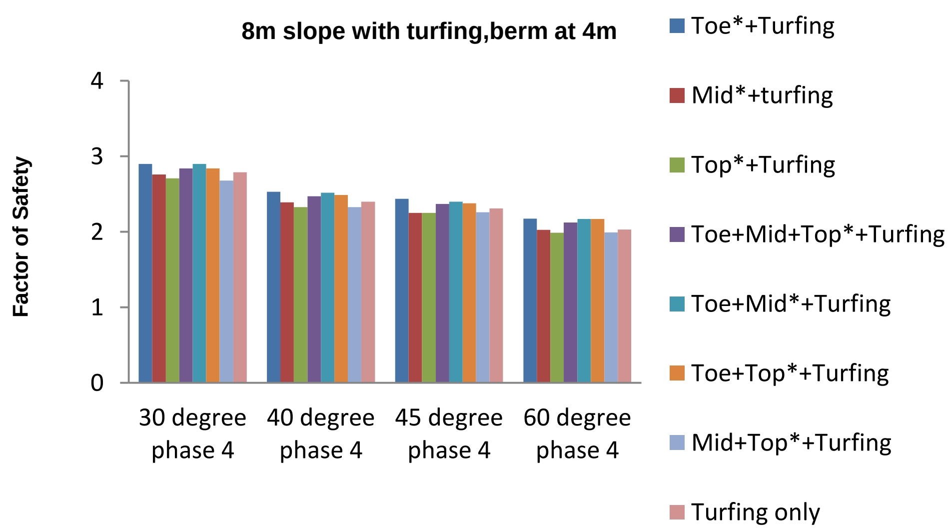


Figure 5.34(b). Variation of factor of safety with respect to slope angle for 8m slope with berm at 4m with turfing (*Tree position)

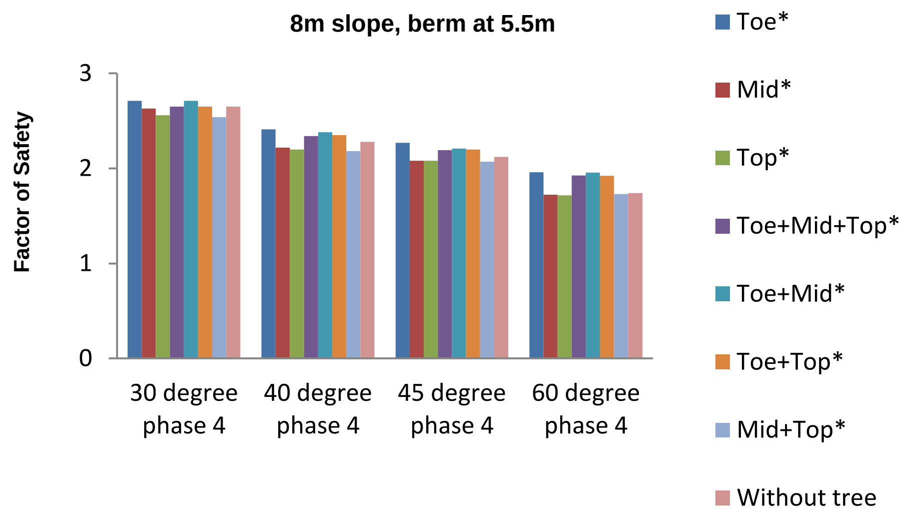


Figure 5.34(c). Variation of factor of safety with respect to slope angle for 8m slope with berm at 5.5m (*Tree position)

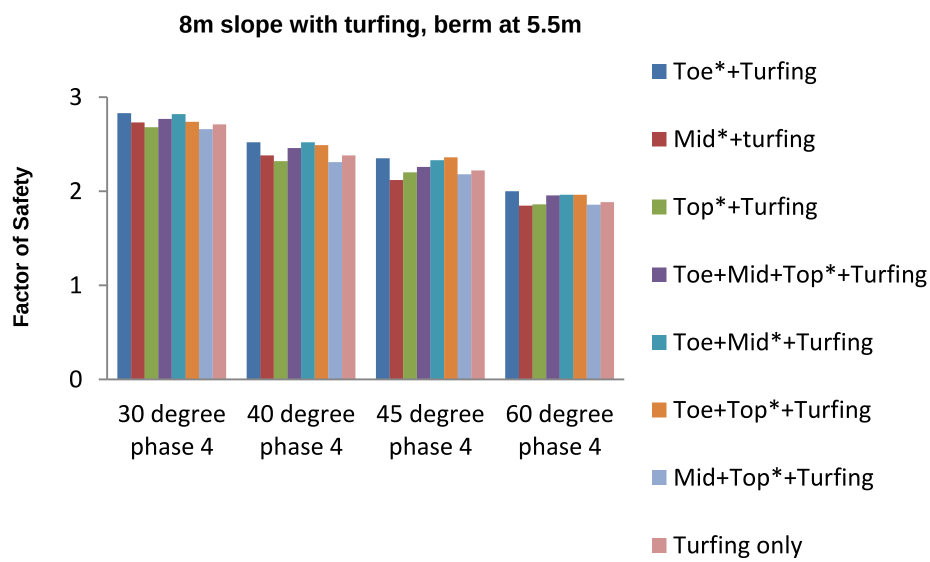


Figure 5.34(d). Variation of factor of safety with respect to slope angle for 8m slope with berm at 5.5m with turfing (*Tree position)

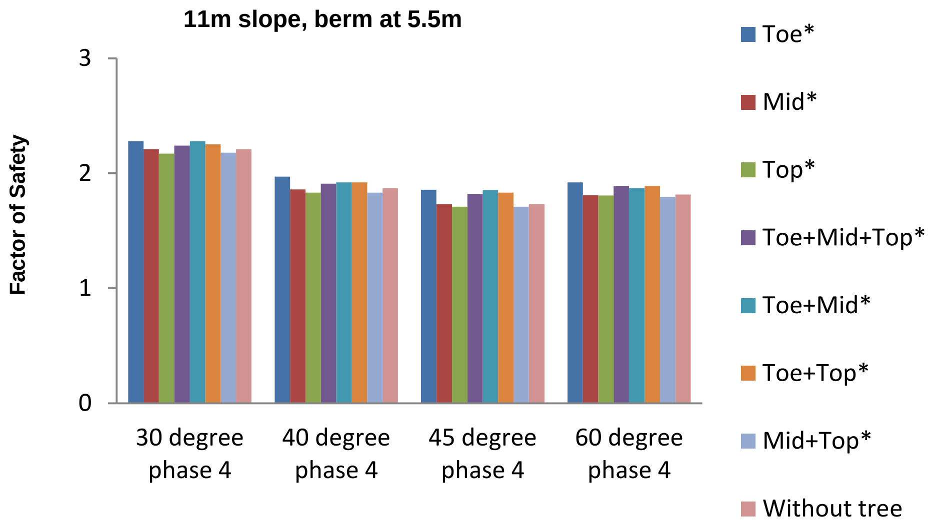


Figure 5.34(e). Variation of factor of safety with respect to slope angle for 11m slope with berm at 5.5m (*Tree position)

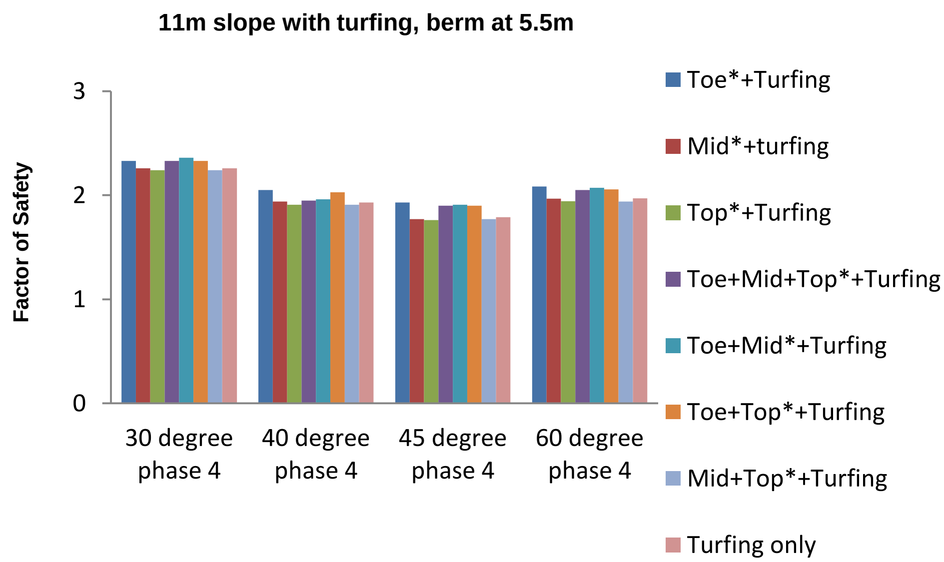


Figure 5.34(f). Variation of factor of safety with respect to slope angle for 11m slope with berm at 5.5m with turfing (*Tree position)

5.2.2.3. Percentage variation in factor of safety with respect to tree position

Figures 5.35 (a) to (c) shows the percentage variation in factor of safety with respect to tree position (Table 5.29 to 5.31) when compared with bare slope. An increasing FOS up to 12% is observed if tree is present at the toe of slope due to reinforcing action of the roots. The excavation in the top does not have much effect on the factor of safety since the excavated soil part lies outside the failure slip circle. When the tree is placed near the mid and top a decreasing factor of safety is observed. This is due to the fact that tree weight contributes to the driving force leading to decrease in FOS. When you consider tree position at Toe+Mid+Top, Toe+Mid, Toe+Top the factor of safety increases. The factor of safety is decreasing up to 6% when trees are provided at Mid+Top position.

