EFFECT OF LEACHATE ON THE STABILITY OF
LANDFILL COMPOSITE LINERS

A Thesis presented to
The Faculty of the College of Engineering and Technology
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In Partial Fulfillment
of the Requirements for the Degree
Master of Science in Civil Engineering

By
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CHAPTER 1

INTRODUCTION

1.1 General Introduction

Environmental pollution has been receiving more attention in recent years. Water and air pollution has been examined thoroughly, significant control measures have been proposed, and a comprehensive volume of legislation has been passed for pollution control and prevention. The pollution of the land surfaces of the United States, which has been called by some the “third pollution”, consists essentially of disposal of solid wastes. Although rather late in starting, the movement to control and prevent pollution due to solid waste disposal has accelerated rapidly within the last few decades, and today the national effort for managing the solid wastes has reached a rather significant level (Hagerty et. al, 1973).

America is a throwaway society. From both industrial and municipal sources, the United States generates about 10 billion metric tons of solid wastes each year. Of this total, municipal wastes represents approximately 140 million metric tons, industrial wastes 140 million tons and agricultural wastes 640 million tones. It is predicted that the total amount of wastes generated from all the sources by the year 2000 may reach 12 billion tones per year (Tchobanoglous et. al, 1977, Schaper, 1991). After solid wastes have been processed and the recovery of conversion products and/or energy has been accomplished, something must be done with the residual matter that is of no further use. The only two alternatives available for the long term handling of solid wastes and residual matter are disposal on or in the earth’s mantle, and disposal at the bottom of the ocean. Disposal on land is by far the most common method in use today.
More than 90 percent of the nation's solid waste is directly disposed of on land. Based on past experience in cities throughout the United States and elsewhere in the world, landfilling has been proved to be the most economical and acceptable method for the disposal of solid waste. Landfill planning and design has evolved into a rather complex technology. The trend towards more sophisticated design will continue as long as landfills are an integral part of solid waste management. Although a sanitary landfill is the last choice in the hierarchy of solid waste management alternatives, it remains an essential part of the total waste management program (Schaper, 1991).

One of the major objectives in designing a landfill should be to reduce environmental and health risks to reasonable levels that will mitigate local, regional, and national concerns. A properly designed and constructed landfill should incorporate proven technology to guard against the release of contaminants into air, surface water, and groundwater, reducing environmental concerns over its entire operating and postclosure life.

While many geological, hydrogeological and engineering problems can be minimized through effective landfill designs, a careful siting process will greatly reduce the risk that may arise during and after operations. While there are many aspects of landfill design that need to be considered, one of the major components that must be addressed is the placement of a composite liner at the bottom of the landfill, which typically consists of 3 to 5 feet of recompacted clay liner overlain by a geomembrane.

Several studies have been conducted to investigate characteristics of the clay/geomembrane interface under unsaturated conditions, utilizing testing devices such as modified direct shear, pull-out box, and others. The results have indicated consistently that the shear strength at the clay-geomembrane interface is always much less than the shear strength of the clay. Also, it was pointed out in some studies that the critical interfaces within the multi-layer landfill system were between the HDPE liner and geotextile layer; between the HDPE liner and geonet layer, and between the HDPE liner and compacted clay
liner (saturated). The interface frictional resistance was affected by various properties, including the degree of polishing and the degree of interface saturation.

However, only a few studies have been conducted on clay/geomembrane interface saturated with water. Yet, this aspect is very important in the slope stability, design, and installation of composite liners in landfills.

A knowledge of solid waste decomposition processes and many influences they exert is essential to proper sanitary landfill site selection and design. Solid waste deposited in a landfill degrades chemically and biologically to produce solid, liquid and gaseous products. Some factors that affect degradation are the heterogeneous characteristics of the wastes, the wastes' physical, chemical and biological properties, the availability of oxygen and moisture within the waste fill, temperature, microbial populations, and type of synthesis (Metry et al. 1976). Ground water, infiltrating surface water moving through the solid waste, or moisture contained in the wastes can produce leachate, a solution containing dissolved and suspended solid matter and microbial waste products. The composition of leachate is important in determining its potential effects on the quality of nearby surface water and ground water. The types and quantities of contaminants that enter the receiving water and the ability of the water to assimilate these contaminants will determine the degree of leachate control needed. Most experts feel that through sound engineering and design, leachate production and movement may be prevented or minimized to the extent that it will not create a water pollution problem.

Several investigators have shown that concentrated organic chemicals can attack compacted clay, effectively destroying the barrier characteristics of the liner material. Shrinkage behavior of clay liner material exposed to landfill leachate has also been studied. More recently, a research work was conducted on a sand/geomembrane interface exposed to kerosene and turpentine. However, no study has been done on clay/geomembrane interfaces exposed to actual landfill leachate. This is a very important aspect, because, over
time, leachate will percolate down to the bottom composite liner, and this aspect needs to be considered in the design.

1.2 Objectives

Based on the general background information summarized in the previous section, the following objectives were established for this investigation:

(1) To establish the clay/geomembrane interface shear strength (or the baseline shear strength) parameters under unsaturated condition.

(2) To determine the effect of water saturation on the clay/geomembrane interface shear strength by performing the modified direct shear tests.

(3) To determine any significant changes in the physical properties of geomembranes due to immersion in actual landfill leachate over 7 day, and 30 day periods.

(4) To study the effect of leachate saturation on the clay/geomembrane interface shear strength by performing modified direct shear tests.

(5) To compare the interface shear strength against clay among geomembranes immersed in leachate for 7 days and 30 days.

Each of these objectives was applied to three different types of clay/geomembrane interfaces in order to conduct a comparative study. These interfaces were (a) clay/smooth HDPE geomembrane interface; (b) clay/textured HDPE geomembrane; and (c) clay/PVC geomembrane interface.

1.3 Outline of Thesis

Chapter 1 has been devoted to the general background information regarding the current state of solid waste management, the current landfilling concept, and the extent of
studies conducted in the past in characterizing the interfaces existing in a modern solid waste landfill.

Chapter 2 presents a more detailed background/literature review of information related to the topics covered briefly in Chapter 1. The current EPA regulations/guidelines concerning the landfill design/construction are first summarized, and the types of interfaces existing in a typical modern landfill are defined. Then, an overview of geomembrane material (types, physical, environmental and strength properties) is presented. And, the last sections of Chapter 2 present results of literature review on the characteristics of the clay/geomembrane interface and on the behavior of clay or geomembrane saturated with chemicals.

Chapter 3 describes the methodology utilized in the current study. Subjects covered in this chapter range from the U.S. EPA Method 9090, and the modified direct shear test, to various tests conducted to characterize the soil and leachate. This chapter also presents the various test conditions which were investigated in this study.

All test results produced from the current study are compiled in Chapter 4. Research findings corresponding to each objective are presented and discussions are developed. Chapter 5 concludes the investigation with conclusions and recommendations for further research.
CHAPTER 2

BACKGROUND INFORMATION AND LITERATURE REVIEW

2.1 Landfills

A landfill or a sanitary landfill is an engineering method of disposing solid wastes on land by spreading them in thin layers, compacting them to the smallest practical volume, and covering them with soil each working day in a manner that protects the environment. According to the U.S. EPA, a sanitary landfill facility means an engineered facility where the final deposition of solid waste on or into the ground is practiced in accordance with chapters 3745-27 and 3745-37 of the administrative code, including areas of solid waste placement, ground water monitoring/control system structures, buildings, explosive gas monitoring/control/extraction system structures, surface water run-on and runoff control structures, sedimentation pond(s), liner systems, management system structures, and areas within the three-hundred-foot radius from the limits of solid waste placement. Careful planning/designing can never be stressed enough before construction and operation of a landfill.

2.1.1 EPA Regulations for Landfills

The guidelines presented by the Federal government (CFR 40 241.200 to 241.212-3) are generally applicable to the land disposal of all solid waste materials. The guidelines are intended to provide for environmentally acceptable land disposal site operations. The guidelines do not establish new standards but set forth requirements and recommended procedures to ensure that the design, construction, and operation of both existing and future land disposal sites meet the health and environmental standards for the area in which they are located.
Routine sanitary landfill techniques of spreading and compacting solid wastes and placing cover material at the end of each operating day should be used to dispose of municipal solid wastes. If special wastes such as hazardous wastes, infectious institutional wastes, and bulk liquids are to be accepted at land disposal sites, a special assessment of site characteristics, nature and quantities of the waste, special design and operations, and precautions is essential.

