VARIATION OF GEOTECHNICAL STRENGTH PROPERTIES WITH AGE OF LANDFILLS ACCEPTING BIOSOLIDS

by

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ABSTRACT

The solid portion of waste disposal, known as Municipal Solid Waste (MSW) can be landfilled. Landfilling has proved to be a safe, sanitary and economical method of disposal. A by-product from wastewater treatment plants called biosolids is sometimes co-disposed along with MSW in landfills. Recent work at the University of Central Florida has focused on the behavior of the mixture of MSW and biosolids.

As an increased amount of waste accumulates in these landfills, it creates a new problem – the geotechnical stability of landfills. In current literature, classical geotechnical testing methods have been followed to find the strength properties of these landfill materials. Furthermore, geotechnical methods of slope stability analyses have been employed to determine the stability of landfill slopes. As these materials have a high organic content, their strength properties may potentially change with time because of the decay of the organic materials.

In the present work, an attempt is made to monitor the change in the geotechnical strength properties of the landfill materials as a function of time. Direct shear tests used for soil testing, with some modifications, were performed on cured compost samples of MSW mixed with biosolids. Geotechnical strength properties of these cured samples were compared to those of an artificially prepared mixture of MSW and biosolids, from the published literature. In addition, direct shear tests are also performed to find the interface properties of a geonet with the cured samples to check the role of a geonet in reinforcing the landfill slopes. A slope stability analysis software SLOPE/W is used to analyze the stability of the landfills.

Cohesion is observed to decrease with time while the friction angle increases with time. Stability (the factor of safety against failure) of landfill slopes increases with time due to increased effective stresses and increased friction angle, as the organic material decays. This may result in additional subsidence but an increase in the effective shear strength with time. Based on the interface test results and subsequent slope stability analyses, it is found that the inclusion of a geonet improves the slope stability of a landfill. This could be a potential benefit to the landfill as reinforcement if properly placed. Based on the slope stability analysis on landfills with different slopes, it is concluded that the slope stability of a landfill is improved by keeping the slopes less steep.

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LIST OF ABBREVIATIONS

MSW	Municipal Solid Waste
CPT	Cone Penetration Test
ASTM	American Standard for Testing Materials
USPHS	United States Public Health Services
APWA	American Public Works Association
EPA	Environmental Protection Agency
FDEP	Florida Department of Environmental Protection
RCRA	Resource Conservation and Recovery Act
GLE	Generalized Equilibrium Method
FS	Factor of Safety
OMC	Optimum Moisture Content
UCF	University of Central Florida

CHAPTER ONE: INTRODUCTION

1.1 Importance of the Subject

With ever growing population, technology boom and increased consumption of man-made things, the handling of the per-capita waste produced is of major concern. With the public participation in reducing the waste production and with recycling techniques, the waste production rate in the United States has been fairly constant for the recent years. But, this rate is projected to be increasing for the coming years (Oweis and Khera, 1998). Waste produced in all the three physical states, has to be treated and disposed in an environmental friendly way. Biosolids is a by-product from waste-water treatment plant. Earlier practices of land application, incineration etc., have proven to be hazardous to some extent, to both the environment and human health. Biosolids, when co-disposed with Municipal Solid Waste (MSW) in a landfill, increases its gas production capacity, thereby increasing its bioreactor efficiency (Sreedharan, 2003). Co-disposal of MSW with biosolids for landfilling has proven to be an economic, safe and viable option for disposal (Koodhathinkal, 2003).

With more and more MSW and biosolids getting placed in landfills, and with limited availability of space, landfills are getting taller. The slopes are, however, strongly controlled by the local regulations. Analyzing the geotechnical stability of these landfills is a challenge, mainly because the materials being analyzed are not a soil whose geotechnical properties can be quantified with established methods. The problem is two fold:

1. The quantification of the geotechnical properties of these materials (MSW and biosolids) at the time of placement and
2. Quantification of the variation of these properties with time, as these properties are bound to change because of the decaying nature of the organics in these materials.

As landfill material can consist of a variety of constituents, it is very difficult to determine the geotechnical properties of these materials using conventional geotechnical testing methods. However, geotechnical tests are an appropriate starting point for characterizing these heterogeneous materials. Koodathinkal (2003) performed direct shear tests on artificially prepared MSW samples shredded to an inch (25.4 mm). He used a larger direct shear box (5.51 x 5.51 inch in plan) rather than the conventional one used for soils. He also tried to find the change in the geotechnical properties of the landfill materials with varying proportions of biosolids and lime sludge. He plotted the variation of factor of safety with varying proportions of biosolids. Vajirkar (2004), on the other hand, obtained field data from cone penetration tests (CPT) conducted on landfill sites which accept biosolids and compared it with the sites that contain only MSW. This study is a continuation of these previous efforts.

1.2 Objective

This research aims at studying the time variation of two geotechnical strength properties, namely the cohesion and angle of internal friction. Consequently, the factor of safety of a typical landfill waste slope containing a mixture of MSW and biosolids is investigated. It also tries to look at the feasibility and effectiveness of a geonet in reinforcing the landfill slopes based on tests conducted on its interface properties with the waste.

Cured compost samples from Sumter County Composting Facility were used to portray very old landfill material in this research. Direct shear tests were conducted on the compost samples according to the current procedure established by ASTM for soils with slight modifications related to the size of the shear box to accommodate for larger particle sizes. Results from direct shear tests on these cured samples are compared to those on synthetically prepared mixture of MSW and biosolids from the published data (Koodathinkal, 2003) that represents new landfill materials. Modeling and slope stability analysis of landfills were then conducted using a software called SLOPE/W. SLOPE/W, which is designed for soil, uses the theory of limit equilibrium of forces and moments to compute the factor of safety against failure. As waste is assumed to act like a cohesive soil (Shafer et al., 2003), SLOPE/W can be adopted for analyzing the slope stability of the landfills.

Interface properties between the compost samples and geonet were also determined by placing the geonet in between the direct shear boxes containing the compost materials and running the direct shear test in the normal procedure. These results were used in checking the suitability of the geonet in reinforcing the landfills at critical locations of a weak plane of failure within the landfill mass.

1.3 Limitations

Cured compost samples were used in this research to represent an old landfill material, typically ranging from 5 to 20 years in age. MSW collected from the households will be brought to the composting facility. After all the recyclables are removed, 25% biosolids will be added and the mixture will be sent to the aerobic digestor for curing. Cured compost samples were obtained from the digestor. In a landfill, it takes years for the material to become like cured compost. Following are the limitations for the cured compost to represent an old landfill material.

1. Because of the high percentage of biosolids, the cured compost may not represent an actual landfill material.
2. Generally, anaerobic conditions prevail inside most of the landfills. Aerobically mixed compost may not represent a landfill material.
3. Mechanically prepared compost may not represent the size and composition of landfill material which has been present inside the landfill for years.

Hence, landfills modeled by using the laboratory test results on the compost samples are named "old" landfills and the actual age of the landfill is not specified.

This research does not address all the factors involved in the general slope stability of landfills. Slope stability of a landfill is governed by many factors, in addition to the strength of the landfill material. These factors include:

1. Allowance of proper drainage for the leachate; ponding of leachate on the bottom clay liner may cause sliding failure.
2. Proper compaction of the individual cells and integrity of the cells and cell covers; improper compaction may cause a particular waste cell to be separated from the landfill thereby causing a local failure which may sometimes be catastrophic and can lead to loss of human life.
3. Slope failure of a real landfill is a three-dimensional problem and the conventional two-dimensional slope stability analysis ignores the resisting forces

from the wedges that are perpendicular to the two-dimensional plane. Hence, two-dimensional slope stability analysis yields lower values of factor of safety, which is a conservative estimate.

This research assumes that the engineered landfill is designed ideally, behaves as a two-dimensional structure and focuses only on the decaying nature of the landfill waste and its impact on the geotechnical properties of the landfill waste and the overall stability of the landfill.

CHAPTER TWO: LITERATURE REVIEW

2.1 Introduction

In this chapter, an overview on the primary role of a landfill in MSW management, the modes of slope failures in landfills, factors responsible for slope-related failures, methods of analysis for landfill slope stability and the evolution of the testing methods for the landfills is presented

2.2 The Practice of "Landfilling"

Urban growth accelerated at the end of World War II. With this growth came increased refuse generation. The impacts of open-burning dumps on public health became a concern. Experience in the military with sanitary fill methods and the interests by some local governments in eliminating open dumps led to increased efforts to dispose of refuse in a sanitary manner, but what prescribed a sanitary manner was unknown. Many local governments that bragged about using the "sanitary landfill" method were actually using modified open dumps.

The research papers from 1940s to '50s noted a growing awareness of settlement, gas generation, and fires in sanitary landfills—the first hints about the potential of groundwater contamination.

The commitment of the US Public Health Service (USPHS) to provide national leadership to eliminate open-burning dumps and replace them with sanitary landfills also served as the basis for a broader national strategy to improve the management of refuse.

The most distinctive characteristic of the sanitary landfill that separates it from all other landfilling and dumping practices is the use of daily cover. In 1955, Ralph Black of the California State Health Department began research on how much cover material was necessary and at what frequency cover should be applied to deal with flies. Over a period of time, working with entomologist A.M. Barnes, their efforts indicated that 2.625 in. of compacted soil would prevent the emergence of flies from a landfill. Operational limitations (compaction equipment would penetrate a cover of that thickness), however, led the researchers, namely, Black and Barnes, (1956), to recommend that 6 in. of compacted soil would eliminate this operational problem.

In 1965, with the passage of the Solid Waste Disposal Act, the USPHS accelerated its efforts to introduce sanitary landfill practices in the US. New publications were developed to better explain the sanitary landfill to regulators, designers, operators, and the public. The term *refuse* was being phased out and replaced with the term *solid waste*. State governments increased their investments in solid waste management. State solid waste programs, normally a part of state vector-control efforts, began to be formed as separate entities. Using USPHS 1961 guidelines, many states began to establish state regulations.

Even with such dramatic progress, an accepted definition and understanding of what actually constituted a sanitary landfill remained an open issue. Landfill gas migration and the possibility of explosions resulted in a need for control measures and eventually birthed a new industry to capture and utilize the gas as an energy source. Studies began to signal that leachate from landfills could contaminate groundwater, resulting in the birth of groundwater monitoring systems for landfills.

Until the early 1970s, the USPHS approach to get states, local governments, and private landfill owners to change to the sanitary landfill had been through research and development, demonstrations of new technological approaches, training, and technical assistance. The USPHS solid waste program was moved to the Environmental Protection Agency (EPA) when it was created in 1970. As an enforcement agency, EPA found the USPHS approach to effecting change inconsistent with its approach. With no enforcement authority over solid waste, EPA interests in solid waste were minimal. As a consequence, from the formation of EPA until 1976, EPA investment in sanitary landfill programs continued to be low.

In 1976, Congress recognized that disposal practices were not improving and that federal attention needed to be increased. The passage of the Resource Conservation and Recovery Act (RCRA) directed EPA to develop criteria for classifying open dumps and sanitary landfills. Building on the USPHS criteria of 1961, the materials and information developed by earlier USPHS research and development efforts, and the now-defunct USPHS training programs, EPA issued the congressional-mandated criteria in 1979. The content of these criteria reflected the early efforts of the USPHS, but EPA added several significant improvements. For the first time, criteria were proposed for landfill gas migration and groundwater protection. In addition, the introduction of bulk liquids into sanitary landfills was discouraged. But again, with no enforcement authority over solid waste, EPA lacked the enthusiasm to encourage the implementation of the criteria. The limited resources of the solid waste program were directed to the development of the RCRA-mandated and enforceable hazardous-waste regulatory provisions.

From 1979 until 1984, EPA’s limited investments in solid waste (nonhazardous waste) were principally in nonlandfill-related programs. Once again, Congress signaled to EPA that nonhazardous-waste landfills had to be addressed. In the RCRA 1984 Hazardous and Solid Waste Amendments, Congress finally granted EPA regulatory authority over landfills and directed the preparation of landfill criteria. EPA responded with a very complete set of criteria (Subtitle D) to be adopted by the states. These rules reflect the almost 50 years of contributions by the USPHS, individuals, organizations, researchers, committees, private solid waste management service providers, and local governments to develop a new technology, the sanitary landfill, and EPA’s work from the hazardous-waste regulations. The RCRA Subtitle D criteria impose a series of design, operating, monitoring, and remediation requirements on MSW landfills. The landfills built to comply with these rules truly eliminate the mosquitoes, flies, rats, swine, and smoke of the open-burning dumps of the 1940s and ’50s and provide what the pioneers sought - a means of disposing of solid waste in a sanitary manner.

2.3 Time Varying Nature of Geotechnical Properties of Waste

Wardwell and Nelson (1981) analyzed the effects of fiber decomposition on the long-term laboratory compression of organic sludge deposits and concluded that many of the engineering properties of saturated sludge landfill deposits are dependent upon its organic content and changes in organic fraction due to fiber breakdown affect the field settlements of these deposits and, in turn, the associated leachate generation from these materials. Since then, not much attention has been given to the time varying nature of the geotechnical properties of waste.

Hence, it is important to study the related landfill properties and the influence of changes in properties of the waste on the slope stability of landfills. This study will provide us with an insight into the specific geotechnical properties to be monitored with time in order to determine the time variation of the stability of a landfill.

2.4 Landfill Failure Types

The stability of landfills is an important issue with catastrophic implications. Some failures have been reported, which have resulted in loss of life. An understanding of the various types of failures that occur in landfills is critical. Several types of failures are possible for landfills. The covers and liners for modern landfills are typically multi-layer composites composed of both soil and geosynthetic materials. The typical landfill cross-section showing the liner system is presented in Figure 1. It contains several interfaces whose resistance against interface shear stresses may be low, and thus act as possible failure surfaces.

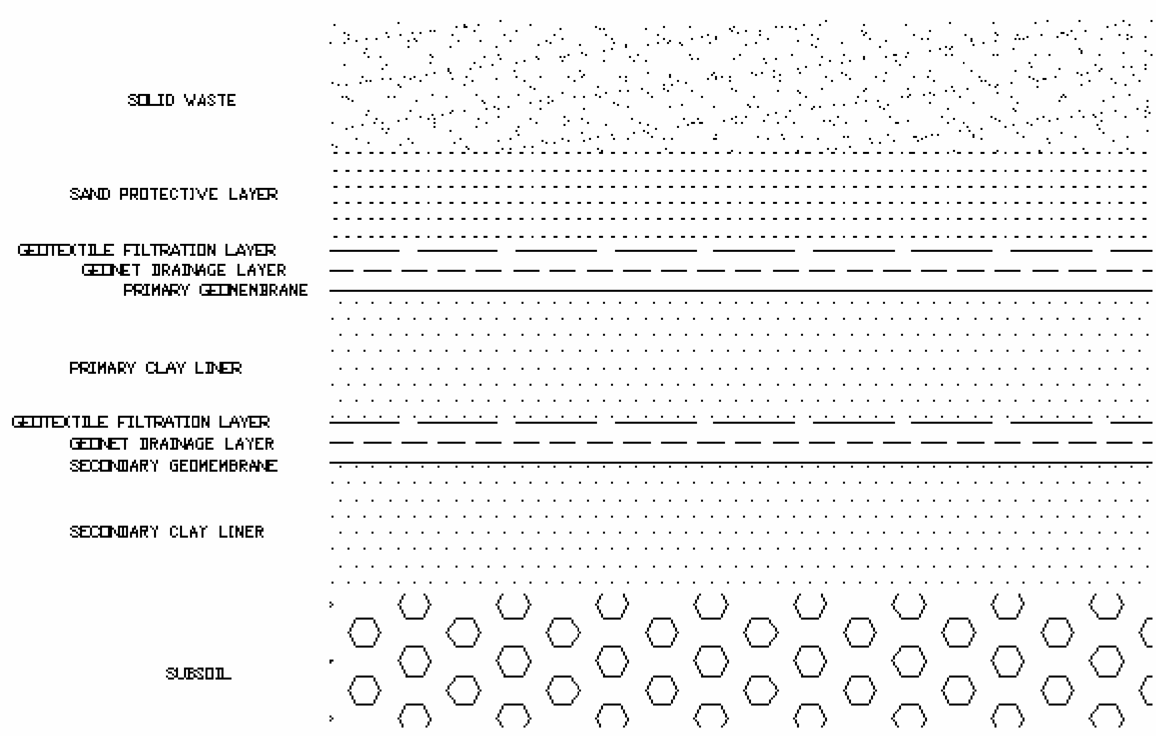


Figure 1: Double Composite Liner System

Additionally, all classical geotechnical failure modes are possible depending upon site-specific conditions (usually involving saturated fine-grained soils) and the placement and geometry of the waste mass. Potential failure modes are described below.

2.4.1. Sliding Failure of Leachate Collection System

As shown in Figure 2, the leachate collection system may slide on the underlying liner system if the slope is too steep. This failure can be expected during heavy rains. This is an interface failure.

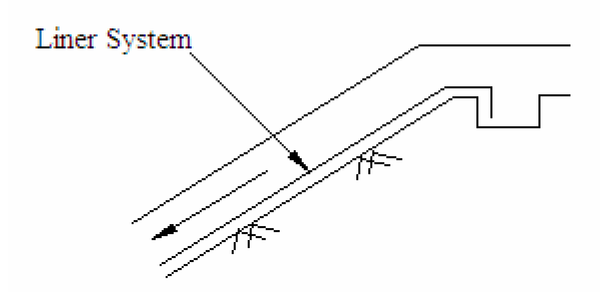


Figure 2: Failure of Leachate Collection system

2.4.2 Sliding Failure of Final Cover System

The final cover system (topsoil and protection soil) shown in Figure 3 may also slide on the liner system if the slope is too steep and is under conditions of heavy rains. This is an interface failure.

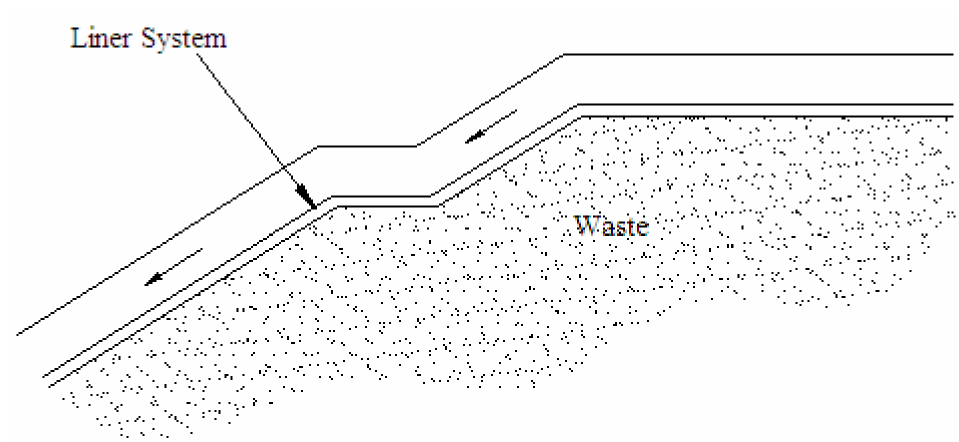


Figure 3: Failure of final cover system

2.4.3 Rotational Failure of Sidewall Slope or Base

As seen in Figure 4, the soil mass behind the waste repository or beneath the site could be unstable and fail. Failure is usually rotational, emerging along the slope, at the toe, or within the foundation.

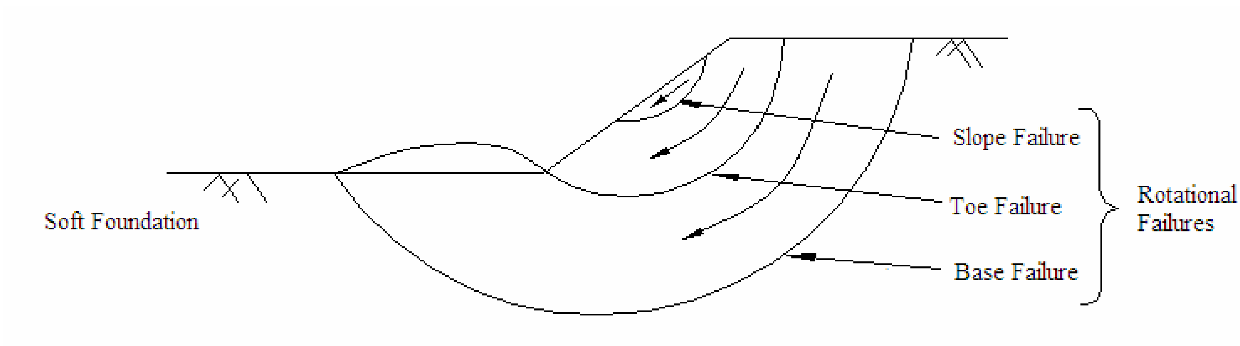


Figure 4: Soil slope, toe and base failures

2.4.4 Rotational Foundation Failure through Waste, Liner, and Subsoil

As seen in Figure 5, a rotational failure can be initiated in a soft foundation soil that can propagate up though the waste mass. The resistance from a liner system, if present, is negligible.