Table 5.29 Percentage variation in factor of safety with respect to tree position for 8m slope with berm at 4m

Slope Angle	Toe*	Mid*	Top*	Toe+Mid +Top*	Toe+Mi d*	Toe+To p*	Mid+T op*
30º	4.12	0	-2.62	1.87	3.75	1.87	-2.62
40º	5.58	-0.43	-3.43	3.01	5.58	3.01	-6.44
45º	5.86	-0.45	-1.80	3.15	5.86	3.15	-3.60
60º	4.07	-1.43	-3.06	2.85	3.91	2.06	-3.12

Table 5.30 Percentage variation in factor of safety with respect to tree position for 8m slope with berm at 5.5m

Slope Angle	Toe*	Mid*	Top*	Toe+Mid +Top*	Toe+Mi d*	Toe+To p*	Mid+T op*
30º	2.27	-0.76	-3.39	0	2.27	0	-4.15
40º	5.70	-2.63	-3.51	2.63	4.39	3.07	-4.39
45º	7.08	-1.89	-1.89	3.30	4.25	3.77	-2.36

60°	12.64	-0.92	-1.44	10.69	12.41	10.52	-0.58

Table 5.31 Percentage variation in factor of safety with respect to tree position for 11m slope with berm at 5.5m

Slope Angle	Toe*	Mid*	Top*	Toe+Mid +Top*	Toe+Mid*	Toe+Top*	Mid+Top*
30°	3.168	0	-1.81	1.36	3.17	1.81	-1.36
40°	5.35	-0.54	-2.14	2.14	2.67	2.67	-2.14
45°	7.28	0	-1.16	5.20	7.17	5.78	-1.16
60°	5.79	-0.33	-0.44	4.08	3.03	4.19	-1.16

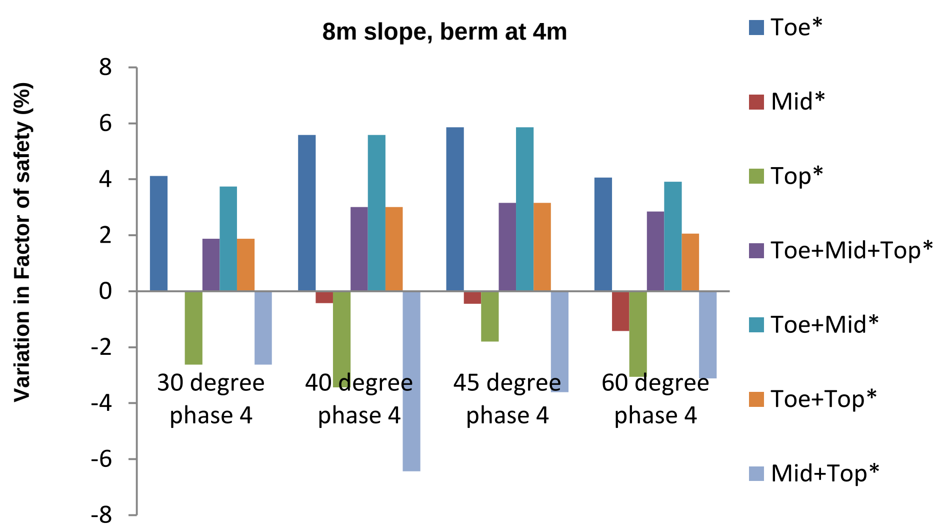


Figure 5.35(a).

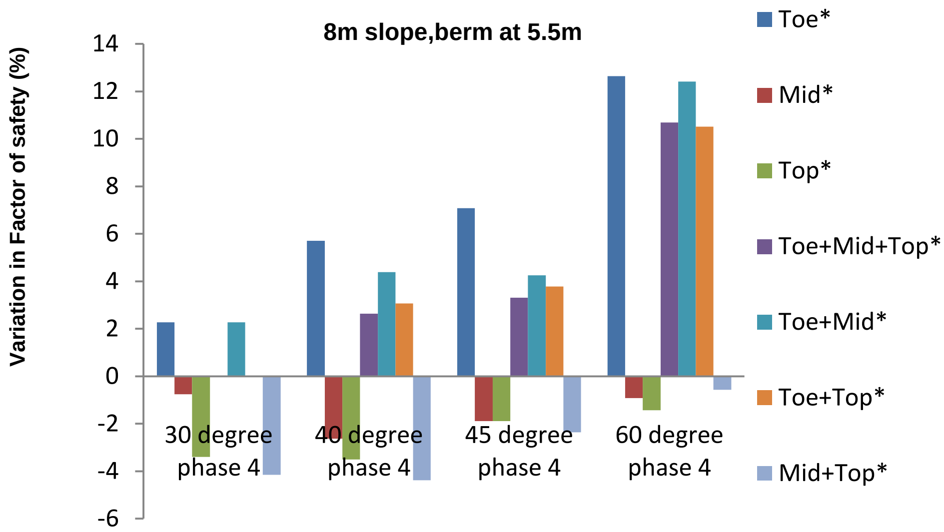


Figure 5.35(b). Percentage variation in factor of safety with respect to tree position for 8m slope with berm at 5.5m (*Tree position)

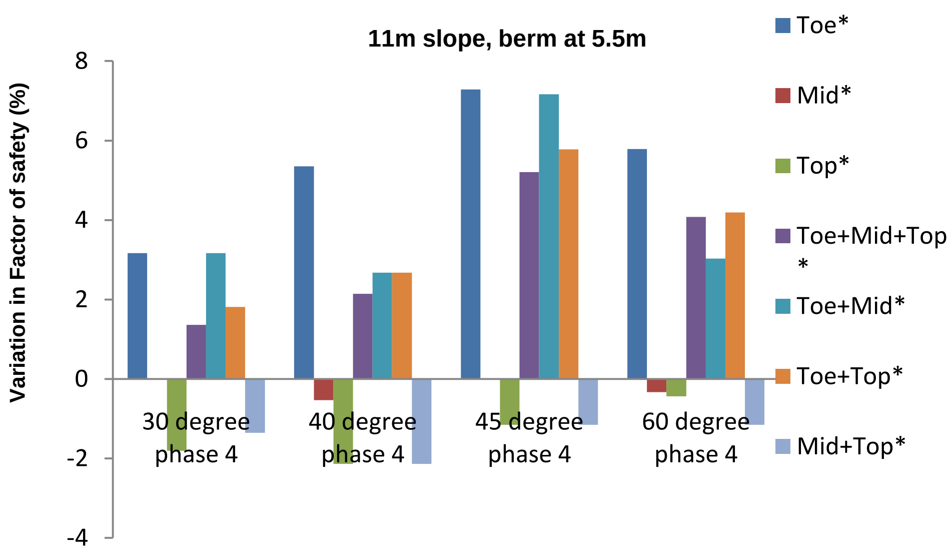


Figure 5.35(c). Percentage variation in factor of safety with respect to tree position for 11m slope with berm at 5.5m (*Tree position)

5.2.2.4. Percentage variation in factor of safety with respect to turfing

Figures 5.36 (a) to (c) shows the percentage variation in factor of safety with respect to turfing (Table 5.32 to 5.34) when compared with bare slope. A percentage increase in factor of safety up to 15% is observed due to turfing along with trees.

Table 5.32 Percentage variation in factor of safety with respect to turfing for 8m slope with berm at 4m

Slope Angle	Toe*+Turfing	Mid*+turfing	Top*+Turfing	Toe+Mid+Top*+Turfing	Toe+Mid*+Turfing	Toe+Top*+Turfing	Mid+Top*+Turfing	Turfing only
30º	8.62	3.37	1.50	6.37	8.62	6.37	0.38	4.49
40º	8.58	2.58	0	6.01	8.16	6.87	0	3.01
45º	9.91	1.35	1.352	6.76	8.11	7.21	1.80	4.056
60º	14.78	7.08	5.07	12.14	14.63	14.68	5.28	7.18

Table 5.33 Percentage variation in factor of safety with respect to turfing for 8m slope with berm at 5.5m

Slope Angle	Toe*+Turfing	Mid*+turfing	Top*+Turfing	Toe+Mid+Top*+Turfing	Toe+Mid*+Turfing	Toe+Top*+Turfing	Mid+Top*+Turfing	Turfing only
30º	6.79	3.02	1.13	4.53	6.42	3.39	0.38	2.27
40º	10.52	4.39	1.76	7.89	10.53	9.21	1.32	4.39
45º	10.85	0	3.77	6.60	9.91	11.32	2.83	4.72
60º	14.94	6.265	6.89	12.41	12.82	12.82	6.67	8.39

Table 5.34 Percentage variation in factor of safety with respect to turfing for 11m slope with berm at 5.5m

Slope Angle	Toe*+Turfing	Mid*+turfing	Top*+Turfing	Toe+Mid+Top*+Turfing	Toe+Mid*+Turfing	Toe+Top*+Turfing	Mid+Top*+Turfing	Turfing only
30^{o}	5.43	2.26	1.36	5.43	6.79	5.43	1.36	2.26
40^{o}	9.63	3.74	2.14	4.28	4.81	8.56	2.14	3.21
45^{o}	11.56	2.31	1.74	9.83	10.41	9.83	2.31	3.47
60^{o}	14.77	8.43	7.05	12.89	14.22	13.28	6.83	8.54

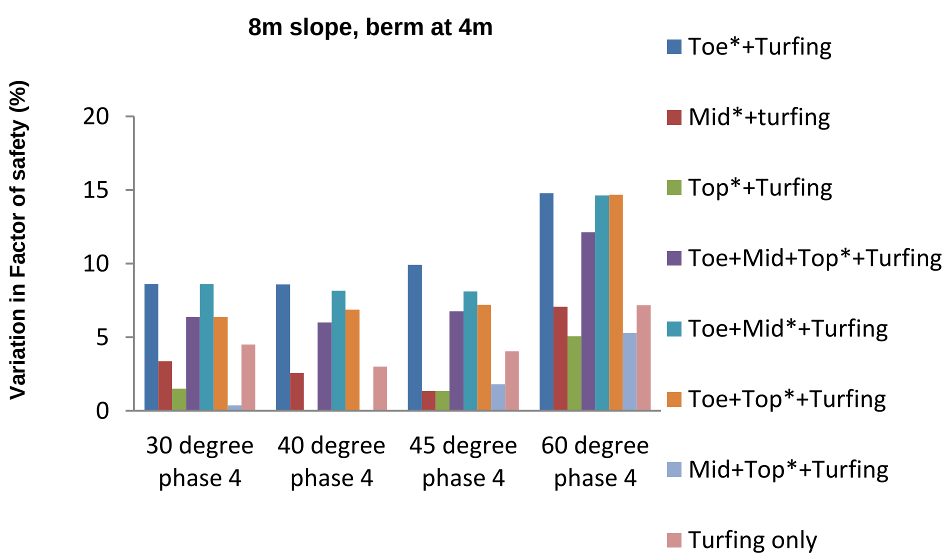


Figure 5.36(a). Percentage variation in factor of safety with respect to turfing for 8m slope with berm at 4m (*Tree position)

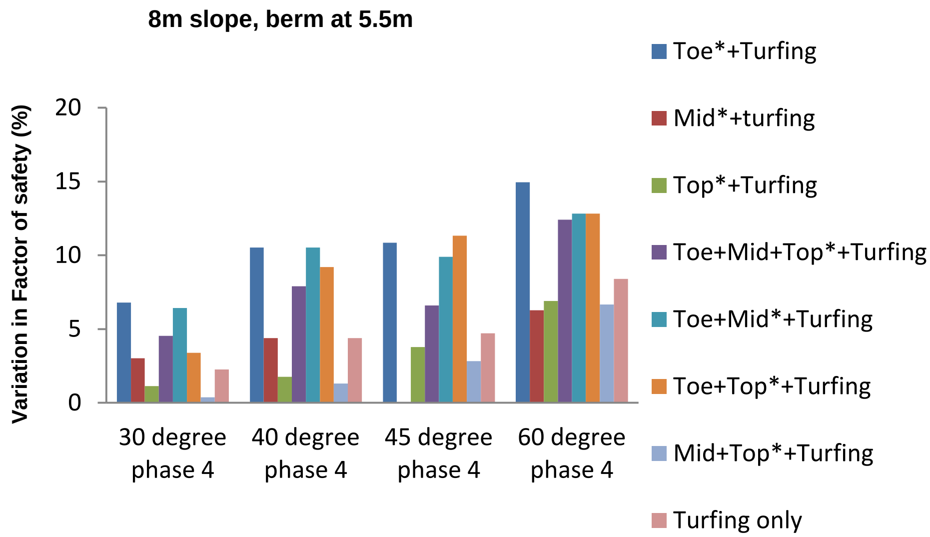


Figure 5.36(b). Percentage variation in factor of safety with respect to turfing for 8m slope with berm at 5.5m (*Tree position)

