STUDIES ON FIELD STABILIZATION METHODS TO PREVENT SURFICIAL SLOPE FAILURES OF EARTHFILL DAMS

by

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ABSTRACT

STUDIES ON FIELD STABILIZATION METHODS TO PREVENT
SURFICIAL SLOPE FIALURES OF EARTHFILL DAMS

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Surficial failures are occurring frequently on the slopes of earthfill dams which are predominantly rainfall induced. Desiccation cracks form on the slope surface during dry environment which accelerates infiltration during rainfall. Infiltration causes increase of pore water pressures and saturation of soil mass in top layer resulting in surficial failures. Numerous surficial failures occurred on the slope of earthfill dams maintained by The United States Army Corps. of Engineers. The current research is undertaken at the University of Texas at Arlington with an objective of exploring the best field stabilization method to mitigate these surficial failures.

Two sites, Joe Pool Dam and Grapevine Dam located in Fort Worth district were selected for the research. The admixtures used to treat the embankment soil were 20% compost, 4% lime with 0.30% polypropylene fibers, 8% lime with 0.15% polypropylene fibers, and 8% lime. These stabilizers were proven to be promising from

a laboratory study in mitigating desiccation cracking. Five test sections including four treated sections and one control section were constructed at each dam site. The test sections were instrumented with moisture probes, temperature probes and inclinometers. The moisture content and temperature was recorded for a period of one year at Joe Pool Dam and the data was analyzed using statistical comparison tools. Besides, the vertical movements of the test sections were monitored by conducting elevation surveys and the lateral movements were monitored by conducting periodical inclinometer surveys. Digital image studies were conducted to monitor the desiccation cracking observed in the test sections. Physical model studies were also carried out to study the relative performance of soil treated with admixtures when it was subjected to a number of alternate wetting and drying cycles. Additional laboratory tests were conducted and the data was used to carry out numerical modeling studies using PLAXFLOW and GSTABL7 software programs.

Based on the analysis of the data, the image studies, and the analytical model studies, the 8% lime with 0.15% fibers is found to be the most effective admixture followed by the 8% lime to prevent desiccation cracking and surficial failures of high plasticity clays.

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INTRODUCTION

1.1 Preamble

Stability of dam and embankment slopes has been a growing concern to Geotechnical engineers in the backdrop of landslides and slope failures of earth dams. Several rolled earthfill embankment dams in the United States and other parts of the world are constructed of clayey soils (McCleskey, 2008). These soils typically exhibit moderate to high plasticity, low to moderate strength and high swell and shrinkage characteristics (Puppala et al. 2006).

Clay soils tend to shrink during drying and swell during wetting. Repeated drying and wetting produces desiccation cracking within the plastic fill materials (McCleskey, 2005). During drying, shrinking of soil occurs due to the development of substantial matric suction in the pore structure of the fine-grained soils (Nahlawi and Kodikara, 2006). If the shrinkage is restrained, soils can crack during desiccation when the tensile stresses developed in the soil exceeds the tensile strength of soil. The depth of desiccation cracks were reported to be varying from as low as few cm to 10 m (few inches to as high as 33 ft) (Nahlawi and Kodikara, 2006).

When it rains, water infiltrates into the soil through the desiccation cracks. Infiltration increases pore water pressure which leads to reduction of shear strength (Rahardjo et al. 1994; Cho and Lee, 2002). As the wetting front increases, the permeability parallel to slope increases and thus the seepage occurs parallel to the slope (Day, 1996). The condition of reduced shear strength and increased shear stresses due to saturation of soil mass leads to surficial failures. Surficial failures are classified as shallow slope failures as the average depth of failure varies from 0.3 m to 1.2 m (1 ft to 4 ft) (Day, 1996). In many cases the failure surface is parallel to the slope face (Day, 1996). Figure 1.1 shows a typical surficial failure occurred at Bardwell Dam of United States Army Corps. of Engineers (USACE).

Figure 1.1 Surficial failure at Bardwell Dam (Source: USACE)

Historically, these surficial failures are repaired by removing the soil within the failure portion and replace with either borrow soil or with same soil with adequate compaction. This practice met with mixed success besides increasing cost of maintenance of dam (McCleskey, 2005). Repair costs of each failure are presently running into few hundred thousands of dollars.

The increasing number of slope failures resulted in the present dissertation research which was conducted at The University of Texas at Arlington (UTA) with the financial support from the United States Army Corps. of Engineers, Fort Worth district.

1.2 Research Objectives

The primary objective of this research is to explore, select and investigate various field stabilization methods to reduce desiccation cracking which will help mitigate surficial failures. Further, the research involves field monitoring of the performance of stabilizers, collecting and analyzing field data, conducting laboratory studies and carrying out slope stability analysis using the laboratory test data to identify the best performing additive.

Four treatment methods comprising of 20% compost, 4% lime with 0.30% polypropylene fibers, 8% lime with 0.15% polypropylene fibers, and 8% lime have been used in the surficial treatment methods. All these methods and their effectiveness were addressed by series of research tasks which are outlined as following:

1.2.1 Research Tasks

- Selection of two dam sites having history of surficial failures and having different types of soils. Joe Pool Dam and Grapevine Dam of USACE are selected for the research study as they underwent number of surficial failures in the past from the beginning of construction of these dams. Besides, the dam sites are having closer proximity to UTA.

- Construction of test sections at both the dam sites using four types of stabilizing agents in addition to one control section for relative performance study. The stabilizing additives used for field trial are 20% compost, 4% lime with 0.30% polypropylene fibers, 8% lime with 0.15% polypropylene fibers and 8% lime.
- Quality control and Quality Assessment tests were carried out to ensure that the field construction activities were executed as per standards. Besides close monitoring of quality of work at construction site, the critical engineering properties of test section material was compared with the laboratory test data of preconstruction soil samples.
- Instrumentation of the test sections with moisture probes, temperature probes and vertical inclinometers.
- Monitoring of data obtained from field instruments, conducting elevation survey and monitoring of sections for cracks with the help of digital images and carrying out image analysis studies.
- Analyzing field data and conducting model studies using computer software PLAXFLOW and GSTABL7.
- Selection of the best field performing additive(s) by conducting a detailed analysis of the field monitoring data and analytical studies.
- Discussion of feasibility for large scale implementation in the field for prevention of surficial failures of slopes of earthfill dams and extending the results of this research to highway embankments and cut slopes.

1.3 Organization of Dissertation

The dissertation consists of 8 chapters. The units indicated are mostly SI units and the results in English units are indicated in parenthesis wherever feasible. Some of the graphs and the drawings prepared were originally in English units and the same are presented here with no alterations.

Chapter 1 consists of introduction of the surficial failures and the proposed methodology of conducting research aimed at mitigation of surficial failures by preventing desiccation cracking and improving shear strength.

Chapter 2 comprises of review of literature relevant to the problems of natural and engineered slopes, slope engineering dealing with design aspects and stability analysis. Details of previous research conducted in the areas of rainfall induced slope failures, influence of soil suction on slope stability, typical case studies of slope failures, current practices of slope stabilization are also presented.

Chapter 3 provides the experimental program aimed at discussing the laboratory results conducted on the preconstruction borrow soil samples of Joe Pool Dam and Grapevine Dam (McCleskey, 2005). All the tests were repeated on the field samples obtained during construction of test sections. The results of tests conducted on all the five test section samples from each dam site are used for QC/QA studies. Additional laboratory tests were performed on the field samples obtained from test sections and the results are used for slope stability analysis.

Chapter 4 illustrates various stages of construction of test sections duly comparing the salient features and engineering properties of both the dam sites.

Measures taken to ensure proper quality control and quality assurance are explained. The results of QC/QA studies are presented for both the dam sites, and the details of instrumentation and field monitoring program are discussed.

Chapter 5 presents the analysis of data collected from the Joe Pool Dam test sections for a period of one year. The data collected from the moisture probes, temperature probes is presented in the form of Tables and Figures. The nomenclature used for each test section is control section, 20% compost, 4% lime with 0.30% fibers, 8% lime with 0.15% fibers and 8% lime. The notation used for by McCleskey (2005) as indicated in Chapter 3 is not used for easier understanding. The data collected from moisture probes and temperature probes is analyzed to explain and compare the effectiveness of each admixture. The relative performance of each treatment is also studied and compared with the data obtained from inclinometer surveys, elevation surveys, Dynamic Cone Penetration (DCP) tests and digital image analysis. Each treatment type is ranked based on the analysis of data and details summarized.

Chapter 6 presents the data collected from Grapevine Dam for a period of three months. Results of physical model studies and supplementary laboratory studies conducted on field soil samples is presented and the effect of wetting and drying on the desiccation cracking pattern and swell potential is discussed.

Chapter 7 presents the finite element model study conducted with the help of PLAXFLOW software to study the influence of rainfall infiltration in saturating the soil mass. Three typical cases of no infiltration, desiccation with high intensity short time rainfall and desiccation with high intensity long time rainfall are studied and results discussed. Stability analysis is carried out for all the three cases using GSTABL7 software program and the factor of safety for each case is calculated and the results are illustrated and discussed in detail.

Chapter 8 presents the summary of the research study, conclusions drawn from the analysis of field data and recommendations for further research.

A list of references indicating the source of information for all the above chapters and biographical information is presented after Chapter 8.

CHAPTER 2

REVIEW OF LITERATURE

2.1 Introduction

Slopes may be either engineered or natural. Engineered slopes include mainly embankments, cut slopes and retaining walls (Abramson et al. 2002). Slope engineering basically involves engineering sustainable slopes of both soil and rock. It involves monitoring, construction, maintenance and repairs to slopes in a safe, effective and economical manner (Abramson et al. 2002).

An essential part of slope engineering is slope stability analysis. The primary purpose of slope stability analysis is to contribute to safe and economic design of manmade slopes of excavations, highway and railway embankments, cut slopes, lands fills and spoil heaps (Abramson et al. 2002).

Slope stability evaluations of natural slopes are concerned with identifying critical geological, material, environmental and economic parameters in addition to nature, magnitude and frequency of potential slope problems (Abramson et al. 2002). In few cases, highways and railways located in hill ranges and valleys are in jeopardy when the slope is not stable (Wyllie and Mah, 2004).

This chapter provides a detailed insight in to the various important aspects of slope stability evaluations including landslides, failure of dam and highway embankment slopes with a specific emphasis on the surficial failures.

2.2 Landslides

Natural slopes that have been stable for many years may suddenly fail due to various reasons like loss of strength, stress changes, ground water changes and climatic conditions (Abramson et al. 2002). Failure of natural slopes is typically studied under the category of landslides. Landslide denotes a process of mass movement of rock, debris or earth down a slope forming materials including rock, soil, artificial fill, or a combination of these materials (Cruden, 1991). Landslides occur worldwide and cause casualties and billions of monetary losses annually. Figure 2.1 shows a schematic of a typical landslide and nomenclature associated with it.

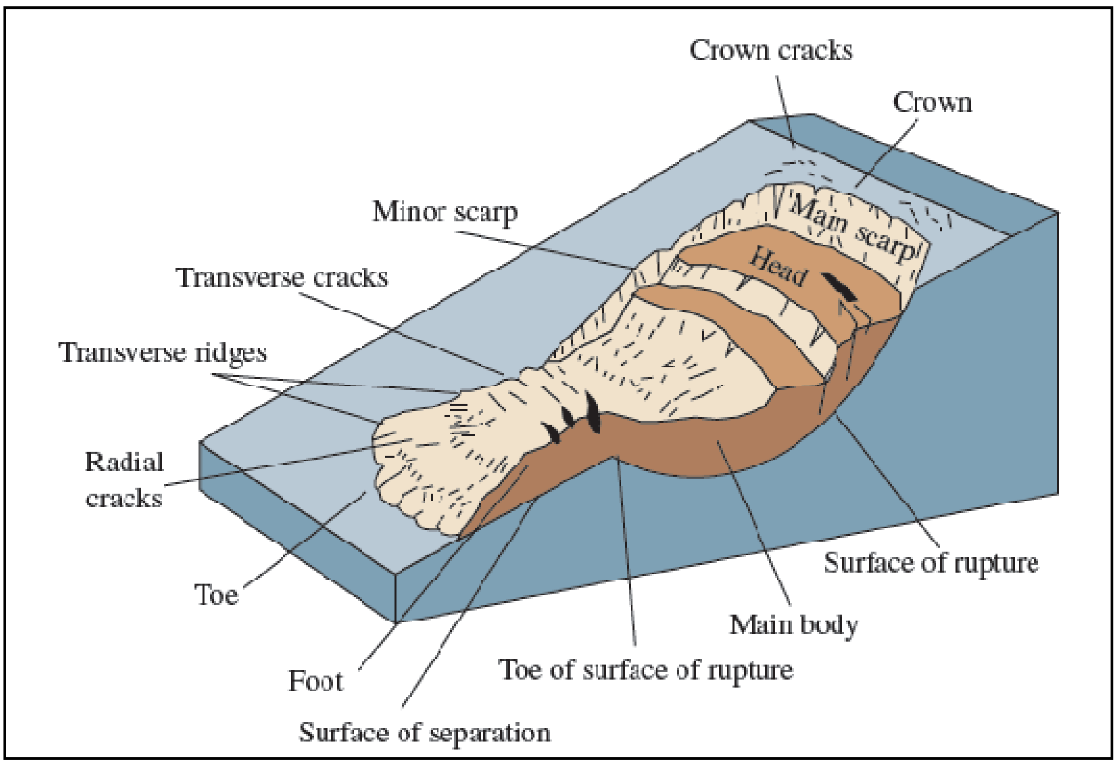


Figure 2.1 Nomenclature used for a landslide (Source: USGS)

2.2.1 Types of Landslides

One of the principal criteria for classifying landslides is based on the kinematics of a land slide. The classification is done observing how the movement is distributed through the displaced mass (Cruden and Varne, 1996). Several distinct types of landslides classified based on kinematics are slides, flows, topples, creep and lateral spread.

2.2.2 Triggering Mechanisms of Landslides

Landslides can have several causes, including geological, morphological, physical and human, but only one trigger (Cruden and Varne, 1996; Wieczorek, 1996). A trigger is an external stimulus such as intense rainfall, earthquake etc that causes almost an immediate response in the form of a landslide. Various triggering mechanisms of landslides are discussed below.

2.2.2.1 Intense Rainfall

Intense rainfall that last for few hours to several days has triggered innumerable landslides in many regions (Wieczorek, 1996). Studies showed that shallow landslides in soils and weathered rock often are generated on steep slopes during the more intense parts of a storm. Rainfall exceeding a threshold of 6.35 mm/hr (0.25 in. /hr) triggered shallow landslides that resulted in a damaging debris flow (Wieczorek, 1996).

During 1982, intense rainfall which lasted for about 32 hr in the San Francisco Bay region of California triggered more than 18,000 predominantly shallow landslides involving soil and weathered rock (Ellen et al. 1988; Wieczorek, 1996). Wieczorek (1996) mentioned that the rapid infiltration of rainfall, causing soil saturation and a temporary rise in pore water pressures, is the mechanism by which landslides are generated during storms.

Figure 2.2 shows a surficial failure on Boulder County road, 3.2 km (2 miles) west of Jefferson County Airport, Colorado, USA. The crown of slide was about 6 m (20 feet) above road level. Steepness of slope and heavy rain triggered the failure on 8th May, 1973. Study of number of slope failures revealed that most of the failures were preceded by rainfall.

Figure 2.2 Surficial failure triggered by heavy rains in Colorado in 1973
(Source: USGS)

2.2.2.2 Rapid Snowmelt

Rapid melting of a snowpack due to sudden warming spells or by rain falling on snow can add water to hillside soil. Studies revealed that the process of melting may provide a more continuous supply of moisture compared to infiltration due to rainfall (Mathewson et al. 1990; Wieczorek, 1996). In California, near Wrightwood, a steady thaw of a heavy snowpack for more than 40 days triggered mudflows (Morton et al. 1979).

Rain-on-snow events reduce the water content of the snowpack and add sufficient moisture content to soils to trigger failure. During mid-April 1982 and mid-March 1983, number of landslides occurred in central Nevada of California due to rain on snow events (Bergman, 1987; Wieczorek, 1996).

2.2.2.3 Water-Level Change

Rapid drawdown of water against a slope can trigger landslides of slopes situated along coastlines, canals and rivers. Rapid drawdown can occur when a river drops following a flood stage. Similarly, water level can change in a canal or a lake. Sea level drops following a storm tide. The slope is safe if pore pressures within the slope adjacent to falling water level dissipate quickly. Slope can fail if the pore pressures are not dissipated quickly which depends on the permeability of soil. Low permeability clays and silts are more susceptible for failure (Wieczorek, 1996).

Rapid drawdown triggered four landslides in very low permeable boulder clay in Fort Henry and Ardclooney embankments in Ireland (Wieczorek, 1996). The slides occurred after a drawdown of 1.1m (3.6 ft) in 10 days and on the previous day of slide, the drawdown was 0.35 m/day (1.14 ft/day) (Massarsch et al. 1987). Also, Koppejan et al. (1948) observed that excessive tidal differences of 2.8 to 4.6 m triggered wet sand flow in the coastal areas of Zeeland, Netherlands. Springer et al. (1985) has examined about 120 landslides sites along Ohio River system when he inspected over 6500 km (4038 miles). Many of these landslides were accounted for sudden drawdown and precipitation.

Between 1941 and 1953, about 500 landslides were noted along shores of Franklin D. Roosevelt Lake during and after construction of Grand Coulee Dam in Washington State (Wieczorek, 1996). Jones et al. (1961) reported frequent landslides during filling stage of reservoir when there was rapid drawdown on two occasions. Schuster (1979) reported even larger drawdown instances between 1969 and 1975 and subsequent earth spreads, earth flows and debris flows.

2.2.2.4 Effect of Increase of Ground Water Level on Slopes

Ground water level on hill slopes rises due to incessant rains or increase of water level in rivers, lakes, canals or reservoirs (Wieczorek, 1996). The rise of ground water table results in increase of pore water pressure and reduction of effective strength of the saturated slope.

This phenomenon will trigger landslides. Lane (1967) cited initial filling of Yellowtail Reservoir, Montana and of the Panama Canal as examples of landslides triggered by initial rising of water levels on natural or cut slopes. Rising ground water levels can also accelerate landslide movement. Lane (1967) observed acceleration of a landslide movement after commencement of initial filling of reservoir of Vaiont Dam, Italy.

The Mayunmarca landslide which occurred on April 25, 1974, dammed the Mantaro River in Peru. The rising water level behind this dam resulted in more landslides along the shores of the lake which destroyed a regional highway (Lee and Duncan, 1975). Wieczorek, (1996) further reported that there was evidence that the rise of ground water due to irrigation coupled with rainfall resulted in landslides.

2.2.2.5 Volcanic Eruption

Deposition of loose volcanic ash on hillsides results in erosion and intense rainfall triggered frequent debris flows (Kadomura et al. 1983). Irazu, a volcano in central Costa Rica, erupted ash almost continuously between March 1963 and February 1965. Intense rain and high runoff accompanied by sheet and rill erosion triggered more than 90 debris flows (Waldron, 1967). Monsoon and Typhoon rains triggered debris flows following the June 1991 eruption of Mt. Pinatubo volcano in the Philippines (Pierson, 1992). Some of the volcanic ash based slides were proved to be very catastrophic in nature.

2.2.2.6 Earthquake Shaking

Earthquake results in strong oscillations triggering landslides, rock falls, debris flows and avalanches (Wieczorek, 1996). Pore water pressures temporarily attain very high values. Increase of pore water pressures reduces the strength of soil. Lateral oscillations result in lateral spread of soil and earth structures. Earthquake occurred in Peru on May 31, 1970 with a magnitude of 7.7 on Richter scale triggered a huge debris avalanche from the north peak of Huascaran Mountain that buried many towns taking a toll of tens of thousands of lives (Plafker et al. 1971). The 7.5 magnitude earthquake of February 4, 1976 triggered more than 10,000 landslides including rock falls and debris slides from steep slopes of Pleistocene pumice deposits (Harp et al. 1981). Pumice deposits are known to lose their strength during seismic loading. Strong shaking increases stresses that may lead to break down of cohesion in cemented soils or brittle rocks such as tephra, loess or sandstone (Sitar and Clough 1983).

2.3 Engineered Slopes

Engineered Slopes or manmade slopes are usually categorized in to three groups viz., Embankments, Cut slopes and Retaining Walls (Abramson et al. 2002). A detailed discussion of various critical aspects from the perspective of slope engineering is presented as following:

2.3.1 Embankments

Fill slopes include earthfill dams, levees, highway embankments, railway embankments and landfills. The parameters involved in the design and analysis are more controlled and usually depend on the properties of borrow material used for the fills. The stability analyses of embankments and fills do not usually involve the same difficulties and uncertainties as natural slopes and cuts. Embankment fills consists of:

- Cohesion less soils (sands and gravels)
- Cohesive soils (silts and clays)
- A mixture of cohesion less soils and cohesive soils, gravels, and cobbles referred to as earth-rock mixture.

Organic and soft clays are usually avoided. Embankments are usually designed for the following conditions.

- All phases of construction
- The end of construction
- The long term condition
- Natural disturbances such as flooding and earthquakes
- Rapid drawdown (for water retaining structures like earthfill dams)
- Steady seepage condition for earthfill dams
- Settlement criteria and
- Bearing capacity of foundation soil

2.3.2 Cut Slopes

Cut slopes are often inevitable in engineering projects like highway and railway construction. In order to achieve economy and based on the stability analysis the angle of cut is usually determined. However, steep cuts are often necessary because of right-of-way and property line constraints (Abramson et al. 2002).

Landfills are a special case which involves both cut and fill slopes. The presence of organic matter, refuse, debris etc., affects the design of landfills.

2.3.3 Retaining Structures

Retaining structures are often constructed to support earth masses including slopes. They include Gravity walls, tie back or soil nailed walls, sheet piles and mechanically stabilized (MSE) walls. MSE walls use different types of reinforcement like steel, geotextiles, geogrids and other types of geosynthetics.

In this research, out of the above mentioned engineered slopes, the study is focused on the safety and stability analyses of earthfill dams and their slopes. The research is aimed at mitigating surficial failures on slopes of earthfill dams of United States Army Corps. of Engineers. As a part of the research, the principles governing the design of an earthfill dam are discussed here.

2.4 Design of Earthfill Dams

An Earth dam, also called earthfill dam should be designed in such a way that the dam is safe and stable during all phases of construction and operation of the reservoir.

Both short term and long term safety of the dam are to be given the highest priority while designing the dam.

2.4.1 Design Principles

The criteria adopted by the Bureau of Reclamation, United States Department of Interior are as following:

- The embankment, foundation, abutments and reservoir rim must be stable and deformations must be within permissible limits.
- Seepage flow through the embankment and foundation must be controlled to prevent generation of excessive uplift pressure, piping, instability, sloughing and other considerations.
- The reservoir rim must be stable so as to prevent triggering of any landslides.

- During the occurrence of IDF (Inflow Design Flood), there should be enough freeboard along with proper spillway arrangements to prevent overtopping.
- Camber should be sufficient to allow for settlement of foundation and embankment.
- The upstream slope must be protected against wave erosion, and the crest and downstream slope must be protected against wind and rain erosion.

2.4.2 Design Procedure

The United States Army Corps. of Engineers (USACE) extensively dealt with various design aspects of earthfill dams and slopes.

USACE documented the design process events in their publication EM 1110-2-1902, dated 31st October, 2003 as detailed below.

- Exploration of soil of foundations and borrow areas.
- Testing of soils in laboratory and in situ tests.
- Preparation of idealized cross section with details of subsurface.
- Measurement or Prediction of seepage and groundwater conditions.
- Selection of relevant loading condition for analysis.
- Selections of trial slip surfaces and computation of factors of safety until location of critical slip surface.
- Comparison of computed factor of safety with the design criteria.
- Proper drafting of specifications, execution of work and quality control/ quality assurance.

2.4.3 Unsatisfactory Slope Performance

USACE has identified the following criteria for a slope to be classified as an unsatisfactory slope.

2.4.3.1 Shear Failure

A shear failure involves sliding of a portion of an embankment and sometimes an embankment along with foundation. A shear surface is conventionally considered to occur along a discrete surface as assumed in stability analysis. Failure surfaces are frequently approximated as circular in shape.

Where zoned embankments or thin foundation layers overlying bedrock are involved, or where a weak strata exist within a deposit, the failure surface may consist of interconnected arcs and planes.

2.4.3.2 Surface Sloughing

Surface sloughing is a kind of shear failure in which a surficial portion of the embankment moves downslope. This kind of surficial failure is considered as a maintenance problem as it does not affect the structural capability of the embankment. If such repairs are not carried out, they can become progressively larger and may be a potential threat to the safety of dam.

2.4.3.3 Excessive Deformation

Some of the cohesive soils need larger strains to develop peak shear resistance. Consequently, the soils deform excessively when loaded. Emphasis is needed to be placed on stress-strain response curve during the design of cohesive embankment and foundation soil.

Strains of more than 15% are considered to be causing excessive deformation (EM 1110-2-1902, dated 31st October, 2003). In such cases, it may be ideal to use the shear strength value at lower strains during design. Excessive settlement may also occur when the cohesive soils are compacted on dry side of optimum moisture content (EM 1110-2-1902, dated 31st October, 2003). Compaction of cohesive soils on dry side of optimum moisture content has significant influence on the stability and seepage conditions of dam as it will induce brittle stress-strain behavior and cracking of embankment (EM 1110-2-1902, dated 31st October, 2003).

When large strains are required to develop shear strengths, surface movement measurement points and piezometers should be installed to monitor movements and pore water pressure during construction. Alternatively, the cross section should be modified or rate of fill placement should be altered.

2.4.4 Liquefaction

Soil Liquefaction reduces shear strength due to shear-induced pore water pressures. However, coarse grained soils are more prone for liquefaction. Cohesive soils that satisfy any of the following criteria are not susceptible for liquefaction.

- 20% soil finer than 0.005 mm.
- Liquid limit greater than or equal to 34 or
- Plasticity Index greater than or equal to 14.

2.4.4.1 Piping and Internal Erosion

Erosion and piping can occur when hydraulic gradients at the downstream end of a hydraulic structure are large enough to move soil particles.

2.5 Slope Stability Analyses

In the last few years, the availability of computers and new software programs has brought revolution in the slope stability analyses leading to an improved understanding of the mechanics of slope stability (Van Impe and Verastegui, 2007). Several methods are practiced for computation of slope stability and few important methods are discussed as following:

2.5.1 Limit Equilibrium Methods

Usually, stability of a slope is expressed in terms of factor of safety computed by means of limit equilibrium methods. The principles underlying these methods are enumerated by Van Impe and Verastegui (2007) as follows.

- A potential failure surface is postulated.
- The shear stresses are calculated by means of statics.
- The calculated shear stresses are compared with available shear strength.
- Factor of safety which is the ratio of shear strength and shear stresses is obtained.
- The failure mechanism with the lowest factor of safety is computed by iteration.

In the simplest case, idealized slopes are assumed to fail along planes or circular sliding surfaces as shown in Figure 2.3 and Figure 2.4 respectively.

More complex failure surfaces can be proposed and analyzed when slope conditions is not uniform (Gray and Sotir, 1996).

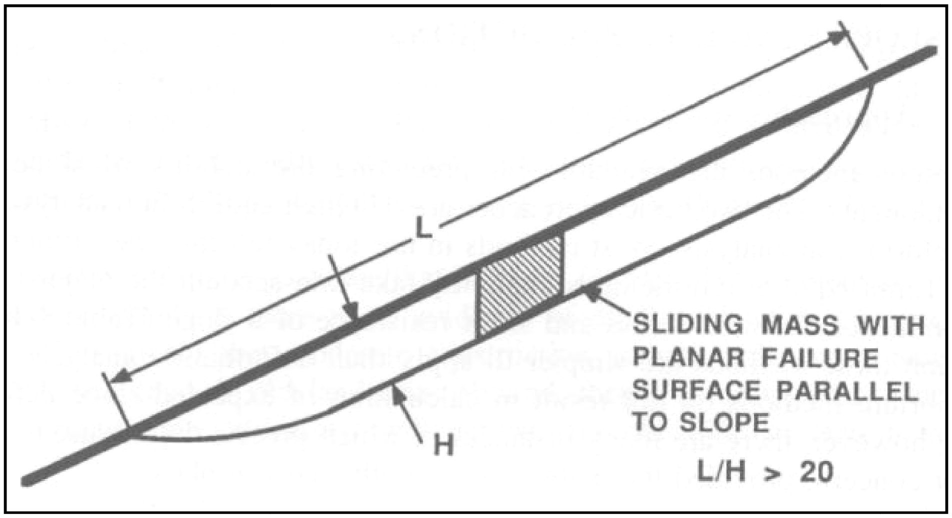


Figure 2.3 Translational failure (Infinite slope model) (Source: Gray and Sotir, 1996)

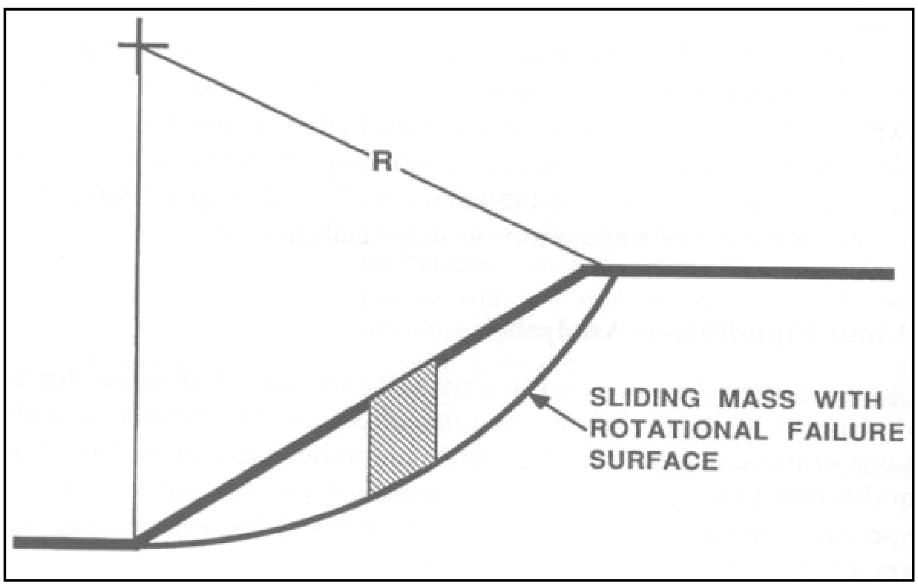


Figure 2.4 Rotational failure (Circular arc model)
(Source: Gray and Sotir, 1996)

A slope that extends for a relatively long distance with consistent subsoil profile may be analyzed as an infinite slope and the limit equilibrium method can be applied readily (Abramson et al. 2002).

2.5.1.1 Infinite Slopes in Dry Sand

A typical slice showing failure surface for a slope in dry sand is shown in Figure 2.5.

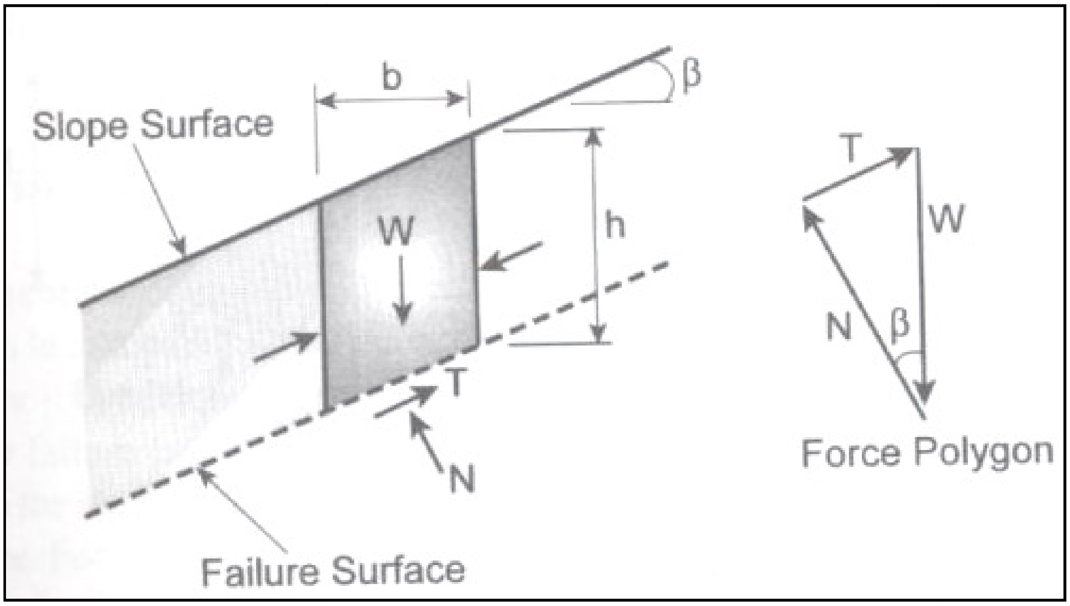


Figure 2.5 Infinite slope failure in dry sand (Source: Abramson et al. 2002)

Normal force $N = W \cos \beta$ (2.1)

Driving force $T = W \sin \beta$ (2.2)

Shear strength $S = N \tan \phi$ (2.3)

Where, $W = \gamma bh(1)$, b and h are width and height of the slice, γ is the unit weight of dry sand and W is the weight of slice

Factor of Safety = $\dfrac{ShearStrength}{ShearStress} = \dfrac{N \tan \phi}{W \sin \beta} = \dfrac{\tan \phi}{\tan \beta}$ (2.4)

In the case of cohesion less soil (dry sand) as an embankment fill material, the factor of safety is independent of height of embankment and depends only on angle of internal friction, ϕ, and the angle of the slope, β. The maximum slope angle is limited to angle of internal friction.

2.5.1.2 Infinite Slopes in $c-\phi$ Soil with Seepage

If a slope is made up of soil having both cohesion and internal friction, and the slope is saturated with seepage parallel to the slope surface as shown in Figure 2.6, the limit equilibrium concept may be applied to compute the factor of safety.

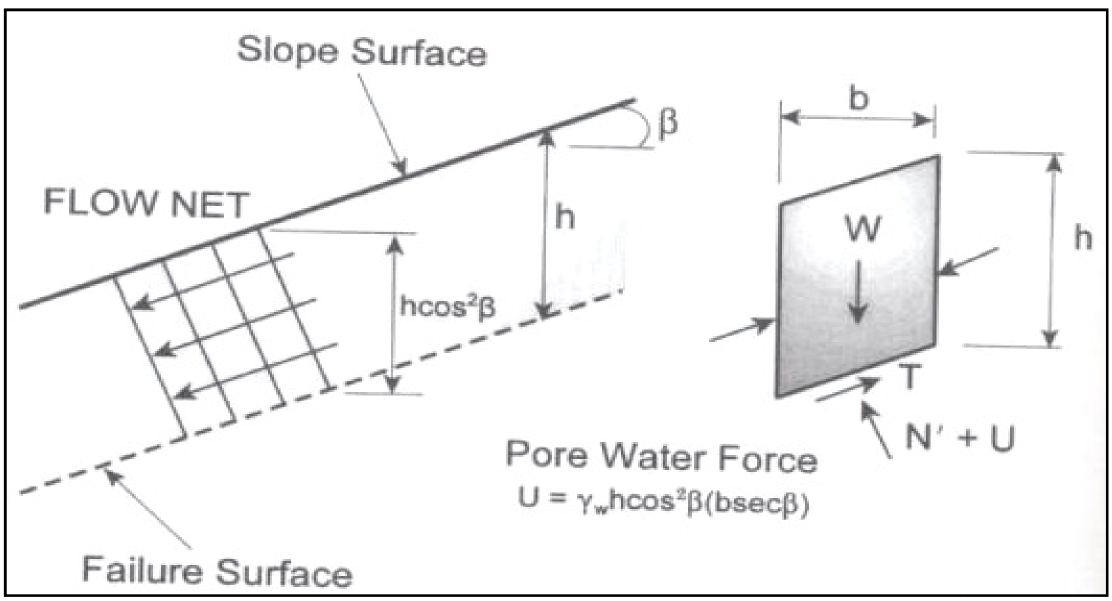


Figure 2.6 Infinite slope failure in soil with seepage parallel to slope
(Source: Abramson et al. 2002)

Factor of Safety, $F = \dfrac{c' + h(\gamma_{sat} - \gamma_w)\cos^2\beta \tan\phi'}{\gamma_{sat} h \sin\beta \cos\beta}$ (2.5)

Where γ_{sat} is the saturated unit weight of soil, c' and ϕ' are cohesion and friction angle expressed in terms of effective stresses respectively. When there is no seepage parallel to flow the above equation reduces to

$$F = \frac{c + \gamma h \cos^2\beta \tan\phi}{\gamma h \sin\beta \cos\beta} \quad (2.6)$$

Where γ is the total unit weight of soil.

2.5.1.3 Method of Slices

Unlike free body approach, in the method of slices by Simplified Bishop Procedure and various other procedures, the soil mass is subdivided in to number of vertical slices as shown in Figure 2.7 and equilibrium of each of the slices is calculated.

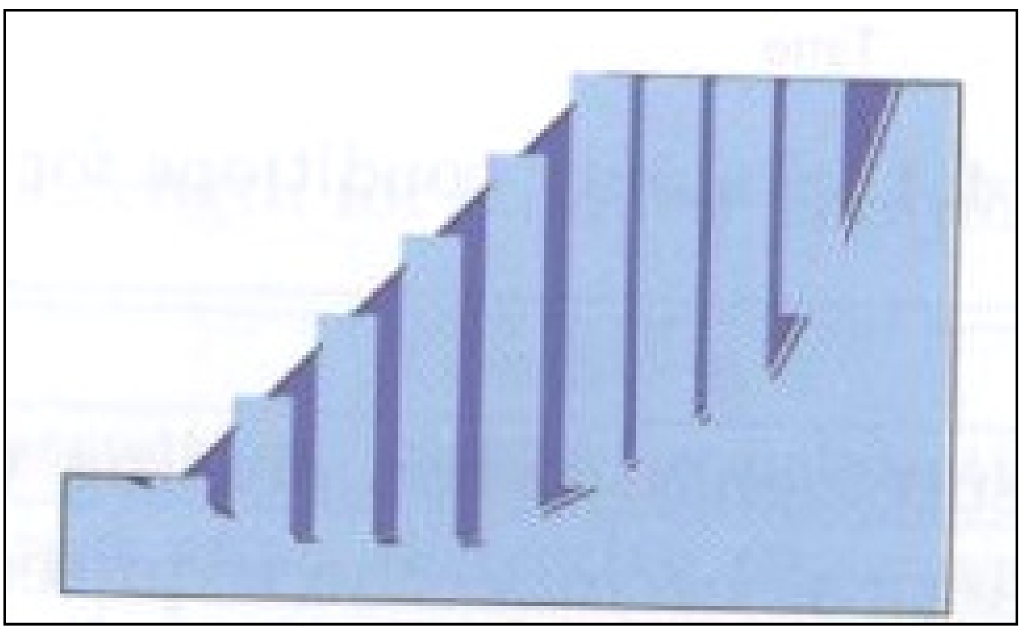


Figure 2.7 Method of slices. (Source: Van Impe and Verastegui, 2007)

The equation for factor of safety in terms of effective stress is,

$$F = \frac{\sum\left[c'\Delta l + (W\cos\beta - u\Delta l)\tan\phi'\right]}{\sum W\sin\beta} \tag{2.7}$$

The method of slices include ordinary method of slices, simplified Bishop procedure and others such as Morgenstern and Price method, Spencer's method, and Taylors' charts (Van Impe and Verastegui, 2007).

In the ordinary method of slices, the forces between two adjacent slides are neglected. Figure 2.8 illustrates the differences among various methods (Van Impe and Verastegui, 2007).

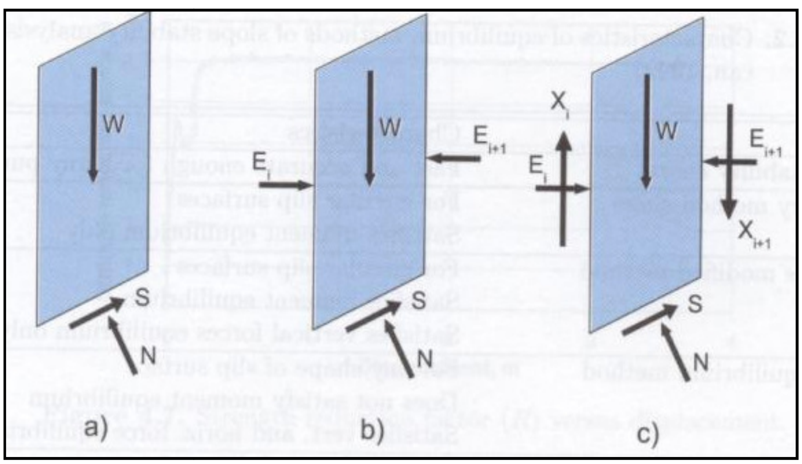


Figure 2.8 Free-body equilibrium of slices (a) Ordinary method of slices, (b) Simplified Bishop procedure, (c) More refined methods (Van Impe and Verastegui, 2007)

USACE preferred use of Spencer's method to Force equilibrium method. Spencer's method assumes that all side forces are inclined at the same angle. In the modified Swedish method, the side force's inclination is not assumed, but calculated as a part of the solution (EM 1110-2-1902, dated 31st October, 2003). USACE recommended use of the Spencer's method where complete solution is required.

Apart from circular failure surface, USACE recommended use of Wedge Method as illustrated in Figure 2.9

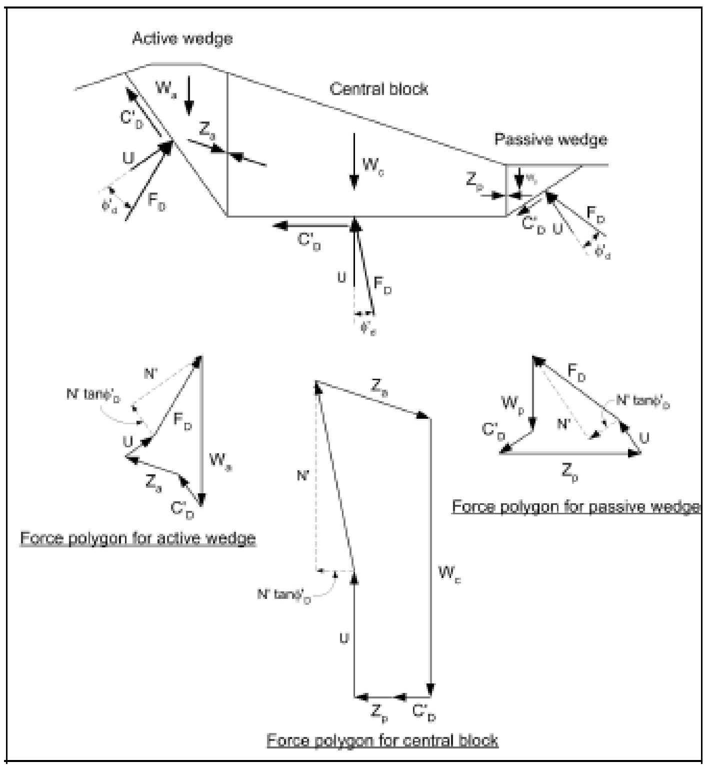


Figure 2.9 Forces and equilibrium polygons for wedge method (Source: USACE)

2.5.2 Strength Reduction Methods

The following definition of factor of safety has gained a lot of importance (Van Impe and Verastegui, 2007). The factor of safety is a factor by which the shear strength parameters may be reduced in order to bring the slope into a state of failure.

This definition has given rise to a new technique called Strength Reduction Method (SRM) and it was implemented by both finite element and finite difference computer programs (i.e., PLAXIS, FLAC) (Van Impe and Verastegui, 2007).

Finite element slope failure prediction is performed by using two reduced shear strength parameters:

$$c_R = \frac{c}{R} \tag{2.8}$$

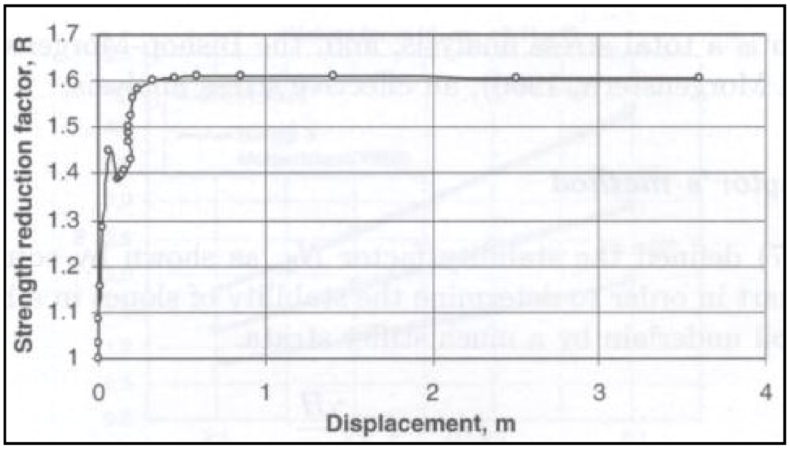
$$\tan \phi_R = \frac{\tan \phi}{R} \tag{2.9}$$

Where, c = cohesion

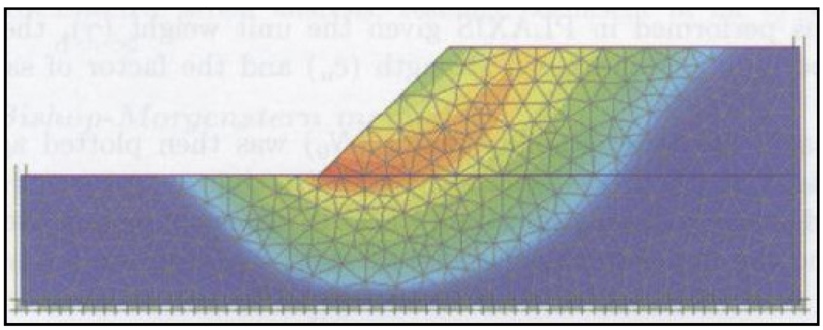
Φ = Friction angle, R = Reduction factor, c_R = Reduced cohesion and

Φ_R = Reduced friction angle

The analysis starts with a value of R equal to 1. R is increased subsequently and the shear strain and displacements are evaluated for each step until failure is reached as shown in Figure 2.10. The shear strength factor at failure is known as critical strength reduction factor and corresponds to an overall safety factor of slope (Van Impe and Verastegui, 2007). Comparison of SRM with other methods has shown that the SRM results are almost in close agreement with the results obtained by the limit equilibrium methods (Van Impe and Verastegui, 2007). The clear advantage of using SRM is that the method does not impose any restriction to the geometry of the failure surface (Van Impe and Verastegui, 2007). In limit equilibrium methods, most of the time, the failure surface is assumed to be circular or parallel to slope (Van Impe and Verastegui, 2007).



(a)



(b)

Figure 2.10 Strength reduction methods. (a) Displacement vs. Strength reduction factor, (b) A Plaxis output (Source: Van Impe and Verastegui, 2007)

2.6 Surficial Failures

Stability of slopes has been a great concern to Geotechnical engineers in view of landslides and slope failures of earth dams, highway embankments and cut slopes. Failures of natural slope are often associated with loss of life and property. Failures of earthfill dams due to seepage have proved to be occasionally catastrophic in nature like the failure of Tetan dam and failure of baldwinhill dam. Earthfill dams, levees, highway embankments, railway embankments, cut slopes, land fill slopes are also susceptible for another kind of slope failure known as surficial failure or surface sloughing as referred in section 2.4.3.2. These failures are classified as shallow slope failures as the average depth of failure varies from 0.3-1.2 m (1-4 ft) (Day, 1996). In many cases the failure surface is parallel to the slope face (Day, 1996). A typical surficial failure occurred at Grapevine Dam in Texas State of USA is shown in Figure 2.11.

Figure 2.11 A Surficial slope failure at the Grapevine Dam, Texas, USA

2.6.1 Surficial Failure Mechanism

Surficial failure mechanism is typical and is different from the mechanism of deep seated slope failures. Desiccation cracking is the root cause of surficial failures. Day (1996) conducted several studies and reported the mechanism of surficial failures as following:

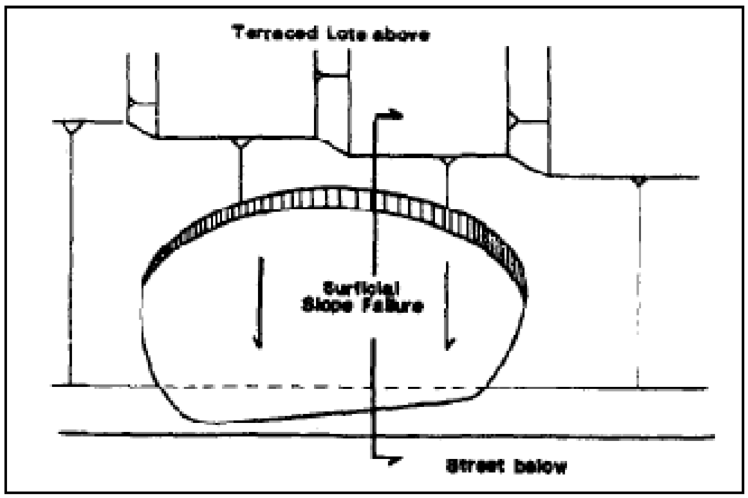
- Slope face gets desiccated and shrinkage cracks develop during hot and dry period when the stresses exceed the tensile strength of soil. Rainwater infiltrates through the cracks and cause swelling of soil increasing void ratio.
- Pore water pressure increases causing reduction of matric suction. Soil mass gets saturated and increases shear stresses.
- Continued rainfall causes seepage flow parallel to slope. Shear strength starts reducing and a surficial failure can occur instantly.
- These failures are common even in highway embankments and cut slopes.

These failures can be sudden and unexpected. Further studies show that the drained cohesion approaches zero as drained shear conditions prevail and the shear strength depends only on drained friction angle (Dronamraju et al. 2008).

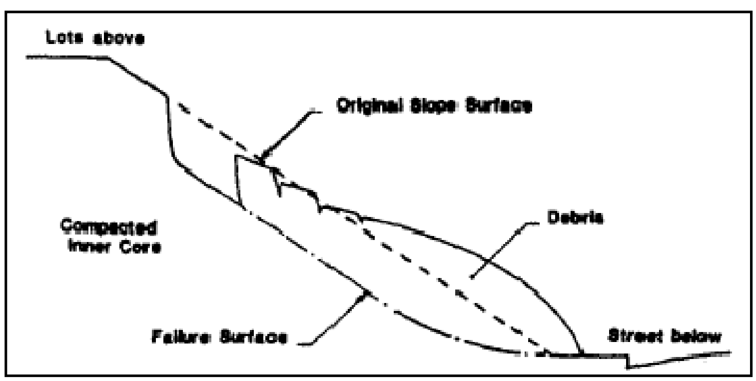
A photograph showing the surficial failure observed during March 2008 on SH 360 near DFW airport, Dallas, Texas, USA (Courtesy:TxDOT) is shown in Figure 2.12.

Figure 2.12 Surficial failure of a cut slope on SH 360 near the DFW airport, Texas

A schematic showing the surficial failure is shown in Figure 2.13. Sometimes these failures are preceded by surficial cracks or other signs of imminent failures (Day, 1996).



(a)



(b)

Figure 2.13 Surficial failure (a) Plan view (b) Cross-sectional view (Source: Day, 1996)

2.7 Desiccation Cracks

Physico-chemically induced cracking may be divided in to three groups, viz., syneresis cracks, cracks induced by freeze-thaw cycles and cracks induced by desiccation (Omidi et al. 1996). Syneresis cracks are induced by changes in the inter-particle forces resulting from replacement of interstitial water with a low dielectric organic solvent or highly aqueous solution (Brown and Anderson, 1983; Omidi eta al. 1996). Various Studies have also shown that freeze-thaw cycles result in formation of cracks and the net result is increase in hydraulic conductivity of soils. During summer or periods of drought, desiccation cracks are induced by evaporation of water and the consequent shrinkage of the soil (Omidi et al. 1996; Othman et al. 1994; Bowders and McClelland, 1994).

2.7.1 Mechanism of Desiccation Cracking

Westergaard (1926) studied the formation of cracks in soil-cement mixture and concluded that the shrinkage and ambient temperature are the vital factors involved. George (1969) showed that effects of temperature on cracking are insignificant compared to the influence of change in moisture content. He found that tensile stresses develop in the soil due to shrinkage and shrinkage stresses reach the maximum value in the early stage of drying near the surface and the shrinkage stresses decrease rapidly with depth. He stated that the stress is relieved either by surface cracking or plastic flow in material. George (1969) explained the mechanism of desiccation cracking as a failure of material in tension. That indicates that when the tensile stresses due to shrinkage exceed the tensile strength of soil, the desiccation cracks form.

2.7.2 Extent of Desiccation Cracking

Clay soils tend to shrink during drying due to development of substantial matric suction in the pore structure of fine grained soils. Nahlawi and Kodikara (2006) reported that soil dries faster when the humidity is lower. Wilson et al. (1990) explained that the rate of evaporation from the soil surface is proportional to the difference in relative humidity of drying environment and the soil pores at the drying surface.

The extent and depth of cracks depend on various factors like temperature, humidity, plasticity of clay, and extraction of moisture by plant roots (Day, 1996). Desiccation cracks up to 3 cm (1.18 in.) wide and 2 m (6.6 ft) deep were reported (Ritchie and Adams, 1974; Bronswijk, 1988). Lecocq and Vandewalle (2002) reported that widest cracks are those that appear first when crack patterns were observed successively.

2.7.3 Effect of Tension Cracks

Omidi et al. (1996) illustrated that permeability increases with desiccation as shown in Figure 2.14.

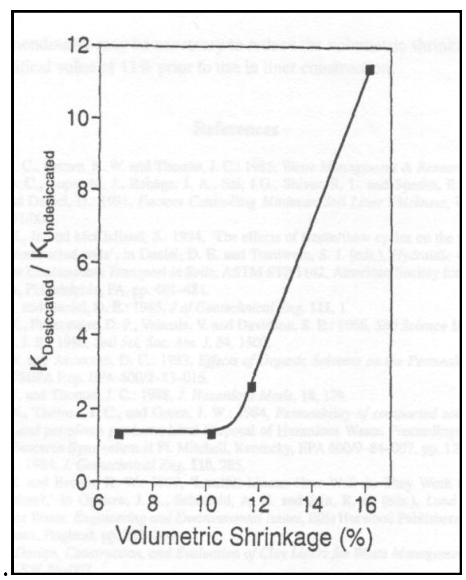
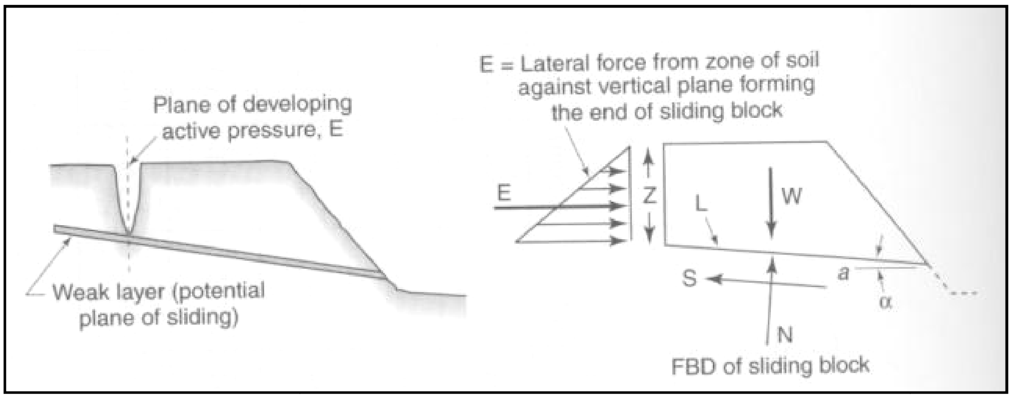


Figure 2.14 Changes in the hydraulic conductivity of soils caused by desiccation (Source: Omidi et al. 1996)

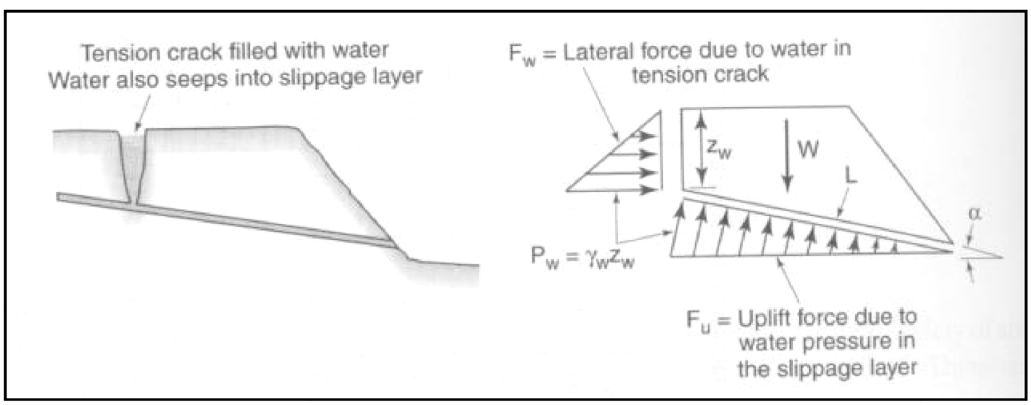
Laboratory experiments on different combination of soil mixtures were conducted by them with the help of permeameters and they arrived at a conclusion that soils with high volumetric shrinkage are more susceptible for desiccation.

2.7.4 Effect of Tension Cracks on Slope Stability

McCarthy (2002) has explained the effect of tension crack as illustrated in the Figure 2.15.



(a)



(b)

Figure 2.15 Block failure (a) Contribution to failure along weak plane by active pressure zone at top of sliding block (b) Contribution to failure where water pressure develops in the tension crack and slippage layer (Source: McCarthy, 2002)

The shrinkage cracks formed due to desiccation during dry season get filled up during precipitation and the water exerts hydrostatic pressure resulting in sliding of block away from the crack duly causing an increase of width of shrinkage crack.

McCarthy (2002) has come up with two formulae for both the cases described in the Figure 2.15 considering the water pressures in the tension crack and uplift pressure exerted by seepage forces along the slippage layer. Equation 2.10 is for case (a) and Equation 2.11 is for case (b) as illustrated above respectively.

$$FS = \frac{cL + (W\cos\alpha + E\sin\alpha)\tan\phi}{W\sin\alpha + E\cos\alpha} \tag{2.10}$$

$$FS = \frac{cL + (W\cos\alpha - F_u + F_w\sin\alpha)\tan\phi}{W\sin\alpha + F_w\cos\alpha} \tag{2.11}$$

Where α is the slope angle and other notations are as shown in the Figure 2.15.

2.7.4.1 Consideration of Tension Crack during Slope Stability Analysis

It is a common practice to neglect the resistance offered by the slip circle near the tension crack as shown in Figure 2.16 (EM 1110-2-1902, dated 31st October, 2003).

When a strength envelope with a significant cohesion intercept is used in slope stability computations, tensile stresses appear in the form of negative forces on the sides of slices and sometimes on the bases of slices.

Such tensile stresses are almost always located along the upper portion of the shear surface, near the crest of the slope, and should be eliminated unless the soil possesses significant tensile strength because of cementing which will not diminish over time (EM 1110-2-1902, dated 31st October, 2003).

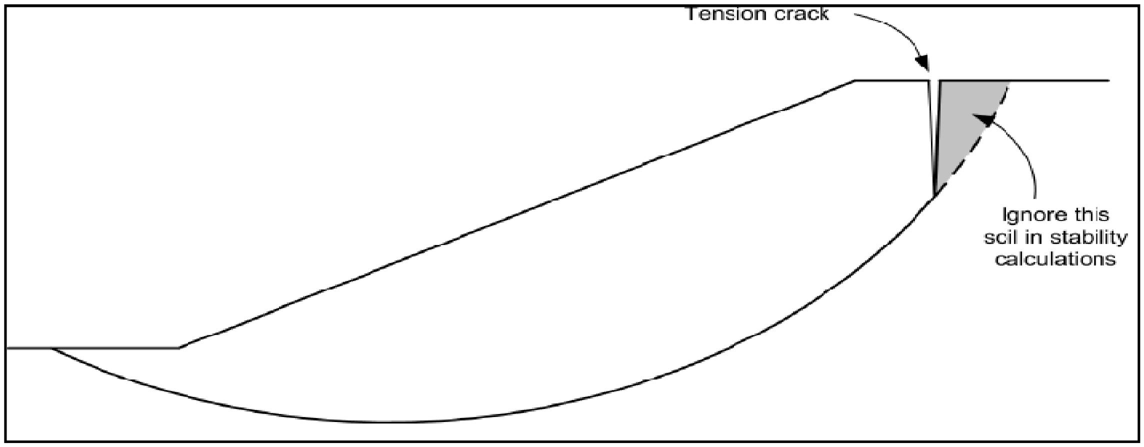


Figure 2.16 Introduction of vertical tension crack to avoid tensile stresses in cohesive soils (Source: USACE)

When a strength envelope with a significant cohesion intercept is used in slope stability computations, tensile stresses appear in the form of negative forces on the sides of slices and sometimes on the bases of slices.

Such tensile stresses are almost always located along the upper portion of the shear surface, near the crest of the slope, and should be eliminated unless the soil possesses significant tensile strength because of cementing which will not diminish over time (EM 1110-2-1902, dated 31st October, 2003).

The tensile stresses are easily eliminated by introducing a vertical crack of an appropriate depth as shown in Figure 2.16. The soil upslope from the crack (to the right of the crack in Figure 2.16) is then ignored in the stability computations.

This is accomplished in the analyses by terminating the slices near the crest of the slope with a slice having a vertical boundary, rather than the usual triangular shape, at the upper end of the shear surface (EM 1110-2-1902, dated 31st October, 2003).

2.8 Unsaturated State of Slopes

Compacted fill soils are used for the construction of earthfill dams, highway embankments and airport runways that are unsaturated in condition (Fredlund and Morgenstern, 1977). Soil below the ground water table is in a state of saturation and soils above the ground water table are in unsaturated condition. In case of earthfill dams, soil below the phreatic line is saturated and above the phreatic line is unsaturated.

Pore pressures are positive within the saturated zone and pore pressures are negative in unsaturated soils. Negative pore water pressures in the soil are the result of change in micro-climate conditions. These changes are more dominant closer to the top surface of the soil. The negative pore water pressures result in de-saturation, shrinkage, and cracking of soil (Fredlund, 1987). As shown in Figure 2.17, Pore water rises above the water table under capillary suction (Lu and Likos, 2004).

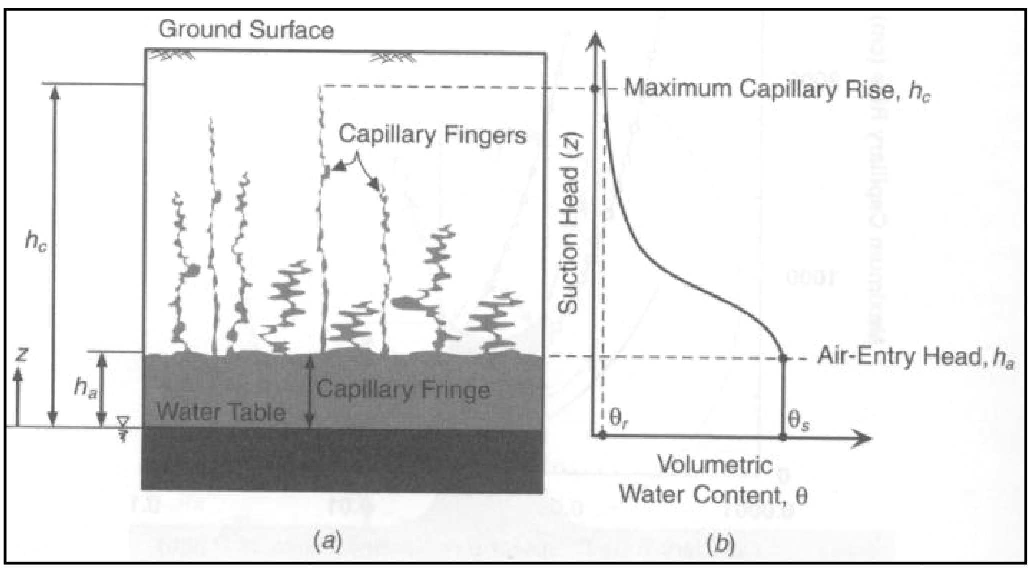


Figure 2.17 Capillary rise in an unsaturated soil (a) Conceptual illustration, (b) SWCC (Source: Lu and Likos, 2004)

The soil remains essentially saturated until the suction head reaches air-entry head within the capillary fringe zone. Above the air entry head, the water content decreases with increasing height though few capillary fingers are present as shown in the Figure 2.17.

2.8.1 Deformation and Flow Phenomena of Unsaturated soils

Swelling or Shrinking is the most vital deformation phenomenon of unsaturated soils in general and of expansive soils in particular. Expansive soils are the soils having a high swelling index, C_s, which are also subjected to frequent change in matric suction (u_a-u_w) (Fredlund, 1987). A structure resting on an expansive soil is subjected to heave or settlement depending up on the fluctuations in the moisture content and matric suction. The mechanism is illustrated in Figure 2.18

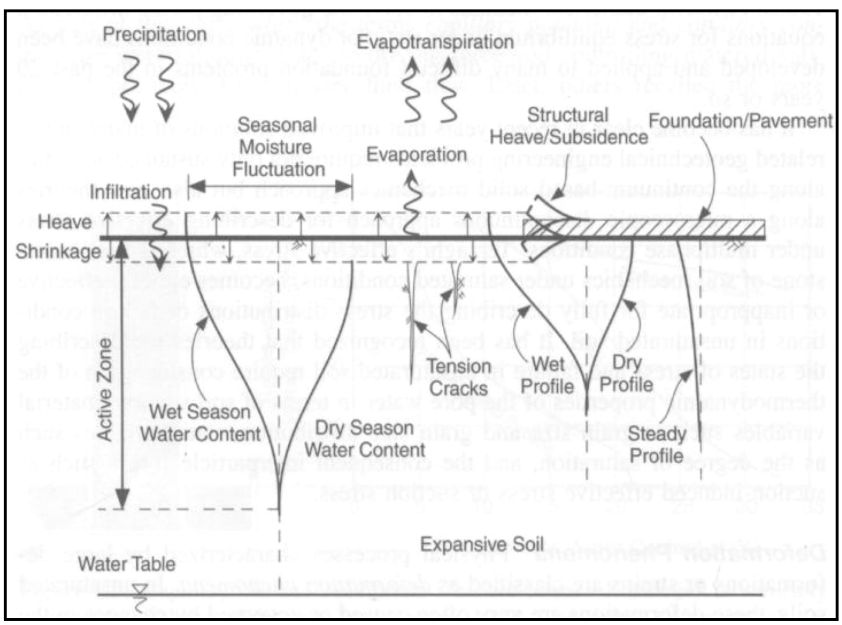


Figure 2.18 Deformation and fluid flow phenomena in a near-surface deposit of unsaturated expansive soil (Lu and Likos, 2004)

Infiltration, Evaporation and the seasonal fluctuation fall in to the category of unsaturated flow phenomenon. Desiccation influence leave the upper portion of soil profile cracked and unsaturated (Fredlund and Morgenstern, 1977).