The hydrogeology of the site should be evaluated in order to design site development in a manner to protect or minimize the impact on the groundwater resources. The site should not be located in an area where the attraction of birds would pose a hazard to low flying aircraft. The location, design, construction, and operation of the land disposal site shall conform to the most stringent water quality standards effective under the provisions of the Federal Water Pollution Control Act.

Cover material should be applied as necessary to minimize fire hazards, infiltration of precipitation, odors, control gas venting and provide a pleasing appearance. Daily cover of at least 6 inches thickness should be applied regardless of the weather. Sources of cover material should therefore be accessible on all operating days. Intermediate cover of at least 1 feet thickness should be applied on areas where additional cells are not to be constructed for extended periods of time, normally, 1 week to 1 year. Final cover of thickness 2 feet should be applied on each area as it is completed or if the area is to remain idle for over 1 year.

In order to conserve land disposal site capacity, and to minimize moisture infiltration and settlement, municipal solid waste and cover material shall be compacted to the smallest practical volume. Solid wastes should be spread in layers no more than 2 feet thick while confining it to the smallest practical volume.
Ohio EPA have presented the guidelines for landfills in more detail for each and every aspect. According to the EPA regulations, sanitary landfills should not be located (OAC 3745-27):

(a) in a sand or gravel pit where the sand or gravel deposit has not been completely removed,
(b) in a limestone quarry or sandstone quarry,
(c) within a close proximity to any preferential stream,
(d) within 1,000 feet radius of drinking water wells,
(e) above an aquifer declared by the federal government under the Safe Drinking Water Act to be a sole source of aquifer prior to the date of receipt of the permit to install application by the Ohio EPA,
(f) within an area of potential subsidence due to an underground mine in existence on the date of receipt of the permit to install application by the Ohio EPA,
(g) above an unconsolidated aquifer capable of sustaining a yield of one hundred gallons per minute for a one day period to a water supply well located within one thousand feet of the limits of solid waste placement.

The following are the specifications in the design and construction of a sanitary landfill facility (OAC 3745-27):

(1) A recompacted soil liner should be constructed using loose lifts of eight inches thick, and its overall coefficient of permeability should be $1 \times 10^{-7}$ cm/sec or less. The soil used in the liner construction should have a maximum clod size of three inches or half the lift thickness whichever is less. The soil should be compacted to at least 95% of the maximum "standard Proctor density" using ASTM D-698 or at least 90% of the maximum "modified Proctor density" using ASTM D-1557 and at a moisture content of 0 to 3% wet of optimum. The soil liner should be placed on the bottom and side slopes of the
landfill and should have a maximum slope based on compaction equipment limitations, slope stability analysis, maximum friction angle between any soil/geosynthetic interface and between any geosynthetic/geosynthetic interface, and resistance of geosynthetic and geosynthetic seams to tensile forces.

(2) A geomembrane placed on the recompacted soil liner should be negligibly permeable to fluid migration, physically and chemically resistant to chemical attack by the solid waste, or other materials that may come in contact with the geomembrane, and seamed properly to allow no more than negligible amounts of leakage.

(3) A leachate management system should be designed to prevent clogging and crushing of the system and to limit the level of leachate in areas other than lift stations to a maximum of one foot. The system should have a drainage layer placed on top of the geomembrane that is able to rapidly collect the leachate entering the system. Collection pipes used to remove leachate from the bottom of the landfill should be embedded in the drainage layer, and have a minimum slope of 0.5%. A filter layer to prevent clogging of the collection system and a protective layer to protect the recompacted soil liner, geomembrane, and collection system from the intrusion of objects during construction and operation should be used.

(4) Collected leachate should be treated and disposed on site at the sanitary landfill facility or pretreated on-site and disposed off-site of the sanitary landfill facility or treated and disposed off-site of the sanitary landfill facility.

(5) Prior to the use of the recompacted soil liner, the following characteristics of the earthen materials must be determined to show that the material is suitable for use in construction of the sanitary landfill facility: recompacted permeability coefficient at construction specifications, moisture content and dry density using an approved ASTM method, grain size distribution using ASTM D-422 for sieve and hydrometer methods, and Atterberg limits using ASTM D-423 and ASTM D-424 methods.

(6) Prior to the installation of a geosynthetic, it should be shown to be physically and chemically resistant to attack by the solid waste, or other materials that may come in
contact with it using U.S. EPA Method 9090, and it should have properties acceptable for installation.

As a summary, Figure 2.1 presents an illustration of the typical landfill vertical profile. As shown in this drawing, contained within a closed landfill are typically the following interfaces:

(a) HDPE liner/geotextile interface
(b) HDPE liner/compacted clay liner interface
(c) HDPE liner/geonet interface
(d) Geotextile/compacted clay liner interface
(e) Geotextile/geonet interface

2.2 Geosynthetics

In the term “geosynthetic”, “geo” refers to the earth and “synthetic” refers to the materials obtained exclusively from human activities. The materials used in the manufacture of geosynthetics are primarily plastics, or polymers.

Geosynthetic materials perform five major functions: separation, reinforcement, filtration, drainage, and moisture barrier. The use of geosynthetics is highly advantageous in restricted spaces or in difficult soil conditions, enhances longevity of the treated earthen structures, and achieves overall cost savings in the long run due to minimum maintenance costs. The five major categories of geosynthetics are:

(a) geotextiles
(b) geogrids
(c) geonets
(d) geomembranes
(e) geocomposites
Figure 2.1 Typical Solid Waste Landfill Vertical Profile
(at Closure)
2.2.1 Geomembranes

In its effort to protect ground water, the EPA is requiring increased and more diversified liner layering for hazardous, solid waste landfills and surface impoundments. And, in doing so the EPA is mandating the increased use of polymer liner materials commonly called flexible membrane liners (FML’s) or geomembranes which form the second largest group of geosynthetics, geotextiles being the first.

The original geomembrane was used as a potable water pond liner. It was butyl rubber, which is a copolymer of isobutylene with approximately 2% isoprene. Butyl rubber is quite impermeable and has found its major use as inner tubes and as the liners of tubeless tires. Another geomembrane, Hypalon, results from the reaction of chlorine and sulfur dioxide which transforms the thermoplastic polyethylene into a vulcanizing elastomers. The resulting rubber is very resistant to ozone, heat and weathering. Most of the geomembranes used in landfills fall into the category of polymer resins and are thus thermoplastic materials. The factors that influence the physical properties of polymers include the size of the molecules, the distribution of individual molecular sizes within a polymer, and the shape and structure of individual molecules. Different additives are added to the polymers to improve manufacturing and the usefulness of the final product (Koerner, 1986).

From the numerous polymers and additives available, a number of possible formulations can be developed. However, in practice only a few are used, and these are named after the major polymers. Butyl rubber, chlorinated polyethylene (CPE), chlorosulphonated polyethylene (CSPE), low density polyethylene (LDPE), high density polyethylene, smooth and textured (HDPE) and polyvinyl chloride (PVC) are the common types (Bagachi, 1990). Table 2.1 summarizes basic material properties for the three most commonly used geomembrane materials.
### TABLE 2.1

Basic Material Properties of Various Geomembranes

<table>
<thead>
<tr>
<th>Property</th>
<th>Smooth HDPE</th>
<th>Textured HDPE</th>
<th>PVC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (pcf)</td>
<td>59.3</td>
<td>69.2</td>
<td>76.8</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>0.94</td>
<td>0.94</td>
<td>1.23</td>
</tr>
<tr>
<td>Young's modulus (psi)</td>
<td>$1.1 \times 10^5$</td>
<td>$1.6 \times 10^4$</td>
<td>$1.0 \times 10^3$</td>
</tr>
<tr>
<td>Tensile stress at break (psi)</td>
<td>4000</td>
<td>583</td>
<td>2300</td>
</tr>
<tr>
<td>Tensile strain at break (%)</td>
<td>700</td>
<td>100</td>
<td>450</td>
</tr>
<tr>
<td>Tear resistance (lbs)</td>
<td>22</td>
<td>30</td>
<td>6</td>
</tr>
<tr>
<td>Puncture resistance (lbs)</td>
<td>30</td>
<td>45</td>
<td>-</td>
</tr>
<tr>
<td>Dimensional stability (% change)</td>
<td>2</td>
<td>2</td>
<td>3.5</td>
</tr>
</tbody>
</table>


Membrane Lining Engineering Specification Guide.
Today, geomembranes are manufactured and distributed almost world wide, making all types of products available. What matters most to the owner/user/designer is to utilize proper materials for each specific project. Since all membranes do not have the same composition, individual testing of each synthetic membrane is essential to determine whether the one proposed for a project conforms to certain standard properties.