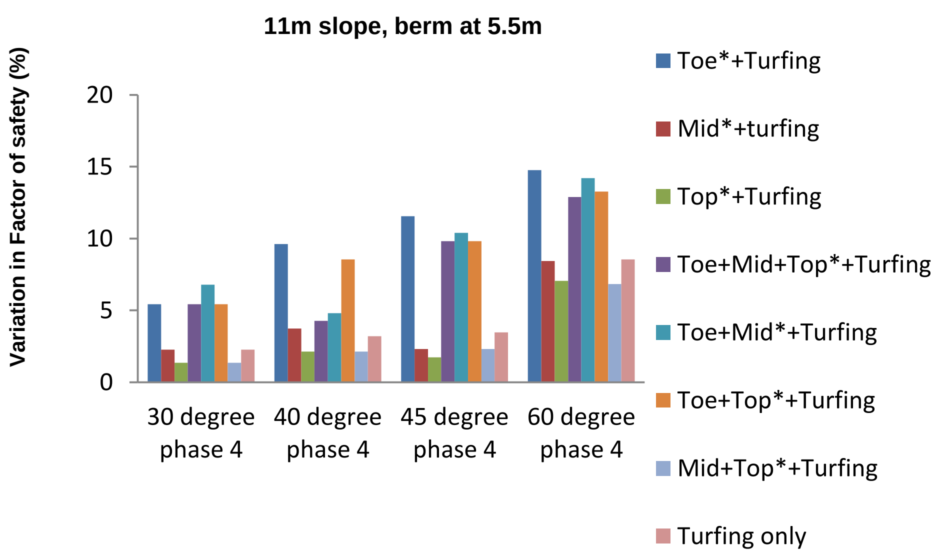


Figure 5.36(c). Percentage variation in factor of safety with respect to turfing for 11m slope with berm at 5.5m (*Tree position)

5.2.2.5. Variation in factor of safety with respect to berm position

By providing berm at different positions for 8m slope, it can be observed that the berm position also affects the factor of safety. When berm is provided at 4m height, an increase in FOS up to 8% is observed when trees are only provided whereas in the case of trees along with turfing there is an increase of FOS up to 10%. The factor of safety is more if we provide berm at mid rather than providing at 3/4th height.

5.2.3. Undrained Condition

Slopes with cut slope angle of 60 degrees failed under undrained conditions for 11m height slope.

5.2.3.1. Variation of Factor of Safety With Respect To Slope Angle

In this case (Table 5.35 to 5.40), it can be observed that, As the slope angle increases the effect of tree at the toe of slope is more i.e., percentage increase in FOS when compared with bare slope is increasing. The variation in factor of safety with respect to slope angle is shown Fig.5.37 (a) to (f).

Table 5.35 Factor of safety with respect to slope angle for 8m slope with berm at 4m

Slope Angle	Toe*	Mid*	Top*	Toe+Mid+Top*	Toe+Mid*	Toe+Top*	Mid+Top*	Without tree
30º	2.68	2.58	2.52	2.6	2.65	2.57	2.5	2.58
40º	2.36	2.22	2.2	2.32	2.35	2.34	2.19	2.27
45º	2.28	2.12	2.1	2.21	2.28	2.26	2.12	2.16
60º	1.35	1.30	1.28	1.32	1.35	1.33	1.27	1.31

Table 5.36 Factor of safety with respect to slope angle for 8m slope with berm at 4m with turfing

Slope Angle	Toe*+Turfing	Mid*+turfing	Top*+Turfing	Toe+Mid+Top*+Turfing	Toe+Mid*+Turfing	Toe+Top*+Turfing	Mid+Top*+Turfing	Turfing only
30º	2.77	2.67	2.6	2.68	2.74	2.67	2.58	2.64
40º	2.45	2.29	2.27	2.4	2.42	2.41	2.39	2.35
45º	2.4	2.2	2.18	2.32	2.38	2.32	2.15	2.21
60º	1.46	1.42	1.38	1.43	1.45	1.43	1.38	1.43

Table 5.37 Factor of safety with respect to slope angle for 8m slope with berm at 5.5m

Slope Angle	Toe*	Mid*	Top*	Toe+Mid+Top*	Toe+Mid*	Toe+Top*	Mid+Top*	Without tree
30º	2.6	2.52	2.47	2.54	2.58	2.57	2.4	2.55
40º	2.29	2.16	2.11	2.24	2.27	2.27	2.08	2.2
45º	2.23	2.08	2.08	2.2	2.25	2.25	2.08	2.14
60º	1.32	1.25	1.24	1.28	1.30	1.29	1.22	1.26

Table 5.38 Factor of safety with respect to slope angle for 8m slope with berm at 5.5m with turfing

Slope Angle	Toe*+Turfing	Mid*+turfing	Top*+Turfing	Toe+Mid+Top*+Turfing	Toe+Mid*+Turfing	Toe+Top*+Turfing	Mid+Top*+Turfing	Turfing only
30º	2.7	2.58	2.57	2.58	2.58	2.63	2.48	2.62
40º	2.4	2.27	2.2	2.35	2.38	2.36	2.3	2.32

45^{o}	2.28	2.12	2.12	2.21	2.26	2.28	2.1	2.17
60^{o}	1.41	1.33	1.32	1.37	1.39	1.38	1.31	1.35

Table 5.39 Factor of safety with respect to slope angle for 11m slope with berm at 5.5m

Slope Angle	Toe*	Mid*	Top*	Toe+Mid +Top*	Toe+Mid*	Toe+Top*	Mid+Top*	Without tree
30^{o}	1.82	1.79	1.77	1.81	1.83	1.82	1.79	1.8
40^{o}	1.6	1.56	1.54	1.59	1.58	1.58	1.53	1.56
45^{o}	1.51	1.45	1.43	1.47	1.5	1.47	1.43	1.45

Table 5.40 Factor of safety with respect to slope angle for 11m slope with berm at 5.5m with turfing

Slope Angle	Toe* +Turfing	Mid* +turfing	Top* +Turfing	Toe+Mid +Top*+Turfing	Toe+Mid*+Turfing	Toe+Top*+Turfing	Mid+Top*+Turfing	Turfing only
30^{o}	1.88	1.87	1.83	1.85	1.86	1.87	1.83	1.86
40^{o}	1.66	1.61	1.62	1.63	1.67	1.63	1.58	1.6
45^{o}	1.56	1.53	1.58	1.53	1.56	1.52	1.49	1.53

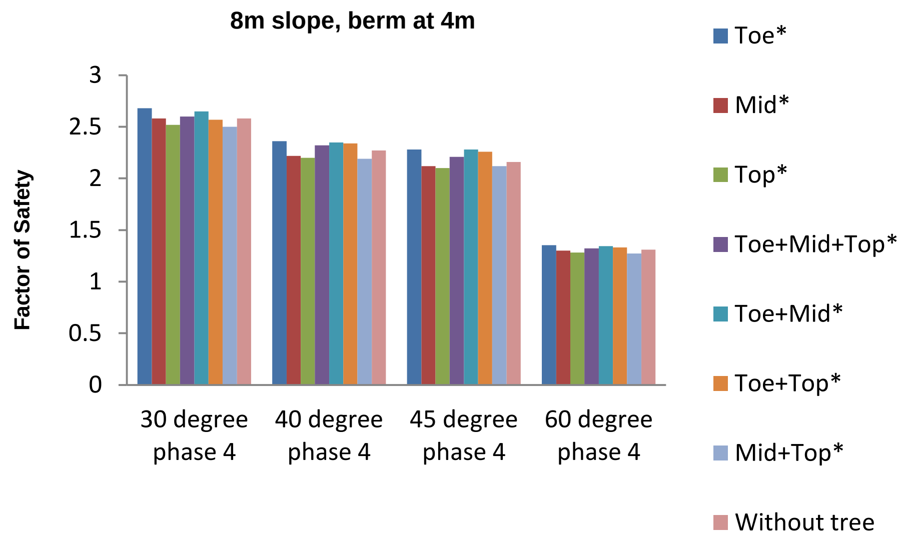


Figure 5.37(a). Variation of factor of safety with respect to slope angle for 8m slope with berm at 4m (*Tree position)

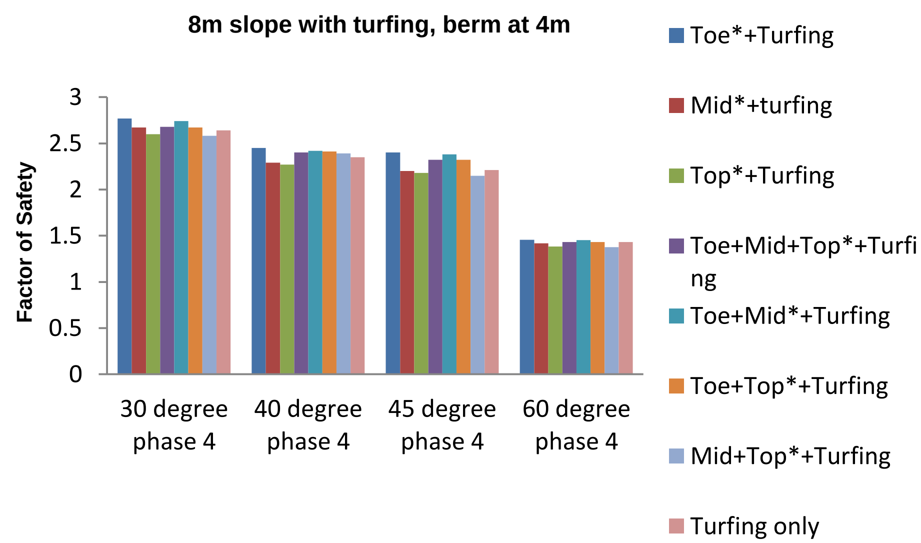


Figure 5.37(b). Variation of factor of safety with respect to slope angle for 8m slope with berm at 4m with turfing (*Tree position)