2.8.2 Pore Pressure Variation

Surficial soils in many part of the world are classified as expansive soils, collapsible soils and residual soils which are unsaturated and having negative pore-water pressures with respect to atmospheric conditions (Fredlund and Rahadjo, 1988). Wetting and drying cycles result in swelling and shrinkage of soil. As a result of these changes, the pore water pressure distribution can take a wide variety of shapes as shown in Figure 2.19. The effect of changing pore pressure distribution is that the positive pore pressure decreases soil strength and suction increases soil strength.

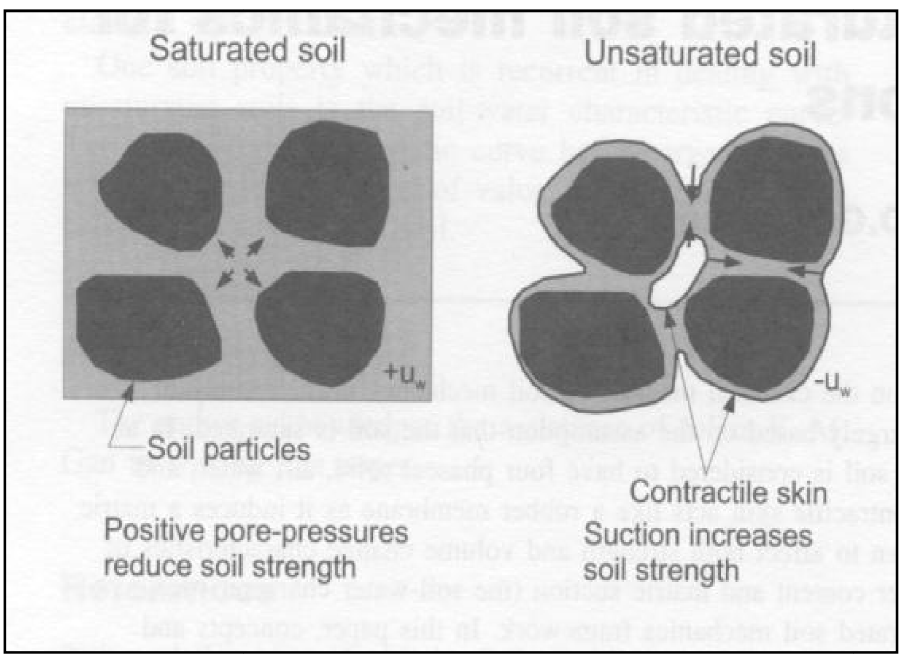


Figure 2.19 Stress distribution during desiccation of a soil (Source: Fredlund, 1978)

2.8.3 Effect of Contractile Skin to form Desiccation Cracks

Air-water interface which is commonly referred to as the contractile skin is considered the fourth phase in the study of stress conditions of unsaturated soils (Fredlund and Morgenstern, 1977). The most distinctive property of contractile skin is to exert a tensile pull. It behaves like an elastic membrane under tension interwoven throughout the soil structure (Fredlund, 1978).

Vegetation growth results in further drying out of soil by applying a tension to the water phase through evapo-transpiration. Most plants are capable of applying 1 to 2 MPa (10 to 20 atm) of tension to the water phase prior to reaching their wilting point. Evapo-transpiration results in further consolidation and de-saturation of the soil.

The tension in the water phase acts in all directions and can readily exceed the lateral confining pressure in the soil mass. At this point, a secondary mode of de-saturation commences (i.e. shrinkage cracking). When the soil is remolded in the compaction process, de-saturation is also the result of artificially subjecting the soil structure to tensile stresses (Fredlund, 1978).

2.8.4 Effect of Soil Suction on Stability of Slopes

A slope exhibits higher strength during dry season as the soil is in unsaturated state with negative pore water pressure or matric suction. Soil-water characteristic curve which relates volumetric water content of the soil with matric suction determines the failure mechanism. The phenomenon is effected by the flux boundary conditions viz., rainfall infiltration, evaporation and evapo-transpiration at the interface of soil and atmosphere (Rahardjo et al. 2007).

Fredlund (1978) presented the shear strength equation for unsaturated soils as following:

$$\tau = c' + (\sigma - u_a)\tan\phi' + (u_a - u_w)\tan\phi^b \qquad (2.12)$$

Where, τ = shear strength of an unsaturated soil

c' = effective cohesion

σ= total normal stress

u_a = pore-air pressure in the soil

u_w = pore-water pressure in the soil

σ' = angle of internal friction

ϕ^b = angle of internal friction with respect to changes in suction

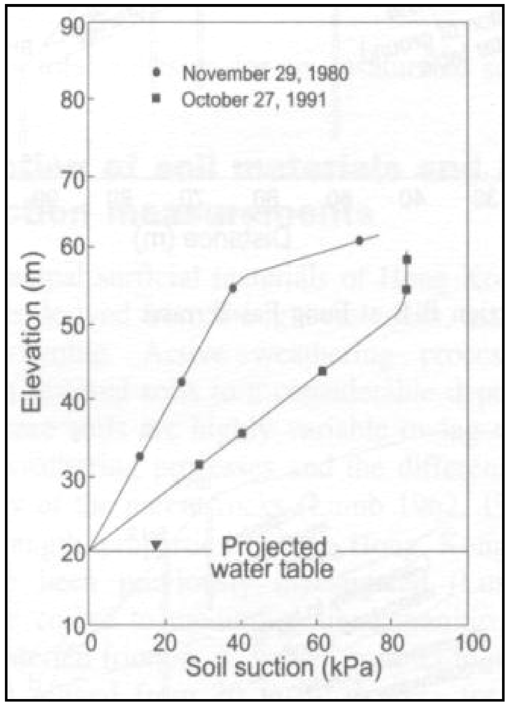
σ– u_a = total stress and

u_a-u_w is matric suction which is the difference of total suction and osmotic suction.

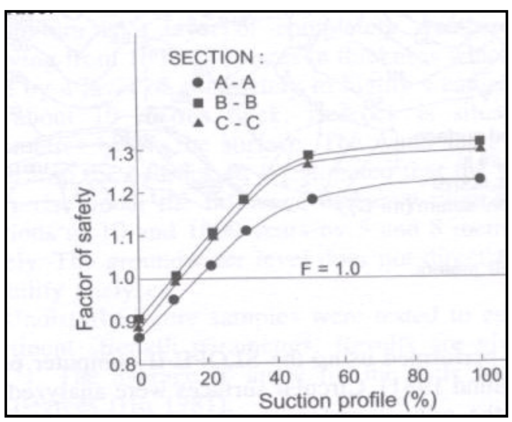
Ching et al. (1999) carried out stability analyses for two cut slopes of Hong Kong. One of the sites was Feng Fair Terrace which has a slope of 60º and a maximum height of 35 m (115 ft).

The slope contained residual soils and weathered granite. The bed rock was located below the slope surface at a depth of 20 to 30 m (66-100 ft). The water table was located near the bed rock. The average ϕ^b was assumed to be 15º. The soil suction was measured by field instrumentation with tensiometers. Figure 2.20 (a) shows the variation of soil suction above water table.

The factor of safety was computed and presented for Fung Fai Terrace slope in Figure 2.20 (b) for three cross sections A-A, B-B and C-C along the length of slope.



(a)



(b)

Figure 2.20 Fung Fai Terrace site, Hong Kong (a) Suction measurements (b) Factor of Safety (Source: Ching et al. 1999)

The computed factor of safety for the cut slopes at Fung Fai Terrace and the second site of Thorpe Manor were approximately 0.86 and 1.05 respectively when the effect of soil suction was neglected. The factor of safety increased to 1.01 and 1.25 when the soil suction was taken into consideration.

2.9 Effect of Rainfall on Surficial Failures

During a rainfall event, water infiltrates into the soil through the desiccation cracks. Infiltration increases pore water pressure which leads to reduction of shear strength triggering failure (Rahardjo et al. 1994, Cho et al. 2002).

As the wetting front increases, the permeability parallel to slope increases and thus the seepage occurs parallel to the slope (Day, 1996). Apart from rain fall intensity, other factors such as rainfall characteristics, antecedent precipitation, soil characteristics, topography also contribute to the failure of any slope (Church and Miles, 1987). The problem is aggravated as the weight of moist soil acts as surcharge. The resisting factors are the drained cohesion and internal friction.

The turfing and vegetation on the slopes too contributes to safety. Reduction of soil moisture content due to transpiration helps in gaining strength. It has been studied that the plant roots enhances shear strength of soil as reinforcement (Waldron, 1977; Day, 1993). Studies of natural and synthetic fiber reinforcement in sand proved increase in shear strength due to reinforcing effect (Gray and Ohashi, 1983; Day, 1996).

Many unsaturated slopes fail during heavy rains following reduction in matric suction and increase in pore water pressures. (Lim et al. 2006).

2.9.1 Suction Measurements at a Slope in Singapore

Lim et al. (1996) conducted suction studies at Singapore, with respect to rainfall at different depths on a soil having a plasticity index of about 30%. The average effective cohesion and friction angle were reported to be 30 kPa (0.62 ksf) and 26º respectively. The field and laboratory coefficient of permeability measured was 1.0×10^{-6} m/sec (3.3×10^{-6} ft/sec) and 1.0×10^{-9} m/sec (3.3×10^{-9} ft/sec) respectively at depths of 1.7 m to 1 m (5.6 – 3.3 ft). Higher value of permeability at field was attributed to the desiccation cracks on soil. The field section was having a width of 15 m (49 ft) and 25 m (82 ft) long along slope with slope of 30º and reducing to 12º to 15º near the toe. The site was originally having grass. Three test sections, each of 5 m (16.4 ft) width were constructed as shown in Figure 2.21.

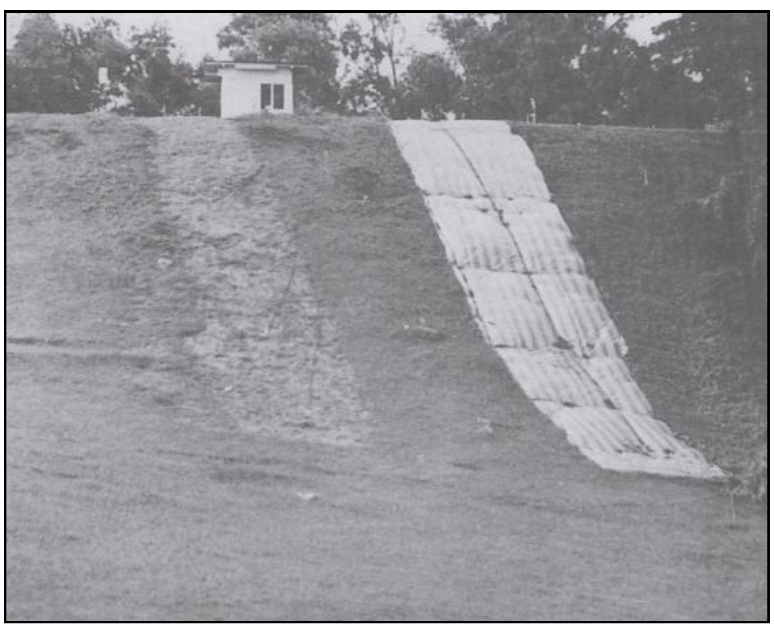


Figure 2.21 Influence of varying ground surface conditions (Source: Lim et al. 1996)

The test sections from left to right are

- Bare ground surface
- Grassed surface and
- Canvas over grass surface

The progressive change in the field matric suction was shown in Figure 2.22. Changes in the matric suction profile under bare surface were reported to be more significant than under the grass surface. There was very little variation of matric suction under the canvas covered surface.

For grass covered surface, the change in the matric suction near surface was reported to be more significant than at deeper depths owing to evaporation and evapo-transpiration. The presence of grass accelerated the removal of water and prevented advancement of water front.

For bear slope, there was only surface evaporation and the wetting front continued to greater depths at the end of each rainstorm. The propagation was up to a depth of 1.5 m (5 ft) or more. It was also observed that the matric suction measured at 1.5 m (5 ft) depth was relatively low.

It could be seen that the soil started showing the trend of changing to saturated condition from unsaturated condition at depths of about 1.5m (5 ft).

Piezometric observations showed that a perched water table probably developed at 1.5m (5 ft) below the ground surface. It can be concluded that with continued rainfall the soil at top few meters get saturated and pore pressures get increased.

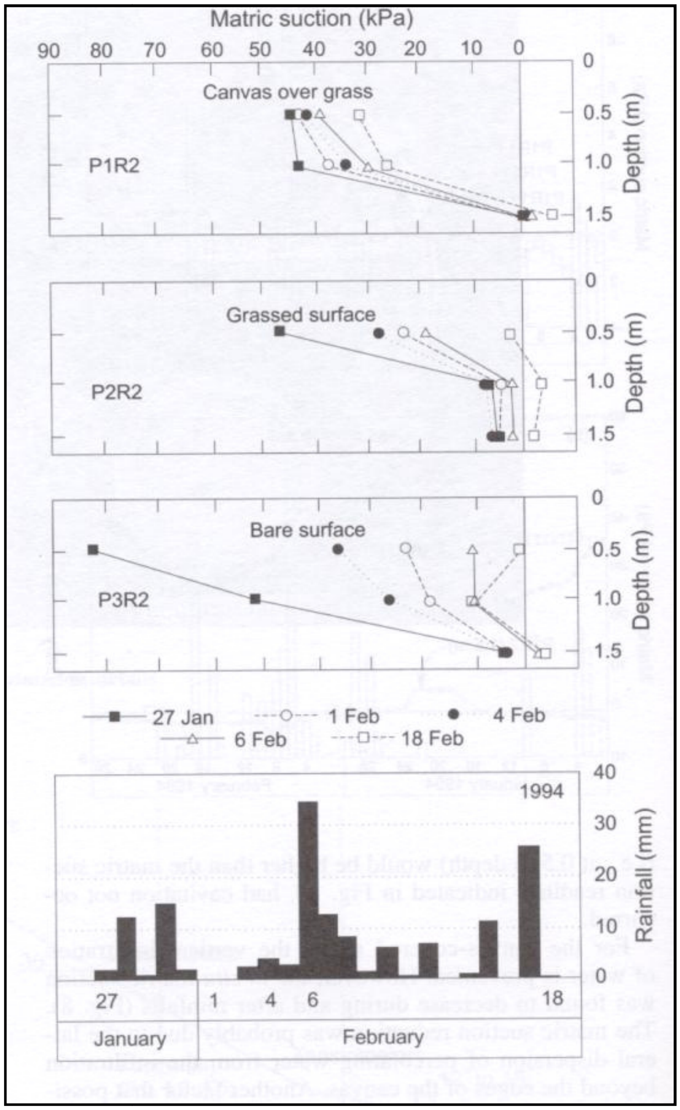


Figure 2.22 In-situ changes in suction due to rainfall (Source: Lim et al. 1996)

2.9.2 Study of the Effect of Antecedent Rainfall in Surficial Failures at Grapevine Dam

McCleskey et al. (2008) reported various failures occurred at Grapevine Dam in the state of Texas, USA. A study has been carried out by referring to rainfall data. It was observed that almost all the failures were preceded by rainfall events as shown in Table 7.5 and the detailed explanation is given in the Chapter 7.

2.10 Case Studies of Typical Slope Failures

It is pertinent to study few typical case studies of slope failures peculiar to earthfill dams and slopes. The analysis of these failures reveals different aspects of various causes of slope failures and helps conduct studies to find pragmatic solution for mitigating surficial slope failures.

2.10.1 Failure of Waco Dam during Construction

The Waco Dam was situated on Pepper Shale formation which was heavily consolidated stiff fissured clay. The height of the Dam was 25.9 m (85 feet). A slope failure occurred in Waco Dam, Texas during its construction in the month of October 1961 (Duncan and Wright, 2005). Figure 2.23 shows the failed portion of Waco Dam. The observations made during site visit are briefly mentioned below.

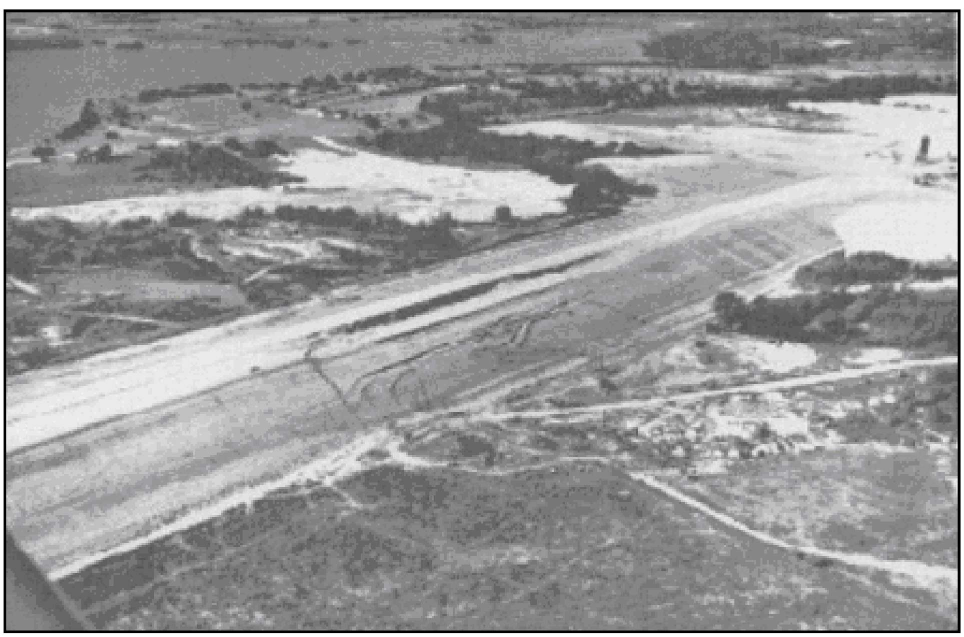


Figure 2.23 Slide on downstream side of Waco Dam
(Source: Duncan and Wright, 2005)

In the slide region, the Pepper shale was geologically uplifted to the surface and it was laterally bound by two faults crossing across the axis of the embankment. The slide was confined to the length of embankment and no significant movements were noticed beyond the fault boundaries.

The investigation of USACE revealed that the slide extended for several hundred meters downstream from the embankment. The Pepper shale was found to be containing horizontal slickensided fissures spaced about 3 mm (0.12 in.) apart. The strength along the horizontal planes was found to be about 40% of the strength of the vertical specimens. The dam would not pass the strength test had the strength along horizontal surface been taken in to consideration at the time of design.

This findings of investigation cautions against testing conventional vertical specimens alone while dealing with stiff fissure clays having single dominant fissure orientation.

2.10.2 Highway 24 Shallow Failure near San Francisco Bay Area

During January 1982, there was 25 cm (10 in.) rainfall (Duncan and Wright, 2005) due to a storm from Pacific Ocean in San Francisco Bay area against the normal yearly rainfall of 63 cm (25 in.). The deluge caused number of landslides which were shallow in nature.

The investigation revealed that the intense rainfall saturated the upper few feet of the ground on the hill sides which came down as slides and flows causing lot of destruction. Duncan and Wright (2005) concluded that long periods of higher than average rainfall causes deep seated, slow moving slides and one or two days of intense rainfall contributes to shallow slides involving few meters of soil.

2.10.3 Failure of Highway Embankment near Houston, Texas

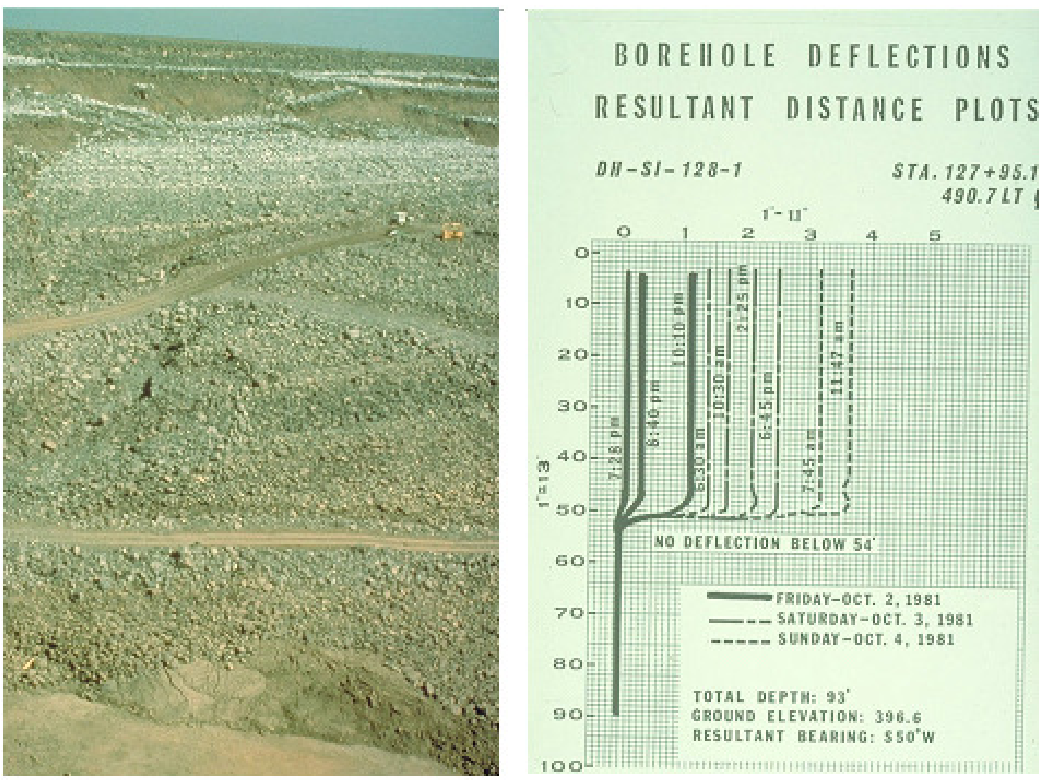
The highway embankment was constructed of highly plastic clay with a side slope of 2H:1V. The fill was well compacted and the embankment remained stable during and after 20 years of construction. However, a failure occurred after 20 years as shown in Figure 2.24.

It was concluded that the soil swelled and grew softer and weaker due to repeated drying and wetting of soil over the period of time as a result of alternating dry and wet seasons, resulting in surficial failure (Duncan and Wright, 2005).

Figure 2.24 A surficial slide near Houston on highway embankment
(Source: Duncan and Wright, 2005)

2.10.4 Slide on Upstream Slope at San Luis Dam, California

San Luis Dam embankment was constructed in 1969 (Duncan and Wright, 2005). On September 4th, 1981 a slide occurred on upstream side where the bank height was 60 m (200 ft). The length of slide was 350m (1100 ft). A photograph showing the failure is shown in Figure 2.25. The cause of failure was investigated and the findings were explained as following (Duncan and Wright, 2005):



(a) (b)

Figure 2.25 San Luis Dam (a) Failed slope (b) Borehole deflections

When the dam was constructed, the highly plastic clay slope-wash that covered the foundation was dry and very strong. It became weak due to wetting by water of reservoir. During the period of 12 years, the water level fluctuated up and down several times due to inflows during rainy season and withdrawals during dry season.

These fluctuations resulted in reducing the strength of soil gradually to a residual value resulting in failure. The failure was also preceded by largest and fastest drawdown during September 1981. The slide was stabilized by rebuilding part of dam and providing additional support in the form of 18m (60 ft) high buttress.

2.11 Slope Stabilization Methods

Slope stabilization methods are aimed at reducing the driving forces and increasing the resisting forces or both. Abramson et al. (2002) summarized the above activities as following.

Driving forces can be reduced by excavating the material and improving the drainage. Resisting forces can be increased by (Abramson et al. 2002):

- Drainage that improves the shear strength
- Elimination of weak strata
- Building of retaining walls, MSE walls
- Provision of soil reinforcement
- Chemical treatment to increase shear strength of the ground

Both biotechnical stabilization and soil bioengineering stabilization entail the use of live materials, specifically vegetation (Gray and Sotir, 1996).

Biotechnical stabilization utilizes mechanical elements or structures in combination with biological elements or plants to prevent slope failures and erosion.

In bioengineering, the plant parts themselves serve as roots and stems and serve as the main structural and mechanical elements in slope protection system. Various conventional and unconventional stabilization methods are discussed as following:

2.11.1 Unloading

Unloading technique implies reduction of driving forces within a slide mass. It includes excavation of upper part of slope in an existing slide. It is called removal of head of slide. Removal of head of a landslide reduces driving forces and tends to balance the failure.

One disadvantage associated with this method is the accessibility to the top of slide as the excavation is to be carried out top downward. Other techniques of unloading include removing all unstable materials of slide, flattening of slope and benching. Benching of slopes is also used to control erosion and to provide vegetation.

2.11.2 Lightweight Fill

Use of lightweight fill during embankment construction also reduces the driving force and thus increases stability of slope. Nelson and Allen (1974) used sawdust and wood fiber with asphalt encapsulation to replace a landslide in a project site of Washington Department of Transportation. Expanded Polystyrene (EPS) blocks were also used as a lightweight fill material. Other promising materials fly ash and geofoams.

2.11.3 Buttressing

Buttressing is a technique used to counter the driving forces of a slope by an externally applied force system that increases resisting force. Buttresses are usually of various types as explained below:

2.11.3.1 Soil and Rock fill

The fill is aimed at providing sufficient dead weight near toe. A schematic showing a rock buttress used to mobilize sufficient dead weight at the toe of an unstable slope to prevent further movement is shown in Figure 2.26.

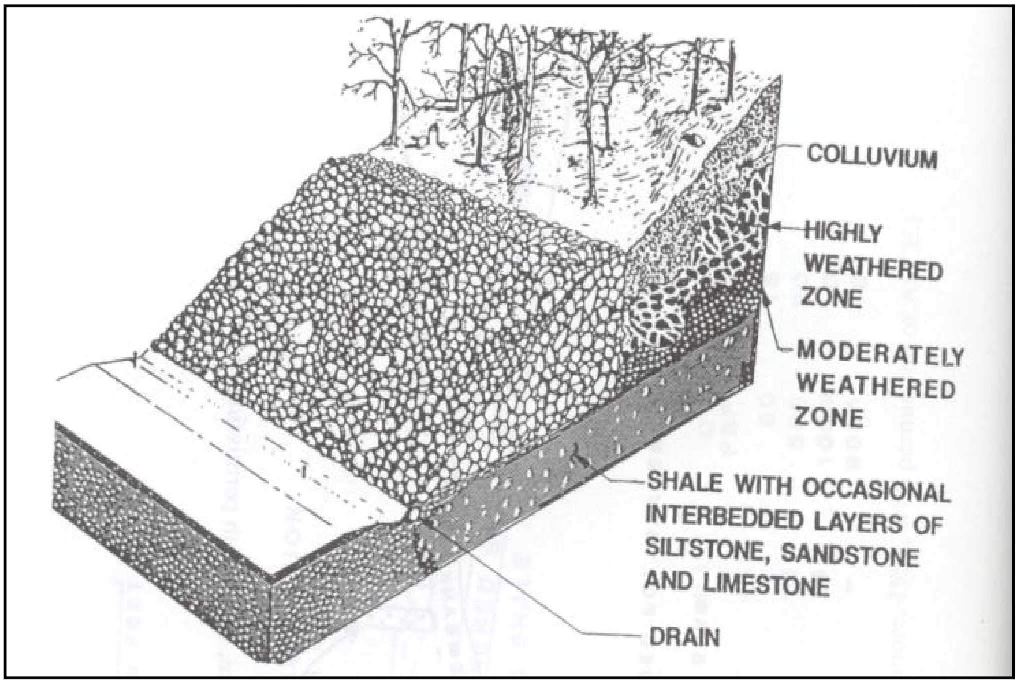


Figure 2.26 Rock buttress used to control unstable slope
(Source: Abramson et al. (2002)

2.11.3.2 Counterberms

A counterberm is used to provide weight at toe of a slope to increase shear strength below the toe. This is useful for embankments on slope and where upheaval is expected to occur. Counterberms also increase resistance against sliding. However, care needs to be taken that counterberm is not counterproductive by increasing driving forces.

2.11.3.3 Shear Keys

Shear keys are used to provide additional resistance to sliding for a counterberm or rocky/soil buttress, and retaining walls.

2.11.4 MSE Walls

Mechanically stabilized embankments are designed to use backfill soil along with thin metallic strips, mesh or geosynthetic reinforcement to support or restrain loads.

2.11.5 Pneusol

Pneusol (tiresoil) uses old tire sidewall mat reinforcement at about 2 feet interval. Design of pneusol is similar to MSE walls. The technique is being widely used in USA , France and many other countries (Abramson et al. 2002). TxDOT repaired a surficial failure near DFW airport on the ramp of 183 W and 360 S using pneusol during September 2008 as witnessed by the author.

2.11.6 Drainage

Proper drainage arrangements reduce destabilizing hydrostatic and seepage forces on a slope. Drainage also reduces the risk of erosion and piping. Various techniques of drainage arrangements generally adopted are discussed as following:

2.11.6.1 Surface Drainage

Drainage system should collect and runoff from the slope and lead water to convenient discharge system (Ortigao and Sayao, 2004). Surface drainage system combined with surface protection should fulfill the objective of reducing infiltration, control soil erosion and improve slope stability. A typical surface drainage system provided at few slopes in Hong Kong showing cross drains is illustrated in Figure 2.27.

Figure 2.27 Surface drainage on a slope at Hong Kong
(Source: Ortigao and Sayao, 2004)

The hydraulic design of surface drainage system depends on various parameters like catchment area, concentration time, and average rainfall intensity, probability of occurrence, slope geometry, infiltration capacity and surface conditions (Ortigao and Sayao, 2004).

2.11.6.2 Subsurface Drainage

The factor of safety of slip surface that passes below the phreatic line can be improved by surface drainage. Properly designed subsurface drains in conjunction with surface drainage helps dissipate excessive pore water pressures resulting in surface slope protection (Ortigao and Sayao, 2004).

Various types of subsurface drains include drain blankets, trenches, cut-off drains, horizontal drains, relief wells and drainage tunnels. Of late, perforated pipes wrapped with geotextiles are widely used for concealed drains. Proper design of proper filter media prevents clogging of subsurface drains.

2.11.7 Soil Reinforcement

Reinforcement of soil mass is generally carried out by soil nailing technique, stone columns, deep mixing, using tie backs etc., (Abramson et al. 2002).

2.11.7.1 Soil Nailing

Soil nailing is used for retrofitting of existing slopes and to repair landslides. Figure 2.28 shows the condition of a slope in Hong Kong before and after retrofitting.

Figure 2.28 Slope before and after retrofitting (Ortigao and Sayao, 2004)

Retrofitting at the Hong Kong slope site included repairing and installing drainage arrangements, adding new vegetation, replacing old surface and providing new surface with soil nails, and shotcrete.

Soil nailing is an in situ reinforcement technique utilizing passive inclusions that will mobilize if movement occurs. The reinforcement consists of steel bars and metal tubes that will resist tensile stresses, shear stresses and bending moments. The design involves spacing, size and length of nails. A proper wall facing like shotcreteing is required.

Yeung et al. (2007) reported the use of glass fiber reinforcement polymer (GFRP) pipe with grouting for soil nailing in the place of traditional galvanized or coated steel reinforcement bars.

2.11.7.2 Tieback Wall

A tie back wall after completion is shown in Figure 2.29.

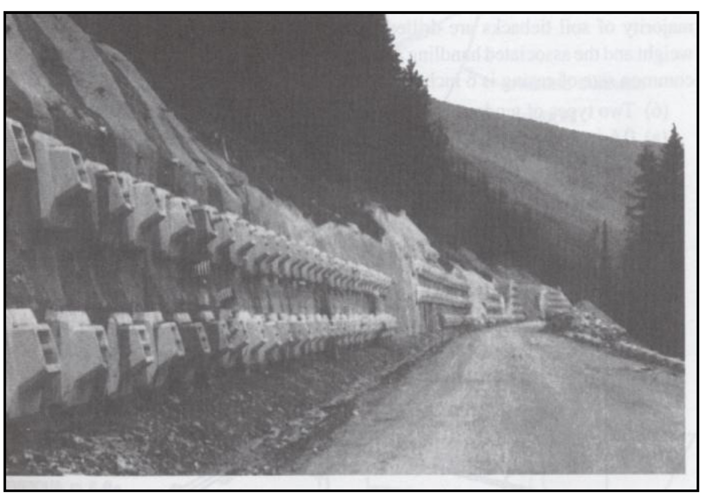


Figure 2.29 Tieback reinforcement for construction of railroad
(Source: Abramson et al. 2002)

The difference between soil nailing and tie back walls is that in tieback walls, a pre-designed amount of prestress is induced in to the reinforcement. Anchors have a free length where as soil nails have a designed length that offers frictional resistance against soil thus transferring load.

2.11.8 Stone Columns

Stone columns can be used to stabilize or prevent landslides The ground improvement technique increases shear resistance of soil along the potential slip surface (Abramson et al. 2002). Figure 2.30 shows an illustration how the stone columns function to improve stability of slopes.

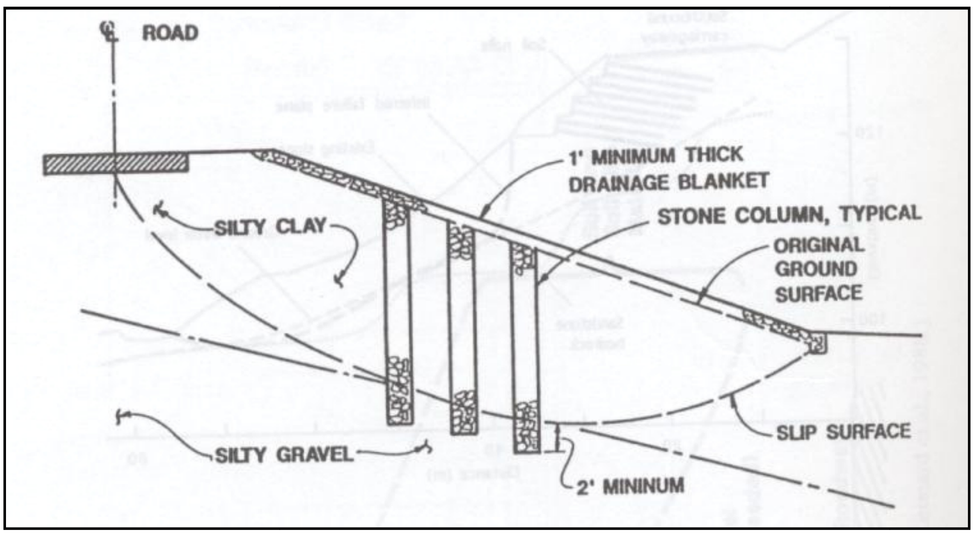


Figure 2.30 Stone columns to stabilize an unstable slope
(Source: Abramson et al. 2002)

2.11.8.1 Deep Soil Mixing

The Deep soil mixing technique is an improvement over using stone columns, but works almost similar to stone columns. Research studies have shown that the use of a properly designed binder which includes lime and cement reduces the swell and shrink movements of soil besides strengthening the soil mass (Puppala et al. 2006).

The deep mixing is an ideal method where provision of flat slopes is not possible in urban areas owing to right of way restrictions. Deep mixing columns are usually spaced at an interval of 3 times the column diameter. The arch action also helps improve the lateral stability.

2.11.8.2 Reticulated Micro Piles

Reticulated micro piles were developed in Italy (Lizzy, 1985; Abramson et al. 2002). A schematic arrangement of using reticulated piles is shown in Figure 2.31.

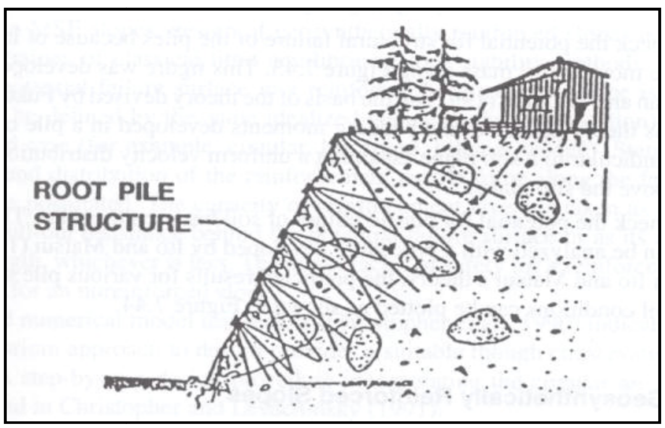


Figure 2.31 Reticulated micro piles to stabilize slopes
(Lizzi, 1985; Abramson et al. 2002)

Reticulated micro piles are used to create a monolithic rigid block of reinforced soil to a depth below the critical failure surface. The major difference between soil nailing and reticulated micro piles is that the behavior of later is influenced significantly by their geometric arrangement. Lizzi (1985) demonstrated after conducting laboratory and field tests that group and network effect of reticulated micro pile system provided higher load bearing and shearing capacities than those of closely spaced vertical piles.

2.11.9 Biotechnical Stabilization and Soil Bio Engineering

Vegetation is highly effective and it stabilizes the soil surface by intertwining of its roots, minimizes seepage of runoff by intercepting rainfall, and retards runoff velocity. A photograph showing the use of geogrid and hydro seed treatment is shown in Figure 2.32.

Figure 2.32 Vegetated surface on geogrid
(Source: Gray and Sotir, 1996)

Vegetation has an indirect effect on soil stabilization by depleting soil moisture, attenuating depth of frost penetration and providing a deep rooted network in the soil (Abramson et al. 2002). Vegetation also has environmental and sustainability advantages. Biotechnical slope protection refers to combined use of living vegetation and inert structural components like concrete, wood, stone and geosynthetics. Soil bio engineering refers to use of live plants and plant parts for slope protection (Gray and Sotir, 1996). Live cuttings and stems are deliberately imbedded on ground or slopes where they serve as soil reinforcement, hydraulic drains, barriers to earth movement and hydraulic pumps or wicks. A conventionally graded and landscaped hill along with drainage ditch is shown below in Figure 2.33. Live staking in conjunction with live fascines can be successfully used to repair the small earth slips and slumps that are quite wet (Gray and Sotir, 1996).

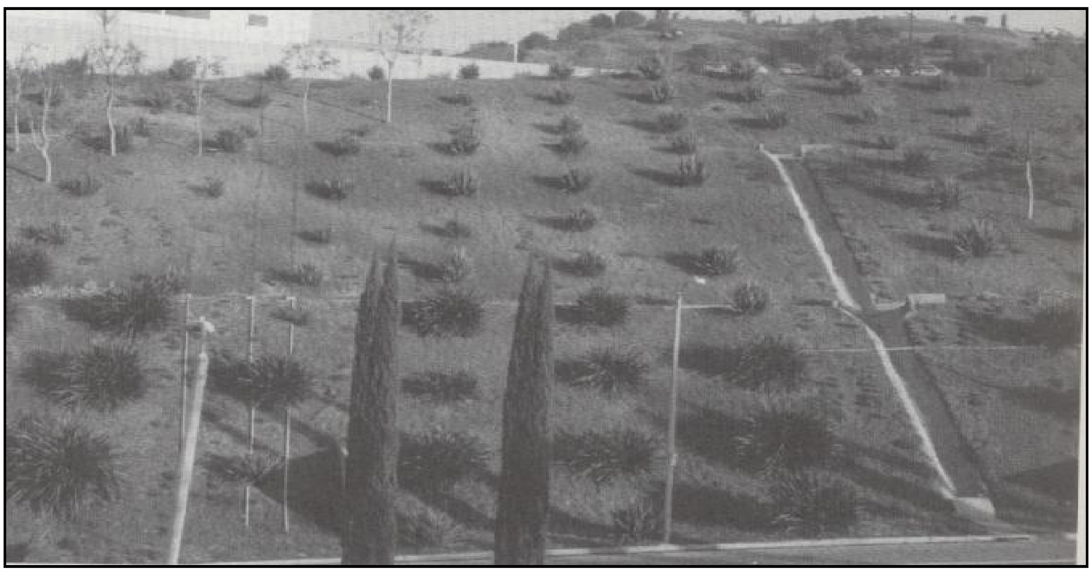


Figure 2.33 Landscaped slope with drainage and plantings
(Source: Gray and Sotir, 1996)

2.11.9.1 Reinforced Grass

The system consists of very porous, synthetic, three dimensional mats also called turf reinforcement mats placed on the ground, followed by filling with soil and seeding.

2.11.10 Measures to Prevent Surficial Erosion

Surficial erosion is the removal of surface layers of soil by the agencies of wind, water and ice. Soil erosion involves a process of both particle detachment and particle transport (Gray and Sotir, 1996). The most common type of soil erosion is rainfall and wind erosion.

Rainfall impact on bare soil initiates soil erosion and runoff transports the soil particles. At the onset of runoff, water collects into small rivulets, which may erode small channels called rills. These rills may intensify and form into larger and deeper channels called gullies. Gray and Sotir (1996) suggested a hierarchy of erodibility based on the Unified Soil Classification System as shown below.

Most Erodible --→ Least Erodible

ML > SM > SC > MH > OL >> CL > CH > GM > SW > GP > GW

where,

ML = low plasticity silt

SM = silty sand

SC = clayey sand

MH = high plasticity silt

OL = low plasticity organic soil

CL = low plasticity clay

CH = high plasticity clay

GM = silty gravel

SW = well graded sand

GP = poorly graded gravel

GW = well graded gravel

Soil erodibility is a measure of the susceptibility of soil particles being eroded by rainfall and runoff. Erosion particularly at the toes of slopes is known to trigger landslides. Soil erodibility depends on the soil texture, length and degree of slope (Abramson et al. 2002). Vegetation helps prevent erosion. Other methods of prevention of soil erosion include use of natural and synthetic mats and blankets, roving and various other self containment systems.

2.11.11 Surface Slope Protection

The objective of surface slope protection is to prevent infiltration by rainfall so that the slope can be maintained dry or partially dry (Abramson et al. 2002).Various surface protection methods in vogue are briefed as following:

- Applying shotcrete on the surface
- Applying Chunam Plaster which is a cement-lime stabilized soil mixture
- Use of masonry blocks with joints filled in cement mortar
- Use of rip-rap

2.11.12 Soil Hardening

Stabilization of slopes by drainage may be hardly effective in cohesive soils as they have very low permeability. The technique of soil stabilization by improving drainage may be more useful for cohesion less soils. For cohesive soils, the following methods may be more effective (Abramson et al. 2002).

2.11.12.1 Compacted Soil-Cement Fill

Compacted soil-cement fill was used to construct embankments where the side slopes could be steepened to 1.5H:1V to 1H:1V because of high shear strength of mix. Use of cement by 1 to 10% by weight had improved the cohesive strength to 25 to 125 pounds per square inch besides reducing permeability (Abramson et al. 2002). Figure 2.34 shows the use of compacted soil-cement fill for slope remedial work to rebuild a failed slope.

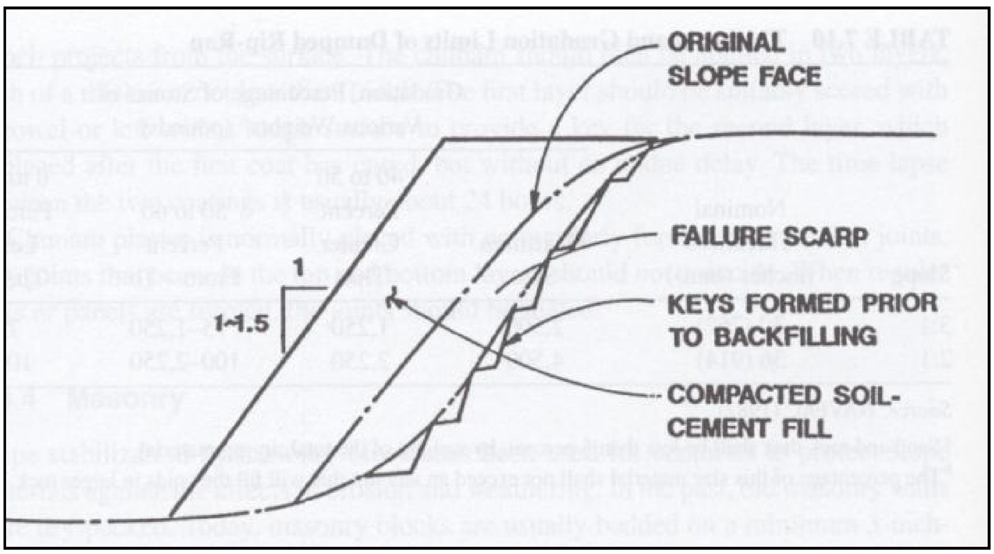


Figure 2.34 Soil-cement fill to stabilize a landslide
(Source: Abramson et al. 2002)

2.11.12.2 Electro-osmosis

Electro-osmosis technique deploys a cathode and an anode. When a potential difference is applied, the water flows towards cathode which has a perforated pipe and the water is pumped out from there. High costs are involved to use this method for cohesive soil stabilization.

2.11.12.3 Thermal Treatment

Thermal treatment was used in Romania and United States of America to stabilize landslides. The high temperature treatment causes a permanent drying of slope. However, it was recommended to be used in exceptional circumstances in view of costs, high energy requirements and environmental impacts.

2.11.12.4 Grouting

Grouting was effectively used to stabilize landslides with shallow movement in clay shale and stiff clays. The effect of grouting is to displace water from fissures and pores in the ground and they are filled with grout under pressure.

2.11.12.5 Lime Injection

Shear strength of clayey and silty soils can be improved by injection of lime columns at designed spacing. A rotating disk auger penetrates the ground to a depth below the slip surface and lime slurry is injected into the kneaded soil column.

2.11.12.6 Preconsolidation

Strength of clayey soils is increased by acceleration of consolidation through preloading. This technique is more suitable for slopes overlying soft foundation soils. Preloading, sand drains, PVDs are commonly adopted for preconsolidation of soil.

2.12 Previous Research on Surficial Slope Failures

Research was conducted to mitigate surficial failures by The University of Texas at Arlington, The University of Missouri at Columbia and The University of Wisconsin at Milwaukee. USACE currently follows certain methods to repair the surficial failures. Various remedial measures attempted to prevent and repair surficial failures are briefly summarized as following:

2.12.1 Current Practice of Repairing Surficial Failures by USACE

USACE has been currently repairing the surficial sloughs by the following methods.

- Excavate the slide mass and rebuilt the slope pushing back the same soil after drying or using borrow soil by compacting in layers.
- Rebuild the slide area by flattening the slope or by providing berms and by using the same soil or borrow soil.

The above methods were reported with mixed success. Recurring failures were noticed even after repairs at few places.

2.12.2 Use of Recycled Plastic Pins to improve Surficial Slope Stability

Loehr and Bowders (2007) conducted research for Missouri Department of Transportation to repair the surficial slides that occurred on the highway embankments and cut slopes. Use of recycled plastic pins for slope stabilization to prevent surficial failures was documented by Missouri Transportation Institute and Missouri Department of Transportation (OR07.006). A schematic of using recycled plastic pins as reinforcing members is shown in Figure 2.35.

Recycled plastic pins are manufactured from industrial or post-consumer waste consisting of polymeric materials like high density polyethylene (HDPE), low density polyethylene (LDPE), polystyrene, polypropylene, polyethylene-terephthalate (PET) and varying amounts of additives like sawdust, fly ash and other waste materials. The average compressive strength ranged from 10 to 21 MPa (1500 psi to 3000 psi).

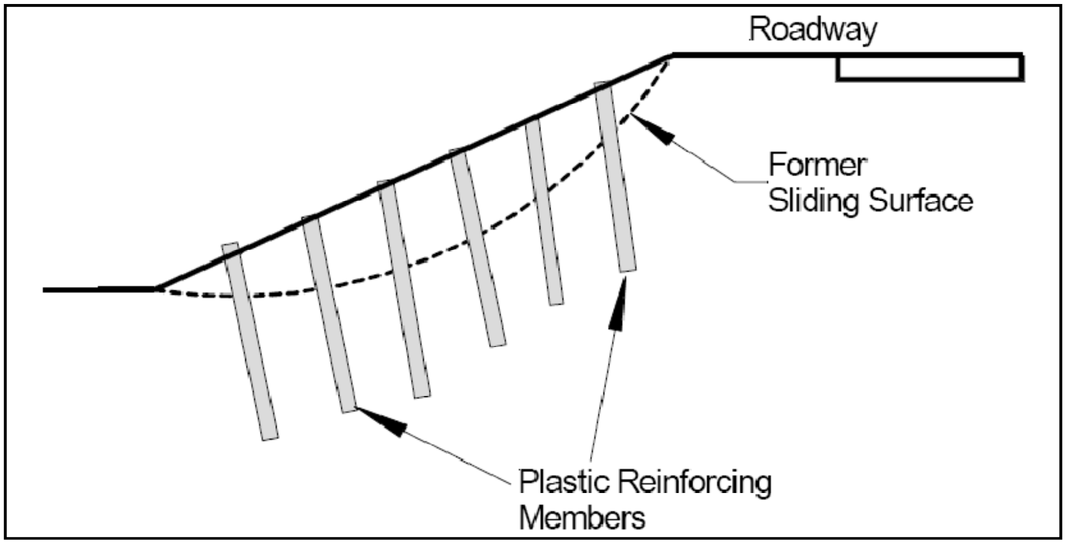


Figure 2.35 Stabilization of surficial slope failures with recycled plastic pins
(Source: Loehr and Bowders, 2007)

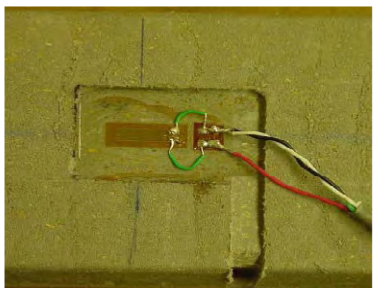
The measured flexural strengths for specimens loaded to failure or 2% strain ranged from 9 MPa to 25 MPa (1300 psi to 3600 psi). The material was found to be resilient to a broad range of exposure to typical environmental conditions.

The eight test sites selected were having embankment slopes varying from 1.7H:1V to 3.2H:1V and the slope heights varying from 4.5 m (15 ft) to 14 m (46 ft). The test sections were instrumented with inclinometers to measure lateral displacements.

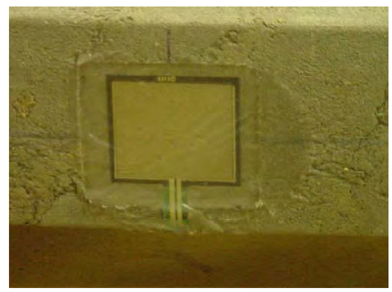
Several reinforcing members were instrumented with strain gages and force-sensing resistors to monitor the loads mobilized. Figure 2.36 shows a typical instrumented recycled plastic member. Figure 2.37 shows the sectional view of installation of these pins.

The sites were also instrumented to measure pore pressures and matric suction using piezometers, thetaprobes and equitensiometers.

(a)



(b)



(c)

Figure 2.36 Instrumentation (a) An instrumented recycled plastic pin,
(b) Electric resistance strain gage (c) Force-Sensing resistor
(Source: Loehr and Bowders, 2007)

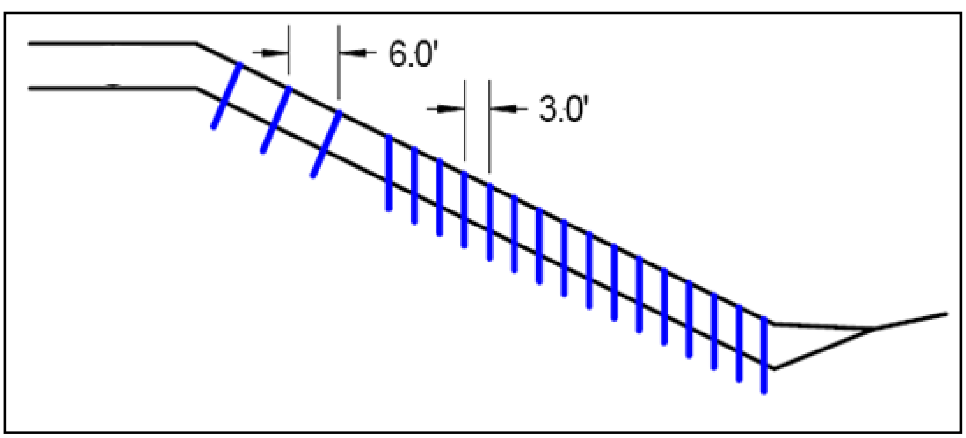


Figure 2.37 Sectional view of installation of plastic pins
(Source: Loehr and Bowders, 2007)

Various observations made during the study are summarized below.

- Pins provided at two slide areas at I70-Emma site were proved to be successful where two control sections failed later.
- For a third slide area at I70–Emma site. The observations were made over a period of about 2 years and there was a failure in the test section. The failure was preceded by displacements of sections ranging from 6 cm to 12 cm (2.5 inches to 5 inches).
- It was concluded based on investigation that the failure did not occur in sections where the pins were closely spaced and failure occurred where the pins were placed at a spacing of 1.8m (6 feet).
- Various other sites were tested with different spacing of recycled plastic pins. The performance of recycled plastic pins was reported to be satisfactory with a spacing of 0.9m (3 feet) to prevent surficial failures.

2.12.3 Use of Nailing and Anchor Techniques to Improve Surficial Slope Stability

Titi and Helwany (2007) have carried out extensive research for Wisconsin Department of Transportation to repair the surficial slides that occurred on the highway embankments and cut slopes. They have documented the use of vertical members for slope stabilization to prevent surficial failures. The studies were compiled as SPR # 0092-05-09. The study team of the University of Wisconsin, Milwaukee visited various sites of surficial failures. One of the locations of surficial failure was along STH-164 in Waukesha County, Wisconsin as shown in Figure 2.38. The team concluded that the cause of failure was prolonged rainfall and snowmelt. When a small hole was dug at another surficial failure site near Burlington, it got filled up with water quickly as shown in Figure 2.39 indicating presence of abundant quantity of water near the failure surface.

Figure 2.38 Surficial failure on a cut slope along STH-164, Wisconsin
(Source: Titi and Helwany, 2007)

Figure 2.39 Perched water on a failure surface through seepage
(Source: Titi and Helwany, 2007)

The report summarizes effectiveness of various repair techniques as following:

2.12.3.1 Pipe Piles and Wood Lagging Method

In the report by Day (1997), the use of pipe piles and wood lagging was reviewed. The pipe pile and wood lagging method consists of disposing the failed debris off the site and cutting benches into the slope below the failure surface as shown in Figure 2.40.

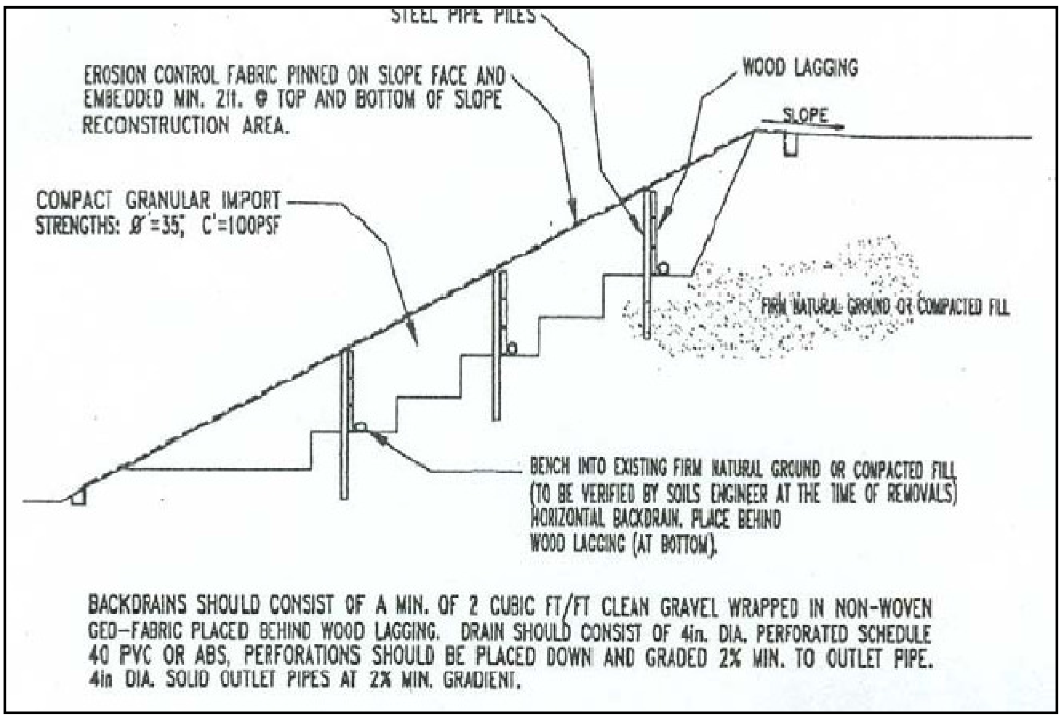


Figure 2.40 Pipe pile and wood lagging repair (Day, 1997)

Wood lagging was placed behind the piles and a drainage system was then built behind the wood. A select fill is compacted in layers and the face of slope was protected with erosion control fabric and vegetation. Titi and Helwany (2007) opined that the disadvantage of this method was that the soil pressure against wood lagging was transferred directly to pipe piles. This results in failure of pile in bending.

Titi and Helwany (2007) have carried out further studies and recommended use of the soil nailing and earth anchors for slope stabilization against surficial failures.

2.12.3.2 Repairs using Soil Nailing

Titi and Helwany (2007) recommended launching of soil nails beyond the failure surface under pressure using Soil Nail Launchers. Figure 2.41 shows the process of soil nail launching on slopes. It was observed that top portion of soil did not move when it was stabilized with soil nails.

Figure 2.41 Installation of soil nails (Source: Titi and Helwany, 2007)

2.12.3.3 Repairs using Earth Anchors

The earth anchoring system consists of a mechanical earth anchor, wire rope and end plate with accessories. The method was recommended for slope stabilization and repairs of surficial failure locations. The technique involves grading of failed slope, providing a turfing mat and then installing earth anchors as shown in the Figure 2.42.

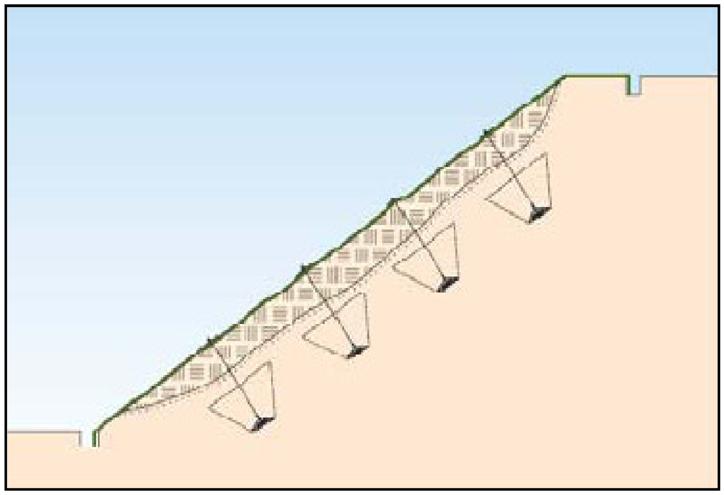


Figure 2.42 Installation of earth anchor
(Source: Titi and Helwany, 2007)

For installing the earth anchors, the anchor is first pushed into the soil below the failure surface. The wire of tendon of the anchor is pulled to move the anchors to its full working position. The wire tendon is locked against the end plate and the system is tightened.

2.12.3.4 Rammed Aggregate Pier

Parra et al. (2007) demonstrated successful use of rammed aggregates pier to repair two deep seated slope failure sites of US Highway 71. The Rammed aggregate piers were installed and it was found that the progressive lateral displacement of slope stopped.

2.12.4 Use of Lime, Fibers and Compost to Improve Surficial Slope Stability

Expansive soils usually have the properties of moderate to high plasticity, low to moderate strength and high swell and shrinkage (Puppala et al. 2006). Chemical stabilization of expansive soils using calcium based stabilizers like lime and cement improved soil strength, stiffness, durability and a reduction in soil plasticity and swell/shrinkage potential (Hoyos et al. 2004). University of Texas at Arlington (UTA) has been conducting research studies of problematic slopes and distressed pavements laid on expansive clays. The studies revealed that lime, fibers and compost were very promising for slope stabilization or mitigation of desiccation cracking of expansive soils. McCleskey (2005) conducted laboratory studies on soils obtained from dam sites of Joe Pool Lake Dam and Grapevine Lake Dam using lime, compost and fibers as the chemical admixtures to treat the soil and found that these admixtures were quite promising to mitigate the desiccation cracking.

2.13 Use of Lime as a Soil Admixture

The use of lime stabilization of clay in construction is 5000 years old (Khattab et al. 2006). The pyramids of shersi in Tibet were built using a compacted mixture of clay and lime (Greaves, 1996; Little, 1995). Lime stabilization is one of the oldest methods to improve soil properties economically (Schoute, 1999). Lime treatment is classified into two processes viz., soil modification and soil stabilization (Source: INDOT manual, Indiana). Soil modification aims at creating a working platform for construction equipment and soil stabilization targets enhancing the strength of soil and improving other desirable properties.

2.13.1 Chemistry of Lime Treatment

Lime used for soil treatment can be in the form of quick lime (calcium oxide, CaO), hydrated lime (calcium hydroxide, Ca [OH]2 or lime slurry (Lime manual, 2004). Quicklime is manufactured by chemically transforming calcium carbonate (lime stone,

$CaCO_3$) into calcium oxide. Hydrated lime is created when quicklime chemically reacts with water.

$$CaCO_3 \rightarrow (Heating\ @\ 900^0 C) CaO + CO_2 \tag{2.13}$$

$$CaO + H_2O \rightarrow Ca[OH]_2 \tag{2.14}$$

Lime cannot react with soils containing as little as 7% clay and Plasticity Indices as low as 10 (Lime manual, 2004). INDOT recommends the following guide lines for classifying the soil to be a reactive soil.

For Modification, % soil passing sieve No. 200 > 35 and PI > 5

For Stabilization, PI > 10 and minimum clay content > 10%

When lime is added to a reactive soil, short term reactions and long term reactions occur. Short term reactions include cation exchange, flocculation, agglomeration and where as long term reactions included pozzolanic reaction and carbonation (Khattab et al. 2006). These reactions result in mineralogical and micro structural changes in the stabilized soils altering the properties of expansive soil (Khattab et al. 2007). A brief description of various reactions involved with lime stabilization is indicated below.

2.13.1.1 Short Term Reactions

Clay particles have negatively charged ions and lime has positively charged ions. After initial mixing, the positively charged calcium ions (Ca^{++}) migrate to the surface of the clay particles and displace the water and other ions adhered to the surface like Mg, K or Na ions. At this stage the Plasticity Index (PI) of soil decreases and the tendency of soil to swell and shrink reduces (Lime manual, 2004). With a mere addition of 1 to 2% of soil, the reactions start immediately and the clay particles are electrically attracted and flocs are formed. The process is called flocculation and agglomeration which occurs within few hours. The workability of soil increases and this process accounts for soil modification.

2.13.1.2 Long Term Reactions

Carbonation and Pozzolanic reactions are time dependent which may take few days to years (Ola, 1978). Carbonation reaction is very slow during which the CaO reacts with atmospheric CO_2 and forms in $CaCO_3$. Addition of adequate quantity of lime beyond the quantity of lime required for soil modification causes a rapid increase of pH of the soil water due to partial dissolution of $Ca(OH)_2$ (Ola, 1978).

Lime reacts with clay minerals and complex chemical reactions or pozzolanic reactions take place forming cementitious products in the form of a water insoluble gel of calcium silicate hydrates. With time, the gel gradually crystallizes into cementing agents such as calcium silicate hydrates (CSH) (tobermorite and hillebrandite) and calcium aluminate hydrates (CAH) (Ingles and Metcalf, 1972; Galvao et al. 2004, Lime manual 2004). CSH and CAH are cementitious products similar to those formed in Portland cement. The reaction occurs only when the water is present and it carries calcium and hydroxyl ions to the clay surface (Galvao et al. 2004). This process results in soil stabilization improving strength of soil significantly besides altering various other properties of soil like swelling, shrinkage, permeability etc.

2.13.2 Selection of Type of Lime

The type of lime selected for stabilization should be based on the several important considerations like type of soil, site conditions, experience of contractor, and availability of equipment and availability of water source (Lime manual, 2004). Quick lime or hydrated lime is usually used for lime stabiliztion as detailed below.

2.13.2.1 Quicklime

Quicklime contains 20 to 24% more available lime oxide content than hydrated lime and hence is economical to use (Lime manual, 2004). Dry quicklime is ideal for drying wet soils. However, quicklime requires more water (about 32% of its weight) for reactions and time for mellowing. Quicklime also raises lot of dust causing environmental concerns.

2.13.2.2 Hydrated Lime or Lime Slurry

Dry hydrated lime can be used for drying clay but it is not as effective as quicklime (Lime manual, 2004). Lime slurry is a hydrated lime mixed with water.

The main advantage of slurry lime is that it ensures a dust free application. It is easier to achieve an even distribution (Lime manual, 2004). However, this method is not suitable for wet soils.

2.13.3 Estimation of Optimum Percentage of Lime

Eades and Grim (1966) and Hill and Davidson (1966) methods are available to determine the optimum percentage of lime (Khattab et al. 2007).

Eades and Grim test as explained in the manual of INDOT and the Hill and Davidson method is briefly explained below.

2.13.3.1 Eades and Grim test

In this method, sufficient amount of lime is added to soils to produce a targeted pH of 12.4 or equal to a pH of lime itself and a graph is plotted between pH percentages of lime. Optimum lime content shall be determined corresponding to the maximum pH of lime-soil mixture.

- Oven dry soil passing No. 40 sieve and transfer a sample of 20 gm into about 5 numbers of 150 ml plastic bottles with screw tops.
- Add lime with different dosages between 3% and 10% and shake the mixture after adding 100ml of distilled water.
- Shake the bottles until there is no evidence of dry material on the bottom and for a minimum of 30 seconds every 10 minutes.
- After one hour, transfer part of the slurry to a plastic beaker and measure the pH value. The pH meter must be equipped with a Hyalk electrode and standardized with a buffer solution having a pH of 12.00.
- Record the pH for each of the lime-soil mixtures. If the pH readings go to 12.40, the lowest percent lime that gives a pH of 12.40 is the percent required to stabilize the soil.

2.13.3.2 Hill and Davidson Method

Hill and Davidson (1960) suggested an empirical expression to obtain the minimum lime percentage (L_m).

$$L_m = \frac{Clay \leq 2\mu m}{35} + 1.25 \qquad (2.15)$$

2.13.3.3 Recommendation of INDOT and Studies by other Researchers

The manual of Indiana Department of Transportation recommends 3-9% of lime to be the optimum percentage. Khattab (2007) reported that swelling pressure decreases with an increase of percentage of lime up to 4% and then stabilizes. Various researchers suggested the use of lime dosage close to 8% by weight and indicated that further additions of dosages of lime did not change the swelling potentials. However, the increase in lime content enhanced the engineering properties of expansive soils (Bell, 1996; Guney et al. 2007).

2.13.4 Effect of Lime Treatment on Properties of Soil

It is generally known that the addition of lime reduces the plasticity index of soil and in most of the cases the liquid limit decreases significantly and plastic limit increases slightly. Studies have shown that (Bell, 1996; Osinubi, 1998) addition of lime to soil results in significant increase in optimum moisture content and decrease of dry density. The compressive strength of soil increases many times.

2.13.4.1 Permeability of Lime Treated Soils

Permeability of soil increases with the addition of lime due to the effects of flocculation and agglomeration. Townsend and Klym (1966) reported an increase of permeability of CH soils from 2×10^{-8} cm/s to 4×10^{-6} cm/s (0.79×10^{-8} in./s to 1.6×10^{-6} in./s) with addition of lime.