2.2.2 Characteristics of Geomembranes

Flexible membrane liners or geomembranes have become required by law for hazardous, solid waste landfills and surface impoundments. With such emphasis given to the flexible membrane liners, how can one be sure that the liner will provide maximum security for containment of hazardous and solid wastes? Thus, the properties of the geomembranes play a significant role in the selection of the proper type for a particular project.

The performance requirements for a flexible membrane liner (FML) include low permeability, chemical compatibility, mechanical strength, and durability. The landfill designer must specify the necessary criteria for each of these properties based on engineering requirements, performance requirements of the landfill, applicable regulatory requirements, and the specific site conditions. The performance requirements determined by a designer/engineer serve as the basis for the selection of a FML for a given facility. The designer must make decisions on the composition, thickness, and construction of an FML. The thickness of the geomembrane should be 30 mil (0.75 mm) or greater and the density of the geomembrane usually varies from 0.85 to 1.5 pcf. Since nothing is impermeable, the assessment of the relative impermeability of the geomembrane is an important issue. Typical values of permeability, as measured by water vapor transmission tests, are in the range of $5 \times 10^{-11}$ to $5 \times 10^{-14}$ cm/sec.
A FML must be mechanically compatible with the designed use of the lined facility in order to maintain its integrity during and after exposure to short-term and long-term mechanical stresses. Mechanical compatibility requires adequate tensile strength, puncture resistance, and geomembrane friction. Tensile strength is a fundamental design consideration. Adequate friction between the components of a liner system, particularly the soil subgrade and the FML, is necessary to ensure that slippage or sloughing does not occur on the slopes of the unit. Geomembranes placed on soil containing stones, sticks, or other debris are very vulnerable to puncture during and after loads are placed on them. Such puncture is an important consideration because it occurs after the geomembrane is covered and cannot be detected until a leak from the completed system becomes obvious.

A FML must exhibit durability, that is, it must be able to maintain its integrity and performance characteristics over the operational life and the post-closure period of the unit. The service life of a FML is dependent on the intrinsic durability of the FML material and on the conditions under which it is exposed. Exposure conditions can vary greatly within a given facility, and a FML must resist the combined effects of chemical, physical, and biological stresses. One indication of a geomembrane's durability is the amount of swelling that occurs because of liquid adsorption during its service life.

Chemical compatibility of the FML with wastes and leachates must be given due consideration. Even in a relatively inert environment, certain materials deteriorate over time when exposed to chemicals contained in both hazardous and non-hazardous environments. It is important to anticipate the kind and quality of leachate a site will generate and select liner materials accordingly. Chemical compatibility requires that the mechanical properties of the FML remain essentially unchanged after the FML is exposed to the waste. Chemical compatibility tests using the EPA method 9090 is used to assess the compatibility of a candidate FML with the specific waste liquid or leachate to be contained. A primary objective of chemical compatibility testing is to ensure that liner materials will remain intact not just during a landfill's operation but also through the post-closure period, and preferably
longer. It is difficult, however, to predict future chemical impacts.

2.3 Production and Characteristics of Leachate in Landfills

Leachate is generated as a result of the percolation of water or other liquid through wastes and the squeezing of the moisture contained in the waste due to its own weight. Thus, leachate can be defined as a liquid that is produced when water or other liquids come in contact with waste. Leachate is a contaminated liquid containing dissolved and finely suspended solid matter and microbial waste products. A part of the precipitation that falls on a landfill reacts both physically and chemically with the waste while percolating downward. During the percolation process it dissolves some of the chemicals produced in the waste through chemical reaction. During the course of migration, the liquid takes on the pollutant characteristics of the wastes. As such, leachate is both site specific and waste specific with regard to both its quantity and quality. The percolating water dilutes the contaminants in addition to aiding its formation. The quantity increases as the percolation of water increases, but at the same time the percolating water dilutes the concentration of contaminants.

The generated leachate will migrate into subsurface water when confining layers beneath the landfill are absent or inadequate. When the landfill sides are steep and lack final cover, springs may break out and contaminate run-offs and surface waters. Physical, chemical and biological factors in a landfill influence waste decomposition rate, generation rate and characteristics. Among these are:

(a) Biological decay of organic, putrifiable material, either aerobically or anaerobically with evolution of gases and liquids.
(b) Chemical oxidation of organic and inorganic material.
(c) Dissolving of organic and inorganic materials by water moving through the fill.
(d) Movement of dissolved materials by concentration gradient and osmosis.
The decomposition of landfill wastes and characteristics depends upon many factors such as waste composition, compaction, original moisture content, inhibition, rate of moisture movement, amount of oxygen available, temperature, and other factors. The rate of chemical and biological reaction in a landfill generally increases with temperature and moisture until an upper limit is reached. Waste decomposition in landfills tend to take place slowly over a very long period of time, as readable newspapers have even been dug out of 10 to 20 year old landfills (Metry et. al 1976). Table 2.2 presents typical characteristics of leachate in municipal solid waste landfills reported in the literature.

2.4 Leachate Collection and Removal System

The leachate collection and removal system (LCRS) regulations for municipal, solid waste and hazardous waste landfills require that the system be designed and operated to ensure that the depth over the liner does not exceed 30 cm. The system must also be resistant to wastes, sufficiently strong to withstand landfill loadings, and protected from clogging. The first part of the collector system to intercept the leachate is the primary collection and removal (PLCR) system located directly below the waste and above the primary liner. This system must be designed and constructed on a site-specific basis to remove the leachate for proper treatment and disposal. The second part of a collection system is between the primary and the secondary liners which is either known as secondary collection and removal (SLCR) system or leak detection, collection and removal (LDCR) system. Each collection and removal system, whether above (primary) or between the liners (secondary), consists of the following components:

(a) A low permeability base which is either a soil liner, composite liner, or flexible membrane liner (FML)

(b) A high permeability drainage layer constructed of either natural granular materials (sand and gravel) or synthetic drainage material (geonet), which is placed directly on the primary and/or secondary liner, or its protective bedding layer.
**TABLE 2.2**

Observed Ranges of Constituent Concentrations in Leachate from Municipal Waste Landfills

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Concentration Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total suspended solids, TSS (mg/L)</td>
<td>10 - 45,000</td>
</tr>
<tr>
<td>Total dissolved solids, TDS (mg/L)</td>
<td>725 - 55,000</td>
</tr>
<tr>
<td>Total solids, TS (mg/L)</td>
<td>1 - 75,000</td>
</tr>
<tr>
<td>Chemical oxygen demand, COD (mg/L)</td>
<td>50 - 90,000</td>
</tr>
<tr>
<td>Total organic carbon, TOC (mg/L)</td>
<td>50 - 45,000</td>
</tr>
<tr>
<td>Hydrogen ion concentration, pH</td>
<td>3.5 - 8.5</td>
</tr>
</tbody>
</table>

**Metals**

<table>
<thead>
<tr>
<th>Metals</th>
<th>Concentration Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chromium, Cr (mg/L)</td>
<td>0.02 - 18</td>
</tr>
<tr>
<td>Magnesium, Mg (mg/L)</td>
<td>3 - 15,600</td>
</tr>
<tr>
<td>Iron, Fe (mg/L)</td>
<td>200 - 5,500</td>
</tr>
<tr>
<td>Manganese, Mn (mg/L)</td>
<td>0.6 - 41</td>
</tr>
</tbody>
</table>

Source: Borden et al. (1989)
(c) Perforated collection pipes within the high permeability drainage layer to collect and carry it rapidly to the sump.

(d) A protective filter material surrounding the pipes, if necessary, to prevent physical clogging of the pipes or perforations.

(e) A collection sump or sumps, where leachate can be removed.

(f) A protective filter layer over the high permeability drainage material which prevents physical clogging of the material.

(g) A final protective layer of material that provides a wearing surface for traffic and landfill operations.

The primary LCRS acts as a collection system to remove leachate from the landfill before it can leak through the primary liner component. This system also aids in reducing leakage by removing or reducing the hydraulic head of leachate that exists within the system. The secondary LCRS acts as a leak detection system by collecting and removing any leachate which leaks through the top or primary liner component of the liner system.

2.5 Clay Liners and Composite Liners

Soil liner materials are selected based on their ability to meet specific performance standards and the costs to bring the materials on site. Requirements that must be met for a soil to perform properly as a liner material include:

(a) Low permeability (usually $1 \times 10^{-7}$ cm/sec when compacted).

(b) Sufficient strength to support itself and the overlying facility components without failure when compacted to the required permeability and thickness.