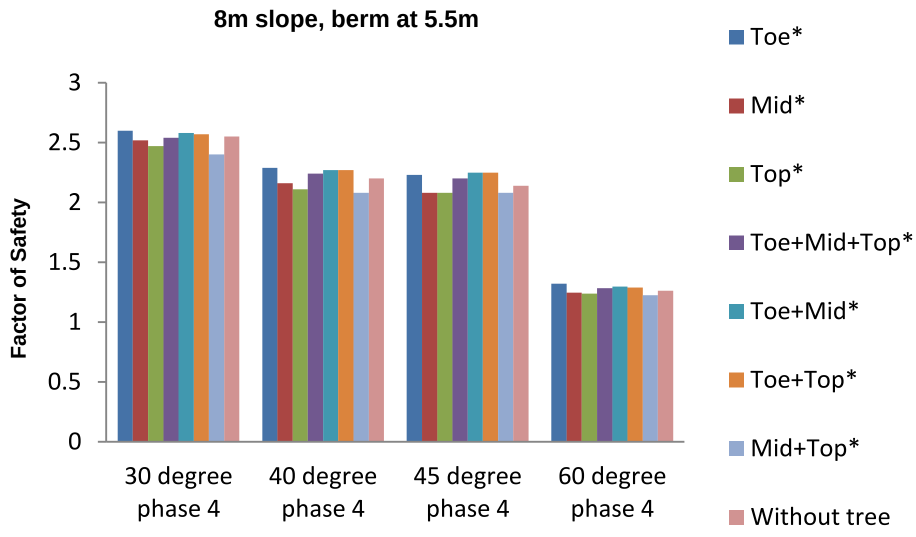


Figure 5.37(c). Variation of factor of safety with respect to slope angle for 8m slope with berm at 5.5m (*Tree position)

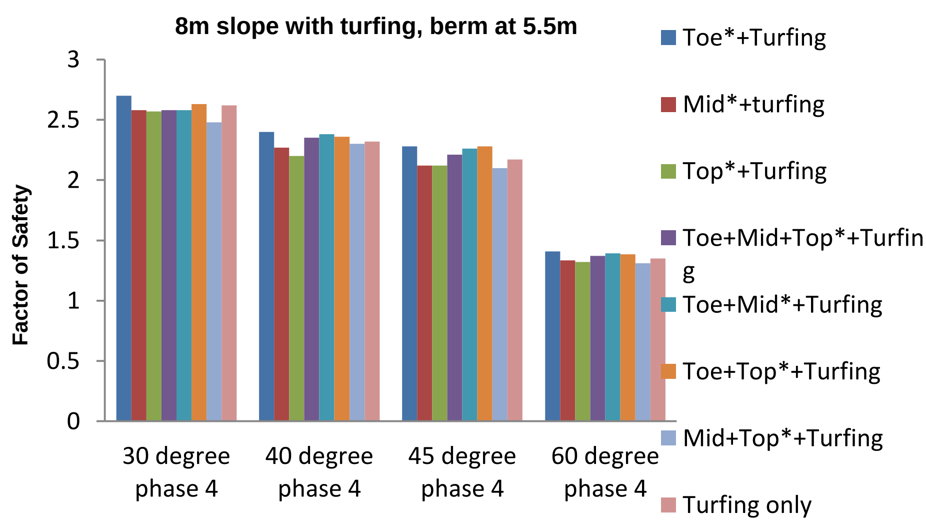


Figure 5.37(d). Variation of factor of safety with respect to slope angle for 8m slope with berm at 5.5m with turfing (*Tree position)

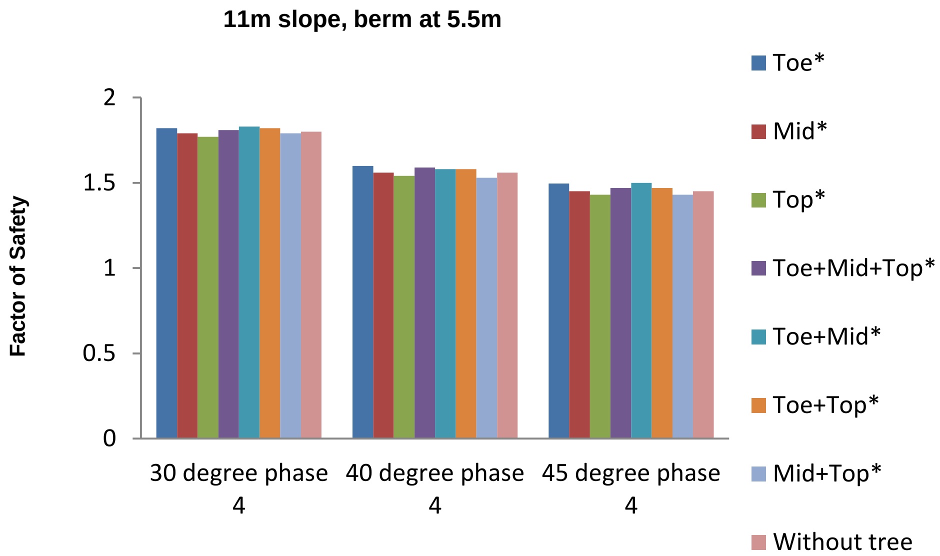


Figure 5.37(e). Variation of factor of safety with respect to slope angle for 11m slope with berm at 5.5m (*Tree position)

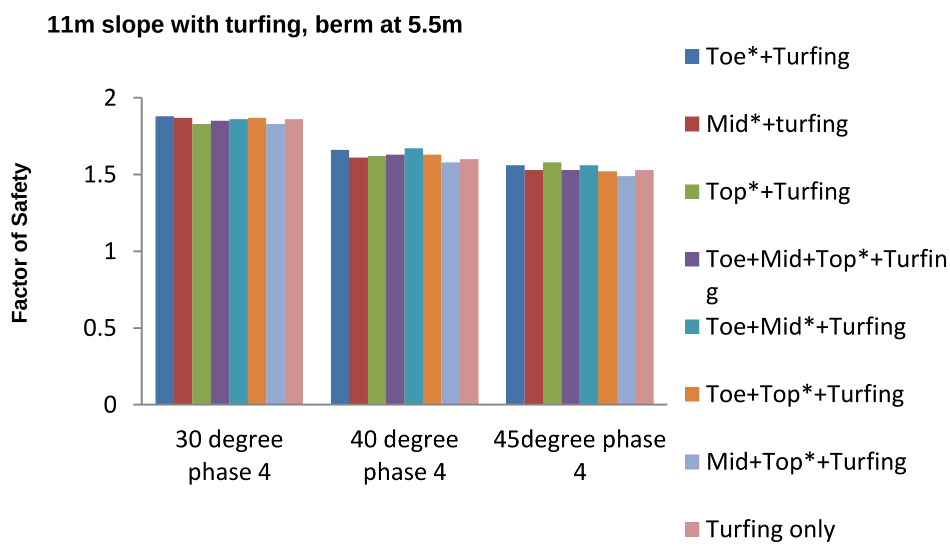


Figure 5.37(f). Variation of factor of safety with respect to slope angle for 11m slope with berm at 5.5m with turfing (*Tree position)

5.2.3.2. Percentage variation in factor of safety with respect to tree position

Table 5.41 Percentage variation in factor of safety with respect to tree position **for 8m slope with berm at 4m**

Slope Angle	Toe*	Mid*	Top*	Toe+Mid+Top*	Toe+Mid*	Toe+Top*	Mid+Top*
30°	3.88	0	-2.33	0.78	2.71	-0.39	-3.10
40°	3.97	-2.20	-3.08	2.20	3.53	3.08	-3.53
45°	6.64	-1.42	-2.37	4.74	6.64	5.69	-2.37
60°	3.20	-0.76	-2.06	0.84	2.59	1.53	-2.89

Table 5.42 Percentage variation in factor of safety with respect to tree position **for 8m slope with berm at 5.5m**

Slope Angle	Toe*	Mid*	Top*	Toe+Mid+Top*	Toe+Mid*	Toe+Top*	Mid+Top*
30°	1.96	-1.18	-3.14	-0.39	1.18	0.79	-5.88
40°	4.09	-1.82	-4.09	1.82	3.18	3.18	-5.46
45°	4.21	-2.80	-2.80	2.80	5.14	5.14	-2.80
60°	4.59	-1.27	-1.90	1.50	2.77	2.14	-3.09

Table 5.43 Percentage variation in factor of safety with respect to tree position **for 11m slope with berm at 5.5m**

Slope Angle	Toe*	Mid*	Top*	Toe+Mid+Top*	Toe+Mid*	Toe+Top*	Mid+Top*
30°	1.11	-0.56	-1.67	0.56	1.67	1.11	-0.56
40°	2.57	0	-1.28	1.92	1.28	1.28	-1.92
45°	3.17	0	-1.38	1.38	3.45	1.38	-1.38

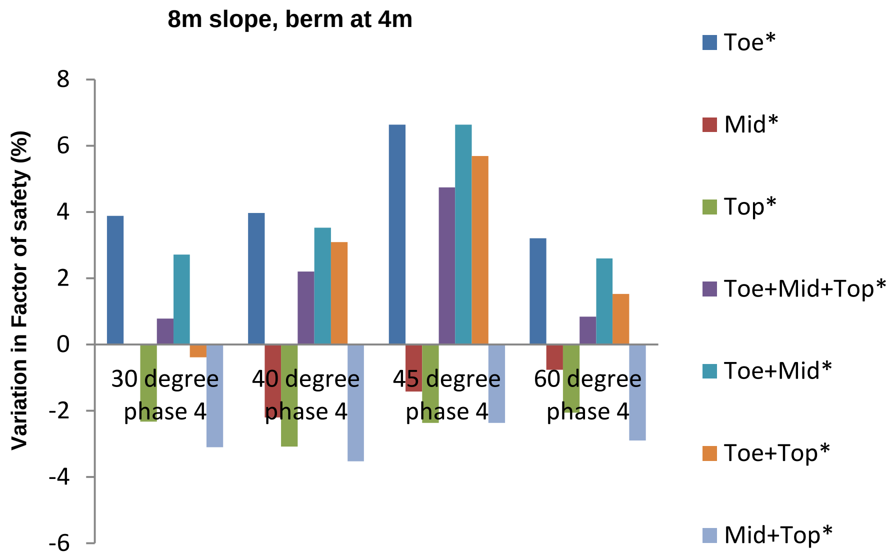


Figure 5.38(a). Percentage variation in factor of safety with respect to tree position for 8m slope with berm at 4m (*Tree position)

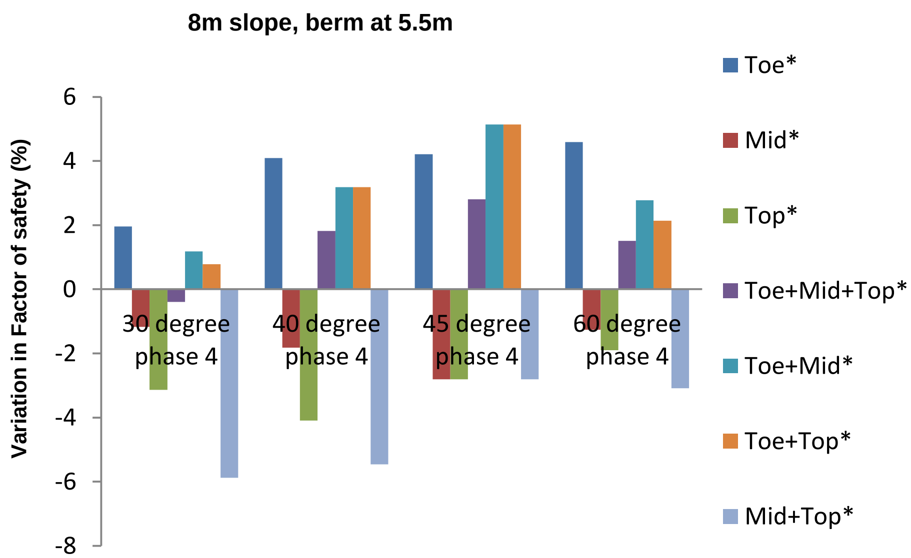


Figure 5.38(b). Percentage variation in factor of safety with respect to tree position for 8m slope with berm at 5.5m (*Tree position)