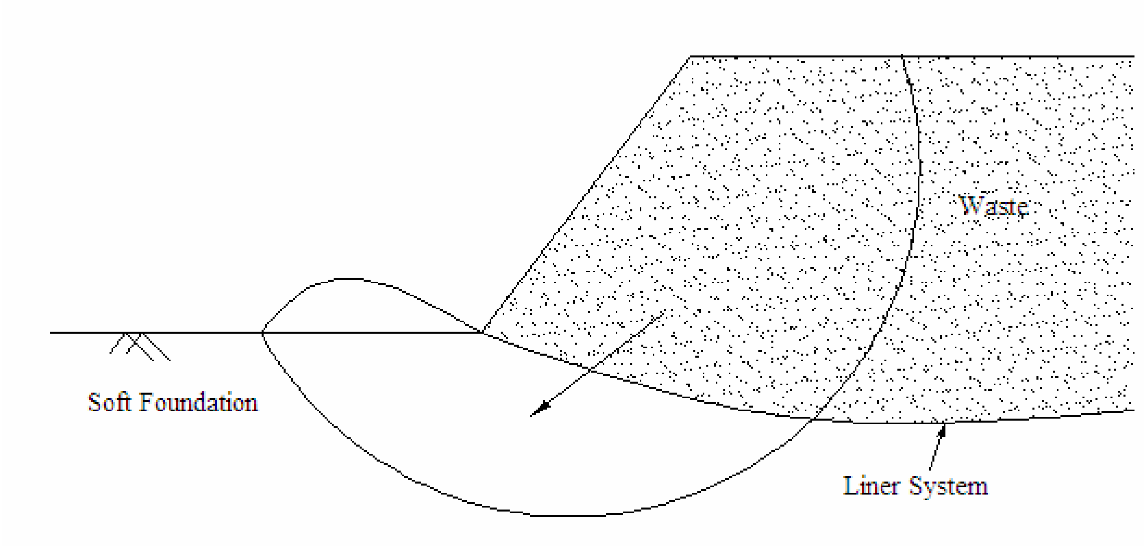


Figure 5: Foundation failure through subsoil, liner and waste

2.4.5 Rotational Failure within the Waste Mass

Failure can also occur within the waste mass, completely independent of the liner system as seen in Figure 6.

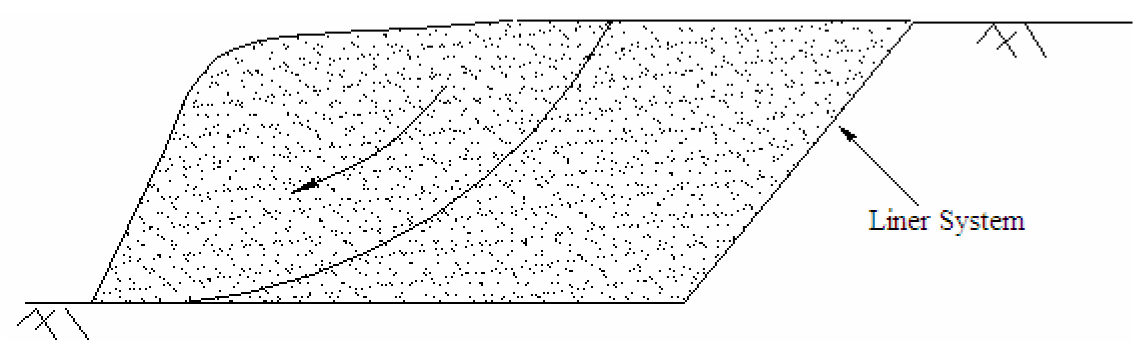


Figure 6: Failure within waste mass

2.4.6 Translational Failure by Movement along the Liner System

As seen in Figure 7, a lateral translational failure can occur with the solid waste sliding above, within, or beneath the liner system at the base of the waste mass. The failure plane can extend back from the toe and propagate up though the waste, or continue in the liner system along the back slope.

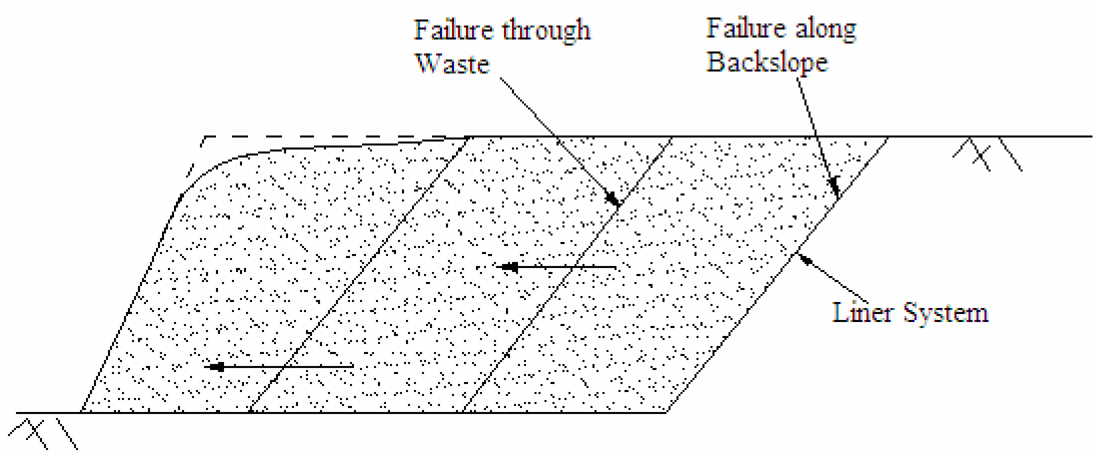


Figure 7: Translational failure along liner system at base and up through waste or liner

2.5 Factors Affecting Slope Stability in Landfills

A slope is said to be stable only when the shear stress developed along the most likely rupture surface is less than the shear strength of the slope material along the surface. The factor by which the shear strength of the slope material must be reduced in order to bring the mass of the slope material into a state of limiting equilibrium along a selected slip surface is called factor of safety for that particular slip surface. The minimum of all the factors of safety possible for a slope is called the overall factor of safety for the slope. The corresponding slip surface is called the failure surface or slip surface of the slope.

Having discussed the various modes of failure possible in a landfill in the previous section, we now have an enhanced understanding of some of the factors that may affect the landfill stability and hence can list them as follows:

2.5.1 Geometry

The exterior slopes, bottom grades, height of fill, and surcharges contribute to the driving forces. Berms at the toe of slopes contribute to the resisting forces. Hence, bottom liner grades, final landfill grades and liner side slope grades should be maintained or designed as flat as practical.

2.5.2 Shear Strength of Materials

The shear strength of liner soils and the shear properties of the interfaces with waste affect the stability of landfill. Lundell and Rohr (1991) studied the impact of the frictional characteristics of interfaces and the related potential stability issues on the design, construction and operation of landfills, from a landfill owner's perspective. They concluded that, when geosynthetics are used, and especially when they are placed adjacent to each other in a liner or final cover configuration, the frictional characteristics of the resulting interfaces can become a very important factor affecting the design, construction and operation of a landfill.

The shear strength of foundation materials or parent soils resists bearing failures beneath the landfill. Significant quantities of liquid and decomposed organic wastes are possible in most bioreactor landfill facilities. Their effect on the slope stability of the landfill as a whole must also be considered.

2.5.3 Loading Conditions

Factors that affect loading conditions in a waste fill are the unit weight of waste that results in insitu overburden stresses and any applied external loads, such as stockpiling of soil or construction equipment on the landfill. Vertical expansions and stockpiles on existing fills increase normal loads on existing waste, liner and base materials. External loads from seismic events can also induce significant stability problems and settlement in the waste facility. Some post closure activities may increase or decrease normal loads on fill materials.

2.5.4 Pore Water Pressure

Increases in pore water pressure can have a significant detrimental impact on the stability of landfills, whereas a decrease in pore water pressure can stabilize a landfill. Pore water pressure change can have an impact on the strength of the waste due to changes in effective stress. Leachate, groundwater, surface water infiltration and recirculation of liquid into the landfill mass can also have a destabilizing affect if not properly controlled. Pore water pressure or piezometric head may also build up on low permeability soil daily cover layers because of ponding and may cause localized liquid outbreaks, shallow slumps and veneer failures of interim and final covers. Bachus et al, (2004) concluded that, although the introduction of water or other liquids to enhance the degradation of the waste has several potentially destabilizing effects, they can be mitigated through sound design, construction, and operating practices. Hence, bioreactor landfills can be designed, constructed, and operated in compliance with regulatory requirements and standards of practice for slope stability. The New River Regional Landfill near Gainesville, Florida and Williamson County Landfill facility, Tennessee are few examples of bioreactor landfills.

2.5.5 Settlement

Settlement of the landfill materials has both stabilizing and de-stabilizing effects on landfill slope stability. Localized settlement and low spots will encourage surface water to infiltrate in the mass, potentially increasing pore water pressure and piezometric head in the waste mass. Bioreactor landfills have experienced accelerated settlements because of decomposition and stabilization of the decomposable organic fraction of the landfill.

Singh and Murphy, (1990) concluded that slope failure may not be the most critical aspect of a sanitary landfill. Settlement and bearing capacity of the foundation soil might be the more significant parameters. Wardwell and Nelson (1981) analyzed the effects of fiber decomposition on the long-term laboratory compression of organic sludge deposits and concluded that many of the engineering properties of saturated sludge landfill deposits are dependent upon its organic content and changes in organic fraction due to fiber breakdown affects the field settlements of these deposits and, in turn, the associated leachate generation from these materials.

2.5.6 Operations

Landfill operations have an impact on landfill stability. Especially, bioreactor landfill operations increase the complexity of a landfill facility and the filling operations (Reinhart and Townsend, 1997). In such landfills, degree of saturation of the waste, the liquid injection system, gas extraction system, temperature of the waste, piezometric head in the fill and isolated pore water pressure should be monitored.

Koerner and Soong (2000) presented a unified approach explaining the influence of leachate on landfill stability in a sequential manner and concluded that the critically important factors both during waste placement operations and quite possibly for the entire service lifetime of the landfill with respect to the overall stability of the waste mass are (1) quantity of leachate in a landfill and/or (2) the site specific liquids management program.

2.6 Methods of Analysis for Slope Stability of Landfills

Pursuants to the study of the various modes of failure and the factors responsible for the failures, the next issue to be addressed is the method of analysis to be adopted to assess the slopes in the stability of the landfills.

According to Shafer et al, (2000), there are two basic approaches to stability analysis. These are the limit equilibrium and the elastic methods. Strain considerations are of little consequence in limit equilibrium whereas in elastic methods, strain and its relationship to stress are of major importance. Hence, elastic models are very complicated and are generally too complex for practical use in basic engineering problems. This research will adopt the limit equilibrium method for further study.

Limit equilibrium approach can be either three dimensional or two dimensional. Two dimensional analysis gives lower factors of safety as it ignores the resisting forces from the wedges that are perpendicular to the plane. However, three dimensional analyses are more complex. Hence, many engineers use the conservative two-dimensional limit-equilibrium approach when analyzing the stability of landfills.

Limit equilibrium approach assumes that Coulomb's failure criterion is satisfied along the assumed failure surface, which may be a straight line, circular arc, logarithmic spiral or other irregular surface. In general, a free body is taken from the slope and, starting from known or assumed values of the force acting upon the free body, the shear resistance for equilibrium is calculated. It is compared to the estimated or available shear strength of the material to give an indication of the factor of safety. The various methods of analysis under limit equilibrium approach are

1. Ordinary Method of Slices,
2. Simplified Bishop's method,
3. Simplified Janbu's method,

4. Spencer's method,
5. Morgenstern-Price method, and
6. Generalized Limit equilibrium

2.6.1 Ordinary Method of Slices

In ordinary method of slices, the soil above the trial failure surface is divided into several vertical slices of arbitrary widths and the forces acting on the slice are analyzed. This method assumes that the resultant of tangential and normal force acting on one side of the slice is equal in magnitude to that acting on the other side and also that their line of actions coincide. Equilibrium equations are then written and the factor of safety is found by dividing the resisting force with the driving force. The factor of safety (F_s) can be found using the following equation,

$$F_s = \frac{\sum_{n=1}^{n=p} (c' \Delta L_n + W_n \cos \alpha_n \tan \phi')}{\sum_{n=1}^{n=p} W_n \sin \alpha_n} \quad \ldots\ldots\ldots\ldots\ldots(2.1)$$

Where,

Fs = factor of safety for the n^{th} slice

c' = cohesion

$\Delta L_n = (b_n) / (\cos \alpha_n)$

b_n = width of nth slice

α_n = angle between W_n and N_r

W_n = weight of the slice

N_r = normal component of reaction force R

φ'= internal friction angle

n = counter of the slices

p = total number of slices

2.6.2 Simplified Bishop's Method

In this method, the forces that are neglected in ordinary method of slices are accounted for to some degree. Factor of safety (F_s) can be found using the following equation,

$$F_s = \frac{\sum_{n=1}^{n=p} (c' b_n + W_n \tan\phi') \frac{1}{m_{\alpha(n)}}}{\sum_{n=1}^{n=p} W_n \sin\alpha_n} \qquad \text{.............(2.2)}$$

where,

$$m_{\alpha(n)} = \cos\alpha_n + \frac{\tan\phi' \sin\alpha_n}{F_s} \qquad \text{..............(2.3)}$$

Equation 2.2 assumes that interslice shear force, $\Delta T = 0$.

All the terms in Equations 2.2 and 2.3 denote the same quantities as in Equation 2.1.

2.6.3 Simplified Janbu's Method

This method also assumes zero interslice shear force, but it satisfies horizontal force equilibrium only. It satisfies overall moment equilibrium and vertical and horizontal force equilibrium, but does not satisfy individual slice moment equilibrium.

2.6.4 Spencer's Method

This method satisfies both force and moment equilibrium. It assumes that the inter-slice forces are parallel and their resultant has a constant inclination to the vertical. It satisfies all states of equilibrium including individual slice moment equilibrium.

2.6.5 Morgenstern-Price Method

This method allows one to specify different types of inter-slice force function. This is very much similar to the Spencer's method except that the inclination of the inter-slice resultant force is assumed to vary according to a portion of an arbitrary function.

2.6.6 Generalized Limit Equilibrium (GLE) Method

This method embodies all the limit equilibrium methods which calculate both force and moment factors of safety for any specified value of inter-slice shear force, λ. This method is very much like the Morgenstern-Price method, except that this method can compute the moment and force factors of safety for a range of λ values.

The methods of stability analysis and equations for factors of safety described in the previous sections are modified, when applied for liner stability. Instability in liners causes sliding failure and can be catastrophic. The geomembranes in these liners are hence anchored to prevent such failures. The method of stability analysis used in this research does not address the issue of liner stability. Assuming that there is no danger of sliding failure from the liner, this research focuses on the overall stability of the landfill slope.

Quan et al, (2003), proposed the translational failure analysis of landfills, which is a new approach to the two-part wedge method for translational failure analysis of landfills. They concluded that this method ensures that the waste strength is not exceeded anywhere in the waste mass and generates a waste filling sequence so as to keep the factor of safety above a stipulated value during the operation phase of the landfilling process.

The reliability of any of the stability analysis methods, discussed above, is highly dependent on the accuracy of the strength properties and the defined geometry. The type of analysis or stability calculation can also introduce variability in the results because of the inherent assumptions and approximations made in developing the methods of analysis. Minimizing variability in controllable items is preferred; therefore choosing analysis methods that minimize approximations is desirable. Hence, a method of analysis that satisfies both moment and force equilibrium is always preferred. Of all the methods of analysis, Janbu or Spencer's method of analysis satisfies the overall moment and force equilibrium requirements and hence, is recommended for the analysis. Any slip surface shape can be analyzed using these two methods of analysis. Generally, Janbu will provide more conservative values of factor of safety.

All the methods of analysis discussed above, are based on dividing the slope into slices and hence can be grouped under "method of slices". Method of slices can readily accommodate complex geometry, heterogeneous waste material properties and external loads. The software SLOPE/W employed in this research, uses this method. The factor of safety in SLOPE/W is formulated in terms of two equations, one satisfying the force equilibrium and the other satisfying the moment equilibrium. Depending on the inter-slice force function adopted, the factor of safety for all the methods can be determined from these two equations.

One key difference among the various methods is the assumption regarding inter-slice normal and shear forces. The relationship between these inter-slice forces is represented by the parameter, λ. For example, a λ value of zero means there is no shear force between the slices, while a λ value that is nonzero means there is shear between the slices. A plot of the factor of safety versus λ is shown in Figure.8.

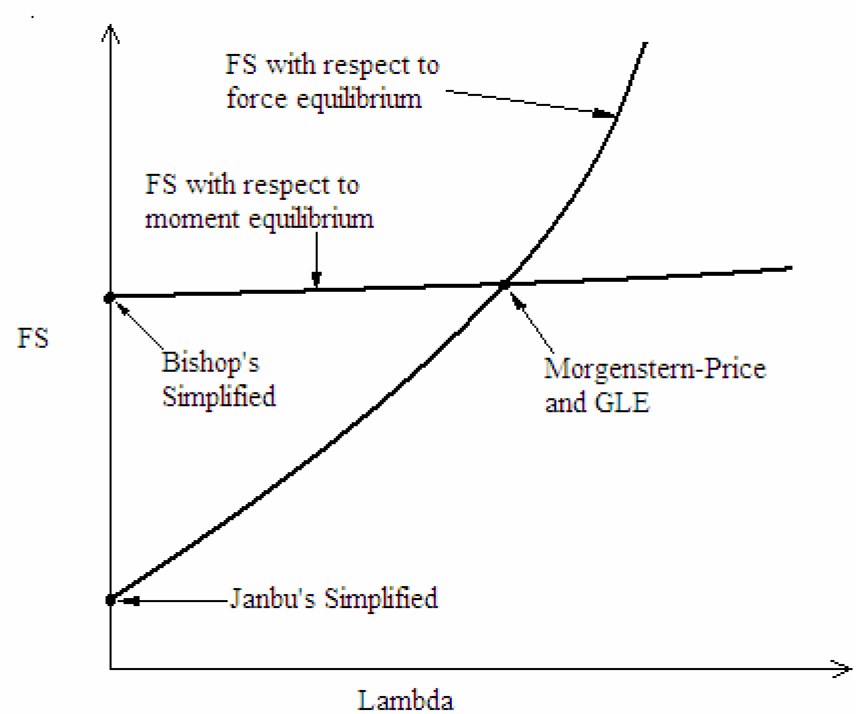


Figure 8: Plot of factor of safety vs. lambda (From GEO-SLOPE Intl. Ltd., Teaching guide for SLOPE/W, 1999)

The factors of safety with respect to force equilibrium and another one with respect to moment equilibrium are shown in figure 8.

Bishop's simplified method uses normal forces but not shear forces between the slices (λ=0) and satisfies only the moment equilibrium. The Bishop factor of safety is on the left vertical axis of figure 10. Janbu's simplified method also uses normal forces but no shear forces between the slices and satisfies only force equilibrium. The Janbu factor of safety is therefore also on the left vertical axis. The Morgenstern-Price and GLE methods use both normal and shear forces between the slices and satisfy both force and moment equilibrium; the resulting factor of safety is equal to the value at the intersection of the two factor of safety curves.

Jones and Dixon (2005), while highlighting the role of waste settlement on landfill lining stability and integrity, tried to find both the global instability and the local failure that could result in the loss of lining integrity. They concluded that the factors of safety relate to global failure (development of a continuous failure surface) and not local failure that can lead to loss of integrity.