The permeability also depends on the type of soil, gradation of soil, dry density and optimum moisture content (OMC). The permeability of compacted clays samples on the dry side of OMC was many times higher than on the wet side (Mitchell and

Dermatas, 1992; Osinubi, 1998). This phenomenon is due to random particle orientations and a large average pore size than when compacted on the wet side of OMC. Osinubi (1998) conducted permeability studies on a CL soil.

It was found that the permeability increased up to 4% of addition of lime and then decreased with further addition of lime. The increase of permeability was attributed to flocculation and increase of pore size. The decrease in permeability due to addition of more than 4% of lime was attributed to increase of pH value as a result of partial disassociation of calcium hydroxide.

It was also opined that the formation of insoluble CAH or CSH gels obstructed the flow through the voids. Presence of excess amount of lime was also said to be responsible for long term pozzolanic reactions.

Galvao et al. (2004) reported that the permeability of lime treated soil decrease with increase of lime content up to 8%. However, the permeability reported by various researchers at 8% lime was still higher than the permeability of untreated soils.

2.13.4.2 Compressibility of Lime Treated Soils

Studies of Galvao et al. (2004) using one dimensional consolidation tests indicated that the soils when treated with lime exhibited significant resistance to compressibility.

However, increase of lime content from 4% to 8% did not have much impact on the resistance to compressibility. It can be attributed to the fact that even with addition of 1% to 2% of lime, the process of cation exchange starts and changes takes place in the physico chemical characteristics of soil surfaces (Galvao et al. 2004).

2.13.4.3 Collapsibility of Lime Treated Soils

Collapsible soils refer to the category of soil deposits that undergo significant decrease in volume when exposed to water (McCarthy, 2002). Tests were conducted by Galvao et al. (2004) using double oedometer method for evaluating wet induced collapse. Samples were prepared at densities lower than the maximum dry density and OMC values obtained from proctor test in favor of collapse. For a given pressure intensity, the difference in strain between sample tested normally and under soaked condition was considered to be the amount of wetting induced collapse. For untreated soils, the wetting induced collapse increased with increase in pressure intensity. Lime treated soils exhibited greater resistance to strains and it was found to be very useful for reducing wetting induced collapse of low density lateritic soils (Galvao et al. 2004).

2.13.5 Long Term Stability Characteristics of Lime Treated Soils

Long term stability characteristics of lime treated soils can also be referred to as durability of lime treated soils. Durability criteria is important when a soil is subjected to wetting and drying cycles, freezing-thawing cycles and leaching. Studies pertaining to durability against these environmental factors are not extensive (Khattab et al. 2007). Various aspects of durability of lime treatment are discussed briefly below.

2.13.5.1 Leaching

Malhotra and Bhasker (1983) and Little (1995) show that leaching has a significant effect both on treated and untreated soils that contain highly soluble salts and minerals. However, leaching has limited detrimental effects on soils that do not contain soluble salts (Khattab et al. 2007).

Various studies have further reported that leaching has little influence in the case of poorly drained soils and detrimental effects were the least at an optimum lime content of 4-6% (McCallister et al. 1990; Parsons and Milburn 2003; McCleskey, 2005). Khattab et al. (2007) has conducted leaching tests for 60 days in laboratory and measured pH, Ca^{++} and flow of water in the leachate. He noticed a slight decrease in pH, Ca^{++} and permeability.

It was concluded that leaching does not reduce the efficiency of treatment as the quantity of lime displaced by the water flow was small, compared to the quantity of lime added initially during treatment.

2.13.5.2 Influence of Wetting and Drying Cycles

Khattab et al. (2007) conducted experiments on FoCa (Clayey soil from France) soil by oven drying at 60º C and submerging in water. The untreated specimen was having a swelling of about 75% with a corresponding void ratio of about 2.5 during wetting. During drying phase, the settlement was about 25% with a corresponding void ratio of about 0.5. The effect of wetting and drying on treated and untreated specimen is shown in Figure 2.43.

It was concluded that due to wetting and drying, lime treated soils have shown reduction in swelling characteristics.

It was emphasized that the efficiency was maximum when the soil was first subjected to wetting. The authors recommended that lime stabilized soils should not be immediately subjected to drying conditions during hot season as soon as the curing is completed.

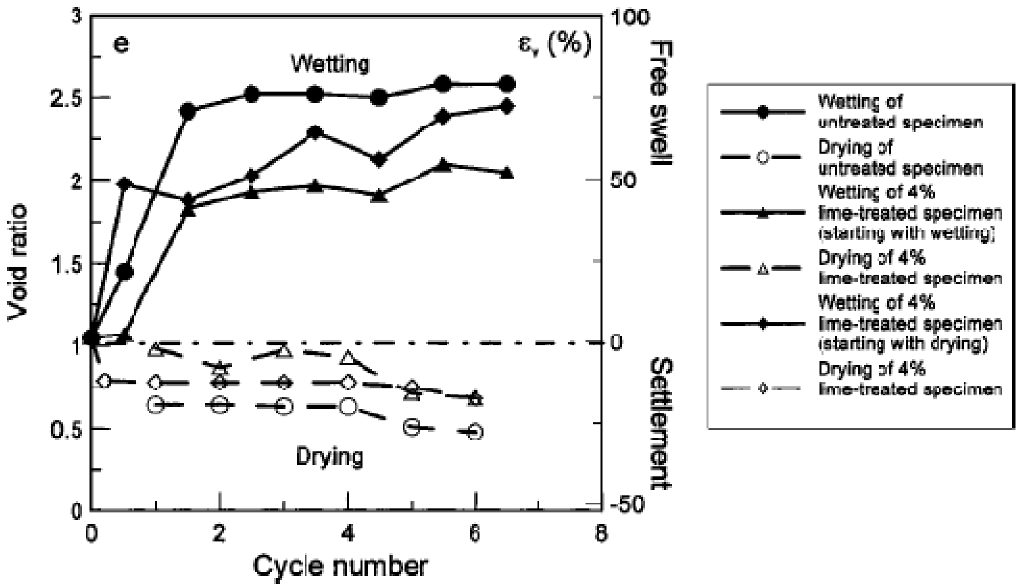


Figure 2.43 Volumetric changes of lime treated and untreated specimens during wetting and drying cycles. (Source: Khattab et al. 2007)

Guney et al. (2007) investigated impact of cyclic wetting and drying on swelling behavior of lime stabilized clayey soils. With cyclic wetting and drying the swelling potential of unstabilized clays was reduced.

Maximum swelling potential was reduced during the first cycle and then reduced gradually with subsequent cycles reaching equilibrium after 4 to 6 cycles.

In the case of lime stabilized soils, the stabilization effect was found to be lost with increase in number of cycles of wetting and drying. The clay content of the cycled samples increased which affected the plasticity index, shrinkage limit and swell potential of the lime treated expansive soil.

Gunery et al (2007) recommended that the lime treatment may not be used at regions susceptible to severe wetting and drying cycles.

2.13.6 Use of Cement as a Soil Admixture

Cement is another commonly used soil stabilizing agent which is composed of calcium, silica, alumina and iron (Guney et al. 2007). Cement stabilization has been practiced for over seventy years. By adding cement to the soils, the calcium ions are released resulting in reduction of the plasticity index (Bugge and Bartelsmeyer, 1961).

The pozzolanic reactions increase strength property and reduce swelling potential (Nelson and Miller, 1992). Cement treated soils are mixed with low cement dosages with or without a targeted strength, where as the cement stabilized soils are mixed with high cement dosages to achieve desired strength (Zhang et al. 2008). INDOT (2002) prescribes a dosage of 3% to 10% apart from the following criteria to be treated as reactive soil for cement stabilization.

For Modification, % soil passing sieve No. 200 $\leq$ 35 and PI < 5

For Stabilization, PI $\leq$ 10 and minimum clay content <20%

Durability of cement treated soils is found to be satisfactory in soils with less free moisture content than with more free moisture content (Zhang et al. 2008). However, the current research is focused on preventing desiccation cracking. Cement stabilization increases susceptibility for shrinkage cracks and makes the surface brittle and more impermeable. As such, cement is not chosen as one of the stabilizing agent for this research conducted at The University of Texas at Arlington.

2.13.7 Use of Lime or Cement for Sulfate Rich Soils

Lime and Cement stabilized sites which were performing satisfactorily after construction have subsequently undergone considerable heaving leading to pavement and other infrastructure failures at number of sites (Mitchell and Dermatas, 1992; Hunter, 1988; Perrin, 1992).

These heave failures have been attributed to the presence of sulfates in soils. The calcium component of lime or cement stabilizer is known to react with clay components and ground sulfates to form a highly expansive crystalline mineral known as ettringite. When water gains access to a treated soil containing these minerals, swelling and softening of soils take place. Millions of dollars are annually spent to repair these distressed structures. Hence, it is important to study the surrounding environments and their influence on soil of heaving caused by sulfates following lime stabilization (Puppala et al. 2007).

Hunter (1988) presented models of lime-montmorillonite reactions that are valid for other clayey soils. In this model, the OH^- retained from lime hydration combines with montmorillonite, $Al_2Si_4O_{10}(OH)_2^-$, to form $2Al(OH)_4$, which then reacts with sulfates to form the ettringite mineral. The normal chemical reactions between these minerals are shown in the following:

$$Al_2Si_4O_{10}(OH)_2*nH_2O + 2(OH)^- + 10H_2O \rightarrow 2Al(OH)_4^- + 4H_4SiO_4 + nH_2O$$

(Dissolution of clay mineral, at pH>10.5)

$$6Ca^+ + 2Al(OH)_4^- + 4OH^- + 3(SO_4)^{2-} + 26H_2O \rightarrow Ca_6[Al(OH)_6]_2*(SO_4)_3*26H_2O$$

(ettringite formation)

These compounds lead to the formation of ettringite crystals ($Ca_6[Al(OH)_6]_2*(SO_4)_3*26H_2O$) at moderate to high temperature conditions (25 to 40°C), which can expand twice or three times of their original sizes when subject to hydration. Once the ettringite crystal is formed, it continues to grow in almost pure form.

Figure 2.44 presents a typical scanning electron image of ettringite mineral. This picture indicates that this mineral is of needle structure, but can also come in different forms such as rod and lathe like structures. Rod like structures form at an early stage during high pH condition and needle like structures form at a later stage when pH is decreased.

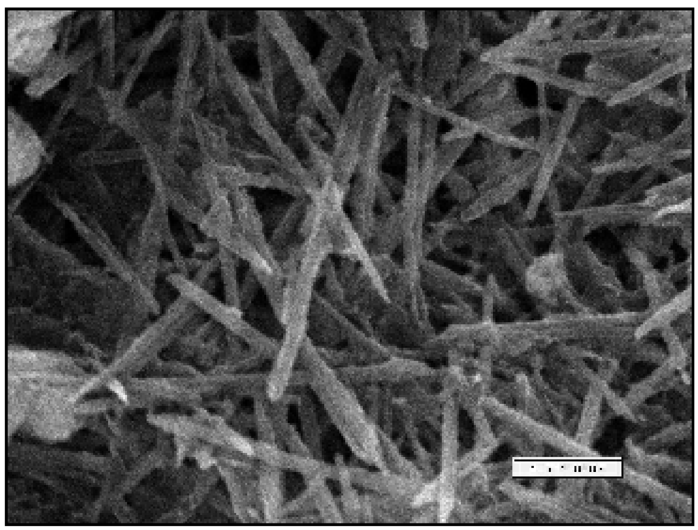


Figure 2.44 Scanning electron micrograph of Ettringite

When the temperature of the system reaches less than 15°C and conditions with abundant presence of soluble carbonate content, ettringite is transformed by a series of intermediate reactions to thaumasite mineral, $[Ca_3Si(OH)_6]_2(SO_4)(CO_3)_2*26H_2O]$. The expansion potential of thaumasite mineral is much higher than that of ettringite (Kollman and Strubel, 1981).

Both ordinary Portland cement and lime treatments were found to be ineffective at high sulfate levels and there is a need to investigate sulfate levels before selecting a soil treatment or modification method in the field (Puppala et al. 2007). Type III cement is more suitable for low sulfate level soils and Type V cement is better suited for sulfate rich soils in case the cement stabilization is inevitable.

University of Texas at Arlington has been of late exploring various alternative methods for stabilization of sulfate rich soils using sulfate resistant cementitious and recycled stabilizers (Hoyos et al. 2004).

2.14 Use of Compost as a Soil Admixture

Texas is one of the largest producers of waste materials in USA (EPA, 1997; Puppala et al, 2004). TxDOT used recycled asphaltic pavement (RAP) and cemented quarry fines (CFQ) for pavement base or sub base materials (Puppala et al. 2008). Other waste materials or recycled materials used for pavement applications include blast furnace slag, steel slag, and coal combustion by products and compost materials in highway construction (Schroeder, 1994).

Desiccation cracks are noticed on the unpaved shoulders of highways in longitudinal and transverse directions. Intrusion of surface runoff or rainfall infiltration into the cracks further weakened the base and subgrade layers (Puppala et al, 2004). Moisture affinity (hydrophilic characteristics) and presence of fibrous material in the compost help reduce the shrinkage of natural expansive subgrades when stabilized with compost.

Compost materials are capable of maintaining uniform moisture level by absorbing moisture from the atmosphere which in turn will help prevent desiccation cracking (Puppala et al, 2004).

2.14.1 Manufacturing of Compost

Compost is relatively a stable and decomposed organic material obtained from the composting process of different types of wastes (Puppala et al, 2004). Composting is recognized as one of the innovative method of recycling organic waste materials.

Composting is a natural process of aerobic, thermophilic, microbiological degradation of organic wastes in to a stabilized and useful product that is free of odors and pathogens (Girovich, 1966). Benefits of compost addition have been identified by various agencies as shown in Table 2.1.

Table 2.1 Benefits of addition of compost identified
by various Agencies / Researchers

	USCC	**US EPA**	**Mitchell, D.**	**Univ. of Georgia**	**Univ. of Florida**
Improves soil structure, porosity, bulk density	✓		✓		
Increases water holding capacity of soil	✓	✓		✓	✓
Increases infiltration and permeability of soils	✓		✓	✓	
Erosion control	✓		✓	✓	
Helps moderate soil temperatures					✓
Adds organic bulk and humus to regenerate poor soils		✓	✓	✓	✓
Helps suppress plant diseases and pests		✓			

(Modified from Jennings et al., 2003)

2.14.2 Application of Compost

Various applications of compost are landscaping, land reclamation, erosion control, and top dressing of golf course / parks, agriculture, residential gardening and nurseries. Intharasombat (2005) conducted using various types of compost as shown in Figure 2.45. These composts were used to treat the top layer of shoulder soil test sections and the performance was monitored with heavy instrumentation, digital imaging and elevation survey for a period of more than 2.5 years.

The studies have shown that right selection of compost helped mitigate shoulder cracking of highway pavements by reducing shrinkage cracking. It was also concluded that out of the various types of composts used for study Bio-solids compost, Cotton Burr compost were found to be more suitable to enhance properties of expansive clayey soils. Dairy manure was not found to be very suitable for preventing shoulder cracking. All the composts were found to be in general good at promoting vegetation growth.

However, compost amended soils have shown high swell strains which were attributed to the hydrophilic characteristics of ingredients of compost (Puppala et al, 2004).

Studies by Xiao et al. (2006) have shown that compost has a good potential for rainfall erosion control. For a road side embankment, filtered compost and vegetated compost were used. Compost of three different pellet sizes was laid on up-slope, mid-slope, and down-slope with finer compost on up-slope and coarser compost on down-slope. Vegetated compost has the composted surface vegetated with grass. The filtered compost application significantly reduced the soil erosion and the vegetated compost showed capability of sustaining repeated rainfall and act like a promising long term erosion control blanket (Xiao et al. 2006). The results also showed that the soil loss due to erosion was within the tolerable limits.

Stephenville		**Dairy Manure Compost** **Organic Content 10%**		**Biosolids Compost** **Organic Content 41%**
Lubbock		**Cotton Burr Compost** **Organic Content 65%**		**Feedlot Manure Compost** **Organic Content 30%**
Bryan		**Biosolids Compost** **Organic Content 58%**		**Wood Compost** **Organic Content 68%**
Corpus Christi		**Cow Manure Compost** **Organic Content 18%**		**Biosolids Compost** **Organic Content 45%**

Figure 2.45 Various types of compost used for research (Source: Intharasombat, 2005)

2.15 Use of Fibers as a Soil Admixture

Deterioration of concrete structures owing to corrosion has called for a new quest of an ideal and durable material which has the desired properties of low shrinkage, good thermal expansion, substantial modulus of elasticity, high tensile strength, improved fatigue and impact resistance. This lead to emergence of fiber reinforcement in concrete applications (Brown et al. 2002). Soon, the application found place in the geotechnical engineering field.

Soil reinforcement implies inclusion of strips, sheets, nets, mats and synthetic fiber to reduce tensile strain (Kumar and Singh, 2008). Strips, geosynthetics consist of continuous inclusions in to earth mass where as fiber reinforcement is injected in a random pattern. These inclusions act to interlock particles as a coherent matrix and the main advantage is the increase in strength of soil (Maher and Gray, 1990).

The mechanism of strength improvement is similar to that of root reinforcement. Roots mechanically reinforce a soil by transfer of shear stress in the soil to tensile resistance of roots (Gray and Sotir, 1996). When shear occurs in the soil, the root fiber or synthetic fiber deforms causing an elongation of fiber provided there is sufficient interface friction along the length of fiber and confining stress to lock the fibers in place and prevent slip or pull (Gray and Sotir, 1996).

The laboratory experiments revealed that the shear strength increase was linear with increase in fiber content. The fiber aspect ratio, L/d has an influence on the shear strength. Higher the L/d ratio, higher is the contribution of fiber to the shear strength (Maher and Gray, 1990).

2.15.1 Types of Fibers

Various natural and synthetic fibers are used for soil reinforcement. The most commonly used natural fiber is coir (Babu and Vasudevan, 2008). Natural Fibers mixed with soil have applications in irrigation and drainage projects such as river levees, bunds, and temporary canal diversions, check dams etc., (Babu and Vasudevan, 2008). Studies by Babu and Vasudevan (2008) using natural coir fiber have shown that with the increase of fiber content, seepage velocity decreased and piping resistance of soil increased. Wood pulp or wood fibers present in the compost also cause similar effect of soil reinforcement. Most commonly used synthetic fibers are polypropylene fibers, polyvinyl chloride and glass. Fibrous carpet waste was also used in some countries like Iran for soil reinforcement (Ghiassain, 2004). Latest developments include use of adhesive coated natural or synthetic fibers to prevent erosion and strength loss in berms and embankments.

2.15.2 Properties of Polypropylene Fibers

The most commonly used synthetic fiber for concrete or soil reinforcement is polypropylene fibers. Polypropylene fibers are available in the form of fibrillated films and tapes or woven meshes. They have better bond than chopped monofilament fibers (Brown et al. 2002). Propylene is an unsaturated hydro carbon, containing only carbon and hydrogen atoms. Polypropylene is a versatile thermoplastic material which is produced by polymerizing monomer units of polypropylene molecules into very long polymer molecules or chains in the presence of a catalyst (Brown et al. 2002). The mechanical properties of polypropylene are as indicated below (Brown et al. 2002).

- Tensile Strength: 25 - 33 MPa (522-689 ksf)
- Flexural Modulus: 1200 - 1500 MPa (25062-31328 ksf)
- Elongation at break: 150 – 300%
- Strain at yield: 10 – 12%

2.15.3 Various Findings and Recommendations by Researchers

Puppala and Musenda (2000) reported that the fibers improved unconfined compressive strength and reduction in volumetric shrinkage strains and swell pressures of expansive clays. Heineck et al. (2005) conducted ring shear tests and bender element tests and concluded that the contribution of polypropylene fiber reinforcement is more effective after a certain level of shear strain.

Tingle et al. (2002) after conducting a series of field studies concluded that discrete geofiber stabilization of sand was a viable alternative to traditional stabilization techniques for low volume road applications.

Miller and Rifai (2004) recommended use of fiber reinforcement for waste containment liners as they found the use of fiber reinforcement reduced the desiccation cracking phenomenon of clay liners. However, increase of fiber content beyond 1% was significantly increasing the hydraulic conductivity of clay liners.

Welker and Josten (2005) carried out direct shear tests and suggested an optimum dosage of 0.2% for reinforcing clayey soil.

2.16 Mixing of Soil, Fibers and Cement / Lime

Consoli et al. (1998) have conducted experiments on fibers and cement mixed with cohesion less soil and compared relative performance. The peak friction angle of uncemented cohesion less soil increased from 35º to 46º due to fiber inclusion. Addition of cement to soil increased stiffness and peak strength. Fiber reinforcement increased both peak and residual triaxial strengths. 3% of Fiber reinforcement of soil mixed with 1% cement decreased stiffness and changed the brittle behavior cemented soil to a more ductile. Cai et al. (2006) conducted experiments on soil mixed with different proportions of lime and fibers and reported beneficial changes in the properties of soil. It was reported that the unconfined strength, cohesion and friction angle increased with increase in curing period. The fiber-lime-soil exhibited high strength, improved toughness, and swell and shrinkage properties.

2.17 Summary

In order to understand the intricacies involved with the surficial failures, a detailed overview of various kinds of slope failures of both natural slopes and engineered studies was reviewed from the available literature. Various measures taken to prevent slope failures and ensure safety of slopes were discussed. A brief account of important research findings of various authors including case studies was discussed which was provided help to carry out the present research tasks.

CHAPTER 3

LABORATORY EXPERIMENTAL PROGRAM

3.1 Introduction

One of the major tasks of this research work is to construct test sections at Joe Pool Dam and Grapevine Dam to study the performance of various admixtures to prevent desiccation cracking and surficial failures. Five test sections include one control section to enable comparison of the relative performance of the treated sections. The treated sections comprise of soil mixed with 20% compost, soil mixed with 4% lime with 0.30% polypropylene fibers, soil mixed with 8% lime with 0.15% polypropylene fibers, and soil mixed with 8% lime.

The details pertaining to construction of test sections is described in Chapter 4. During the construction of test sections field soil samples were obtained from both the dam sites at the time of mixing and placement to conduct laboratory tests on all the ten samples.

The laboratory testing program is designed to carry out all the basic engineering tests, required mineralogical tests, strength tests, and swell and shrinkage tests on the field construction samples. The detailed procedure of carrying out these tests is illustrated in this chapter.

The results are compared with the test results of earlier investigations performed by McCleskey (2005) on the borrow soils obtained from Joe Pool Dam and Grapevine

Dam to ensure quality control. Additional laboratory studies including torsion ring shear and hydraulic conductivity tests are conducted on field treated soil mixtures and the results are also presented in this chapter.

3.2 Laboratory Test Procedures

Laboratory test procedures are briefly explained as following:

3.2.1 Sieve Analysis Test

The grain size distribution of the soil was determined using TxDOT procedure Tex-110-E. Sieve analysis test was carried out on the control soil samples obtained from both the Dam sites for the purpose of classification of soil. Dry soil was pulverized with a rubber tipped pestle and the soil was passed through a set of sieves. The stack of sieves was kept in a mechanical shaker shown in Figure 3.1 for 15 minutes. The percentage of soil retained was calculated.

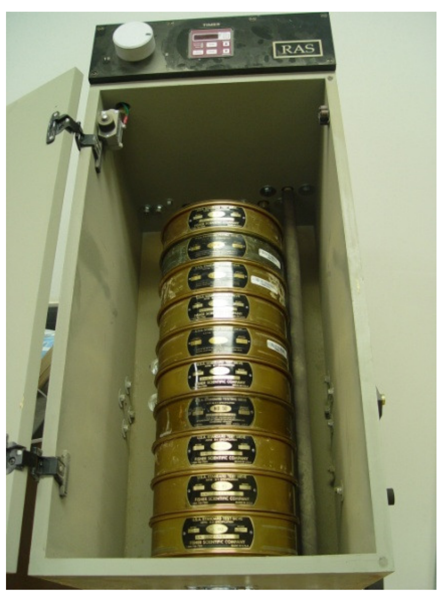


Figure 3.1 Stack of sieves in a mechanical shaker

Wet analysis was carried out as for the cohesive soil by washing the soil retained on sieve No. 200. Soil passing through 75 micron size was dried and hydrometer analysis was conducted as explained in the section 3.2.2.

3.2.2 Hydrometer Analysis

Hydrometer Analysis was carried out to study the micro level distribution of silt and clay fraction present in the field soil of Joe Pool Dam and Grapevine Dam. At first, 50 g of oven dried and well pulverized soil was mixed with a solution containing a 4% deflocculating agent (Calgon) and soaked for about 8 to 12 hours.

A 1000 cc graduated cylinder was kept ready with 875 cc of distilled water mixed with 125 cc of deflocculating agent. The temperature was recorded. Meniscus correction and zero corrections are observed.

The prepared soil was thoroughly mixed in a mixer cup and all the soil solids inside the mixing cup was transferred to a 1000 cc graduated cylinder. The graduated cylinder was filled with distilled water till the mark. The hydrometer readings were recorded at cumulative time of 0.25 min., 0.5 min., 2 min. 4 min., 8 min., 15 min., 20 min., 2 hr., 4 hr., 8 hr., 12 hr., 24 hr., 48 hr., and 72 hr.

After taking the readings initially for the first 2 minutes, the hydrometer was taken out and kept in another cylinder filled with distilled water. Necessary temperature corrections, zero corrections and meniscus corrections were made to the hydrometer readings as per procedure. The percentage finer was calculated and the grain size distribution curve was plotted for the soil and presented in Figure 4.9.

3.2.3 Specific Gravity Test

Specific gravity is defined as the ratio of the mass of a given volume of solid to the mass of an equal volume of distilled water. The specific gravity was determined as per ASTM standard method D854-06.

3.2.4 Atterberg Limits

Atterberg limit tests were conducted on field soil samples to determine the plasticity properties of the soils. Both liquid and plastic limit tests were conducted as per ASTM D-4318 standard test methods. Atterberg limit tests reveal properties related to consistency of the soil. LL is measured as the water content at which the soil flows and the PL is determined as the water content at which the soil starts crumbling when rolled into a 3 mm (1/8 in.) diameter thread. These plasticity tests are somewhat operator sensitive.

The numerical difference between LL and PL values is known as plasticity index (PI) and this index characterizes the plasticity nature of the soil. The water content of the samples during tests is measured using oven drying method. The plasticity index is used to classify the soil as per USCS.

3.2.5 Standard Proctor Tests

Standard Proctor test method using Tex-114-E procedure was followed to determine moisture content versus dry density relationships. The optimum moisture content of the soil is the water content at which the soils are compacted to a maximum dry unit weight condition. The optimum moisture content and maximum dry density values were initially obtained by conducting proctor tests on the borrow soils of both the dam sites. Standard Proctor tests were conducted on both control and treated soils. By adding the stabilizer agent such as lime or compost to the control soil, both physical and chemical properties of the mixed soils change. In order to ensure quality control during construction, the optimum moisture content and maximum dry unit weight required to be achieved were specified in the specifications supplied to the contractor before commencement of test section construction. The actual moisture content and dry density achieved was verified by conducting non-destructive field test using nuclear gage and desired level of quality control was achieved. The results are presented in Tables 4.6 and 4.7.

3.2.6 Linear Shrinkage Bar Test

The Linear shrinkage bar test used in this research was based on the procedure established by the Texas Department of Transportation (Tex-107-E standard method). This test measures the volumetric shrinkage (width, depth, and height) of the soil samples in the shrinkage mold shown in Figure 3.2.

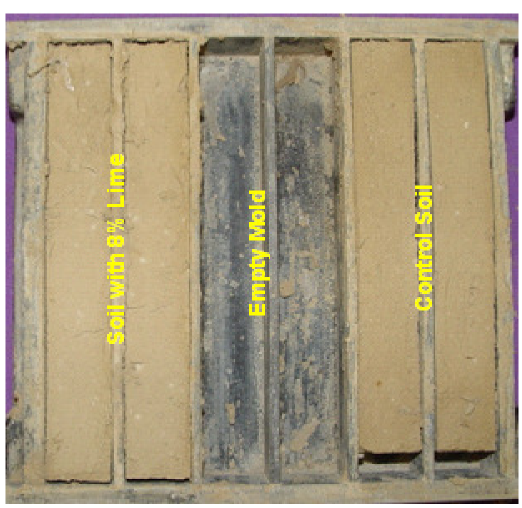


Figure 3.2 Linear shrinkage test setup

Soil samples were first mixed with water level corresponding to the liquid limit state, and then the samples were molded and placed in a linear shrinkage block, which are 12.7 cm (5 in.) long and 1.9 cm (0.75 in.) width and depth.

Soil samples were kept at room temperature condition for twelve hours. Then, the soil samples were dried in the oven at 110°C. The length, width and height of dried sample were measured by vernier calipers and the volumetric shrinkage was calculated and expressed as a percent of its original volume.

3.2.7 Free Swell Test

One dimensional free swell test represents the field condition of dam slope. The result of the test gives the heave potential of soil. Samples of control soil and treated soils were obtained from both the dam sites and remolded samples are prepared to fit a conventional oedometer steel ring of size 64 mm (2.5 in.) in diameter and 25 mm (1 in.) in height.

The free swell is measured by observing the change in dial gage readings for a period of 24 hours. The free swell is measured from the dial gage having a least count of 0.001 in. the swell measured is presented in the form of percentage over the thickness of 1 in. of soil sample.

The results are used for QC/QA assessments as explained in Chapter 4. Alternative wetting and drying effect on swell of soil samples was also studied and the results are presented in section 6.3.

3.2.8 pH Determination

Determination of pH of the soil helps in studying the solubility of soil minerals and the mobility of ions in the soil. The pH measuring equipment used in this research is shown in Figure 3.3.

Figure 3.3 Test setup for measuring pH

This test was performed by following ASTM D-4972 specification. In order to find the pH value of the soil samples, a 1:1 ratio of dried soil to distilled water was used in this method. The soil samples were first mixed with distilled water and then shaken and mixed again to ensure thorough mixing. Then, the pH was monitored by inserting an electrometric indicator into the soil mix, which provides the pH conditions in the soil. This pH test results were used while determining soluble sulfate levels and for quality control tests.

3.2.9 Soluble Sulfate Determination

The soluble sulfate content was required to be determined at the time of selection of borrow soil for the Joe Pool Dam site. Later the tests were also carried out on the control soil of both Joe Pool Dam and Grapevine Dam to determine if the soil was having any soluble sulfates.

The soluble sulfate content in the soil is an important test property that is known to assess the sulfate heaving process in chemically treated soils. Hence, an attempt was made to measure soluble sulfates in soil. The established test procedures for sulfate analysis yield results with high standard deviation values.

This makes it difficult to assess or interpret which sulfate levels the soil samples truly contained. Hence, an attempt was made to evaluate the current sulfate determination methods and modify the method that provides repeatable results with low standard deviations.

The method used in this research was a modified procedure from the standard gravimetric method outlined in the 17^{th} edition of Standard Methods for the Examination of Water and Wastewater.

10 grams of dried soil was diluted with 100 mL of distilled water. The extraction of the solution was obtained by centrifuging with the speed of 14,000 rpm.

The pH values of the solution were controlled within the range of 5 to 7 by Hydrochloric acid. Barium Chloride ($BaCl_2$) was then added in the boiling solution to bring out sulfate in the form of Barite ($BaSO_4$).

The solution was placed in an 85°C oven for 12 hours to continue the digestion process in which precipitation takes place to obtain Barite by gravimetric process.

The barite precipitated from this process was calculated to obtain the soluble sulfate contents in the soil samples. Puppala et al. (2002) used a smaller pore size filter of 0.1 μm and higher speed of centrifuging of 14000 rpm with longer time in order to segregate the small particles from the solution. This modified method provided results that match with ion chromatography measurements.

Various sequences of soluble sulfate test carried out are depicted in Figure 3.4.

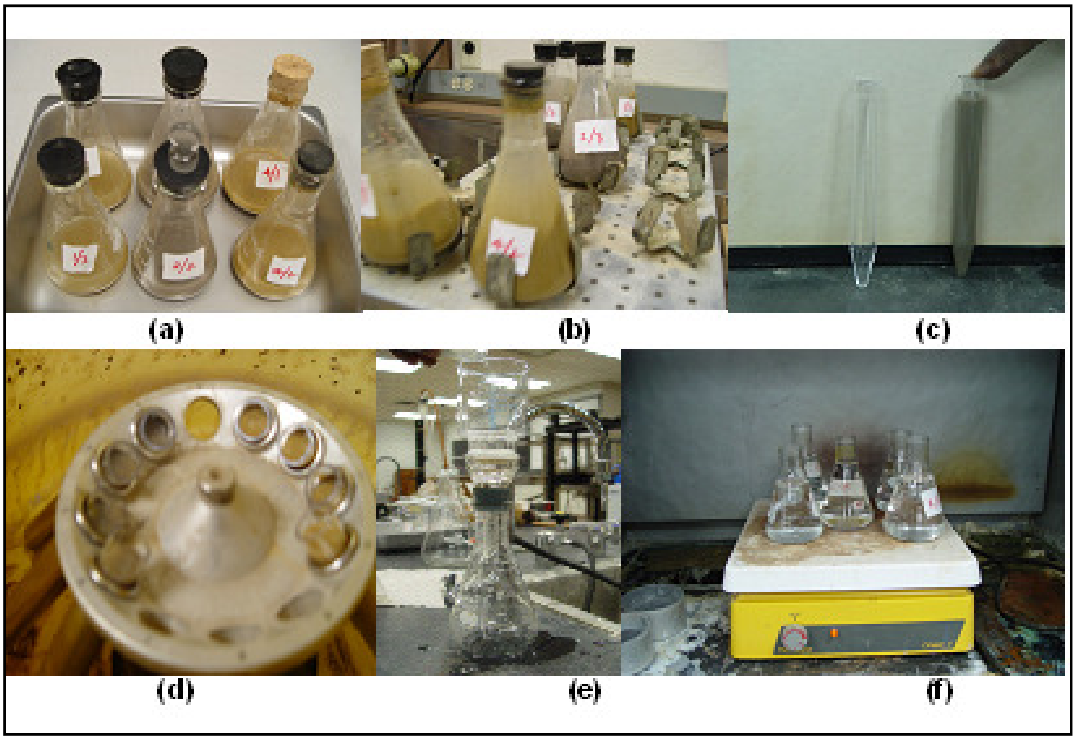


Figure 3.4 Sequence of soluble sulfate test

The flowchart showing the procedure to carry out the test is shown in Figure 3.5. The results of the test were found to be useful during the construction of test sections.

Borrow soils having higher concentration of soluble sulfates were discarded and soil having very less concentration of soluble sulfates ware used for making up the bottom soil of the Joe Pool Lake Dam test section during construction.

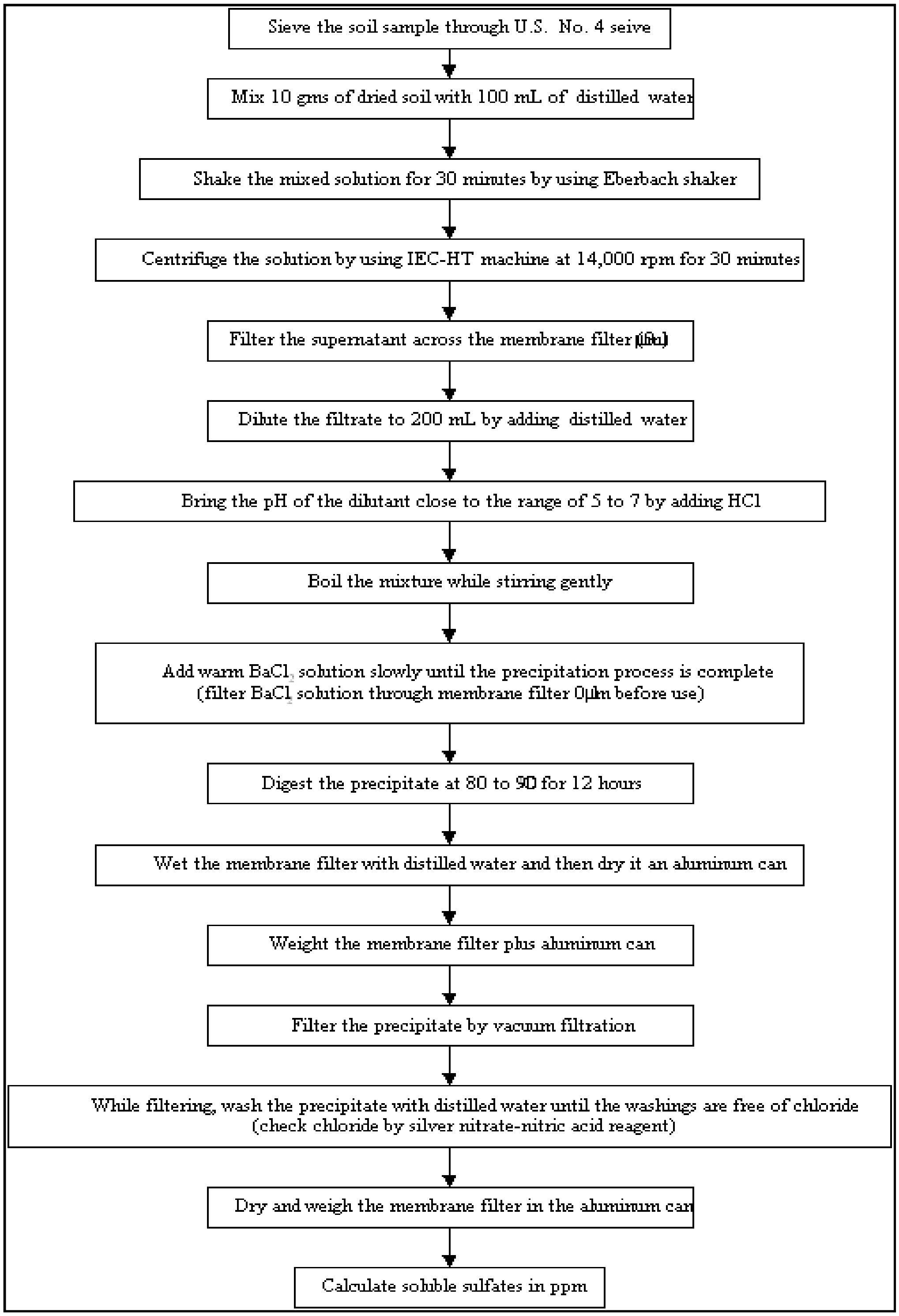
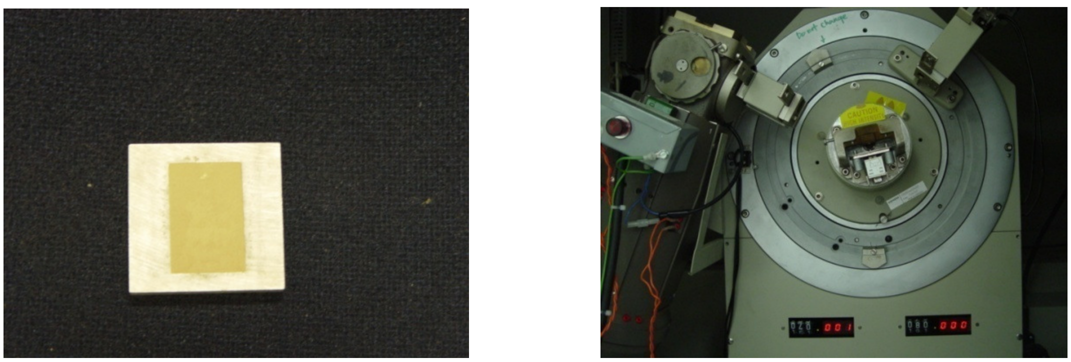


Figure 3.5 Flow chart showing sulfate test procedure

3.2.10 X-Ray Diffraction Test

Powder X-Ray diffraction method was used to identify the mineral composition of the soil samples. This test can only identify the presence of the minerals in the samples without any effort to measure the quantity. The soil samples were first dried and sieved through a US No. 230 standard size. The soils in the form of powder were then subjected to CuKα radiation with the speed of 2 degrees per minute in order to read the basal spacing of different minerals present in soils.



(a) (b)

Figure 3.6 X-ray diffraction test (a) Soil sample (b) D-500 assembly

The data was recorded and analyzed to determine the presence of minerals that causes heaving problems in this research study. Figure 3.6 shows the X-Ray diffraction set up. The test was conducted to confirm that there is no ettringite formation in the lime treated soil of Joe Pool Dam.

3.2.11 Scanning Electron Micrograph

Scanning electron microscope (SEM) studies were conducted in order to visually observe and record the crystal shapes of the ettringite minerals in microscopic orientation views of the samples. This method was used for the micro fabric study of clays. The magnification of SEM can go up to 150,000 times the original size. Gold coating was used on the powder samples prior to scanning.

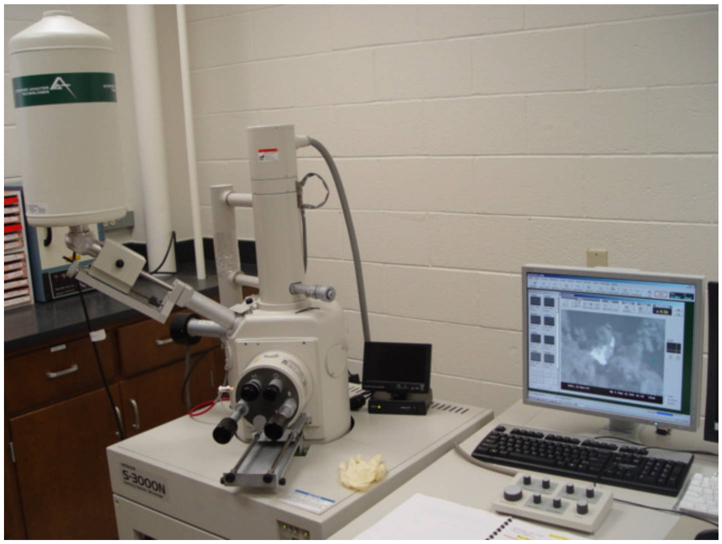


Figure 3.7 Scanning electron micrograph equipment

Once scanning was performed on gold-coated soil samples, they were pictured in digital format. These pictures were used to identify the presence of ettringite in the sample. Typically, the ettringite minerals appear in needle shapes at higher magnifications in the digital photographs. The test set up is shown in Figure 3.7.

3.2.12 Direct Shear Apparatus

Direct shear test was performed on all the untreated and treated soil specimens to obtain drained shear strength parameters of the test soils. Normal stresses applied were 25 kPa (0.52 ksf) and 50 kPa (1.04 ksf) which are representative of overburden stresses for shallow overburden depths. Soil specimens were sheared at a slower rate of 0.035 in./min (0.09 cm/min). Computer controlled equipment available in UTA laboratory was used for carrying out the direct shear tests.

3.2.13 Bromhead Ring Shear Apparatus

In this research, Bromhead ring shear test apparatus available in UTA laboratory was used to obtain the drained shear strength parameters of soil and soil treated with stabilizers. Test procedure prescribed by ASTM D 6467-99 was followed.

3.2.13.1 Description of Test Apparatus

The equipment contains a shear device which holds the specimen securely between two porous inserts and provides a means of applying normal stress to the faces of the specimen, permitting drainage of water through the top and bottom boundaries of specimen. The device is capable of applying a torque to the specimen along a shear plane parallel to the faces of specimen. At the inner and outer walls of the specimen container friction is developed during shear.

The device is capable of shearing the specimen at a uniform rate of displacement. The rate of displacement can be selected using a combination of gear wheels from 44.52 mm/min. travel to 0.018 mm/min. travel. The specimen container is annular in shape with an inner diameter of 70 mm and outer diameter of 100 mm. the container radially confines the 5 mm thick soil specimen. Due to this confinement wall friction is developed at the inner and outer circumference of the specimen. The magnitude of the wall friction is the least at the top porous stone and the soil interface and increases with the depth of the specimen. Thus the failure plane occurs at the surface of the top porous stone where the wall friction is the least. This type of failure condition is referred to as *smear* condition.

3.2.13.2 Test Procedure

The equipment contains a Bromhead torsion ring shear device. The preparation of soil specimen and the set up for preconsolidation is shown in Figure 3.8.

Figure 3.8 Description of torsion ring shear test procedure

Soil specimen is prepared at a water content equal to the liquid limit and placed in the annular space of the bottom platen and the top platen is placed over it. The specimen is pre-consolidated under a water bath at a load increment ratio of one at applied normal stresses of 25 kPa (0.52 ksf), 50 kPa (1.04 ksf), 100 kPa (2.08 ksf). For each load increment, it was ensured that primary consolidation was complete. In order to reduce the amount of horizontal displacement required to reach a residual condition, the specimen was pre-sheared at a constant rate of displacement of 18 mm/min.

After completing pre-shearing, identical soil specimens were sheared at various normal stresses and at a very slow rate of displacement. A similar rate of displacement used for direct shear test is used for torsion ring shear test also. Slow rate of displacement allows dissipation of pore pressures and helps obtain realistic drained shear strength values. As the analysis was focused on shallow slope failures, the results obtained at lower normal stresses is discussed as it simulates the field condition.

3.2.14 Suction Measurements by Pressure Plate and Filter Paper Method

Several test methods including filter paper and pressure plate method are commonly used to develop Soil Water Characteristic Curves (SWCCs) of unsaturated soils studies. The limitation of the pressure plate device is that it can measure matric suction up to only 1,000 kPa (20.9 ksf) to 1,500 kPa (31.3 ksf). The capacity is sometimes limited by availability of pressure plate and capacity of compressor. Therefore, filter paper method was used to measure soil suction ranging more than 1,000 kPa (20.9 ksf). Hence, both pressure plate and filter paper methods were employed in the development of a complete SWCC of the present soils.

3.2.14.1 Pressure Plate Method

Figure 3.9 shows the schematic of a typical pore water extraction testing setup using a pressure plate apparatus. The primary components of the system are a steel plate pressure vessel and a saturated High Air Entry (HAE) ceramic plate. As shown, a small water reservoir is formed beneath the plate using an internal screen and a neoprene diaphragm.

The water reservoir is vented to the atmosphere through an outflow tube located on top of the plate, thus allowing the air pressure in the vessel and the water pressure in the reservoir to be separated across the air-water interfaces bridging the saturated pores of the HAE material (Lu and Likos, 2004). Specimens are initially saturated, typically by applying a partial vacuum to the air chamber and allowing the specimens to imbibe water from the underlying reservoir through the ceramic disk.

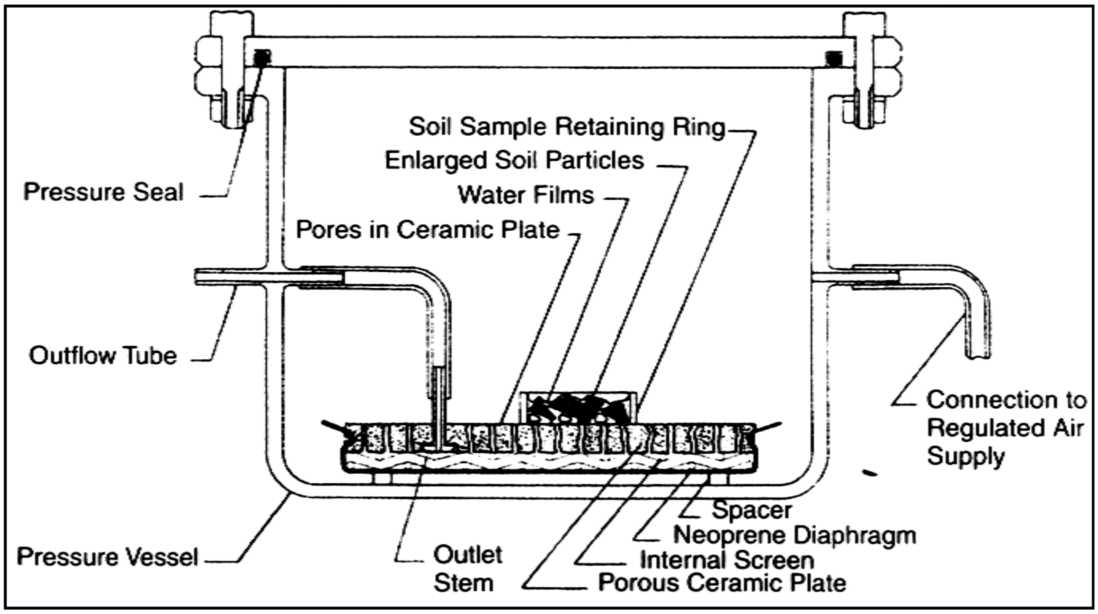


Figure 3.9 Schematic drawing of pressure plate
(Soil-Moisture Equipment Corp., 2003)

Figure 3.10 shows the set up used for conducting pressure plate equipment to measure matric suction. Air pressure in the vessel is then increased to some desired level while pore water is allowed to drain from the specimens in pursuit of equilibrium.

The outflow of water is monitored until it ceases, the pressure vessel is opened, and the water content of one or more of the specimen is measured, thus generating one point on the soil-water characteristic curve.

Subsequent increments in air pressure are applied to generate addition points on the curve using the other specimen.

Figure 3.10 Closed pressure vessel after applying air pressure

To measure the higher suction values, filter paper method was used. Description of this method is as following:

3.2.14.2 Filter Paper Method

For filter paper method, a filter paper (Schleicher & Schuell No. 589-WH type) is suspended in the headspace above the specimen such that moisture transfer occurs in the vapor phase.

The equilibrium amount of water absorbed by the filter paper is a function of the pore-air relative humidity and the corresponding total soil suction. The water content of the filter paper was measured after it reached equilibrium with the soil through vapor for a period of ten days. The calibration chart is shown in Figure 3.11.

The matric suction was estimated from the filter paper's moisture content using the calibration curve proposed by Bulut, Lytton, and Wray (2001).

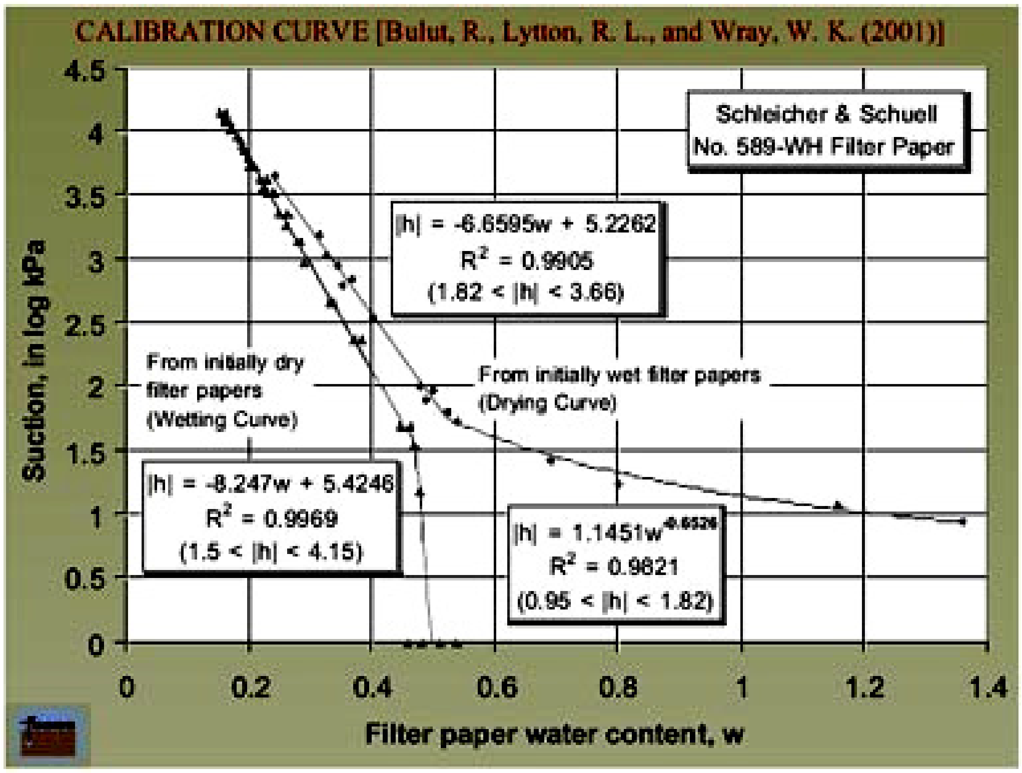
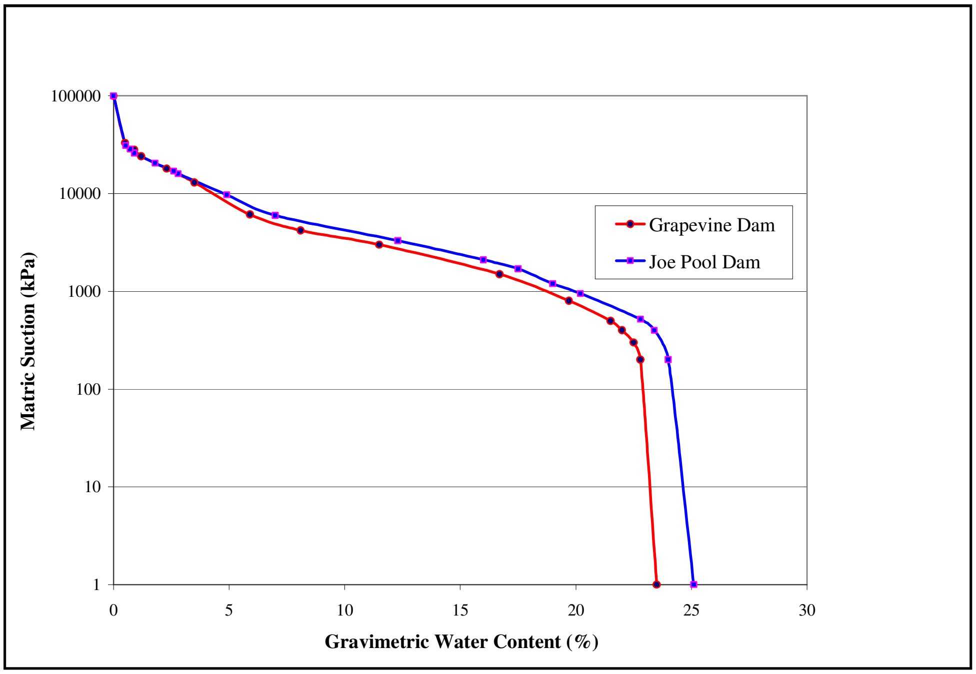


Figure 3.11 Calibration curves (Bulut, Lytton and Wray, 2001)

3.2.14.3 Suction Measurements

Number of samples at varying moisture content was used to obtain suction measurements using filter paper technique. Using the calibration chart the moisture content absorbed by the filter paper was converted in to suction. The gravimetric moisture content of each sample was measured. Suction measured by conducting pressure plate test and filter paper test for both the Joe Pool Dam and the Grapevine Dam is presented in Figure 3.12. Gravimetric water content is plotted on the ‘x-axis’ and suction is plotted in kPa on the ‘y-axis’.



3.12 SWCC for control soil of Joe Pool Dam and Grapevine Dam

The SWCC is found to be useful while carrying out analytical model study using PLAXFLOW software to input material properties. The SWCC can also be used to assess the amount of suction in the field by knowing the soil moisture content at any given time.

3.2.15 Permeability Test

Falling head permeability test was conducted on all the soil samples obtained from test sections. The equipment used for the test is shown in Figure 3.13. The results of the permeability test are presented in Table 3.4.

The Grapevine Dam soil is found to be having higher hydraulic conductivity than the Joe Pool Dam soil. The treated soils are found to be having higher permeability than the control soil.

3.13 Setup of permeability test

3.3 Research Conducted by McCleskey (2005)

A brief summary from the laboratory research of McCleskey (2005) conducted at the University of Texas at Arlington, Texas, USA is presented below. The study was conducted on soil samples obtained from borrow sites of Joe Pool Dam and Grapevine Dam during 2004. The index properties of borrow soil are reported in Table 3.1.

Table 3.1 Index properties of borrow soil (McCleskey, 2005)

Property	Joe Pool Dam	Grapevine Dam
% Passing No. 200 sieve	69.4	57.5
% Clay fraction	10.5	15.5
Liquid limit	58	30
Plastic limit	24	17
Plasticity index	34	12
USCS classification	CH	CL

All the tests conducted on borrow soil are repeated on soil samples obtained from the field test sites. The index properties of the soil obtained from field samples from construction site is presented in Table 4.2. The type of soil encountered during construction almost matched with the type of soil obtained from borrows pits of both the dam sites which were used for laboratory studies by McCleskey (2005).

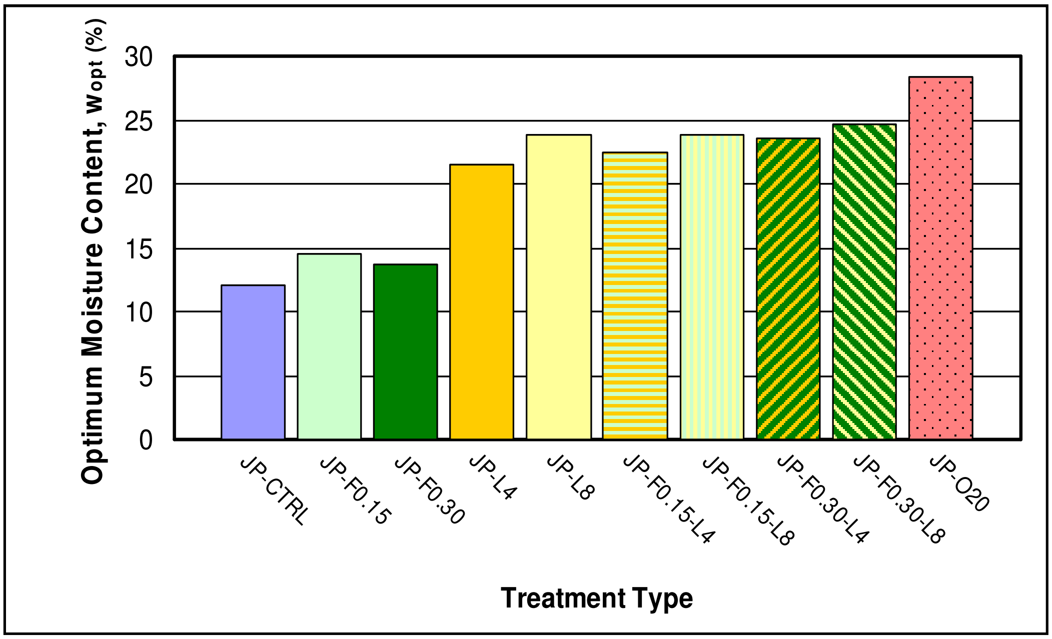
3.3.1 Results of Laboratory Tests

Various tests including standard proctor compaction, direct shear, volumetric shrinkage, one dimensional free swell test were conducted to study the effect of stabilizers. Various types of admixtures and soil samples used and their notation followed is indicated in Table 3.2.

Table 3.2 Types of sample specimens tested and notation used

Treatment type	Notation for Joe Pool Dam	Notation for Grapevine Dam
Dam control soil	JP-CTRL	GV-CTRL
Soil with 0.15% polypropylene fibers	JP -F0.15	GV-F0.15
Soil with 0.30% polypropylene fibers	JP -F0.30	GV-F0.30
Soil with 0.40% polypropylene fibers	JP -F0.40	GV-F0.40
Soil with 4% hydrated lime	JP -L4	GV-L4
Soil with 8% hydrated lime	JP -L8	GV-L8
Soil with 0.15% polypropylene fibers and 4% lime	JP -F0.15-L4	GV-F0.15-L4
Soil with 0.15% polypropylene fibers and 8% lime	JP -F0.15-L8	GV-F0.15-L8
Soil with 0.30% polypropylene fibers and 4% lime	JP -F0.30-L4	GV-F0.30-L4
Soil with 0.30% polypropylene fibers and 8% lime	JP -F0.30-L8	GV-F0.30-L8
Soil with 20% Compost (Organic)	JP -020	GV-020

The control soil from Joe Pool Dam and Grapevine Dam was classified as Sandy Lean Clay (CL) and Sandy Fat Clay (CH), respectively. The maximum dry density and optimum moisture content for control soils and treated soils is shown in Figures 3.14 and 3.15. Figure 3.16, Figure 3.17 and Figure 3.18 shows the test results of all the soil samples compacted at optimum moisture content.