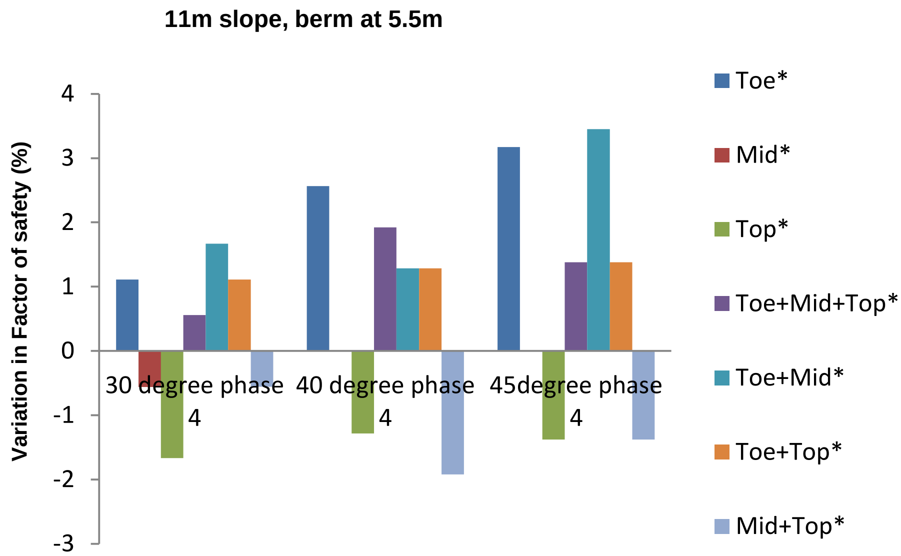


Figure 5.38(c). Percentage variation in factor of safety with respect to tree position for 11m slope with berm at 5.5m (*Tree position)

An increase of factor of safety up to 9% is observed when tree is at toe and a 5% decrease in factor of safety is observed when tree is at Mid+Top. Figures 5.38 (a) to (c) shows the percentage variation in factor of safety with respect to tree position (Table 5.41 to 5.43) when compared with bare slope.

5.2.3.3. Percentage variation in factor of safety with respect to turfing

Table 5.44 Percentage variation in factor of safety with respect to turfing for 8m slope with berm at 4m

Slope Angle	Toe*+Turfing	Mid*+turfing	Top*+Turfing	Toe+Mid+Top*+Turfing	Toe+Mid*+Turfing	Toe+Top*+Turfing	Mid+Top*+Turfing	Turfing only
30°	7.37	3.49	0.78	3.88	6.20	3.49	0	2.33
40°	7.93	0.88	0	5.73	6.61	6.17	5.29	3.53
45°	11.11	1.85	0.93	7.41	10.19	7.41	-0.46	2.32
60°	10.98	8.16	5.42	9.08	10.76	9.23	5.04	9.08

Table 5.45 Percentage variation in factor of safety with respect to turfing for 8m slope with berm at 5.5m

Slope Angle	Toe*+Turfing	Mid*+turfing	Top*+Turfing	Toe+Mid+Top*+Turfing	Toe+Mid*+Turfing	Toe+Top*+Turfing	Mid+Top*+Turfing	Turfing only
30º	5.88	1.18	0.79	1.18	1.18	3.14	-2.75	2.75
40º	9.09	3.18	0	6.82	8.18	7.27	4.55	5.46
45º	6.54	-0.93	-0.93	3.27	5.61	6.54	-1.87	1.40
60º	11.48	5.54	4.67	8.55	10.21	9.58	3.64	6.81

Table 5.46 Percentage variation in factor of safety with respect to turfing for 11m slope with berm at 5.5m

Slope Angle	Toe*+Turfing	Mid*+turfing	Top*+Turfing	Toe+Mid+Top*+Turfing	Toe+Mid*+Turfing	Toe+Top*+Turfing	Mid+Top*+Turfing	Turfing only
30º	4.45	3.89	1.67	2.78	3.33	3.89	1.67	3.33
40º	6.41	3.21	3.85	4.49	7.05	4.49	1.28	2.57
45º	7.59	5.52	8.97	5.52	7.59	4.82	2.76	5.52

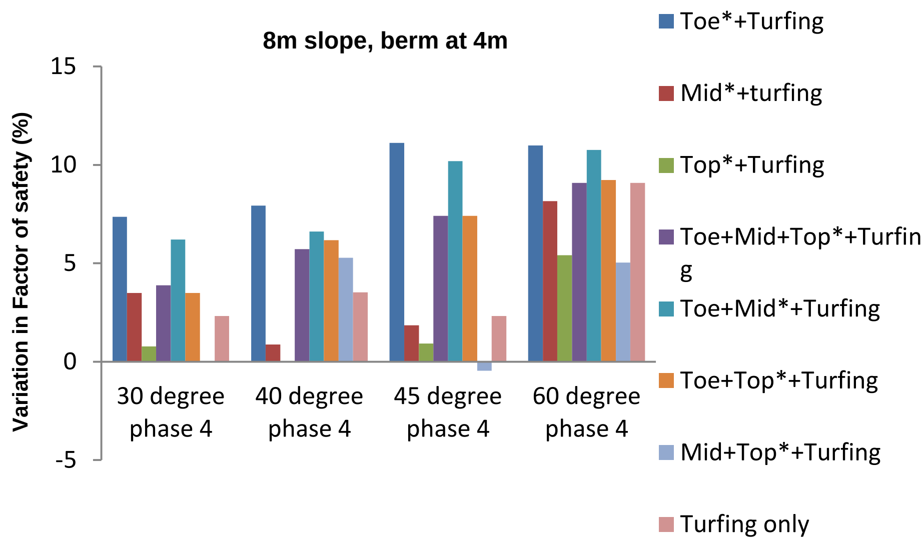


Figure 5.39(a). Percentage variation in factor of safety with respect to turfing for 8m slope with berm at 4m (*Tree position)

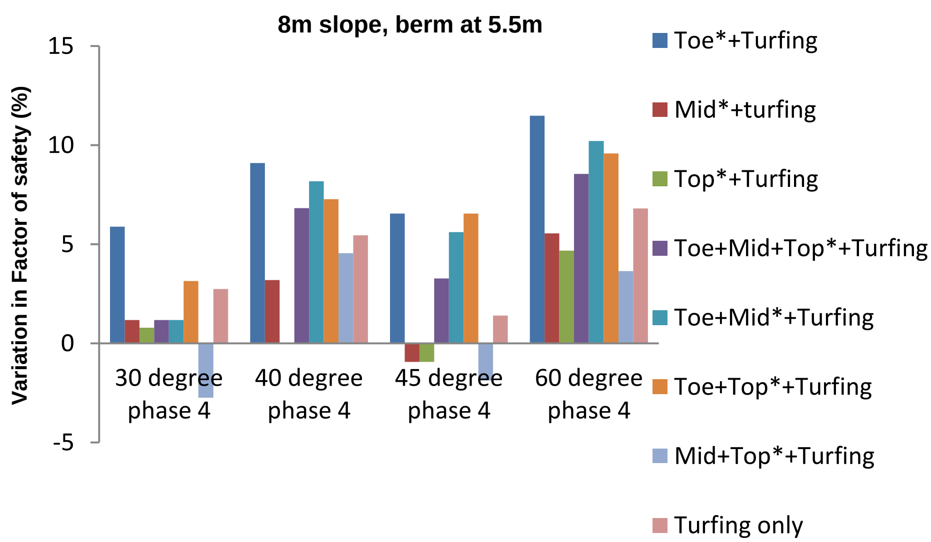


Figure 5.39(b). Percentage variation in factor of safety with respect to turfing for 8m slope with berm at 5.5m (*Tree position)

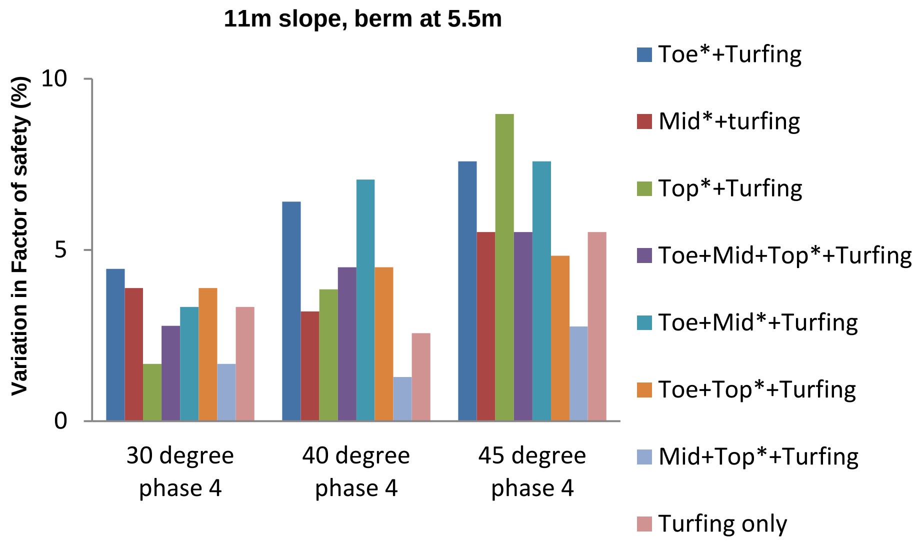
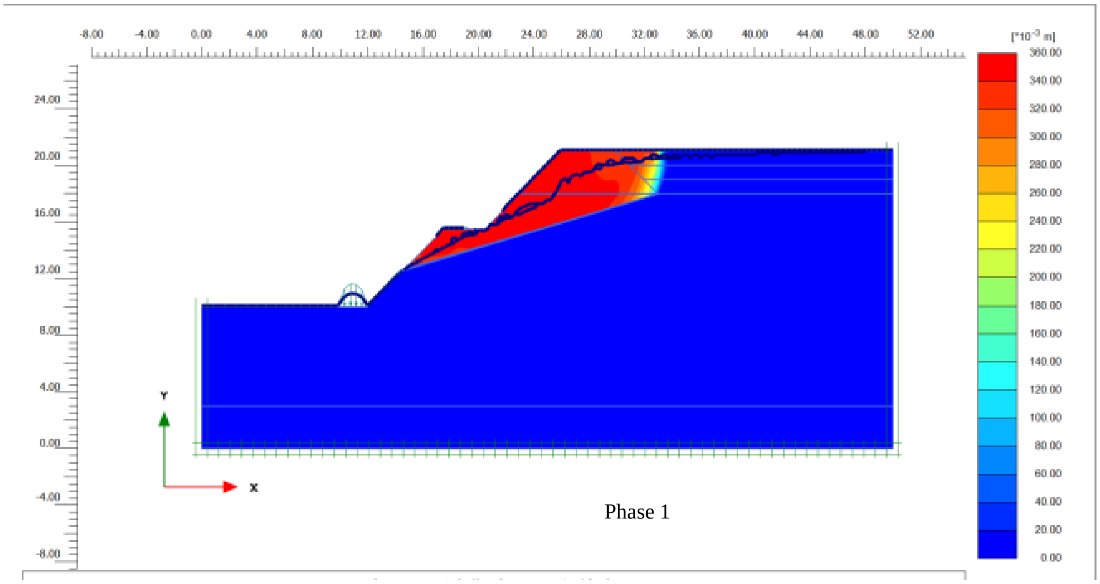