(c) Compatibility with waste or waste leachate to be contained, i.e., no significant loss in permeability or strength when exposed to waste or waste leachate.
The U.S. EPA has compiled data on the characteristics of soils used for constructing liners in a variety of locations nationwide (Elsbury et. al 1985, Ely et. al 1983). In addition to soil characteristics, cost considerations also enter into material selection when liner material must be brought from off site. The in situ soil at the facility site is the ideal liner material from the standpoint of cost and convenience. This material will be excavated during foundation preparation and therefore does not need to be transported to the site.

Composite liner systems should outperform either flexible membrane liners (FML’s) or clay liners alone. Leachate lying on top of a clay liner will percolate down through the liner at a rate controlled by the hydraulic conductivity of the liner, the head of the leachate on top of the liner, and the total area available for the flow in the liner. With the addition of a FML placed directly on top of the clay liner and sealed against its upper surface, leachate moving down through a hole or defect in the FML may not spread significantly out between the FML and the clay liner. The composite liner system allows much less leakage than a clay liner acting alone, because the area of flow through the clay liner is much smaller.

A FML placed on a bed of sand, geotextiles, or other highly permeable materials would allow liquid to move through the defect in the FML, spread laterally over the whole area of the clay liner, and percolate down as if the FML was not there. With clay liner soils that contain some rock, it is sometimes proposed that a woven geotextile be placed on top of the soil liner under the FML to prevent the puncture of the FML. A woven geotextile between the FML and the clay, however, creates a highly transmissive zone between the FML and the clay. The surface of the soil liner instead should be compacted and the stones removed so that the FML can be placed directly on top of the clay.

Understanding the basic hydraulic mechanisms for synthetic liners and clay liners is very important in appreciating the advantages of a composite liner. Hydraulic performance of the clay liners may be represented by Darcy’s law. In clay liners, the factors that most
influence liner performance are hydraulic head and soil permeability. Clay liners have a higher hydraulic conductivity and thickness than do synthetic liners. Additionally, leaking through a clay liner will undergo chemical reactions that reduce the concentration of contaminants in the leachate. Leakage through a synthetic liner is controlled by Fick's law, which applies to the process of liquid diffusion through the liner membrane. The diffusion process is similar to flow governed by Darcy's law except it is driven by concentration gradients and not by hydraulic head. Diffusion rates in geomembranes are very low in comparison to hydraulic flow rates even in clays.

2.6 Strength Characteristics of Clay/Geomembrane Interfaces

Geosynthetics have been used in civil engineering applications since the mid 1960's (Barrett 1966). In the beginning, they were applied to facilitate construction under difficult conditions and in low-risk engineering works, since few relevant geotechnical data or long-term performance records were available.

As positive experience was gained from "rule of thumb" designs, geosynthetics were used in more critical engineering applications such as reinforced earth structures, lined slopes of dams and canals and lined side slope, bottom liner and composite liner in landfills. Thus, it became apparent that more quantitative engineering data would be required to develop rational design criteria. During the last few decades, numerous testing methods have been established by various government agencies, manufacturers, and individuals to provide the necessary engineering design information and to describe the engineering properties of geosynthetics in quantitative terms. Nevertheless, generally recognized standard testing methods still do not exist, not only because of the diversity of the materials involved, but also as a result of the large number of combinations of materials used in a broader area of possible applications.
In solid and hazardous waste landfills, installation of a leachate collection system and a composite liner is necessary to reduce discharges of landfill leachate into the environment. A critical element in designing these systems is determining the maximum slope of the landfill in order to maximize the air space. Evaluation of slope stability requires knowledge of the strength characteristics of the interfaces between the soil and geosynthetic materials, or among different geosynthetics.

In order to determine the shear strength at the soil/geosynthetic interface, several different types of tests have been devised: pull-out tests (Giroud 1980; Ingold 1982; McGown et. al 1982), large scale direct shear tests (Myles 1982; Saxena et. al 1984); small scale direct shear tests (Giroud 1980; Martin et. al 1984; Akbar et. al 1985; Williams and Hovlihan 1987). Ingold (1982) reported and discussed significant differences between the results from shear box tests and pull out tests. He concluded that the pull out test is more appropriate than the direct shear test to determine strength parameters required for the design of reinforced earth structures. In other applications, however, where the geosynthetic is not required to provide the soil reinforcement as in the case of the lined slopes of landfills, the amount of differential movement between the soil and geosynthetic determines the mobilized resistance. Here the direct shear test is perceived to simulate the field behavior. Thus, the direct shear test is the appropriate test for evaluating the strength characteristics of the soil/geosynthetic interface (Collios et. al 1980; Ingold 1982). Several studies have been conducted to evaluate interface properties between geosynthetics and cohesionless soils using direct shear test devices (Degoutte and Mathieu, Negussey et. al, 1986; O'Rourke et. al, 1990). More recently Mitchell et. al (1990) studied the interface properties between geomembranes and cohesionless soils using small and large scale direct shear test devices.

According to current U.S. EPA regulations for liners at containment or disposal sites, in most situations geomembranes need to be placed directly on cohesive soils. Several studies have been performed to determine interface strength characteristics between
Takasumi et al (1991) reviewed the state of the art of the testing procedures to evaluate the soil/geosynthetic interface strength characteristics. They concluded that the selection device and size of the testing device is very important to evaluate the soil/geosynthetic characteristics. Koerner et al (1986) conducted direct shear tests on various geomembranes (PVC, CPE, EPDM, smooth HDPE and embossed or textured HDPE) against a number of different cohesive soils. The soil was compacted in the shear box at target values of dry density and water content. It was seen that adhesion of the geomembrane to the soil was markedly reduced from the cohesion of the soil itself unless the liner material was very soft or heavily textured. On the other hand, the frictional angle at the interface between the geomembrane and the soil was relatively high for all the tests.

Eigenbrod (1987) conducted extensive direct shear tests on four different soils using various geotextiles and geomembranes. He found that the friction between soil and geosynthetics was less than that within the soil. He concluded that the plane with the lowest soil/geosynthetic friction value is always the most critical one.

Fourie and Fabian (1987) conducted direct shear and pull out tests on a silty clay in contact with a needle punched, non woven and a woven geotextile for both drained and undrained conditions. For the undrained condition, they found that the bond strength develops differently for non woven and woven geotextiles. For non woven geotextile, the strength envelope was normal stress dependent. For a woven geotextile, the strength envelope was normal stress dependent up to 100 kPa. The difference in the bond strength was attributed to the high permeability and high transmissivity of the non woven fabric. It was observed that at any stress level, the non woven geotextile produced a higher bond strength than the woven geotextile. For the drained test, the strength envelope was friction like and had a zero cohesion intercept. The woven geotextile had higher frictional angles than did the non woven geotextile. For both woven and non woven geotextiles, the
undrained condition produced lower strength values than did the drained condition.

Swan et. al (1991) studied the effect of compaction water content and dry unit weight on the interface shear strength between a cohesive soil and a 1.5 mm thick high density polyethylene (HDPE) smooth geomembrane. Interface direct shear tests were conducted at seven different combinations of compaction water content and dry unit weight at the same normal pressure of 140 kPa. They found that the peak shear stress increased with both increasing water content and increasing dry unit weight and also peak shear stress increased with increasing compactive effort.

Seed et. al (1991) conducted direct shear tests on HDPE/compacted clay interface for two different kinds of clay obtained from two hazardous waste repositories. The first one was a low plasticity, sandy clay bentonite mix and the other was a medium to high plasticity clay bentonite mix. They found that the as-compacted HDPE/clay liner interface frictional angle changed a lot even for relatively minor variations in as-compacted dry density and water content. They found that the contours of the strength values could be drawn in zones roughly parallel to the zero air voids curve.

Daniel et. al (1990) suggested that compacting the soil to a maximum dry unit weight over a specified range in molding water content (w) is based on historical practice for structural fills, where strength and compressibility are the main concerns and not the low hydraulic conductivity. But for clay liners, hydraulic conductivity is an important criterion, therefore a different approach based on water content - density requirements related to hydraulic conductivity and other relevant factors should be applied to assure a good quality control/quality assessment program.

Pein et. al (1991) suggested that the clay liner moisture content has a significant effect on geomembrane/clay interface strength. Firstly, field conditions may be simulated in the laboratory by wetting the surface of the soil and allowing it to swell before placing the
geomembrane, and then the normal load is applied and the interface is sheared. Secondly, by covering the geomembrane with drainage layer material to force the geomembrane into contact with the soil and to minimize temperature effects, swelling of clay can be prevented.