Qian et al (2002) suggest a minimum factor of safety of 1.5 for stability analysis. Shafer et al. (2000) suggest factor of safety for failures be based at a critical liner interface and range from 1.4 for assumed large deformation, residual interface and waste strengths to 1.5 for assumed peak interface and waste strengths. For failures within the waste or subgrade, a minimum acceptable final condition factor of safety is considered to be 1.5. For interim conditions, peak interface strengths are typically used. Acceptable factors of safety for interim conditions typically ranged from 1.2 to 1.3.

2.7 Geotechnical Testing Methods for Waste

Having discussed the various modes of failure in a landfill, some of the factors responsible for these failures and the various methods of analyses, it is now appropriate to look for suitable testing methods to be used for testing the landfill materials.

Landva and Clark (1987), conducted field and laboratory tests on waste materials, with particular emphasis on refuse landfills and woodwaste such as barkfill (hogfuel), sludge and ash wastes. They concluded that geotechnical investigations of these unusual and difficult materials are indeed feasible, as long as it is recognized that conventional testing methods and analyses may not apply and that a different approach is required.

Duplancic et al, (1990) analyzed the long-term deformation monitoring data of a hazardous landfill through in-situ borings, installation of inclinometers and performing laboratory tests such as index properties tests and triaxial shear strength test and concluded that standard field and laboratory geotechnical techniques may be applied to landfills, but with slight modifications. Many attempts have been made to quantify the geotechnical properties of the waste materials through various testing methods.

Singh and Murphy, (1990) prepared a compilation of these data which noted the waste strength properties. The angle of internal friction ranged from about 1 degree with cohesion as large as 2200 psf to an angle of internal friction as high as 36 degrees with zero cohesion. These values are in agreement with a similar study performed by Kavazanjian et al (1995), in which the drained strength was estimated as the greatest of 500 psf cohesion and 0 degree internal friction angle or 0 psf cohesion and 33 degree internal friction angle. They agreed that in direct shear testing of waste, significant consolidation of the waste during normal load application and drainage of pore water during shearing resulted in internal friction angles greater than typical waste strengths (33 degrees).

2.8 Summary of the Literature Review

After going through an initial phase of evolution, it is now agreed that landfilling waste is a safe and sanitary way of disposal. The stability of a slope in a landfill is a serious issue that requires study. Various modes of failure are possible for a modern landfill and factors such as geometry of the landfill, shear strength of materials, loading conditions, pore water pressure, settlement and landfill operations affect its stability. A method of analysis that satisfies both moment and force equilibrium and which minimizes approximations is preferred. Standard field and laboratory geotechnical techniques can be applied for landfills, but with slight modifications.

CHAPTER THREE: METHODOLOGY

3.1 Introduction

In this chapter, a description of the methodology adopted in this research is presented, which includes the laboratory tests conducted and the modeling and analysis of the landfill slope stability using the software SLOPE/W.

3.2 Basis of the Work

The basic idea is to monitor the shear strength parameters of the landfill material with time, in order to find the effect of decomposition of the organic part of the waste on its strength properties. Based on the literature review, it was ascertained that direct shear tests on representative samples is a suitable approach to determine the shear strength parameters. The usual direct shear apparatus for soils is modified into a larger direct shear apparatus as discussed in Koodathinkal (2003).

3.3 Sampling of the compost material

Cured compost was used to represent an ‘old’ landfill material. Samples of cured compost were brought from Sumter County Composting Facility. MSW, collected from households is brought to the Composting Facility. Typical values of various constituents of MSW in Sumter County are presented in Table 1.

Table 1: MSW Composition for Sumter County, Florida

Material	*Percent by Weight*
Newspaper	6
Glass	3
Yard Trash	8
Construction & Demolition (C & D) Debris	31
Food Waste	5
Textiles	2
Metals	18
Plastics	7
Other Paper	16
Miscelaneous	4

(Source: Florida Department of Environmental Protection (FDEP) Report dated 11/3/2004)

After removing recyclables like paper, glass, yard trash etc., 25% by weight of biosolids, from Wastewater Treatment Plant, are added. The mixture is then sent to an aerobic digestor for curing. It was this product of the aerobic digestion that was brought to UCF to conduct direct shear tests. The higher values of unit weights than that of a normal waste recorded in this research can be attributed to the larger composition of food waste and C & D debris, which have high values of unit weights. Piles of cured compost samples were present in the Composting Facility. Each pile differs in its age. The age of the pile is counted from the day of collection of MSW from the households. As the cured compost is mechanically digested in an aerobic digestor, it is relatively homogeneous, when compared to its particle size and composition at the time of its collection from the households. Samples were collected with shovels from the top, mid-portion and bottom of each pile to minimize the sampling errors, which can arise because of the differences in unit weight values of the constituents. Fifty to sixty pounds of the compost samples from each pile were collected in polythene covers and brought to UCF. The age of the cured compost samples brought were one month (cured sample - 1), twelve months (cured sample - 2), thirteen months (cured sample - 3) and sixteen months (cured sample - 4). Direct shear tests were performed on these samples. Results of similar tests conducted on a fresh mixture of artificially prepared MSW and 30% biosolids (Koodathinkal, 2003) were used to represent the geotechnical strength properties of a 'new' landfill material.

3.4 Overview of Laboratory Testing

Direct shear test was conducted on the cured compost samples. All the constituents of the samples were less than an inch in size. A large size direct shear mold, as shown in Figure 9, was designed and fabricated to accept a sample size of 5.51 x 5.51 x 4.33 inches (Koodathinkal, 2003). As the particle size is less than $1/6^{th}$ the mold size, it is assumed that neither the mold size nor the particle size will affect the test results significantly (Wa'il, 2004).

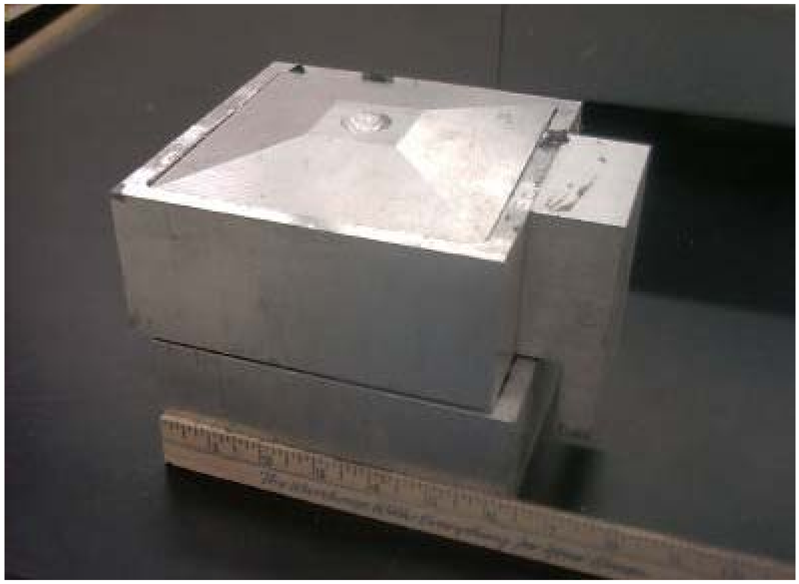


Figure 9: Large size direct shear test box

Initially an effort was made to test the samples at their optimum moisture contents (OMC) by conducting compaction tests to find the OMC using the Auto-Compaction Machine. However the compost samples were found to be too compressible to run the test. The sample in the mold was splattering with a great energy, very similar to soft wet clay, for each blow from the automatic hammer. Hence, direct shear test was run on the samples at their natural moisture contents at room temperature. Typically, the moisture content of most of the specimen was around 60% on a wet basis.

The only control available was on the amount of compaction in the test box. Therefore the samples were compacted to an arbitrary set standard of a 200 g. weight, tamped 10 times for each of the three layers up to the top of the mold. Some portion of the sample was placed in 3 to 5 containers and dried in an oven kept at a constant temperature of

105^{o}C for 24 hrs. Then, moisture content (wet basis) is determined as the weight of water divided by the moist weight of the sample.

The shear box was weighed when full and empty to find the unit weight. Moisture content and unit weights data are presented in Appendix B.

3.5 Direct Shear Test

The direct shear test assembly is shown in Figure 10, and consisted primarily of the direct shear box of dimensions specified earlier. The direct shear tests in this research were performed according to ASTM D 3080. The direct shear box is split into two halves (top and bottom), holding the waste specimen. A proving ring is used to measure the resistance from the horizontal deformation. Two dial gauges (one horizontal and one vertical) are used to measure the deformation of the specimen during the test. The apparatus also has a yoke by which a vertical load can be applied to the specimen.

Figure 10: Shear box assembly for Direct Shear test

A constant horizontal deformation can be applied to the top box by manually rotating a handle. The direct shear test conducted is strain-controlled and hence constant horizontal strain is applied and the resistance from the specimen is calculated from the calibrated proving ring reading. The rates of shear displacement applied were between 0.03 to 0.08 in./min. Incremental horizontal strains were applied until the proving ring dial gauge reading reaches a maximum and then falls, or the proving ring dial gauge reading reaches a maximum and remains a constant. At each equal horizontal deformation, readings from the horizontal dial gauge, vertical dial gauge and proving ring dial gauges were taken.

First, a vertical (normal) load is applied and the compost specimen is sheared until the proving ring dial gauge reaches a maximum and either decreases or remains constant. This procedure is repeated for at least 5 different normal loads. Normal stress is the normal load divided by the cross section area of the mold, which is 30.36 square inches.

For any given set of horizontal and vertical gauge readings, the shear force is calculated as the number of divisions moved by the proving ring dial gauge needle multiplied by the proving ring calibration factor, which is 0.32 lb/div for the proving ring used in this project. The shear stress is the shear force divided by again the mold cross section area. The maximum shear stress is the failure shear stress or shear strength. For each direct shear test conducted, shear strength is plotted against normal stress to get the failure envelope. The scattered points are used to generate a best fit straight line. The intercept of the line on the shear stress (vertical) axis is the cohesion value and the angle it makes with the normal stress (horizontal) axis is the friction angle.

3.6 Geonet as a Reinforcement Material

Though the direct contact of any geosynthetic with waste is not prevalent in the current landfill design, it is proposed that a geonet be, placed within the waste mass. This may be used as a reinforcement material in preventing the slope failure, thereby improving the factor of safety of a landfill. Figure 11 shows the potential placement of a geonet such that the resistance against a slope failure may be increased.

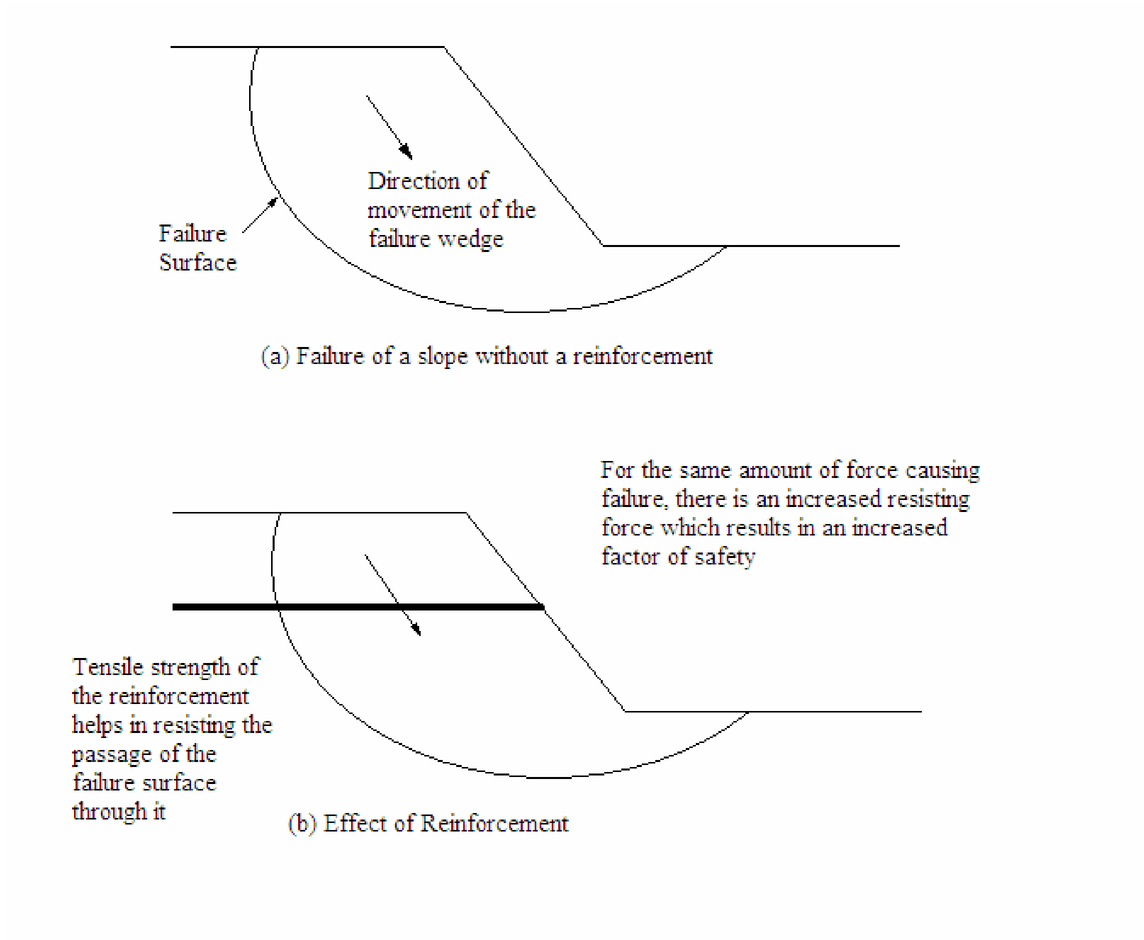


Figure 11: Effect of Reinforcement in Improving the Stability of a Landfill Slope

3.7 Direct Shear Test for Interface Properties

As outlined by Jones and Dixon (2005), and Lundell and Rohr (1991), interface strength of geomembranes and immediate soil layers and integrity of the geomembrane plays an important role in the stability of a modern landfill. For the present day landfill, the geomembrane in the composite liner interfaces with sand on the top and clay at the bottom. Published data (Quian et al, 2002) are available for the shear strength of these two interfaces.

From section 3.6, we find that in order to investigate the potential use of geonets as reinforcement layers, the tensile strength value of the geonet and the interface properties of a geonet with compost are required. The tensile strength of the geonet is obtained from its manufacturer (POLY-FLEX Inc. Review manual, 2005). Interface properties of geonet with the compost samples were obtained from the direct shear tests conducted in this study by placing the geonet in between the shear boxes which are filled with the compost. The experimental set up is shown in Figure 12. The direct shear test was run following the normal procedure.

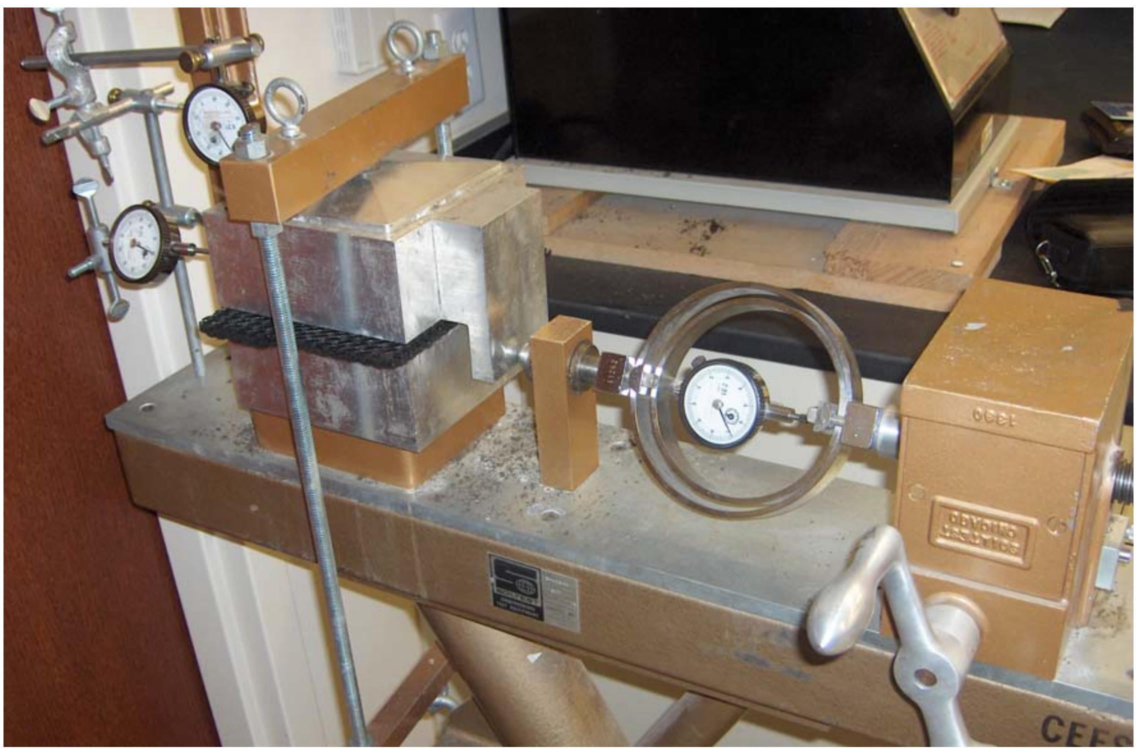


Figure 12: Direct Shear Test Assembly for Testing Interface Properties

3.8 Modeling and Slope Stability Analysis of Landfills using SLOPE/W software

The values of cohesion, friction angle and the unit weight obtained from the direct shear tests for the various samples were used as input parameters for conducting the slope stability analysis of a landfill slope, using the software called SLOPE/W which is a part of a commercially available software family called GEO-SLOPE (GEO-SLOPE International Ltd., 1999).

3.8.1 Overview of SLOPE/W – Theoretical Basis

SLOPE/W uses the theory of limit equilibrium of forces and moments to compute the factor of safety against failure. The assumptions in this limit equilibrium formulation are:

1. The factor of safety of the cohesive component of strength and the frictional component of strength are equal for all soils involved and
2. The factor of safety is the same for all slices.

For an effective stress analysis, the shear strength of the material may be defined as:

$$S = c' + (\sigma_n - u)\tan\phi'$$

where,

S = shear strength

c' = effective cohesion

φ' = effective angle of internal friction

σ_n = total normal stress and

u = pore water pressure

For a total stress analysis, the strength parameters are defined in terms of total stresses and pore water pressures are not required. In that case,

$$S = c + \sigma_n \tan\phi$$

The stability analysis involves passing an imaginary slip surface through the landfill and dividing the inscribed portion into vertical slices. The slip surfaces may be circular, composite (i.e., combination of circular and linear portions) or consist of any shape defined by a series of straight lines (i.e., fully specified slip surface).

Next, as shown Figure 13, the forces acting on all the slices are identified.

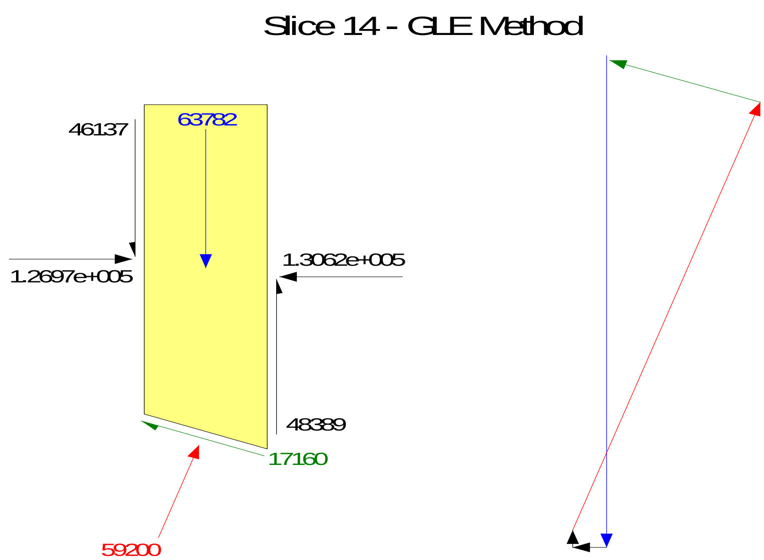


Figure 13: Forces acting on a sample slice and the force polygon representing the equilibrium of the forces

The ultimate aim is to find the factors of safety along the chosen slip surface. In achieving this, the following equations of statics from fundamental mechanics are used:

1. The summation of forces in a vertical direction for each slice. The equation is solved for the normal force at the base of the slice.
2. The summation of forces in a horizontal direction for each slice is used to compute the interslice normal force. This equation is applied in an integration manner across the sliding mass.
3. The summation of moments about a common point for all slices. The equation can be rearranged and solved for the moment equilibrium factor of safety, F_m.
4. The summation of forces in a horizontal direction for all slices, giving rise to a force equilibrium factor of safety, F_f.