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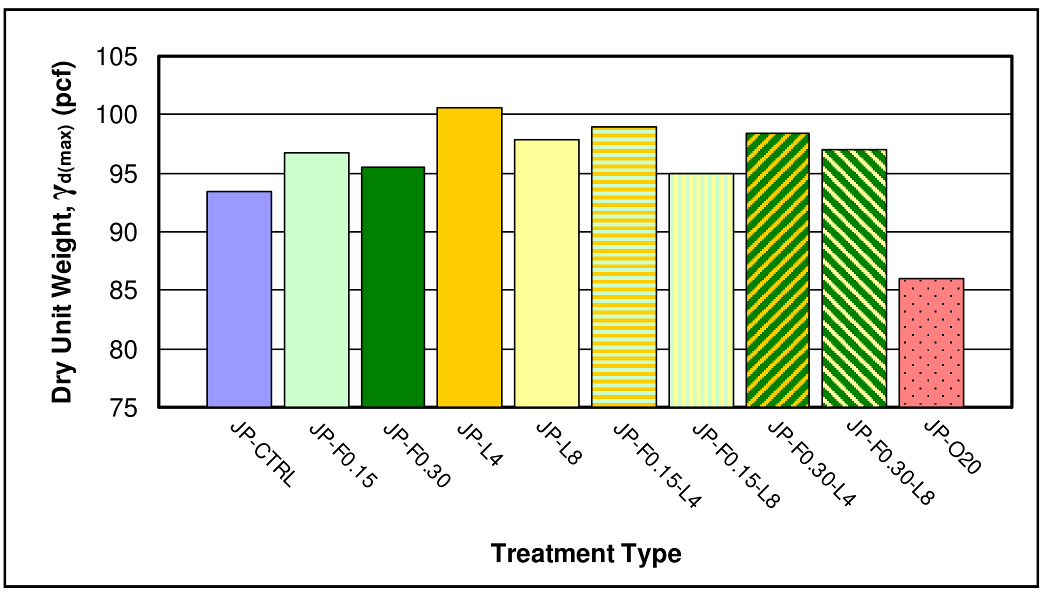
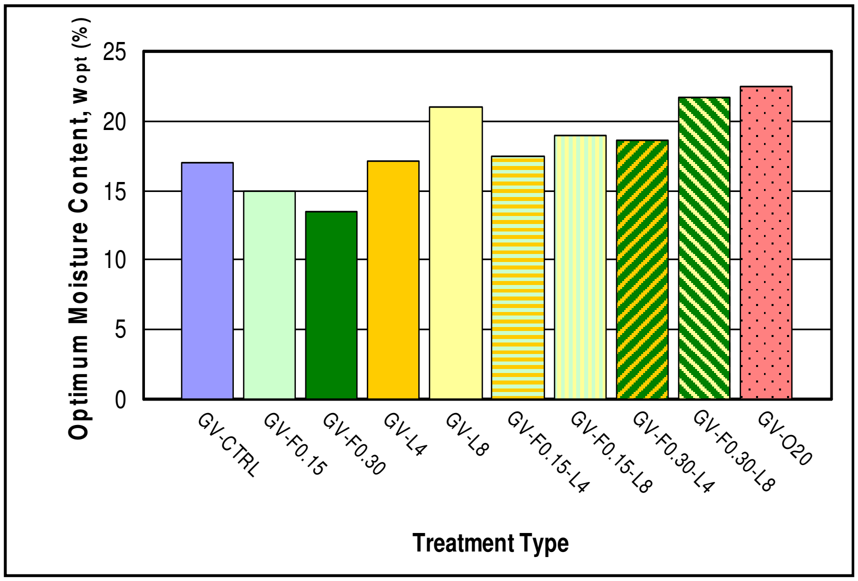
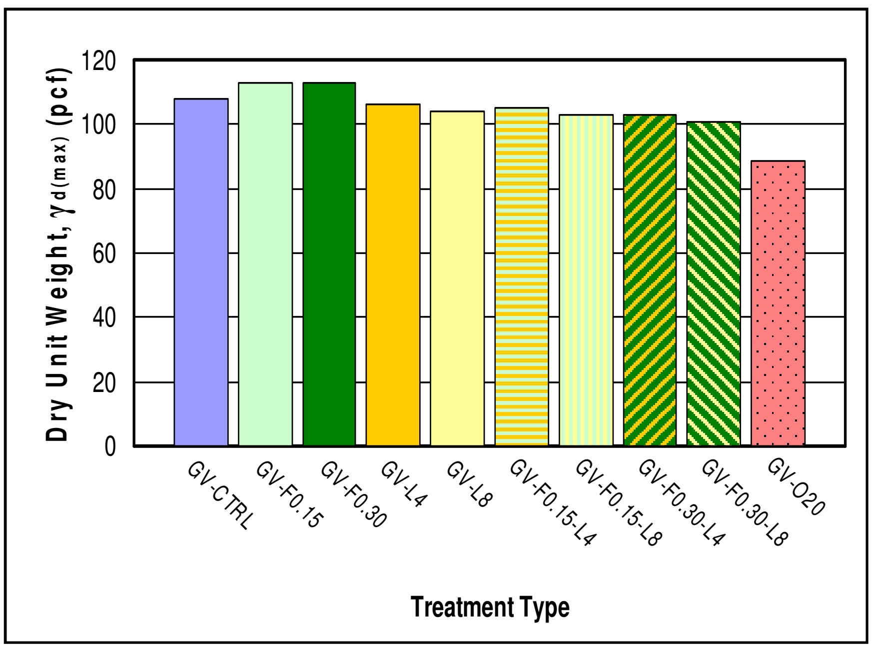


(b)

Figure 3.14 Joe Pool Dam Soil (a) Optimum moisture content (b) Maximum dry density (McCleskey, 2005)

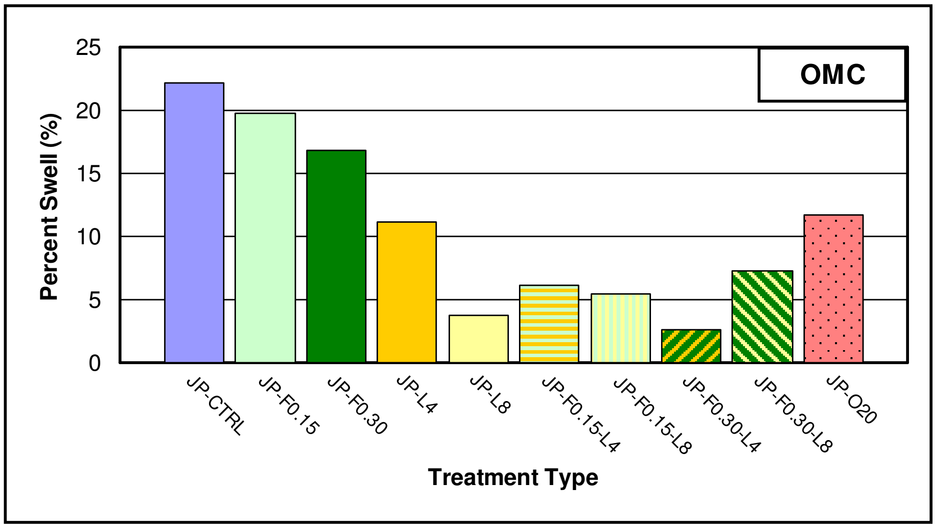


(a)



(b)

Figure 3.15 Grapevine Dam Soil (a) Optimum moisture content
(a) Maximum dry density (McCleskey, 2005)



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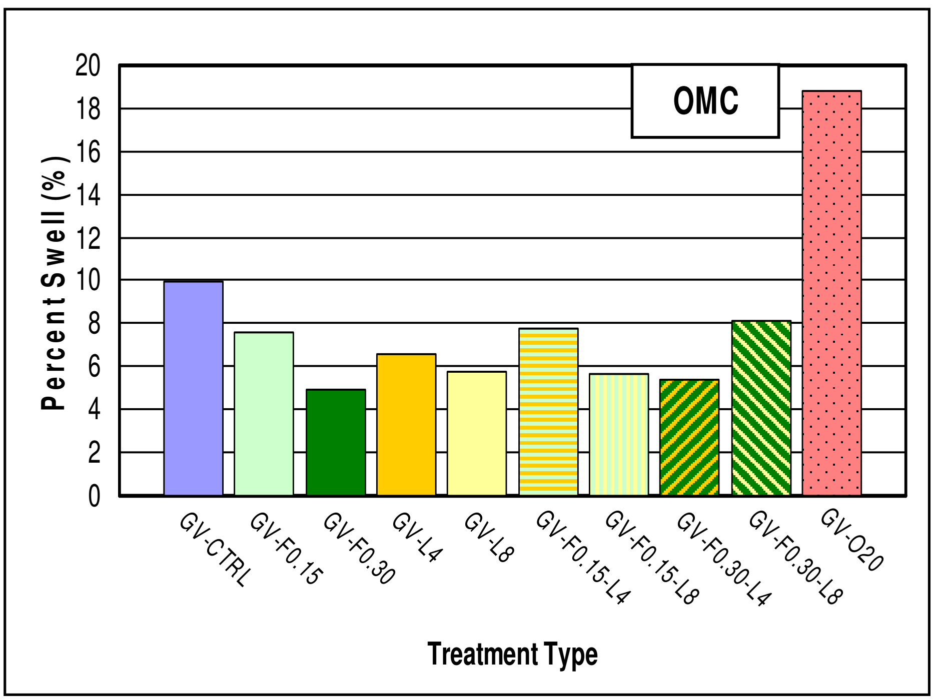
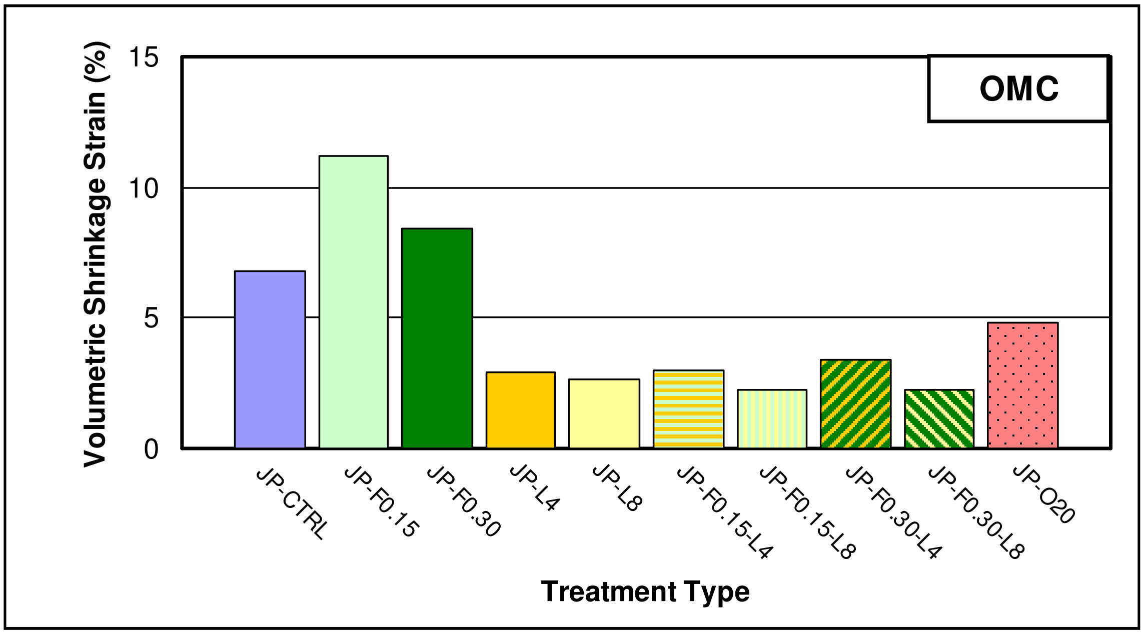
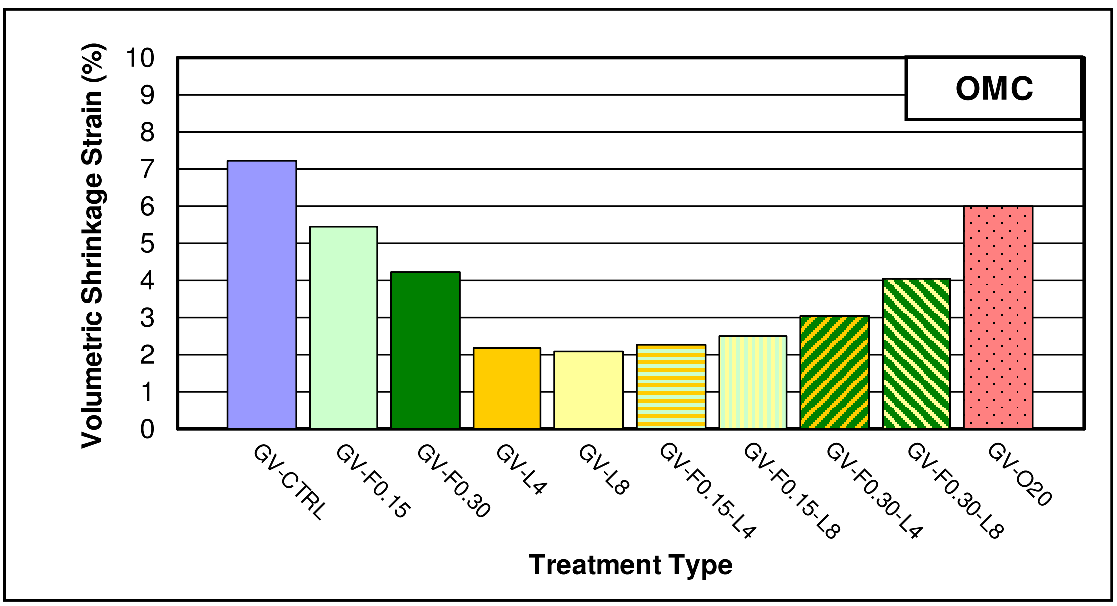


(b)

Figure 3.16 Swell strain data (a) Joe Pool Dam soil and (b) Grapevine Dam soil (Source: McCleskey, 2005)

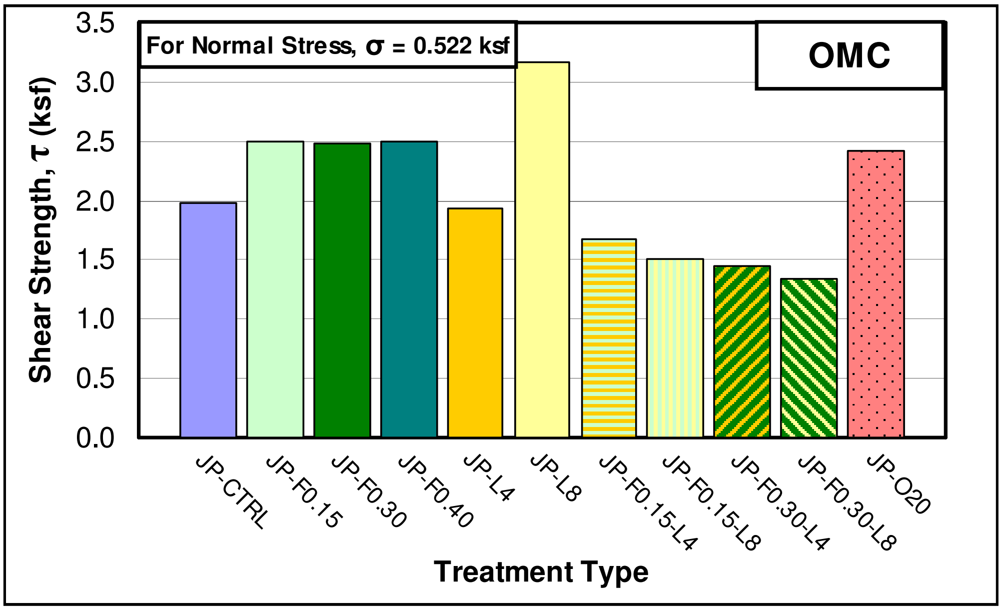


(a)

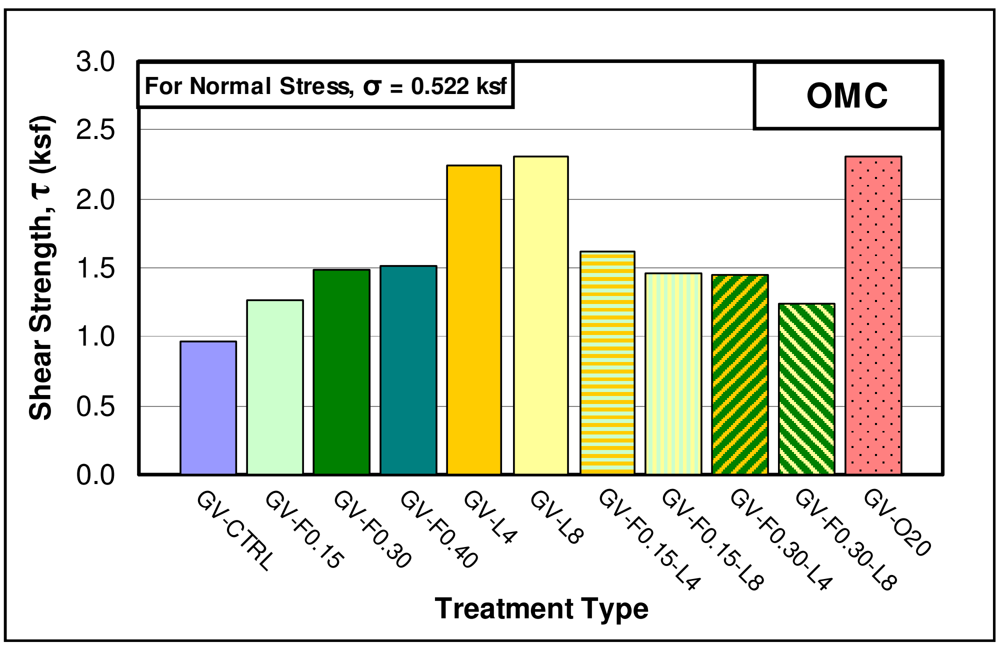


(b)

Figure 3.17 Volumetric shrinkage strain data (a) Joe Pool Dam soil and (b) Grapevine Dam soil (Source: McCleskey, 2005)



(a)



(b)

Figure 3.18 Direct shear test Results data (a) Joe Pool Dam soil and (b) Grapevine Dam soil (Source: McCleskey, 2005)

3.3.2 Recommendations of McCleskey (2005)

Important interpretations of the laboratory tests conducted and conclusions based on the analysis of data are discussed here.

Swell test results indicated that the lime and lime with fiber treatment has reduced the swelling potential of soil. Addition of fibers to the soil did not improve the swell properties of soil. Swelling was also observed to be high in the case of soil specimens prepared with compost amended soil.

Shrinkage test results indicated that both Lime and Lime with fibers were proven to be an effective treatment to prevent shrinkage cracks in the soil. Addition of lime has significantly improved the shear strength of soil. Addition of fibers to the soil has also improved the tensile strength of soil mixtures.

In order to achieve the goal of mitigating the desiccation cracks of the soil, the treatment should be aimed at reducing shrinkage and swelling tendencies besides possessing adequate shear resistance and possible tensile strength. Lime and fibers were found to be most effective to reduce the volumetric changes of soil besides improving strength.

Lime in conjunction with fibers was also found to be very effective. Compost treatment is preferred from the view point that it is hydrophilic and preserves enough moisture content always which helps prevention of desiccation cracks. The performance of compost treatment to highway shoulders from another study conducted at UTA was instrumental in adopting similar treatment method for surficial slope stability.

McCleskey (2005) based on the above study, made the following recommendation with respect to field additive treatments to construct test sections at Joe Pool Dam and Grapevine Dam sites:

- 8% Lime
- 8% Lime with 0.15% Fiber
- 4% Lime with 0.30% Fiber
- 20% Compost
- Control section to compare the performance

3.4 Additional Tests on Field Samples

This section covers both torsion ring shear and hydraulic conductivity studies conducted by the author as a part of the present dissertation work. The main intent of these studies is to input these parameters for numerical studies attempted to address the stability of the dam embankments.

It should be noted that the soils used in these investigations are directly taken from the field construction test sections. Torsion ring shear test and hydraulic conductivity tests are conducted on all the field test section samples and the results are discussed as following:

3.4.1 Torsion Ring Shear Test Results

Torsion ring shear tests were conducted as per the test procedures explained in section 3.2.13 and a typical set of test results of both untreated control soil and lime treated soil from Joe Pool Dam are presented in Figure 3.19.

These results are used to determine the residual shear strength parameters. Peak envelopes were not prominent on the present soils as the embankment soils used for compaction are normally consolidated. Table 3.3 presents a complete summary of residual shear strength parameters of the control and treated soils from both Dam sites.

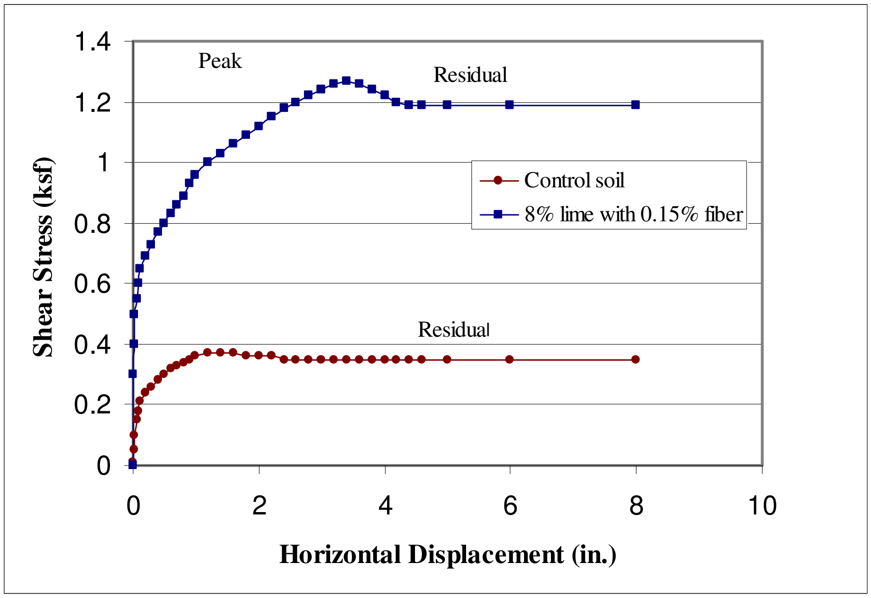


Figure 3.19 Torsion ring shear test data for the control soil and the 8% lime with 0.15% fibers

Table 3.3 Results of torsion ring shear test on field samples

Treatment	Joe Pool Dam		Grapevine Dam	
	Cohesion kPa (ksf)	Friction Angle (degrees)	Cohesion kPa (ksf)	Friction Angle (degrees)
Control	0 (0)	20	0 (0)	18
20% Compost	2.9 (0.06)	19	3.4 (0.07)	20
4% Lime with 0.30% Fibers	10.5 (0.22)	33	10.5 (0.22)	35
8% Lime with 0.15% Fibers	16.8 (0.35)	39	16.3 (0.34)	40
8% Lime	12.5 (0.26)	36	12.9 (0.27)	38

3.4.2 Hydraulic Conductivity Tests

Falling head tests were conducted on both control and treated soils to determine their hydraulic conductivity. Summary of these test results are presented in Table 3.4.

Table 3.4 Hydraulic conductivity of field samples

Treatment	Joe Pool Dam cm/sec (in./sec)	Grapevine Dam cm/sec (in./sec)
Control	2.7×10^{-6} (1.06×10^{-6})	8.1×10^{-6} (3.19×10^{-6})
20% Compost	8.6×10^{-5} (3.39×10^{-5})	9.3×10^{-5} (3.66×10^{-5})
4% Lime with 0.30% Fibers	7.7×10^{-6} (3.03×10^{-6})	3.1×10^{-5} (1.22×10^{-6})
8% Lime with 0.15% Fibers	2.7×10^{-5} (1.06×10^{-5})	4.4×10^{-5} (1.73×10^{-5})
8% Lime	1.1×10^{-5} (4.33×10^{-5})	2.3×10^{-5} (9.06×10^{-6})

3.5 Summary

This chapter provides a complete description of various test procedures used in the experimental program. Earlier research findings of McCleskey (2005) based on the laboratory tests conducted on the soil samples obtained from borrow sites of Joe Pool Dam and Grapevine Dam were summarized. All the basic engineering tests were carried out on the field soil samples and these results were primarily used to address quality control issues related to field treatment methods adapted in the field, which are explained in Chapter 4. The field performance of admixtures was studied comparing the relevant soil properties based on the laboratory test results. The analysis of field data is carried out to determine the best performing additives for field sections which are aimed at reducing desiccation cracks and mitigation of surficial failures in the field conditions. Torsion ring shear and hydraulic conductivity data from the laboratory studies performed on field samples during test section construction are also summarized and these results are used for analytical model studies.

CHAPTER 4

CONSTRUCTION OF FIELD TEST SECTIONS

4.1 Introduction

The detailed study of failure mechanism of surficial failures from the literature review underlines the fact that the desiccation cracks are the root causes of these problems. Desiccation cracks increase the infiltration because of increased hydraulic conductivity. When the rainfall encounters the bottom layer of soil having lesser hydraulic conductivity, seepage takes place parallel to slope. The process leads to saturation of top layer of soil mass resulting in increase of pore water pressure, soil softening and reduction of shear strength which may ultimately lead to a surficial failure. The imminent danger emphasizes the need to mitigate the desiccation cracks. Prevention of desiccation cracks reduces the infiltration during rainfall and it will prevent the top layer of soil from getting totally saturated. This will ensure unsaturated conditions and higher shear strength in top layers of soil which will help avert surficial failures.

Laboratory studies on representative soil samples obtained from borrow pit locations of Joe Pool Dam and Grapevine Dam of United States Army Corps. of Engineers (USACE) have proven that use of lime, lime mixed with fibers and compost as admixtures helped improve the shrinkage and swelling properties of soil (McCleskey, 2005). Different combination and proportions of lime, fibers and compost were studied in the laboratory of The University of Texas at Arlington, Texas, USA and the most optimum levels of dosages were recommended for field implementation.

4.1.1 Criteria for Selection of Test Sections

The USACE, as a part of its Water Resources mission under direct control maintains 609 dams, maintains and/or operates 257 navigation locks, and operates 75 hydroelectric facilities, generating 24% of the nation's hydropower and three percent of its total electricity. The Fort Worth district of USACE operates 25 reservoirs in the State of Texas. Out of the dams maintained in Fort Worth district, Joe Pool Dam and Grapevine Dam were selected for field implementation of the stabilizer treatments selected from the analyses of laboratory study results.

The specific site for construction of test section at each dam is selected in such a way that the slope is having virgin soil and it was not previously subjected any kind of failure and no repair work was done at the site location. Accessibility of site for the construction vehicles, tool and plants is also considered while selecting the site.

Both Joe Pool Dam and Grapevine Dam are located in Dallas Fort Worth Metroplex in the state of Texas, USA. Both the dams selected have experienced huge number of surficial failures since their inception (McCleskey 2005). The type of soil at these dam sites is different from each other which is expected to give a better insight into the aspects of the behavior of clayey soils when treated with chemical admixtures.

The dam sites are having a closer proximity to The University of Texas at Arlington. The Joe Pool Dam test section site is at a distance of about 24 km (15 miles) and the Grapevine Dam test section is at a distance of 40 km (25 miles) from UTA. A map showing the location of these two dams is shown in Figure 4.1.

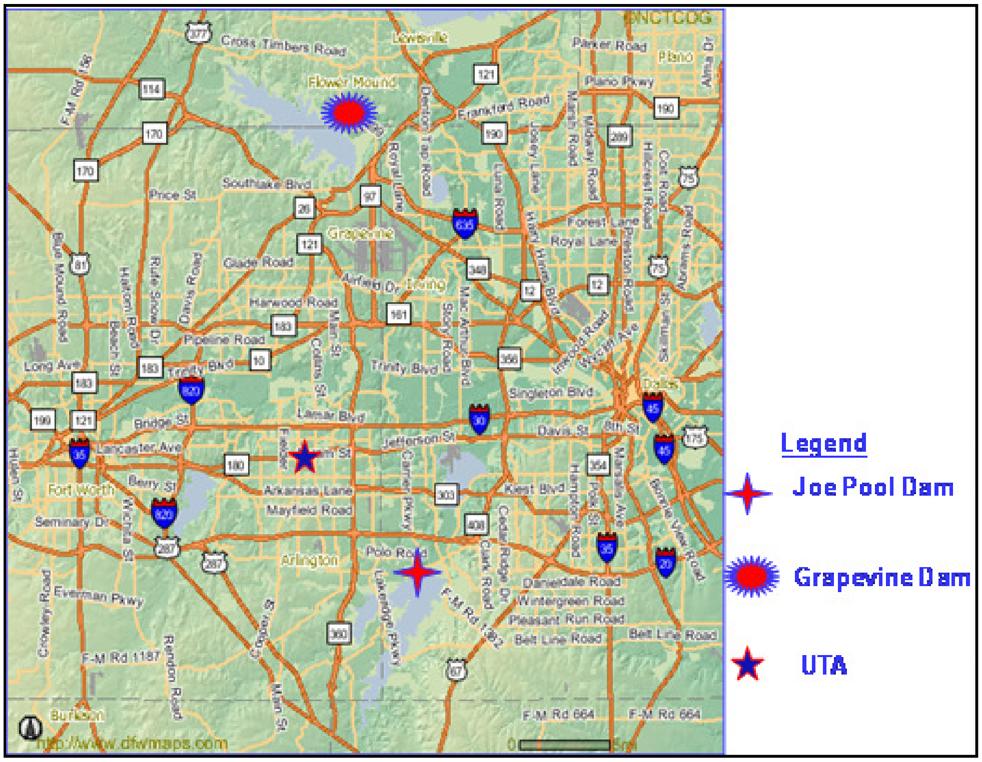


Figure 4.1 Map showing the location of Joe Pool Dam and Grapevine Dam
(Source: dfwmaps.com)

The climate of North Central Texas is humid subtropical with hot summers. There is a wide range of temperature variation in either extreme. Precipitation also varies from 500 mm to 1000 mm (20 to 40 in.).

The climatic conditions are found to suit ideally for a study involving alternate drying and wetting of soil. Before the commencement of work, all the activities were meticulously planned by USACE and UTA besides awarding the work to contractors to mobilise materials, equipment, and installing inclinometers.

4.2 Salient Features of Joe Pool Dam and Grapevine Dam

The details of Joe Pool Dam and Graprvine Dam are presented concisely in Table 4.1 indicating salient features.

Table 4.1 Details of Joe Pool Dam and Grapevine Dam (Source: USACE)

Item	Joe Pool Dam	Grapevine Dam
Location (Test Section)	Latitude : 32º 38’ 39.26” N Longitude: 97º 00’ 6.76” W	Latitude : 32º 58’ 27.48” N Longitude: 97º 03’ 23.31” W
Name of Creek	Denton Creek	Mountain Creek
Year of Construction	December 1979 – April 1986	January 1948 – June 1952
Year of Impounding	January 1986	July 1952
Age of the Dam	22 years	56 years
Length of Bank	6760 m (22180 feet)	3764 m (12350 feet)
Max. Height of Dam	33.07 m (108.5 feet)	39m (128 feet)
Width of Crest	9.1 m (30 feet)	8.5m (28 feet)
Vehicular Traffic	Not Allowed	Traffic Allowed
Crest Elevation (NGVD)	564.5 feet	588.0 feet
Impounding Capacity of dam	176,900 acre-feet of water	188,550 acre-feet of water
Geology of Dam Site	Upper Cretaceous age Eagle Ford Formation and Quaternary Age Alluvial and Terrace Deposits.	Upper Cretaceous age Woodbine Formation and Quaternary Age Alluvial and Terrace Deposits.
Slope of Dam	2.8H:1V	2.5H:1V

4.2.1 Joe Pool Dam

Joe Pool Lake is situated in the south part of the Dallas-Fort worth Metroplex. The location of test site is shown in Figure 4.2. The work of construction of test sections commenced during September 2007 and completed in three weeks.

Figure 4.2 Location of test sections at Joe Pool Dam
(Source: www.commons.wikimedia.org)

4.2.2 Grapevine Dam

Grapevine Dam was constructed for flood control purposes and was located 20 miles (32 km) northwest of Dallas. A layout plan of Grapevine Dam is shown in Figure 4.3. The location of proposed construction is shown in red in the plan. The work of construction of test sections commenced during July 2008 and completed in 4 weeks.

4.3 <u>Construction of Test Sections</u>

A layout showing the details of the proposed construction of test sections is shown in Figure 4.4. A detailed drawing showing plan and section prepared by USACE is shown in Figure 4.5 and 4.6 for Joe Pool Dam and Grapevine Dam respectively. The sequence of construction operations is illustrated as following:

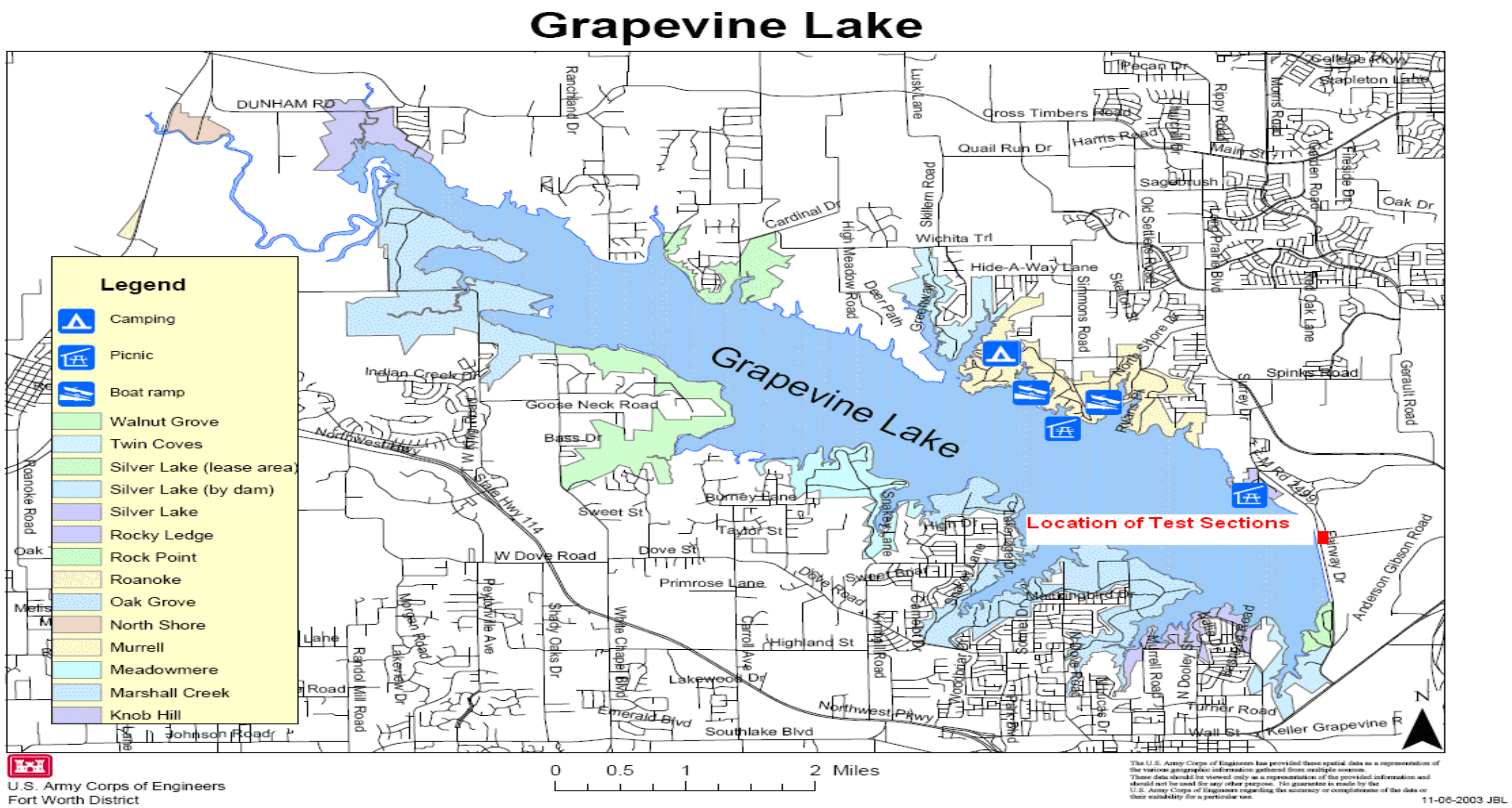


Figure 4.3 Map of Grapevine Dam (Source: USACE)

Figure 4.4 Layout for construction of test section 18 m x 7.5 m (60 ft x 25 ft)

4.3.1 Excavation of Top Soil

The grass and other vegetation on the surface were removed by mowing. After mowing, the proposed test sections are marked on the slope as per the drawing shown in Figure 4.5 and 4.6 with the help of wooden stakes.

During the construction of dam, the core soil was overlain by a top soil of about 23 cm (9 in.) thick for the purpose of vegetation growth. The treatment of admixtures is intended to be mixed with the core soil of dam.

As such, the top soil was excavated first using a back hoe as shown in Figure 4.7. The same soil was stockpiled aside for reuse to place it back over treated section after compaction of the 45 cm (18 in.) thick soil layer mixed with admixtures on the slope surface.

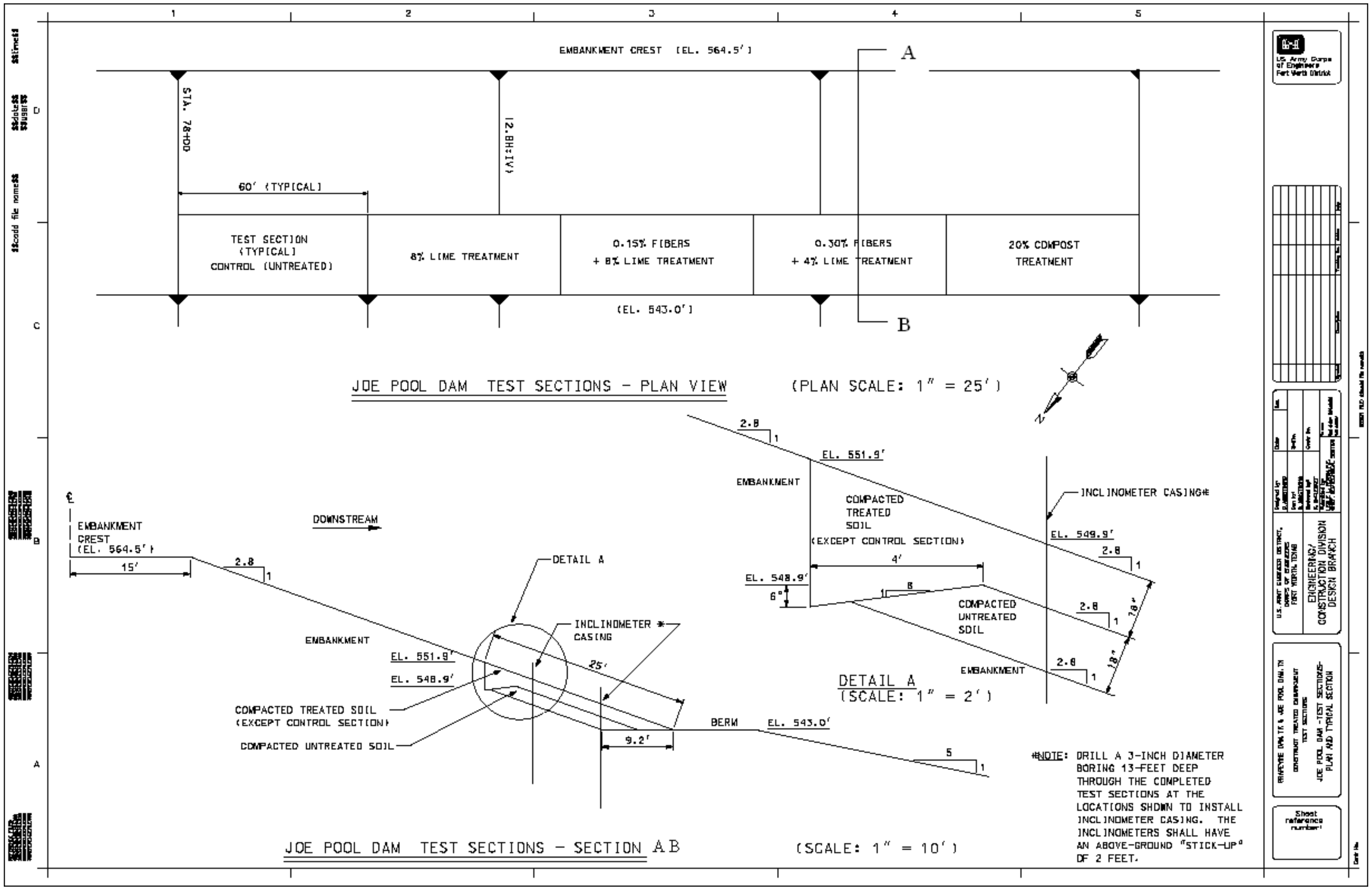


Figure 4.5 Plan and section showing the construction of test sections at Joe Pool Dam

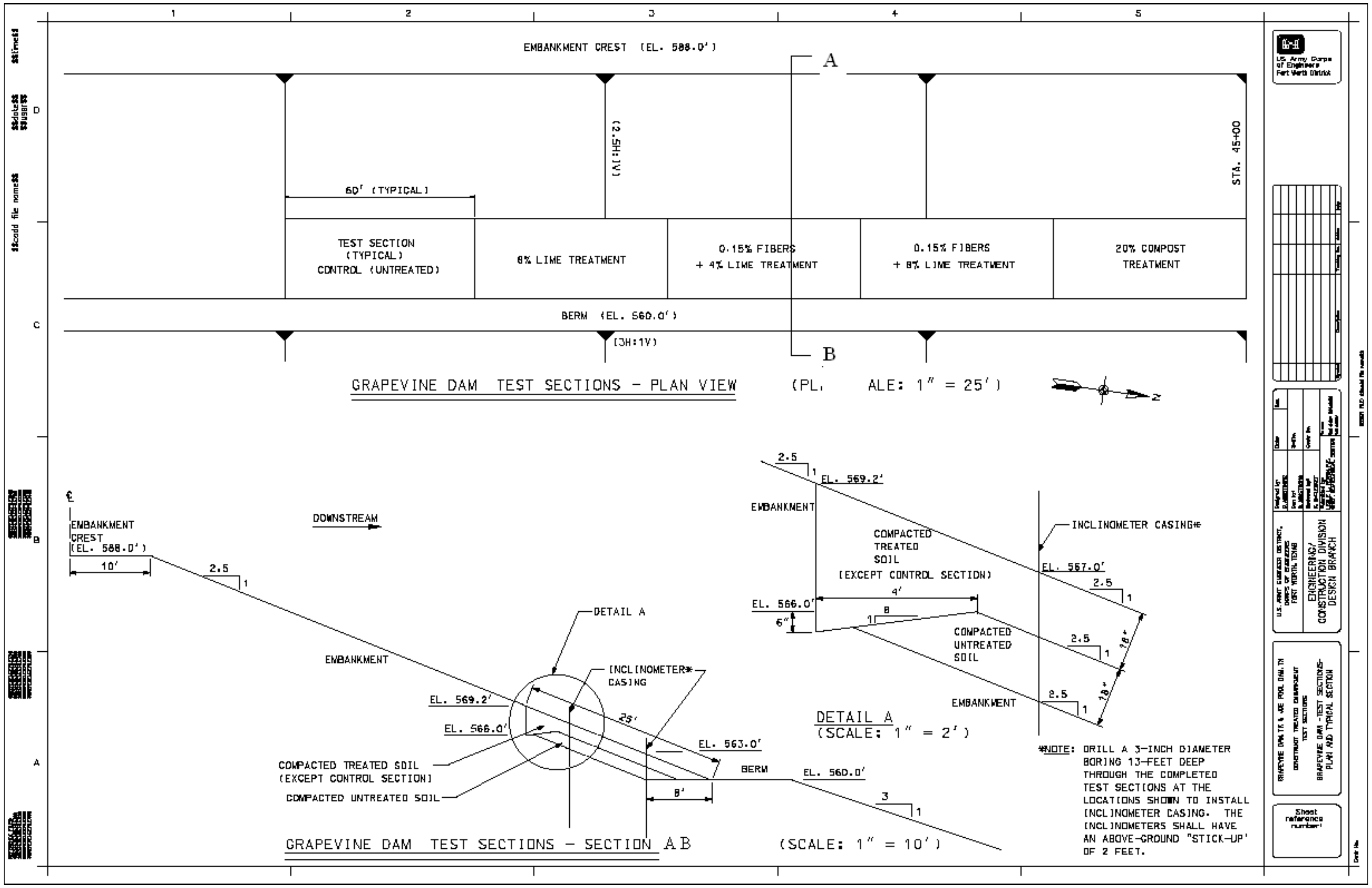


Figure 4.6 Plan and section showing the construction of test sections at Grapevine Dam

(a)

(b)

Figure 4.7 Excavation of top Soil (a) During excavation (b) Completed work

4.3.2 Excavation of Core Soil of Dam

After excavation of the top soil, the excavation of core soil of the dam was commenced as per the plan. The depth of excavation near the top was about 1.06 m (42 in.) as shown in Figure 4.8.

Figure 4.8 Depth of excavation near top of berm

After completion of the excavation of the core soil of the dam, the soil samples were collected from the field and all the basic and engineering laboratory tests were conducted. At Joe Pool Dam, the thickness of the top soil was found to be more than 150mm (6 in.). As such it became necessary to identify proper borrow pits so as to make up the bottom portion of the excavation.

Two borrow sites were selected for the Joe Pool Dam and the laboratory tests revealed that one of the borrow soils was from Eagle Ford formation location. The soluble sulfate content was found to be 2482 ppm. The soil was classified as high sulfate clay as the soluble sulfate content is more than 2000 ppm. When the soil sulfate soils come into contact with lime, one of the additives of this research, it can lead to the formation of ettringite mineral which further leads to heaving of soil. In order to prevent this heaving of lime treated soil, the borrow pit containing high sulfates was abandoned.

The other borrow soil is found to be having 29 ppm of soluble sulfate which is very low. The soil was classified as Fat clay, CH as per Unified Soil Classification System. The optimum moisture content of the soil was 22% and the maximum dry density was 15 kN/m^3 (96 pcf).

The approved borrow soil was used to prepare the bottom surface of the test section as shown in Figure 4.9 to receive 45 cm (18 inches) of treated soil. A berm was excavated with a reverse slope of 8H: 1V as shown in Figure 4.9 to have proper anchorage of treated soil with existing core soil of dam.

Figure 4.9 Preparation of bottom surface of excavation
(Inset: preparation of berm)

Laboratory tests were conducted on the core soil of both dam sites. Important basic properties of the soil are presented in Table 4.2. The test results were compared with the properties of soil samples of both dams as reported by McCleskey (2005).

It was confirmed that the properties of the borrow soils used in the laboratory studies and the actual field samples were almost similar and there were no major differences between them.

Table 4.2 Physical properties of core soil

Soil Property	Joe Pool Dam Soil	Grapevine Dam Soil
% Passing No. 200 Sieve	80	52
% of Clay Fraction	12	9
% Gravel	0	0
Specific Gravity, (Gs)	2.71	2.73
Liquid Limit (%)	58	29
Plastic Limit (%)	21	15
Plasticity Index	37	14
Activity	Active (3)	Active (1.6)
Optimum Moisture Content (%)	14	14
Maximum Dry Density	15.1 kN/m^3 (97 pcf)	17.96 kN/m^3 (115 pcf)
USCS Classification	CH (Fat clay with sand)	CL (Sandy Lean Clay)

Figure 4.10 shows the graph showing the particle size distribution of finer fraction of field soil sample obtained from Joe Pool Dam and Grapevine Dam based on the hydrometer tests.

As the soil is predominantly having aggregate finer than 75 micron, the grain size distribution of finer aggregate is presented. The grain size distribution data is found to be useful for classification of soil and also to obtain relevant material model while using PLAXFLOW software which is explained in Chapter 7.

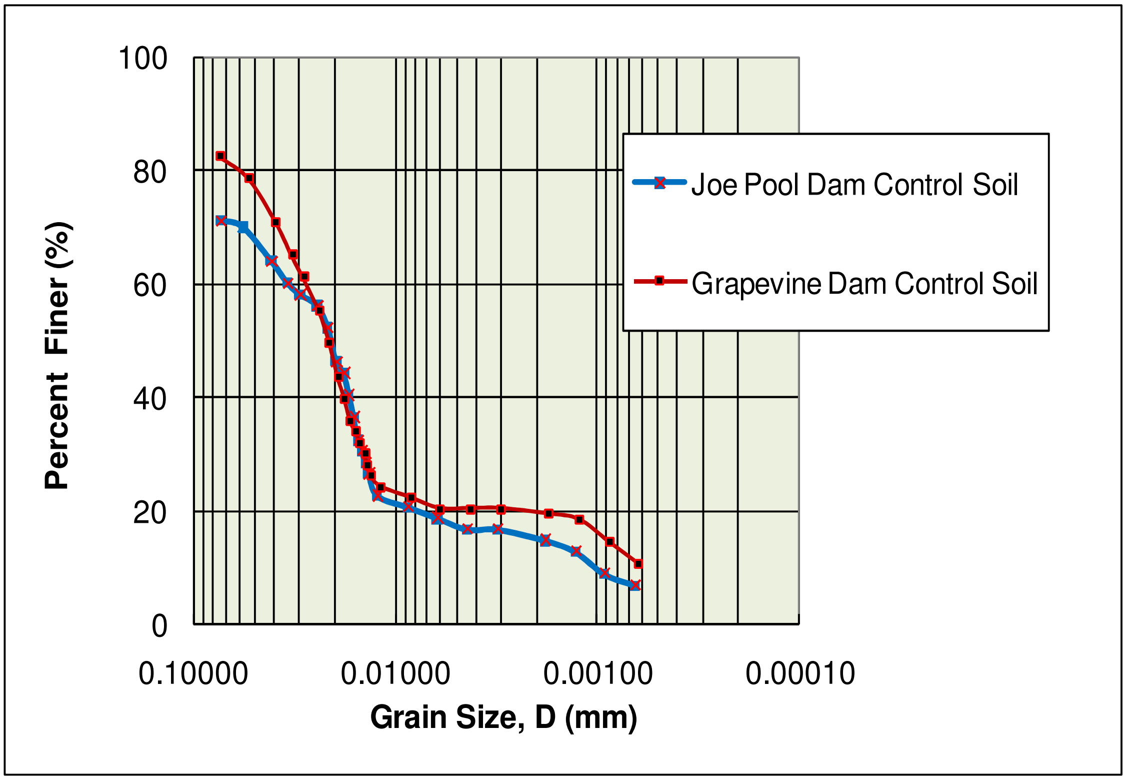


Figure 4.10 Hydrometer analysis – Grain size distribution of finer aggregate of Joe Pool Dam and Grapevine Dam

4.3.3 Handling of Excavated Soil

A suitable soil processing area was identified near the toe of the dam. The site was mowed, and the top 10 to 15 cm (4 to 6 in.) of soil was removed and the area leveled.

The excavated soil was loaded into the trucks as shown in Figure 4.11 and transported to the soil processing area hereafter called as level pad.

The excavated soil was placed in the level pad area in a layer of 15 cm to 20 cm (6 to 8 inches) thickness. The level pad area locations for both the dam sites are shown in Figure 4.11.

(a)

(b)
Figure 4.11 Soil processing area (a) Joe Pool Dam (Inset: Handling of soil)
(b) Grapevine Dam (Inset: Handling of soil)

The soil in the level pad area was pulverized, moistened and kept ready for mixing with admixtures. Figure 4.12 shows the pulverized soil at the level pad where the soil will be mixed with compost, lime and fibers.

Figure 4.12 Pulverized and moistened soil at level pad
ready for mixing with admixtures

Similar to the layout of treated sections, zoning of the soil in the level pad was done. Zone 1 was control soil and Zone 2 to 5 include soil to be mixed with 20% compost, 4% lime with 0.30% fibers, 8% lime with 0.15% fibers and 8% lime respectively.

The compost and lime was brought through trucks and unloaded in respective zone. The fibers were received in bags at the time of supply and the same were spread manually as explained further.

4.3.4 Addition of Compost for Zone 2

Compost was obtained from City of Denton, Texas where the nomenclature used is “Dyno dirt”. Dyno dirt is nutrient rich compost material. The compost products are made primarily from yard trimmings which are collected by the Solid Waste Department.

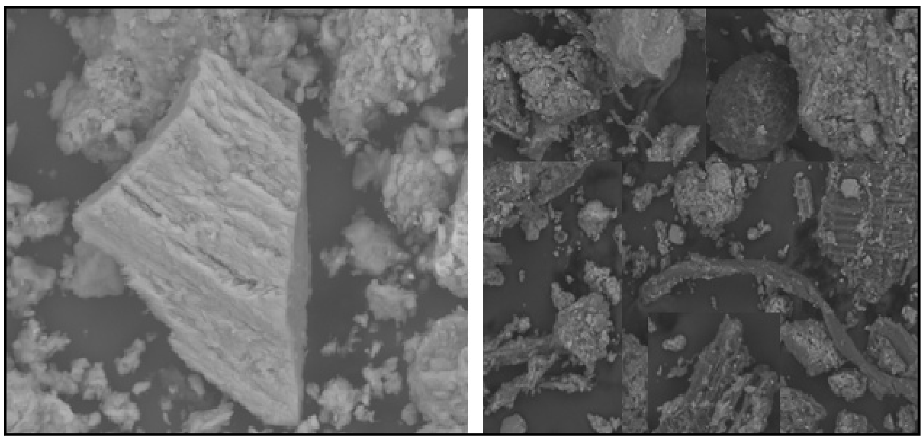
Figure 4.13 shows the unloading of compost at the site in Zone 2 of level pad area. The compost was spread uniformly with machinery and mixed with the soil.

Figure 4.13 Unloading of compost in Zone 2 of level pad

The compost sample was tested in the UTA laboratory. Certain vital properties of the compost are presented in Table 4.3. Figure 4.14 shows Scanning Electron Micrograph image of the control soil and compost sample to have a better comparison of the particle structure.

Table 4.3 Properties of compost

Description	Value
Organic Content	36.6 %
Ash Content	60.4 %
Specific Gravity	1.25
Moisture Content when Supplied	28.5 %
pH	7.6



(a) (b)

Figure 4.14 SEM image (a) Control Soil (b) Compost sample

4.3.5 Mixing of Lime

The specifications of hydrated lime used are presented in Table 4.4.

Table 4.4 Properties of hydrated lime

Component	Value
Calcium Oxide (CaO)	72.65%
Silicon Dioxide (SiO_2)	< 1
Magnesium Oxide (MgO)	<1
Trace Elements (Al, Fe Oxide)	< 1.24
Chemically Combined Water	23.76
Specific Gravity	2.2 to 2.4
Boiling Point (Decomposes to CaO)	1076˚ F
pH	12.454 at 25º C
Flash Point	Non Explosive and Non Flammable

4.3.5.1 Mixing of Lime Slurry at Level Pad

After pulverization and scarification of soil, 4% of lime was mixed in Zones 3, 4 and 5. Hydrated lime in slurry form was delivered in a self unloading transport truck conforming to AASHTO designation M 216 and the lime was spread on the entire soil in the three Zones accounting for 4% of lime by weight. The application of lime with the help of pressure distributors is shown in Figure 4.15.

The mixing commenced as soon as spreading pass of lime was completed so as to prevent slurry run off. Lime and soil were mixed thoroughly. During preliminary mixing of soil with lime additive, adequate water was also added so as to initiate chemical reaction.

Mellowing period of one day was allowed and the lime added soil was remixed. Soil in Zone 4 and 5 required double application of the lime additive at the rate of 4% each time.

After mixing of soil with initially applied 4% of lime, the soil was scarified in zone 4 and 5 to apply remaining 4% of lime. With the help of self unloading lime slurry trucks, the balance 4% quantity of lime was also applied.

The lime was immediately mixed with soil duly adding sufficient quantity of water. Due to mellowing, the clay became friable and pulverization could be readily attained during final mixing.

Figure 4.16 shows the process of mixing of lime with mechanical rotary mixing equipment that scarified, mixed and graded the soil at level pad with each pass of the rotary mixer.

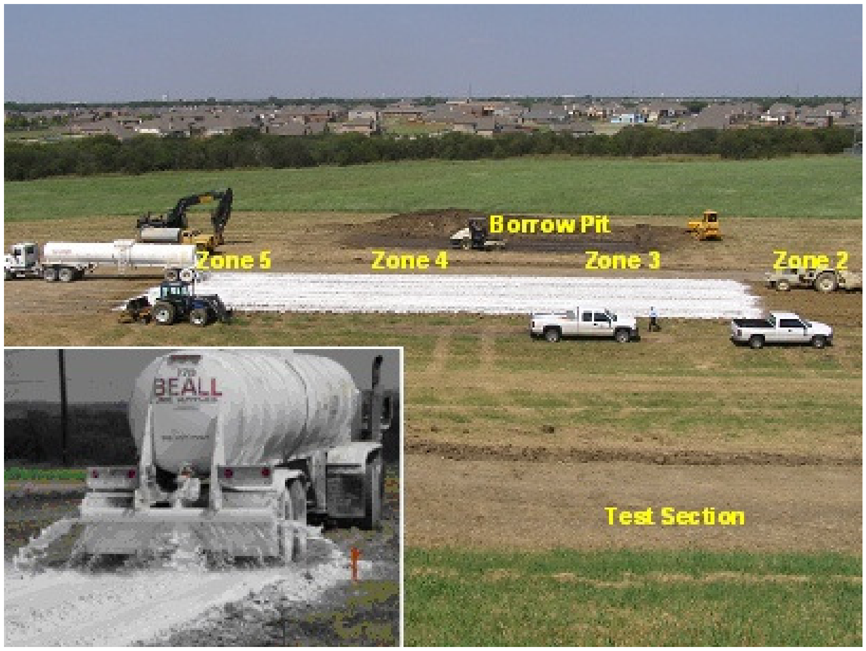


Figure 4.15 Spreading of lime slurry on the pulverized soil

Figure 4.16 Mixing of lime with soil on level pad

Final mixing was completed in Zone 5 duly adding adequate water. Several passes of rotary pulverizer was continued till 100% sample passed through 25mm (1 in.) and 60% sample passed through # 4 sieves. In Zone 5, the soil mixed with 8% lime was thoroughly mixed so as to transport it and place on the slope of the dam over the compacted surface.

4.3.6 Mixing of Polypropylene Fibers

The polypropylene fibers, having brand name of Geofibers, were used for mixing with the soil. The properties of fibers are presented in Table 4.5. Figure 4.17 shows the mobilization of polypropylene fibers at site along with a scanning electron micrograph image in the inset.

Table 4.5 Properties of polypropylene fibers

Description of Property	Value
Tensile Strength	669 MPa (97 ksi)
Specific Gravity	0.91
Melting Point	330˚ F
Acid, Salt and Alkali Resistance	High
Water Absorption	111 %
Length of fiber	50 mm (2 in.)
Length / dia ratio	2173 (dia: 23microns)
Young's Modulus	4000 MPa (580 ksi)

For each dam site, 455 kg (1000 lb) of fibers were procured and about 303 kg (667 lb) of soil was spread on 4% lime and soil mixture in Zone 3 to account for 0.30% of fibers. In Zone 4, about 333 lb of fibers was spread on 8% lime and soil mixture to account for 0.15% of fibers. Figure 4.18 shows the fibers spread on Zones 3 and 4 at a rate of 0.30% and 0.15 % respectively.

Figure 4.17 Mobilization of polypropylene fiber (Inset: SEM image of fibers)

Figure 4.18 Polypropylene fibers spread on soil mixed with lime

After spreading the fibers on Zones 3 and 4 as per the proportions specified, the lime soil mixture was mixed with fibers duly adding adequate quantity of water. The mixing was carried out with a rotary mixer as shown in Figure 4.19.

Figure 4.19 Mixing of fibers with lime soil mix

4.3.7 Placement of Treated Soil at the Test Section Locations

The treated soil with admixtures was transported and placed back in the embankment immediately. The treated soil was compacted adding adequate water in two layers of 23 cm (9 in.) each. Adequate water was added to the compacted sections during and after compaction to have proper in-place curing of lime soil and lime fiber soil mixtures. The test section was constructed by proper compaction with a sheep foot roller. After compacting of the treated soil and control soils of 45 cm (18 in.) thickness each, a 23 cm (9 in.) top soil was placed and compacted. Various stages involved in the placement of soil on the embankment slope are shown in Figure 4.20 through 4.25.

Figure 4.20 Loading of treated soil in to truck

Figure 4.21 Unloading of treated soil on slope and leveling

Figure 4.22 Watering of test sections

Figure 4.23 Compaction with sheep foot roller

Figure 4.24 Compacted section with 8% lime

Figure 4.25 Placement of treated soil in the embankment (Inset: Planning of work)

4.3.8 Turfing

After the completion of construction of test sections, the embankment area, level pad area and other borrow pit areas were fertilized and seeded. Seeding of hulled

Bermuda grass (C.Dactylon) was applied at a rate of 4.9 gm of seed per square meter (1 pound of seed per 1000 square foot). The seeded areas were watered at 2 day interval and a total of 3 watering in 5 days.

4.3.9 Final Compaction

All the test sections, processing area and borrow areas were compacted with one pass over the entire surface after completion of seeding.

4.4 Quality Control and Quality Assurance Studies

Quality Control is product oriented and includes testing and inspections. Quality assurance is a process oriented and involves methodologies and standards prescribed. The standards required, detailed specifications and drawings of work were well documented and furnished to all the agencies involved with the work. A preconstruction meeting was held before commencement of work at each site. Number of brainstorm sessions was held at the construction site frequently to monitor the progress of work, quality and plan further activities in a coordinated manner. The details of quality control and quality assurance activities described as following:

4.4.1 Formation of Ettringite

Whenever lime is added to soil containing soluble sulfates, it interacts with sulfates and results in the formation of ettringite. Ettringite mineral causes heaving of soil. Precautions were taken to ensure that the borrow pit soil selected at Joe Pool Dam did not contain soluble sulfates. One borrow pit containing eagle ford formation soil having high content of soluble sulfate levels was abandoned. Laboratory tests were still conducted and the images obtained from X-ray diffraction (XRD) test and Scanning

Electron Micrograph test were studied to ensure there was no formation of ettringite. Figure 4.26 and 4.27 presents the XRD image and SEM image of the 8% lime treaded soil and it could be observed that there are no traces of any ettringite mineral.

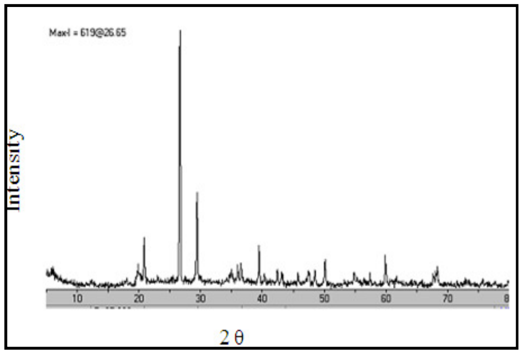


Figure 4.26 X-ray diffraction test image of lime treated soil

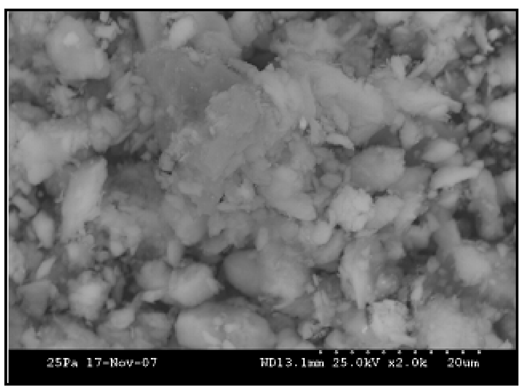


Figure 4.27 SEM image of 8% lime treated soil

4.4.2 Laboratory Tests on Soils

Quick laboratory tests were conducted on the soil samples obtained from the construction site to determine properties such as moisture content and Atterberg limits of control and treated soils. These results were compared with targeted or design properties for evaluating the quality control aspects of field construction.

4.4.3 Effectiveness of Mixing of Fibers

The fibers were spread manually on the lime mixed soil layer on the level pad area. In order to check the effectiveness of mixing of soil with admixtures, observation pits were excavated. Figure 4.28 shows the process of mixing of fibers and the inset picture shows the distribution of fibers over the entire depth of soil layer. Besides, soil samples from Zone 3 and Zone 4 were collected and sieved to verify the percentage of fibers present in the soil. These studies showed that the desired proportion of fibers was present in both the Zones.

Figure 4.28 Spreading of fibers on soil in Zone 3 and Zone 4
(Inset: Observation pit)

4.4.4 Quality Control of Field Compaction

Nuclear gage tests were conducted for each layer of the compacted soil to check whether the targeted density is achieved. The test set up with nuclear gage is shown in Figure 4.29.

Figure 4.29 Nuclear gage for compaction quality control

The nuclear gage also provided the information on moisture content of compacted soil. The tests also helped control the watering and compaction activities during the course of work. The moisture content and degree of compaction were controlled based on the results of the nuclear gage test. Tables 4.6 and 4.7 presents the dry unit weight and OMC targeted and achieved for the Joe Pool Dam and Grapevine Dam respectively. The nuclear gage test results have shown that the compaction achieved was more than the targeted value of 95% of Standard Proctor test results.

Table 4.6 Targeted and achieved dry unit weight and optimum moisture content for Joe Pool Dam test sections

Description	Dry Unit Weight		Moisture Content	
	Targeted kN/m^3 (pcf)	Achieved kN/m^3 (pcf)	Targeted (%)	Achieved (%)
Control	14 (89)	16 (100)	14	20%
20% Compost	13 (82)	14 (90)	28	29.2%
4% Lime with 0.30% Fibers	15 (93)	15 (95)	24	25.6
8% Lime with 0.15% Fiber	14 (90)	15 (95)	25	25.4
8% Lime	14 (92)	15 (97)	24	25

Table 4.7 Targeted and achieved dry unit weight and optimum moisture content for Grapevine Dam test sections

Description	Maximum Dry Unit Weight		Optimum Moisture Content	
	Targeted kN/m^3 (pcf)	Achieved kN/m^3 (pcf)	Targeted (%)	Achieved (%)
Control	17 (110)	17 (110)	14	17.5
20% Compost	13 (84)	16 (105)	22.5	20
4% Lime with 0.30% Fibers	15 (98)	16 (103)	18.6	19.7
8% Lime with 0.15% Fiber	15 (98)	16 (102)	18.9	20.1
8% Lime	15 (99)	16 (100)	21	21.5

4.4.5 Quality Assurance (QA) studies

Quality assurance studies were conducted by performing engineering swell, shrinkage and direct shear tests on the field treated specimens.

Table 4.8 to 4.10 presents a summary of comparisons between the results of preconstruction laboratory tests conducted by McCleskey (2005) and present field mixed samples obtained during construction of test sections.

Table 4.8 Comparison swell strain results

Description	Joe Pool Dam		Grapevine Dam	
	Pre construction sample (%)	Field Samples (%)	Pre construction sample (%)	Field Samples (%)
Control	22.1	19	10.0	14.0
20% Compost	11.8	18	18.8	7.8
4% Lime with 0.30% Fibers	2.6	6	5.4	1.7
8% Lime with 0.15% Fiber	5.5	2	5.7	0.2
8% Lime	3.8	0.1	5.8	0.3

Table 4.9 Comparison of volumetric shrinkage strain results

Description	Joe Pool Dam		Grapevine Dam	
	Pre construction sample (%)	Field Samples (%)	Pre construction sample (%)	Field Samples (%)
Control	6.8	9	7.2	12
20% Compost	4.8	10	6.0	14%
4% Lime with 0.30% Fibers	3.4	2	3.1	4%
8% Lime with 0.15% Fiber	2.2	3	2.5	3%
8% Lime	2.6	3	2.1	3%

Table 4.10 Comparison of direct shear test results

Description	Joe Pool Dam		Grapevine Dam	
	Pre construction sample (%)	Field Samples (%)	Pre construction sample (%)	Field Samples (%)
Cohesion kPa (ksf)				
Control	76 (1.58)	80 (1.67)	38 (0.80)	53 (1.10)
20% Compost	93 (1.94)	86 (1.80)	90 (1.89)	81 (1.70)
4% Lime with 0.30% Fibers	56 (1.17)	57 (1.20)	50 (1.04)	62 (1.30)
8% Lime with 0.15% Fiber	55 (1.16)	62 (1.30)	47 (0.98)	72 (1.50)
8% Lime	141 (2.94)	105 (2.20)	94 (1.97)	100 (2.10)
Friction Angle (degrees)				
Control	37	36	17	18
20% Compost	42	40	39	41
4% Lime with 0.30% Fibers	28	38	38	42
8% Lime with 0.15% Fiber	34	42	42	44
8% Lime	24	43	33	44

Comparison of test results between preconstruction laboratory test results on borrow soil samples and post construction field test samples show a good, if not complete agreement indicating that the mixing was fairly successful in the field during construction.

4.5 Instrumentation

The test sections are to be monitored continuously in order to determine the most effective treatment which prevents formation of desiccation cracks and exhibits least swelling and shrinkage.

Lateral displacement of the slope portion of treated test sections were to be monitored with respect to time. As such, vertical inclinometers were provided in the test sections at a rate of two casings in each section totaling ten inclinometers for each dam site.

Elevation pegs are also installed in the dam site to monitor the vertical movements of surface caused by swelling or shrinkage. 125 pegs at Joe Pool Dam and 85 pegs at Grapevine Dam were installed to monitor the elevation fluctuations of the treated sections.

The treated sections were also monitored visually and with digital imaging to analyze the shrinkage crack development and its propagation. The relevant details pertaining to field instrumentation is summarized below.

4.5.1 GroPoint Moisture Probes

In order to monitor the soil moisture content and soil temperature, it was planned to install moisture probes along with a data logger to continuously record the moisture content and temperature as explained below. Figure 4.30 shows the schematic indicating the working principle of moisture probes.

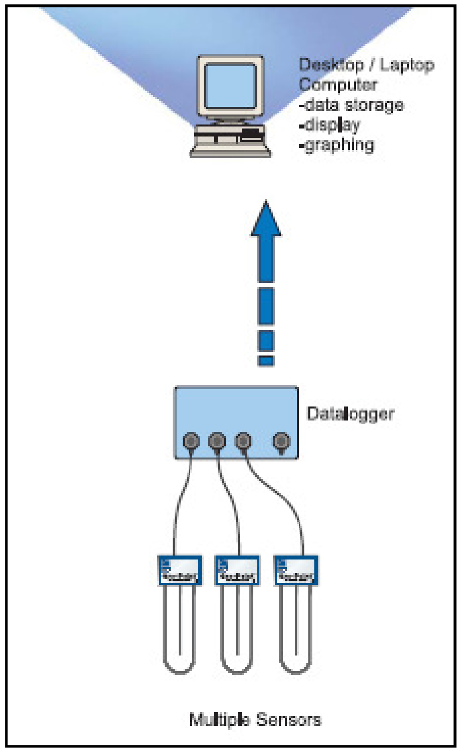


Figure 4.30 Schematic showing the working of moisture probes (Courtesy: ESI)

The moisture sensor works on the principle of 'Time Domain Transmission' or (TDT) technology and provides volumetric moisture contents. It measures the one-way propagation time. The pulse reading is observed at the other end of the transmission line from the transmitter of the sensor. The propagation time of an electromagnetic wave along a given length of transmission line is proportional to the square root of the permittivity of the medium the transmission line is immersed in. For the medium of soil/water/air, in this project the permittivity of the water dominates the mixture of permittivity and the measurement can then be used to determine the volumetric water content of the soil mixture. Volumetric moisture contents are related to gravimetric moisture contents by the density of the soil medium. The relationship is shown in the following equation.

$$\theta_G = \theta_v * \frac{\rho_w}{\rho_s} \tag{4.1}$$

Where, θ_G = Gravimetric soil moisture content;

θ_S = Volumetric soil moisture content;

ρ_w = Density of water and

ρ_s = Bulk density of soil.

Two moisture probes were placed in each treated test section. Figure 4.31 shows the installation of moisture probes.

The bottom probe was placed at a depth of 50 cm (20 inches) from the top of surface, i.e., at the middle of 45 cm thick treated section. The second moisture probe was placed at the top of treated section near the interface with top soil at a depth of 25 cm (10 inches) from the top surface of slope.

The temperature probe was placed in the treated section nearer to the top moisture probe in the treated portion of the soil. Both moisture and temperature probes provide real time volumetric moisture content and temperature data.

The data was stored in a data logger stationed at each test site, and the data was downloaded to a laptop computer periodically during site visits by connecting to the data cable and using Grograph software program. The moisture content and temperature data recorded every hour was transferred to an excel file for further analysis.

(a) (b)

Figure 4.31 Moisture sensors (a) Installation at site (b) Data logger with battery and temperature probe

4.5.2 Installation of Inclinometer Casings

The length of each inclinometer casing is 4.5 m (15 feet) and the diameter of the casing is 70 mm (2.75 inches). Two pipes of 3 m (10 feet) and 1.5 m (5 feet) were joined together by providing a ‘O’ ring between the pipes and locking them together with its self locking arrangements. A bottom cap was provided to prevent ingress of water in to the casing. The assembled casings are shown in Figure 4.32.

Figure 4.32 Assembled inclinometer casings of 4.5 m (15 ft) long

All ten locations of the inclinometers were marked on the field as per the drawings so as to commence the drilling operations at the marked locations. The drill rigs used for drilling bore holes for inclinometer installation are shown in Figure 4.33.

(a)

(b)

Figure 4.33 Drilling of inclinometer casing hole (a) Joe Pool Dam (b) Grapevine Dam

After drilling the holes, the preassembled casings were inserted in the holes and one pair of grooves was aligned in the direction of movement along the slope.

A grout was prepared with one part of portland cement, 0.3 parts of bentonite and 2.5 parts of water by weight. The grout was poured around the annular space between hole and the casing as shown in Figure 4.34. The grout was cured for an adequate period.

To avoid casing buoyancy, the casing was filled with water before carrying out grouting work. The grouted casings were cured for a period of one week. Later, the water in the casings was bailed out with the help of a bailer.

Figure 4.35 shows the completed portion of test sections at Joe Pool Dam and Grapevine Dam.

Figure 4.34 Grouting of inclinometer casing bore hole

(a)

(b)

Figure 4.35 Completed test Section (a) Joe Pool Dam and (b) Grapevine Dam

Vertical inclinometer probe was used to take the readings of all the inclinometer casings periodically and the data was analyzed with the help DMMWin and Digipro software as prescribed by the manufacturer of inclinometer probe.