Clay liner properties are heavily dependent on the type of clay and the site specific control of its preparation, deposition and final compaction. On the other hand, high density polyethylene (HDPE) properties are far more accurately defined because they are 100% factory made and controlled in a factory environment. Struve (1991) suggested that a factory controlled clay liner, self sealing composite geomembrane (Gundseal) has advantages in terms of homogeneity, purity, hydraulic conductivity, cost etc. over the in-situ clay. The hydraulic conductivity of Gundseal composite geomembranes is reported to be $4 \times 10^{-10}$ cm/sec, in comparison to the hydraulic conductivity of $1 \times 10^{-7}$ cm/sec. for clay alone.

Lauwers (1991) conducted direct shear tests for a geocomposite consisting of a polyvinyl chloride (PVC) geomembrane laminated to a non-woven polyester fabric and a PVC geomembrane against lean clay, sand and concrete sand. A significant improvement in the friction, puncture and physical properties of the geocomposite over geomembrane was noted. Interface frictional angle increased by 10 degrees by using the geocomposite.

Tensile stresses or “downdrag” may develop in a geomembrane installed over the landfill side slopes due to the settlement of wastes. The geomembrane may be protected by reducing the interface friction above the geomembrane so that it is less than or equal to interface friction below the geomembrane. Even though, a geotextile is often placed above the geomembrane to accomplish this purpose, the resulting low interface friction angle may adversely affect the landfill stability. Pein et. al (1991) recommended that the strains should be calculated along the geomembrane interface. For this, it is necessary to monitor several landfills, measure the movement along the sideslope, test various models, monitor field
installations and develop analytical models. Koerner et. al (1991) described several design models and their development into design equations for cover soil stability and for the induced tensile stresses that are mobilized in the underlying geomembrane.

Carey et. al (1991) suggested that to enhance long term performance of final cover utilizing clay as the barrier layer, the cover design must include a significant amount of soil above the barrier layer. Design of final cover for stability should utilize undrained parameters to predict the response during construction and drained parameters for performance during post - construction. They concluded that the shear stress displacement of a clay/geomembrane interface is dependent on the particular soil and geomembrane used. Therefore testing needs to be performed on the actual materials to verify conformance with the design.

The design and construction of solid - hazardous landfills involve the use of a multi-layered liner system to form a barrier between the waste materials and the underlying foundation soils. The multi-layered liner system includes multiple layers of geonet, filter and drain fabric, compacted clay liner and a layer of high density polyethylene (HDPE) geomembrane.

One of the uncertainties concerning proper landfill design with geosynthetics deals with the interaction of the many components within the cap and/or bottom liner of the landfill. A common failure mechanism of landfills constructed with geosynthetics is slippage between the components of the cap or bottom liner system. As a result, the soil/geosynthetic and geosynthetic/geosynthetic interface friction is an important variable in the proper design of cap and liner systems.

In March of 1988, a slope stability failure occurred in the 6.1 hectare Kettlemen hills hazardous waste landfill. Mitchell et. al (1990) conducted direct shear tests using a modified Karol-Warner direct shear testing apparatus and pull-out tests to evaluate the
interface shear strength characteristics of the various components of the multi-layer liner system of the Kettlemen hills hazardous waste landfill. The interface combinations tested were HDPE geomembrane/goextile interface, HDPE geomembrane/compacted clay liner interface, HDPE geomembrane/geonet interface, geotextile/compacted clay liner interface and geotextile/geonet interface. There was good agreement between the residual interface shear strength characteristics measured in direct shear and pull out box tests for all interface combinations. They recommended a frictional value of 8.5 degrees for dry-liner interface conditions and a frictional value of 8 degrees for submerged conditions. The most critical interfaces within the liner systems were between the HDPE liner and geotextile layer; between the HDPE liner and geonet layer and between the HDPE liner and compacted clay liner (saturated). They reported that the frictional resistance is affected by various properties, including the degree of polishing, whether the interfaces are wet or dry.

Seed et. al (1990) conducted two-dimensional and three-dimensional slope stability analyses to determine the cause of the failure of the Kettleman hills hazardous waste landfill. They found that although wetting of the HDPE liner/compacted clay liner interface may have contributed in some way to the observed slope failure, it is probable that a slope failure of this type would have occurred at about this same stage of fill placement regardless of the actual extent of wetting of this clay/HDPE interface. They suggested that the use of liner interface shear strengths measured in the laboratory test program (Mitchell et. al, 1990) in conjunction with three-dimensional stability analysis methods provided results in good agreement with the failure that occurred at Kettleman hills facility.

Koutsourais et. al (1991) extended the data base for interface friction values of the various components of cap and liner systems for proper landfill design with special emphasis on friction under low to moderate normal loads. They conducted modified direct shear tests for three soil types including a conventional sand, kaolinite clay and fly ash against five types of geotextiles, one flexible geogrid, five geomembranes including HDPE with smooth and roughened surfaces, a very low density polyethylene (VLDPE), a
chlorosulphonated polyethylene (CSPE) and a polyvinyl chloride (PVC) and one geonet. They tested for various interface combinations that corresponded to a typical landfill cross section. They concluded that surface roughness, flexibility and elasticity of the geosynthetic have a great impact on the interface friction angle. The interface friction between layers of geosynthetics and/or soils varies with normal stress. Under low normal stresses, dense cohesionless soils tested against geosynthetic exhibited a higher interface friction angle than under higher normal stresses. Finally, interface frictional angle should be determined on a soil to soil basis. Table 2.3 summarizes the results of the shear tests reported by the earlier researchers till to date.

However, very few studies have been conducted on clay/geomembrane interface saturated with water. (Eigenbrod and Locker 1987; Saxena and Budiman 1985; Mitchell et al 1990; and Seed et. al 1990).

2.7 Behavior of Clay Exposed to Chemicals

It has been known for many years that the permeability of clay soils may be drastically altered by chemicals present in the permeating liquid. Over the last several decades, a number of studies have been carried out to determine the effects of various chemicals on agricultural soils or in geological formations important for oil product. The potential effects of certain organic fluids on clay permeability were recognized as early as 1942, when Macey’s experiments with fireclay showed that the rates of flow for certain organic liquids through clay were of an enormously higher order than for water (Macey, 1942).

Mechanisms whereby the chemical nature of a permeant fluid may alter clay soil permeability and theories to predict clay - chemical interactions have been described by earlier researchers (Mitchell, 1976; Brown and Anderson, 1980). Changes in the permeability of clay soils to chemical interactions may result from:
<table>
<thead>
<tr>
<th>Researcher</th>
<th>Type of Device</th>
<th>Soil type Shearing rate</th>
<th>Type of geosynthetic</th>
<th>Condition tested</th>
<th>Results reported</th>
</tr>
</thead>
<tbody>
<tr>
<td>Koerner et al (1986)</td>
<td>Small direct shear box (102mmx102mm)</td>
<td>Lean clay (CL) 0.127 mm/min.</td>
<td>1. Smooth HDPE 2. Embossed HDPE 3. PVC 4. CPE-5. EPDM</td>
<td>Unsaturated</td>
<td>$\phi = 30^\circ$ 1. $\delta = 15^\circ$ 2. $\delta = 27^\circ$ 3. $\delta = 16^\circ$ 4. $\delta = 17^\circ$ 5. $\delta = 23^\circ$</td>
</tr>
<tr>
<td>Mitchell et al (1990)</td>
<td>Modified Karol-Warner direct shear box (2.8 in x 2.8 in)</td>
<td>Clayey soil 0.127 - 1.27 mm/min.</td>
<td>Smooth HDPE</td>
<td>1. Unsaturated 2. Saturated condition (UU)</td>
<td>$\delta = 11^\circ$ - 14$^\circ$ 2. peak strength = 800 - 1000 psf</td>
</tr>
<tr>
<td>Seed et al (1991)</td>
<td>Modified Karol-Warner direct shear box</td>
<td>Clayey soil + 5% bentonite mix</td>
<td>Smooth HDPE</td>
<td>1. Unsaturated 2. Prolonged saturation (UU)</td>
<td>$\delta = 11.5^\circ$ - 12.5$^\circ$</td>
</tr>
<tr>
<td>Carey et al (1991)</td>
<td>Not mentioned</td>
<td>Medium Plastic calyey soil</td>
<td>Smooth HDPE Textured HDPE</td>
<td>Drained</td>
<td>For smooth HDPE, $\delta = 14^\circ$</td>
</tr>
<tr>
<td>Swan et al (1991)</td>
<td>Direct shear box (305 mm x 305 mm)</td>
<td>Cohesive clay 0.5 mm/min.</td>
<td>HDPE</td>
<td>Unsaturated</td>
<td>Peak shear stress increased with both increasing water content and increasing dry unit weight.</td>
</tr>
<tr>
<td>Koutsourais et al (1991)</td>
<td>Modified direct shear box (305 mm x 305 mm)</td>
<td>Kaolinite clay 0.0025 - 0.25 mm/min.</td>
<td>1. Smooth HDPE 2. Rough HDPE 3. VLDPE 4. CSPE 5. PVC</td>
<td>Unsaturated</td>
<td>$\phi = 6^\circ$ 1. $\delta = 5^\circ$ 2. $\delta = 11^\circ$ 3. $\delta = 4^\circ$ 4. $\delta = 8^\circ$ 5. $\delta = 6^\circ$</td>
</tr>
</tbody>
</table>