Even after the application of all these equations, the overall analysis still remains indeterminate, and a further assumption is made regarding the direction of the resultant interslice forces. The direction is assumed to be described by an interslice force function. The factors of safety can now be computed based on moment equilibrium (F_m) and force equilibrium (F_f). These factors of safety may vary depending on the percentage (λ) of the force function used in the computation.

The factor of safety satisfying both moment and force equilibrium is considered to be the converged factor of safety of the GLE method. Using the same GLE approach, it is also possible to specify a variety of interslice force conditions and satisfy only the moment or force equilibrium conditions according to the various commonly used methods of slices as discussed in the literature review section.

3.8.2 Modeling and Analysis using SLOPE/W

Having discussed the theoretical basis of the SLOPE/W software, an overview of the modeling of a typical landfill and the subsequent stability analysis is now discussed. SLOPE/W contains a user-friendly Graphical User Interface to prepare the model. The Graphical User Interface window named DEFINE is used to initiate the model of the landfill. Since the main interest is in the slope stability of the landfill, only the sloping portion of the landfill is modeled. Due to symmetry, only one-half of the landfill is modeled. For the present work, typical Floridian landfill slopes of 1:3 and 1:4 are used in the model. The profile of the slope is drawn using the 'sketch' and 'draw' tabs. A typical landfill model is shown in Figure 14, having several layers which are discussed in the following paragraph.

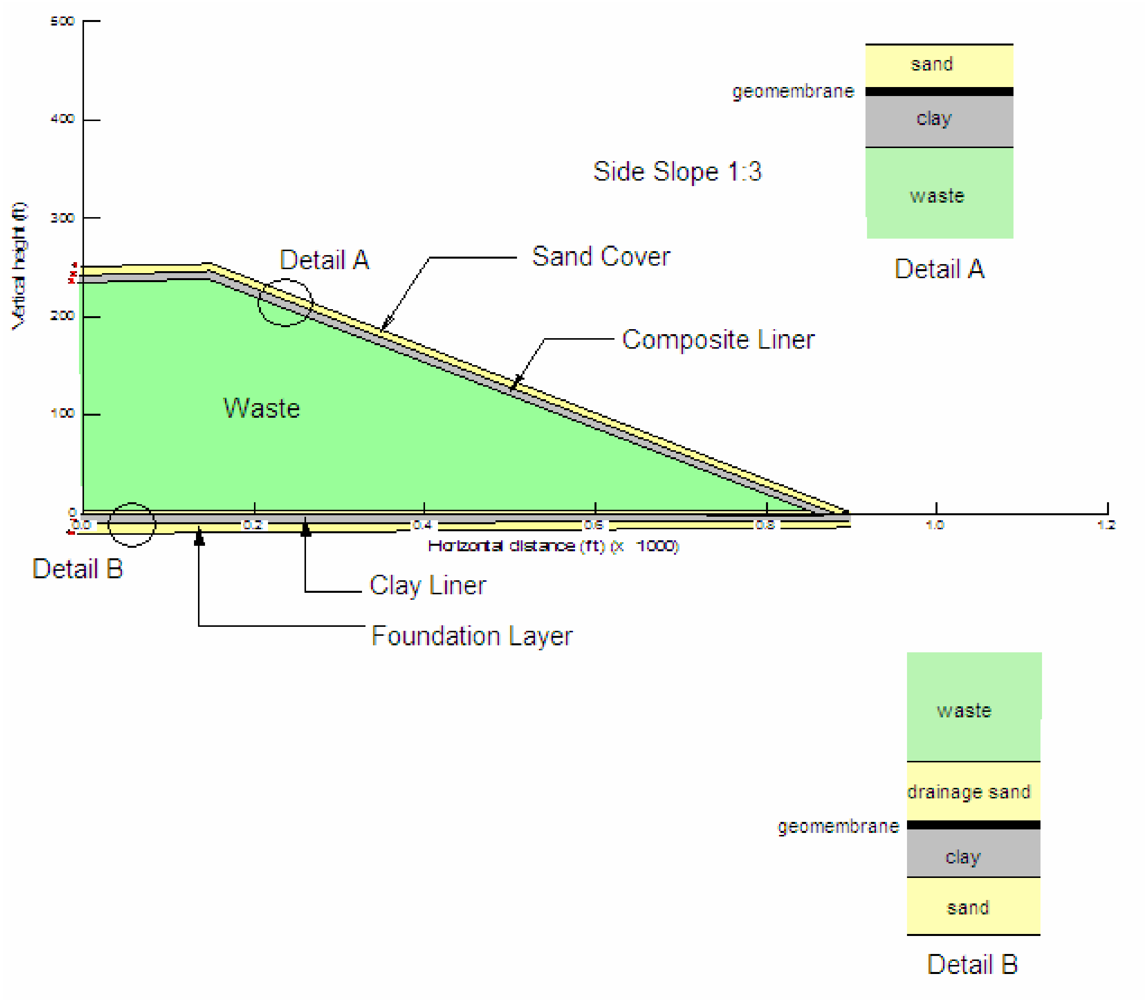


Figure 14: General profile of the landfill used in the modeling

As can be seen from Figure 14, waste (MSW mixed with biosolids) is covered by a composite liner and then overlain by a sand cover layer. Some other variations are also possible based on the specific needs of the landfill. For example, a bioreactor landfill may require some modifications. For the present modeling, only two layers namely, the composite liner and soil cover are modeled for simplicity. In addition, the thicknesses of these layers are slightly exaggerated in the model; or else the inclusion of such small layers into the model was almost impossible using this software.

In the bottom section (Detail B), there is a composite liner below the waste mass and the composite liner is underlain by the foundation soil. The grey colored layer in Figure 14 is a composite liner. The green colored mass is the landfill material and the top yellow colored layer represents the protection layer. Finally, the bottom yellow colored layer represents the foundation soil. The grey colored layer is modeled as a composite liner containing a geomembrane interfaced with clay in the bottom and sand in the top. Interface properties for these combinations are taken from published data (Qian et al, 2002). Typical cohesion and friction angle values are taken for the sand layer. These values are presented in Table 2. For the landfill material, results from the direct shear tests on composts were taken.

Table 2: Landfill Layers – Model Input Values

Layer	*Cohesion/Adhesion*	*Friction Angle*	*Unit Weight*
Sand Cover	0	40	110 pcf
Geomembrane/Sand	25psf	37	140pcf
Geomembrane/Clay	120 psf	29	150 pcf
Waste	*	*	*
Sand/Geomembrane	25 psf	37	140 pcf
Geomembrane/Clay	120 psf	29	150 pcf
Foundation Layer	0	40	110 pcf

* denotes experimental results

After entering the soil properties and selecting the method of analysis, a range of trial slip circles have to be defined. It is enough to specify the radius and the center of the circle and the software generates a number of potential slip circles. It is important that the range of radii and centers must span all potential areas of the landfill where failure surfaces may develop. The radii and surfaces of the slip circles are shown in Figure 15.

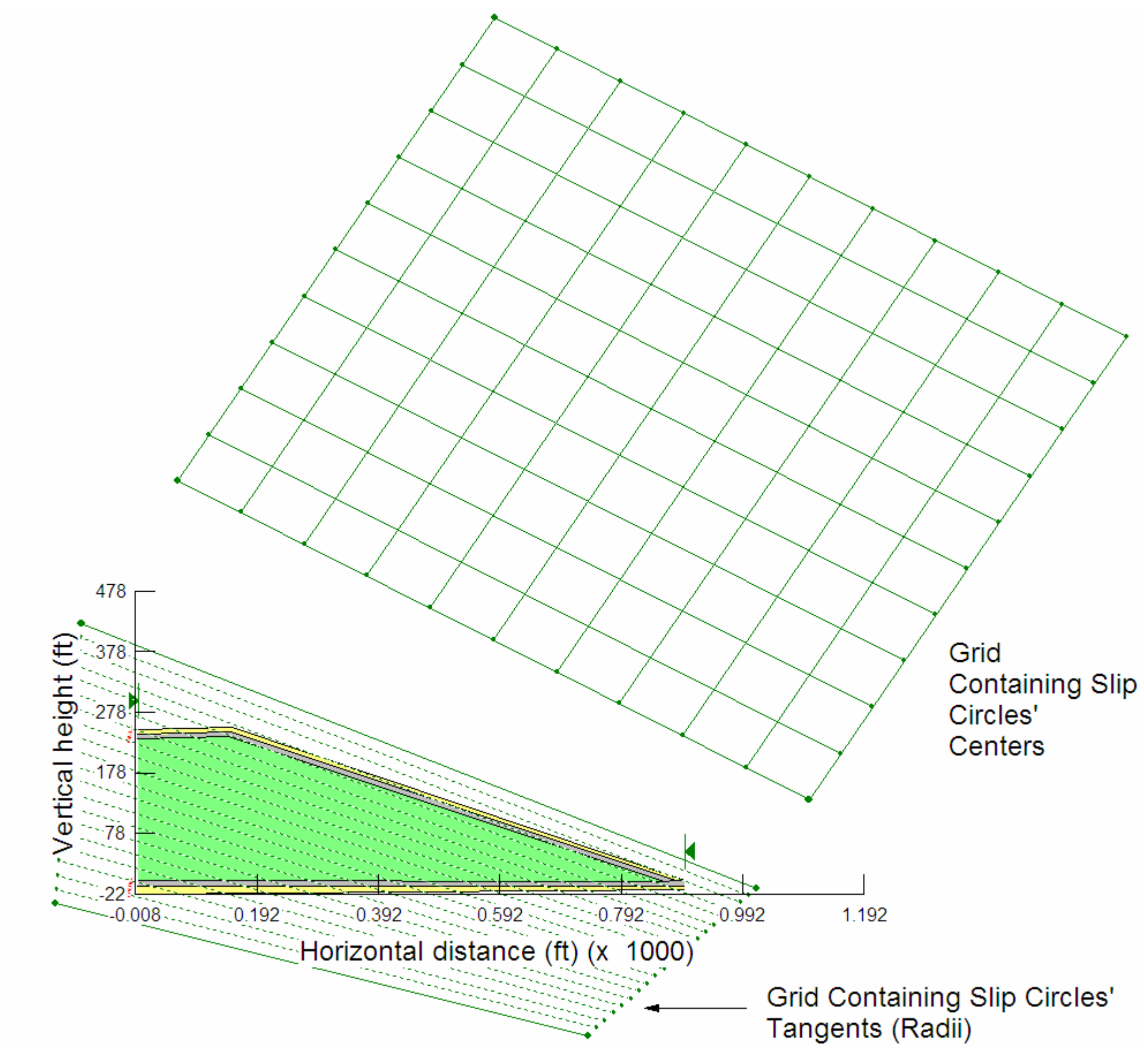


Figure 15: Slip circles' centers and their radii

The interesting thing about this program is that combinations resulting in as many as 10,000 circles at a given time can be specified and the failure surface can be obtained within a few seconds. Thus, a grid of centers of circles and a contour of tangents or slip surfaces from which radii can be found should be specified. Then, the software program has to be verified for data input errors or data insufficiency from the ‘verify’ option in the ‘Tools’ menu. If there are no errors, the program allows us to execute the analysis and solve for the factor of safety using the various methods discussed previously. The minimum factor of safety is then computed and displayed at the location of the corresponding center of the slip surface as seen in Figure 16. The different contours of the factors of safety, corresponding slip surfaces, slices associated with that slip surface and forces acting on each slice can be viewed using the CONTOUR tab.

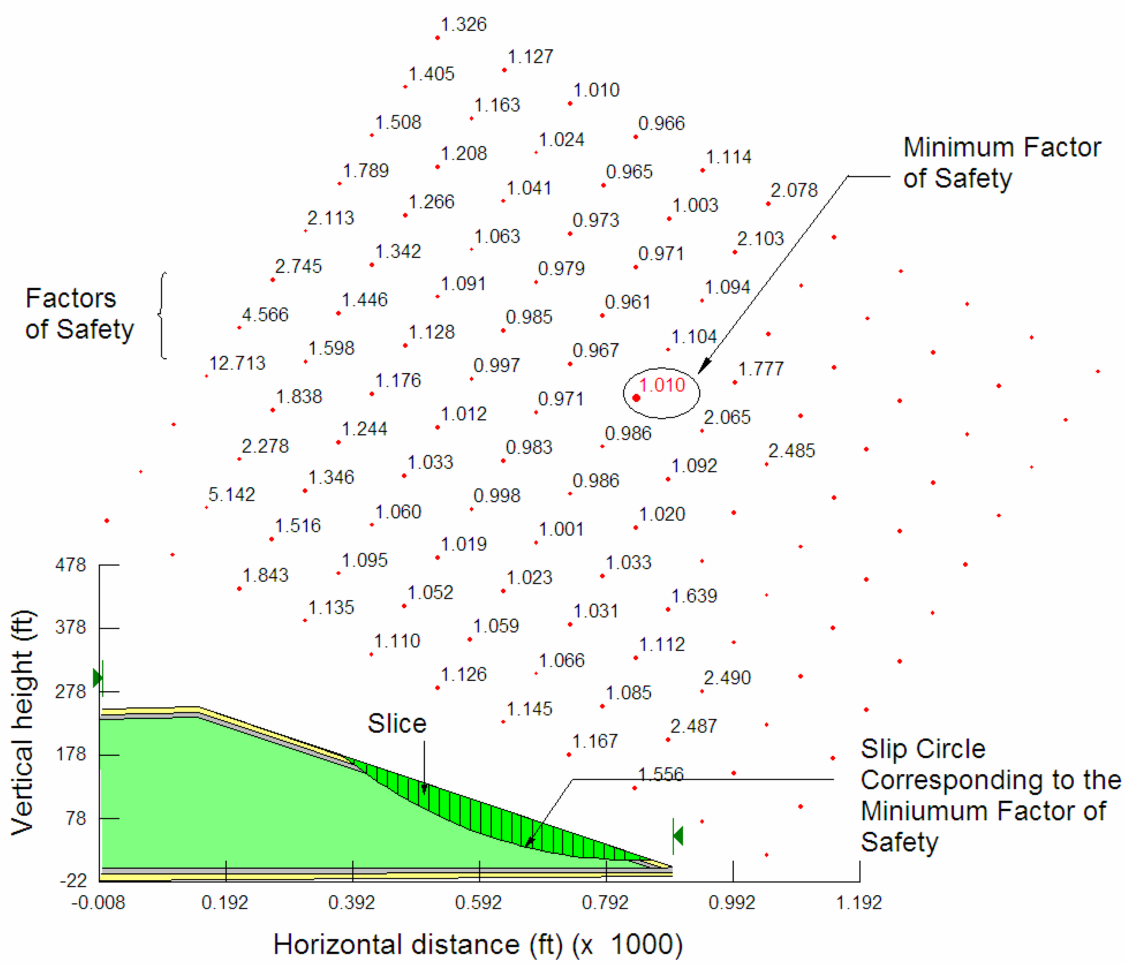


Figure 16: Minimum Factor of safety and corresponding slip circle

The CONTOUR window is well designed and can display the results from a micro level to a macro level, right from the force acting on an individual slice to the minimum factor of safety corresponding to a specified method of analysis. Summation of forces along a slice is shown in Figure 17.

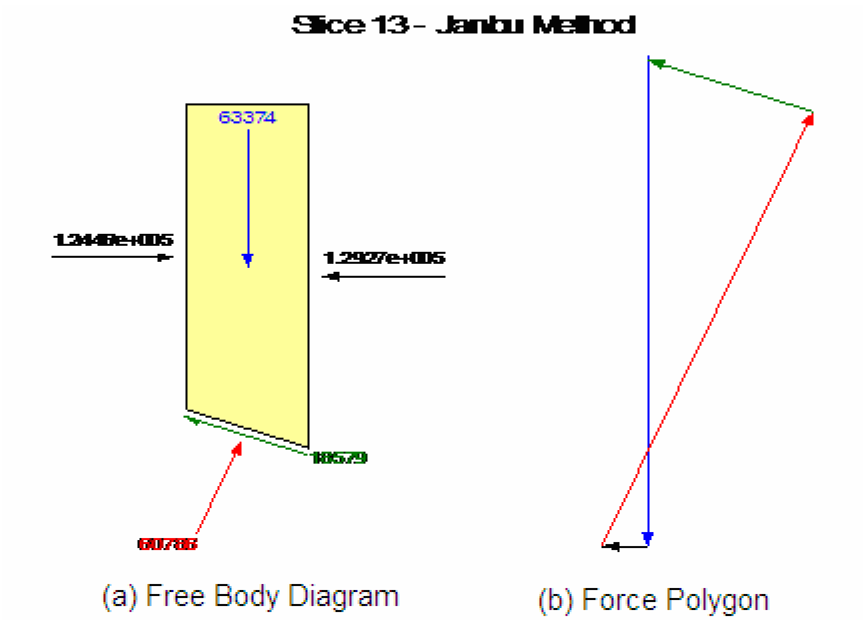


Figure 17: Summation of forces acting on a sample slice

The factors of safety, thus obtained are tabulated and presented in chapter 4.

CHAPTER FOUR: RESULTS OF TESTING AND MODELING

4.1 Introduction

In this chapter, the results of the direct shear tests conducted on individual samples of compost representing old landfill material are presented. They are compared to the properties of new landfill materials using shear tests conducted previously by Koodathinkal (2003). In addition, the results of the interface tests conducted with geonet and two of the compost samples are presented. Lastly, modeling and analysis results from SLOPE/W for landfill slopes with two different inclinations are presented.

4.2 Tests conducted on the Cured Compost Samples Alone

Direct shear tests were conducted using the modified large scale shear mold as described in section 3.5, on the cured compost samples from the Sumter County Composting facility. The samples had the following ages – 1 month (cured compost sample - 1), 12-months (cured compost sample - 2), 13-months (cured compost sample - 3) and16-months (cured compost sample - 4). The composition and sampling of these samples were presented in section 3.3.

4.2.1 Cured Sample - 1

Specimen from cured sample - 1 was placed in the shear mold and compacted. The mold was then placed in the direct shear test assembly as shown in Figure 10. The direct shear test was run according to the procedure described in section 3.5. Moisture content and unit weight of the sample were determined as described in section 3.4. During the direct shear test, normal loads of 1013g, 2014g, 3019g, 4000g and 5013g were applied. The detailed test readings were recorded, and are tabulated in Table A-1.1. Normal stresses and the corresponding shear stresses at failure were calculated by dividing the corresponding loads by the shear box cross section area. These values are tabulated in Table 3 and a plot of shear stress at failure versus normal stress is shown in Figure 18. A straight line regression curve best fitting the data was generated, whose intercept with the shear stress axis provides the cohesive strength, or cohesion, of the sample and the angle it makes with the positive normal stress axis provides the angle of internal friction of the specimen. For this sample, a cohesion of 0.363 psi and an angle of internal friction of 16.08 degrees were obtained.