4.6 Summary

Test sections were constructed at Joe Pool Dam and Grapevine Dam slopes. Length of each section was 18 m x 7.5 m (60 ft x 25 ft). At each dam site, the construction of the control section, the 20% compost section, the 4% lime mixed with 0.30% fibers section, the 8 % lime mixed with 0.15% fibers section and the 8% lime section was completed. Both quality control and quality assurance aspects of field construction of test sections were addressed by comparing present laboratory results on the field-treated samples with the previous laboratory investigation studies performed by McCleskey (2005). These comparisons show a reasonably good agreement, indicating that the field mixing was successful.

Test sections were instrumented with moisture probes, temperature probes and vertical inclinometers. Elevation survey and digital imaging of surface of test sections was also carried out apart from monitoring data from instrumentation. The results obtained are analyzed and presented in Chapter 5.

CHAPTER 5

ANALYSIS OF FIELD DATA OF JOE POOL DAM SITE

5.1 Introduction

Five tests sections were constructed at Joe Pool Dam and these sections were instrumented with moisture probes, temperature probes and inclinometers to study the moisture absorption, moisture retention and deformations of test sections. The field data has been monitored for a period of one year and the analysis of the monitored data is presented in this Chapter. Moisture sensors were installed at two different levels within the 45 cm (18 in.) thick treated section at a depth of 25 cm (10 in.) and 50 cm (20 in.) from the top surface. The temperature probe was installed at a depth of 25 cm (10 in.) from the surface. The schematic showing the location of moisture probes and temperature probe is shown in Figure 5.1.

In each test section, the moisture probes and temperature probe were connected to a data logger buried at the site leaving a data cable projected above the ground surface as shown in Figure 5.1. The data logger was connected to a battery which is also buried in the soil. For all five sections of each dam sites, a total number of 10 moisture probes, 5 temperature probes and 5 data loggers were installed. The data of TDT type moisture probe and the temperature was continuously recorded at every one hour interval. The data collected was downloaded to a laptop computer once a month during site visits.

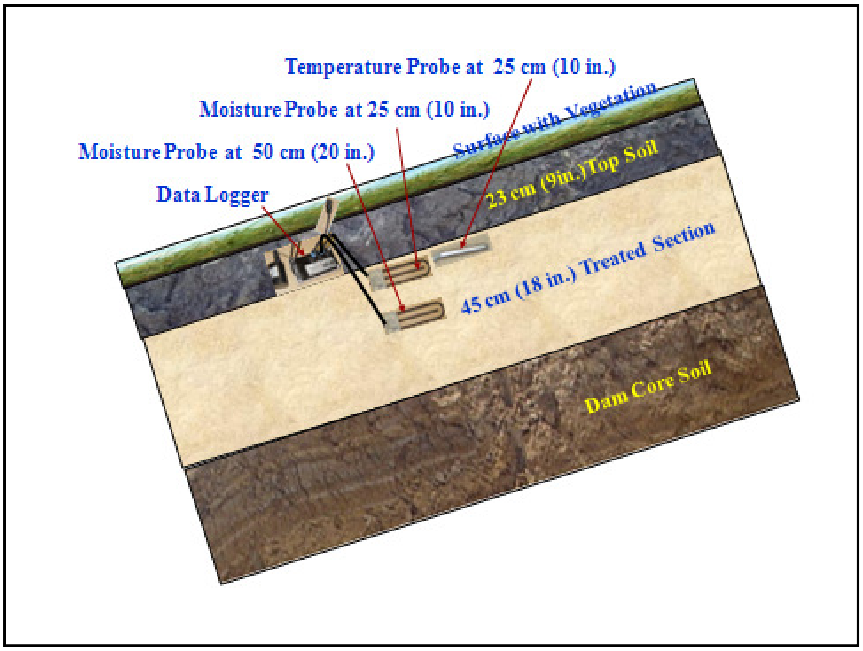


Figure 5.1 Locations of moisture probes and temperature probe

In addition, two vertical inclinometers were installed at each test section, totaling ten for each dam site to monitor the lateral displacements. The lateral displacement of the test section was being monitored closely by taking readings periodically with the help of a vertical inclinometer probe. The frequency of the observations was about once a month and more visits were made immediately after extreme weather conditions. It is hypothesized that soils swell during wet season and shrink during dry season. Hence, elevation surveys were conducted to monitor both swelling and shrinkage behaviors of the treated site sections. Elevation surveys were also performed following extreme weather events. The site was inspected at a frequency of once in every two weeks to physically observe the conditions. Emphasis was given to observe the features such as growth of vegetation and formation of cracks on the slope surface during these site inspections.

5.1.1 Rainfall Data

While studying the moisture content fluctuations and the temperature variations of the test sections, site precipitation details over the same period were also collected. During the period of monitoring between October 2007 and September 2008, there was a total rainfall of 89 cm (35 in.) against the average annual rainfall of 94 cm (37 in.) for Dallas, Texas.

As such, it is considered that the site has received considerable amount of precipitation so as to compare the performance of the test sections. The distribution of monthly rainfall during the period of monitoring for Joe Pool Dam is shown in Figure 5.2 along with the average annual rainfall of Dallas.

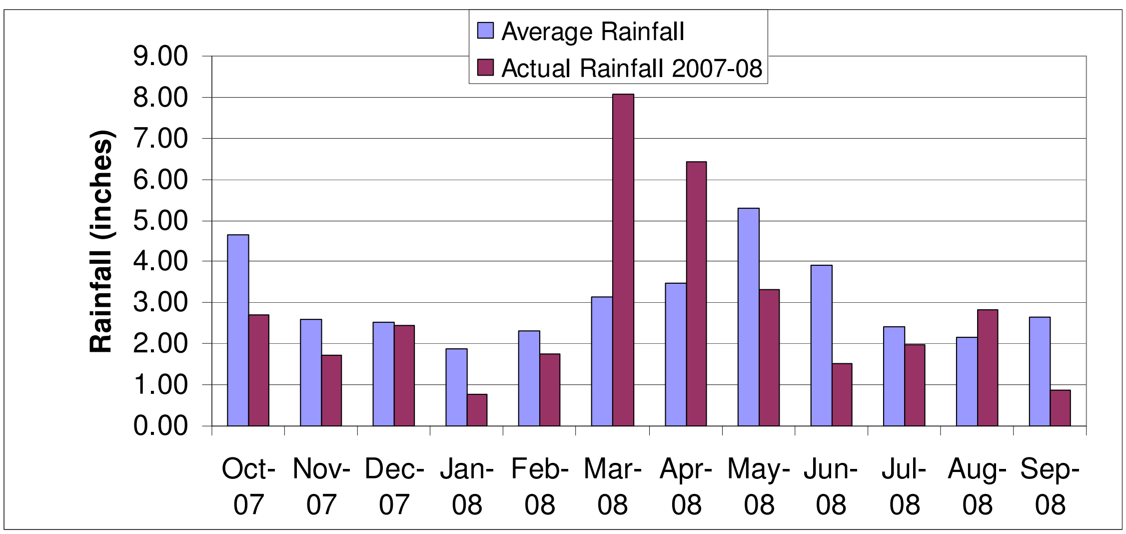


Figure 5.2 Monthly rainfall data (Source: USACE) (1 in.= 2.54 cm)

5.2 Moisture Sensors Data of Joe Pool Dam

Volumetric moisture content data as recorded by the moisture sensors at Joe Pool Dam test sections was collected from the data loggers with the help of the

Grograph software. A typical graph showing the volumetric moisture content and temperature record on ‘y-axis’ with time period on ‘x- axis’ is presented in Figure 5.3 for the control section of Joe Pool Dam.

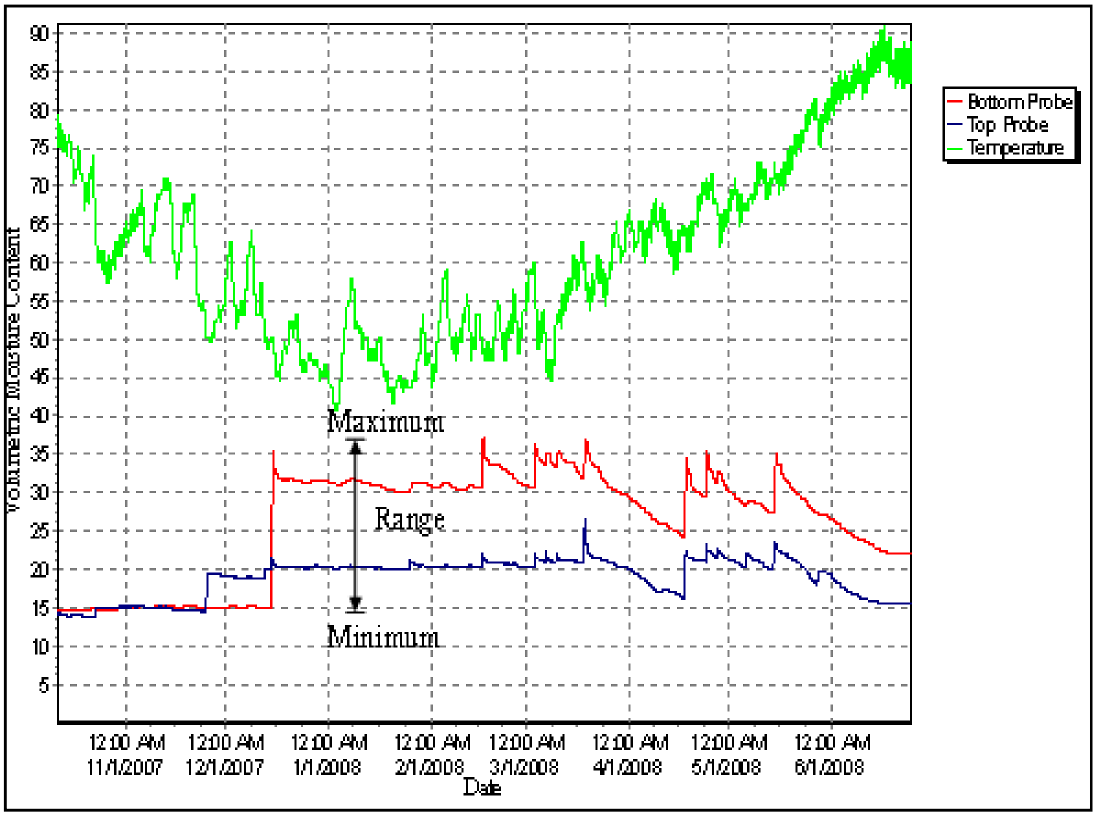


Figure 5.3 Schematic showing the moisture probe data for the control section of Joe Pool Dam

The data collected was analyzed to study the response of the sensors to different weather conditions at the test site. Emphasis was given to observe the maximum and minimum volumetric moisture content and the average moisture content of each test section.

The data was compiled month wise and depicted in the form of tables and figures for quick interpretation and analysis.

5.2.1 Initial Response of the Control Section to Rainfall

The daily rainfall data for the period of monitoring between October 2007 and September 2008 is shown in Figure 5.4.

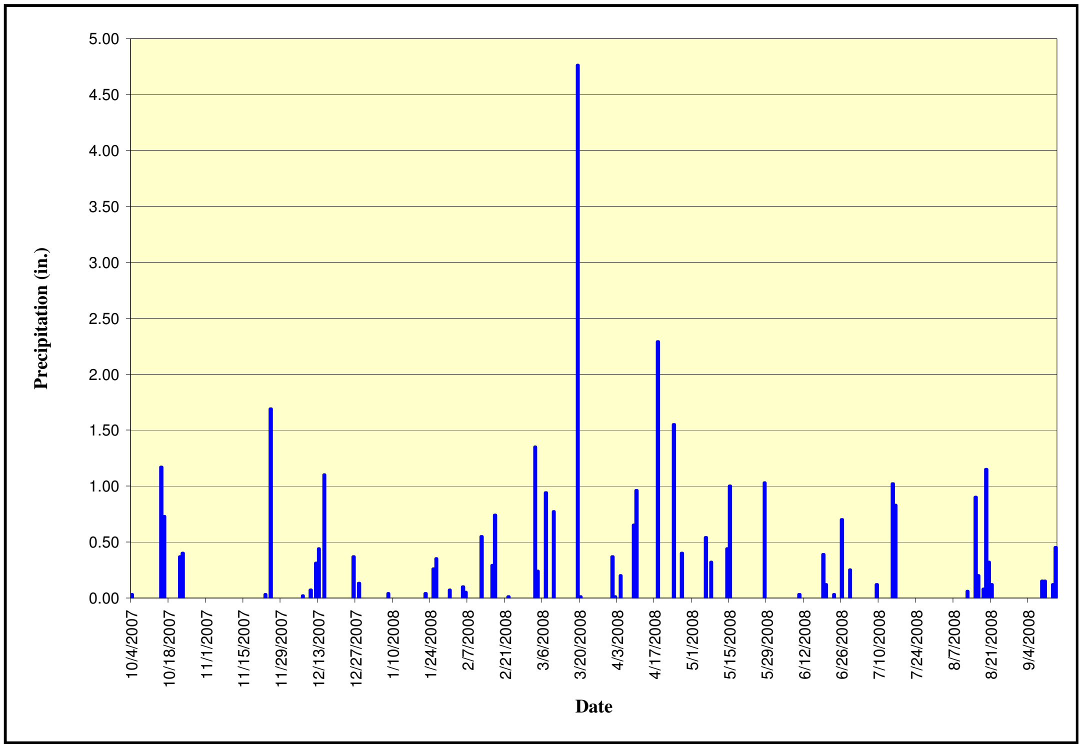


Figure 5.4 Daily rainfall data from October 2007 to September 2008 (1 in.= 2.5 cm)

It can be seen from Figure 5.3 that after the completion of construction of test sections, the soil volumetric moisture content of the control section for the top probe was 14% and for bottom probe was 15%. The moisture absorption and retention were prominent in the top moisture probe measurements for various rainfall events during the first three months. The bottom moisture probe has shown an increase of volumetric moisture content up to 34% after a period of about two and half months and continued to retain higher moisture content than the top probe.

This phenomenon indicates that the rainfall during first two months has caused wetting of top layer of soil only. This is apparent because the coefficient of permeability of clay is low as shown in Table 5.1 and it takes more time for the water to percolate to bottom layers. Besides, during the construction, the test section is in unsaturated condition. The coefficient of permeability of unsaturated soil is lesser than the coefficient of permeability of saturated soil (Lu and Likos, 2004). Continuous rainfall event during the month of December for a period of 5 days has resulted in an increase of wetting front beyond the 50 cm (20 in.) depth from the surface.

5.2.2 Initial Response of the Compost Section to Rainfall

Compost is hydrophilic and previous research studies show that it can hold higher moisture content under normal weather conditions. The volumetric moisture content of the compost test section is shown in Figure 5.5.

The initial moisture content of top probe is about 25% which is higher than the moisture content value of 15% of top probe of control section during the same period. However, the bottom probe moisture content is almost same as that of control section.

The top probe was measuring an increase in volumetric moisture content when following various rainfall events during the first three months.

However, the bottom probe measured higher moisture content during the rainfall event in the second month.

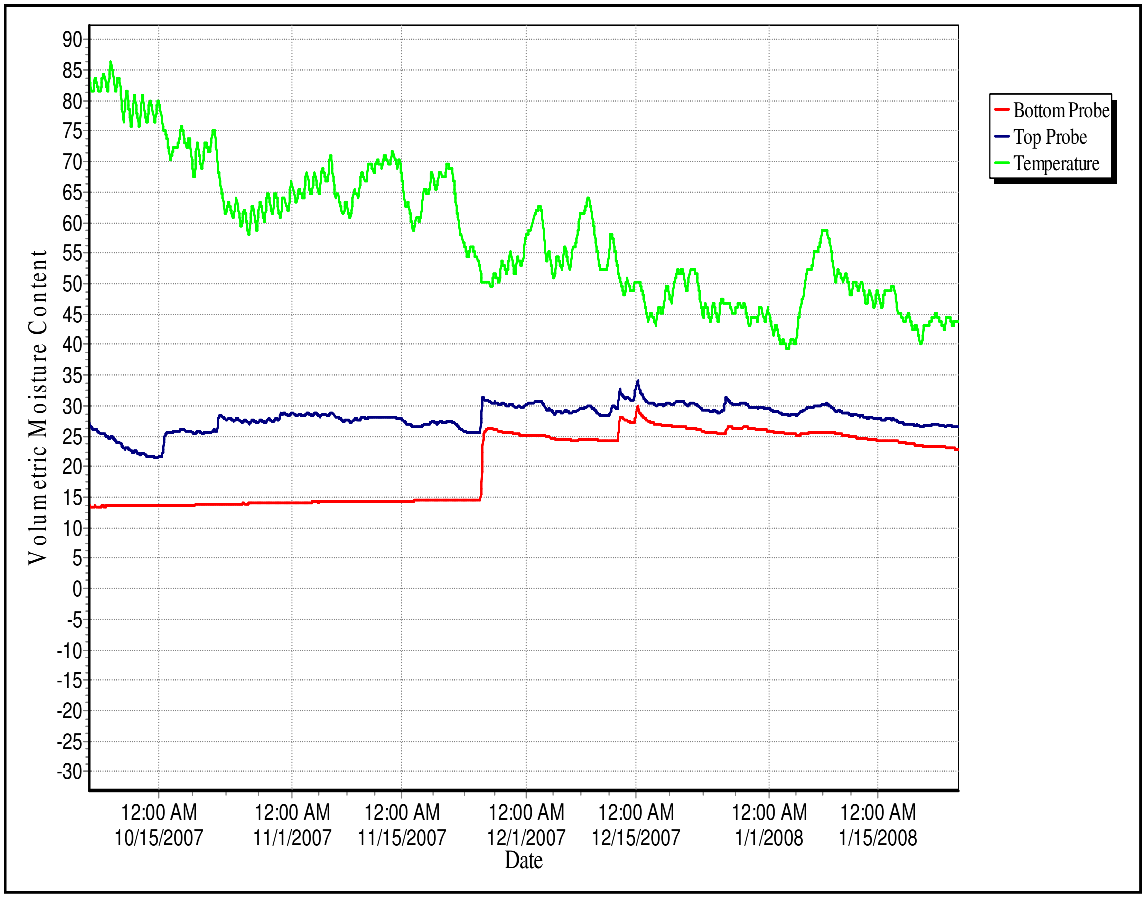


Figure 5.5 Output of moisture sensors for the 20% compost treated section

This indicates a faster increase of wetting front beyond 50 cm (20 in.) depth from the surface when compared with the time taken for control section.

This is attributed to the fact that the coefficient of permeability of compost treated section is higher than that of the control soil section as shown in Table 5.1.

The phenomenon of absorbing and retaining relatively higher moisture content in the top layer of soil is expected to help prevent desiccation cracking.

Prevention of desiccation cracks in turn retards the infiltration, thereby resulting in saturation of the soil mass.

5.2.3 Initial Response of the 4% Lime with 0.30% Fibers Treated Section to Rainfall

The initial volumetric moisture content levels of top and bottom probes of the test section with 4% lime with 0.30% fibers are higher than the moisture content levels measured in the control section as can be seen from Figure 5.3 and Figure 5.6.

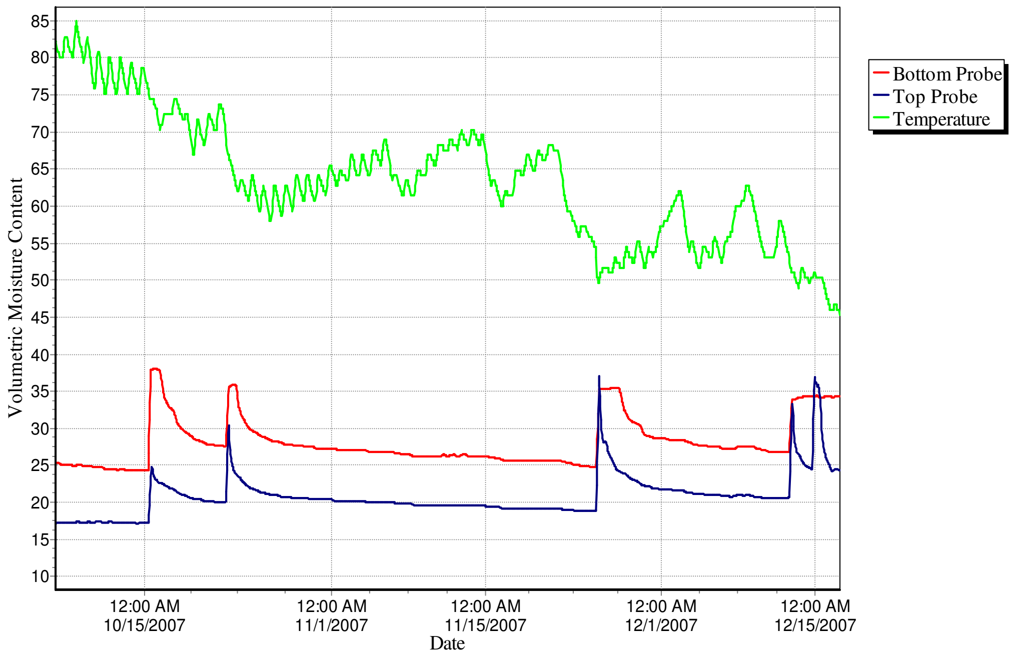


Figure 5.6 Output of moisture sensors for the 4% lime with 0.30% fibers treated section

The higher volumetric moisture content is attributed to the fact that polypropylene fibers are hydrophilic in nature and hold higher moisture content. Almost all the significant rainfall events are followed by an increase in moisture content of both top and bottom probes during the first three months period as can be seen in Figure 5.6.

5.2.4 Initial Response of the 8% Lime with 0.15% Fibers Treated Section to Rainfall

The initial moisture contents monitored from top and bottom moisture probes are higher than that of control section as seen from Figure 5.3 and Figure 5.7.

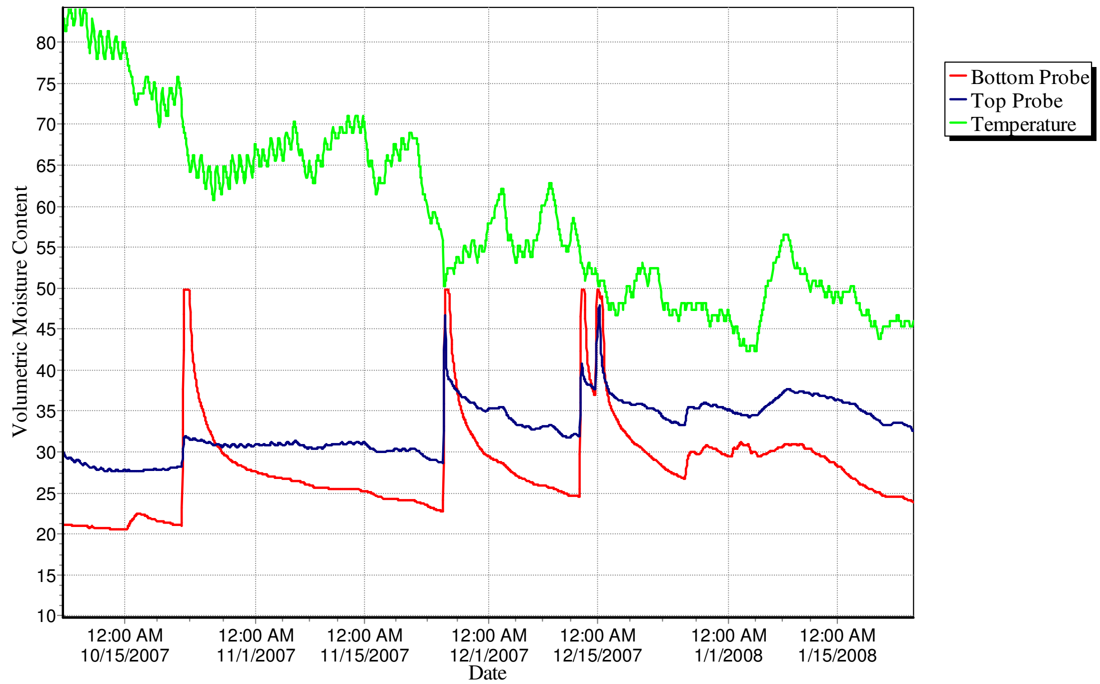


Figure 5.7 Output of moisture sensors for the 8% lime with 0.15% fibers treated section

Each rainfall event was followed by a rise in volumetric moisture content levels of both the probes and the bottom probe was found to have reached the saturation stage showing a volumetric moisture content level of about 50%.

Apparently, higher moisture content level is attributed to the presence of fibers in the treated section, which makes the soil more open in structure that absorbs the moisture from rainfall events.

The saturation of bottom probe is possibly due to ponding near the interface of treated section and the dam core surface. The coefficient of permeability of the 8% lime with 0.15% fiber treated section is higher than the control section as shown in Table 5.1. The bottom probe readings reached equilibrium levels from the condition of saturation gradually indicating gradual percolation of water into the underlying embankment layers and occurrence of seepage parallel to slope.

5.2.5 Initial Response of the 8% Lime Treated Section to Rainfall

The volumetric moisture content for the 8% lime treated section is shown in Figure 5.8. The initial moisture content of the 8% lime treated section is found to be higher than the control section.

The top probe was showing small increase in moisture content for various rainfall events.

For the continuous rainfall events during November and December 2007, the bottom probe in the 8% lime treated section measured higher moisture content levels indicating percolation beyond 25 cm (10 in.) depth.

However, the moisture absorption levels in the 8% Lime with 0.15% fiber treated section is higher than the 8% lime treated section apparently due to the presence of polypropylene fibers.

It is noticed from the initial observations of moisture sensors that the treated test sections absorbed more moisture content than the control section.

The presence of polypropylene fibers increased the moisture absorption and retention in the 8% lime with 0.15% fiber treated section when compared with the 8% lime treated section.

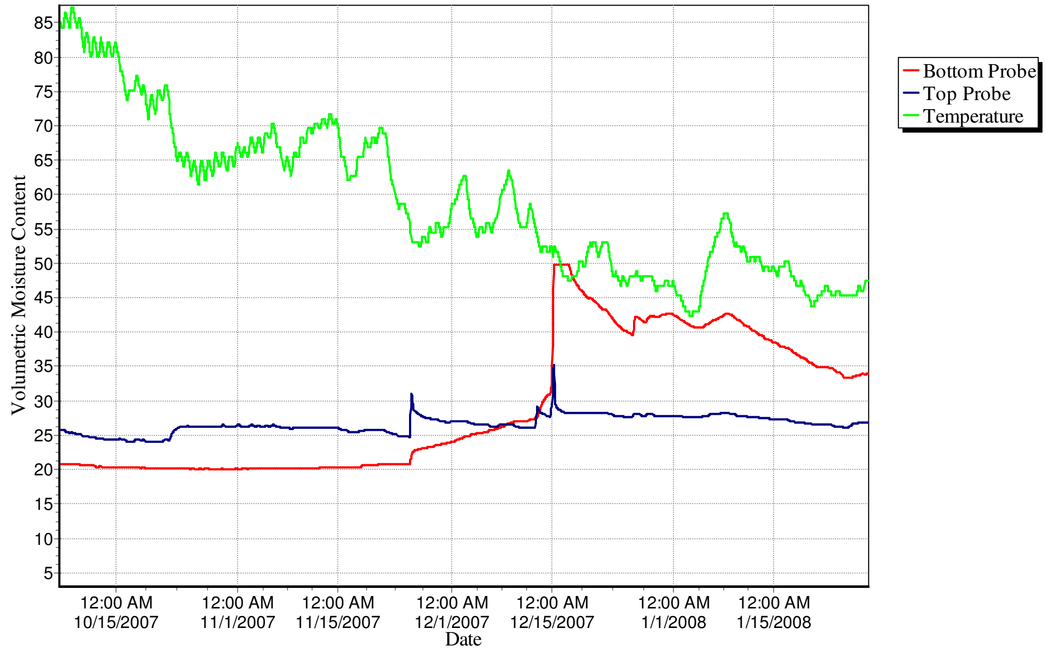


Figure 5.8 Output of moisture sensors for 8% lime treated section

5.2.6 Response of Treated Sections to a High Rainfall Event

On March 19th, 2008, a rainfall event with a maximum amount of total rain of 12 cm (4.76 in.) was recorded in Dallas - Fort Worth Metroplex. On that day, there was a new surficial failure occurred within a few hundred yards from the test section location at Joe Pool Dam. About four surficial failures were also noticed on highway embankment slopes and cut slopes near the DFW airport in Irving, Texas. These failures led to further examine the response of the treated sections to this particular rainfall event.

The variation of volumetric moisture content reading in the top and bottom probes at different time periods during this event is presented in Table 5.1.The coefficient of permeability is reproduced in metric units from Table 3.4. Prior to the storm, the average moisture content levels were higher in treated sections than in the control section. With the advent of the storm, the moisture levels increased in all the test sections. The soil reached highest levels of 50% in the case of the 8% lime with 0.15% fiber treated sections.

Table 5.1 Volumetric moisture content variation (%) to the 12 cm rainfall event

Description	Control	Compost	4% Lime with 0.30% Fibers	8% Lime with 0.15% Fibers	8% Lime
Prior to Storm Event	24	28	27	32	30
During storm Event	32	47	37	50	46
Increase of Moisture Content	8	19	10	18	16
Coefficient of Permeability (least to highest)	2.7×10^{-6} cm/sec	8.6×10^{-5} cm/sec	7.7×10^{-6} cm/sec	2.7×10^{-5} cm/sec	1.1×10^{-5} cm/sec
	1	5	2	4	3

The rise in moisture levels is highest in the compost section followed by the 8% lime with 0.30% fibers section, the 8% lime, the 4% lime with 0.30% fibers and the control section. It is observed that the rise is proportional to the hydraulic conductivity of the treated soil mass.

It can be seen from the above table that control section, 8% lime, 4% lime with 0.30% fibers, 8% lime with 0.15% fibers, and 20% compost sections are having the hydraulic conductivity in the increasing order from least to highest. From the above analysis, it can be concluded that for less intensity rainfall, the infiltration is limited to top few centimeters. For heavy and prolonged rainfall, the depth of infiltration is higher and the bottom layers of the embankment section are more prone to becoming saturated.

5.2.7 Response of Test Sections to Dry Season

Summer period of 2008 showed that the month of June was found to be the driest month. Prolonged dry spell resulted in huge desiccation cracks formed on the embankment surface. Volumetric moisture contents in top and bottom probes during the fourth week of June 2008 are shown in Table 5.2.

Table 5.2 Moisture content during desiccation cracking

Treatment	Volumetric Moisture Content (%)	
	Bottom Probe	Top Probe
Control	22	16
20% Compost	13	17
4% Lime with 0.30% Fibers	19	15
8% Lime with 0.15% Fibers	26	21
8% Lime	18	19

It could be observed that the low moisture content level recorded by the top probe is an indicator of conditions leading to desiccation cracking in both control and compost sections. Though the moisture content monitored in 4% lime with 0.30% fibers treated section was 15%, there was no visible desiccation cracks observed in the test section at Joe Pool Dam. This is attributed to the fact that the section has a higher tensile strength due to the presence of polypropylene fibers, which enhanced overall tensile strength of the treated soil.

5.2.8 Analysis of the Moisture Content Data Monitored for One Complete Year

The summary of monthly moisture content data showing rainfall amounts per month, minimum moisture content, maximum moisture content, range, and average moisture content during the month are shown in Tables 5.3 to 5.7.

Table 5.3 Moisture content data of the control section

Month	Data Points	Rainfall (in.)	Location of Probe	Minimum	Maximum	Range	Average
Oct-07	492	4.65	Bottom	14.51	15.1	0.59	14.9
			Top	13.92	15.29	1.37	14.55
Nov-07	719	2.61	Bottom	15.1	15.29	0.19	15.19
			Top	14.51	19.61	5.1	15.86
Dec-07	744	2.53	Bottom	15.1	37.06	21.96	24.27
			Top	18.82	22.16	3.34	19.94
Jan-08	745	1.89	Bottom	30	31.96	1.96	31.02
			Top	20	21.37	1.37	20.4
Feb-08	696	2.31	Bottom	30.59	37.84	7.25	31.79
			Top	20.2	22.35	2.15	20.68
Mar-08	744	3.13	Bottom	29.41	37.45	8.04	32.82
			Top	20.2	38.43	18.23	21.23
Apr-08	720	3.46	Bottom	24.31	37.06	12.75	28.78
			Top	16.27	23.92	7.65	19.59
May-08	744	5.3	Bottom	26.47	38.24	11.77	29.11
			Top	17.84	24.71	6.87	20.71
Jun-08	718	3.92	Bottom	21.96	26.47	4.51	23.22
			Top	15.49	19.22	3.73	16.23
Jul-08	744	2.43	Bottom	21.37	21.96	0.59	21.67
			Top	15.49	20.39	4.9	17.75
Aug-08	722	2.17	Bottom	21.76	25.29	3.53	23.19
			Top	17.25	23.53	6.28	19.3
Sep-08	720	2.65	Bottom	23.53	25.1	1.57	24.4
			Top	19.22	20.59	1.37	19.74
Total Average			Bottom	22.84	29.07	6.23	25.03
			Top	17.43	22.63	5.20	18.83

Table 5.4 Moisture content data of the 20% compost section

Month	Data Points	Rainfall (in.)	Location of Probe	Minimum	Maximum	Range	Average
Oct-07	492	4.65	Bottom	13.53	14.12	0.59	13.87
			Top	21.37	29.41	8.04	25.94
Nov-07	719	2.61	Bottom	14.12	26.27	12.15	16.54
			Top	25.49	31.76	6.27	28.07
Dec-07	744	2.53	Bottom	24.12	31.18	7.06	25.82
			Top	28.24	35.69	7.45	29.98
Jan-08	745	1.89	Bottom	22.94	25.88	2.94	24.24
			Top	26.47	30.39	3.92	28.16
Feb-08	696	2.31	Bottom	19.8	49.8	30	23.66
			Top	25.1	32.75	7.65	27.93
Mar-08	744	3.13	Bottom	15.88	49.8	33.92	26.18
			Top	20.59	49.8	29.21	28.9
Apr-08	720	3.46	Bottom	13.33	36.47	23.14	20.54
			Top	17.45	30.78	13.33	22.59
May-08	744	5.3	Bottom	15.1	39.02	23.92	20.55
			Top	17.84	27.84	10	21.85
Jun-08	718	3.92	Bottom	12.94	15.1	2.16	13.51
			Top	16.27	18.04	1.77	16.8
Jul-08	744	2.43	Bottom	12.94	13.73	0.79	13.14
			Top	16.47	17.84	1.37	16.72
Aug-08	542	2.17	Bottom	12.94	13.73	0.79	13.22
			Top	16.47	24.31	7.84	19.38
Sep-08	720	2.65	Bottom	13.73	14.31	0.58	14.15
			Top	18.82	22.55	3.73	20.11
Total Average			Bottom	15.95	27.45	11.50	18.79
			Top	20.88	29.26	8.38	23.87

Table 5.5 Moisture content data of the 4% lime with 0.30% fibers section

Month	Data Points	Rainfall (in.)	Location of Probe	Minimum	Maximum	Range	Average
Oct-07	492	4.65	Bottom	24.31	38.04	13.73	28.84
			Top	17.06	37.25	20.19	20.61
Nov-07	719	2.61	Bottom	24.71	35.49	10.78	27.29
			Top	18.82	38.43	19.61	20.49
Dec-07	744	2.53	Bottom	26.86	34.51	7.65	29.98
			Top	20.59	38.04	17.45	23.04
Jan-08	745	1.89	Bottom	26.47	33.53	7.06	28.33
			Top	20.2	24.12	3.92	21.74
Feb-08	696	2.31	Bottom	27.25	33.92	6.67	28.96
			Top	20.98	38.04	17.06	22.83
Mar-08	744	3.13	Bottom	27.06	35.69	8.63	30.86
			Top	18.82	43.33	24.51	26.96
Apr-08	720	3.46	Bottom	21.37	36.86	15.49	27.71
			Top	15.88	39.02	23.14	21.06
May-08	744	5.3	Bottom	20.39	38.04	17.65	23.83
			Top	15.49	38.63	23.14	18.87
Jun-08	718	3.92	Bottom	19.02	20.59	1.57	19.48
			Top	14.31	16.08	1.77	14.86
Jul-08	744	2.43	Bottom	19.41	23.14	3.73	21.15
			Top	14.51	22.94	8.43	17.67
Aug-08	722	2.17	Bottom	20.59	40.39	19.8	23.6
			Top	16.27	26.86	10.59	18.82
Sep-08	720	2.65	Bottom	20	21.18	1.18	20.35
			Top	16.47	17.45	0.98	16.57
Total Average			Bottom	23.12	32.62	9.50	25.87
			Top	17.45	31.68	14.23	20.29

Table 5.6 Moisture content data of the 8% lime with 0.15% fibers section

Month	Data Points	Rainfall (in.)	Location of Probe	Minimum	Maximum	Range	Average
Oct-07	492	4.65	Bottom	20.59	49.8	29.21	26.85
			Top	27.65	31.96	4.31	29.33
Nov-07	719	2.61	Bottom	22.75	49.8	27.05	27.32
			Top	28.63	49.8	21.17	31.76
Dec-07	744	2.53	Bottom	24.51	49.8	25.29	30.32
			Top	31.76	49.8	18.04	35.1
Jan-08	745	1.89	Bottom	23.92	31.18	7.26	27.28
			Top	32.35	37.65	5.3	34.97
Feb-08	696	2.31	Bottom	23.33	49.8	26.47	32.84
			Top	29.61	49.8	20.19	39.88
Mar-08	744	3.13	Bottom	25.69	49.8	24.11	41.91
			Top	29.22	49.8	20.58	46.98
Apr-08	720	3.46	Bottom	17.45	49.8	32.35	29.48
			Top	20.78	49.8	29.02	33.37
May-08	744	5.3	Bottom	30	49.8	19.8	39.5
			Top	30.78	49.8	19.02	42.9
Jun-08	718	3.92	Bottom	17.06	35.88	18.82	24.55
			Top	20.59	36.08	15.49	23.66
Jul-08	744	2.43	Bottom	25.88	49.8	23.92	34.51
			Top	20.39	31.57	11.18	25.92
Aug-08	722	2.17	Bottom	37.84	49.8	11.96	43.99
			Top	28.24	46.08	17.84	34.6
Sep-08	720	2.65	Bottom	36.86	48.24	11.38	43.16
			Top	30	39.61	9.61	35.32
Total Average			Bottom	25.49	46.96	21.47	33.48
			Top	27.50	43.48	15.98	34.48

Table 5.7 Moisture content data of the 8% lime section

Month	Data Points	Rainfall (in.)	Location of Probe	Minimum	Maximum	Range	Average
Oct-07	492	4.65	Bottom	20	20.59	0.59	20.22
			Top	24.12	26.47	2.35	25.17
Nov-07	719	2.61	Bottom	20	24.12	4.12	20.95
			Top	24.71	31.57	6.86	26.21
Dec-07	744	2.53	Bottom	24.12	49.8	25.68	36.07
			Top	26.08	37.84	11.76	27.55
Jan-08	745	1.89	Bottom	32.75	42.75	10	37.72
			Top	26.08	28.24	2.16	27.18
Feb-08	696	2.31	Bottom	25.69	49.8	24.11	35.36
			Top	24.12	36.47	12.35	26.57
Mar-08	744	3.13	Bottom	23.53	49.8	26.27	40.03
			Top	23.73	36.67	12.94	28.49
Apr-08	720	3.46	Bottom	17.65	49.8	32.15	29.21
			Top	20.2	37.25	17.05	24.7
May-08	744	5.3	Bottom	22.75	49.8	27.05	29.02
			Top	23.73	36.67	12.94	26.45
Jun-08	718	3.92	Bottom	17.06	24.12	7.06	19.03
			Top	19.22	25.29	6.07	20.53
Jul-08	744	2.43	Bottom	17.25	29.8	12.55	21.3
			Top	19.22	24.51	5.29	20.75
Aug-08	722	2.17	Bottom	19.22	40	20.78	24.71
			Top	20.2	28.82	8.62	22.98
Sep-08	720	2.65	Bottom	19.02	27.06	8.04	21.42
			Top	20.2	24.31	4.11	21.5
Total Average			Bottom	21.59	38.12	16.53	27.92
			Top	22.63	31.18	8.54	24.84

The data of monthly rainfall is depicted in Figure 5.2 for a period of one year from October 2007 to September 2008. During the observed period, the maximum moisture content recorded was 38% in control section, 50% in compost section, 43% in 4% lime mixed with 0.30% fibers section, 50% in 4% lime mixed with 0.30% fibers section and 50% in 8% lime treated sections.

It could be seen from the data that on a number of occasions the maximum moisture content in the treated section was approaching 50% indicating a condition of saturation of soil.

The annual average maximum moisture content measured in the increasing order for the test sections is 26% for control section, 29% for compost section, 33% for 4% lime with 0.30% fibers section, 35% for 8% lime section and 45% for 8% lime with 0.30% fibers section. Higher moisture content is expected to prevent desiccation cracking during dry season.

The minimum moisture content recorded in the test sections can be termed as the residual moisture content of soil and this is important for the growth of vegetation on the test section. Wilting point is defined as the lowest moisture content below which the roots of plant will not be able to absorb any moisture from soil and as a result it wilts (Fredlund, 1978). If the residual moisture content is less than the wilting point then the vegetation will not grow. The wilting point however depends on type of soil, type of plant and atmospheric conditions.

Further it can be noticed that lesser residual moisture content leads to complete shrinkage conditions resulting in desiccation cracking in soils. Huge cracks appeared at the surface during the early summer periods of 2008 at the test section site of Joe Pool Dam in control section and compost treated section as shown in Figure 5.32. Prominent size cracks were later appeared in the control section and the compost section during June 2008 as shown in Figure 5.34 and 5.35 respectively. The residual moisture contents at depths of 25 cm (10 in.) were 15% for control section and 16% for compost treated section. As such, soil having high residual moisture content is preferred so as not to have desiccation cracking and to enhance vegetation growth.

The residual moisture content noticed during the year-long monitoring period is given below in Table 5.8. The minimum moisture contents recorded during October 07 are ignored as the test section was fairly new at that time as the construction was completed during September and October 2007.

Table 5.8 Residual moisture content of the treated Sections

Test Section	Residual Moisture Content	Remarks
Control	15%	Nov. 07, June 08 and July 08
20% Compost	16%	June, July and August 08
4% Lime with 0.30% Fibers	14%	June and July 08
8% Lime with 0.15% Fibers	21%	April and June 08
8% Lime	19%	Top – June and July 08

It was hypothesized that the surface moisture contents were lesser than the values mentioned above, leading to decay of vegetation growth and formation of cracks. However no cracks were noticed in both lime and lime - fiber treated sections, which have higher residual moisture contents. It is always preferred to have higher residual moisture content so that no cracking occurs due to drying.

Based on the residual moisture content criterion, the control section, the compost section, the 4% lime with 0.30% fiber section, the 8% lime with 0.15% fiber section and the 8% lime sections are ranked as 4, 3, 5, 2 and 1, respectively.

5.2.9 Annual Moisture Content Variation or Range

Moisture content range is the difference between maximum moisture content and minimum moisture content of a soil. The moisture content variation parameter is complex as it depends on the coefficient of permeability of embankment, rainfall, infiltration capacity of embankment, temperature and evaporation of the site.

However, this data explains the effectiveness of the field treatment method with respect to the moisture content variation. Usually, the treatment can be mentioned as effective if the range is small and moisture content is uniform over a period of time.

Figure 5.9 shows the average variation of moisture content range over a 12 month period as observed at the Joe Pool Dam site.

It can be seen that the range is minimum for control section and compost sections. The range is highest for the 8% lime with 0.15% fibers section followed by the 8% lime and the 4% lime with 0.30% fibers section.

The range is highest in the 8% lime with 0.15% fibers because of the presence of fibers and the section reaching the values of saturation often immediately after the rainfall.

The range in the control section is the least as the soil has hardly reached saturation stage for various rainfall events.

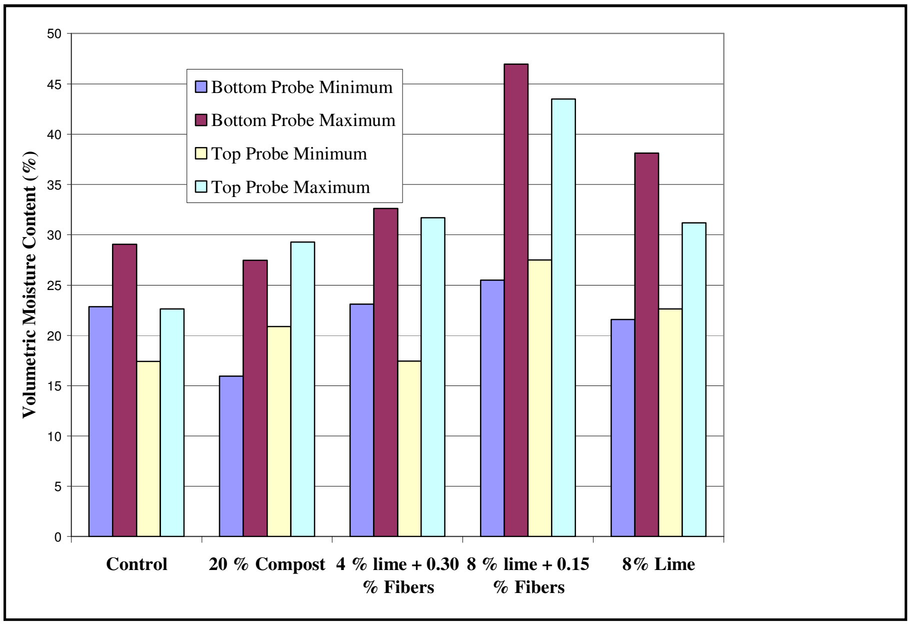


Figure 5.9 Minimum and maximum moisture content recorded in top and bottom probe

5.2.10 Average Annual Moisture Content

Figures 5.10 and 5.11 show the average annual moisture content of both top and bottom probes. For top layer of soil, compost holds more moisture content than the control section. This is apparently due to the presence of wood fibers and hydrophilic property of compost material. Moisture retention is also better in lime with fiber treated sections. Presence of fibers enhanced the porous nature of the soil, which in turn resulted in the higher absorption of moisture. Based on the moisture absorption and retention capacity of the treated sections as per observations of top probe, the control section, the compost section, the 4% lime with 0.30% fiber section, the 8% lime with 0.15% fiber section and the 8% lime sections are ranked as 4, 5, 3, 1 and 2, respectively.

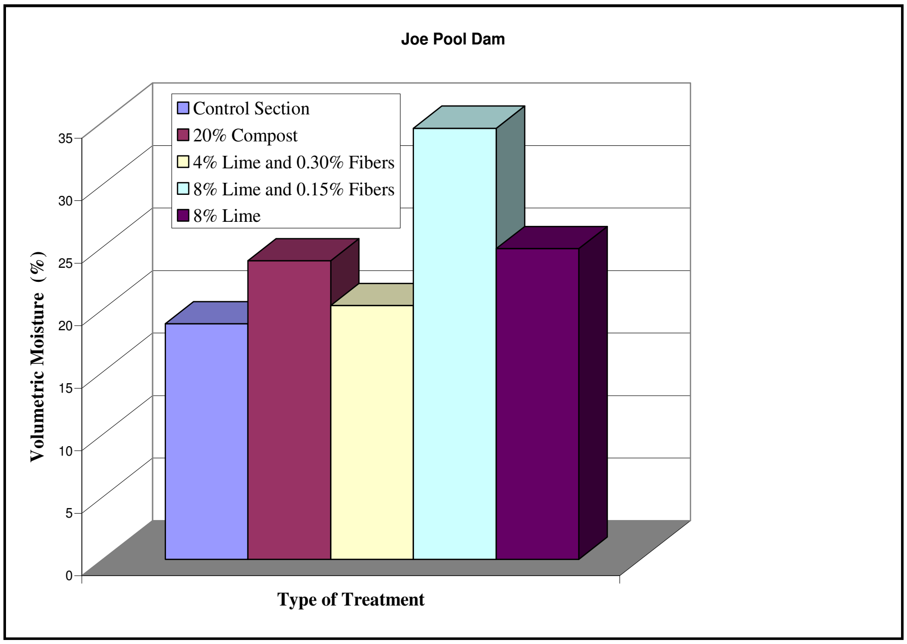


Figure 5.10 Average annual moisture content of top probe

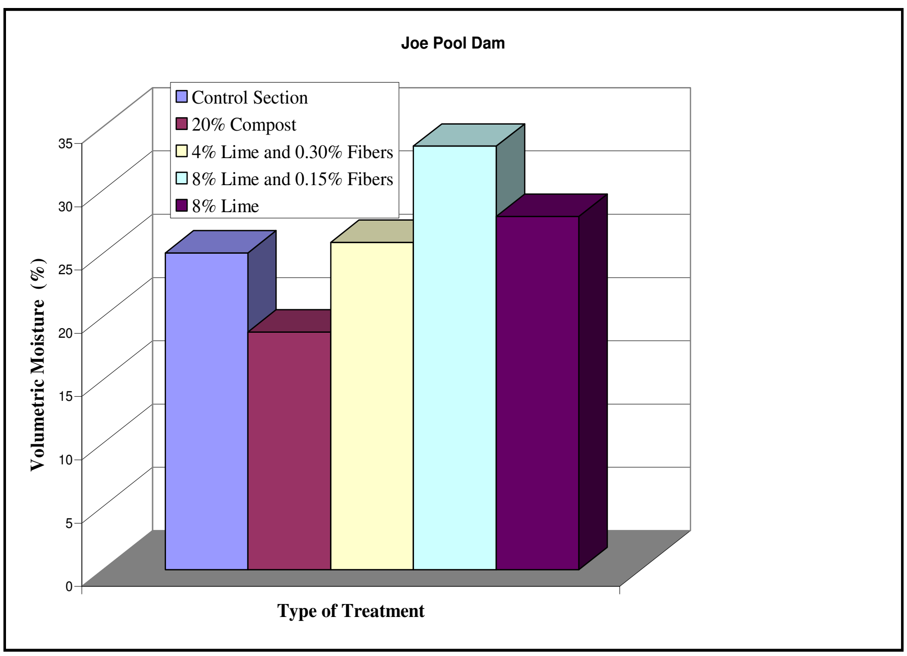


Figure 5.11 Average annual moisture content of bottom probe

Based on the moisture absorption and retention capacity of the treated sections as per the observations of bottom probe the control section, the compost section, the 4% lime with 0.30% fiber section, the 8% lime with 0.15% fiber section and the 8% lime sections are ranked as 4, 5, 3, 1 and 2, respectively.

The above ranking is based on the inference from the average annual values of moisture range and average moisture content as per the data compiled month wise. On most of the occasions, the data and interpretation can be misleading. There may be an apparent difference between two data sets and in real case, the difference may not be significant enough to rank the two treatments differently. A t-test is therefore conducted on the above data of both range and average moisture content values to arrive at reasonable conclusions about the relative effectiveness of various treatment methods in regard to the retention or absorption of moisture.

5.3 t-test

5.3.1 Significance of Statistical t-test

The statistical test is aimed at carrying out analysis of data for rational decision making regarding the usefulness of the treatment methods in solving the problem of surficial failures. The test allows incorporating the variability of data into the decision making process. Various terms involved with the statistical analysis are described below in brief.

The sample mean is the average value of all the observations in the data set. The average of the data taken from a set of larger population is known as population mean and its symbol is μ. If there are N observations in a population the population mean is

Population Mean, $$\mu = \frac{\sum_{i=1}^{N} x_i}{N} \quad (5.1)$$

The mean value always does not convey all the information about the sample of data. Population variance or population standard deviation is useful to study the variation of data. Standard Deviation is the positive square root of the variance.

Variance, $$\sigma^2 = \frac{\sum_{i=1}^{N} (x_i - \mu)^2}{N} \quad (5.2)$$

5.3.2 Hypothesis Testing with Test Statistic

Suppose the population of data has a normal distribution with mean μ and variance σ^2 the hypothesis is tested as following:

Null hypothesis: $$H_0 : \mu = \mu_0 \quad (5.3)$$

Alternative Hypothesis: $$H_0 : \mu \neq \mu_0 \quad \text{or} \quad (5.4)$$

$$H_0 : \mu > \mu_0 \quad (5.5)$$

Test statistic $$t = \frac{\bar{X} - \mu_0}{\sigma / \sqrt{n}} \quad (5.6)$$

is said to be having a t distribution with n-1 degrees of freedom.

The t value leads to explain whether to reject a null hypothesis. The critical t value (t_c) is obtained from the standard tables which depends on the value of α and degrees of freedom. If the absolute of an observed 't' is more than the critical value, the null hypothesis is rejected. Rejecting a null hypothesis when it is true is called a type I error. The probability of type I error is called the significant level of the test and is indicated as α. The usual value of α is taken as 5% (0.05).

The 'P' value of the statistical test is the smallest level of significance that would lead to rejection of the null hypothesis. That means when the 'P' value is calculated, it is compared with the significance level of 0.05 to make a decision. The smaller the 'P' value, the greater is the evidence against the null hypothesis. It is a usual practice to report the 'P' value along with the decision regarding the null hypothesis.

In this chapter, the t-test is carried out on range and average moisture content recorded by moisture probes of Joe Pool Dam test section sites.

The t-test results for the Top and Bottom probe for range of moisture content are shown in Table 5.9 to 5.12.

Table 5.9 t-test results for moisture content variation range for top probe

Parameter	Control	20% Compost vs. Control	4% Lime with 0.30% Fibers vs. Control	8% Lime with 0.15% Fibers vs. Control	8% Lime vs. Control
Mean	5.20	8.38	14.23	15.98	8.54
Variance	21.71	54.69	75.57	51.54	23.11
Observations	12	12	12	12	12
Hypothesized Mean Difference		0	0	0	0
Df		19	17	19	22
t Stat		1.26	3.17	4.36	1.73
P(T<=t) one-tail		0.11	0.00	0.00	0.05
t Critical one-tail		1.73	1.74	1.73	1.72

Table 5.10 t-test results for moisture content variation range for bottom probe

Parameter	Control	20% Compost vs. Control	4% Lime with 0.30% Fibers vs. Control	8% Lime with 0.15% Fibers vs. Control	8% Lime vs. Control
Mean	6.23	11.50	9.50	21.47	16.53
Variance	43.20	162.09	37.41	60.54	112.46
Observations	12	12	12	12	12
Hypothesized Mean Difference		0	0	0	0
Df		16	22	21	18
t Stat		1.28	1.26	5.18	2.86
P(T<=t) one-tail		0.11	0.11	0.00	0.01
t Critical one-tail		1.75	1.72	1.72	1.73

Table 5.11 t-test results for average moisture content variation for top probe

Parameter	Control	20% Compost vs. Control	4% Lime with 0.30% Fibers vs. Control	8% Lime with 0.15% Fibers vs. Control	8% Lime vs. Control
Mean	18.83	23.87	20.29	34.48	24.84
Variance	4.83	23.69	10.52	43.92	7.60
Observations	12	12	12	12	12
Hypothesized Mean Difference		0	0	0	0
Df		15	19	13	21
t Stat		3.27	1.29	7.76	5.90
P(T<=t) two-tail		0.01	0.21	0.00	0.00
t Critical two-tail		2.13	2.09	2.16	2.08

Table 5.12 t-test results for average moisture content variation for bottom probe

Parameter	Control	20% Compost vs. Control	4% Lime with 0.30% Fibers vs. Control	8% Lime with 0.15% Fibers vs. Control	8% Lime vs. Control
Mean	25.03	18.79	25.87	33.48	27.92
Variance	35.58	27.75	15.86	49.00	59.15
Observations	12	12	12	12	12
Hypothesized Mean Difference		0	0	0	0
Df		22	19	21	21
t Stat		2.72	0.40	3.18	1.03
P(T<=t) one-tail		0.01	0.35	0.00	0.16
t Critical one-tail		1.72	1.73	1.72	1.72

The t-test result for one tail test is more relevant for this study as it is expected that the treated sections will have an improved performance over the control section. The t-test results for one tail test are compared for all the four treatments relative to the performance of Control Section for top probe as shown in Table 5.13.

Table 5.13 Comparison of treated sections with control section for top probe

Treatment	t Statistic	t Critical	p Value	Difference Significant?	Conclusion
20% Compost- Range	1.26	1.73	0.11	No	Same
20% Compost- Average	3.27	1.75	0	Yes	Good
4% Lime with 0.30% Fibers - Range	3.17	1.74	0	Yes	Good
4% Lime with 0.30% Fibers -Average	1.29	1.73	0.11	No	Same
8% Lime with 0.15% Fibers - Range	4.36	1.73	0	Yes	Good
8% Lime with 0.15% Fibers - Average	7.76	1.77	0	Yes	Good
8% Lime - Range	1.73	1.72	0.05	Yes	Good
8% Lime - Average	5.90	1.72	0	Yes	Good

The t-test results reveal that there is no significant difference between the moisture content range of control soil and that of the compost treated soil, as seen in Table 5.13.

The t-test data indicates that the compost treated section performs better and holds higher moisture content than the control section in terms of holding higher moisture content at a depth of 25 cm (10 in.) from the surface.

In regard to the 4% lime with 0.30% fibers treated section, there is significant difference between the ranges and there is no significant difference between the average volumetric moisture content values.

In regard to both the 8% lime with 0.15% fibers section and the 8% lime treated section have shown significant difference with respect to control section for range and average moisture content.

Based on the t-test results of the moisture range and the average moisture content of the control section, the compost section, the 4% lime with 0.30% fiber section, the 8% lime with 0.15% fiber section and the 8% lime sections are ranked as 5, 4, 3, 1, and 2, respectively.

5.4 Soil Temperature Data

The temperature probes are located at a depth of 25 cm (10 in.) from the surface within the treated sections. The data recorded in Fahrenheit by the temperature probe is shown in Table 5.14 to 5.18

Table 5.14 Temperature data for control section of Joe Pool Dam

Month	Number of Data Points	Minimum (F)	Maximum (F)	Range (F)	Average (F)
Oct-07	492	57	79	22	68
Nov-07	719	50	71	21	62
Dec-07	744	45	64	20	51
Jan-08	745	41	58	17	47
Feb-08	696	44	59	16	51
Mar-08	744	45	67	22	57
Apr-08	720	59	72	13	65
May-08	744	65	83	18	73
Jun-08	718	79	91	12	85
Jul-08	744	82	95	12	90
Aug-08	722	80	96	16	88
Sep-08	720	72	89	16	80
Average		60	77	17	68

Table 5.15 Temperature data for compost section of Joe Pool Dam

Month	No. of Data	Minimum (F)	Maximum (F)	Range (F)	Average (F)
Oct-07	492	58	81	23	69
Nov-07	719	50	72	22	62
Dec-07	744	43	64	21	51
Jan-08	745	39	59	19	47
Feb-08	696	50	60	18	51
Mar-08	744	44	66	22	56
Apr-08	720	57	71	14	64
May-08	744	64	81	17	72
Jun-08	718	78	91	13	85
Jul-08	744	81	95	15	89
Aug-08	722	81	97	16	89
Sep-08	720	72	89	17	80
Average		60	77	18	68

Table 5.16 Temperature data for 4% lime with 0.30% fibers section of Joe Pool Dam

Month	No. of Data	Minimum (F)	Maximum (F)	Range (F)	Average (F)
Oct-07	492	58	80	20	68
Nov-07	719	50	70	21	62
Dec-07	744	38	63	19	51
Jan-08	745	40	57	17	47
Feb-08	696	44	59	15	51
Mar-08	744	45	68	23	57
Apr-08	720	59	73	14	66
May-08	744	67	86	19	75
Jun-08	718	81	95	14	87
Jul-08	744	84	97	13	91
Aug-08	722	81	98	17	90
Sep-08	720	74	89	15	81
Average		60	78	17	69

Table 5.17 Temperature data for 8% lime with 0.15% fibers section of Joe Pool Dam

Month	No. of Data	Minimum (F)	Maximum (F)	Range (F)	Average (F)
Oct-07	492	61	81	21	71
Nov-07	719	50	71	21	64
Dec-07	744	45	63	18	52
Jan-08	745	42	57	14	48
Feb-08	696	45	57	12	51
Mar-08	744	46	64	18	55
Apr-08	720	59	71	12	64
May-08	744	67	81	14	73
Jun-08	718	89	79	10	84
Jul-08	744	81	92	11	88
Aug-08	722	81	94	12	88
Sep-08	720	74	86	13	80
Average		62	75	15	68

Table 5.18 Temperature data for 8% lime section of Joe Pool Dam

Month	No. of Data	Minimum (F)	Maximum (F)	Range (F)	Average (F)
Oct-07	492	61	83	21	71
Nov-07	719	52	72	19	64
Dec-07	744	46	63	17	52
Jan-08	745	42	57	15	48
Feb-08	696	36	57	12	51
Mar-08	744	37	63	15	55
Apr-08	720	58	70	12	63
May-08	744	66	79	13	72
Jun-08	718	77	86	9	82
Jul-08	744	80	91	11	86
Aug-08	722	81	92	11	87
Sep-08	720	73	86	13	79
Average		59	75	14	68

The temperature probes in the test sections were located at a depth of 25 cm (10 in.) from surface as shown in Figure 5.1. It is noted from Figure 5.12 that there was no significant difference between the maximum and minimum temperatures recorded for the treated sections and those of control sections.

There is approximately 11ºF difference between maximum and minimum ambient temperature. The difference for the control section, the compost section, the 4% lime with 0.30% fibers, the 8% lime with 0.15% fibers and the 8% lime, is 17ºF, 18ºF, 17ºF , 15ºF and 14ºF, respectively.

From the temperature data, it is not possible to identify the best treatment to be adopted for dam slopes. Hence, temperature data is not considered for further analyses.

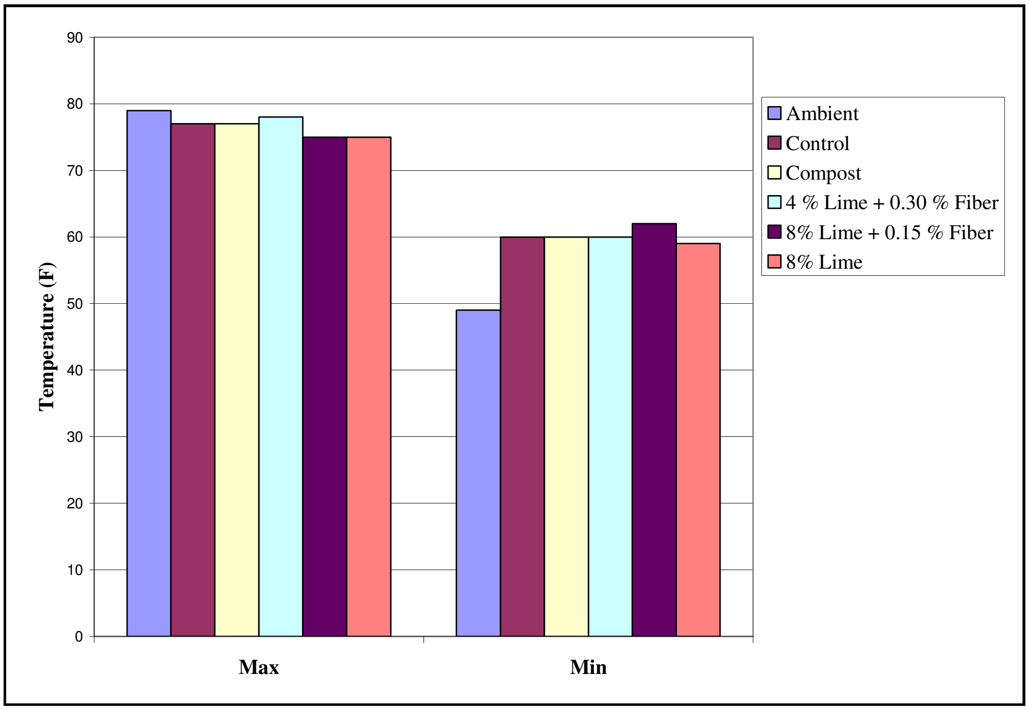


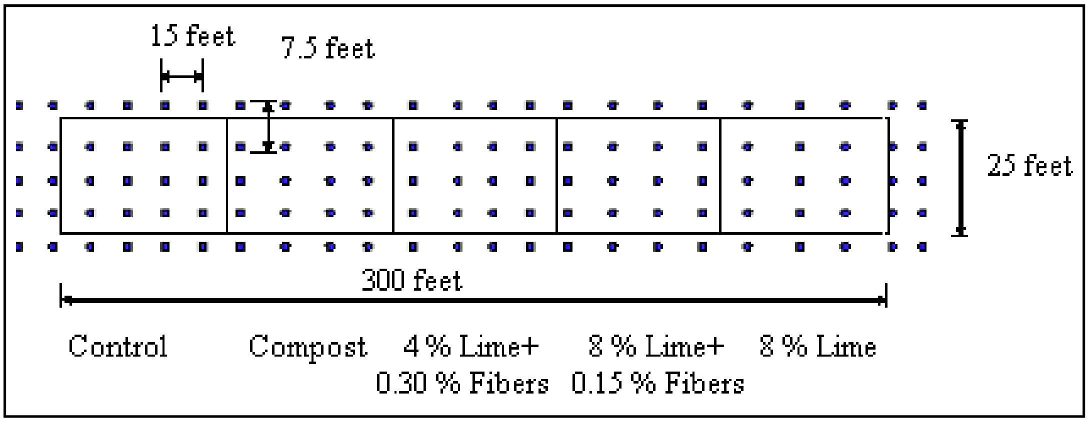
Figure 5.12 Average maximum and minimum soil temperature recorded

5.5 Elevation Surveys

Elevation surveys were carried out on the test sections during the field monitoring period. Steel pegs were driven on the ground at intervals of 4.57 m (15 ft) horizontally and 2.29 m (7.5 ft) vertically in five rows as shown in Figure 5.13.

It was important to fix the elevation pegs as the observations were to be taken at the same point every time to observe the pattern of vertical movements within the ground with respect to changes in moisture contents.

Elevation surveys were carried out using the total station device, which has a resolution of 0.25 cm (0.1 in). The first elevation survey was conducted in November 2007 after the completion of the construction of test sections including final compaction and seeding. The details of subsequent elevation surveys are presented in Table 5.19 and the vertical movements are compared with the first time readings taken during November 2007. The details of antecedent rainfall are also shown in Table 5.19.



(a)

(b)

Figure 5.13 Elevation survey (a) Layout (b) Steel pegs for survey

Table 5.19 Date of conducting elevation surveys

Date of Survey	Number of Days from First Survey	Details Antecedent Rainfall
11/15/2007	First Survey	10/15/2007 to 10/23/2007 – 6.8 cm (2.67 in.)
12/19/2007	34 days	12/12/2007 to 12/15/2007 – 4.7 cm (1.85 in.)
02/04/2008	81 days	01/23/2007 to 01/26/2008 – 1.5 cm (0.61 in.)
04/07/2008	144 days	03/19/2008 – 12 cm (4.76 in.); 04/04/2008 – 0.5 cm (0.20 in.)
05/23/2008	190 days	05/14/2008 to 05/15/2008 – 3.7 cm (1.44 in.)
06/28/2008	226 days	06/26/2008 – 1.8 cm (0.70 in.)
08/02/2008	261 days	07/15/2008 to 07/16/2008 – 4.7 cm (1.85 in.)
10/04/2008	324 days	09/14/2008 – 1.1 cm (0.45 in.)