Figure 5.39(c). Percentage variation in factor of safety with respect to turfing for 11m slope with berm at 5.5m (*Tree position)

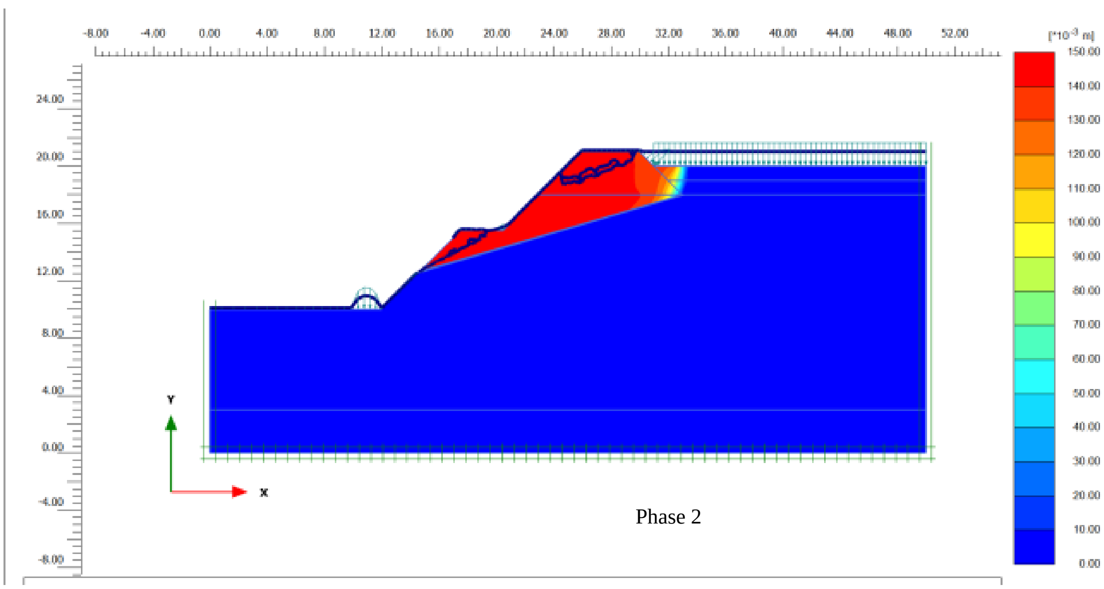
A percentage increase up to 12% is obtained when turfing is provided in slopes along with the trees in undrained conditions. Figures 5.39 (a) to (c) shows the percentage variation in factor of safety with respect to turfing (Table 5.44 to 5.46) when compared with bare slope.

5.2.4. Effect of Piping on Excavated Slopes

A pipe of diameter 0.1m is provided in the lithomargic clay layer for bare excavated slopes of height 8m and 11m under undrained conditions. The slopes with cut slope angles 30, 40, 45 and 60 degrees are being analyzed. A decrease in factor of safety is obtained for slope angles 30, 40, and 45 degrees, whereas 60 degree slope failed for both 8m and 11m height slopes when piping is provided. The failure surface is seen to pass along the pipe in all the phases. (Fig.5.40)



Phase 1



Phase 2

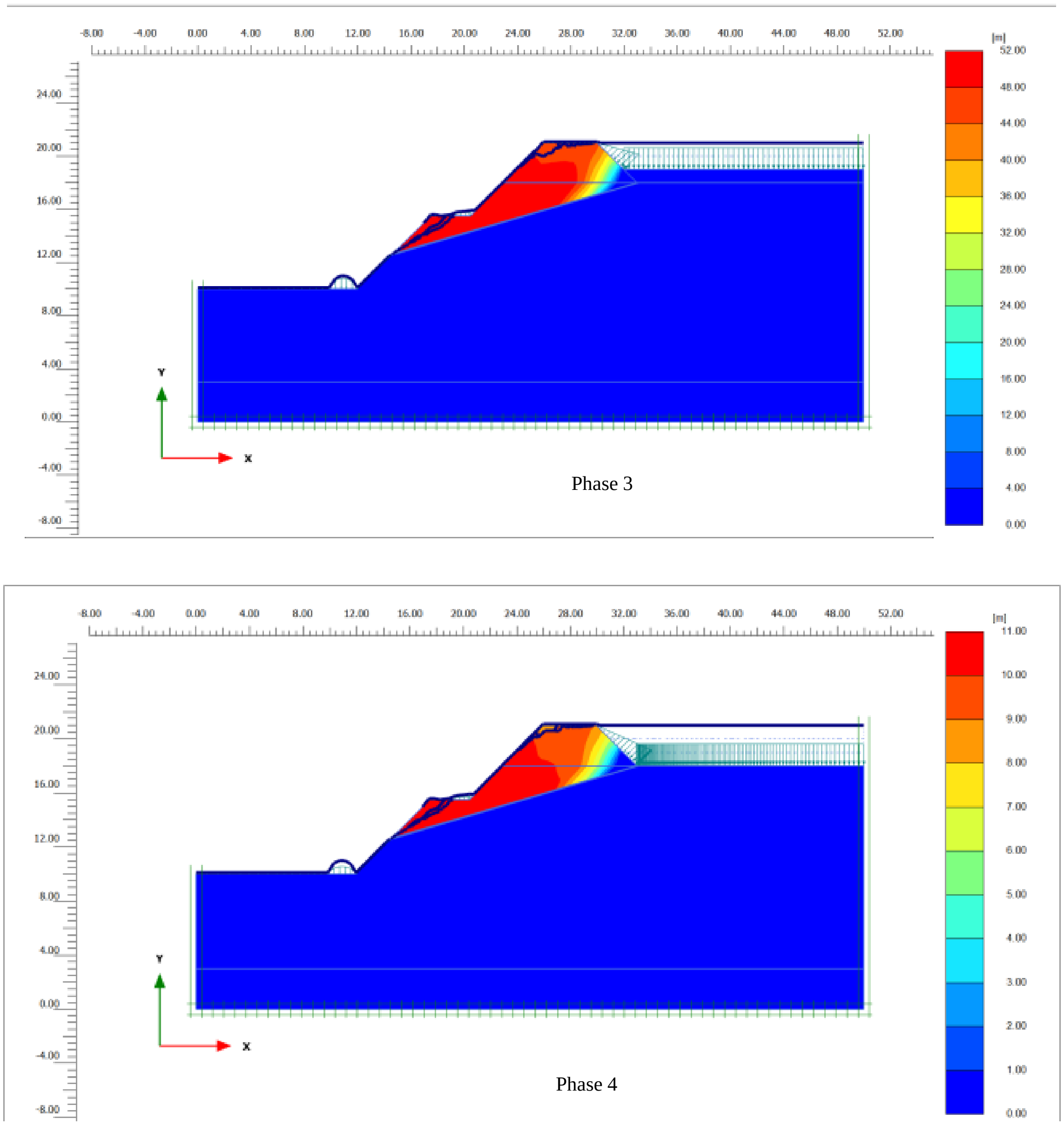


Figure 5.40. Failure surfaces passing along the pipe for Phase 1 to Phase 4

CHAPTER 6

CONCLUSIONS

6.1. GENERAL

The final conclusions of this research work are discussed in this chapter. The erosion characteristics of controlled lithomargic clay samples are studied and the conclusions are summarised. The variation of critical shear stress for samples with higher clay fraction and higher silt fraction was studied under different compaction conditions and the conclusions are stated in this chapter. The conclusions from the results of slope stability analyses studying the effect of vegetation excavation, seepage, precipitation and piping are also summarised in this chapter.

6.2. HOLE EROSION TESTS

- Hole erosion tests are conducted on lithomargic clay samples containing higher clay fraction (C samples) and higher silt fraction (M samples).
- Soils containing higher clay fraction showed higher resistance to erosion with critical shear stress varying from 200N/m^2 to 400N/m^2 whereas in the case of soils with higher silt fraction lower erosion resistance was observed, with critical shear stress varying from 45N/m^2 to 125N/m^2.
- Critical shear stress was observed to be higher for all the samples prepared at OMC conditions indicating greater resistance to erosion.
- Soils with high plasticity index were less susceptible to erosion than soils with lower plasticity index.
- Lithomargic clays with higher clay fraction could be classified under moderately slow erosion, whereas soils with higher silt fraction indicated very rapid to moderately rapid erosion.

6.3. SLOPE STABILITY ANALYSES

- There is noticeable change in factor of safety when shedi soil becomes drained to undrained condition (about 24%).

- The factor of safety is more if we provide berm at mid rather than providing at 3/4th height for slopes of 8m height.
- Vegetation (turfing) is multifunctional, relatively inexpensive and visually attractive and help to stabilize the slopes. It increases the factor of safety to about 12%. The roots of the turf vegetation act as soil reinforcement and increase the cohesion thus reducing the soil erosion.
- The effect of trees over slope is more as the slope becomes steeper. The factor of safety of vegetated slope increases with increase in cohesion if properly located.
- Tree at toe provides better stability whereas trees at mid and top gives lower factor of safety. The factor of safety of vegetated slope with both turfing and tree at toe gives more stability to the slope compared to only tree at toe position.
- Piping in the excavated slopes results in a decrease in stability of the slope. The failure surface was observed to pass along the pipe present in the slope.

REFERENCES

[1] Adams, B.T., Xiao, M., and Wright, A. (2013). "Erosion mechanisms of organic soil and bioabatement of piping erosion of sand." *Journal of Geotechnical and Geoenvironmental Engineering* 139,1360–1368.

[2] Achari, R., and Shivashankar, R. (2005). "Studies on Anisotropy and Behaviour of Blended Shedi Soil". *M Tech thesis*, Department of Civil Engineering, NITK Surathkal, India.