Table 3: Direct Shear Test Results for Cured Sample - 1

Normal Stress (psi)	Shear Stress (psi)
0.074	0.39
0.147	0.401
0.219	0.422
0.292	0.443
0.366	0.474
C = 0.363 psi	
Φ = 16.08 degrees	
Moisture Content = 47.94%	

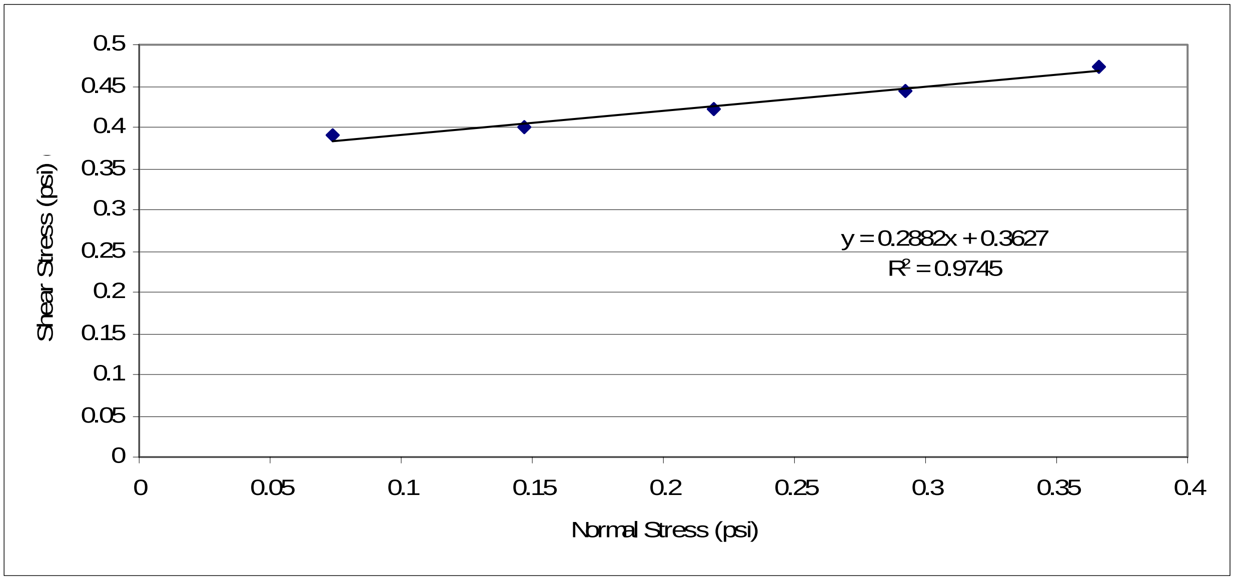


Figure 18: Variation of shear stress with normal stress (Cured Sample - 1)

Similarly, direct shear tests were conducted on cured samples-2, 3 and 4 respectively. An effort was made to ensure the specimen material that was placed in the shear box was a representative of the actual sample using visual inspection of size and composition. Table 4 lists the entire set of direct shear tests conducted on the cured samples alone, and the location of the test readings and the final results in this thesis.

Table 4: List of Direct Shear Tests Conducted

Sample Description	*Detailed Test Readings*	*Moisture Content Data*	*Unit Weights Data*	*Summary of the Test Results*	*Graph of Shear Stress vs. Normal Stress*
Cured Sample - 1	Table A-1.1	Table B-1.1	Table B-2	Table 3	Figure 18
Cured Sample - 2	Table A-1.2	Table B-1.2	Table B-2	Table 5	Figure 19
Cured Sample - 3	Table A-1.3	Table B-1.3	Table B-2	Table 6	Figure 20
Cured Sample - 4	Table A-1.4	Table B-1.4	Table B-2	Table 7	Figure 21

4.2.2 Cured Sample - 2

Table 5: Direct Shear Test Results for Cured Sample - 2

Shear Stress (psi)	Normal Stress (psi)
0.074	0.527
0.147	0.559
0.219	0.59
0.292	0.611
0.366	0.632
c=.505psi	
Φ = 19.76 degrees	
Moisture Content = 57.30%	

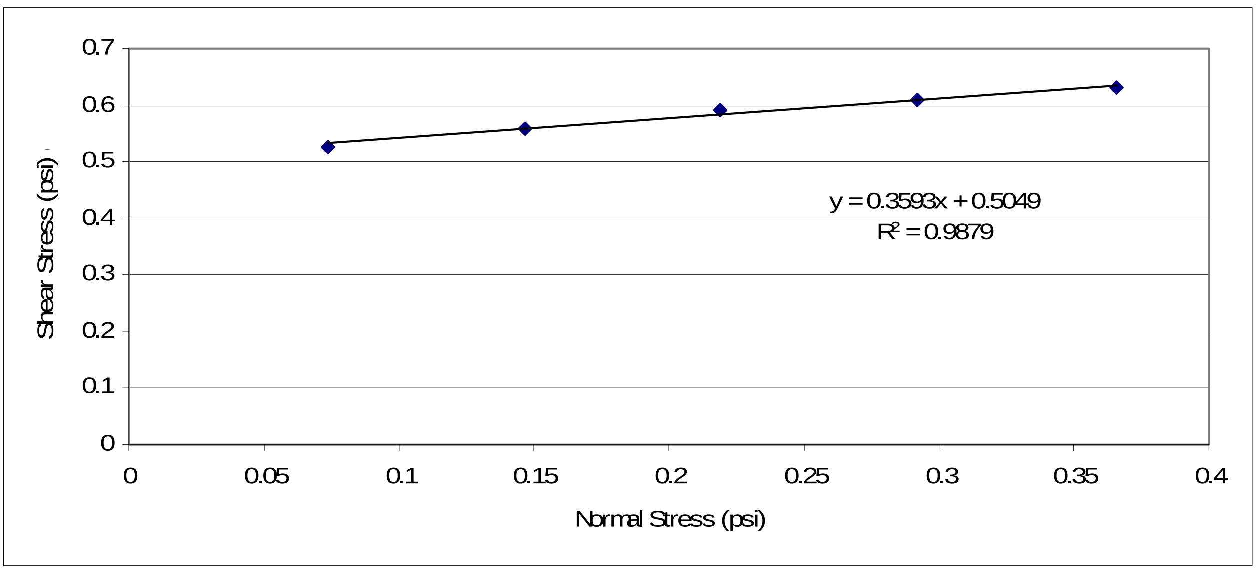


Figure 19: Variation of shear stress with normal stress (Cured Sample - 2)

For the cured sample - 2, the cohesion was found to be 0.505 psi and the angle of internal friction was found to be 19.76 degrees.

4.2.3 Cured Sample - 3

Table 6: Direct Shear Test Results for Cured Sample - 3

Normal Stress (psi)	Shear Stress (psi)
0.043	0.348
0.073	0.411
0.116	0.432
0.147	0.464
0.219	0.464
c = 0.3504 psi	
Φ = 31.55 degrees	
Moisture Content = 66.56 %	

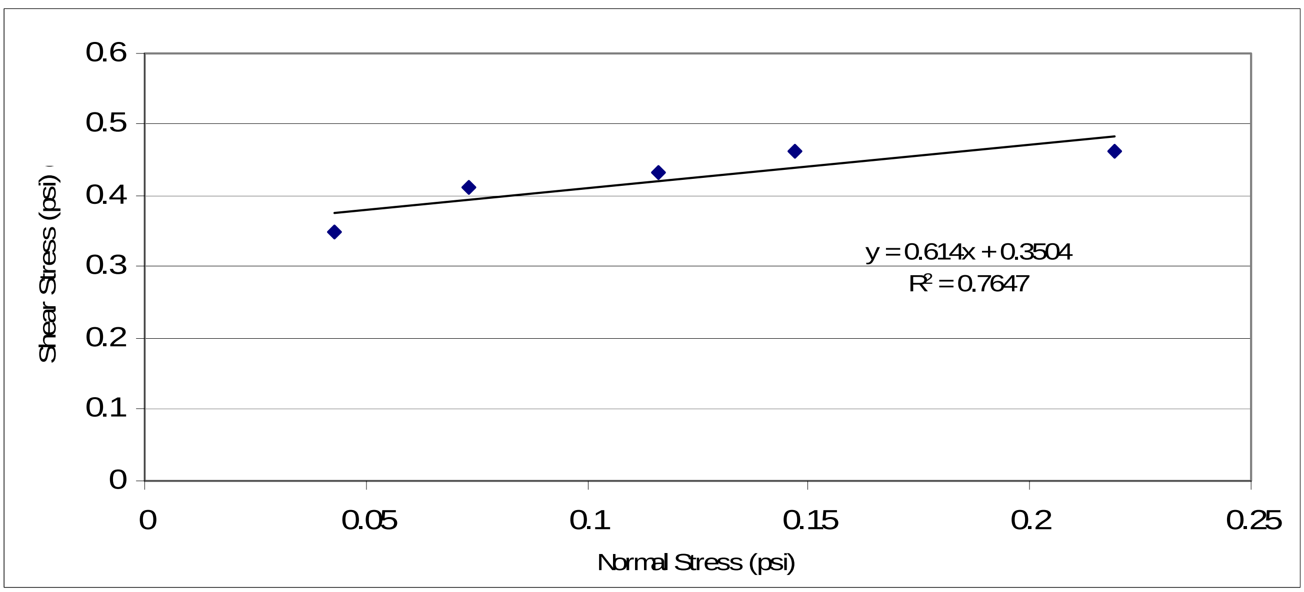


Figure 20: Variation of shear stress with normal stress (Cured Sample - 3)

For the cured sample - 3, the cohesion was found to be 0.3504 psi and the angle of internal friction was found to be 31.55 degrees.

4.2.4 Cured Sample - 4

Table 7: Direct Shear Test Results for Cured Sample - 4

Normal Stress (psi)	Shear Stress (psi)
0.073	0.390
0.146	0.432
0.189	0.432
0.219	0.464
c = 0.3578 psi	
Φ = 24.58 degrees	
Moisture Content = 66.57 %	

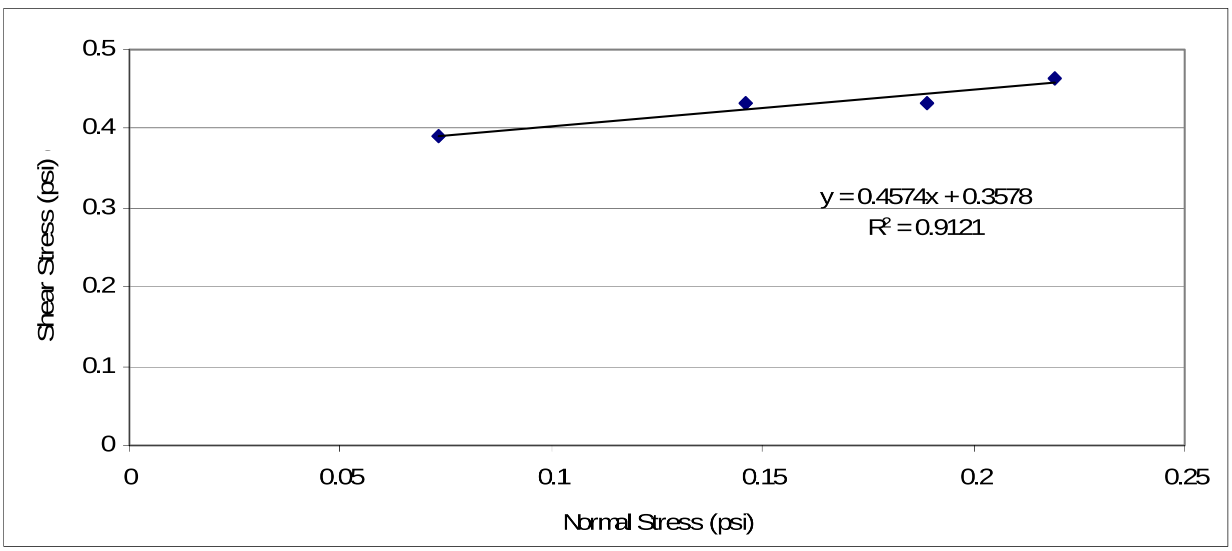


Figure 21: Variation of shear stress with normal stress (Cured Sample - 4)

For the cured sample - 4, the cohesion was found to be 0.3578 psi and the angle of internal friction was found to be 24.58 degrees.

A summary of the above results is presented in Table 8.

Table 8: Summary of the Direct Shear Tests on Different Samples

Sample Description	*Age (months)*	*Cohesion (psi)*	*Angle of Internal Friction (degrees)*
Cured Sample - 1	1	0.363	16.08
Cured Sample - 2	12	0.505	19.76
Cured Sample - 3	13	0.350	31.55
Cured Sample - 4	16	0.358	24.58

From Table 8, it is observed that cohesion and friction angle do not vary substantially with time over the time period studied for the compost. Since the changes in the geotechnical strength properties of the cured compost samples is not significant in the time span observed, average values of cohesion (0.394 psi = 56.74 psf, standard deviation = 10.68 psf) and angle of internal friction (23.00 degrees, standard deviation = 6.68 degrees) are computed and are assumed to represent the geotechnical properties of an old landfill. Average values of normal stresses and corresponding shear stresses in the experiments conducted are plotted in Figure 22, which also contains the shear strength data on MSW obtained from laboratory and in-situ testing supplemented by the strengths obtained from back-analysis of existing stable solid waste landfill slopes (Kavazanjian et al., 1999).

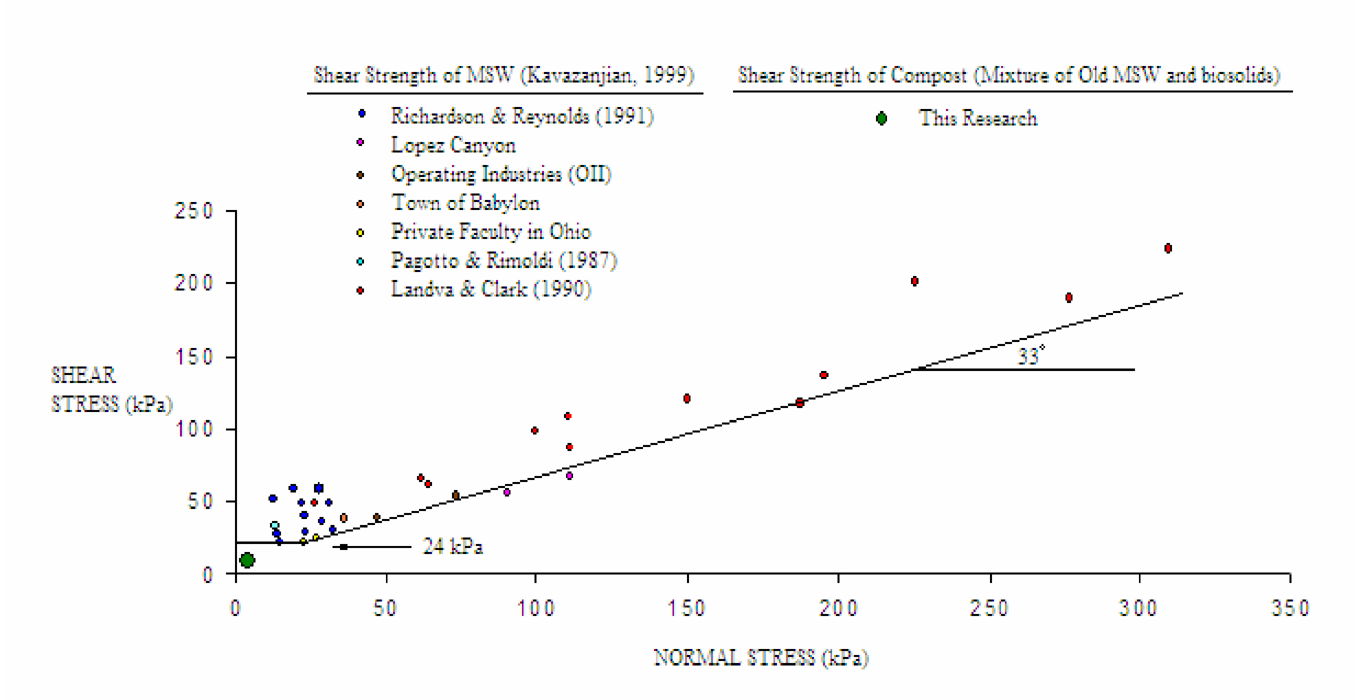


Figure 22: Shear strength of Waste

From Figure 22, it is observed that for the range of normal stresses applied in this research, the cohesion value obtained (0.394 psi = 2.72 kPa) is less than 24 kPa and the value of friction angle (23 degrees) is more than 0 degrees, which were suggested by Kavazanjian (1999). He was referring to the tests conducted on MSW and the direct shear tests conducted in this research were on compost, which represent very old MSW. Moreover, 25% biosolids were present in the compost.

Koodathinkal (2003), conducted direct shear tests on freshly prepared artificial samples of MSW mixed with 30% biosolids and reported that the mixture has a cohesion value of 674 psf and an angle of internal friction of 11.21 degrees. This test was conducted at a moisture content of 60% (wet basis). The average value of the moisture contents of the cured samples conducted in the present research is also 60% (wet basis). Assuming that the 5% difference in the biosolids proportion does not affect the geotechnical strength properties significantly, a comparison of these two results may be used as an indication of the change in these properties with time. These values are compared in Table 9 and the time variation of cohesion and friction angle are depicted in Figure 23.

Table 9: Comparison of Geotechnical strength properties of Fresh and Cured Landfill Materials

Landfill Material	*Cohesion (psf)*	*Friction Angle (degrees)*	*Source*
Fresh (New)	674.00	11.21	Koodathinkal (2003)
Cured (Old)	56.74	23.00	This Research

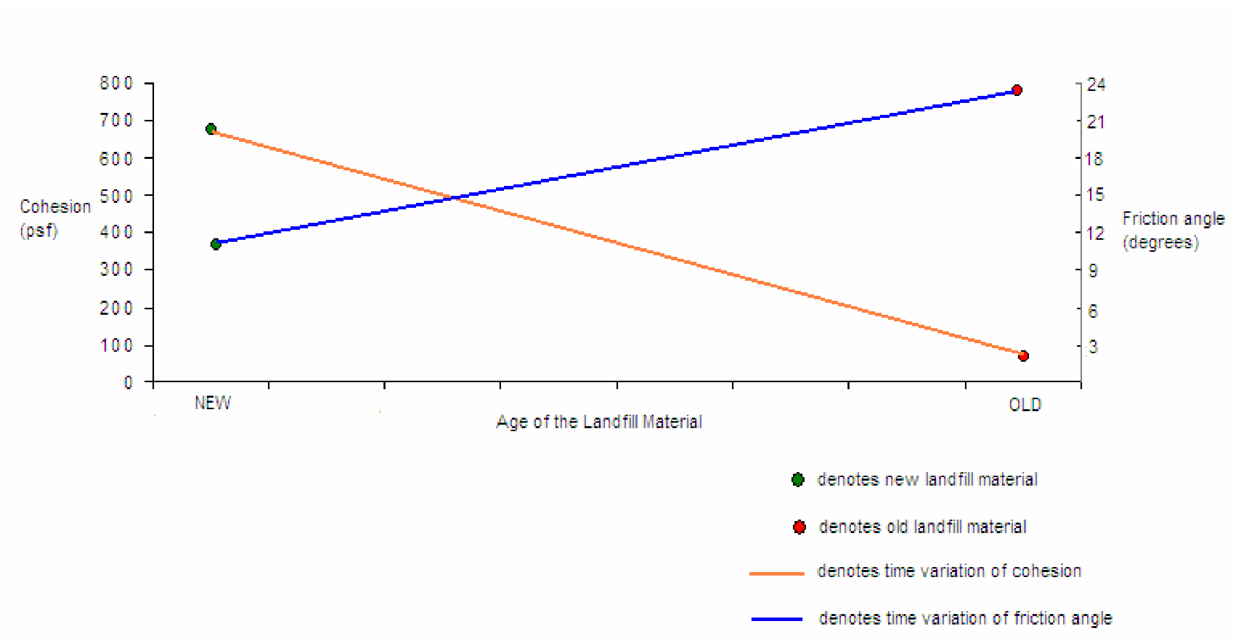


Figure 23: Time variation of cohesion and friction angle of landfill materials

From Table 9 and Figure 23, we observe that there is a substantial reduction in the cohesion value of the landfill material (MSW + biosolids) with time. On the other hand, the angle of friction shows a significant increase with time. The reduction in the cohesion value may be due to the decay of the softer organic material. Softer organic material binds the particles. With the decay of the softer organic material, the dense residual material has an increased strength which might be reflected by the increase in the friction angle of the material.

Figure 24 shows the cohesion and friction angle values for a variety of waste from various sources, which reveals a huge scatter in these values which may be an indication of the variety in the constituents or the heterogeneity of the waste.