The total station instrument was always stationed at the middle of entire length in front of the test sections and at the same location on a firm concrete pedestal to yield the highest accuracy.

Apart from the fixed station point, five bench marks were set up on the crest of the dam on the bitumen paved surface. The readings of the total stations were used to calculate the movement of each point on the treated sections.

The rise in the elevation is treated as swelling of the surface and the fall in the elevation is treated as the shrinkage of the surface. The effect of soil erosion is neglected as it is the same for all the points located on the surface of the treated sections overlaid by top soil. The relative swell or shrinkage of each test section is shown in Figure 5.14.

The performance of the treated sections can be compared with the relative movement of control section and also with the movements of the original ground surface adjacent to the treated section.

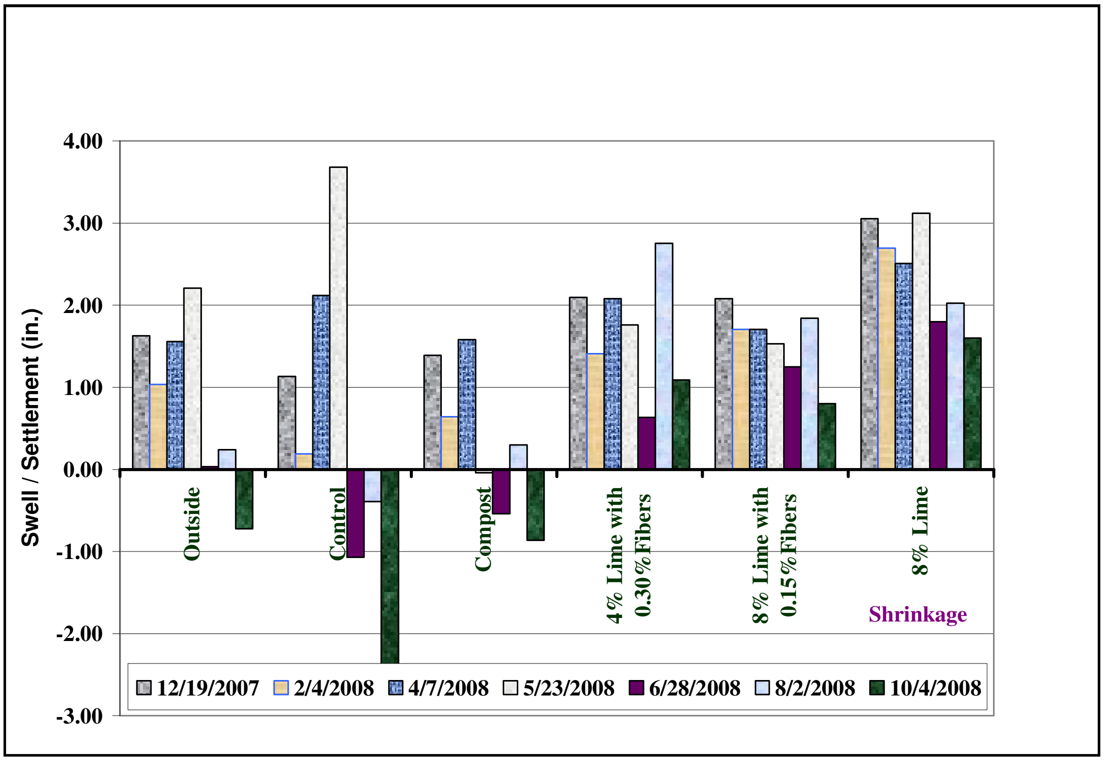


Figure 5.14 Summary of elevation survey results on test sections

It could be inferred from the above graph that the control section has been exhibiting highest movements implying swelling and shrinkage movements with respect to the environmental changes. Higher rainfall events induced swelling in the soil and dry periods resulted in shrinkage of soil. Besides the control section, swelling of soil was higher in the 8% lime treated section and it was least in the compost treated section. However, shrinkage was the highest in the control section followed by the compost section. It was also observed that the slope as a whole is experiencing swell and shrinkage movements as it could be noticed from the elevation survey results of section outside the test section. The newly constructed control test section experienced more swell and shrinkage movements than the outside ground surface.

The high intensity rainfall from December 08 onwards caused the soil to swell gradually reaching the highest value during May 08. The shrinkage tendencies started after May 08 onwards and these could be well correlated with the high average temperatures recorded at the site as shown in Tables 5.13 to 5.17. The shrinkage continued despite a few spells of small intensity rainfalls between June 08 and October 08.

During the field survey on 6/28/2008, it was noticed that there was a huge tension crack along the construction joint in control and compost sections. Prominent settlement was also noticed in the sections.

The laboratory results of swell and shrinkage tests presented in Table 4.8 and 4.9 show that the control soil and compost soil specimens exhibited high swell and shrinkage behavior than the lime with fibers treated soil specimens in laboratory conditions. The control and the compost soils exhibited higher swell and shrinkage in the field also. However, the field results show swell of about 5 cm (2 in.) in lime with fibers treated soils too. This is attributed to the fact that the treated sections were encapsulated with top soil, which has higher potential of swelling.

However, during dry weather the lime with fibers treated soils did not shrink as much as the control and compost soils. No shrinkage cracks were noticed during dry season in the lime with fiber treated sections. From the elevation survey results the total maximum and minimum surface variation for each test section during the monitoring period of one year is presented in Table 5.20.

Table 5.20 Maximum swell and shrinkage as per elevations survey

Treatment Type	Average cm (in.)	Maximum Variation cm (in.)	Minimum Variation cm (in.)	Range cm (in.)	Rank
Control	1.19 (0.47)	9.35 (3.68)	-6.03 (-2.37)	15.38 (6.05)	5
20% Compost	0.90 (0.35)	4.01 (1.58)	-2.19 (-0.86)	6.20 (2.44)	4
4% Lime with 0.30% Fibers	4.29 (1.69)	6.99 (2.75)	1.61 (0.63)	5.38 (2.12)	3
8% Lime with 0.15% Fibers	3.96 (1.56)	5.28 (2.08)	2.03 (0.80)	3.25 (1.28)	1
8% Lime	6.10 (2.40)	7.92 (3.12)	4.06 (1.60)	3.86 (1.52)	2
Outside	1.69 (0.66)	5.61 (2.21)	-1.84 (-0.72)	7.44 (2.93)	-

(+) indicates swell movement and (-) indicates shrinkage movement

From the above table it could be seen that the maximum variation of swell or shrinkage movements occurred in Control section and the least variation occurred in the 8% lime treated with 0.15% fibers. Based on the criteria of the total volume change shown as range in Table 6.20, the control section, the compost section, the 4% lime with 0.30% fiber section, the 8% lime with 0.15% fiber section and the 8% lime sections are ranked as 5, 4, 3, 1, and 2, respectively.

5.6 Inclinometer Surveys

Two vertical inclinometers were installed in each test section. The schematic indicating the notation of inclinometers is shown in Figure 5.15.

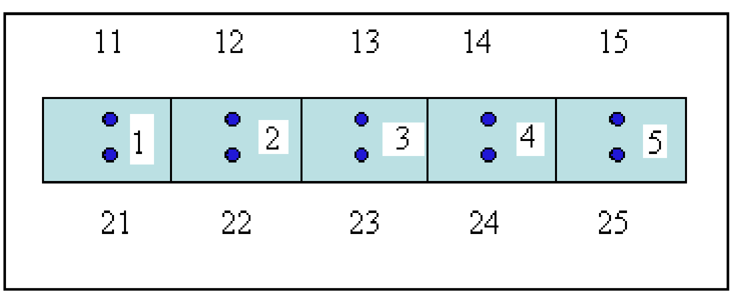


Figure 5.15 Notation used for inclinometers

The profile readings of the inclinometers were taken periodically with the help of an inclinometer probe and a data logger. The data is presented in the forms of the cumulative displacement versus depth of the inclinometer casing. The observations were taken at 0.30 m (1 ft) intervals. All the inclinometers were installed to a depth of about 4 m (13 ft) into the ground with a projection of 0.60 m (2 ft) above the ground surface. For Joe Pool Dam, the inclinometer 12 was installed to a depth of 4.3 m (14 ft) in to the ground due to site conditions with 0.60 m (2 ft) projection above the surface. The inclinometer cumulative displacements recorded on seven different dates is shown in the Figures 5.16 to 5.20. The cumulative displacement is calculated with reference to the first reading taken on 10/10/2007. The observations based on the profile of inclinometer readings as shown in Figures 5.16 to 5.20 is detailed as following:

A lateral displacement of soil was detected in all the five (5) test sections. The displacement monitored in the second row of inclinometers is higher than the first row. The displacement is the highest along the A-axis in the direction of the slope and is negligible along the B-axis in the direction perpendicular to slope is negligible. Till June 2008, the displacement was towards the down direction when the soil was swelling. The displacement started receding and moving in the upward direction once the shrinkage of soil commenced from June 2008. The movements of casing are prominent from ground level to about 3-4 ft of the soil mass. In order to study the displacement in both upward and downward directions in detail, the displacement graphs are presented for each inclinometer in Figures 5.21 to 5.30 and the summary of movements is shown in Table 5.21.

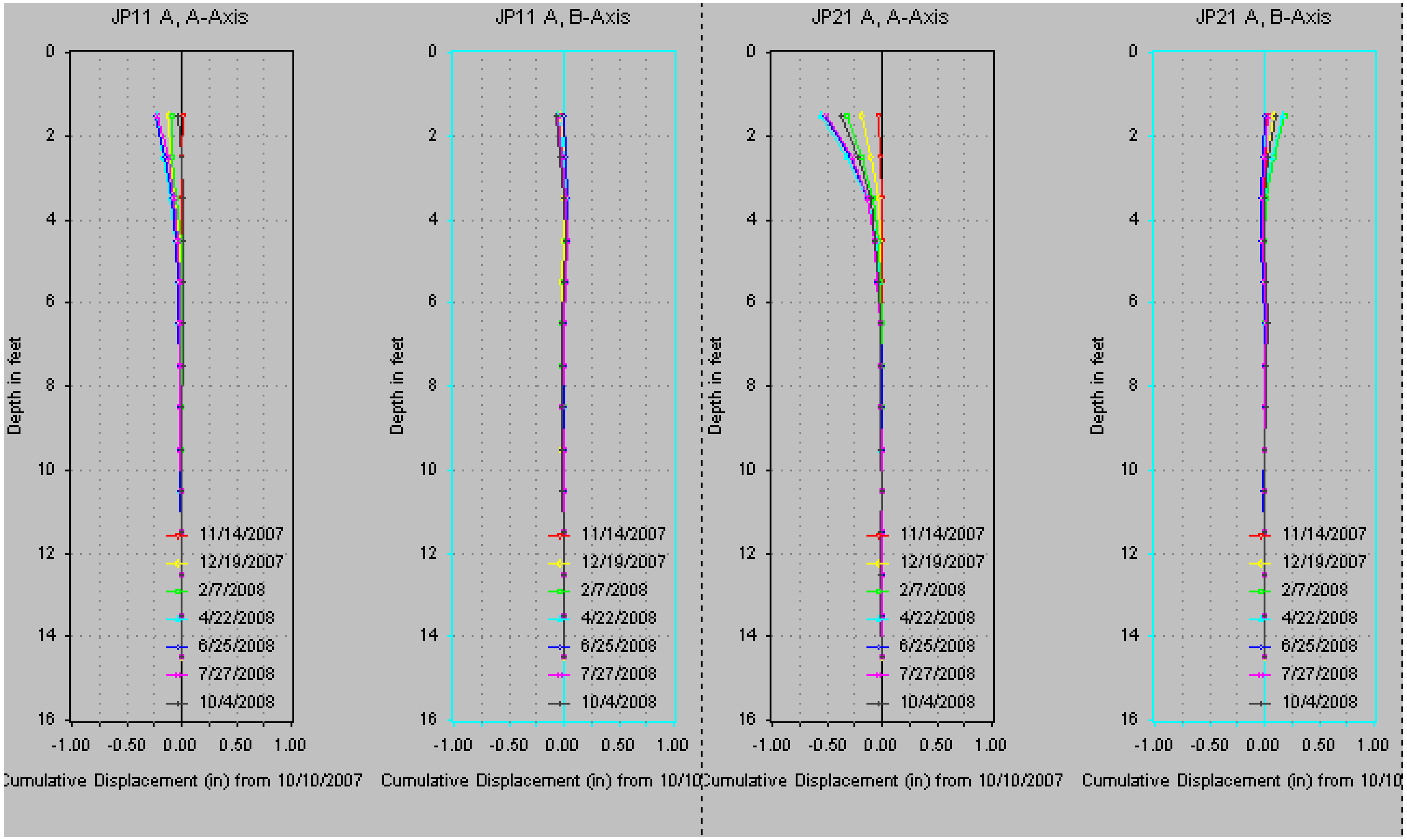


Figure 5.16 Inclinometer data for control section

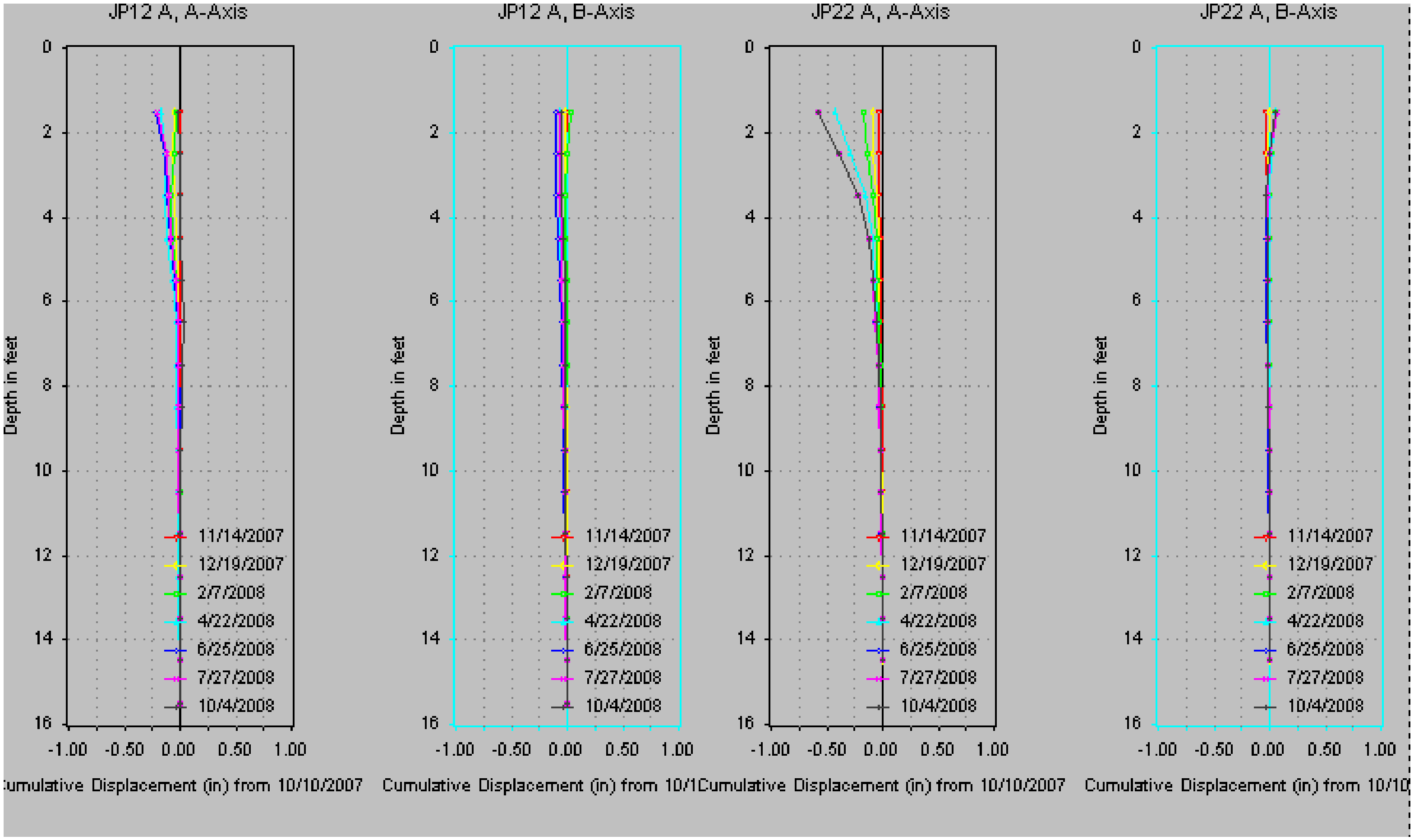


Figure 5.17 Inclinometer data for compost section

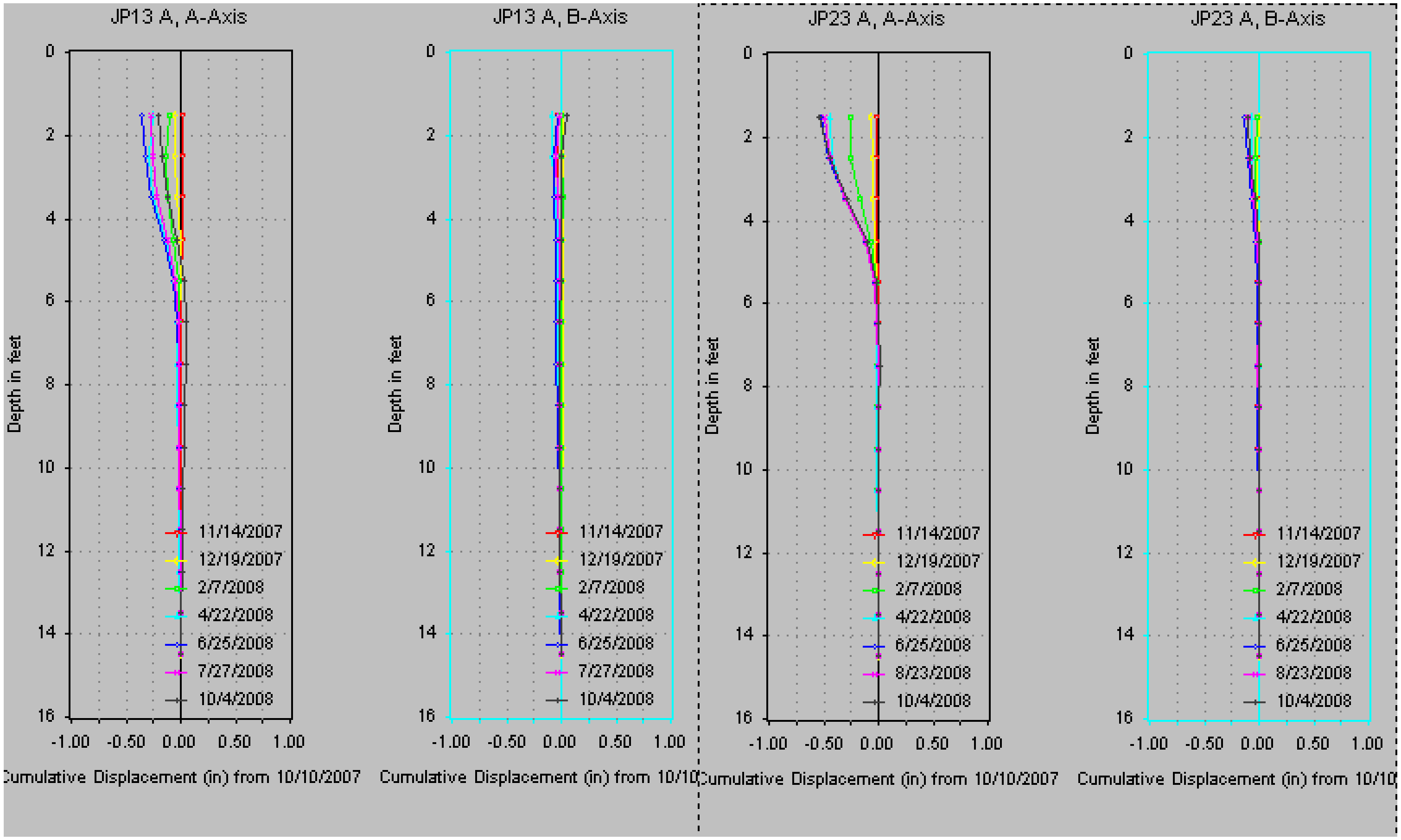


Figure 5.18 Inclinometer data for 4% lime with 0.30% fibers section

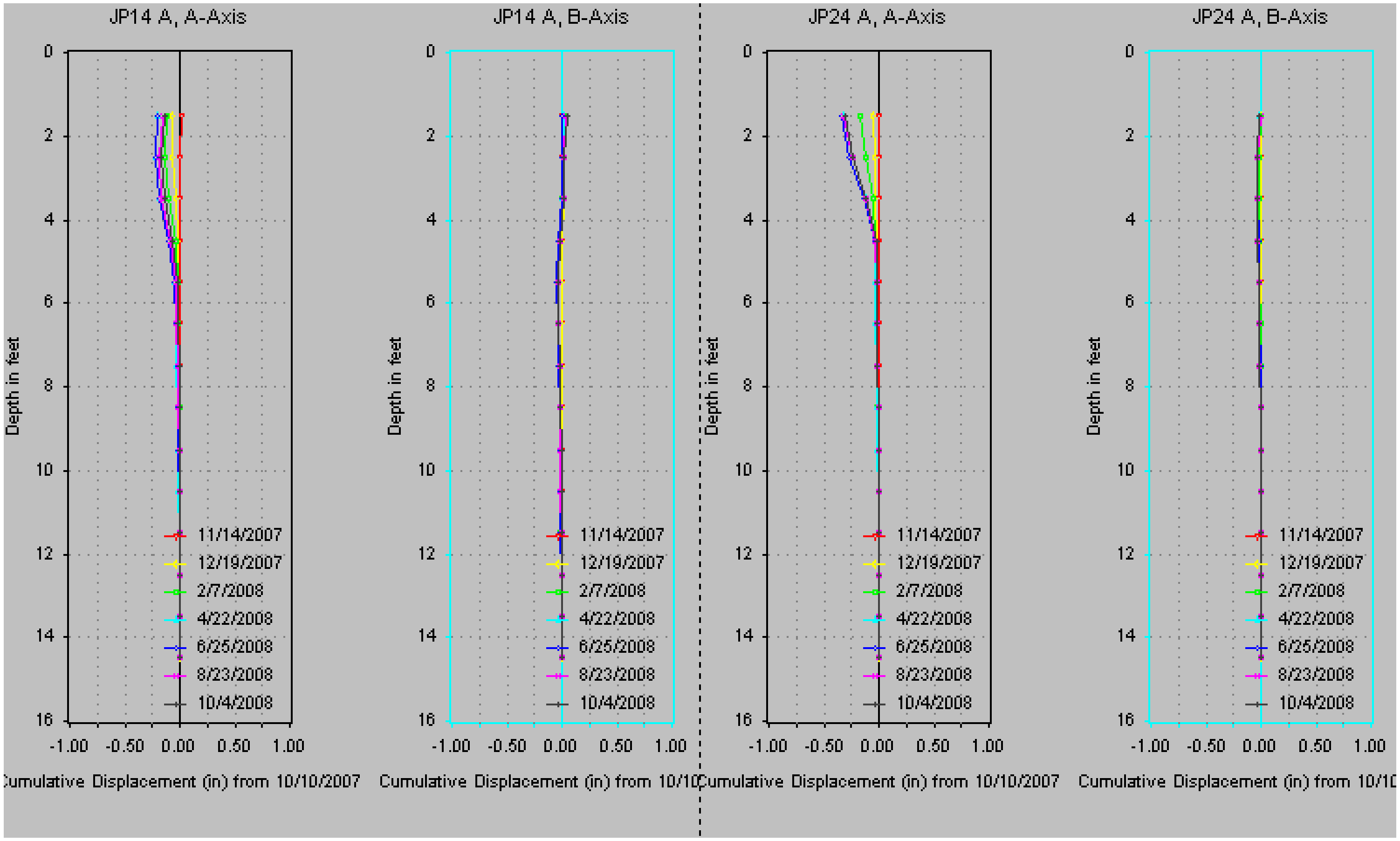


Figure 5.19 Inclinometer data for 8% lime with 0.15% fibers section

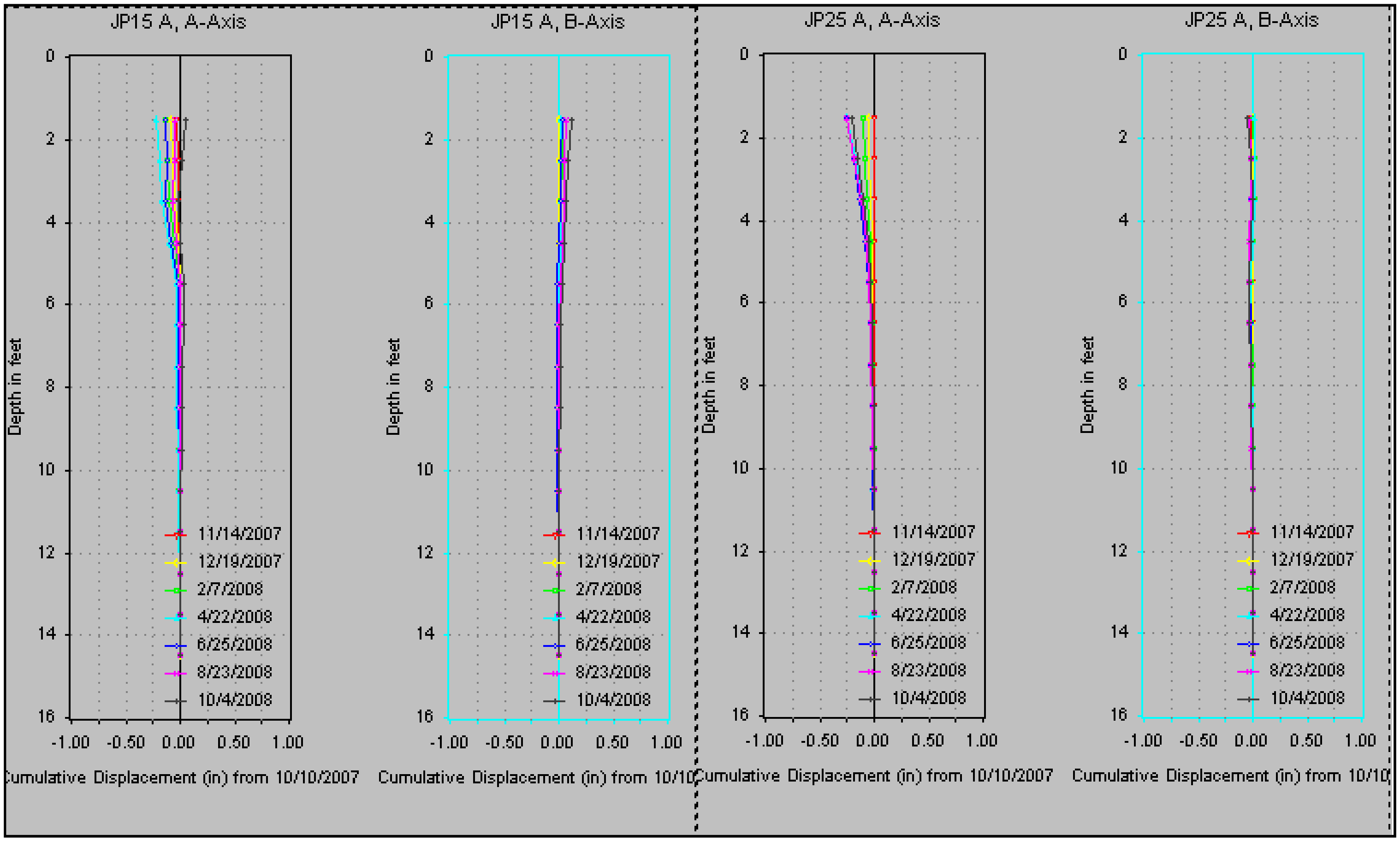


Figure 5.20 Inclinometer data for 8% lime section

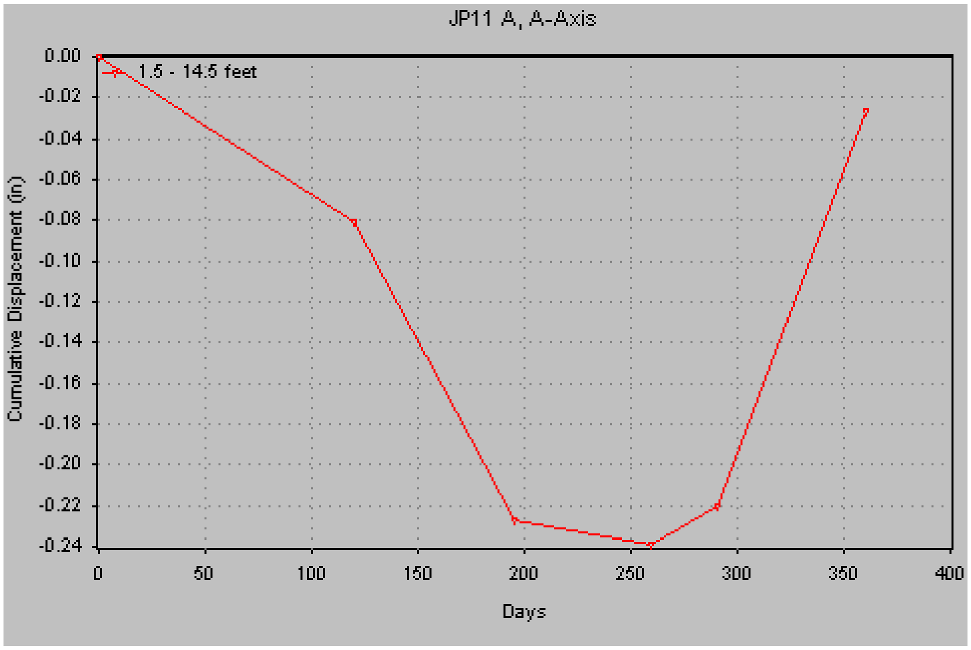


Figure 5.21 Time displacement graph of Inclinometer No. 11

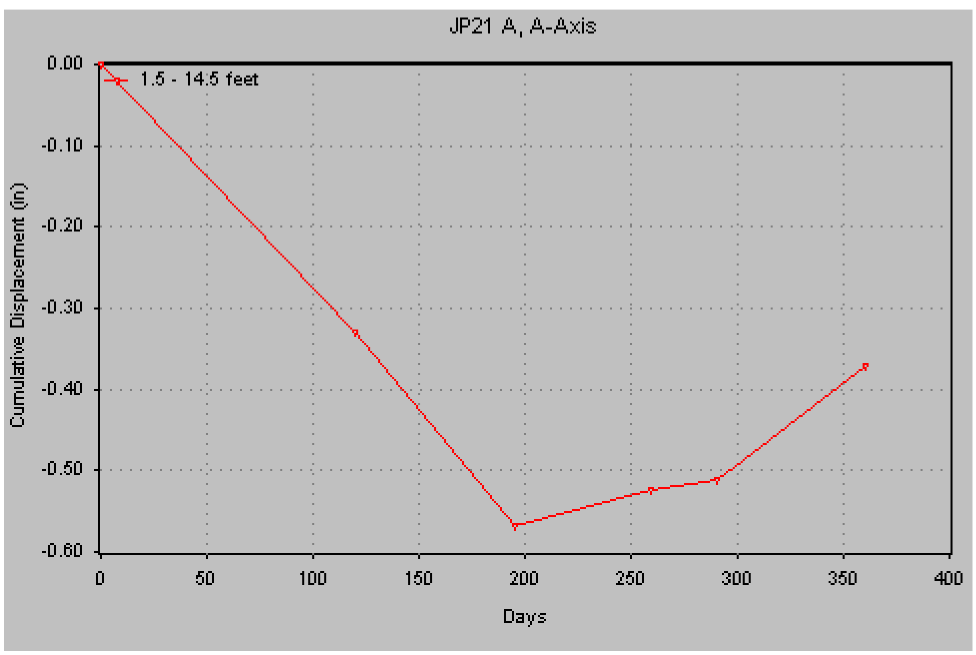


Figure 5.22 Time displacement graph of Inclinometer No. 21

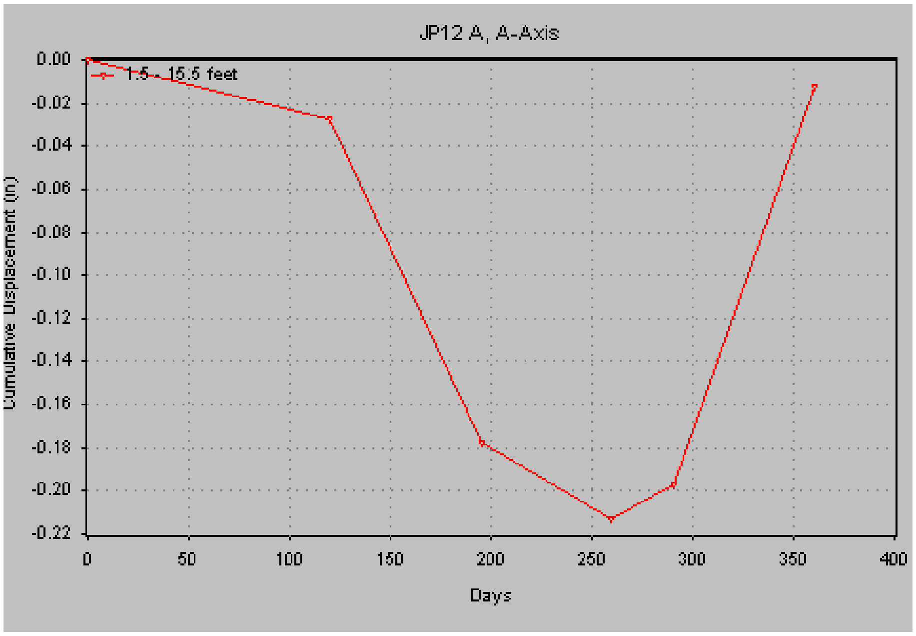


Figure 5.23 Time displacement graph of Inclinometer No. 12

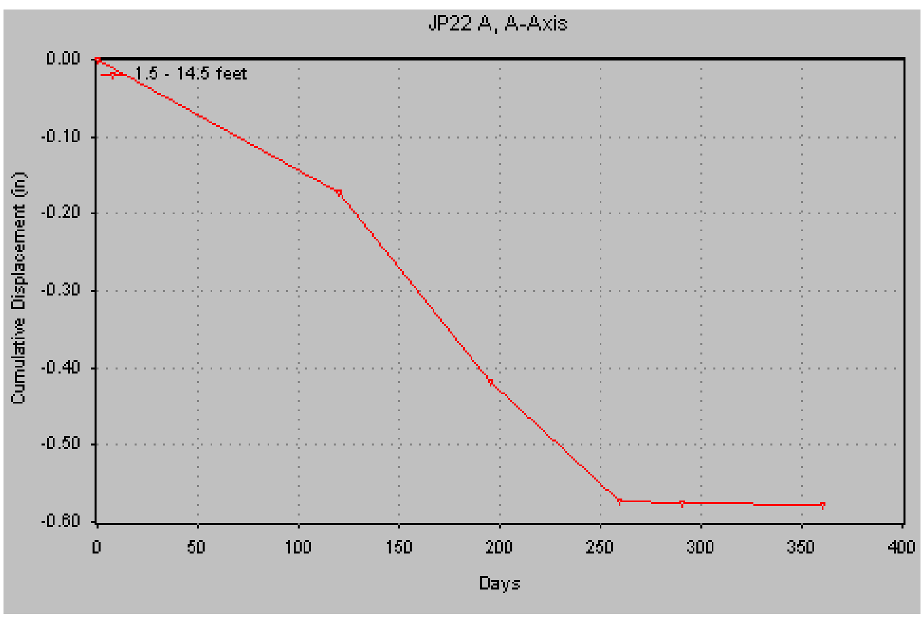


Figure 5.24 Time displacement graph of Inclinometer No. 22

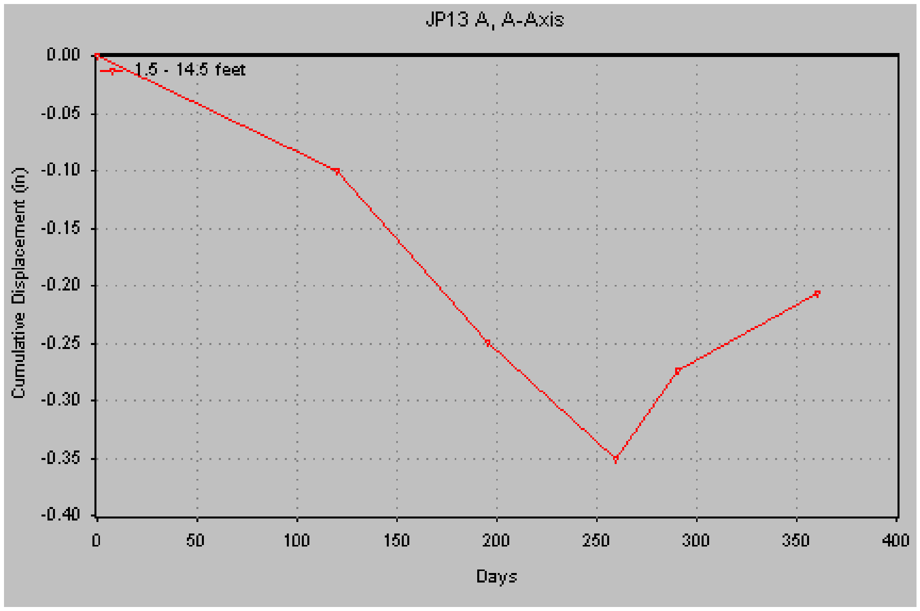


Figure 5.25 Time displacement graph of Inclinometer No. 13

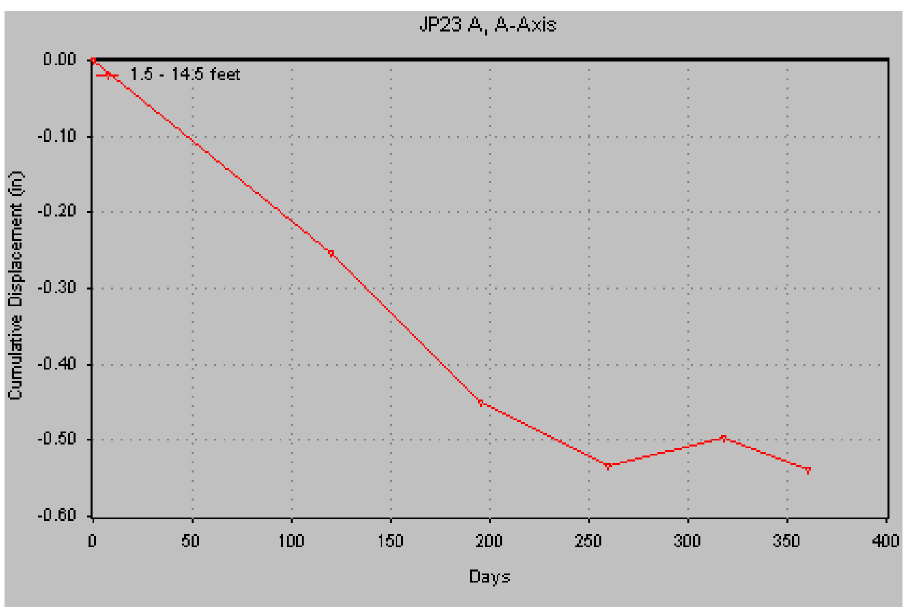


Figure 5.26 Time displacement graph of Inclinometer No. 23

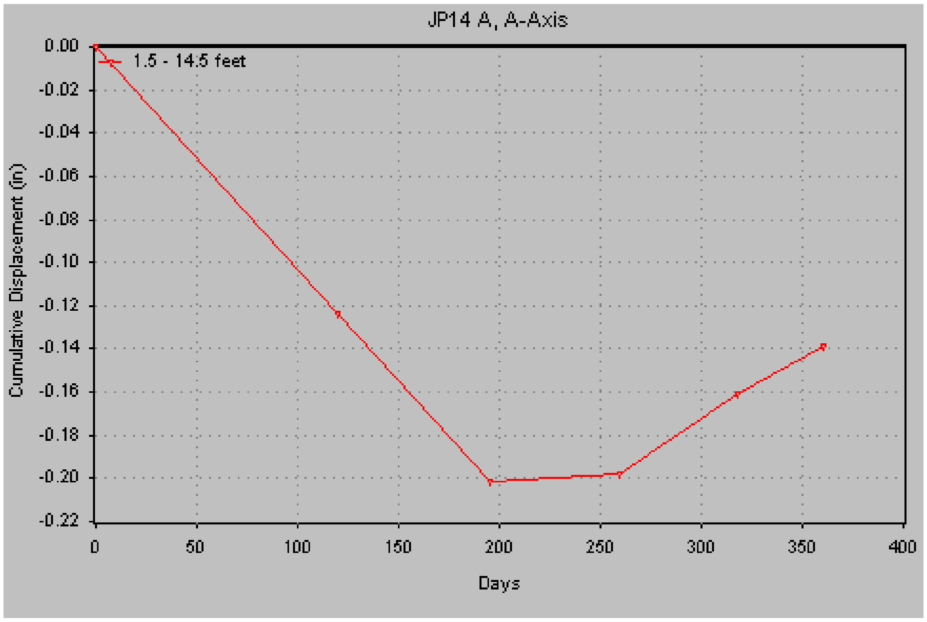


Figure 5.27 Time displacement graph of Inclinometer No. 14 of

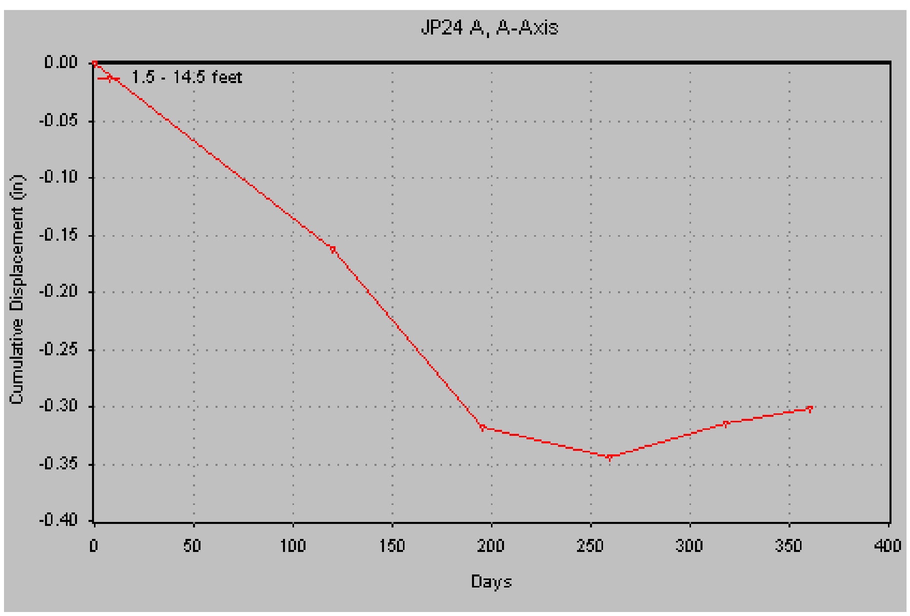


Figure 5.28 Time displacement graph of Inclinometer No. 24

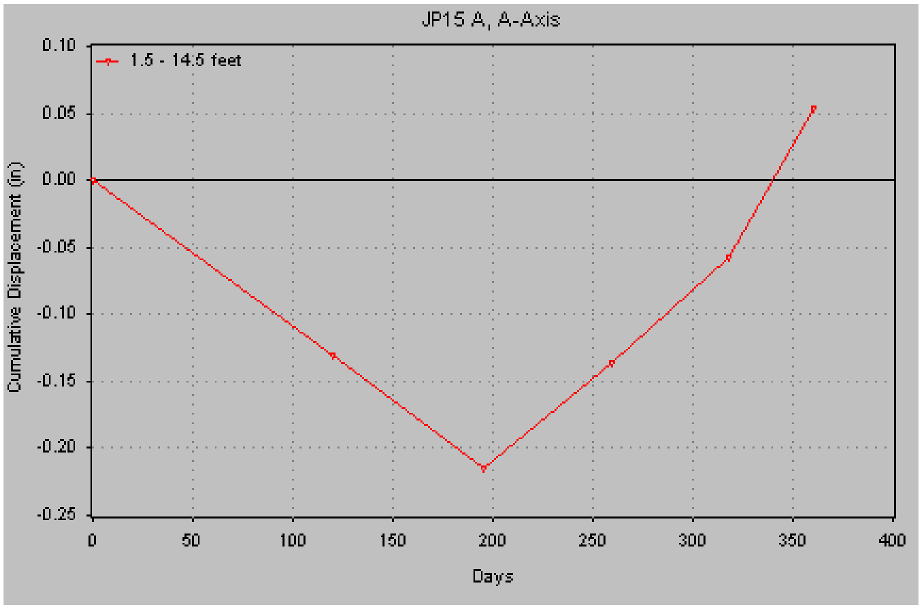


Figure 5.29 Time displacement graph of Inclinometer No. 15

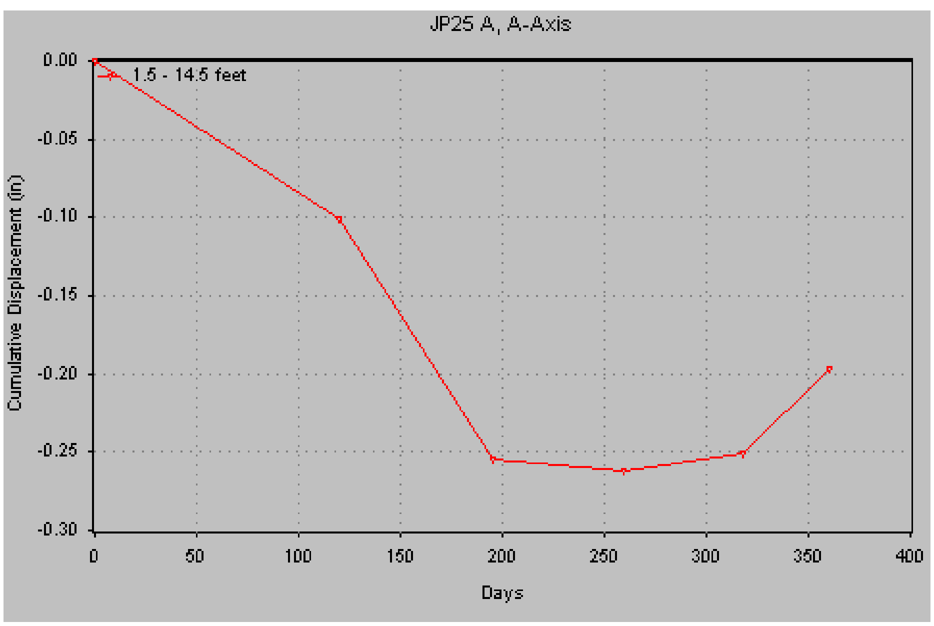


Figure 5.30 Time displacement graph of Inclinometer No. 25

Table 5.21 Summary of inclinometer casing movements

Section	Inclinometer Number	Maximum Movement cm (in.)	Movement after 1 year cm (in.)
Control	11	0.61 (0.24)	0.08 (0.03)
	21	1.47 (0.58)	0.97 (0.38)
20% compost	12	0.53 (0.21)	0.03 (0.01)
	22	1.47 (0.58)	1.47 (0.58)
4% Lime with 0.30% Fibers	13	0.89 (0.35)	0.51 (0.20)
	23	1.32 (0.52)	1.35 (0.53)
8% Lime with 0.15% Fibers	14	0.51 (0.20)	0.36 (0.14)
	24	0.86 (0.34)	0.76 (0.30)
8% Lime	15	0.56 (0.22)	-0.15 (-0.06)
	25	0.66 (0.26)	0.51 (0.20)

It is proposed to rank the effectiveness of the treatment with reference to the maximum swell related displacement recorded during inclinometer surveys as the focus of this research is on the surficial failures which occur during wet season.

Based on this criterion and the results presented in Table 5.21, the control section, the compost section, the 4% lime with 0.30% fiber section, the 8% lime with 0.15% fiber section and the 8% lime sections are ranked as 5, 4, 3, 1, and 2, respectively.

5.7 Surface Cracking and Digital Image Analysis

One of the vital areas of focus of this research is desiccation cracking during dry period. Desiccation cracks are seen at many places on the dam slope outside the test section.

Cracks were noticed on the paved surface on the crest of the dam as shown in Figure 5.31 even before the construction of test section commenced.

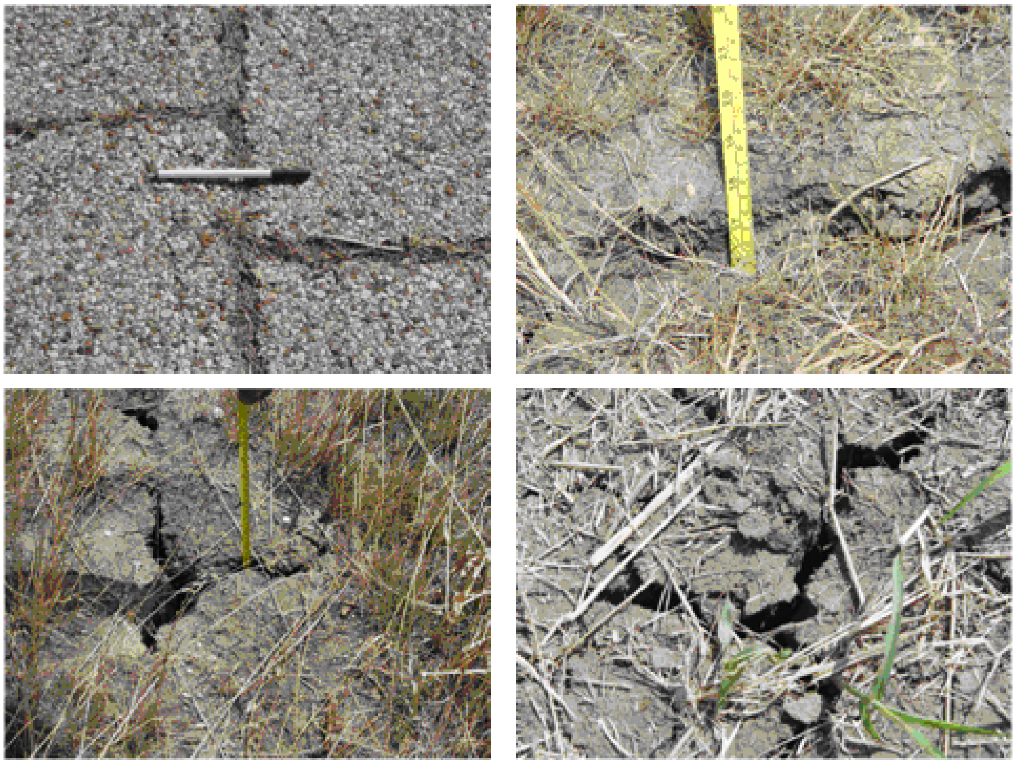


Figure 5.31 Cracks noticed on the crest and slope of
Joe Pool Dam outside test sections

However, in the treated sections, no cracks were noticed till June 2008 since the construction of test sections.

On March 17th 2008, a few thin desiccation cracks were noticed at various parts of the test sections but they were not deep enough as shown in Figure 5.32. The minor cracks detected on the surface layer of soil disappeared following a heavy downpour event on March 19th, 2008.

No further desiccation cracks surfaced till June 2008. On June 26th 2008, huge cracks were noticed on the surface of control section and compost section. Cracks were prominent along the construction joint of old surface and new test section on the top side of test sections.

Figure 5.32 Minor cracks noticed during March 2008 at Joe Pool Dam

Few cracks were as wide as 6 inches and as deep as 18 inches. On the control section, 6 feet long crack along with few other cracks were also noticed in the middle of section and also near the top inclinometer casing. Compost section has fewer cracks on the surface but has severe cracks along the construction joint on the top side. The schematic of cracks noticed is shown in Figures 5.33 to 5.35 by combining number of photos taken.

However, No cracks were noticed in the remaining three test sections constructed with lime and lime with fibers. The cracks were disappearing whenever there was rain and sometimes they were surfacing partly during dry weather. Sowers (1979) observed that the fissures formed as a result of desiccation or weathering appear to be closed, but they still remain planes of weakness and paths of seepage.

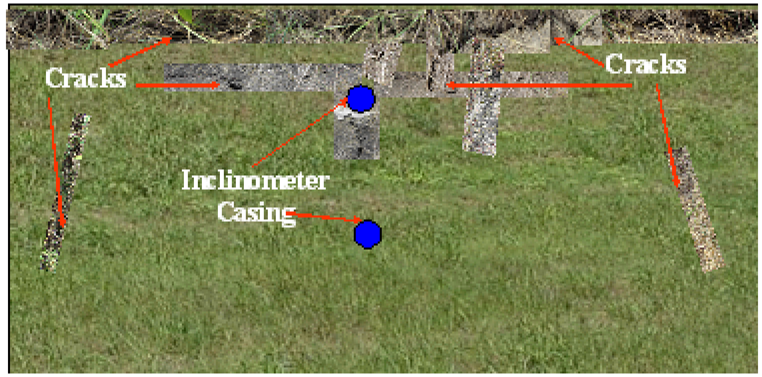


Figure 5.33 Approximate view of surficial cracks in control section formed in June 2008 at Joe Pool Dam

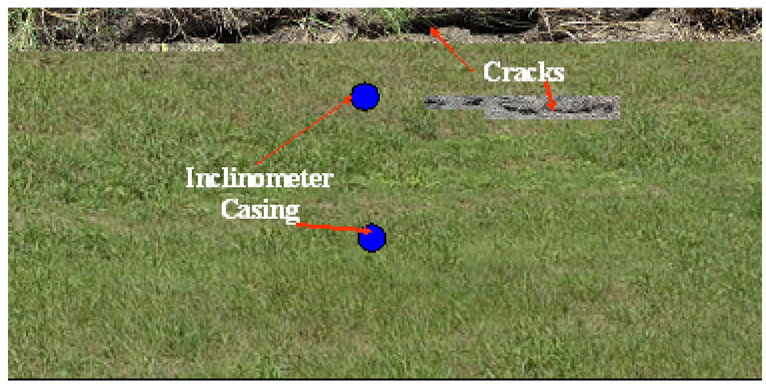


Figure 5.34 Approximate view of cracks in compost section formed in June 2008 at Joe Pool Dam

Figure 5.35 Cracks near construction joint of control and compost sections

As such, the effectiveness of treatment is to be judged carefully with respect to the extent of cracking. In order to assess the extent of cracking, digital image analysis is carried out using software called Scion Image. The digital images are converted into bit map images to be made accessible by software. The software also accepts TIFF images apart from BMP images.

A known distance measured actually in the field is correlated with the distance on the image in the form of number of pixels with the help of scion software. Thus the scale is set and the length and area measured on the image can be obtained in terms of actual field measurements.

In the photograph, two pegs were driven at a distance of 0.9 m (3 ft). The distance on the image was 500 pixels. The scale is set so that 500 pixel of length is equal to 3 feet. This gives the total area of digital image as 0.66 m^2 (7.09 ft^2). The image is converted to a gray scale. A suitable threshold value is selected so that the crack alone is prominent. The unwanted dark images if any are removed using an eraser tool.

The software provides the total area of the cracked section. The total cracked area of the section is 0.0084 m^2 (0.09 ft^2). The percent area of the cracked surface in this image is 1.27%.

All the digital images showing the cracks are analyzed and the percentage of cracked section in the images is reported. Figure 5.36 shows the digital image of cracked surface of control section and Figure 5.37 shows the image after threshold respectively. Table 5.22 presents the digital crack image collection data and monitoring details of all the test sections and their shrinkage cracking data.

Figure 5.36 Digital image of cracked surface in control section

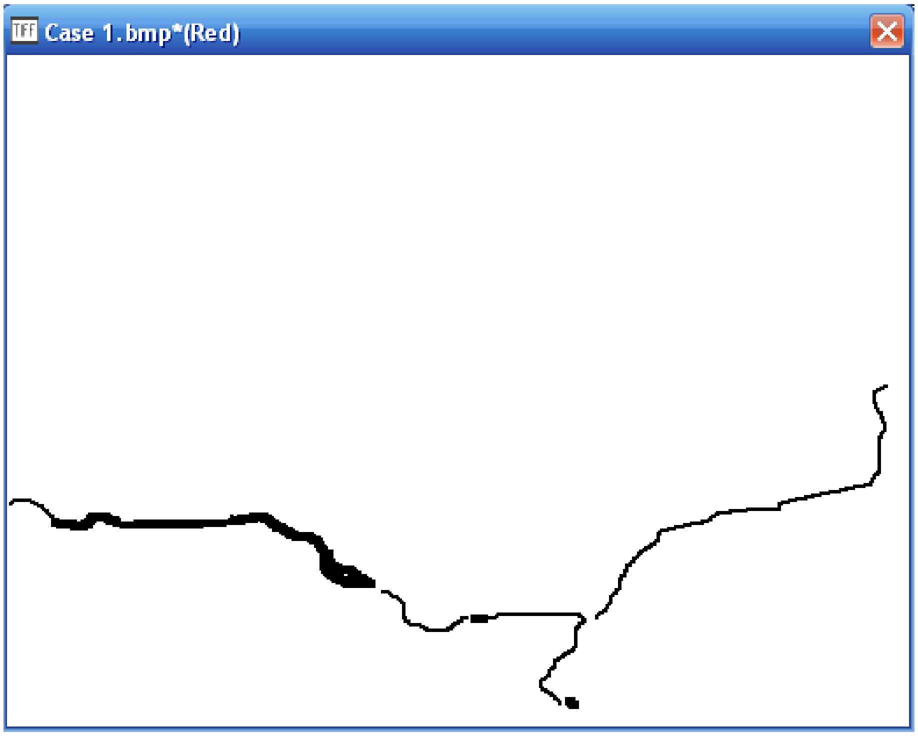


Figure 5.37 Image of crack after threshold

Table 5.22 Shrinkage crack monitoring data

Month	Control Section	Compost Section	4% Lime with 0.30% Fibers	8% Lime with 0.15% Fibers	8% Lime
October 07	None	None	None	None	None
November 07	None	None	None	None	None
December 07	None	None	None	None	None
January 08	None	None	None	None	None
February 08	None	None	None	None	None
March 08	Minor	Minor	Minor	None	None
April 08	None	None	None	None	None
May 08	None	None	None	None	None
June 08	1.08%	1.37%	None	None	None
August 08	None	None	None	None	None
September 08	None	None	None	None	None
October 08	None	None	None	None	None

Based on the shrinkage cracks observed and analyzed on each test section, the control section, the compost section, the 4% lime with 0.30% fiber section, the 8% lime with 0.15% fiber section and the 8% lime section are ranked as 4, 5, 3, 1, and 1, respectively.

5.8 In Situ DCP Tests

In order to assess the strength of the test sections, in situ tests were carried out using dynamic cone penetration (DCP) tests. Research conducted based on laboratory tests on sulfate rich soils with 8% lime with 0.30% fibers by Sappington IV (2003) at The University of Texas at Arlington, showed that the strength increased with wetting and drying cycles. The results of the DCP tests conducted during October 2008 were compared in Table 5.23 with the results of tests conducted immediately after initial curing period of construction of test sections. It could be seen that in a period of one year, the undrained shear strength of control soil and compost treated soil reduced where as the strength of lime with fiber treated soil has increased.

Table 5.23 DCP test results

Treatment	Average Number of blows per 3 Penetrations of 44 mm each	
	October 2007	October 2008
Control	10+ 10 +12 = 34	6 + 8 + 8 = 22
20% Compost	8+ 8+ 13 = 34	7 + 11 + 13 = 31
4% Lime with 0.30% Fibers	11 + 20 + 24 = 55	31 + 35 + 26 = 92
8% Lime with 0.15% Fibers	15 + 50 + 75 = 140	58 + 72+ 73 = 203
8% Lime	16 + 34 + 55 = 105	69 + 77 + 112 = 258

Based on the results of DCP test as shown in Table 5.23, the control section, the compost section, the 4% lime with 0.30% fiber section, the 8% lime with 0.15% fiber section and the 8% lime sections are ranked as 5, 4, 3, 2, and 1, respectively.

5.9 Vegetation Growth

The vegetation growth is important from the point of view of preventing soil erosion, aesthetic view, moisture retention, uniform infiltration and positive environmental impact. During the monitoring period of complete one year, photographs were taken during site visits to document the growth pattern of vegetation on the test sections.

From Figure 5.38 (a) and (b), it can be seen that the vegetation growth was faster in 8 % lime treated section which can be correlated with the highest moisture content of about 26 % available as per top moisture probe measurement. The moisture content as measured in the top probe of control section shows that the moisture levels were gradually increasing till February 2008 representing slower growth of vegetation.

The growth in the compost section, the 4 % lime mixed with 0.30 % fiber section and the 8 % lime mixed with 0.15 % fibers section is not faster though the moisture levels till February were high as per the top probe measurements. This is apparently due to the fact that the higher moisture content is measured due to the presence of fibers and growth of vegetation was slow till February 2008. However, the vegetation has grown well by April 2008 in all sections.

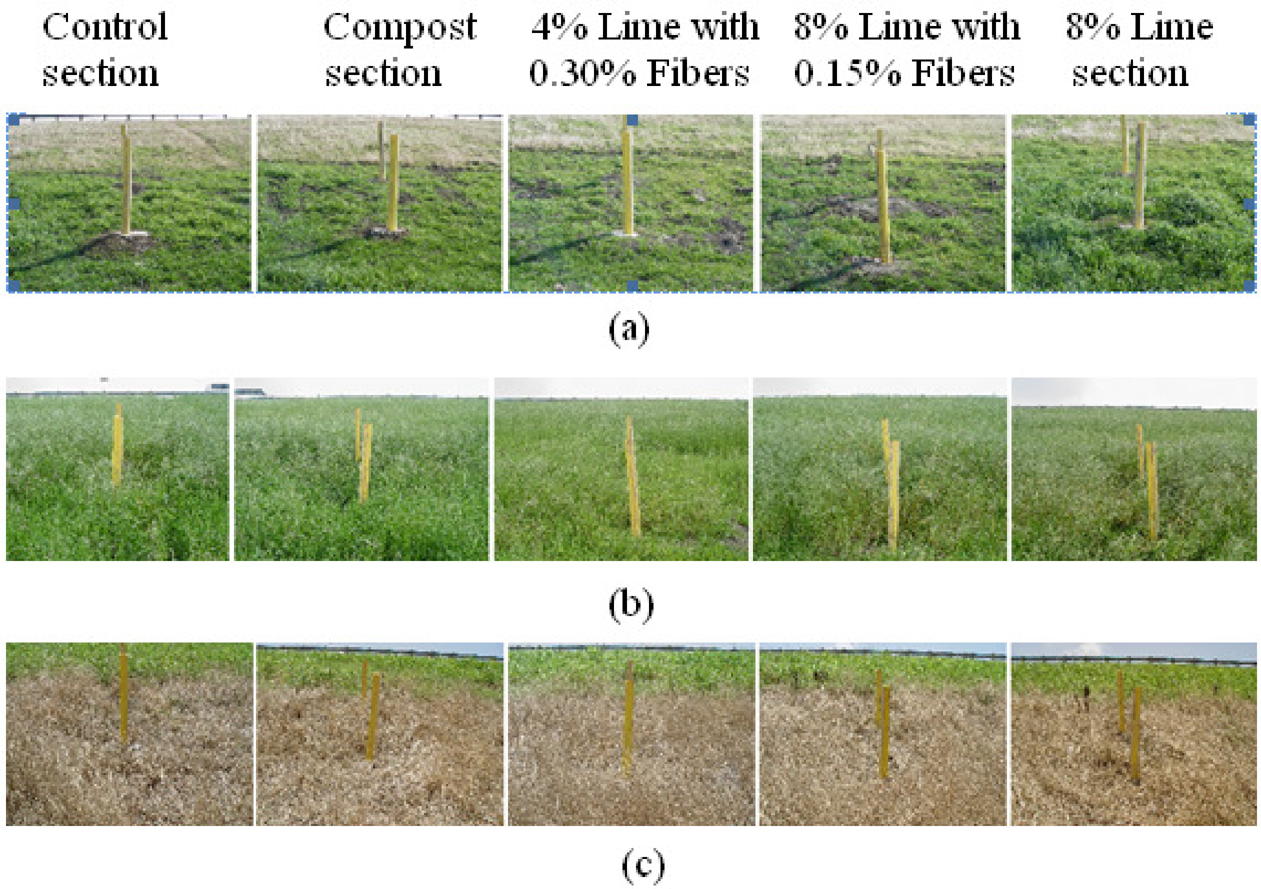


Figure 5.38 Vegetation growth at Joe Pool Dam (a) February 2008 (b) April 2008 (c) June 2008

Based on the vegetation growth the control section, the compost section, the 4% lime with 0.30% fiber section, the 8% lime with 0.15% fiber section and the 8% lime sections are ranked as 4, 3, 5, 2, and 1, respectively.

The vegetation growth diminished during June 2008 due to availability of less moisture content in the top layer of soil. The availability of residual moisture content in the top probe is presented in Table 5.7. Obviously, the residual moisture content in the top layer was less than the moisture content recorded in Table 5.7 at a depth of 25 cm (10 in.) from surface. It can be observed that the vegetation wilted in the new test section location alone and in the rest of the dam portion, the vegetation survived. This is attributed to the fact that the length of root of grass in the test section location was smaller as such it could not absorb moisture available at deeper layers of soil. Figure 5.39 shows the length of roots of grass collected from test site location and from other part of test section on the dam.

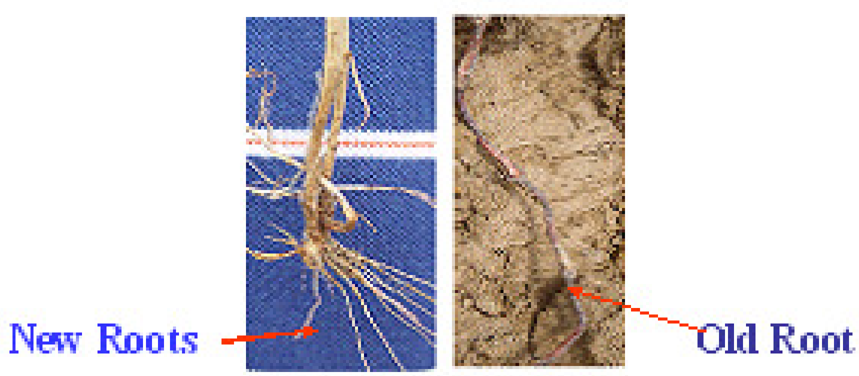


Figure 5.39 Length of root in new test section and existing slope

5.10 Ranking Summary

The performance of test sections at Joe Pool Dam has been evaluated based on the measured volumetric moisture content at the 25 cm (10 in.) and the 50 cm (20 in.) levels from surface of the slope. The performance was also studied based on the elevation surveys, inclinometer surveys1, desiccation cracking and vegetation growth. The ranking of each test section based on various factors considered is summarized in Table 5.24. From the ranking of test sections shown in Table 5.23 the control section, the compost section, the 4% lime with 0.30% fiber section, the 8% lime with 0.15% fiber section and the 8% lime sections are finally ranked as 4, 3, 5, 2, and 1 respectively.