UU = Unconsolidated undrained  \( \phi \) = Internal frictional angle  \( \delta \) = Frictional angle of soil to geomembrane
(a) Alterations in soil fabric stemming from chemical influences on the diffuse double layer surrounding clay particles.
(b) Dissolution of soil constituents by strong acids or bases.
(c) Precipitation of solids in soil pores.
(d) Soil pore blockage due to the growth of microorganisms.

Compacted clay is widely used for lining landfills and impoundments in an effort to isolate hazardous and other types of waste materials from the surrounding environment. One major concern about clay liners is that they may be attacked by the chemical wastes or leachates they contain. Although there have been a number of studies conducted to evaluate the potential for migration of industrial wastes, none of these has focused on the rate of movement of organic solvents through soils. The coefficient of permeability is probably the single most important laboratory determined parameter for predicting the movement of solvents through clay liners (Green et al. 1981).

Several investigators have shown that concentrated organic chemicals can attack compacted clay, effectively destroying the barrier characteristics of the liner material (Green et al, 1981; Anderson 1982; Brown et. al, 1983, 1984; Fernandez & Quigley 1985; Foreman and Daniel 1986; and others). Foreman and Daniel (1986) investigated the effects of hydraulic gradient and type of permeameter upon the hydraulic conductivity and intrinsic permeability of compacted clay permeated with organic chemicals. Kaolinite, illite clay and a smectitic clay were permeated with water, methanol, and heptane over a range in hydraulic gradient. When the permeant liquid was water, the hydraulic conductivity of the three clays was virtually identical for all kinds of permeameters tested whereas for clays permeated with concentrated organic chemicals, hydraulic conductivity varied significantly with type of permeameter. They concluded that clays subjected to high confining stress tend to have a reduced susceptibility to attack by organic chemicals.
Permeation of compacted clay with real-world dilute, organic waste liquids did not cause any detrimental effects upon the hydraulic conductivity of compacted clay. However, the concentration limit that separates solutions that will not affect hydraulic conductivity has not been identified. Four organic chemicals over a range of concentrations were permeated through compacted specimens of kaoilinite and illite-chlorite in both rigid and flexible wall permeameters. It was found that dilute organic chemicals less than 80% by volume in an aqueous solution have little effect on the hydraulic conductivity of compacted clay (Bowders and Daniel, 1987). Mechanical stabilization using a large compactive effort (modified proctor compaction) or application of a compressive stress greater than or equal to 10 psi is found to render a compacted clay invulnerable to attack by concentrated organic chemicals under laboratory test conditions (Broderick and Daniel, 1990).

In the landfill, the geomembranes are subjected to harsh components which they must safely contain. Such an environment has been shown to have an extreme effect on the physical properties of geomembranes (Miller, et al., 1991). It is critically important that the permeant liquid used in the compatibility test should be representative of the worst-case that is to be contained at the facility. The interface friction angle, which is a function of the physical properties of the liner and the soil, will also be altered.

Hettiaratchi et al. (1988) studied the shrinkage behavior of clay liner material exposed to simulated municipal solid waste landfill leachate. Recently, Randolph and Raschke (1991) studied how specific chemicals affect the interface friction properties between geomembranes and sand. High density polyethylene (HDPE) was subjected to organic chemicals namely ethyl acetate, turpentine and a single volumetric dilution of nitric acid (50%). Polyvinyl chloride (PVC) was subjected to various chemicals namely kerosene, ethanol, and two volumetric dilutions of nitric acid (30% and 50%), and the changes in the physical properties were measured at 30 days intervals. Modified direct shear tests of degraded HDPE and PVC membranes were conducted against sand. They found that there was a significant drop in the friction angle for the degraded PVC whereas it was constant in
the case of the HDPE.

However, to the author’s knowledge no study has been conducted on effects of landfill leachate on the geomembrane/clay liner interface shear strength.
CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 Introduction

As seen in the literature review, characteristics of clay/geomembrane interface saturated with water or leachate could be critical for the design of landfills. To identify the characteristics, an experimental program was established. The soil utilized in the investigation was obtained from a site located near Ohio University in Athens, Ohio. The required amount of a clayey soil was removed from the ground using a hand auger and shovel. The soil was oven dried at a temperature of 105 - 110° C, then pulverized and passed through different sieves depending upon the type of test to be conducted. The sieved soil was mixed thoroughly and stored in sealed bins. High density polyethylene (HDPE) geomembrane samples were provided by Gundle Lining Systems and Polyvinyl chloride (PVC) geomembrane material was supplied by Water Saver. The leachate was obtained from the Suburban landfill in Brownsville, located about 10 miles from Columbus, Ohio.

3.2 Characteristics of Soil

According to a survey report of Athens County, Ohio (1985), the site, from where the soil samples were obtained is in the unglaciated Allegheny Plateau Region. It is underlain by sedimentary rock. Subsoils found within the top 5 feet belong to the Guernsey - Upshur complex. These soils are commonly situated on ridge tops, side slopes, and narrow benches. They are dark yellowish brown or brown, friable silty clay loam and have low permeability and a high shrink - swell potential. Depth to bedrock ranges typically from 50 to 80 inches, and the bedrock is relatively soft. The soils may develop perched
groundwater at a depth of 2 to 3.5 feet during the period of January to April.

In order to make sure that the clay material present at the site was suitable for landfill liner construction, a series of standard laboratory tests were performed on the samples obtained from a depth of 2 feet. These tests included the natural moisture content test, specific gravity, the Atterberg liquid and plastic limit tests, hydrometer analysis, soil classification, falling head permeability test and the Standard Proctor test. The natural moisture content test was conducted according to ASTM D2216, and specific gravity was determined according to ASTM D70. The Atterberg limits, liquid limit and plastic limit tests were conducted according to ASTM D4318.

About 250 gm of soil passed through sieve No. 40 was used in the test. The liquid limit test was conducted starting from wet to dry using Casagrande’s liquid limit apparatus in which the portion of the prepared soil was placed in the cup of the liquid limit device, and a groove was formed in the soil pat using the grooving tool. The number of blows required to close the groove was recorded, and a small amount of soil was taken from the closed portion for moisture content determination. The tests were repeated four to five times to obtain the corresponding number of blows between 10 and 40.

About 20 gm of soil was selected from the material prepared for the liquid limit test. The water content of the soil was reduced to a consistency at which it could be rolled without sticking to the hands by spreading and mixing continuously. The soil mass was rolled between the palm and the glass plate with just sufficient pressure to roll the mass into a thread of uniform diameter of 3.2±0.5 mm. The crumbled thread was retained for moisture content determination. The process was repeated four to five times on different 2 gm portions of soil and kept for moisture content determination to obtain an average water content. The Atterberg limits were used for soil identification and classification.
The hydrometer method of grain size analysis (ASTM D421) was performed on soil passing through sieve No. 200. About 50 grams of soil was mixed with 125 ml of 4% sodium metaphosphate solution and was allowed to stand for a period of 16 hours. This mixture was then transferred to a sedimentation cylinder and distilled water was added to the 1000 ml mark. The solution was agitated for 1 minute and the hydrometer and temperature readings were taken at 1, 2, 3, 4, 8, 16, 30, 60 minutes, ......., for a 96 hour period. This test determines the percentage of silt and clay present in the soil.

The falling head permeability test was attempted by using a 6.35 cm diameter permeameter with a 100 ml burette (of cross sectional area 1.6135 cm²). The test was conducted for a soil sample of 4.064 cm thickness over 48 hours with the initial hydraulic gradient of about 15.3%. In the falling head permeability test, no clear evaluation of the coefficient of permeability was possible since no measurable drawdown was observed in the standpipe. If the coefficient of permeability of the sample was higher than 1 x 10⁻⁷ cm/sec, a drawdown of 2.54 cm (=1 inch) or more would have resulted within 24 hours. Therefore, it may be concluded that the clay soil at the site has a coefficient of permeability of less than 1 x 10⁻⁷ cm/sec.