[3] Aleva, G.J.J., and Creutzberg, D. (1994). "Laterites: concepts, geology, morphology, and chemistry". Wageningen, The Netherlands: International Soil Reference and Information Center.

[4] Alkasawneh, W., Husein Malkawi, A.I., Nusairat, J.H., and Albataineh, N. (2008). "A Comparative Study of Various Commercially Available Programs in Slope Stability Analysis." *Comput. Geotech*. 35, 428–435.

[5] Al-Samarrai, L.B. (2014). "Soil stabilization against internal erosion using cement and fiber-cement mixture ", *M.Sc.Thesis*, American University of Sharjah, UAE.

[6] Ali, N., Farshchi, I., Mu'azu, M. A., & Rees, S. W. (2012). "Soil-root interaction and effects on slope stability analysis." *Electronic Journal of Geotechnical Engineering*, *17*, 319-328.

[7] Anaswara, S., and Shivashankar, R. (2015). "Effect of vegetation on stability of slopes." *50th Indian Geotechnical Conference*, Pune, Maharashtra, India.

[8] ASTM D4647/D4647M-13.(2013). "Standard test methods for identification and classification of dispersive clay soils by the pinhole test". *ASTM International*, West Conshohocken, PA. https://doi.org/10.1520/D4647_D4647M-13.

[9] Benahmed, N., and Bonelli, S. (2012a). "Investigating Concentrated Leak Erosion Behaviour of Cohesive Soils by Performing Hole Erosion Tests." *Eur. J. Environ. Civ. Eng*. 16, 43–58.

[10] Benahmed, N., and Bonelli, S.(2012b)."Internal erosion of cohesive soils: laboratory parametric study." *6th International Conference on Scour and Erosion*, Paris, France. 8.

[11] Benaissa, K., Angel, P.V.M., Dolores, R.C.M., Philippe, D., Abdellatif, K., Mohammed, B., and Larbi, E.B. (2012). "Predicting initial erosion during the hole erosion test by using turbulent flow CFD simulation." *Appl. Math. Model*. 36, 3359–3370.

[12] Berilgen, M.M. (2007). "Investigation of stability of slopes under drawdown conditions." *Comput. Geotech*. 34, 81–91.

[13] Bezzazi, M., Khamlichi, A., Vera, M.P., Rubio, M.D.C., and Olegario, C.L. (2010). "A Simplified analytical modeling of the hole erosion test." *Am. J. Eng. Appl*. Sci. 3, 765–768.

[14] Bonelli, S., Benahmed, N., and Brivois, O. (2006a) "On modelling of the hole erosion test." *3rd International Conference on Scour and Erosion*, Amsterdam, Netherlands. 6.

[15] Bonelli, S., Brivois, O., Borghi, R., and Benahmed, N. (2006b). "On the modelling of piping erosion." *Comptes Rendus Mécanique* 334, 555–559.

[16] Bonelli, S., and Brivois, O. (2008). "The scaling law in the hole erosion test with a constant pressure drop." *Int. J. Numer. Anal. Methods Geomech*. 32, 1573–1595.

[17] Boukhemacha, M.A., Bica, I., and Mezouar, K. (2013). "New Procedures to Estimate Soil Erodibility Properties from a Hole Erosion Test Record." *Period. Polytech. Civ. Eng*. 57, 77.

[18] Cazzuffi, D., Cardile, G., and Gioffrè, D. (2014). "Geosynthetic engineering and vegetation growth in soil reinforcement applications." *Transp. Infrastruct. Geotechnol.* 1, 262–300.

[19] Chanmee, N., Bergado, D. T., Hino, T., and Lam, L. G. (2016). “Analysis and simulations of erosion protection designs using the PLAXIS 2D and slide programs”. *Japanese Geotechnical Society Special Publication*, 2(29), 1075-1078.

[20] Chevalier, C., Haghighi, I., Pham, T.L., and Reiffsteck, P. (2010). “Two complementary tests for characterizing the soil erosion.” *American Society of Civil Engineers*, 152–161.

[21] Chok, Y.H. (2008). " Modelling the effects of soil variability and vegetation on the stability of natural slopes", PhD Thesis, The University of Adelaide School of Civil Environmental and Mining Engineering.

[22] Coppin, N.J., Richards, I.G., and Construction Industry Research and Information Association (2007). “Use of vegetation in civil engineering”. (London: CIRIA).

[23] Dalton, P. A., Smith, R. J., and Truong, P. N. V. (1996). “Vetiver grass hedges for erosion control on a cropped flood plain: hedge hydraulics.” *Agricultural Water Management*, 31(1-2), 91-104.

[24] Darshan, C.S., and Sitaram, N.(2017). “SEM and XRD investigations on lithomargic clay stabilized using granulated blast furnace slag and cement.” *International Journal of Geotechnical Engineering*. https://doi.org/10.1080/19386362.2017.1380355

[25] Daware, S.N., and Hegde, R.A. (2010). " Daware effect of alternate wetting and drying on laterite and their engineering behaviour." *Indian Geotechnical Conference*, GEOtrendz, Mumbai, 16-18.

[26] D’Souza D.N., Choudhary A.K., Basak P., and Shukla S.K. (2019). “Assessment of vetiver grass root reinforcement in strengthening the soil. in: Thyagaraj T. (eds)”. *Ground Improvement Techniques and Geosynthetics*. Lecture Notes in Civil Engineering, 14. Singapore.

[27] Duncan, J.M. (1996). "State of the Art: Limit Equilibrium and Finite-Element Analysis of Slopes." *J. Geotech. Eng.* 122, 577–596.

[28] Dupuy, L., Fourcaud, T., and Stokes, A. (2005). "A Numerical Investigation into the Influence of Soil Type and Root Architecture on Tree Anchorage." *Plant Soil.* 278, 119–134.

[29] Fell, R., Wan, C.F., Cyganiewicz, J., and Foster, M. (2003). "time for development of internal erosion and piping in embankment dams." *J. Geotech. Geoenvironmental Eng.* 129, 307–314.

[30] Gasmo, J.M., Rahardjo, H., and Leong, E.C. (2000). "infiltration effects on stability of a residual soil slope." *Comput. Geotech.* 26, 145–165.

[31] Gidigasu, M.D. (1976). “Laterite soil engineering: pedogenesis and engineering principles”. Amsterdam ; New York: Elsevier Scientific Pub. Co.

[32] Gray, D.H., and Leiser, A.T. (1982). “Biotechnical slope protection and erosion control”. New York: Van Nostrand Reinhold Co.

[33] Gray, D.H., and Sotir, R.B. (1996). “Biotechnical and soil bioengineering slope stabilization: a practical guide for erosion control”. New York: John Wiley & Sons.

[34] Greenway, D. R. (1987). "Vegetation and slope stability. in slope stability", edited by M. F. Anderson and K. S. Richards. New York: John Wiley & Sons.

[35] Haghighi, I., Chevalier, C., Duc, M., Guédon, S., and Reiffsteck, P. (2013). "Improvement of Hole Erosion Test and Results on Reference Soils." *J. Geotech. Geoenvironmental Eng*. 139, 330–339.

[36] Hamdhan, I.N., and Schweiger, H.F. (2011). "Slope stability analysis of unsaturated soil with fully coupled flow-deformation analysis". In *Annual Conferenceof the International Association for Mathematical Geosciences*, 1-18. IAMG

[37] ICOLD (2016). Bulletin 164 : "Internal erosion of existing dams, levees and dikes, and their foundations. Volume 2: Investigations, testing, monitoring and detection, remediation and case histories. Internal erosion processes and engineering assessment." *International Commission on Large Dams*, Paris. Preprint 5 February 2016 from: http://www.icold-cigb.org.

[38] Indraratna, B., Athukorala, R., and Vinod, J. (2013). "estimating the rate of erosion of a silty sand treated with lignosulfonate." *J. Geotech. Geoenvironmental Eng.* 139, 701–714.

[39] Indraratna, B., Muttuvel, T., and Khabbaz, H. (2008). "investigating erosional behaviour of chemically stabilised erodible soils." *American Society of Civil Engineers*, 670–677.

[40] IS 2720-11: Methods of tests for soils: Determination of shear strength parameters of a specimen tested in unconsolidated undrained triaxial compression without the measurement of pore water pressure. *Bureau of Indian Standards*, New Delhi (1993–Reaffirmed 2002)

[41] Jotisankasa, A., Mairaing, W., and Tansamrit, S. (2014). "Infiltration and stability of soil slope with vetiver grass subjected to rainfall from numerical modeling." In Proceedings of the 6th International Conference on Unsaturated Soils, *UNSAT*, 1241-1247.

[42] Koohpeyma, H.R., Vakili, A.H., Moayedi, H., Panjsetooni, A., and Nazir, R. (2013). "Investigating the effect of lignosulfonate on erosion rate of the embankments constructed with clayey sand." *Sci. World J*. 2013, 1–6.

[43] Lewis, D.A., and Schmidt, N.O. (1977). "Erosion of unsaturated clay in a pinhole test." *American Society for Testing and Materials*, 623, 260–273.

[44] Lim, S.S. (2006). "Experimental investigation of erosion in variably saturated clay soils", PhD Thesis, University of New South Wales, Australia.

[45] Luthi, M., (2011) “A Modified Hole Erosion Test (HET-P) to Study Erosion Characteristics of Soil,” Ph.D. thesis, The University of British Columbia, Vancouver, BC.

[46] Luthi, M., Fannin, R.J., and Millar, R.G. (2012). "A modified hole erosion test (HET-P) device." *Geotech. Test. J*. 35, 104336.

[47] Maneecharoen, J., Htwe, W., Bergado, D.T., and Baral, P. (2013). "ecological erosion control by limited life geotextiles (LLGs) as well as with vetiver and ruzi grasses." *Indian Geotech. J.* 43, 388–406.

[48] Matthews, C., Farook, Z., and Helm, P. (2014). “Slope stability analysis–limit equilibrium or the finite element method”. *Ground Engineering*, 48(5), 22-28.

[49] Mehenni, A., Cuisinier, O., and Masrouri, F. (2016). “Impact of lime, cement, and clay treatments on the internal erosion of compacted soils.” *Journal of Materials in Civil Engineering* 28:04016071. https://doi.org/10.1061/(ASCE)MT.1943-5533.0001573.

[50] Merat, S., Djerbal, L., and Bahar, R. (2019). “Rainfall effect on slope stability using numerical analysis.” *In Recent Advances in Geo-Environmental Engineering, Geomechanics and Geotechnics, and Geohazards*, 419-424. Cham.

[51] Mickovski, S.B., and Beek, L.P.H. van (2009). "Root Morphology and Effects on Soil Reinforcement and Slope Stability of Young Vetiver (Vetiveria Zizanioides) Plants Grown in Semi-Arid Climate." *Plant Soil* 324, 43–56.

[52] Morgan, R.P.C. (1995). “Slope stabilization and erosion control: a bioengineering approach”.London: Spon.