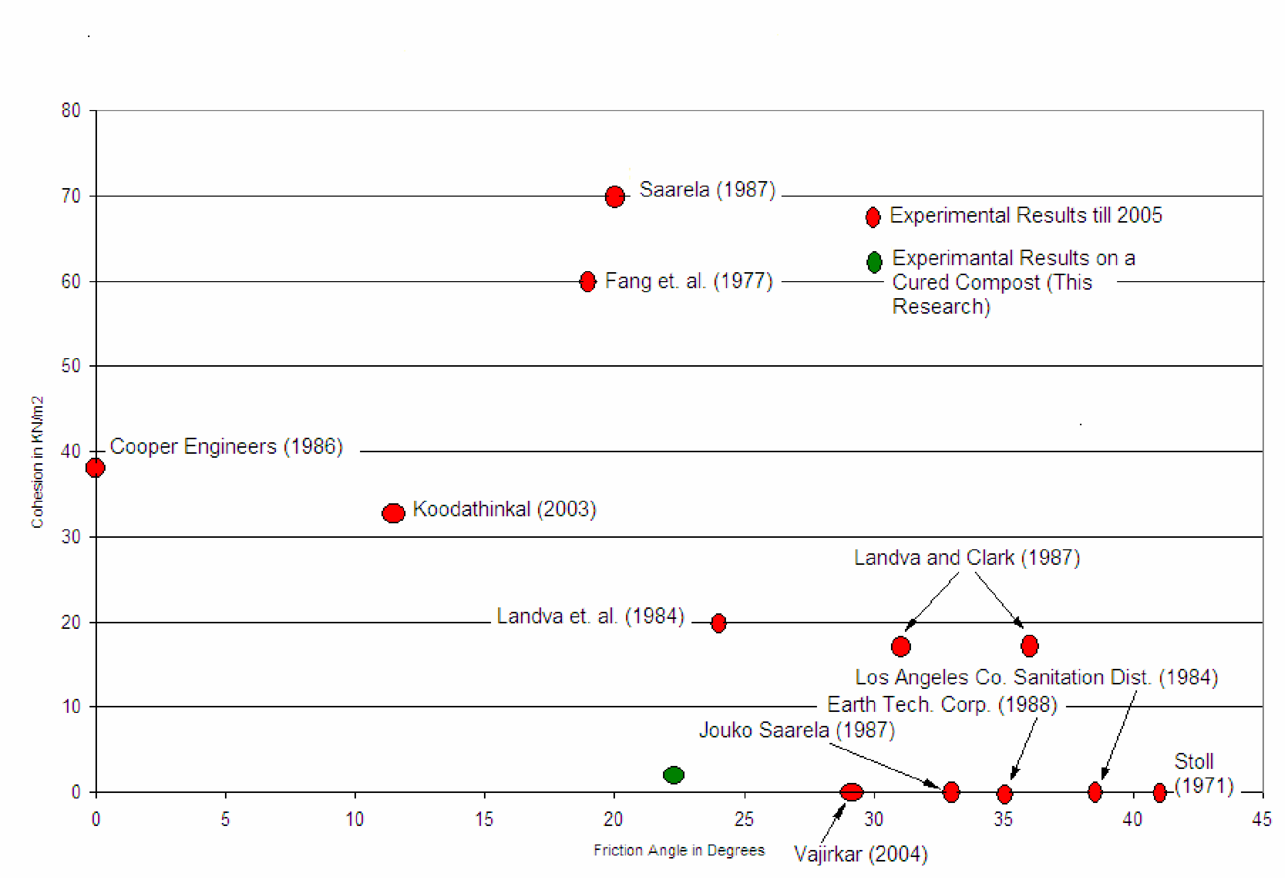


Figure 24: Cohesion and Friction angle values of waste with variety of constituents

4.3 Tests Conducted to Determine Interface Shear Strength

Geonet, which is normally used for drainage, is tested for its interface properties with cured samples, in order to check its usefulness as a reinforcing material for the slopes. The potential mechanism of reinforcement is described in section 3.6. Direct shear tests were conducted on geonet interfaced with cured samples -1 and 2. Figure 25 shows the geonet used in the direct shear test. A detailed description of the geonet is presented in Appendix C.

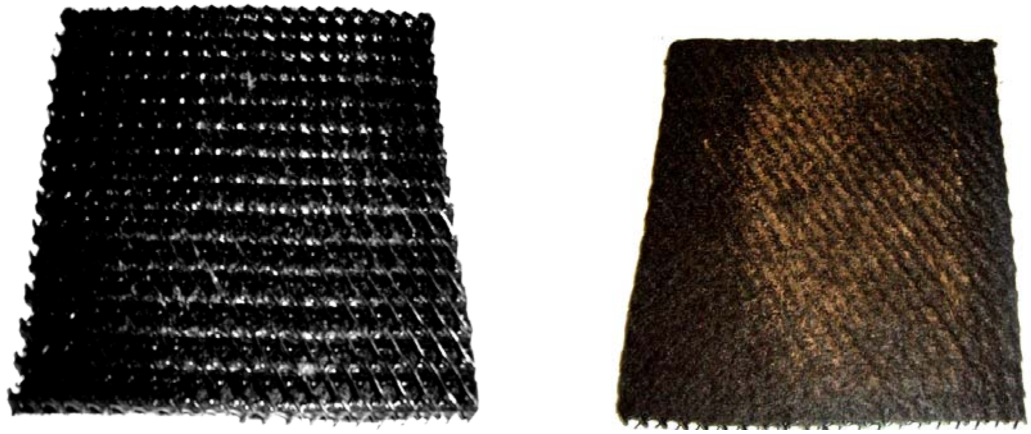


Figure 25: Geonet

The top and bottom half boxes of the direct shear test mold were filled with the cured compost samples with the geonet placed in between them. The direct shear test was run using the same procedure as described in section 3.7.

4.3.1 Interface Testing (Geonet with Cured Sample - 1)

Cured sample - 1 was placed in the lower shear box and compacted. The shear box was kept in the direct shear test assembly. The geonet was placed over the lower box and the upper shear box is placed on it. The top shear box is filled with the one month old sample and compacted. The final assembly is shown in figure 12. Direct shear test was then conducted. The moisture content of the sample was determined. In the direct shear test, increasing normal loads of 1013g, 2017.5g, 3018.5g, 4000g and 5013g were applied. The test readings were recorded and presented in the appendix in Table A-2.1. Normal stresses and corresponding shear stresses at failure are presented in Table 10 and a plot of shear stress at failure versus normal stress is shown in Figure 26. For this interface, a cohesive strength of 0.0796 psi and a friction angle of 4.55 degrees were obtained using the intercept and slope of the regression curve respectively.

Table 10: Direct Shear Test Results of Geonet Interfaced with Cured Sample - 1

Normal Stress (psi)	Shear Stress (psi)
0.074	0.116
0.147	0.158
0.219	0.19
0.292	0.242
0.366	0.264
c = 0.0796 psi	
Φ = 4.55 degrees	
Moisture Content = 49.41%	

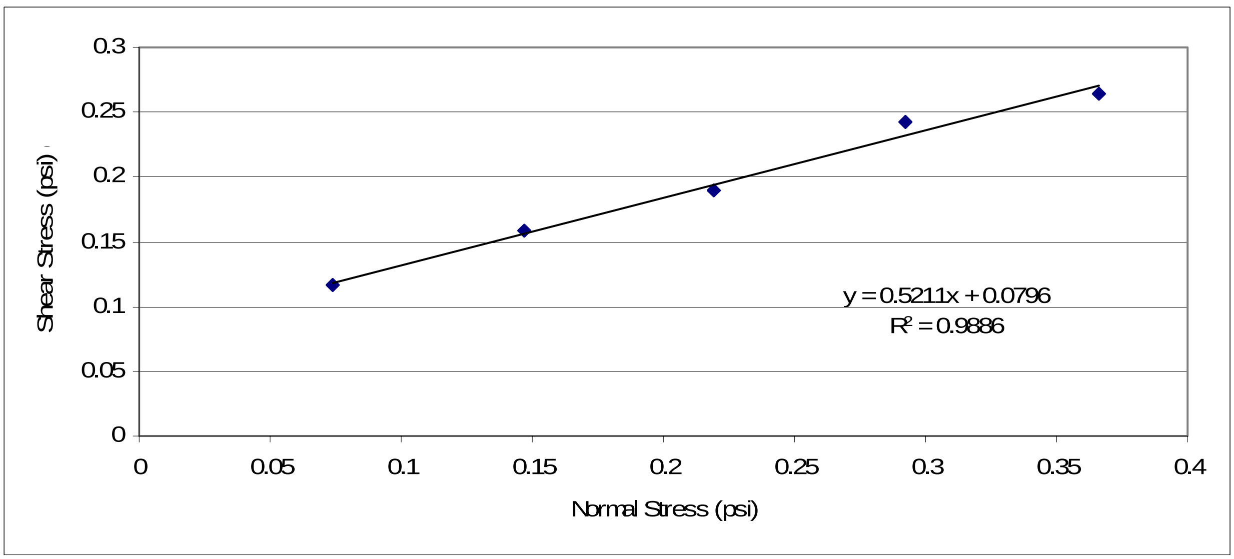


Figure 26: Variation of shear stress with normal stress (Geonet interfaced with Cured Sample - 1)

4.3.2 Interface Testing (Geonet with Cured Sample -2)

Similarly direct shear tests was conducted on geonet interfaced with cured sample - 2. Detailed test readings are presented in Table A.2.2. The results are presented below.

Table 11: Direct Shear Test Results of Geonet Interfaced with Cured Sample - 2

Normal Stress (psi)	Shear Stress (psi)
0.074	0.158
0.147	0.158
0.219	0.232
0.292	0.221
0.366	0.242
c = 0.1327 psi	
Φ = 7.56 degrees	
Moisture Content = 58.01%	

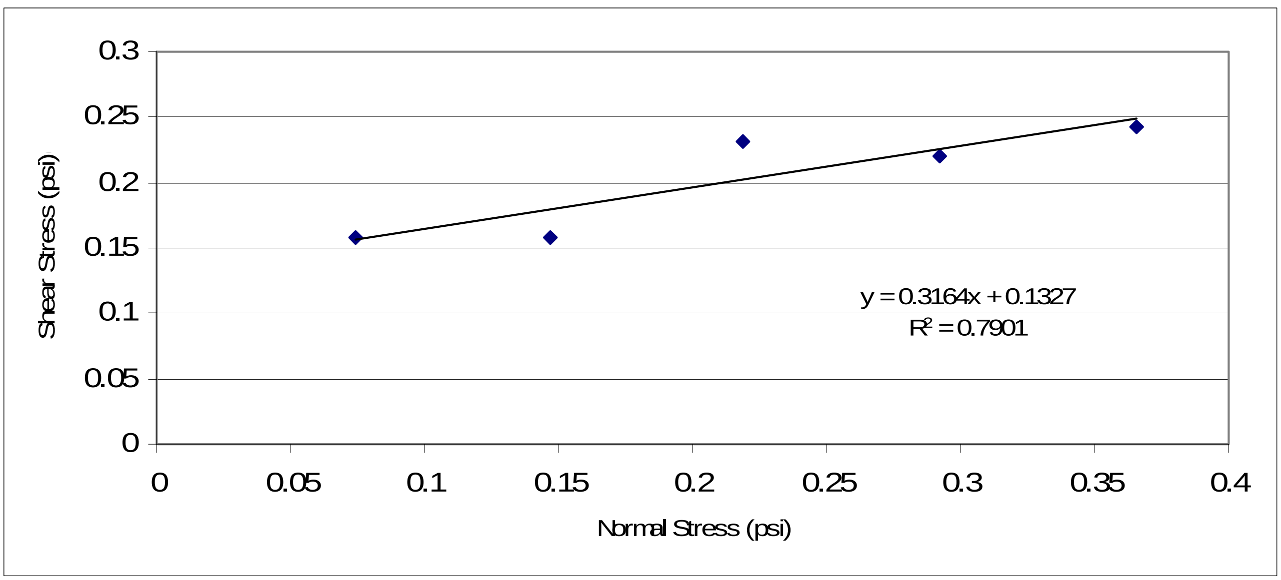


Figure 27: Variation of shear stress with normal stress (Geonet interfaced with cured sample - 2)

For this interface, a cohesive strength of 0.1327 psi and a friction angle of 7.56 degrees were obtained and using once again, the intercept and slope of the regression curve respectively.

A summary of the two interface shear testing results is presented in Table 12.

Table 12: Summary of the Results from Direct Shear Tests Conducted on Interfaces

Interface	*Adhesion (psi)*	*Friction angle (degrees)*
Geonet with cured sample - 1	0.0796	4.55
Geonet with cured sample - 2	0.1327	7.56

Very nominal increase in adhesion and friction angle for the interface properties were noted form Table 12. Hence, average values of adhesion (0.1062 psi = 15.29 psf) and a friction angle of 6.1 degrees are regarded as the interface properties of a cured sample with a geonet.

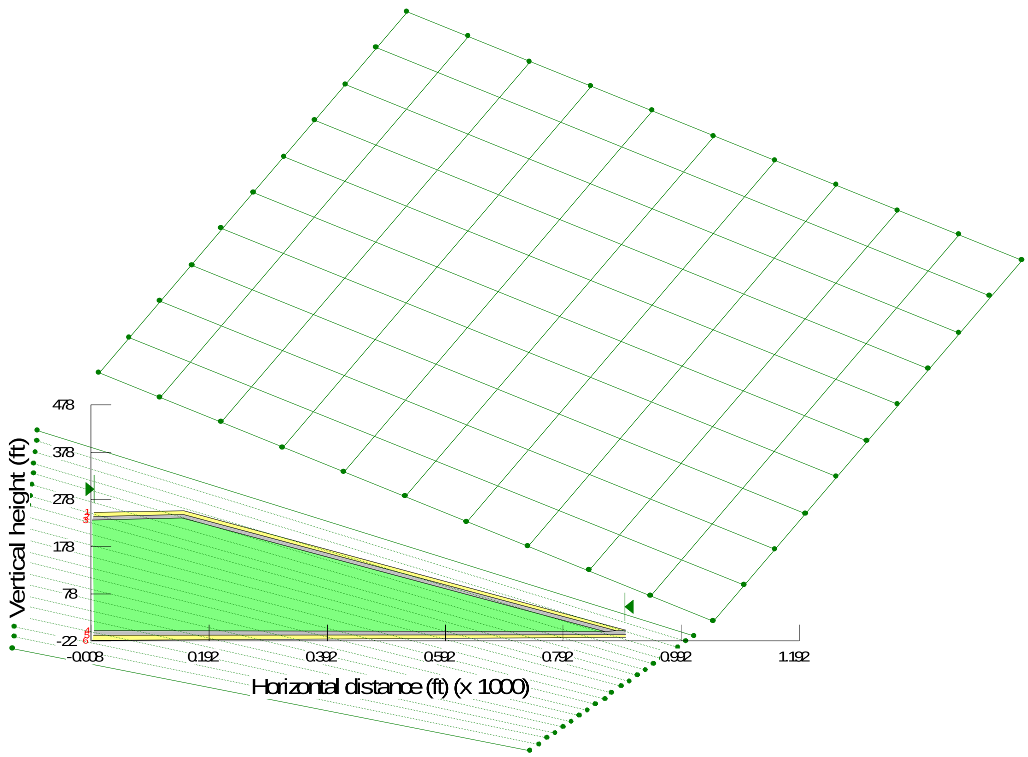
4.4 Results from Modeling and Analysis in SLOPE/W

Values of cohesion, angle of internal friction and unit weights of the landfill samples are used as input parameters to model different scenarios in SLOPE/W. Side slopes of the landfill of 1:3 and 1:4 are considered.

4.4.1 New Landfill (Slope 1:3)

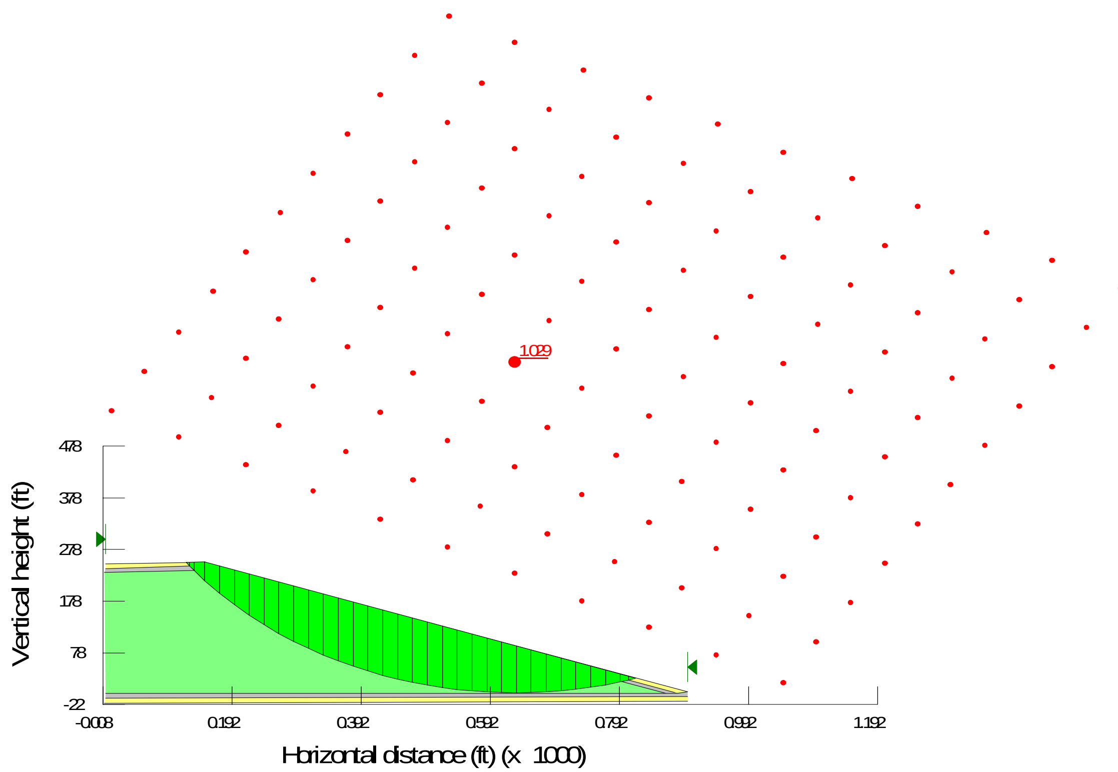
Cohesion, friction angle and unit weight values (Table 9), which are the results of the direct shear test conducted by Koodathinkal (2003) are used to define the properties of a new landfill material which is a mixture of MSW and 30 % biosolids. With a side slope of 1:3, minimum factor of safety is found to be 1.052. The model and slip surface are shown in Figure 28. This figure also displays the minimum factors of safety from the various methods of analysis.

Model:



New Landfill
Soil Model Mohr-Coulomb
Unit Weight 63 pcf
Cohesion 674 psf
Phi 11.21 degrees

Analysis results:



Factor of safety = *1.029 (J) – Janbu Method- Minimum factor of safety*
1.052 (O) – Ordinary method of slices
1.106 (B) – Bishop's method of slices
1.105 (Mp-M) – Morgenstern and Price's method – Moment based
1.106 (Mp-F) – Morgenstern and Price's method – Force based

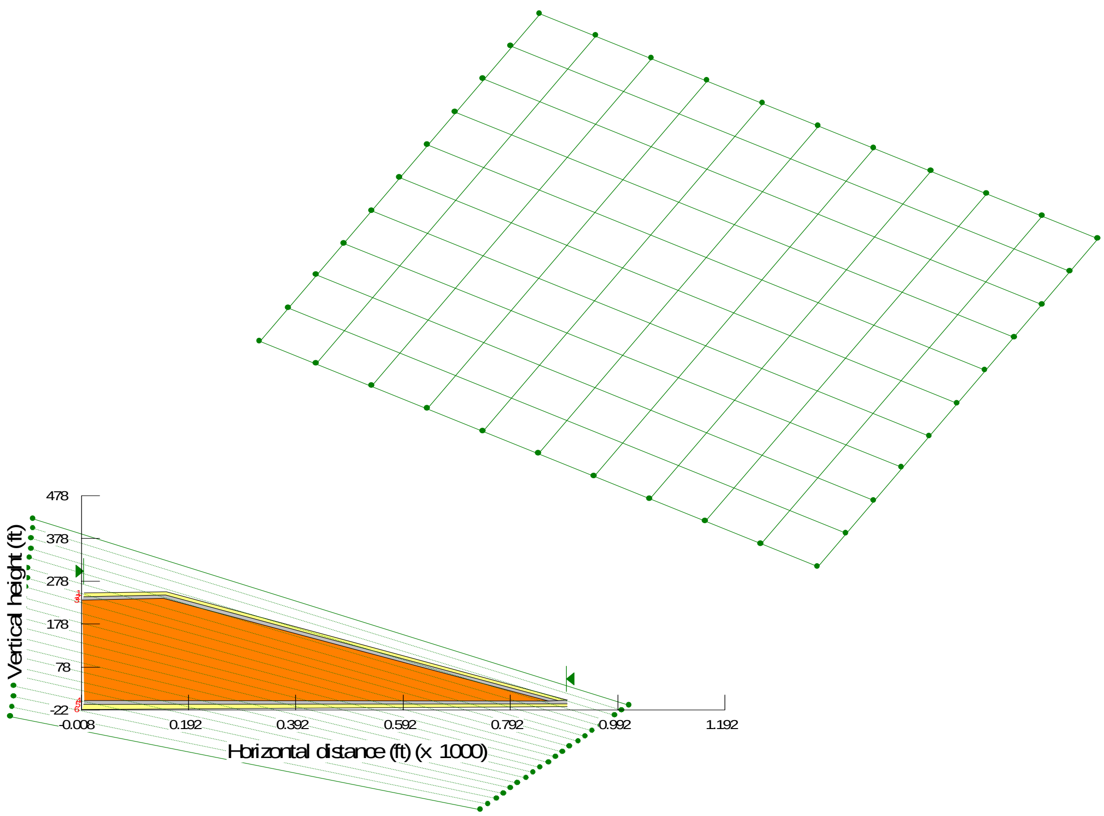
Figure 28: Stability Analysis (New Landfill)

4.4.2 Old Landfill (Slope 1:3)

Similarly, the values of cohesion, angle of internal friction from Table 9 corresponding to the old landfill material and the average unit weight value from Appendix B are used as input parameters to model the old landfill slope. The values of input material parameters and factors of safety along with the minimum factor of safety are presented in Figure 29.