Compaction tests were conducted using a Standard Proctor compaction mold, according to ASTM D698, in which soil passing through sieve No. 4 was used. About 2 kg of soil were mixed with an initial water content of 12%. Soil was compacted in three equal layers in the compaction mold using a rammer weight of 2.5 kg and 25 blows of the rammer to each layer. After compacting each layer, care was taken to scratch the surface in order to ensure bondage between the layers. After compacting the third layer extra projecting soil was trimmed, and the weight of the mold and soil was determined. The cylinder of soil was extruded from the mold and water content was determined for the soil sample, one near the top and one near the bottom. The process was repeated for different water contents with increments of 3% until the decrease in weight of the mold and soil was
noted. Then the process was repeated three more times to ensure sufficient number of points were on either side of the peak value. Since the direct shear tests were performed on soil passing through sieve No. 10, compaction tests were also run on soil passing through sieve No. 10. Each point in both the tests was repeated twice, and the average value was taken in the computations.

Table 3.1 presents the results of the tests described above. Figure 3.1 shows the resultant grain size distribution curve for the soil. Based on these test results, the soil can be classified as CL ("lean clay") type by the Unified Soil Classification System and as A-7-6 by the AASHTO soil classification system.

Figure 3.2 shows the plot of dry density versus moisture content for soil passing through sieves No. 4 and 10. As seen in the figure, dry density increases as the water content increases up to a particular point after which the dry density decreases. The point at which this transition takes place corresponds to optimum moisture content and the corresponding density is maximum dry density. The parameters, maximum dry density and optimum moisture content, are very important, because in the field the soil is generally compacted at 2-3% wet of optimum and 95% of maximum dry density to achieve a relatively low coefficient of permeability.

3.3 Characteristics of Leachate

The leachate samples were collected from the Suburban landfill located in Brownsville, Ohio. The landfill is operated by Waste Management of North America, Inc. The total landfill area is about 21 acres. The landfill accepts solid wastes from Newark, Zanesville and surrounding areas and a small amount of solid waste from Columbus. When leachate samples were taken, the construction of the landfill was almost complete and the preparation of the landfill cap was getting ready. Leachate samples were taken from the storage tank.
TABLE 3.1

Index Properties of the Soil

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>In situ moisture content (%)</td>
<td>27 - 28</td>
</tr>
<tr>
<td>Hygroscopic moisture content (%)</td>
<td>3.3 - 4.0</td>
</tr>
<tr>
<td>Liquid limit, WL (%)</td>
<td>48.0</td>
</tr>
<tr>
<td>Plastic limit, Wp(%)</td>
<td>24.5</td>
</tr>
<tr>
<td>Plasticity Index, Ip(%)</td>
<td>23.5</td>
</tr>
<tr>
<td>Specific gravity, G</td>
<td>2.66</td>
</tr>
<tr>
<td>Maximum dry density, d (pcf)</td>
<td></td>
</tr>
<tr>
<td>Passing through sieve No. 4</td>
<td>99.8</td>
</tr>
<tr>
<td>Passing through sieve No. 10</td>
<td>102.0</td>
</tr>
<tr>
<td>Optimum moisture content, OMC (%)</td>
<td>19.5</td>
</tr>
</tbody>
</table>
Figure 3.1 Grain Size Distribution Curve for Soil
Figure 3.2 Dry Density Versus Moisture Content Relationship for Clay
Various tests such as suspended solids, dissolved solids, total solids, chemical oxygen demand (COD), total organic carbon (TOC), pH, and metal analysis were conducted to characterize the leachate. Procedures followed Standard Methods for the Examination of Water and Wastewater (1985).

The membrane filter used in the determination of suspended solids was 47 mm in diameter and 0.45 micrometer in pore opening. The residue retained on the filter was dried to a constant weight in an oven at 103 - 105°C to determine the mass of suspended solids. The solution which passed through the filter was evaporated in a weighed dish and dried to constant weight in an oven at 103 - 105°C to determine the dissolved solids. Total solids were measured by evaporating a known amount of leachate sample in a weighed dish and drying to a constant weight in an oven at 103 - 105°C.

The chemical oxygen demand (COD) test is used to measure the content of organic matter in the leachate sample. The COD test was performed using a Hach model COD reactor and Spectronic 20 spectrometer. The procedure outlined in the Hach manual was followed.

The total organic carbon (TOC) test is used to measure the amount of organic matter present in the leachate sample, and is especially applicable to small concentrations of organic matter. The TOC test was performed using a DC-80 series carbon analyzer provided by the Dohrman company and following the procedures outlined in the DC-80 manual.

The measurement of pH or hydrogen ion concentration in the leachate sample was made using a pH meter model 290A, provided by the Orion company. Metal analysis was performed using a Perkin-Elmer model 2380 atomic absorption spectrophotometer. The standard solutions for the metals used in the analysis were obtained from the Fisher
Two samples of leachate from the same landfill were used in the study since the tests were conducted at different times. The landfill was nearing closure at the time of sampling, and the final cover was being constructed. Leachate I was obtained on October 25, 1991, and Leachate II was obtained on January 15, 1992. Table 3.2 presents the results of the tests described above for both samples of leachate. As seen from the table, there is a decrease in the concentration for leachate II when compared to leachate I which can be attributed to the fact leachate II was collected at a later time. When these characteristics are compared with the typical values presented in Table 2.2, it is observed that the leachate used in this investigation is weak.

3.4 Immersion of Geomembranes in Leachate I (EPA Method 9090)

Method 9090 is the standard test established by the U.S. Environmental Protection Agency (U.S. EPA) to determine the chemical compatibility of flexible membrane liners to be used in solid waste containment applications. This test simulates the conditions that the flexible membrane liner (FML) may encounter in service and assesses what effects, if any, the exposure to the leachate has on the FML. The test procedure involves several steps, including:

(a) Selection of representative samples of the leachate and the FML
(b) Preparation of the exposure cells for operation during the exposure period.
(c) Exposure of the FML samples to the leachate in the simulated service environment.
(d) Physical and analytical testing of the unexposed and exposed FML samples, and
(e) Analysis of test data for trends during exposure period.
TABLE 3.2

Characteristics of the Leachate

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Leachate I</th>
<th>Leachate II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total suspended solids, TSS (mg/L)</td>
<td>83</td>
<td>66</td>
</tr>
<tr>
<td>Total dissolved solids, TDS (mg/L)</td>
<td>3676</td>
<td>1375</td>
</tr>
<tr>
<td>Total solids, TS (mg/L)</td>
<td>3836</td>
<td>1486</td>
</tr>
<tr>
<td>Chemical oxygen demand, COD (mg/L)</td>
<td>619</td>
<td>550</td>
</tr>
<tr>
<td>Total organic carbon, TOC (mg/L)</td>
<td>83.2</td>
<td>54.1</td>
</tr>
<tr>
<td>Hydrogen ion concentration, pH</td>
<td>8.1</td>
<td>7.0</td>
</tr>
<tr>
<td><strong>Metals</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Calcium, Ca (mg/L)</td>
<td>90.7</td>
<td>82.8</td>
</tr>
<tr>
<td>Magnesium, Mg (mg/L)</td>
<td>2247</td>
<td>900</td>
</tr>
<tr>
<td>Iron, Fe (mg/L)</td>
<td>21.3</td>
<td>20.4</td>
</tr>
<tr>
<td>Manganese, Mn (mg/L)</td>
<td>1.4</td>
<td>0.9</td>
</tr>
</tbody>
</table>
The FML samples selected for exposure and testing should be free of flaws and defects in order that the test specimens prepared from them are, as nearly as possible, influenced only by exposure to the leachate in the simulated service environment. Maintenance of constant exposure conditions is important since variation in any of them may influence the quality of the data collected from testing the exposed samples. Factors that are critical to the performance of Method 9090 include:

(a) Use of exposure cells made of materials that are not reactive with the leachate.
(b) Constant temperature of the leachate.
(c) Stirring of the leachate to prevent stratification of phases.
(d) Exchange of the leachate in the exposure cells monthly or more frequently, to maintain constant concentrations of constituents that may be reduced due to their uptake by the FML samples or volatilization into a headspace in the exposure cell
(e) Maintenance of zero headspace in sealed exposure cells to reduce the potential loss of volatile organic constituents from the leachate.