[53] Mu’azu, M.A., Farshchi, I., Ali, N., and Rees, S.W. (2012). "Soil-Root Interaction and Effects on Slope Stability Analysis." *EJGE*. 17, Bund C.

[54] Muttuvel, T. (2008). "Erosion rate of chemically stabilised soils incorporating tensile stress-deformation behaviour," PhD thesis. School of Civil, Mining & Environmental Engineering, University of Wollongong, Australia.

[55] Nayak, S., and Sarvade, P.G. (2012). “Effect of cement and quarry dust on shear strength and hydraulic characteristics of lithomargic clay.” *Geotechnical and Geological Engineering* 30:419-430. https://doi.org/10.1007/s10706-011-9477-y.

[56] Vincent, N.A., Shivashankar, R., Lokesh, K.N., and Jacob, J.M. (2017). “Laboratory electrical resistivity studies on cement stabilized soil.” *International Scholarly Research Notices.* https://doi.org/10.1155/2017/8970153..

[57] Norris, J.E. (2008). “Slope stability and erosion control ecotechnological solutions”. Dordrecht; [London]: *Springer*.

[58] Osman, N., Abdullah, M. N., and Ali, F.H. (2011). " Soil-root Shear Strength Properties of Some Slope Plants." Sains Malaysiana. 40, 1065–1073.

[59] Quinton, J.N., Edwards, G.M., and Morgan, R.P.C. (1997). "The Influence of Vegetation Species and Plant Properties on Runoff and Soil Erosion: Results from a Rainfall Simulation Study in South East Spain". *Soil Use and Management*. 13,143-148.

[60] Ramesh, H.N., and Nanda, H.S. (2007). “Strength behaviour of shedi soil treated with fly ashes”. *13th ARC 2007*, Kolkata, 893–896.

[61] Ravi Shankar, A.U., Chandrashekhar, A., and Prakash Bhat, H. (2012). “Experimental investigation on lithomargic clay stabilized with sand and coir”. *Indian Highways, Indian Roads Congress* 40:21-31.

[62] Reddi, L.N., Lee, I.M., and Bonala, M.V.S. (2000). “Comparison of internal and surface erosion using flow pump tests on a sand-kaolinite mixture.” *Geotechnical Testing Journal*. 23:116–122. https://doi.org/10.1520/GTJ11129J

[63] Sabhahit, N., and Rao, A.U. (2004). "Failure Analysis of Excavated Slopes in Laterite Soils." *Proceedings of the International Symposium on Lowland Technology*.

[64] Saliba, F., Nassar, R. B., Khoury, N., and Maalouf, Y. (2019). "Internal Erosion and Piping Evolution in Earth Dams Using an Iterative Approach." In Geo-Congress 2019: Soil Erosion, Underground Engineering, and Risk Assessment ,67-75. Reston, VA: *American Society of Civil Engineers*.

[65] Sargunan, A. (1977). "Concept of critical shear stress in relation to characterisation of dispersive clays." Dispersive clays, related piping, and erosion in geotechnical projects, *ASTM STP 623*, Sherard and Decker, Eds., 390-397.

[66] Savage, S., Douglas, K., Fell, R., Pierson, W., and Berndt, R. (2019). "Modeling the Erosion and Swelling of the Sides of Transverse Cracks in Embankment Dams". *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 145 (5).

[67] Sekhar, D.C., Nayak, S., and Preetham, H.K. (2017). "Influence of granulated blast furnace slag and cement on the strength properties of lithomargic clay." *Indian Geotechnical Journal* 47:384-392. https://doi.org/10.1007/s40098-017-0228-8.

[68] Shaikh, A., Ruff, J.F., Charlie, W.A. and Abt, S.R (1988). "Erosion rale of dispersive and non-dispersive clays". *Journal of Geotechnical Engineering Division*, ASCE, Vol. 114, 589-600.

[69] Sherard, J., Dunningan, L., Decker, R., and Steele, E. (1976). "Pinhole test for identifying dispersive soils." *Journal of the Geotechnical Engineering* 102:69-85.

[70] Shivashankar, R., Hridya, S., Thomas, Biji and Reddy, D. Venkat. (2014), "Effect of vegetation on shear strength of shedi soils and slope stability",

Indian Geotechnical Conference IGC-2014, Kakinada, India, 10 pages, pg. 269 in volume of abstracts.

[71] Shivashankar, R., Ravi Shankar, A.U., and Jayamohan, J. (2015). “Some studies on engineering properties, problems, stabilization and ground improvement of lithomargic clays.” *Geotechnical Engineering Journal of SEAGS and AGSSEA*, 46:68-80 .

[72] Taccari, M. L., and van der Meij, R. (2016). “Investigation of the influence of animal burrowing on the failure of the levee of San Matteo along the Secchia river.”In E3S Web of Conferences .9, 19001). EDP Sciences.

[73] Truong, P. (2000). “The global impact of vetiver grass technology on the environment.” In Proceedings of the Second International Conference on Vetiver. Office of the Royal Development Projects Board, Bangkok, 48-61.

[74] Umesh, T.S., Dinesh, S.V.,and Sivapullaiah, P.V. (2011). “Characterization of dispersive soils.” *Materials Sciences and Applications*, 2:629-633.

[75] Vinod, J. S., Indraratna, B., and Mahamud, M. A. A. (2010). "Stabilisation of an erodible soil using a chemical admixture." *Institution of Civil Engineers. Proceedings. Ground Improvement*, 163, 43-52.

[76] Wahl, T. L. (2010)." Relating Het And Jet Test Results To Internal Erosion Field Tests." 2nd Joint Federal Interagency Conference, Las Vegas, NV.

[77] Wahl, T. L., Hanson, G.J. and Regazzoni, P., (2009). "Quantifying Erodibility of Embankment Materials For The Modeling Of Dam Breach Processes." ASDSO Dam Safety, Hollywood, Florida.

[78] Wahl, T. L., Regazzoni, P., and Erdogan, Z. (2009). "Partical Improvment for the Hole Erosion Test." 33rd IAHR Congress: Water Engineering For A Sustainable Environment, Vancouver.

[79] Wan, C.F., and Fell, R. (2002). “Investigation of internal erosion and piping of soils in embankment dams by the slot erosion test and the hole erosion test.” *UNICIV Report No.R-412*, UNSW, Sydney.

[80] Wan, C.F., and Fell, R. (2004a). "Investigation of Rate of Erosion of Soils in Embankment Dams." *J. Geotech. Geoenvironmental Eng.* 130, 373–380.

[81] Wan, C.F., and Fell, R. (2004b). "Laboratory tests on the rate of piping erosion of soils in embankment dams." *Geotechnical Testing Journal*, 27, 295–303.

[82] Wilson, G.V., Nieber, J.L., Sidle, R.C.,and Fox,G.A. (2013). “Internal erosion during soil pipe flow: State of the science for experimental and numerical analysis.” *Transactions of the ASABE* 56(2):465–478. doi:10.13031/2013.42667.

[83] Xiao, M., Abela, C., Mortensen, K., Beltran, M., and Oliver, A. (2008). “Rainfall Erosion Resistance of Various Compost Soils on Roadside Embankment.” *American Society of Civil Engineers*,377–384.

[84] Xiao, M., and Gomez, J. (2009). "Rainfall Erosion Resistance and Stability of Various Composts." *J. Soil Water Conserv.* 64, 233–242.

[85] Xiao, M., Reddi, L.N., Howard, J., Devine, A., and Stott, R.R. (2006). “Rainfall Erosion Control on Roadside Embankment Using Compost Soils.” *American Society of Civil Engineers*, 226–231.

[86] Ma, Y., Xiao, M., and Kermani, B. (2020). “Experimental Investigation of the Effects of Fluid’s Physicochemical Characteristics on Piping Erosion of a Sandy Soil under Turbulent Flow.” *Geotechnical Testing Journal*, 43(2).

LIST OF PUBLICATIONS

Journal:

1. Shivashankar R., Thomas, Biji Chinnamma; Krishnanunni K.T., Reddy D.V. (2019) Slope Stability Studies of Excavated Slopes in Lateritic Formations. In: I.V. A., Maji V. (eds) Geotechnical Applications. *Lecture Notes in Civil Engineering, vol 13. Springer*, Singapore. doi:10.1007/978-981-13-0368-5_14.
2. Thomas, Biji Chinnamma, Shivashankar R., Sarah Jacob and Meera Susan Varghese (2020), "Erosion Studies on Lithomargic Clays", *Indian Geotechnical Journal, Springer*. 50:142-156. https://doi.org/10.1007/s40098-019-00364-8
3. Shivashankar, R. and Thomas, Biji Chinnamma (2020) "Laterites And Lateritic Soils: Geology, Engineering Properties And Problems", *Lowland Technology International Journal*. 21(4):205-214.

International and National Conferences:

1) Thomas, Biji Chinnamma and Shivashankar R. (2019), "Study of Critical Shear Stresses of Soils for Progressive Erosion", *International Conference TMSF – Civil Engineering Today*, Don Bosco College of Engg. Goa, India.
2) Shivashankar, R. and Thomas, Biji Chinnamma (2019), "Hole Erosion Studies On Lithomargic Clays", *TC107 Symposium on Laterites and Lateritic Soils*, Goa Engineering College, Farmagudi, India.
3) Thomas, Biji Chinnamma and Shivashankar R. (2019), "Laterites: Slope Stability and Erosion Studies" , *Symposium on Laterites and Lateritic Soils*, Thiruvananthapuram, India.
4) Thomas, Biji Chinnamma , Shivashankar, R., Varghese, Meera Susan and Prabhu, Yashvantha N. (2018), "Slope Stability Studies On Lateritic Formations", *International Symposium on Lowland Technology* 2018, Hanoi, Vietnam.

5) Thomas, Biji Chinnamma , Krishnanunni, K. T.,Shivashankar, R. and Reddy, D. Venkat(2016), “Slope Stability Studies Of Excavated Slopes In Lateritic Formations, Including Erosion Studies On Lithomargic Clays”, *10th International Symposium on Lowland Technology 2016*, Mangalore, India.
6) Shivashankar, R., Thomas, Biji Chinnamma, Krishnanunni, K. T. and Reddy, D. Venkat(2016), “Slope Stability Studies Of Excavated Slopes In Lateritic Formations”, *Indian Geotechnical Conference IGC2016,* Chennai, India.
7) Shivashankar, R., Thomas, Biji Chinnamma, Krishnanunni, K. T. and Anaswara, S.(2016),“Slope Stability Analysis And Erosion Studies With Lithomargic Clays”, Extended Abstract Volume of *International Geotechnical Engineering Conference on Sustainability in Geotechnical Engineering Practices and Related Urban Issues*, Mumbai, India.
8) Shivashankar, R., Hridya,S. Thomas, Biji and Reddy, D. Venkat (2014),”Effect Of Vegetation On Shear Strength Of Shedi Soils And Slope Stability”, *Indian Geotechnical Conference IGC-2014*, Kakinada, India, 10 pages, pg. 269 in volume of abstracts.

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