5.11 Summary

Field data was collected and processed to analyze the performance of treated sections with respect to control section at Joe Pool Dam. Data recorded of Moisture Probes placed at the 10 in. (25 cm) and the 20 in. (50 cm) depth from the top surface is analyzed. Similarly, the data recorded by temperature probe is analyzed and presented. Elevation survey was carried out at the test section locations and the result of swell and shrinkage is presented. Inclinometer data, DCP data are presented and analyzed. It could be seen that the overall performance of treated section is ranked in the order as 8% lime with 0.15% fibers, 8% lime, 4% lime with 0.30% fibers and 20% compost and control section.

Table 5.24 Summary of rankings of control and test sections at Joe Pool Dam

Treatment	Residual Moisture Content	Moisture Absorption Top and Bottom Probes	t-test on Moisture Content	Elevation Survey	Inclinometer Surveys	Desiccation Cracking	DCP Results	Vegetation growth
Control Section	4	4	5	5	5	4	5	4
Compost Section	3	5	4	4	4	5	4	3
4% Lime with 0.30% Fibers	5	3	3	3	3	3	3	5
8% Lime with 0.15% Fibers	2	1	1	1	1	1	2	2
8% Lime	1	2	2	2	2	2	1	1

CHAPTER 6

RELATIVE PERFORMANCE OF DAM SECTIONS

6.1 Introduction

In this chapter, the data collected from Grapevine Dam test sections is presented and the engineering performance is relatively compared with those of Joe Pool Dam. The construction of test sections at Grapevine Dam site was not initiated along with the Joe Pool Dam test sites due to the budgetary issues. The construction of test sections at Grapevine Dam was commenced during July 2008 and was completed three weeks after initiation. The data collected during the three months period was only used for analysis in this chapter.

Supplementary studies were planned and conducted on physical models prepared in the laboratory using field construction soil samples of both the dam sites to study the influence of alternate wetting and drying conditions simulated in the laboratory environment.

The model studies have given an insight into the cracking behavior of soil mixed with additives for alternate wetting and drying condition. The influence on swelling behavior is also studied by conducting free swell stain measurement tests with alternate drying and wetting.

6.2 Moisture Sensors Data

The rainfall recorded at Grapevine Dam during the observation period of three months is shown in Figure 6.1. Volumetric moisture content data as recorded by the moisture probes positioned at 25 cm (10 in.) and 50 cm (20 in.) from surface of slope for each of the five test sections is shown in Figures 6.2 to 6.6.

The average volumetric moisture content recorded by top and bottom probes for three months is shown in Figures 6.7 and 6.8. It can be seen from Figure 6.6 that the total rainfall received during the observation period of 3 months is 17.8 cm (7 in.) and the highest rainfall received is 4.85 cm (1.91 in.) during September 2008.

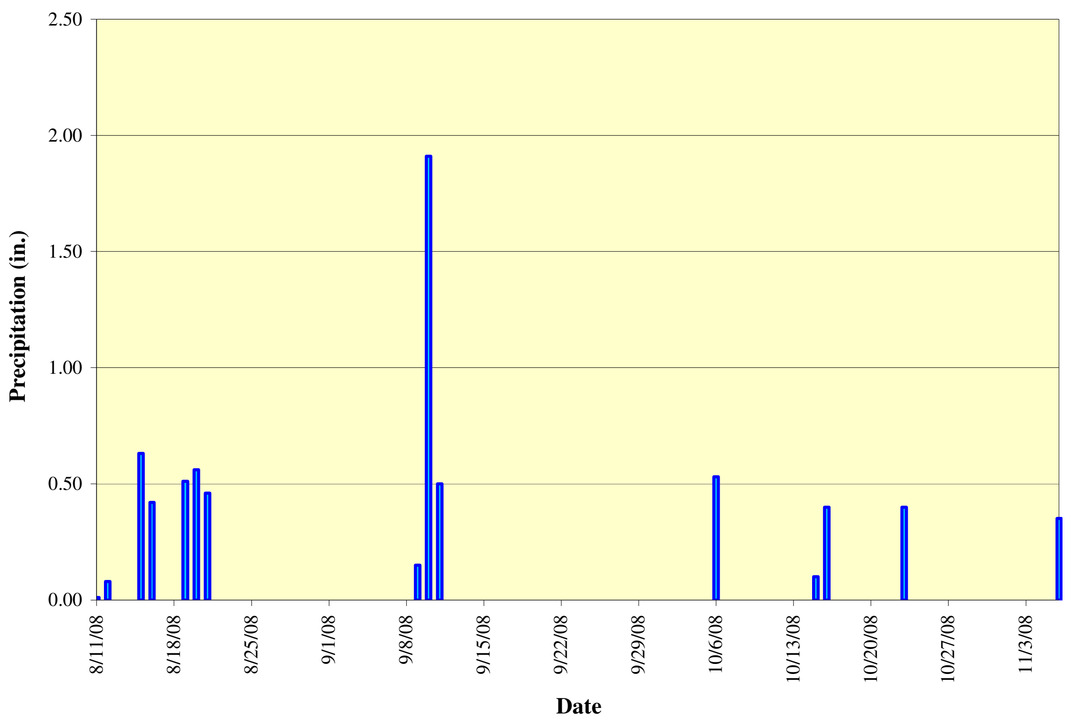


Figure 6.1 Rainfall data between August 2008 and November 2008 (1 in. = 2.5 cm)

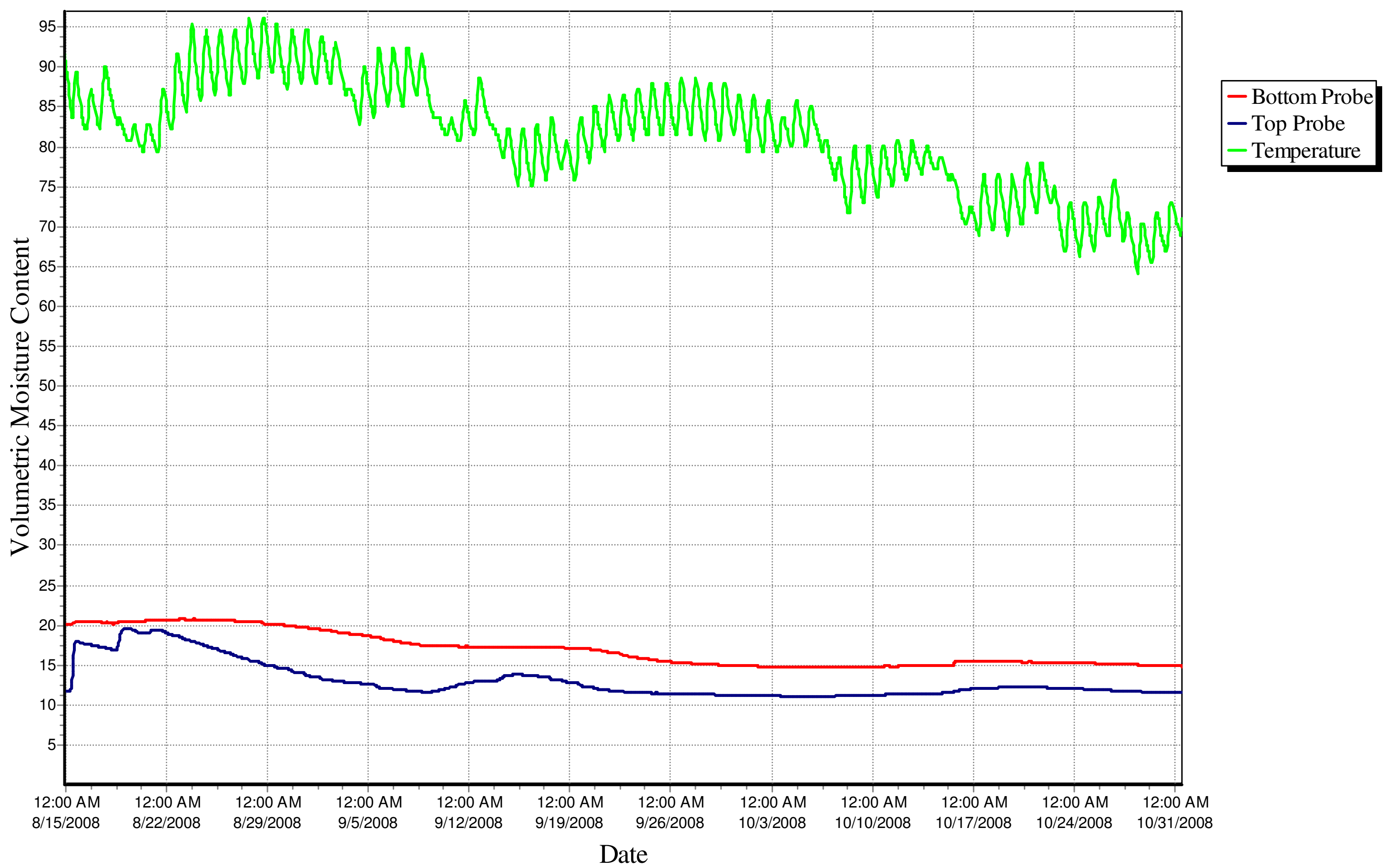


Figure 6.2 Output of moisture sensors and temperature sensor for the control section

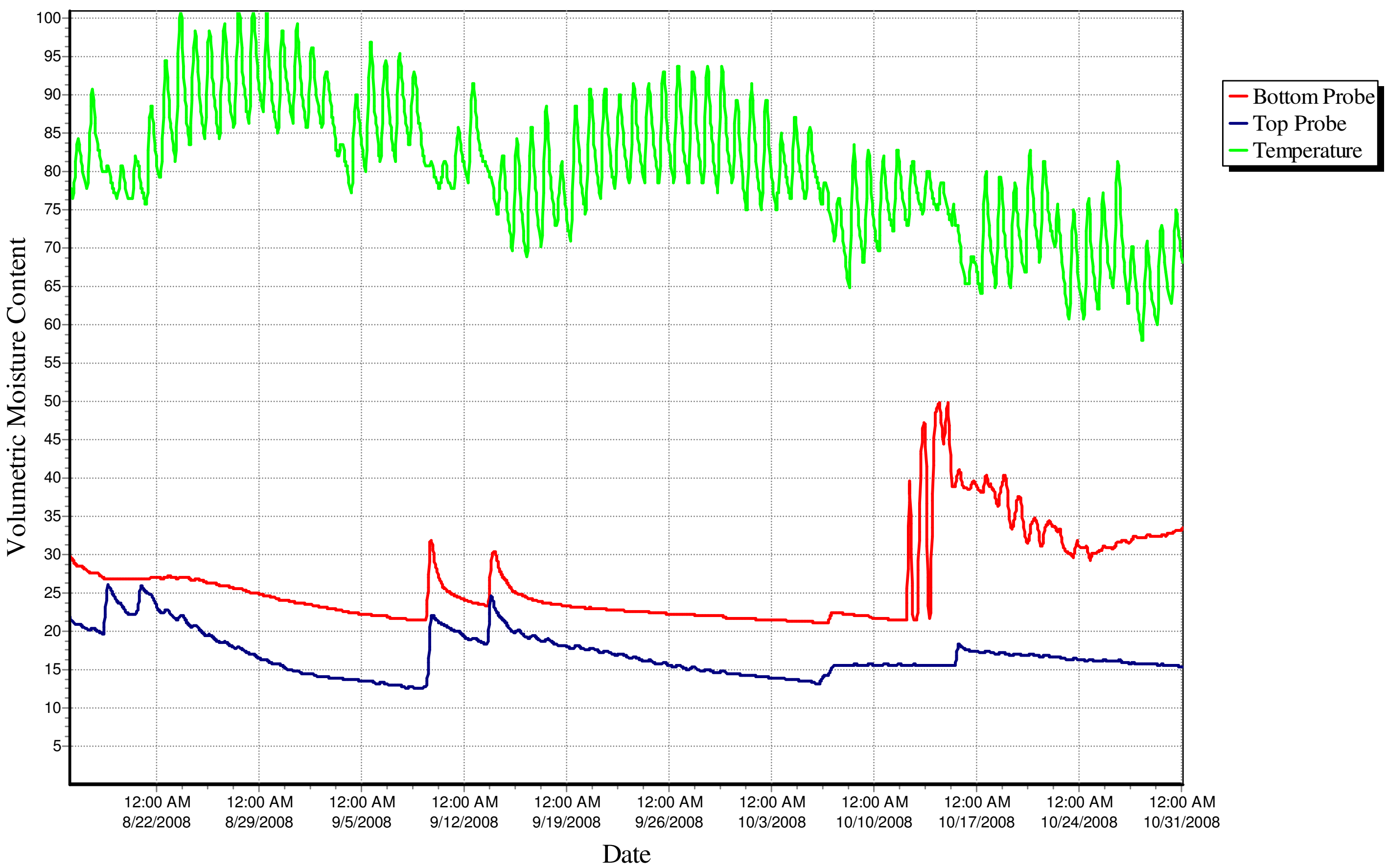


Figure 6.3 Output of moisture sensors and temperature sensor for the compost section

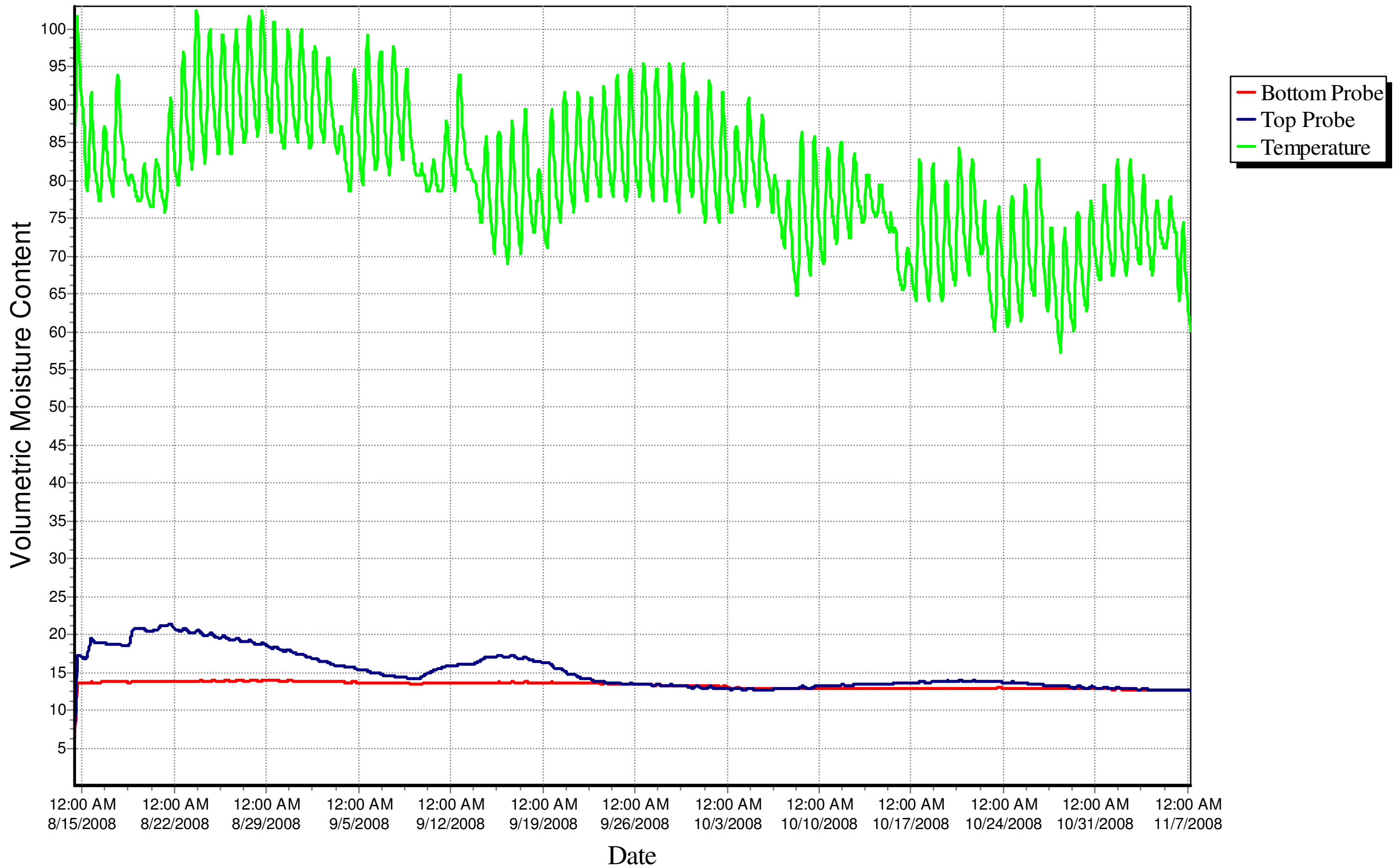


Figure 6.4 Output of moisture sensors and temperature sensor for the 4% lime with 0.30% fibers section

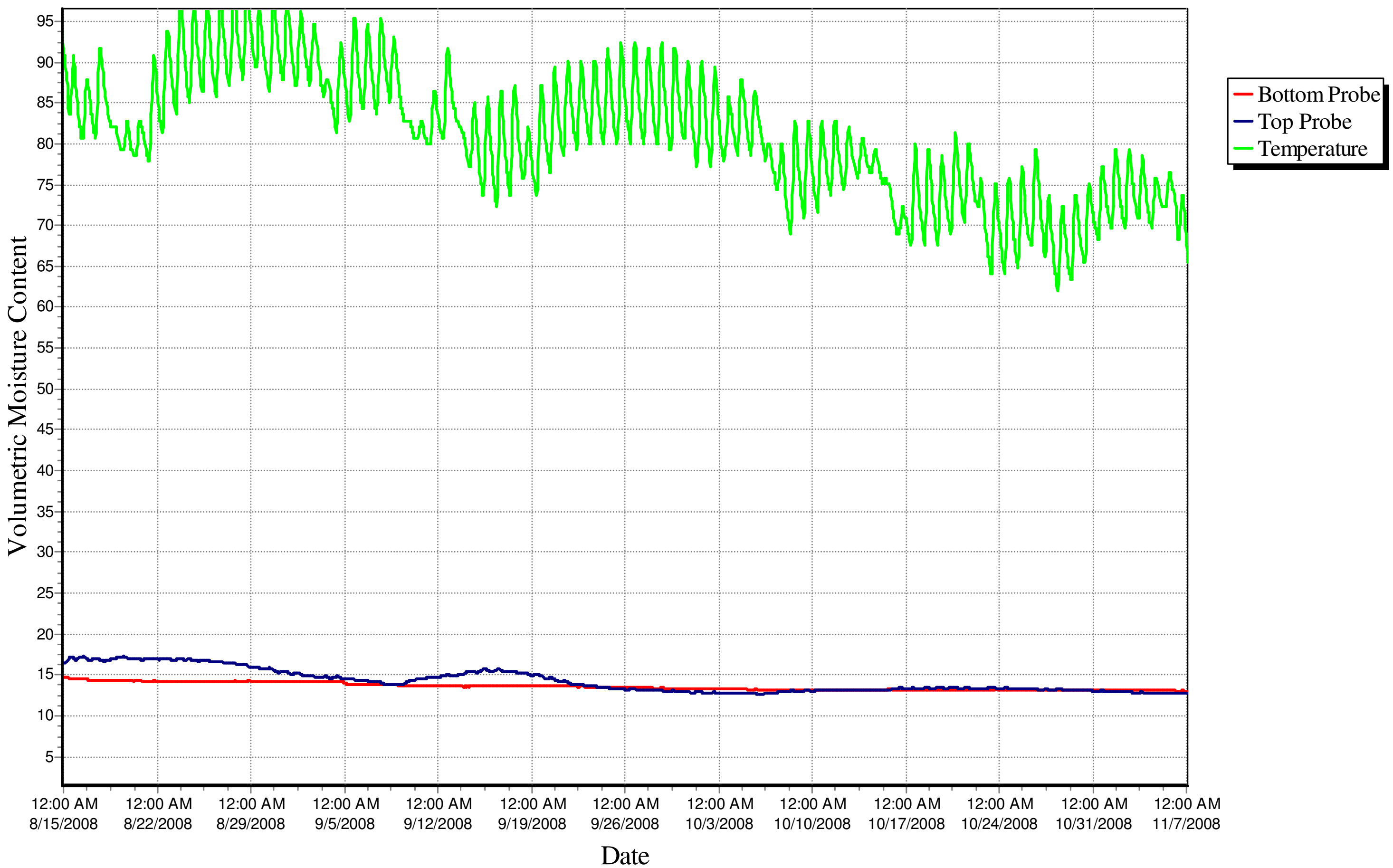


Figure 6.5 Output of moisture sensors and temperature sensor for the 8% lime with 0.15% fibers section

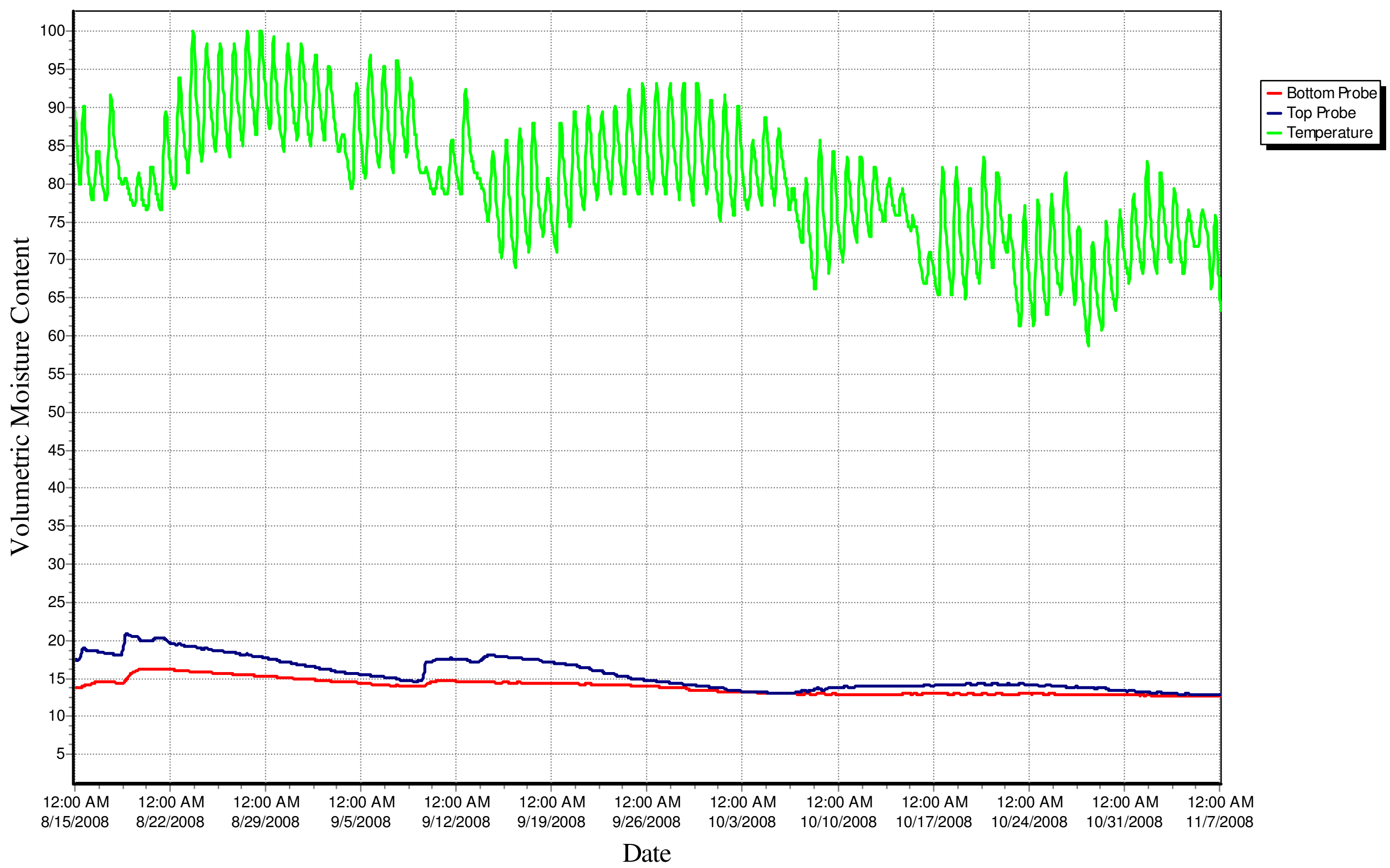


Figure 6.6 Output of moisture sensors and temperature sensor for the 8% lime treated section

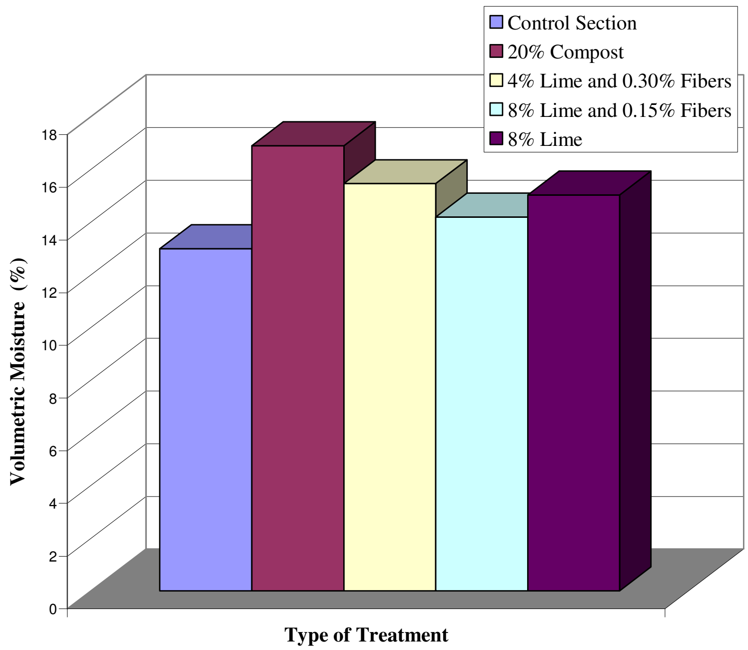


Figure 6.7 Average moisture content of top probe at 25 cm (10 in.) depth

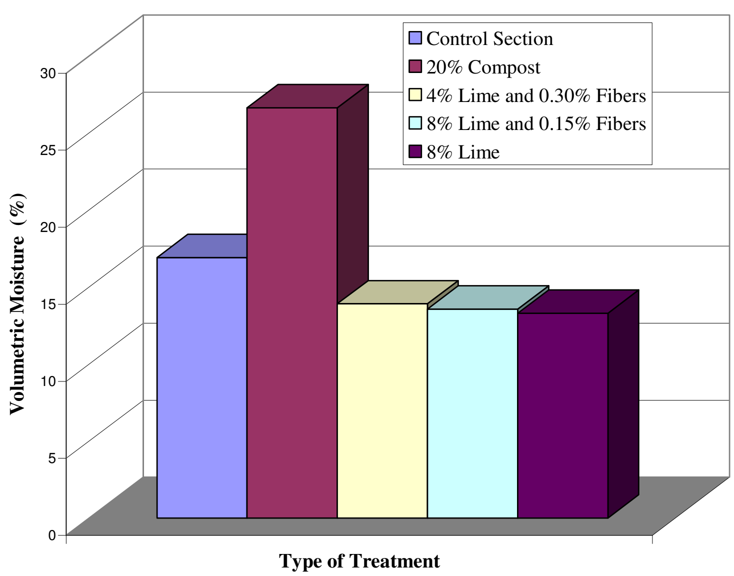


Figure 6.8 Average moisture content of bottom probe at (20 in.) depth

The rainfall caused significant increase in volumetric moisture content of the compost section, as shown in Figure 6.3. In all other sections the response to each rainfall event was not significant. This is attributed to the fact that there were longer spells of dry season with sporadic rainfall events of less intensity.

It is observed that the highest average volumetric moisture content is recorded in both top and bottom probes of the compost section. The average moisture content for the compost section is about 17% and 27% for top probe and bottom probe as against 13% and 17 % for top and bottom probes of the control section. The compost section is holding higher moisture content than the control section.

For the remaining treated sections, the moisture level was measuring about 15 % in the top probe and 13% in bottom probe. This clearly indicates that the section did not receive high rainfall events after construction.

However, the average annual volumetric moisture content recorded at the Joe Pool Dam test section is higher than the three month average volumetric moisture content recorded for the Grapevine Dam as can be seen from Figures 5.9, 5.10, 6.7 and 6.8.

This phenomenon indicates that the Joe Pool Dam soil holds higher moisture content than the Grapevine Dam soil. The hydraulic conductivity of the Grapevine Dam soil is higher than that of the Joe Pool Dam soil, which may result in quick flow through bottom layers leaving lesser moisture content in top layers.

The data needs to be monitored for longer period so that comparison can be made between the annual moisture contents of both the dam sites so as to evaluate effectiveness of treatment.

6.2.1 Gravimetric Moisture Content Observed

Physical soil samples were collected from a depth of 25 cm (10 in.) on October 10th 2008, to compare the gravimetric moisture content present in the soil. The gravimetric moisture content measured is presented in Table 6.1.

Table 6.1 Gravimetric moisture content measured in the field

Treatment	Joe Pool Dam (%)	Grapevine Dam (%)
Control	19	11
20% Compost	17	12
4% Lime with 0.30% Fibers	20	13
8% Lime with 0.15% Fiber	27	13
8% Lime	25	13

The results further show that Joe Pool Dam section is holding higher moisture content than the Grapevine Dam, primarily due to high plasticity properties of the Joe Pool embankment soil.

6.2.2 Other Surveys

Elevation and inclinometer surveys were conducted and no significant movements were observed so far. There is no cracking on the surface except for few minor surficial cracks on control and compost section. It is noted that the data needs to be monitored for additional period to make reasonable conclusions about the relative performance.

6.3 Supplementary Physical Model Studies

Physical models were prepared using the field soil samples obtained from all the test sections. The soil was well compacted in a box of 35 x 25 x 15 cm (14 x 10 x 6 in.) with bottom free drainage arrangement. The soil samples were subjected rigorous alternate wetting and drying cycles. The sample was submerged in the laboratory to create severe wetting conditions. Later, the samples were kept outside the laboratory building exposing them to field atmospheric conditions. This exposure was maintained for an adequate time period to allow for natural drying. Both wetting in laboratory and drying in outside temperatures were carried out for a period of three months.

During drying of the sample, desiccation cracks were appearing on the surface. During the wetting cycle, the cracks were appearing to be closed. During the next drying phase cracks were again resurfacing. The crack pattern at the end of three months period is relatively compared as shown in Figures 6.9 to 6.13.

With the help of scion software, digital image analysis was performed on the cracked surfaces. Figure 6.14 and 6.15 shows the scion image after the threshold process for control soil and compost soil treated sections of both the dam sites.

Table 6.1 shows the ultimate percentage of cracks that occured in the treated sections when they are directly exposed to severe weather conditions.

The difference of crack patterns with respect to the actual field condition is that in the field, the treated sections are encapsulated with top soil.

The cracks observed in the field typically represent the cracks of top soil overlying the treated section.

As such, in judging effectiveness of each treatment, the maximum cracking potential calculated and compared in Table 6.2 based on the physical model studies is more realistic.

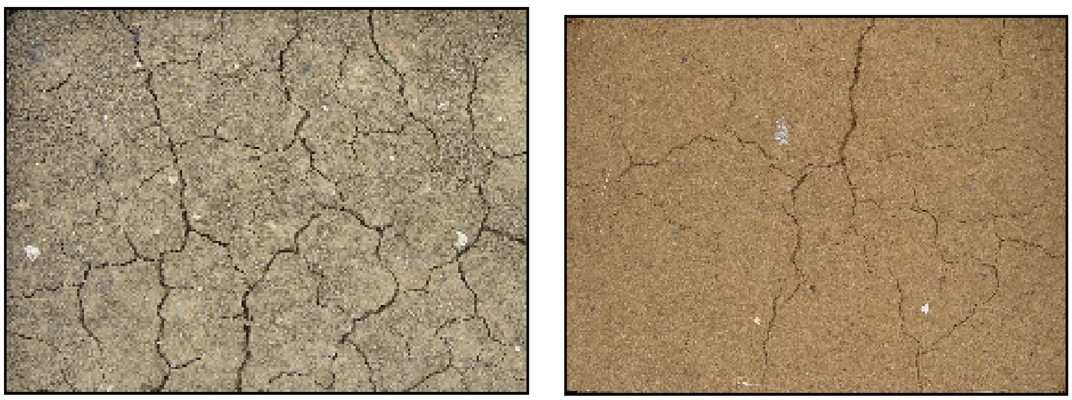


Figure 6.9 Shrinkage cracking pattern of control soil,
Joe Pool Dam vs. Grapevine Dam

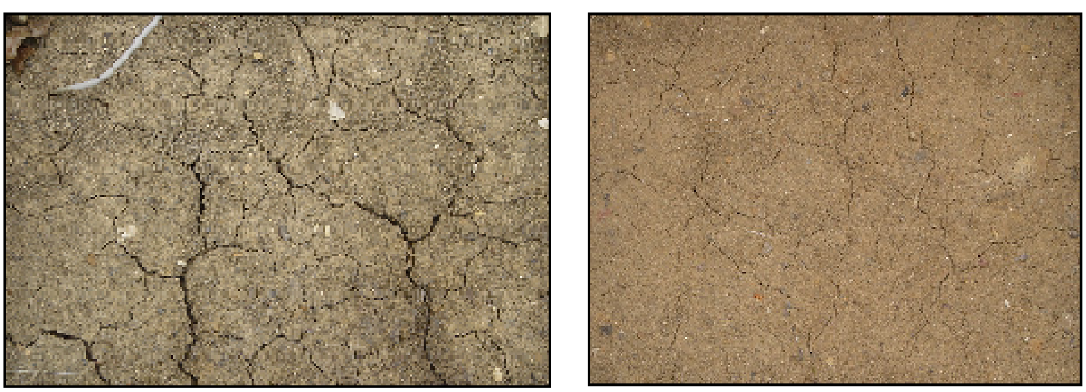


Figure 6.10 Shrinkage cracking pattern of 20% compost treated soil,
Joe Pool Dam vs. Grapevine Dam

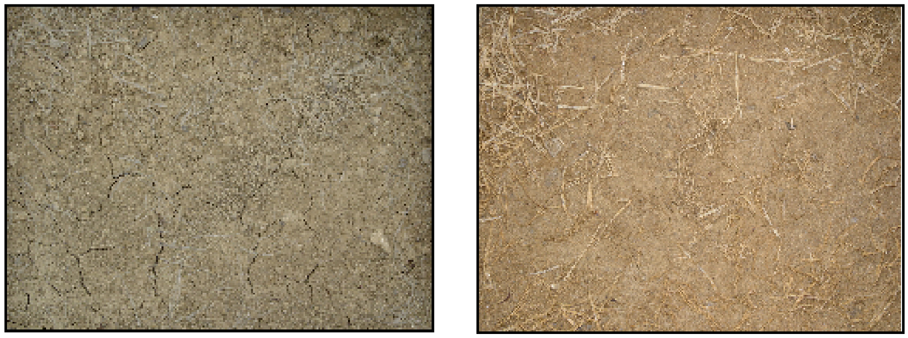


Figure 6.11 Shrinkage cracking pattern of 4% lime with 0.30% fiber treated soil, Joe Pool Dam vs. Grapevine Dam

Figure 6.12 Shrinkage cracking pattern of 8% lime with 0.15% fiber treated soil, Joe Pool Dam vs. Grapevine Dam

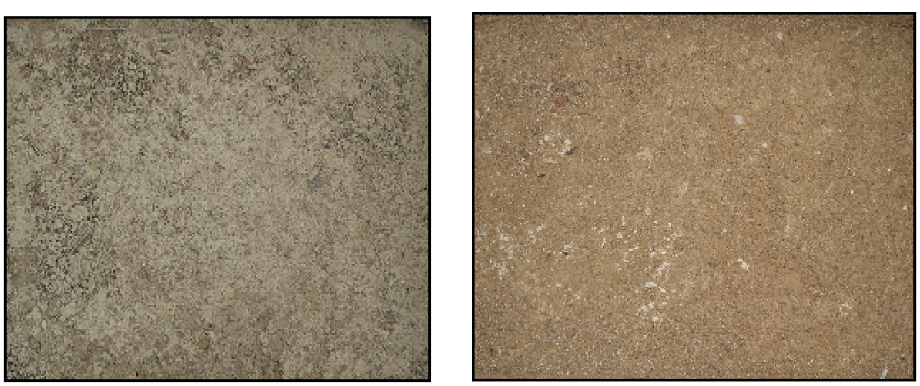


Figure 6.13 Shrinkage cracking pattern of 8% lime treated soil, Joe Pool Dam vs. Grapevine Dam

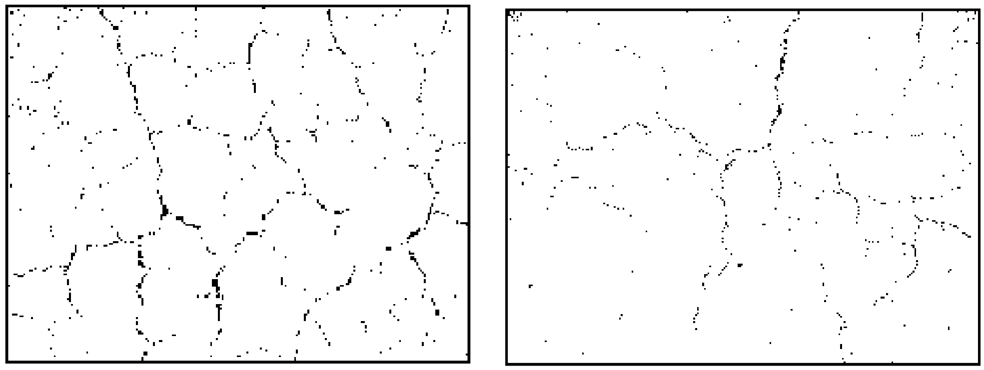


Figure 6.14 Digital image of control soil after threshold,
Joe Pool Dam vs. Grapevine Dam

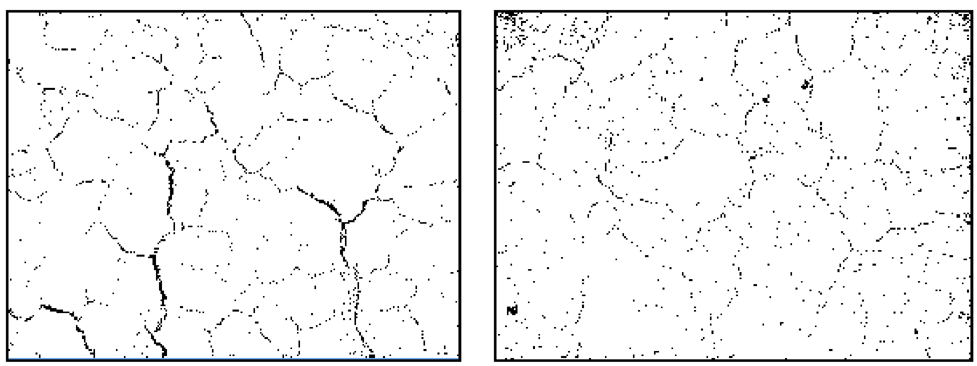


Figure 6.15 Digital image of 20% compost treated soil after threshold,
Joe Pool Dam vs. Grapevine Dam

Table 6.2 Comparison of percent shrinkage crack potential of test sections of Joe Pool Dam and Grapevine Dam

Treatment	Joe Pool Dam (%)	Grapevine Dam (%)
Control	2.2	1
20% Compost	2.9	2.8
4% Lime with 0.30% Fibers	0.4	0.35
8% Lime with 0.15% Fiber	< 0.1	< 0.1
8% Lime	< 0.1	< 0.1

From the above studies, it can be observed that the 8% lime with 0.15 % fibers and the 8% lime sections are more effective than the 4% lime and 0.30 % fiber treated section. Compost section has exhibited highest shrinkage cracks.

Grapevine Dam control soil exhibited lesser shrinkage cracks during drying than the Joe Pool Dam soil primarily due to the reason that the Grapevine Dam soil is sandy lean clay (CL) where as the Joe Pool Dam soil is predominantly a fat clay (CH).

6.4 Swelling Properties with respect to Alternate Wetting and Drying

Supplementary Laboratory Studies were conducted on the field soil samples collected from Grapevine Dam. The normal set up used for carrying out one dimension free swell test is used for the five samples as shown in Figure 6.16.

Figure 6.16 Test set up for vertical swell strain test

Swelling of soil is an important parameter related with surficial failures. Swelling occurs during wet season and it will increase the void ratio, which in turn can increase the moisture content holding capacity of soil and ultimately resulting in the saturation of the soil mass. The mechanism of swelling is more complex than shrinking and depends on various phenomena like elastic rebound of soil grains, the attraction of clay minerals to water, the electric repulsion of clay particles, the expansion of air trapped in the soil voids (Sowers, 1979).

6.4.1 Influence of Density on Swelling Potential

Sowers (1979) stated that the potential swell of clay increases with an increase in compaction density of the soil. Swell tests conducted on control soil samples prepared with different densities at same moisture content have shown an increase of about 4 % swell for the samples compacted with higher density. Laboratory tests conducted on soil samples has confirmed it to some extent. However, the relation of swell vs. compaction density should be better studied in the field rather than in the laboratory.

6.4.2 Influence of Cyclic Wetting and Drying on Swelling Potential

One dimensional swelling test is conducted on the field test section samples of Grapevine Dam.

The sample was allowed to swell for a period of 24 to 72 hours till it reached a stabilized value and the maximum value of swell was recorded. Later, the water was removed from the assembly. The soil sample was allowed to dry under natural conditions and the minimum value of swell was recorded. The test was repeated for 4 cycles and the test results of are presented in Figure 6.17.

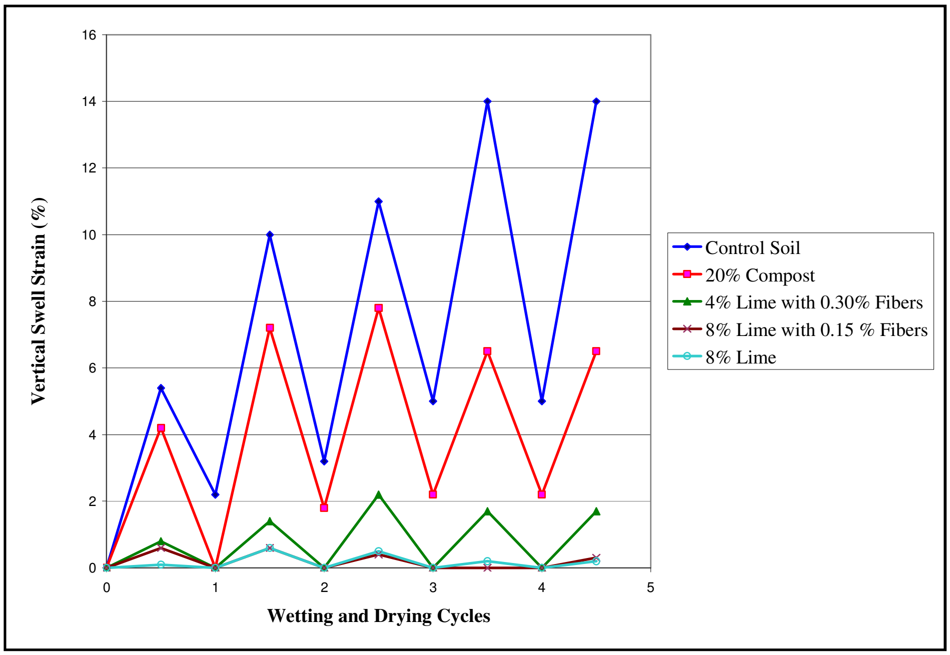


Figure 6.17 Swelling potential of treated sections with wetting and drying cycles

The following observations can be made from the test results:

- The swelling potential has been increasing with an increase in each cycle of wetting and drying for the control soil.
- For the compost treated soil, the swelling strain potentials increased for the first three cycles and thereafter it was slightly decreased.

- With an increase in the number of cycles, there is a plastic deformation in the soil and the minimum swelling observed is increasing and then reached an equilibrium value. The phenomenon was noted in both the control and the compost sections and was not detected in the lime and fiber treated sections.
- From the test results it can be inferred that swell strain potentials of both the lime and lime with fiber treated soils are lesser than that of the control section.

6.5 Summary

In this chapter, the moisture sensor results of the Grapevine Dam is presented and the average moisture content observed for the three month period is compared with the average annual moisture content results of the Joe Pool Dam. The Grapevine Dam sections are holding less average moisture content due to prolonged dry season with intermittent rainfall events. The data needs to be monitored for longer periods for more statistically significant comparisons. The cracking pattern is studied by making physical models in the laboratory and the results are compared for both dam test sections. The Joe Pool Dam control soil is exhibiting higher percentage of cracks due to wetting and drying cycles. The results of swell tests for the treated soil samples of Grapevine Dam are presented for alternate wetting and drying cycles. The lime and fiber treated sections have shown lesser potential for swelling. The discussion of results further strengthens the observations that the 8% lime with 0.15% fiber treated section and 8% lime treated sections are performing better due to cyclic wetting and drying scenarios.

CHAPTER 7

ANALYTICAL MODEL STUDY

7.1 Introduction

Various factors that cause a slope failure are complex and often difficult to describe mathematically (Lu et al. 2003). Conventionally, the limit equilibrium method is used for the slope stability analysis. However, the finite element method is being increasingly used due to many advantages such as study of failure mechanism at critical equilibrium, monitoring of progressive failure resulting from shear strength reduction, possibility of study of different construction procedures or stages, study of influence of additional complicated factors like rainfall (Zheng et al. 2006). There are numerous commercial computer software programs available to analyze the slope stability using the finite element method and the limit equilibrium method.

Dam slopes are usually designed for safety during and at the end of construction, during sudden drawdown and during steady seepage conditions. While performing the stability analysis for safety at the end of construction, the USACE (EM-1110-2-1902 dt. 10/31/2003) recommends use of drained shear strength related to effective stresses for free draining soils and use of undrained strengths related to total stress analysis of undrained soils. Data from Consolidated-Undrained and Unconsolidated-Undrained shear tests are generally used in addressing construction stability analyses. Consolidated-Drained tests are typically used to determine the effective stress shear parameters for steady seepage conditions. However, no specific analysis is carried out to check the safety and stability of slope against surficial failures (McCleskey et al. 2008).

Soil above the phreatic line is in unsaturated condition and soil below the phreatic line is in saturated condition. Usually, matric suction contributes to higher shear strength of soil near surface of slope. However, the desiccation cracks that form during dry season have an adverse affect of accelerated infiltration process resulting in a decrease of soil suction associated with increase of pore pressure (Day, 1996). During an intense rainfall, infiltration of rainwater causes the formation of wetted zone near the slope surface. This phenomenon results in an overall reduction of effective normal stress, thus reducing the shear strength of soil.

Alternate wetting and drying cycles also result in the creep of clay and thereby a reduction of shear strength in soils. Apart from reduction of shear strength, the shear stresses increase when desiccation cracks are filled with infiltrated water. With continued rainfall, seepage develops parallel to the slope (Day, 1996). The rainwater induced moisture results in swelling of clay which results in increase of void ratio and permeability. Under these circumstances, drained and residual shear conditions typically prevail in the desiccation zone parallel to slope.

As the drained cohesion parameter of saturated clay tends to approach zero, the shear strength of a soil decreases and the resisting shear strength is only dependent on drained friction angle.

Due to low overburden stresses for the shallow failure conditions, the shear stresses are higher than the mobilized or available shear strength of the soil. These circumstances hence eventually lead to surficial failures during the prolonged rainfall event (Brand, 1981; Cho et al. 2002; Chen et al. 2004; and Day, 1996).

Considering the above aspects, the dam slopes are modeled using two software programs. First, finite element software, PLAXFLOW was used to model the dam section to study saturation condition for various rainfall events. Then, the GSTABL7 software, which works on the principle of the limit equilibrium method, was used to compute the factors of safety under various saturation conditions against surficial slope failures. The influence of treating the upper 45 cm (18 in.) layer of soil with admixtures is also studied and the results showing a factor of safety of each section against surficial slope failure is compiled and discussed.

7.2 Basic Concepts of Infiltration

Few basic concepts associated with the infiltration process are presented below.

7.2.1 Unsaturated Soil Parameters

Soil mass consists of soil solids and voids. The voids of soil may be filled with water or air. The soil is filled with both water and voids during an unsaturated state. When a soil is completely saturated all the pores are filled with water.

Saturation water content is obviously the quantity of water in the soil when all the pores are filled with water (Marinoschi, 2004).

Various fundamental equations involved are described below:

- Porosity:

$$n = \frac{V_v}{V_b}, \quad V_v = \text{ volume of void}, \quad V_b = \text{ soil bulk volume} \tag{7.1}$$

- Volume of voids:

$$V_v = V_w + V_a \tag{7.2}$$

.

$V_w =$ water volume, $V_a =$ air volume

- Degree of saturation:

$$S_w = \frac{V_w}{V_v} \tag{7.3}$$

- Degree of Saturation

$$S_w = \frac{V_w}{V_v} = \frac{V_w / V_b}{V_v / V_b} = \frac{\theta_w}{n}, \tag{7.4}$$

- Volumetric Moisture content:

$$\theta_w = \frac{V_w}{V_b} \tag{7.5}$$

or

$$\theta_w = n\, S_w \tag{7.6}$$

7.2.2 Equations of Subsurface Flow in PLAXFLOW

The governing equation for the subsurface flow through saturated-unsaturated porous media is given by

$$\frac{\rho}{\rho_o} F \frac{\partial h}{\partial t} + \nabla \cdot [-\mathbf{K} \cdot (\nabla h + \frac{\rho}{\rho_o} \nabla z)] = \frac{\rho^*}{\rho_o} q \quad (7.7)$$

where,

ρ = density of water as a function of salinity and temperature

ρ_0 = reference density of water

ρ^* = density of the source water

q = source per unit volume per unit time

z = elevation head

K = hydraulic conductivity tensor

F = water capacity which depends on compressibility of medium and water, effective moisture content, effective porosity and degree of saturation.

- Water capacity is defined as

$$F = \alpha' \frac{\theta_e}{n_e} + \beta' \theta_e + n_e \frac{d S_w}{d h} \quad (7.8)$$

where

α' = modified compressibility of medium;

β' = compressibility of water;

θ_e = effective moisture content;

n_e = effective porosity;

S_w = degree of saturation.

Darcy's law governs the flow in a porous medium which is given by the equation 7.9. The Darcy velocity or flux (discharge per unit area, q_D) is computed using Darcy's law as:

$$\mathbf{q}_D = -\mathbf{K} \cdot \left(\frac{\rho_o}{\rho} \nabla h + \nabla z\right) \tag{7.9}$$

For the present analysis, the infiltration model requires permeability properties of the soil, which were measured during additional laboratory studies conducted. These results are reported in section 3.4.

7.2.3 Materials Model of PLAXFLOW (Source: PLAXFLOW Manual)

The modeling of unsaturated flow is mostly based on a Van Genuchten material description. The software gives a variety of default models relating relative permeability and saturation besides giving an option to input user defined parameters. The model study includes study of the extent of saturation due different rainfall events.

7.2.4 Boundary Conditions

Dirichlet boundary condition is applied whenever the flow domain is adjacent to a body of open water. By Neumann boundary condition, the normal derivative of pressure is prescribed on the boundary. By the flux boundary condition, the flux (or infiltration) normal to the boundary surface is prescribed. For an impermeable boundary or closed boundary the flux is indicated as zero.

7.3 Slope Modeling Studies using PLAXFLOW

A typical dam section is modeled to study the effect of infiltration in the surficial layers. The soil properties obtained from laboratory studies are used to model the dam. Brief details of the properties used in PLAXFLOW software is given in Table 5.1 based on the experimental results on the field samples as presented in Chapters 3 and 4.

Table 7.1 Properties of Grapevine Dam soil used for Modeling

Properties	Grapevine Dam
< 2 μ	9%
>2 μ and < 50 μ	0%
> 50 μ and < 2mm	91%
Coefficient of Permeability	
Control Soil	8.1 x 10^{-6} cm/sec (3.19 in./sec)
Desiccation Zone (assumed)	8 x 10^{-5} cm/sec (1.13 in./sec)

7.3.1 Material modeling

Initially, the geometry showing the cross-section of dam is drawn as shown in Figure 7.1. The input coordinates of the points are shown in Table 7.2. PLAXFLOW program allows for assigning standard, Hypres, USDA, Staring series of soil properties to the model.

The standard option allows selection of most common soil types like coarse, medium, fine, very fine and organic material. Hypres series is an international classification system and Staring series is based on the soil classification in Netherlands (Reference: PLAXFLOW manual). USDA series is based on the soil classification of the United States Department of Agriculture. The user can use any of the available soil model or choose to input the parameters manually.

Figure 7.2 (a) shows the materials model used in this section and Figure 7.2 (b) shows an example of modeling a rainfall intensity of 5 cm per day (2 in. per day) for 2 days.

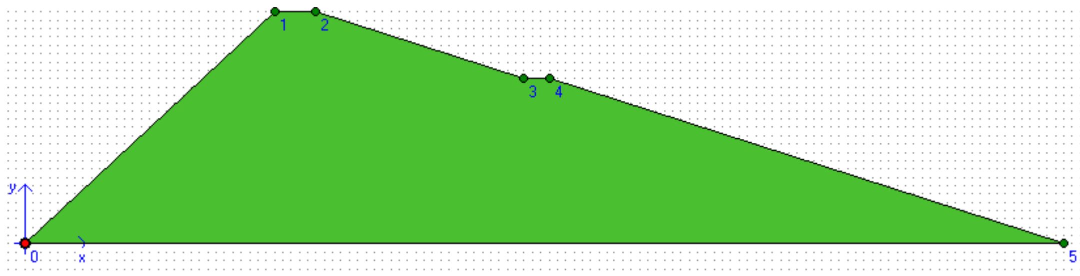


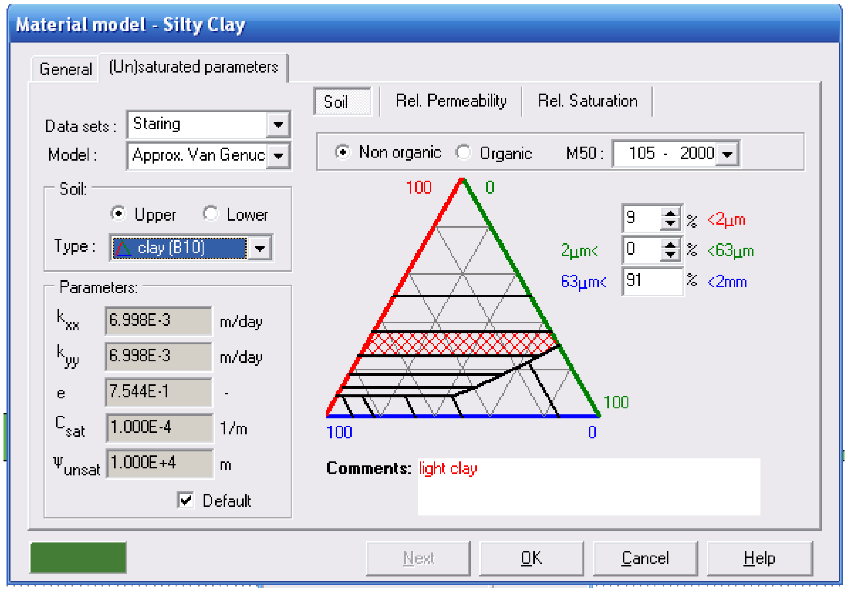
Figure 7.1 Geometry of dam model

Table 7.2 X and Y coordinates of model

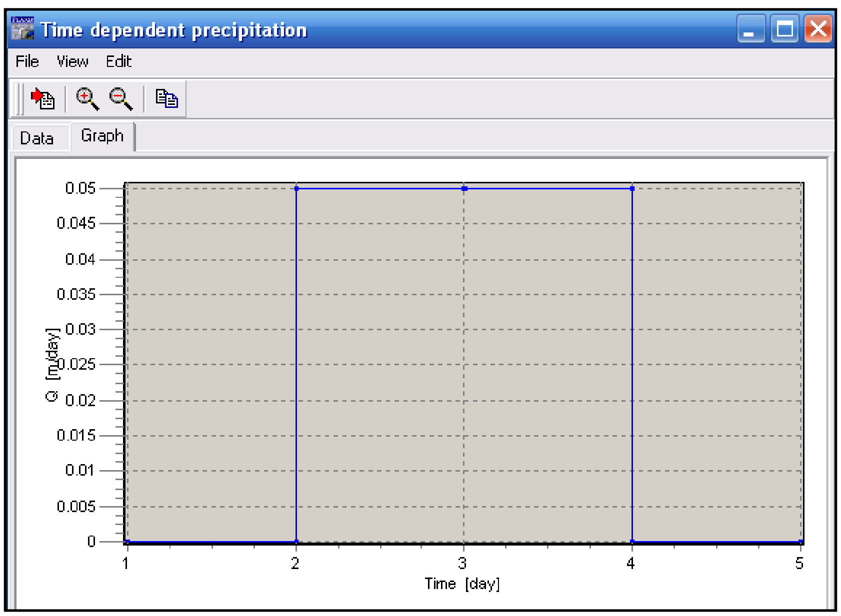
Point No.	X-coordinate m (ft)	Y-coordinate m (ft)
1	30 (98)	28 (92)
2	35 (115)	28 (92)
3	60 (197)	20 (66)
4	63 (207)	20 (66)
5	125 (410)	0 (0)

PLAXFLOW uses "3-node triangular" elements in ground water flow calculations. A 15 node element is selected under plain strain condition. Initial mesh is generated using the mesh generation option. The mesh in the desiccation zone is refined to have more number of elements at the interface as shown in Figure 7.5.

The slope configuration used was analyzed for various infiltration conditions due to different environmental boundary conditions The results are analyzed for various conditions including no rainfall event, normal rainfall event with no desiccation zone, short time high intensity rainfall with desiccation zone present and long time high intensity rainfall with desiccation zone present and these results are presented in the following categories:



(a)



(b)

Figure 7.2 PLAXFLOW (a) Material's model (b) Rainfall data

7.3.2 Case 1 – No Rainfall Event

When there is no rainfall, under normal conditions, seepage takes place because of the reservoir water level alone.

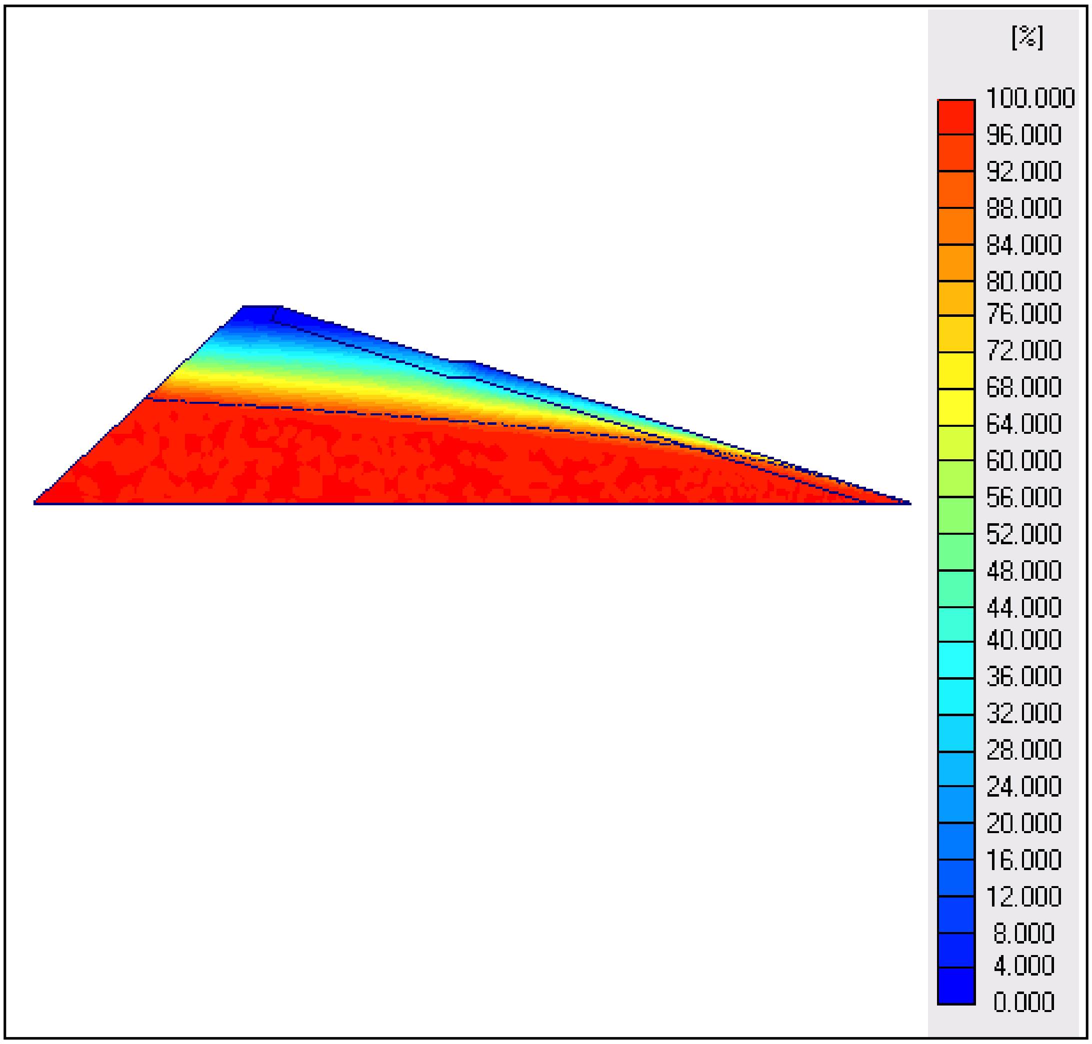


Figure 7.3 Degree of saturation during normal seepage

Figure 7.3 shows the degree of saturation across the dam cross section with the reservoir water level at 15 m (50 ft). As per legend shown, the red color indicates 100% saturation level and the blue color indicates no saturation.

The immediate effect of seepage of reservoir water is to saturate the soil below the phreatic line. Soil above the phreatic line is in unsaturated condition beyond the capillary fringes. FEM Results of Case 1 confirm this observation.

7.3.3 Study with Normal Rainfall Infiltration Event

The increase of wetting front depends on the intensity of rainfall, duration, the permeability of soil, the pore pressure and degree of saturation.

A low intensity rainfall of 2.5 cm (1 in.) for a day caused a moderate increase of wetting front as shown in Figure 7.4.

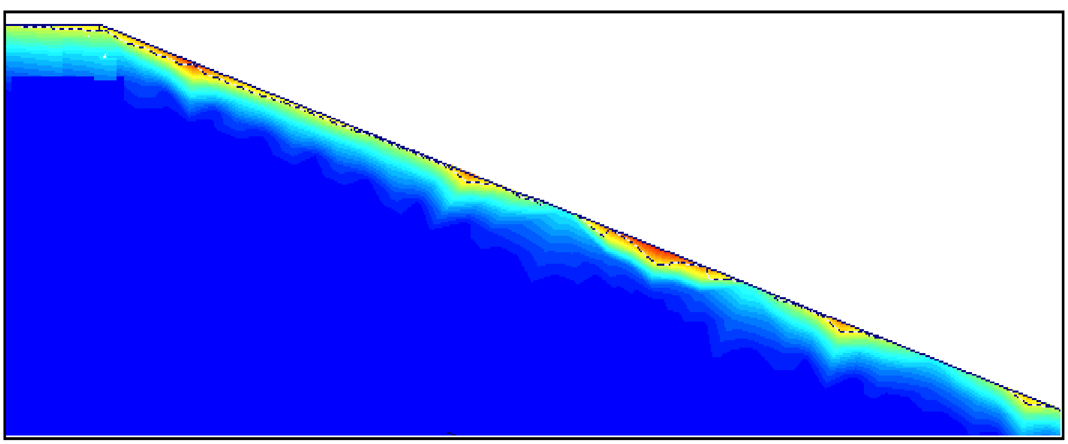


Figure 7.4 Increase of wetting front for low intensity rainfall for 1 day

It is noticed that the soil below the surface is not getting saturated beyond a few centimeters under normal rainfall condition. A cross-section along the middle of slope on the downstream side has shown that the degree of saturation was high near the top surface and started decreasing with depth and the degree of saturation started increasing again nearer to phreatic line. Soil below the phreatic line was completely saturated.

7.3.4 Effect of Desiccation and High Intensity Rainfall Events

Alternate wetting and drying cycles result in desiccation cracking of soils during dry period. During rainfall, the cracks are filled up with rainwater. The presence of moisture content results in swelling of soils and thereby an increase in both the void ratio and permeability.

The effect of alternate wetting and drying result in the formation of desiccation cracking, and the depth of desiccation zone assumed to be around 0.90-1.20 m (3-4 ft) at the surficial soil (Dronamraju et al. 2008).

Hydraulic conductivity of the desiccation zone is higher than the soil below it based on the explanation presented in section 2.7.3. As such, higher coefficient of permeability of 0.248 m/day (0.81 ft/day) is considered for this top 1 m (3.3 ft) layer of desiccation zone as shown in Table 7.1.

In Figure 7.5, the desiccation zone is shown in blue and the soil below the desiccation zone is shown in green which is the dam core soil.

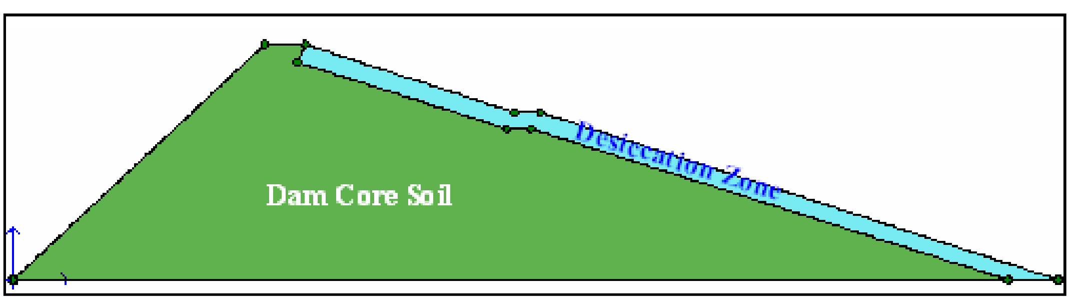


Figure 7.5 Material model of Grapevine Dam with desiccation zone near surface

7.3.4.1 Mesh Generation

Numerical results have shown that at the interface of different types of materials, higher resolution grid is necessary to have reliable solution. As such, in the desiccation zone, a higher resolution grid is selected, which is shown in Figure 7.6.

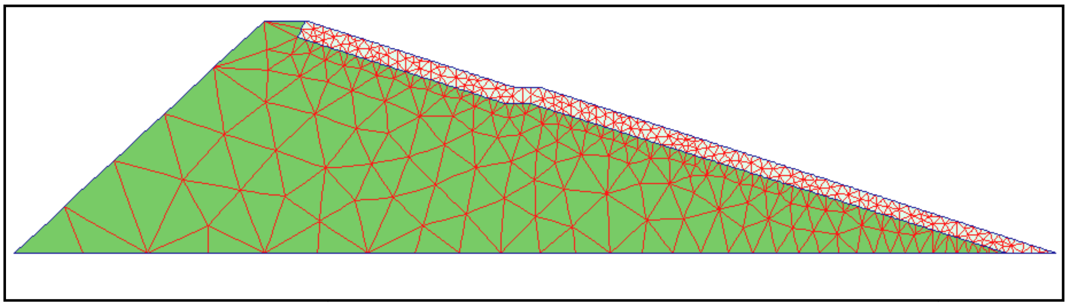


Figure 7.6 Higher resolution grid near interface

7.3.4.2 Influence of Desiccation Zone

Abramson et al. (2002) mentioned that infiltration through an unsaturated zone is vertical and causes no positive pore pressures. If the infiltrating rainfall encounters a material of low permeability, flow will be impeded and a perched water table may form. This results in the increase of saturated zone in the surficial layers.