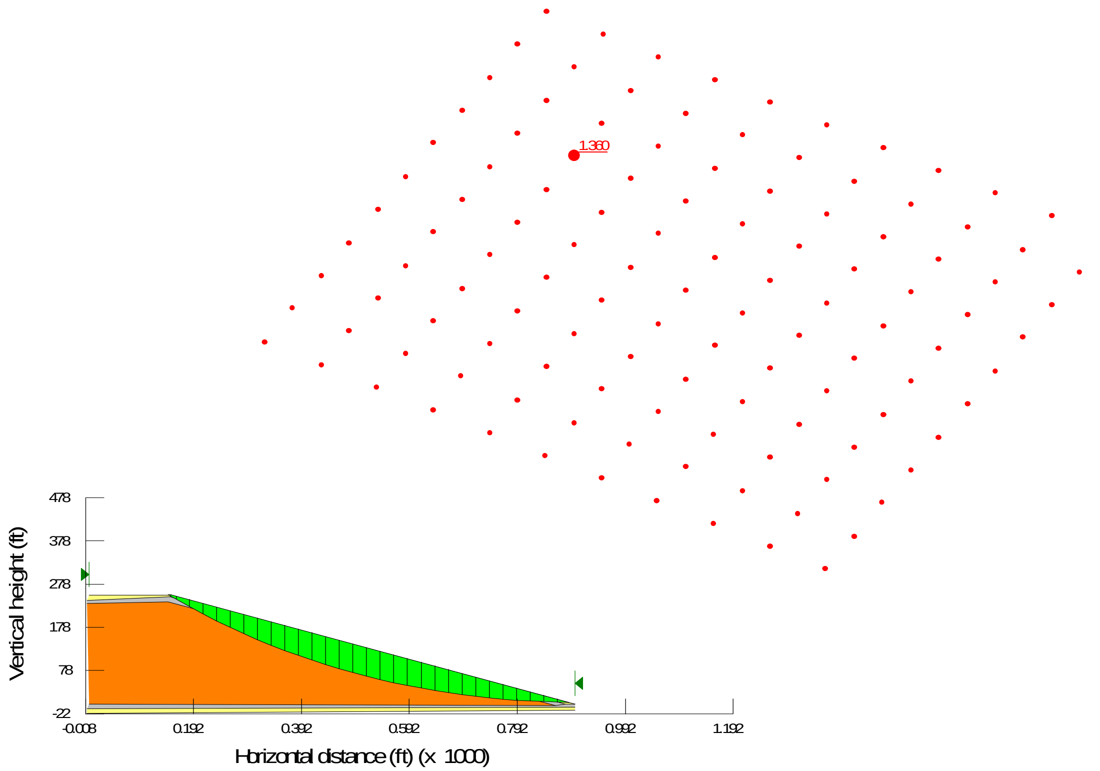
For a side slope of 1:3, the factor of safety is found to be 1.360.

Model:



Old Landfill
Soil Model Mohr-Coulomb
Unit Weight 62.83 pcf
Cohesion 56.74 psf
Phi 23 degrees

Analysis Results:



Factor of safety = *1.360 (J) – Janbu Method- Minimum factor of safety*
1.360 (O) – Ordinary method of slices
1.393 (B) – Bishop's method of slices
1.393 (Mp-M) – Morgenstern and Price's method – Moment based
1.393 (Mp-F) – Morgenstern and Price's method – Force based

Figure 29: Stability Analysis (Old Landfill)

In the above analyses, the sand layer which is used as a top protection layer and bottom foundation layer is assumed to have a unit weight of 110 pcf, an angle of internal friction of 40 degrees and a zero cohesion value which are typical for normal sand. The composite liner is modeled as a geomembrane interfaced with clay in the bottom and sand in the top (Figure 14). The model input values are presented in Table 2

Minimum factors of safety for the two cases from the above analyses are compared in Table 13, assuming that the 5% difference in the biosolids content between both the samples does not have a significant influence.

Table 13: Summary of Slope Stability Analyses (side slope 1:3)

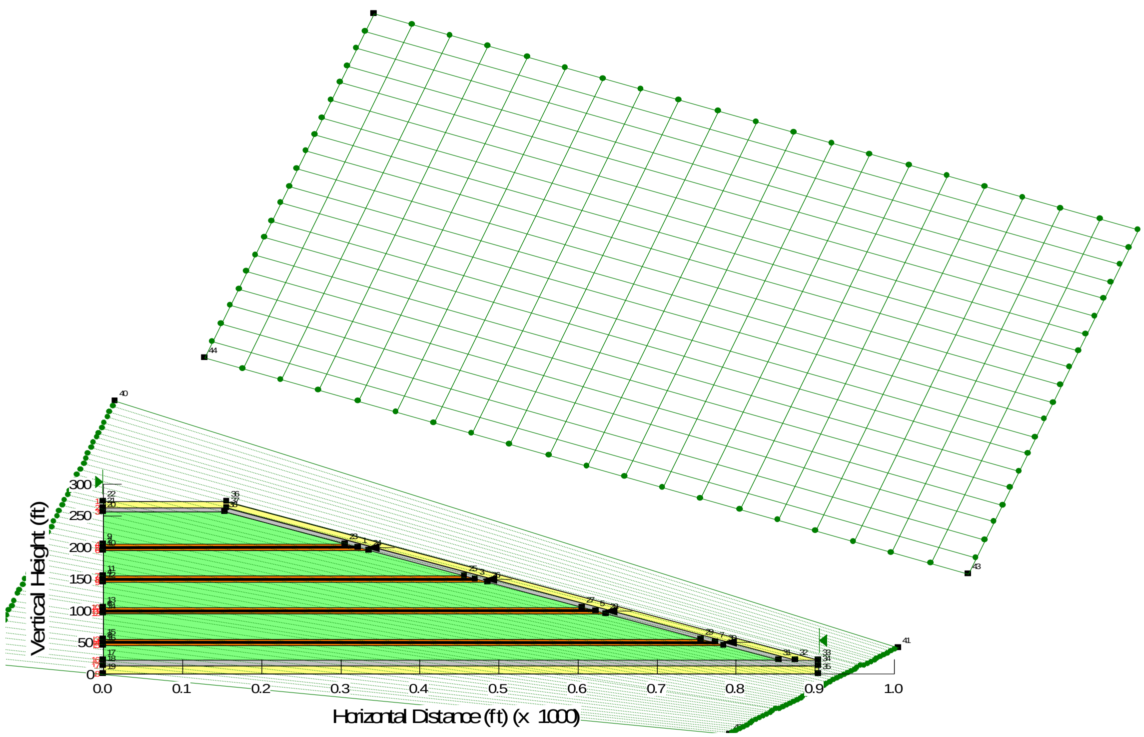
Landfill	*Minimum Factor of Safety*
New	1.029
Old	1.360

From Table 13, it is observed that factor of safety of the landfill materials increases over time.

4.5: Potential use of Geonet as Reinforcement

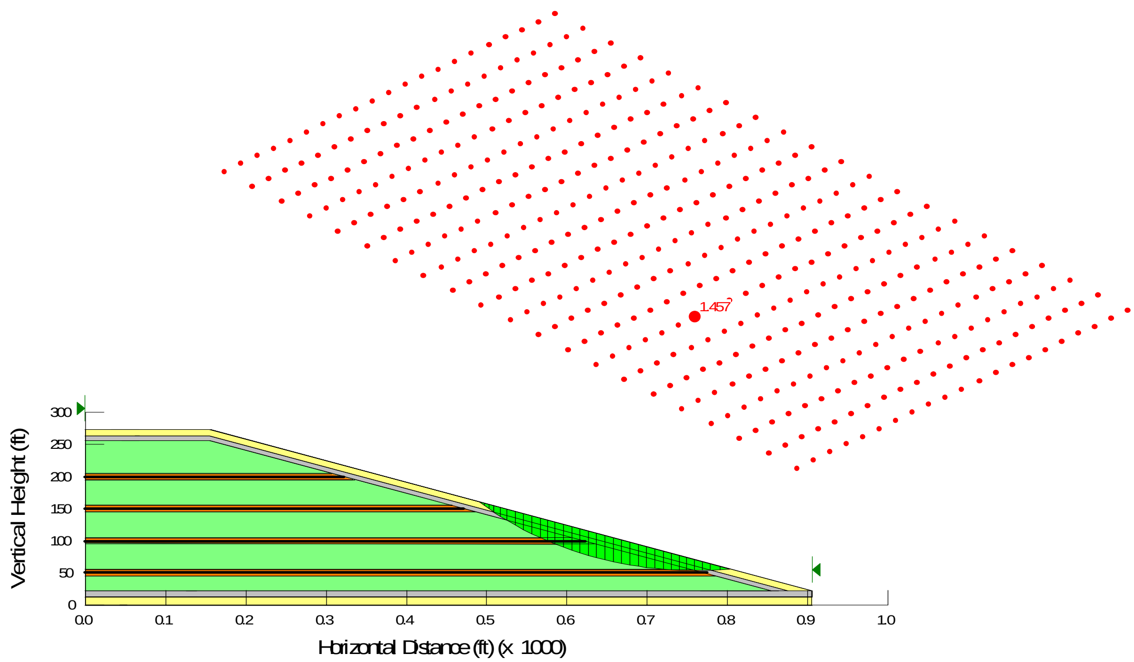
As discussed in sections 3.6, 3.7 and 4.3, direct shear test results for the interfaces of cured samples with geonet and tensile strength were used to model the landfill slope to check the suitability of geonet as reinforcement to the slope. Tensile strength value of the geonet from the manufacturer's manual (POLY-FLEX, 2005) is presented in Appendix C and is used in the modeling. Thin layers of waste on the top and bottom of the geonet are used to model the interface behavior. The properties of these layers are taken as the average values of Table 12. Because of their small size, these layers are almost invisible in Figure 30. Landfill slopes are modeled such that geonets are placed inside the landfill in a configuration which utilizes the tensile strength of the geonets to the maximum. Geonet is placed as periodic covers between the waste cells (periodic covers). In the landfill model in Figure 30, the geonets were placed at 50 ft. intervals within the 250 ft. height of the landfill. The slope is modeled with 1:3 inclinations. Minimum factor of safety for this case is found to be 1.457.

Model:



Geonet & Old landfill material
Soil Model Mohr-Coulomb
Unit Weight 60 pcf
Cohesion 15.29 psf
Phi 6.1 degrees

Analysis Result:



Factor of safety = *1.457 (O) – Ordinary Method of Slices- Minimum factor of safety*
1.459 (J) – Janbu method
1.484 (B) – Bishop's method of slices
1.496 (Mp-M) – Morgenstern and Price's method – Moment based
1.496 (Mp-F) – Morgenstern and Price's method – Force based

Figure 30: Stability Analysis with a periodic cover (Effect of geonet inclusion inside the waste mass as a reinforcement material)

From Figure 30, we observe that the inclusion of a geonet inside the landfill in the manner proposed in this study does improve the factor of safety and stability of a landfill. This issue needs to be further investigated with different types of geonet and different landfill materials.

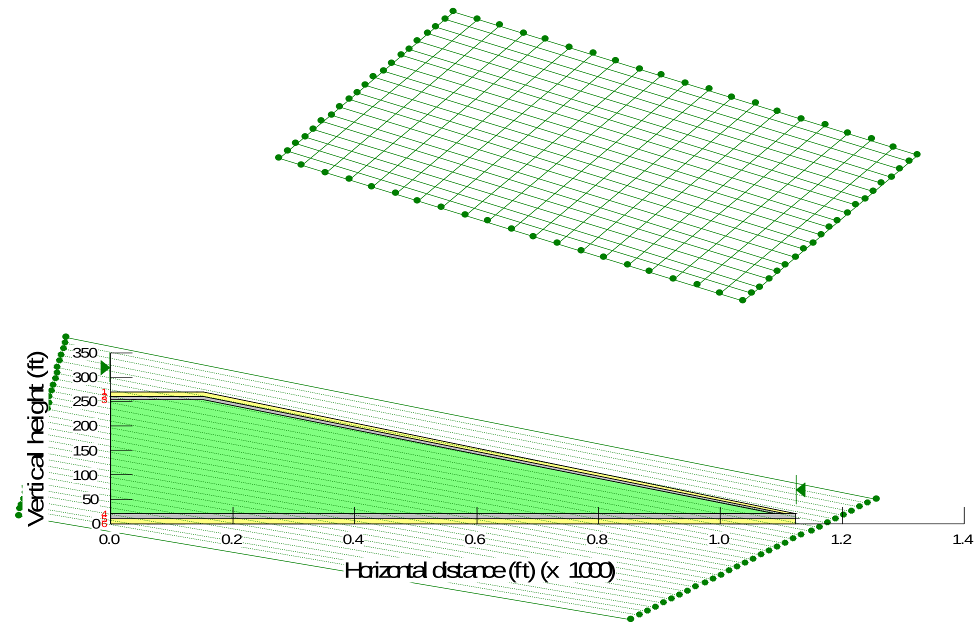
4.6 Modeling and Analysis of Landfill Slopes with reduced slopes (1:4)

Effect of reducing the slope of a landfill is presented in this section. Similar procedures described in sections 4.4 were followed to model and analyze the old and new landfill slopes, except that the inclination of the landfill slopes are changed from 1:3 to 1:4. The models and analyses results are presented in the following sections.

4.6.1 New Landfill (Slope 1:4)

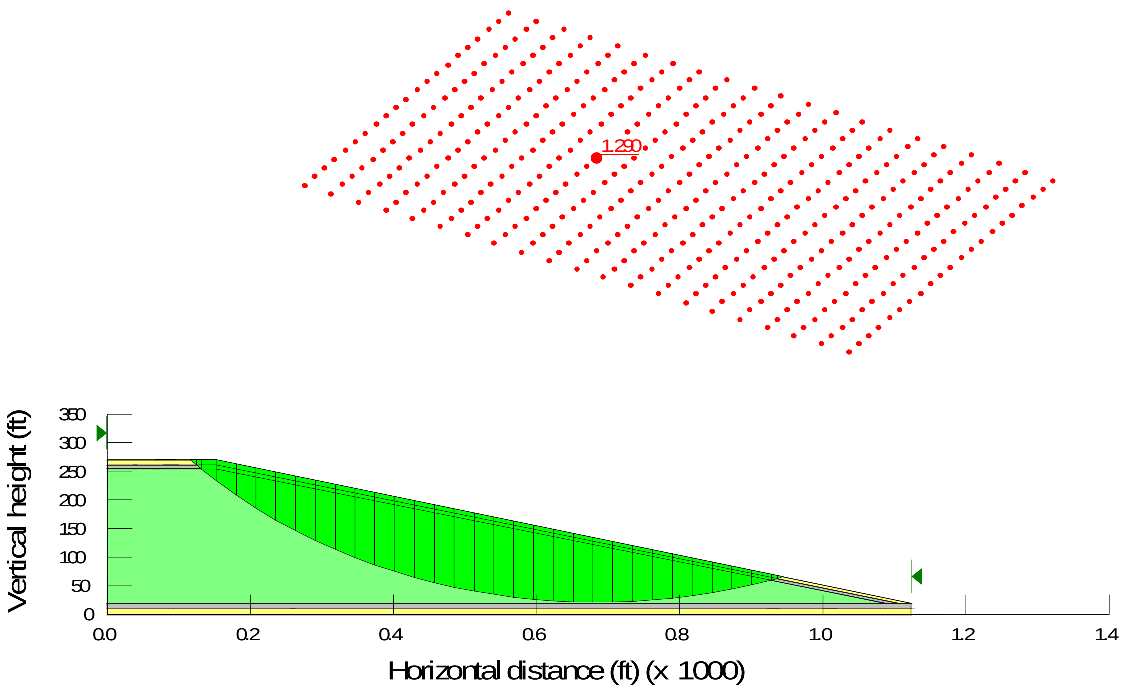
Cohesion, friction angle and unit weight values (Table 9) which are the results of direct shear test performed by Koodathinkal (2003), are used to define the properties of a new landfill material which is a mixture of MSW and 30 % biosolids. With a side slope of 1:4, minimum factor of safety is found to be 1.290. The model and slip surface are shown in Figure 31. This figure also displays the minimum factors of safety from the various methods of analysis.

Model:



New Landfill
Soil Model Mohr-Coulomb
Unit Weight 63 pcf
Cohesion 674 psf
Phi 11.21 degrees

Analysis Result:



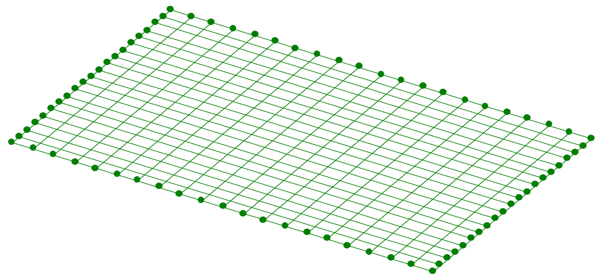
Factor of safety = *1.290 (J) – Janbu Method- Minium factor of safety*
1.313 (O) – Ordinary method of slices
1.368 (B) – Bishop's method of slices
1.367 (Mp-M) – Morgenstern and Price's method – Moment based
1.368 (Mp-F) – Morgenstern and Price's method – Force based

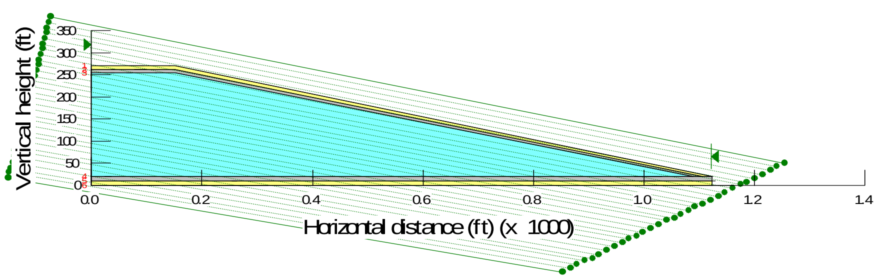
Figure 31: Slope stability analysis for a new landfill slope (Slope 1:4)

4.6.2 Old Landfill (Slope 1:4)

Similar model was generated for an old landfill with a 1:4 slope. Values of cohesion and friction angle from Table 9 and an average value of unit weight from Table B-2 were used to model the slope in Figure 32. The model along with the analyses results are presented in the following section. Factor of safety for this case is found to be 1.735.

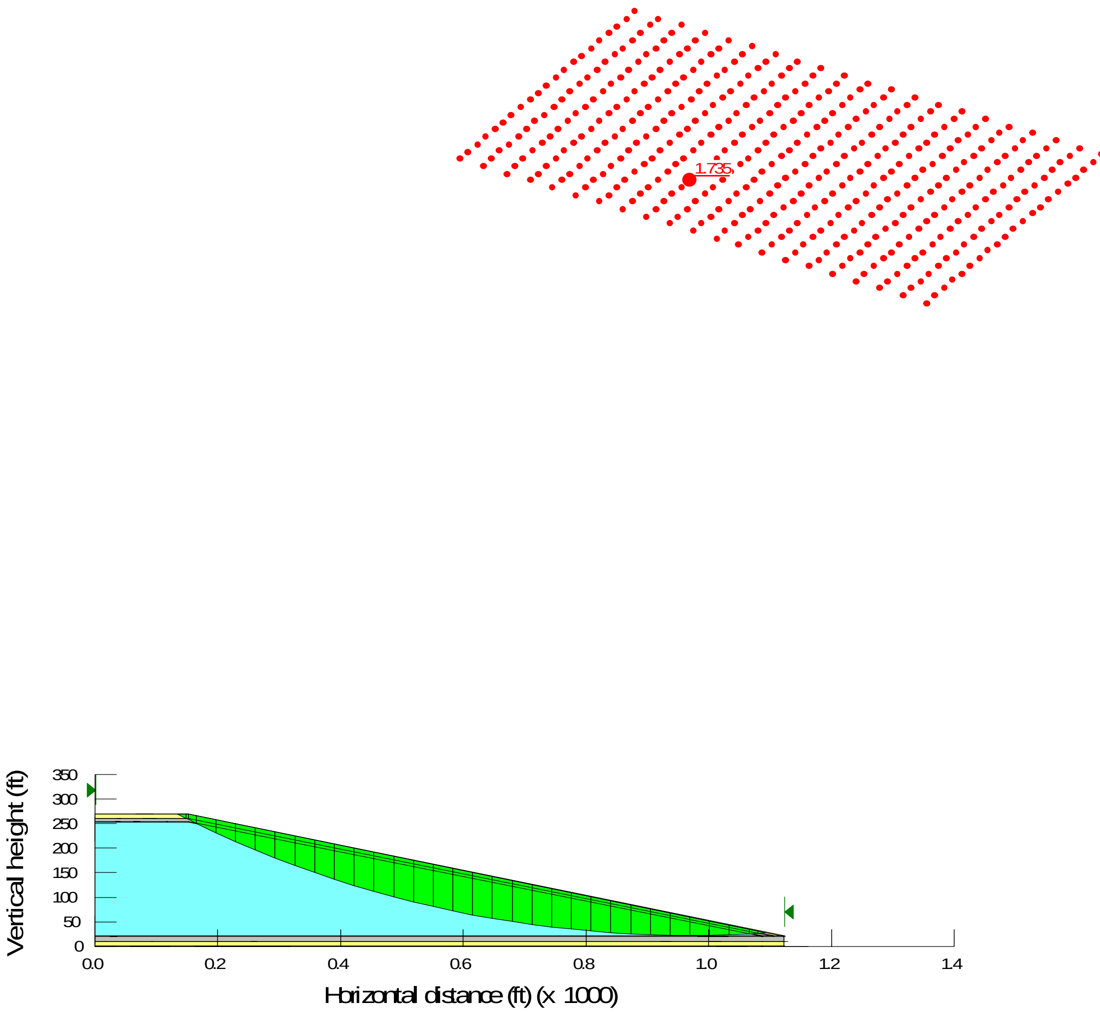
Model:





Old Landfill
Soil Model Mohr-Coulomb
Unit Weight 62.83 pcf
Cohesion 56.74 psf
Phi 23 degrees

Analysis Result:



Factor of safety = *1.735 (J) – Janbu Method- Minimum factor of safety*

1.736 (O) – Ordinary method of slices

1.767 (B) – Bishop's method of slices

1.767 (Mp-M) – Morgenstern and Price's method – Moment based

1.767 (Mp-F) – Morgenstern and Price's method – Force based

Figure 32: Slope stability analysis for an old landfill slope (Slope 1:4)

Again assuming that the 5% difference in the biosolids content in the new landfill doesn't affect the results significantly, a summary of factors of safety with 1:3 slope and 1:4 slopes are presented and the effect is compared for both old and new landfill slopes in Table 14.

Table 14: Summary of the Factors of Safety with 1:3 and 1:4 Slopes

Landfill's Age	*Factor of Safety with 1:3 Slope*	*Factor of Safety with 1:3 Slope*
New	1.029	1.290
Old	1.360	1.735

From Table 14, we see that the stability of a landfill slope increases almost by a constant factor if the side slopes are reduced. This emphasizes the role of geometry in the overall stability of a landfill slope. However, reinforcement may be used with steeper slopes, if land area is not available to flatten the slopes.

4.7 Summary

In this chapter, the direct shear test results of cured compost samples and their interfaces with geonet were presented. Landfill slope models using SLOPE/W software were also presented along with the analysis results. The effect of geonet inclusion in landfills to potentially increase its stability was also investigated. Time variation of cohesion, friction angle and factor of safety were summarized. Finally, the role of geometry in the slope stability of a landfill is emphasized by showing that the stability of a slope may be increased by using a less steep geometry if land is available.

CHAPTER FIVE: CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Based on the research conducted in this thesis, the following conclusions may be drawn:

- Cohesion value decreases by a huge amount, with time. The landfill materials seem to loose the cohesive forces because of the timely decay of the softer organic material which usually binds the particles.

- The angle of internal friction is seen to be increasing with time. In other words, the slopes can be gradually increased from flatter to steeper by adding more material and hence a schedule can be prepared for addition or expansion of landfills.

- High values of moisture contents were recorded, which may be a result of moisture addition inside the digestor of the Composting Facility. The temperature of the oven, and the time the samples have to retained in the oven should also be reduced. There seems to be an additional loss of moisture not because of the evaporation of escaping of water from the sample, but from the evaporation of some of the highly volatile organic materials.

- The overall factor of safety is seen to be increasing with time. Hence the stability of the landfill during the early construction phase is more critical. As observed from the stability analysis of the old and new slopes, it is noted that both cohesion and friction angle are important geotechnical strength properties of the waste, and influence the stability of a landfill slope to a significant extent.

- The geometry of a landfill plays an important role in the slope stability of a landfill. Change of landfill slope from 1:3 to 1:4 invariably increases the factor of safety by almost a constant value.

- Geonets, which are primarily used for drainage, may also be used as a reinforcing material to improve the stability of a landfill slope. Geonet, when placed within the waste mass in a periodic cover fashion can improve the stability of a landfill slope. On the other hand, inclusion of a geonet over a large area of a landfill is a costly issue and has to be called for only when the slope stability of a landfill is a serious threat to stability. Hence, when slope stability of the landfill is the only concern and all other issues like drainage, gas generation etc. have been properly accounted for and there is no concern related to ponding of leachate or rainwater over the geonet due to its high permeability value, then it is shown that geonets may be successfully used as reinforcement materials. More study is needed to confirm this finding under different conditions.

- There should be some modifications in the compaction tests to be conducted on compost materials due to high moisture contents. Either the compaction energy or the number of blows or both should be reduced.

5.2 Recommendations

The results from this work can be more meaningful, if there is correlation of the shear strength properties with changes in moisture. Hence, the change in the shear strength properties of landfill materials can be monitored with the change in moisture content.

As the cohesion and friction angles of the landfill material are derived from a best-fit straight line approximation of the shear stress variation with normal stress, Mohr-coulomb's failure theory is used in the analysis. More tests have to be conducted to confirm this behavior.

More sophisticated experiments like triaxial tests have to be conducted to get precise values of the shear strength properties under natural loading conditions. This will simulate natural slip surface frictional behavior.

As shown in Figure 24, previous studies reveal that the shear strength parameters of waste vary over a wide range. This can be attributed to the variety in its composition. Hence a study of variation of geotechnical properties of waste with its composition should be conducted. Some preliminary work on this has been carried out by Koodathinkal (2003).

Installation of geonets as a reinforcement material throughout the landfill is a costly issue. Analysis can be carried out to determine the anchor length that is sufficient to generate the required tensile strength, thereby reducing the geonet material required which provides an economic design.

APPENDIX A: DIRECT SHEAR TEST DATA

Table A-1.1 Direct Shear Test Readings for Cured Sample - 1

Cured Sample - 1								
HDG – Horizontal Dial Reading 1 div = 0.001 inch								
VDG - Vertical Dial Reading 1 div = 0.001 inch								
PRR - Proving Ring Reading Proving ring factor = 0.32 lb/div								
Normal Load = 1013 g			*Normal Load = 2014 g*			*Normal Load = 3018.5 g*		
HDG	VDG	PRR	HDG	VDG	PRR	HDG	VDG	PRR
800	732	970	700	611	970	800	600	970
760	730	987	650	610	984	750	611	976
720	733	993	600	613	988	700	615	986
680	723	1001	550	610	987	650	620	986
640	710	1007	500	608	998	600	624	996
600	889	1006	450	595	1001	550	606	1001
560	874	1007	400	574	1006	500	597	1000
520	854	1007	350	553	1008	450	584	1006
480	834	1006	300	533	1007	400	566	1008
			250	514	1002	350	549	1010
						300	531	1010
						250	516	1008
						200	503	1007

Normal Load = 4000 g			*Normal Load = 5013 g*		
HDG	VDG	PRR	HDG	VDG	PRR
700	695	970	800	620	970
640	707	979	740	630	984
580	713	989	680	634	993
520	717	986	620	630	997
460	721	991	560	623	1000
400	715	999	500	633	990
340	702	1004	440	626	1002
280	687	1004	380	617	1007
220	669	1010	320	606	1010
160	648	1012	260	590	1013
100	630	1012	200	572	1015
40	618	1010	140	555	1014
			80	546	1013

Table A-1.2 Direct Shear Test Readings for Cured Sample - 2

Cured Sample - 2								
HDG - Horizontal Dial Reading 1 div = 0.001 inch								
VDG - Vertical Dial Reading 1 div = 0.001 inch								
PRR - Proving Ring Reading Proving ring factor = 0.32 lb/div								
Normal Load = 1013 g			*Normal Load = 2014 g*			*Normal Load = 3018.5 g*		
HDG	VDG	PRR	HDG	VDG	PRR	HDG	VDG	PRR
965	614	970	800	614	970	900	603	970
925	606	990	760	609	986	850	601	984
885	592	1010	720	599	995	800	596	985
845	571	1015	680	594	1001	750	588	1000
805	544	1020	640	566	1013	700	582	1002
865	520	1019	600	544	1020	650	563	1010
825	496	1016	560	530	1022	600	541	1018
			520	508	1023	550	522	1021
			480	490	1022	500	501	1024
			440	478	1020	450	482	1026
						400	464	1025
						350	450	1025
						300	436	1024

Normal Load = 4000 g			*Normal Load = 5013 g*		
HDG	VDG	PRR	HDG	VDG	PRR
900	603	970	850	572	970
840	600	983	790	569	988
780	599	985	730	577	980
720	594	998	670	567	997
660	585	1002	610	552	1000
600	571	1008	550	550	1015
540	559	1018	490	553	1008
480	541	1018	430	540	1017
420	524	1024	370	527	1025
360	502	1028	310	511	1030
300	483	1022	250	498	1024
240	464	1023	190	481	1028

Table A-1.3 Direct Shear Test Readings for Cured Sample - 3

Cured Sample - 3								
HDG - Horizontal Dial Reading 1 div = 0.001 inch								
VDG - Vertical Dial Reading 1 div = 0.001 inch								
PRR - Proving Ring Reading Proving ring factor = 0.32 lb/div								
Normal Load = 597 g			*Normal Load = 1004.5 g*			*Normal Load = 1598 g*		
HDG	VDG	PRR	HDG	VDG	PRR	HDG	VDG	PRR
760	451	969	780	400	969	800	407	968
700	447	980	740	402	977	730	408	982
640	440	989	700	403	978	660	404	991
580	416	989	660	402	988	590	402	993
520	412	993	620	397	993	520	392	997
460	396	995	580	394	995	450	378	1001
400	378	996	540	385	998	380	362	1002
340	360	998	500	374	996	310	344	1005
280	337	1001	460	365	997	240	317	1009
220	315	1002	420	355	997	170	295	1009
160	299	1002	380	344	1000			
			340	332	1001			
			300	320	1004			
			260	304	1007			
			220	286	1008			
			180	267	1007			

Normal Load = 2017.5 g			*Normal Load = 3018.5 g*		
HDG	VDG	PRR	HDG	VDG	PRR
700	380	968	720	416	968
640	384	983	660	416	982
580	385	991	600	412	993
520	380	997	540	399	998
460	371	1000	480	384	1001
400	361	1003	420	373	1003
340	346	1007	360	360	1007
280	328	1009	300	388	1012
220	311	1012	260	384	1011
160	276	1008			
100	254	1008			

Table A-1.4 Direct Shear Test Readings for Cured Sample - 4

Cured Sample - 4					
HDG - Horizontal Dial Reading 1 div = 0.001 inch					
VDG - Vertical Dial Reading 1 div = 0.001 inch					
Normal Load = 1001 g			*Normal Load = 2005.5 g*		
HDG	VDG	PRR	HDG	VDG	PRR
640	253	968	650	264	968
600	252	992	600	262	982
560	229	1000	550	254	991
520	213	1004	500	246	999
480	195	1005	450	233	1004
440	174	1003	400	213	1009
400	157	1003	350	190	1009
			300	170	1009
			250	153	1008

Normal Load = 2603.2 g			*Normal Load = 3018.5 g*		
HDG	VDG	PRR	HDG	VDG	PRR
660	392	968	540	554	969
590	393	982	480	552	987
520	394	992	420	545	997
450	386	996	360	532	1004
380	375	999	300	517	1006
310	358	1003	240	498	1012
240	340	1002	180	477	1012
170	320	1007	120	459	1013
100	300	1009	60	484	1008
74	230	1009			

Table A-2.1 Direct Shear Test Readings for Geonet Interface with Cured Sample - 1

Geonet with Cured Sample - 1								
HDG - Horizontal Dial Reading 1 div = 0.001 inch								
VDG - Vertical Dial Reading 1 div = 0.001 inch								
PRR - Proving Ring Reading Proving ring factor = 0.32 lb/div								
Normal Load = 1013 g			*Normal Load = 2017.5 g*			*Normal Load = 3018.5 g*		
HDG	VDG	PRR	HDG	VDG	PRR	HDG	VDG	PRR
800	482	970	700	462	970	750	462	970
760	481	977	650	459	982	700	463	983
720	475	981	600	457	981	650	462	988
680	472	981	550	455	980	600	461	988
640	471	980	500	454	984	550	460	988
600	471	979	450	453	985	500	459	988
560	471	980	400	452	985	450	457	988
520	471	980	350	451	985	400	456	988
480	470	980	300	451	985			
440	470	980	250	449	984			
400	469	979						

Normal Load = 4000 g			*Normal Load = 5013 g*		
HDG	VDG	PRR	HDG	VDG	PRR
850	468	970	760	473	970
790	468	988	700	473	992
730	466	993	640	472	993
670	465	992	580	471	994
610	465	991	520	471	995
550	465	991	460	471	995
490	464	991	400	470	995
430	463	991	340	408	995
			280	370	995

Table A-2.2 Direct Shear Test Readings for Geonet Interface with Cured Sample - 2

Geonet with Cured Sample - 2								
HDG - Horizontal Dial Reading 1 div = 0.001 inch								
VDG - Vertical Dial Reading 1 div = 0.001 inch								
PRR - Proving Ring Reading Proving ring factor = 0.32 lb/div								
Normal Load = 1013 g			*Normal Load = 2014 g*			*Normal Load = 3018.5 g*		
HDG	VDG	PRR	HDG	VDG	PRR	HDG	VDG	PRR
600	532	970	600	561	970	600	570	970
560	533	980	550	564	979	550	570	980
520	531	985	500	564	983	500	570	985
480	524	983	450	561	982	450	568	986
440	523	982	400	561	983	400	565	986
400	523	981	350	561	983	350	586	992
360	523	981	300	561	984	300	539	988
320	523	980	250	560	984	250	583	988
280	524	980	200	596	985	200	510	988
			150	560	985	150	518	987
			100	575	984			

Normal Load = 4000 g			*Normal Load = 5013 g*		
HDG	VDG	PRR	HDG	VDG	PRR
1600	579	970	1600	595	970
1540	579	989	1540	594	989
1480	575	989	1480	590	992
1420	577	991	1420	590	992
1360	577	991	1360	588	991
1300	577	990	1300	588	992
1240	577	990	1240	588	992
1180	577	990	1180	592	991
1120	577	990	1120	586	991
1060	577	990	1060	586	992
1000	578	991	1000	586	993
940	539	991	940	586	993

APPENDIX B: MOISTURE CONTENT AND UNIT WEIGHTS DATA

Table B-1.1 Moisture content data of Cured Sample - 1

Cured Sample - 1				
Serial Number	*Weight of container (g)*	*Weight of the Container and Moist sample (g)*	*Weight of the container and Dry sample (g)*	*Moisture Content(%)*
1	50.1	95.3	73.1	49.11
2	49.7	95.1	65.7	48.91
3	25.1	36.2	30.7	50.00
4	25.1	40.9	33.5	46.80
5	25.1	34.9	30.5	44.90
			Average	47.94

Table B-1.2 Moisture content data of Cured Sample - 2

Cured Sample - 2				
Serial Number	*Empty weight Of the container (g)*	*Weight of the container and Moist sample (g)*	*Weight of the container and Dry sample (g)*	*Moisture Content(%)*
1	50	115.3	77.6	57.73
2	42	89.8	62.4	57.32
3	24.8	40.3	31.5	56.77
4	24.7	43	32.5	57.37
5	24.9	42.2	-	-
			Average	57.30

Table B-1.3 Moisture content data of Cured Sample - 3

Cured Sample - 3				
Serial Number	*Empty weight Of the container (g)*	*Weight of the container and Moist sample (g)*	*Weight of the container and Dry sample (g)*	*Moisture Content(%)*
1	49.9	100.7	67.17	66.00
2	49.9	102.7	67.22	67.20
3	41.9	84.5	56.00	66.90
4	24.7	51.1	34.12	64.30
5	24.8	48.5	32.30	68.40
			Average	66.56

Table B-1.4 Moisture content data of Cured Sample – 4

Cured Sample - 4				
Serial Number	*Empty weight Of the container (g)*	*Weight of the container and Moist sample (g)*	*Weight of the container and Dry sample (g)*	*Moisture Content(%)*
1	49.4	96.2	65.64	65.30
2	41.3	121	69.22	64.97
3	42	94.3	59.10	67.30
4	24.7	53.6	34.26	66.93
5	24.7	58.1	35.27	68.35
			Average	66.57

Table B-1.5 Geonet Interface Test with Cured Sample - 1: Moisture Content Data

Geonet with Cured Sample - 1				
Serial Number	*Empty weight of the container (g)*	*Weight of the container and Moist sample (g)*	*Weight of the container and Dry sample (g)*	*Moisture Content(%)*
1	50.1	70.4	60.6	48.27
2	49.7	68.7	59.3	49.47
3	25.1	33	29.1	49.37
4	25.1	32.4	28.7	50.69
5	25.1	32	28.6	49.27
			Average	49.41

Table B-1.6 Geonet Interface Test with Cured Sample - 2: Moisture Content Data

Geonet with Cured Sample - 2				
Serial Number	*Empty weight of the container (g)*	*Weight of the container and Moist sample (g)*	*Weight of the container and Dry sample (g)*	*Moisture Content(%)*
1	50	87.3	65.9	57.37
2	41.9	84.7	59.7	58.41
3	24.8	39.8	30.9	59.33
4	24.7	39.1	30.9	56.94
			Average	58.01

Table B-2 Unit Weights Determination Readings

Serial Number	*Sample*	*Empty weight of the Shear test box (g)*	*Weight of the shear box + sample (g)*	*Weight of the sample (g)*	*Unit weight of the sample (pcf)*
1	Cured Sample - 1	4630.9	6356.30	1725.40	50.1
2	Cured Sample - 2	4630.9	6701.40	2070.50	60.1
3	Cured Sample - 3	4630.9	7046.44	2415.54	70.4
4	Cured Sample - 4	4630.9	7058.24	2427.34	70.7

APPENDIX C: GEONET DETAILS

Geonet

It consists of approximately 97 % polyethylene, 1.5 to 3.0 % carbon black, and other additives. Following are its properties.

Table C.1 Physical Properties of Geonet

Property	*Testing Method*	*Minimum Average Value*
Thickness, mm	ASTM D 5199	5.1
Density, min., g/cc	ASTM D 1505	0.94
Carbon Black content, %	ASTM D 1603	1.5 - 3.0
Tensile Strength, (Peak, MD), kN/m	ASTM D 5035	7.9
Transmissivity, (MD), m^2/sec	ASTM D 4716	0.001

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