The input parameters were altered to study the influence of desiccation zone on account of both short time and long time high intensity rainfall events.

7.3.4.3 Case 2 - Desiccation and Short Time High Intensity Rainfall Event

An intensity of 2.5 cm (1 in.) per day for a period of one day caused saturation of part soil near the crest as shown in Figure 7.7. This is termed as Case 2 for the slope stability analyses. It can be seen from Figure 7.7 that the wetting front starts increasing from surface near the crest.

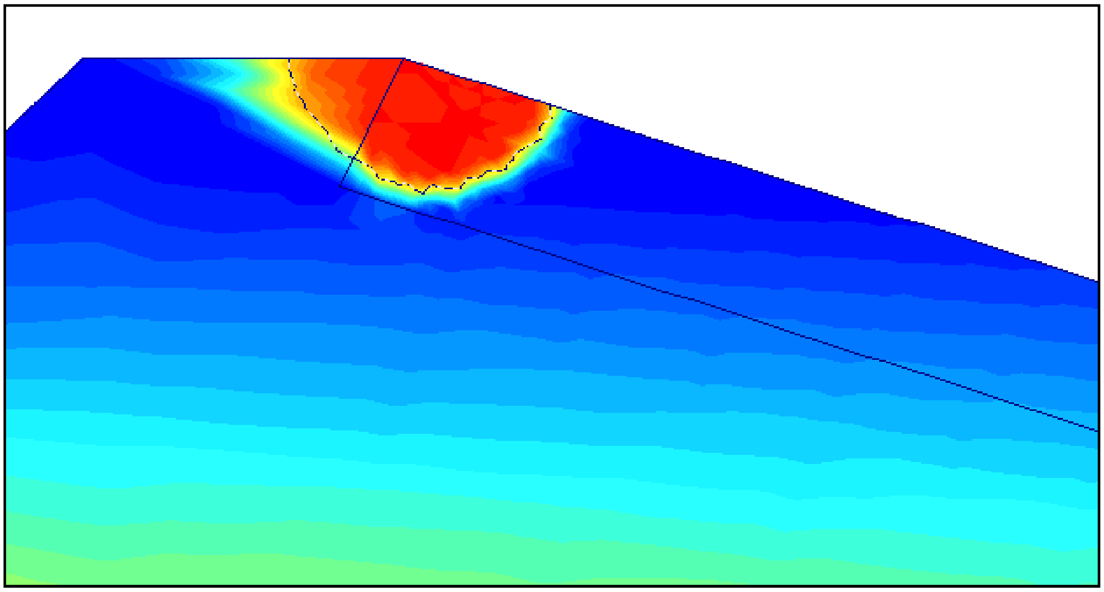


Figure 7.7 Saturation of soil near the crest high intensity rainfall for 1 day (Case 2)

7.3.4.4 Case 2 - Desiccation and Short Time High Intensity Rainfall Event

Further, the rainfall is simulated for a continuous period of 2 days with an intensity of 5 cm (2 in.) per day. The continuous high intensity rainfall event resulted in increase of wetting front in the desiccation zone. as shown in Figure 7.8. This is termed as Case 3 condition for the slope stability analysis.

The saturation of the desiccated zone is apparently due to the reduction in rate of infiltration below the desiccated zone. The desiccated zone is having higher permeability than the soil below it.

When the infiltration reaches the interface soil, flow is impeded and flow starts parallel to slope causing increase in degree of saturation of the soil in the desiccated zone.

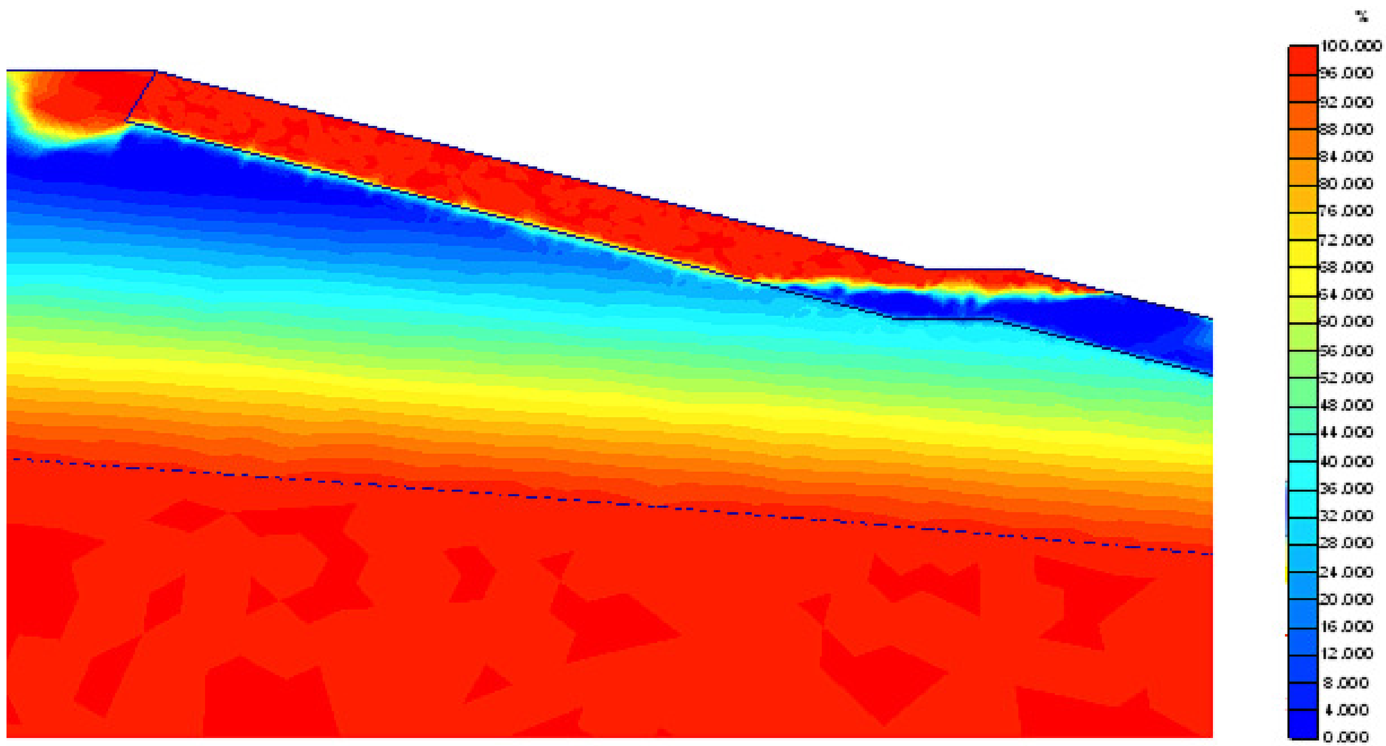


Figure 7.8 Complete saturation of soil for case 3 – desiccation and high intensity rainfall for a long time

7.4 Drained Condition in Saturated Desiccated Zone

It is apparent from the above discussion that rainfall infiltration leads to saturation of desiccated zone along the slope. Alternate wetting and drying cycles also result in creep of clay and thereby a reduction of shear strength. Apart from reduction of shear strength, the shear stresses increase when cracks are filled with infiltrated water. The rainwater induced moisture content results in swelling of clay which results in increase of void ratio and permeability.

Duncan and Wright (2005) explains that drained conditions exist if changes in load is slow enough that the soil reaches a state of equilibrium and no excess pore pressures are caused. Under drained conditions, pore pressures are controlled by hydraulic boundary conditions. The water in the soil may be static or it may be seeping steadily. When drained conditions prevail, there is no decrease or increase in the water within the soil.

Under these circumstances, it is assumed that drained conditions typically prevail in the desiccation zone parallel to slope (Dronamraju et al. 2008).

As such, drained cohesion and drained friction angle determines the shear strength of soil. As the drained cohesion parameter tends to approach zero, the shear strength of a soil decreases and resisting shear strength is only dependent on drained friction angle. The reduction of effective stress due to pore pressure decreases the shear strength of soil further. The reduction in shear strength results in occurrence of surficial failures.

7.5 Residual Shear Strength Conditions

Residual shear strength of a soil represents the lowest possible shear strength that a soil exhibits at large shear strains and this mobilized strength plays a significant role in the stability analyses. A slope does not fail if the applied shear stress is less than the residual shear strength (Gilbert et al. 2005). Residual shear strength of a cohesive soil is applicable to slopes that have already undergone a shear failure (Stark et al. 2005). Once sliding has occurred in a clay, the clay particles become reoriented parallel to the slip surface, and the strength decreases progressively reaching a low residual value (Duncan. 1996).

Slickensides develop in clays due to shear on distinct planes of slip from the realignment of clay particles on the same planes (Meehan et al. 2008). Friction angle on the slickensided surfaces is termed as the residual friction angle (Duncan and Wright, 2005). Skempton showed that once a failure has occurred and a slickensided failure surface has developed, only residual shear strength is available to resist failure.

The drained fully softened shear strength is another parameter used for evaluating first time slope failures (Stark et al. 2005). Strain softening of soil may lead to progressive failures. In case of a progressive failure, the soil particles along the failure surface remain in residual state and they will not attain its peak shear strength value (Duncan and Wright, 2005). The fully softened strength is measured by remolding the clay in the laboratory at a water content equal to the liquid limit of the soil and then measuring its strength in a normally consolidated condition.

Mesri and Shagein (2003) showed that slopes in non-homogeneous stiff clay and clay shale exhibit a residual strength along a portion of slip surface for first time slides. Many studies have further shown that during the first time slope failure, part of the slip surface is in residual condition.

As such, the residual shear strength parameters are considered to be relevant for carrying out surficial slope failure analysis.

7.6 Drained Conditions and Effective Stress Analysis

A slope failure may occur under drained or undrained conditions of soils (Duncan, 2005). Undrained strengths are important for short term loading and drained strengths are vital for long term loading conditions (Duncan et al. 1996).

Considering the mechanism of surficial failures, measurement of residual strength parameters under drained conditions in the desiccation zone is considered to be vital for study of surficial failures.

7.7 Slope Stability Analysis Using GSTABL7 Software Program

Torsion ring shear tests were conducted on the field samples obtained during construction of test sections. The direct shear test and torsion ring shear test results of control and all treated sections were reported in sections 3.3 and 3.4.

The properties of Joe Pool Dam control soil and treated soil sections as shown in Table 7.3 are used for slope stability analysis. The analysis is carried out for all the three cases mentioned in Section 7.3.

For Case 1 study of no rainfall event, strength parameters for all sections are considered from direct shear test results, which are undrained strength parameters. For Case 2 with desiccation and short time high intensity rainfall, the drained residual parameters are considered for soil portion saturated near the crest. For the rest of soil in the embankment, undrained strength parameters from direct shear test results are considered. For Case 3 with desiccation and high intensity rainfall event, the torsion ring shear test results are considered for the control soil and other treated sections in the desiccated zones. Direct shear test result of control soil is adopted for the soil below the desiccation zone. As the analysis was considered for surficial failures which are shallow and local in nature, the effect of saturated zone below phreatic line was ignored. The models considered for all the three cases are described in the following.

Table 7.3 Shear strength parameters of Joe Pool Dam field samples

Treatment	Direct Shear Test		Torsion Ring Shear Test	
	Cohesion kPa (ksf)	Friction angle (degrees)	Cohesion kPa (ksf)	Friction angle (degrees)
Control	80 (1.67)	36	0 (0)	22
Compost	86 (1.80)	40	3.4 (0.07)	20
4% Lime with 0.30% Fibers	57 (1.20)	38	10.5 (0.22)	35
8% Lime with 0.15 % Fibers	62 (1.30)	42	16.3 (0.34)	40
8 % Lime	105 (2.20)	43	12.9 (0.27)	38

7.7.1 Slope Stability Analysis

Slope stability analysis was carried out on the above mentioned three specific cases analyzed in section 7.3 using GSTABL7 software. The geometry of slope and soil parameters were given as input parameters. Bishop slip circle method was used for the analysis. The software also allows to use various other methods of analysis including Janbu method, Spencer method, or to select user's own failure surface.

The location of initiation and termination points is to be specified to carry out the analysis. The number of initiation points, number of slips circles from each point, and length of each segment of slip circle are to be specified.

The program creates a number of slip circles and then calculates the factor of safety for all these slip circles. The ten failure slip circles having the smallest factor of safety values are plotted in the output results. The circle having the least factor of safety is displayed in red.

7.7.1.1 Case 1- No Rainfall Event

Under this study, a dam slope model with a height of 15 m (50 ft) and 2.5H:1V slope is considered as shown in Figure 7.9.

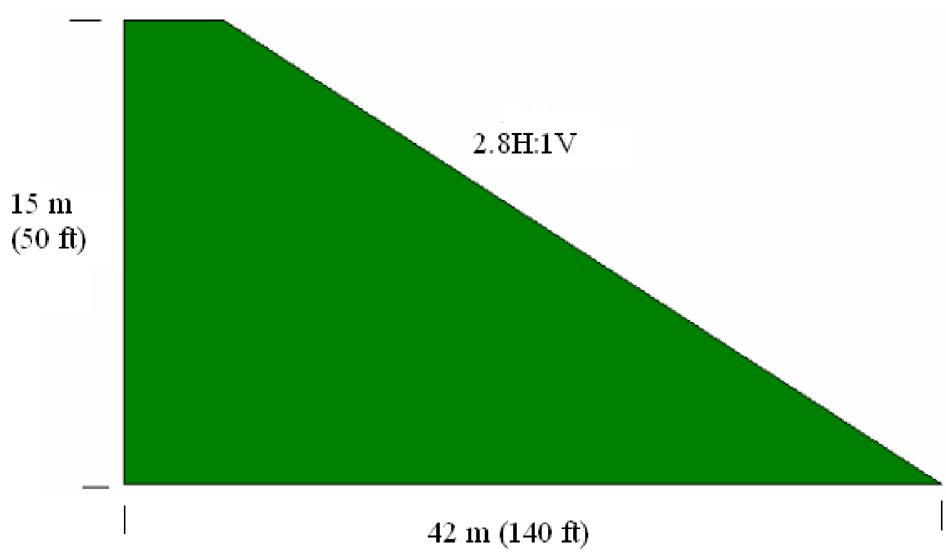


Figure 7.9 Dam slope under normal condition with no rainfall (Case 1)

To study and evaluate the influence of each treatment method, a uniform thickness of 0.50 m (20 in.) treated layer is assumed along the slope surface of the embankment configuration. The factor of safety from the stability studies are presented in Table 7.4 and Figure 7.18.

7.7.1.2 Case 2 – Desiccation and High Intensity Short Time Rainfall Event

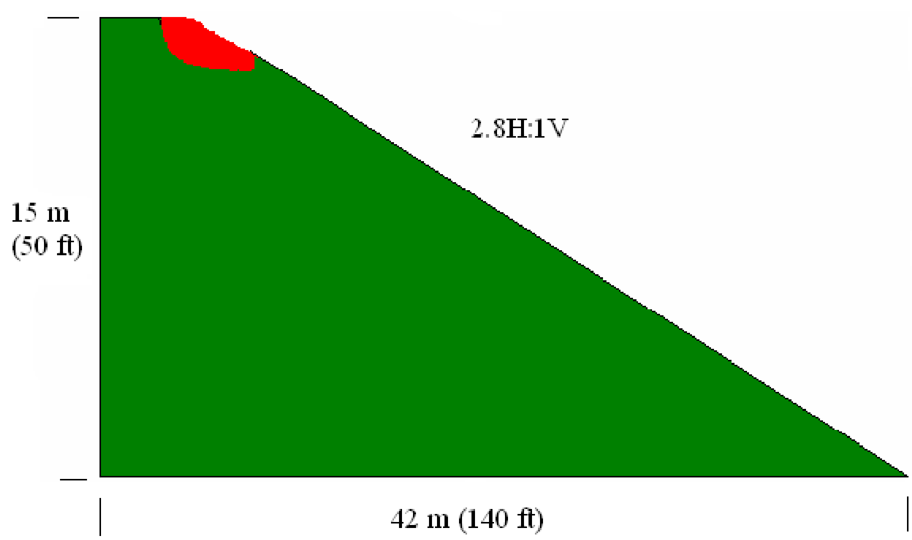


Figure 7.10 Dam slope with partly saturated area near crest (Case 2)

In Case 2, it is assumed that part of the treated section is saturated near the crest of slope as shown in Figure 7.10 in order to simulate the condition observed in section 7.3.4.2 and as depicted in Figure 7.7. For the saturation portion shown in red, the torsion ring shear test results are used and for the remaining portion of soil shown in green, the direct shear test results are used. The factor of safety is shown in Table 7.4 and Figure 7.18.

7.7.1.3 Case 3 – Desiccation and High Intensity Long Time Rainfall Event

Under this study, slope model is considered when the entire length of slope in the top 0.50 m (20 in.) treated layer is saturated completely. Torsion ring shear test results are used in the saturated area shown in red for control and other treated sections. For the remaining portion of soil below the desiccation zone shown in green, the direct shear test results of control soil are used. The factor of safety obtained from the three cases mentioned is tabulated in Table 7.4 and Figure 7.18.

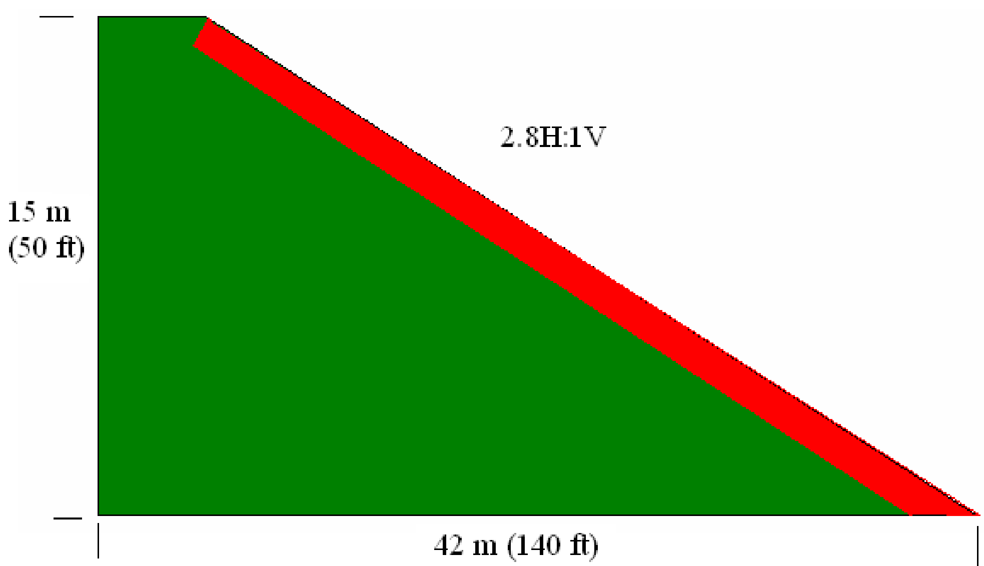


Figure 7.11 Dam slope with desiccation zone completely saturated (Case 3)

Field data analysis presented in Chapter 5 reveals that the 8% lime with 0.15% fiber treated section is the best performing admixture out of the four admixtures studied in the test sections. The results from GSTABL7 software showing the factor of safety for control section and 8% lime with 0.15% fiber treated section for all the three cases is shown in Figures 7.12 to 7.17.

Table 7.4 Factor of safety for different cases

Treatment for a Depth of 50 cm below Surface	Case 1 No Rainfall Event	Case 2- Desiccation and Short Time High Intensity Rainfall Event	Case 3 - Desiccation and Long Time High Intensity Rainfall Event
Control Soil	9.17	1.83	0.88
20% Compost	9.23	2.28	1.57
4% Lime with 0.30% Fibers	8.94	4.68	4.20
8 % Lime with 0.15% Fibers	8.99	6.66	6.21
8 % Lime	9.40	5.89	4.76

It is assumed that the minimum factor of safety required for stability against surficial failure is 1.3. The factor of safety is higher than 1.3 for case 1 for all the soil admixtures when there is no rainfall and no saturated zone is present in the top layer. The factor of safety is higher than 1.3 even in case 2 where the top soil near crest got saturated. However, the factor of safety is 0.9 which is less than 1.3 for control section when the entire desiccation zone is saturated due to the desiccation cracking and high rainfall events.

7.7.2 Discussion of Results from Slope Stability Analysis

7.7.2.1 Case 1- No Rainfall Event

The results from the slope stability analysis indicate that under normal conditions the slope is safe against failure when there is no rainfall. The surface treatment has minor influence on the factor of safety values.

Results from Figure 7.12 indicate that the failure surface having the least factor of safety is of deep seated failure with a factor of safety of 9.17. As such, there is no possibility for any kind of surficial failure in the control section or treated section.

7.7.2.2 Case 2 – Desiccation and High Intensity Short time Rainfall

In actual field conditions, number of surficial failures was observed to be originating from the crest location. The short time rainfall event saturated the soil near the crest location.

The factor of safety of control section is the least followed by 20% compost treated section. The section treated with 8% lime with 0.15% fiber is having the highest factor of safety.

All the sections are having higher factor of safety. However, the control section and the compost section are likely to reach the unsafe levels with the increase of wetting front and saturation of larger areas in the direction parallel to slope surface.

7.7.2.3 Case 3- Desiccation and High Intensity Long time Rainfall

The factor of safety is the least for control section indicating the fact that the slope is vulnerable for surficial failure.

Addition of admixtures to soil has improved the strength and the factor of safety is varying from 1.57 for 20% compost treated section to 6.21 for the 8% lime with 0.15% fibers.

Based on the factor of safety obtained for case 3, the control section, the 20% compost section, the 4% lime with 0.30% fibers section, the 8% lime treated with 0.15% fiber section and the 8% lime section are ranked as 5, 4, 3, 1 and 2 respectively.

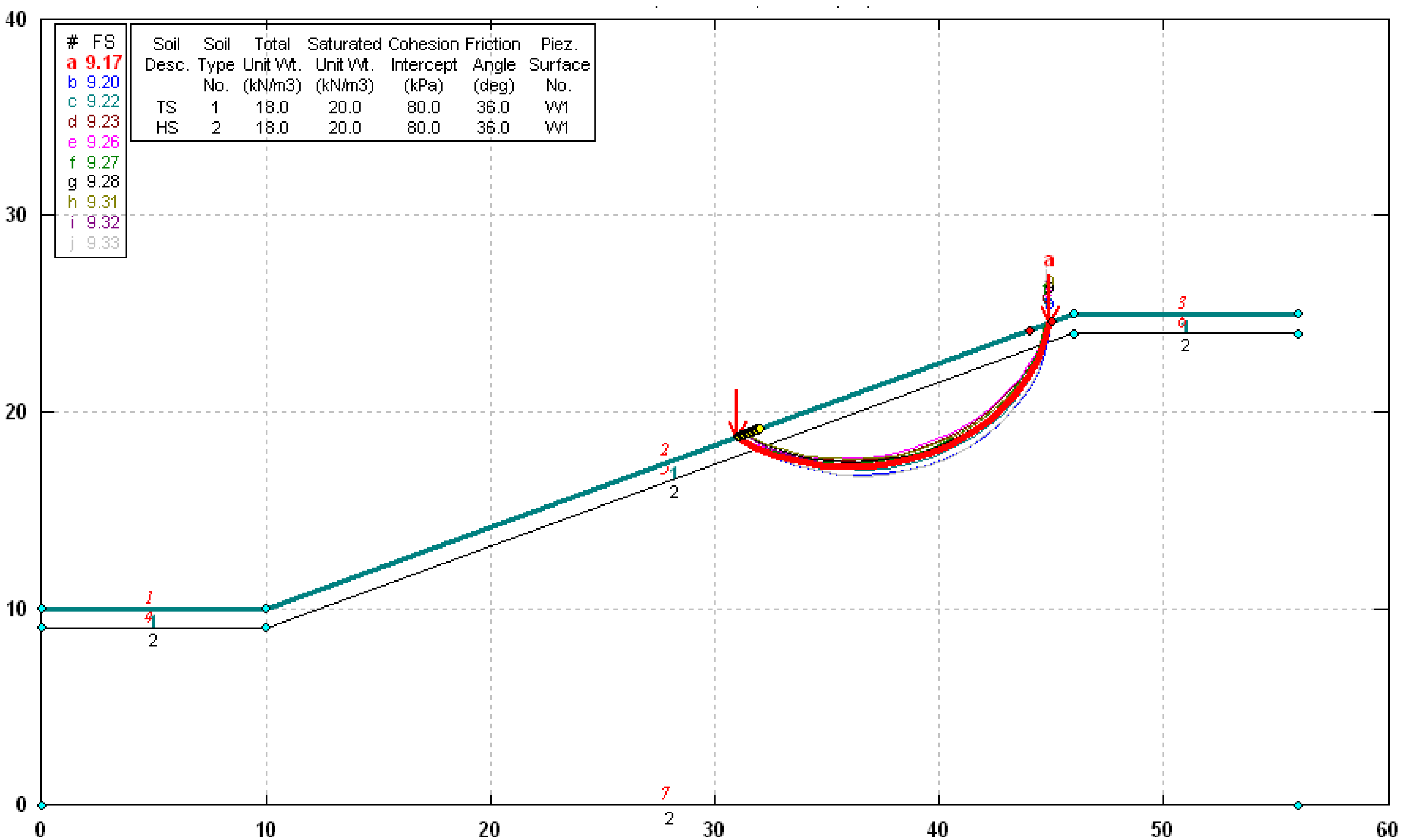


Figure 7.12 Factor of safety for the control section of case 1- no rainfall

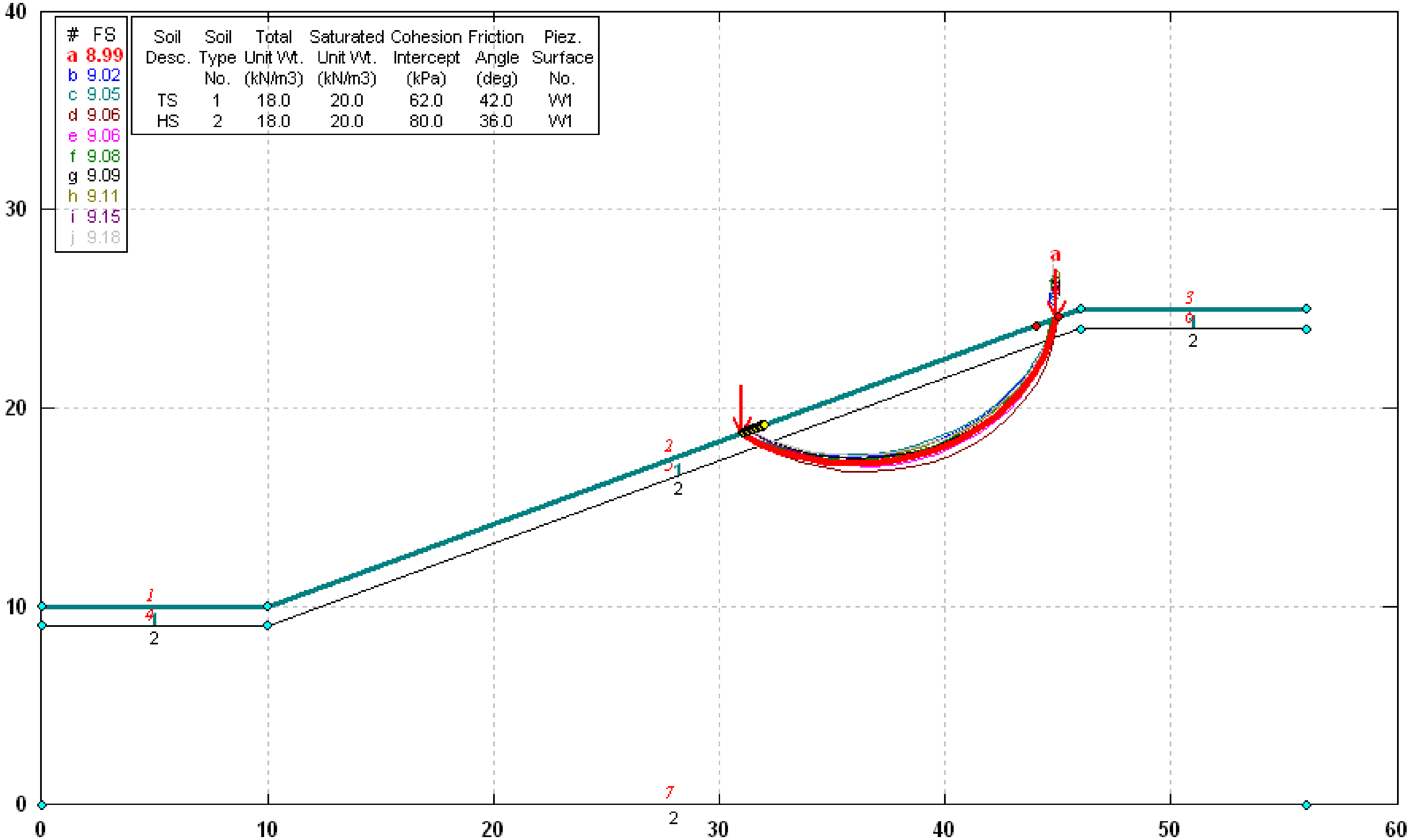


Figure 7.13 Factor of safety for the 8% lime with 0.15% fiber treated section of case 1- no rainfall

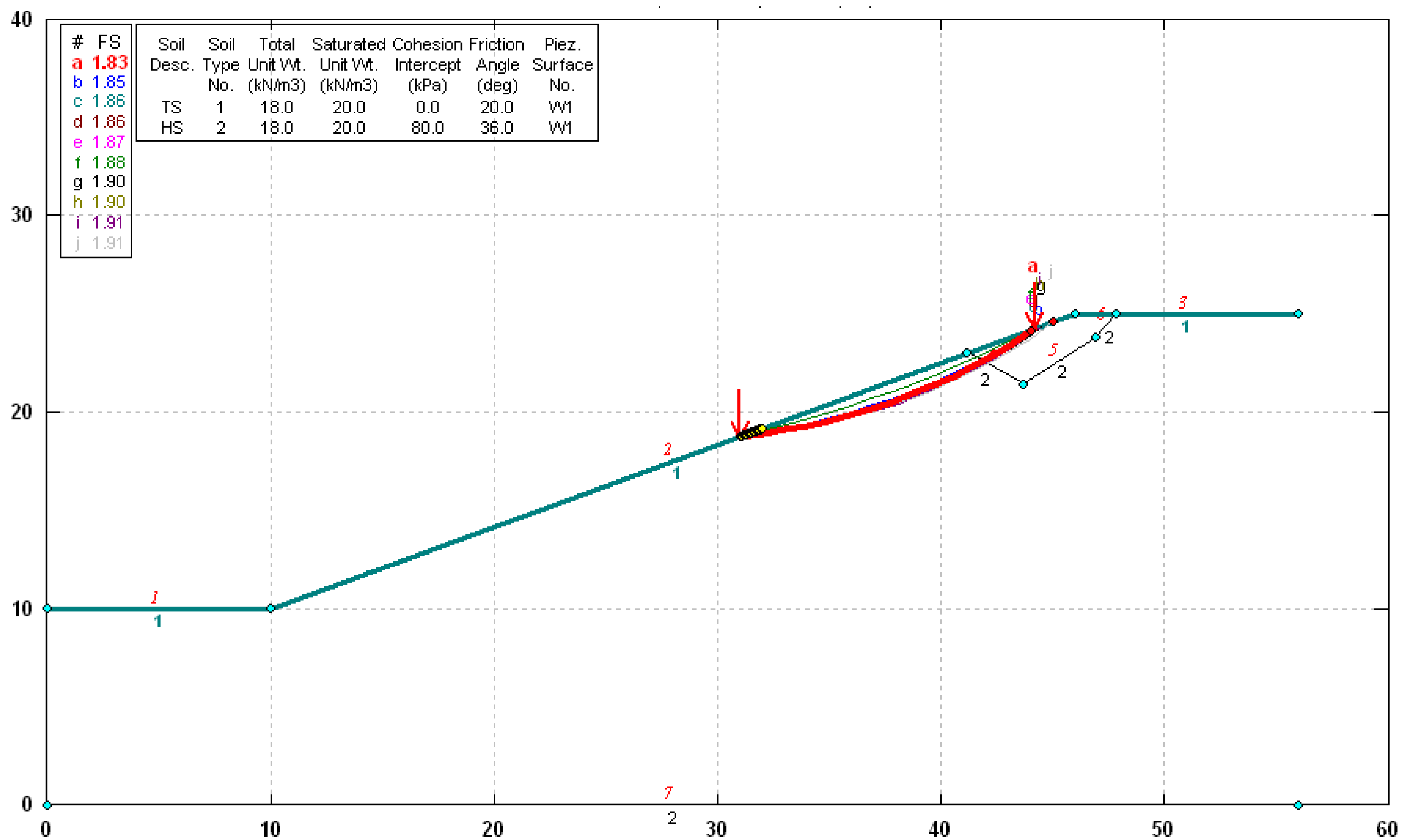


Figure 7.14 Factor of safety for the control section of case 2- high intensity short time rainfall

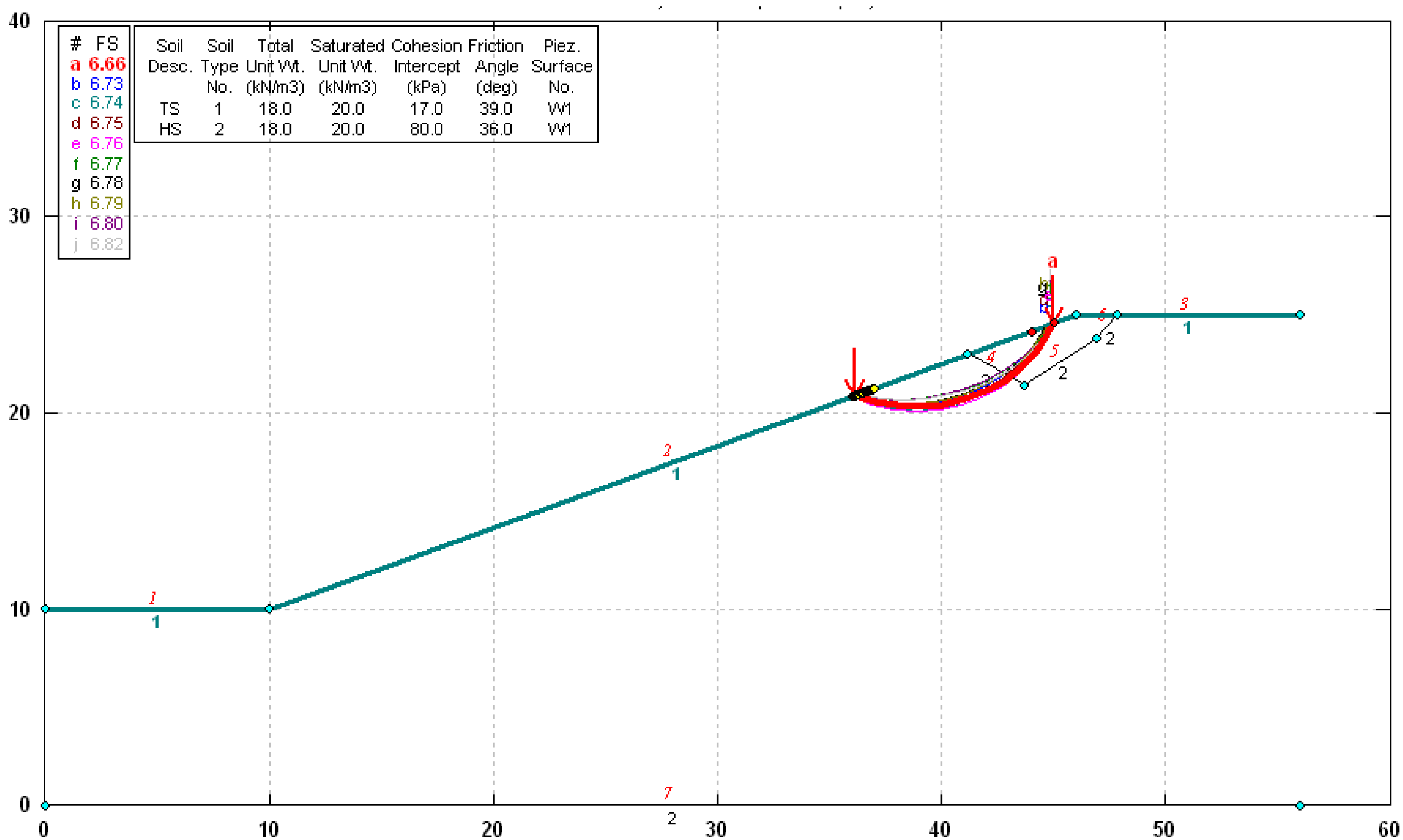


Figure 7.15 Factor of safety for the 8% lime with 0.15% fiber treated section of case 2- high intensity short time rainfall

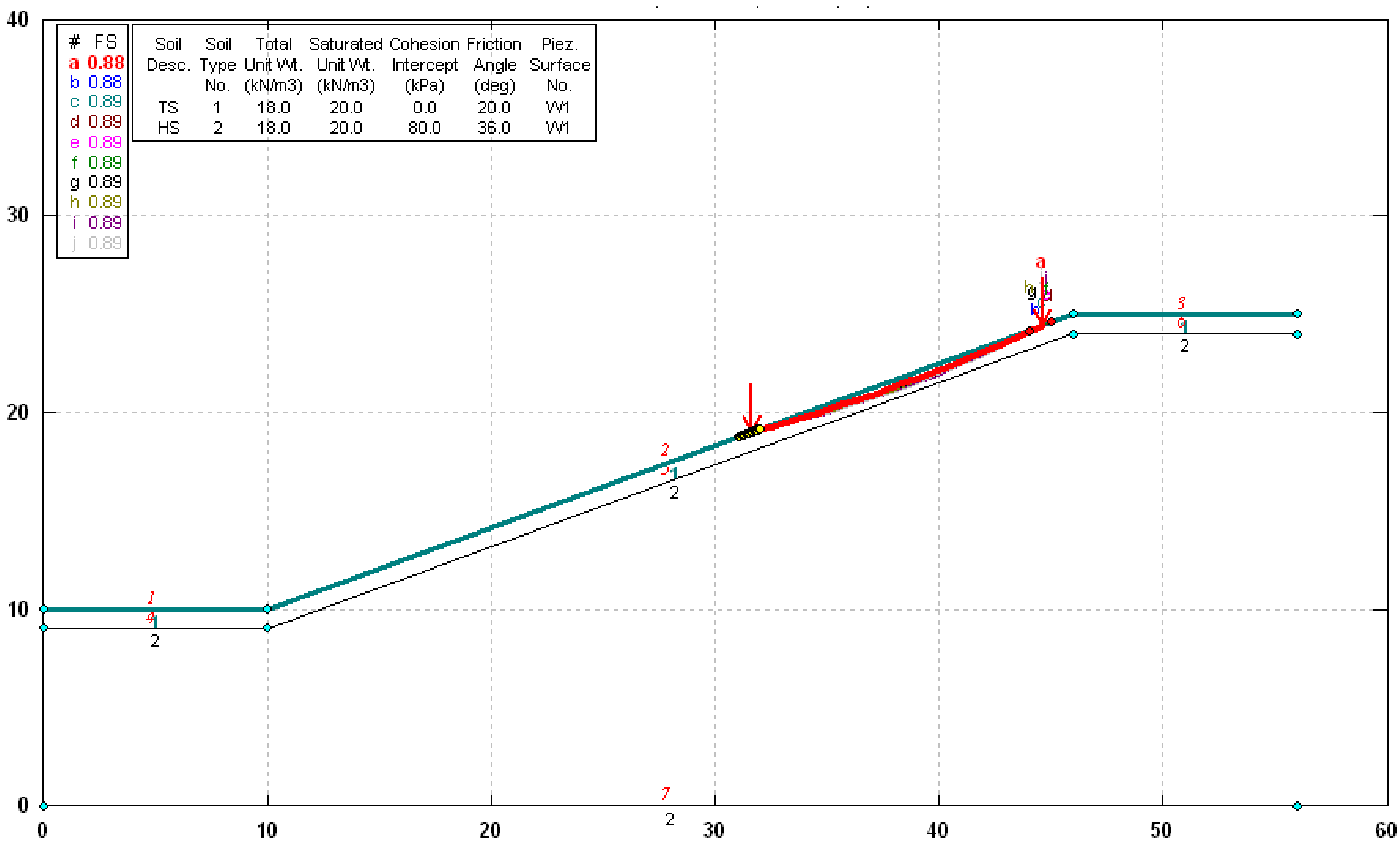


Figure 7.16 Factor of safety for the control section of case 3- desiccation and high intensity long time rainfall

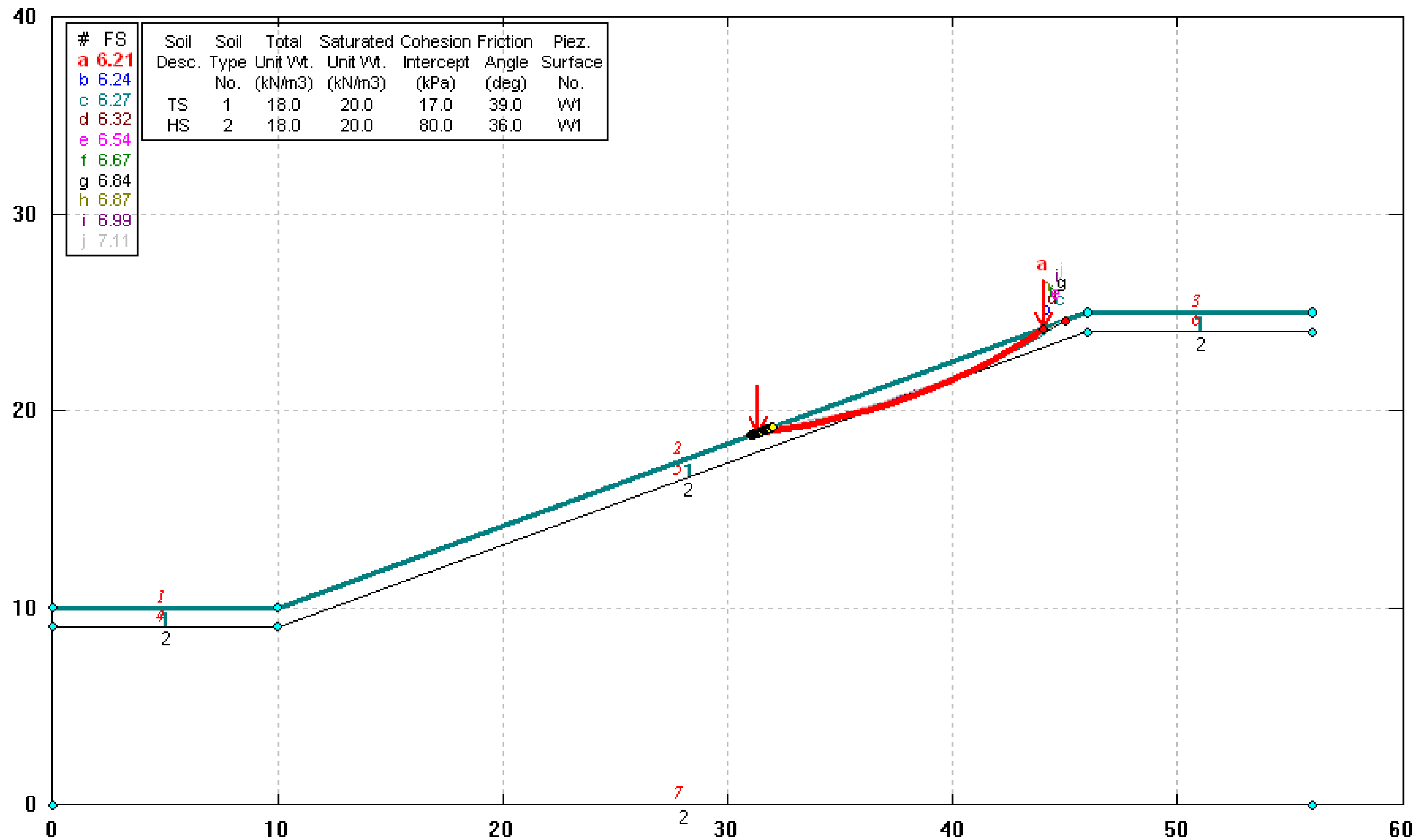


Figure 7.17 Factor of safety for the 8% lime with 0.15% fiber treated section of case 3- desiccation and high intensity long time rainfall

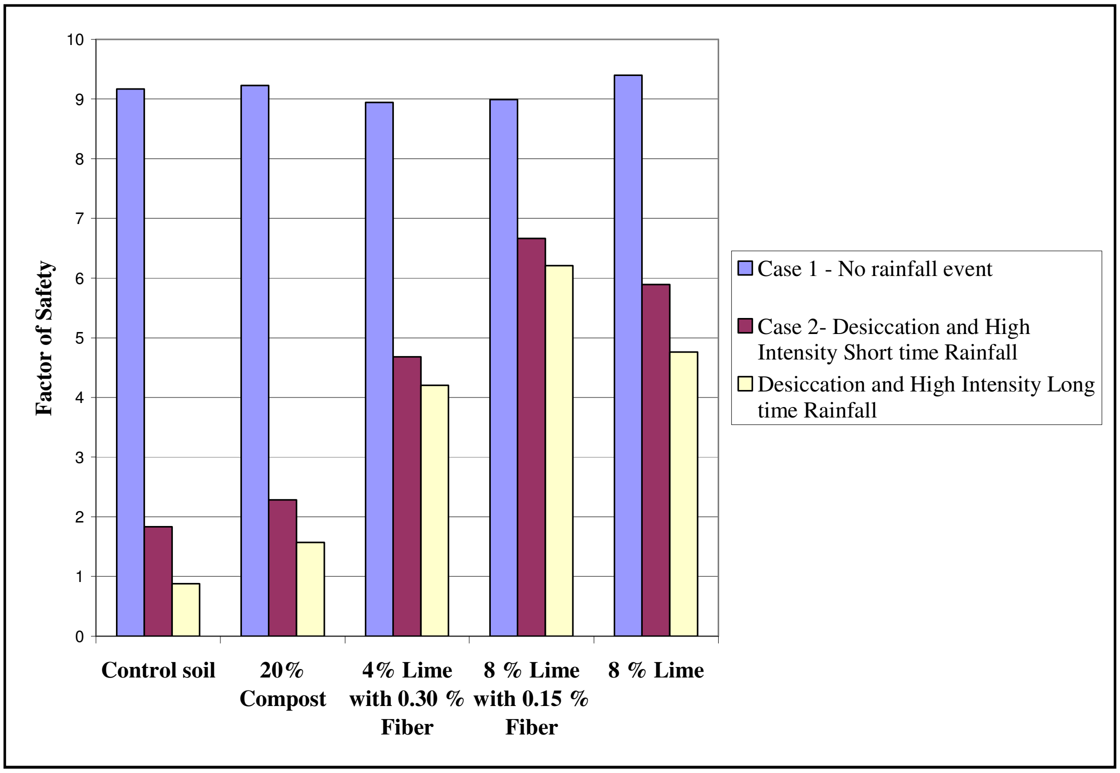


Figure 7.18 Factors of safety for 3 case studies

7.8 Validation of Modeling Results

It is observed from the above analysis that the single major cause for surficial failures is the high intensity rainfall preceded by desiccation cracking. An insight into the failures observed indicate that almost all the surficial failures were preceded by various intensities of rainfall.

7.8.1 Influence of Antecedent Rainfall

McCleskey et al. (2008) reported from available records that various failures occurred at the Grapevine Dam. The details of rainfall events preceding each failure are studied and presented in Table 7.5. It can be seen from Table 7.5 that antecedent rainfall is the primary reason to trigger most of the surficial failures as observed at Grapevine Dam.

Table 7.5 History of surficial slope failures at Grapevine Dam

Date of slide	Slide Width × Length (m)	Rainfall observations during the month (and preceding month where necessary)
26 Feb 1965	30 x 12	157 mm- Feb
05 Jun 1970	38 x 14	16 mm - June, 92 mm- May
09 Feb 1973	24 x 5	49 mm-Feb, 83 mm-Jan
23 Apr 1973	23 x 11	154 mm-Apr,
03 Apr 1974	60 x 21	64 mm-Apr, 58 mm-Mar
10 Apr 1974	15 x 11	64 mm-Apr, 58 mm-Mar
--Jun 1976	18 x 21	36 mm-June, 153 mm-May
-- Jun 1976	27 x 21	36 mm-June, 153 mm-May
17 Jan 1977	45 x 20	61 mm-Jan
17 Jan 1977	15 x 15	61mm-Jan
07 Feb 1977	42 x 15	43 mm-Feb
-- Jun 1977	46 x 15	17.5 mm-June, 25 mm-May
27 Oct 1981	16 x 21	360 mm-Oct
27 Oct 1981	16 x 18	360 mm-Oct
10 Jan 1982	46 x 21	59 mm-Jan,4 mm-Dec81
19 May 1982	70 x 18	347 mm-May
19 May 1982	32 x 18	347 mm-May
09 July 1982	33 x 21	69 mm-July, 109 mm-July
13 Mar 1989	30 x --	95 mm-Mar, 94 mm-Feb
04 Nov 2004	45 x 23	127 mm-Nov, 145 mm-Oct

The rainfall data was collected from the data base of USACE. The rainfall data is also presented for three specific cases of failure showing the antecedent rainfall particulars, as shown in Figures 7.19 to 7.21. Figure 7.19 indicates that a longer duration rainfall during the month of October 1981 triggered a surficial failure on October 27th. It can be inferred from Figures 7.20 and 7.21 that high intensity rainfall events preceded surficial failures.

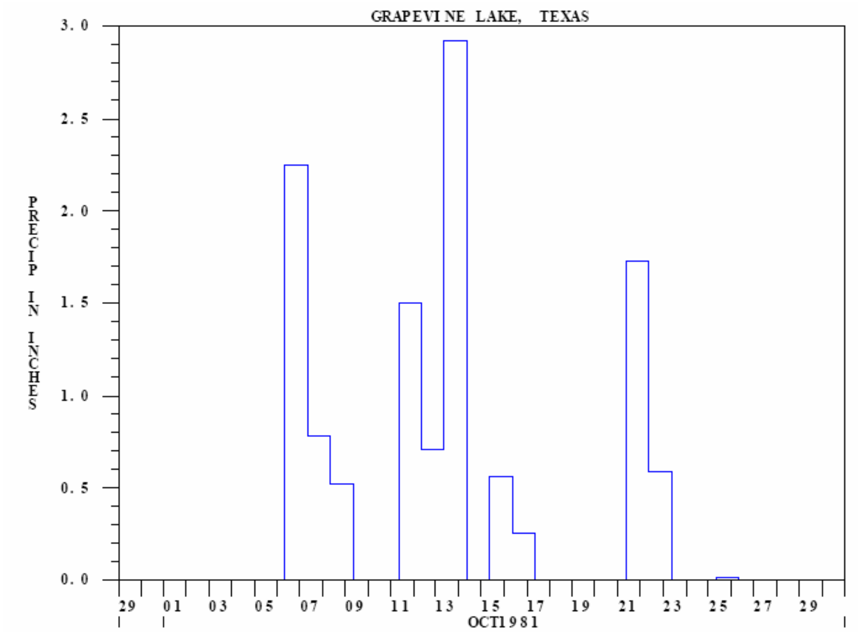


Figure 7.19 Rainfall data for October 1981 (1 in. = 2.54 cm)

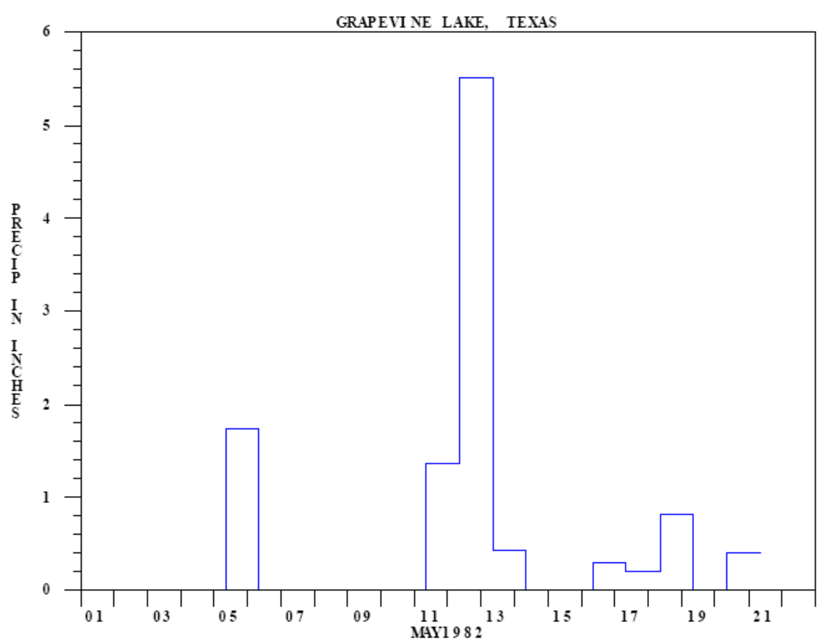


Figure 7.20 Rainfall data for May 1982 (1 in. = 2.54 cm)

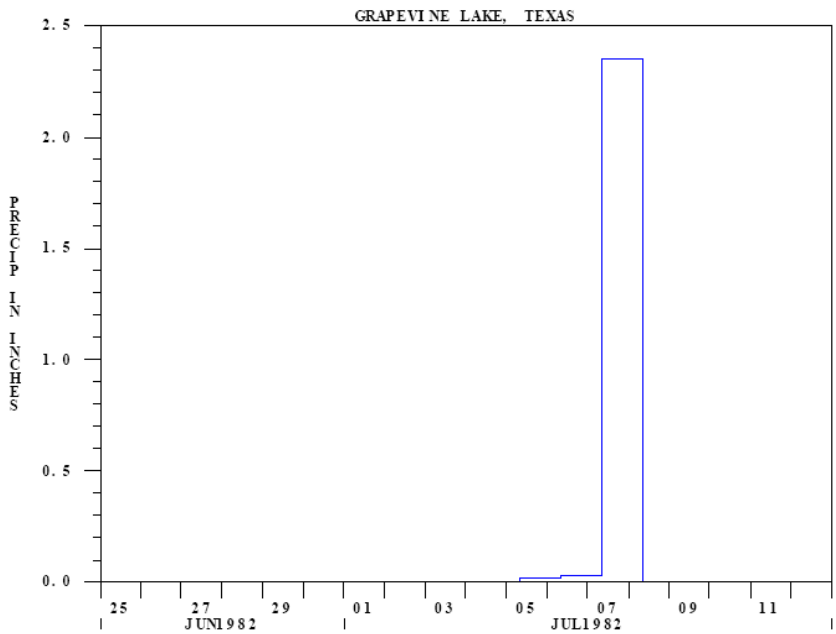


Figure 7.21 Rainfall data for July 1982 (1 in. = 2.54 cm)

7.8.2 Failures Observed in Recent Years

There was a high intensity rainfall of 12 cm (4.76 in.) during March 2008 which has resulted in two surficial failures at Joe Pool Dam within few hundreds of metres from the test section location as shown in Figure 7.22.

7.8.3 Extent of Failures

The maximum length of surficial failure is about 23 m (75 ft) as observed from Table 7.5. Dronamraju et al. (2008) reported that the maximum length of failure depend on the depth of desiccation zone. It was also explained with the help of Gstabl software results that the factor of safety increases once the failure circle intersects the unsaturated zone where the strength parameters are higher than those of the saturated desiccation zone. Small increase in drained cohesion of the surficial desiccated zone can significantly enhance safety of slope. Hence surficial treatments with admixtures such as lime and fibers were recommended to mitigate the surficial failures.

(a)

(b)

Figure 7.22 Surficial failure at Joe Pool Dam due to high rainfall event in March 2008 (a) Long view (b) Close up view

7.8.4 Drained Shear Strength Parameters

The laboratory tests conducted on the control section field sample have shown a drained cohesion of zero on control soil and a drained friction angle of 20º for Joe Pool Dam soil. University of Missouri, Columbia conducted tests for Missouri Department of Transportation, USA on soil obtained from various surficial failure sites of highway embankments (Loehr and Bowders, 2007). They have reported an effective cohesion of zero for almost all the sites. Previous studies based on stability analyses carried out for various types of soils reported that the drained friction angle varied from 5º to 40º (Mesri et al. 2003).

7.9 Summary

The finite element software PLAXFLOW was used to study the influence of rainfall infiltration to increase the degree of saturation. The influence was studied for three cases of no rainfall event, moderate rainfall event and desiccation with high intensity rainfall event. The results have shown that the soil starts getting saturated near the crest for the case 2 with moderate rainfall. The soil quickly gets saturated in the event of a prolonged or high rainfall event as the top layer of soil is having higher hydraulic conductivity due to desiccation. The strength of soil gets reduced as a result of saturation of top layer of soil and the slope is prone to surficial failure.

The Gstabl software program was used to determine factors of safety for the three different cases. It was found that there is hardly any influence of treatment when there is no rainfall and the factor of safety of all the five sections is around 9. The factor of safety of slope is decreasing when the soil near the crest starts getting saturated. With the combined effect of desiccation and high intensity rainfall events, the factor of safety of control section got reduced to less than one, indicating imminent failure. The treated sections have shown a good improvement from the strength aspect and the factor of safety is more than the minimum desirable value of 1.3. The analytical model study has further strengthened the conclusions drawn from field performance of treated sections as explained in Chapter 5.

CHAPTER 8

SUMMARY AND CONCLUSIONS

8.1 Introduction

The main objective of conducting this research was to study different surficial soil treatments to mitigate the shallow slope failures and also to select the best performing additive(s). These objectives were assessed based on the laboratory results, field monitoring studies, laboratory model studies and analytical model studies.

As a part of research, literature review was carried out and various causes of slope failures were studied. The infinite slope approach and the strength reduction methods to carry out the slope stability analysis were discussed. The mechanism of surficial failures was discussed in detail based on various previous research studies. The mechanism of desiccation cracking and the influence of unsaturated condition of slopes were also discussed. Various case studies of slope failures with specific emphasis on surficial failures were discussed. Various slope stabilization methods aimed at reducing driving forces and increasing resisting forces were illustrated.

Two locations, Joe Pool Dam and Grapevine Dam located in the Fort Worth district, Texas, of the United States Army Corps. of Engineers were considered for the present research evaluations. Borrow pit soil samples obtained from Joe Pool Dam and Grapevine Dam were experimentally characterized by McCleskey (2005) and from these results, both stabilizers and their dosages were selected for field studies.

As a part of this research, five test sections were constructed at each site with four additives that include 20% compost section, 4% lime with 0.30% fibers, 8% lime with 0.15% fibers and 8% lime. One control section was constructed along with treated section for relative comparison of effectiveness of treatment methods. The 45 cm (18 in.) thick treated soil was covered with top soil of 23 cm (9 in.) thick and the sites were instrument with moisture probes, temperature probes and vertical inclinometers. Elevation surveys and digital image surveys were conducted to monitor the performance of test sections.

Laboratory tests were repeated on the soil samples obtained from the field test sections and these results were used for addressing the Quality Control and Quality Assurance studies (QC/QA). Additional test results of the torsion ring shear strength tests and hydraulic conductivity tests were conducted on the field samples and these results are used for slope stability analysis. QC/QA data revealed that the field construction standards were achieved and most of the properties of treated sections were in close agreement with the original laboratory studies.

Joe Pool lake test sections were monitored for more than 12 months while Grapevine Dam was monitored for three months. The field data collected at Joe Pool Dam test section was analyzed with statistical methods. Statistical t-tests were used for relative comparison of treatment methods. The admixtures were ranked based on the moisture content data and field performance enhancements. The performance of each treatment method was also compared based on other results including temperature probe data, inclinometer survey data, elevation survey data, dynamic cone penetration test data and digital images of cracks in the test sections. The effectiveness of each type of treatment was also assessed by conducting swell tests with alternate wetting and drying on samples of Grapevine Dam test sections.

In addition, the effects of rainfall infiltration to saturate the soil mass of top layer of slope were studied using the finite element software, PLAXFLOW. Three cases of no rainfall, desiccation with short time high intensity rainfall, desiccation with long time high intensity rainfall were considered. Adverse effect of desiccation and how the proposed treatments impact the saturation and slope stability enhancements were also analyzed and addressed.

Table 8.1 shows the summary of various observations of Joe Pool Dam test sections as discussed in Chapters 5, 6 and 7.

8.2 Summary and Conclusions

The results of the findings of this dissertation research are summarized as follows:

- Based on the average annual volumetric moisture content results, the 8% lime with 0.15% fiber treated section is holding higher volumetric moisture content than the other treated sections. As such this treatment section is classified to be the best performing section. The presence of polypropylene fibers in the lime helped in loosening the soil mass thus holding higher moisture content.

Table 8.1 Summary of performance of Joe Pool Dam test sections

Treatment	Average Top Probe Moisture Content	Average Bottom Probe Moisture Content	Residual Moisture Content	Maximum Variation of Elevation in. (cm)	Maximum Inclinometer Movement in. (cm)	Desiccation Cracking	DCP Results Oct. 08 (per 44mm)	Factor of Safety (case 3)
Control Section	19%	25%	15%	6.05 (15.4)	0.58 (1.5)	1.08%	7	0.88
Compost Section	24%	19%	16%	2.44 (6.2)	0.58 (1.5)	1.37%	10	1.57
4% Lime with 0.30% Fibers	20%	26%	14%	2.12 (5.4)	0.52 (1.3)	Nil	31	4.20
8% Lime with 0.15% Fibers	34%	33%	21%	1.28 (3.3)	0.34 (0.9)	Nil	68	6.21
8% Lime	25%	28%	19%	1.52 (3.9)	0.26 (0.7)	Nil	86	4.76

- Based on the statistical t-test results of moisture data, the best performing field treatment sections in the decreasing order with respect to holding higher moisture content are the 8% lime with 0.15% fibers, the 8% lime, the 4 % lime with 0.30% fibers, the 20% compost and the control sections. The residual moisture content is also found to be the highest during the dry season in the 8% lime with 0.15% fibers treated section.
- Elevation survey results have shown that the control soil experienced maximum swelling during wet season and maximum shrinkage during dry season. In control section, the average maximum swell recorded was 6.03 cm (2.37 in.) and average minimum swell recorded was 9.35 cm (3.68 in.) The total variation of swell or shrinkage in the control section, the compost section, the 4% lime with 0.30% fibers, the 8% lime with 0.15% fibers, and the 8% lime section of Joe Pool Dam was 15.38 cm (6.05 in.), 6.20 cm (2.44 in.), 5.38 cm (2.12 in.), 3.25 cm (1.28 in.) and 3.86 cm (1.52 in.) respectively. Of all the test sections, the 8% lime with 0.15% fibers treated section is observed to be undergoing the least vertical movements.
- The inclinometer survey results show lateral displacement in the direction of slope in all the five test sections of the Joe Pool Dam. The maximum displacement was observed in control and compost sections. The displacement was less in case of both the lime and lime with fiber treated sections. Addition of lime has chemically altered the soil and improved its properties.

- Shrinkage or desiccation cracks were noticed prominently during the driest month (June 2008) during monitoring period in Joe Pool Dam. The desiccation cracks surfaced in the control section and compost section. The widest cracks were seen near the construction joint of the old section and the new control and compost section inspite of taking precautions during construction. This calls for additional precaution to be taken at the time of construction to have proper benching with the old section. However, no cracks were noticed in the lime and lime with fiber treated sections of the Joe Pool Dam.
- The model studies using PLAXIS and GSTABL7 indicate that the slope is safe against surficial failure when there is no rainfall event. The factor of safety is high even against a deep seated failure.
- The model studies indicated that under normal rainfall event, the top layer of soil is not getting saturated. However, due to the effect of desiccation zone, during a rainfall event, the wetting front is initiated in the upper layer. This front is resulting in the saturation of soil in the desiccated zone. The factor of safety values are reduced with an increase of the wetting front due to infiltration.
- The factor of safety is found to be less than 1.3 due to effects of desiccated zone and a long time high intensity rainfall. The top layer of soil mass is getting saturated and the slope becomes unsafe against surficial failure due

to reduction of shear strength. However, the slopes treated with admixtures are found to be safe.

- The section treated with the 8% lime and 0.15 % fibers admixture is having the highest factor of safety of above 6 against surficial failure even during these adverse conditions. The sections with the 8% lime and the 4% lime with 0.30% fibers are also having factor of safety values well above 4.
- There was no failure at any of the test sections except for a few cracks that appeared on the surface and near the construction joints of the control and compost sections of the Joe Pool Dam. However, as the sections were only monitored for one year, no major assessments can be made at this time with respect to failure of sections. Nevertheless, certain positive indicators are seen from the field monitoring of the test sections built with lime and lime with fibers.
- As explained in the Chapter 6, the shrinkage cracking potential of the control soil of the Joe Pool Dam having a CH soil is found to be higher than the cracking potential of the control soil of Grapevine dam having a CL soil.
- The data collected from the Grapevine Dam was for three months period. Additional data needs to be collected to make relative comparison of the performance of test sections.
- The performance of compost section needs to be monitored for a longer period before judging its usefulness.

8.3 Specific Conclusion and Recommendations

The objective of the research study is accomplished. Based on the analysis of data collected, the 8% lime with 0.15% fiber treated section is found to the best performing admixture followed by the 8% lime, the 8% lime with 0.15% fibers and the 20% compost section. The use of 8% lime with 0.15% polypropylene fibers is recommended as admixture to strengthen the vulnerable slopes of dam embankments and while repairing the failed slopes.

Soil stabilization with the recommended dosage of chemical admixture of 8% lime with 0.15% fibers is a pragmatic solution for prevention of surficial failures. Lime and polypropylene fibers are readily available in the market and the machinery required for construction work is also available in the market. No special expertise or skills are required for field construction. During construction of the test sections at Joe Pool Dam and Grapevine Dam, no typical problems were faced. Similarly no failures were noticed at the site during the observation period. Caution should be exercised to have better anchorage with old soil by proper benching.

The cost benefit ratio is not studied in this research as the work carried out is for a test section of 18 m x 7.5 m (60 ft x 25 ft). The approximate cost for construction of the test sections is \$108/m^2 (\$10/ft^2). However, the cost will reduce if the treatment is carried out over a large area.

The recommended admixture can be used at vulnerable locations of dam sections where desiccation cracks formed predominantly. The admixture can also be used during repair and rehabilitation of surficial failure locations.

It is observed that at Joe Pool dam and other locations, a concrete apron is placed at the edge of crest where tension cracks are formed. The self weight of concrete might have been a destabilizing force in the event of a surficial failure. In view of the above, the edge of the crest including a part of slope area can be treated using 8% lime with 0.15% fibers admixture in lieu of concrete apron. This will be more economical and it does not increase destabilizing driving forces. This will in turn prevent desiccation cracking and the ensuing surficial failures.

8.4 Future Research Needs

As a part of future research needs, it is recommended that the moisture sensors data from both the test sections be collected for longer periods. The test section should be monitored visually and with the help of digital images to observe the cracking pattern. Inclinometer surveys and elevation surveys are to be continued during the monitoring period. The possibility of extending the research findings to solve the problems of surficial failures of highway embankments and cut slopes is to be explored.

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