Nine smooth HDPE and nine textured HDPE specimens were cut to an approximate size of 180 x 180 mm. Nine PVC specimens were cut to an approximate size of 200 x 200 mm. The difference was due to the expected shrinkage in the PVC specimens.

Containers chosen to contain the specimens for immersion were of sufficient size to contain the samples with provisions for supporting the liner specimens so that they did not touch the bottom or sides of the container or each other, and for stirring the leachate in the container. Two holes were drilled near the top on either side of the container at a sufficient distance so that glass rods could be inserted. Two holes were punched at the top of the liner specimens, so that they could be suspended from glass rods by means of nylon thread. Two holes were punched at the bottom of the liner specimens, and small weights were attached to ensure that the specimen was suspended by its own weight and did not touch adjacent sheets. Different containers were used for each kind of specimen. The containers
were closed airtight to ensure that there was no evaporation of the leachate. Each container was filled with sufficient leachate so that all the specimens were immersed fully. Mixing was done once each week. The temperature was maintained at 23 ± 2°C as required by the EPA test method.

One specimen from each container was taken out after periods of 7 days and 30 days. Testing was conducted for the following physical properties prior to and after immersion:

(a) gauge thickness in inches - average of the four corners and the center.
(b) mass in grams.
(c) length in mm - average of the lengths of the two sides and the length measured at the center of the liner specimen (vertical immersion dimension).
(d) width in mm - average of the widths of the two sides and the width measured at the center of the liner specimen (horizontal immersion dimension).

Leachate samples were taken from each container after a period of 30 days and were tested for various characteristics as described in section 3.3.

3.5 Direct Shear Tests

A conventional small scale direct shear device, having a split box size of 2 inch square, was utilized in the investigation. This device was equipped with an electric motor, a force ring, two displacement dial gages, and weights on a suspending arm. The electric motor was required to achieve a displacement controlled mode. The force ring had a capacity of 2,000 lbs. When tests were conducted with the clay/geomembrane interface, the device was modified. The geomembrane specimen was attached to a rigid steel plate anchored on top of the lower box, and the soil was placed in the upper box. Different
combinations of weights were applied to vary the normal stress level from 9.08 psi to 35.48 psi. All direct shear tests were conducted inside a chamber where both temperature and humidity can be controlled. All tests were conducted at a temperature of 23± 2º C.

3.5.1 Preparation of Specimen

The soil passing through sieve No. 10 was mixed at a water content of 2 - 3 % wet of optimum. The mixed soil was compacted in the Proctor’s mold according to ASTM D698. After weighing the compacted soil to calculate dry density, the cylinder of soil was extruded from the mold. Using the trimmer, samples were taken from the compacted soil and both sides trimmed to have a smooth surface. The trimmer had dimensions 2 x 2 x 0.75 inches. After weighing the trimmer and soil, the sample from the trimmer was transferred to the shear box very carefully. To ensure smooth transfer from the trimmer to the direct shear box, a coating of oil was applied on the inner faces of the trimmer while taking out the samples from the compacted soil.

3.5.2 Test Conditions

Direct shear tests were conducted under the following three different conditions for the clay/geomembrane interface:

(a) unsaturated condition,
(b) condition in which the interface was saturated with water, and
c) condition in which the interface was saturated with leachate.

Tests were also conducted on clay alone under the unsaturated condition and saturated condition in which the saturating media were water and leachate. Unsaturated condition means running the direct shear test on clay compacted at 95% maximum dry
density and 2-3% wet of optimum.

3.5.2.1 Unsaturated Condition

In the first part, the effect of shearing rate on the frictional angle of clay was investigated. For this, direct shear tests were conducted on clay alone at eight different shearing rates, i.e. 0.778, 0.605, 0.535, 0.462, 0.451, 0.432, 0.428, and 0.425 mm/min. At each shearing rate, the normal stress was varied from 9.08 psi to 35.48 psi. For each shearing rate, two of the four tests were duplicated.

In the case of clay/geomembrane interfaces under unsaturated conditions, tests were conducted under three different conditions:

(a) fresh geomembrane
(b) geomembrane immersed in leachate for a period of 7 days.
(c) geomembrane immersed in leachate for a period of 30 days.

For the unsaturated condition, either with clay alone or the clay/geomembrane, after the sample was transferred to the direct shear box, the box was moved to the equipment, and the normal stress was applied. Once the sample had reached equilibrium under the load (constant vertical deformation dial reading), the sample was sheared at a constant strain rate of 0.425 mm/min. Four tests were performed for each kind of geomembrane under each condition outlined above, and two of them were duplicated.

3.5.2.2 Condition in Which the Interface was Saturated with Water

Tests were conducted both on clay alone and the clay/geomembrane. In both cases, after the sample had reached equilibrium under the application of the normal load (constant vertical deformation dial reading), the clay sample was saturated with water for a period of
24 hours and then sheared at a rate of 0.425 mm/min. Four tests were performed for each kind of geomembrane and two of them were duplicated. To ensure that 24 hour saturation was sufficient, some samples were saturated for 2, 3, and 4 days for different samples under the same normal stress and then sheared.

3.5.2.3 Condition in Which the Interface was Saturated with Leachate

Tests were conducted on both clay alone and the clay/geomembrane. In the case of clay/geomembrane interfaces under saturated condition, tests were conducted under the three different conditions outlined in section 3.5.2.1.

In all the cases, after the sample had reached equilibrium under the normal load application (constant vertical deformation dial reading), the sample was saturated with leachate for a period of 24 hours and then sheared at 0.425 mm/min. Four tests were performed for each kind of geomembrane under each condition outlined in section 3.5.2.1, and two of them were duplicated. To ensure that 24 hour leachate saturation was sufficient, some samples were saturated for 1, 2, 3, and 4 days for different samples under the same normal stress and then sheared.
CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 Introduction

Experimental work was carried out according to the methodologies described in Chapter 3. Results of all the experimental and analytical studies are presented along with relevant discussions in this chapter. By the end of the experimental program, a total of 216 direct shear tests were conducted, 75 tests for clay alone and 141 tests for the geomembrane/clay liner interfaces. Because of the time consuming nature of the experimental program as a whole and a limited amount of leachate collected at the landfill each time, two types of leachate samples were utilized in the experimental program. Leachate I was used for performing short term saturation tests on the clay liner and clay liner/geomembrane interfaces. It was also used in conducting EPA Method 9090 type tests. Leachate II was used to evaluate interface shear strength between the clay liner and geomembranes submerged in leachate for 7 and 30 days.

Before the initiation of the actual direct shear testing, a background study was undertaken to determine the effect of shearing rate on the internal friction angle (φ) of the compacted clay. This was necessary to establish a standard shearing rate for all subsequent tests. The first section of this chapter presents findings of the shearing rate effect study.

4.2 Effect of Shearing Rate on the Frictional Angle of Clay

Figure 4.1 shows shear stress vs. horizontal displacement curves for shearing of compacted clay at different shearing rates under a normal stress of 13.63 psi. It can be seen that lowering the shearing rate increased the maximum shear stress reached in the clay.
Figure 4.1 Shear Stress Versus Horizontal Displacement for Shearing of Compacted Clay at Different Shearing Rates
For a shearing rate of 0.778 mm/min, the shear stress mobilized is far less when compared to the shear stress mobilized for other shearing rates. Figures 4.2-(a) and 4.2-(b) present maximum shear stress vs. normal stress plots for the clay sheared at different shearing rates. In general, both internal frictional angle ($\phi$) and cohesion ($c$) increased as the shearing rate decreased.

Figure 4.3 shows the plot of frictional angle vs. shearing rate for the compacted clay. It can be seen that as the shearing rate decreases, frictional angle increases. This trend continues up to a certain shearing rate after which the frictional angle becomes almost constant (i.e. the curve becomes asymptotic). When the shearing rate was slower than this threshold value, a steady frictional angle of 27.8 psi was observed. Therefore, for all the subsequent direct shear tests it was determined to conduct the test at a fixed shearing rate of 0.425 mm/min. This shearing rate is comparable to the shearing rate chosen by the other researchers. O'Rourke et. al. (1990) conducted their direct shear tests at a shearing rate varying from 0.4 to 0.6 mm/min., and, Koerner et. al (1986) selected a fixed shearing rate of 0.127 mm/min. for their experiments. Mitchell et. al (1990) conducted their experiments at a shearing rate between 0.127 - 1.27 mm/min. to analyze the failure of Kettleman hills waste landfill slope. Swan et. al (1991) used a shearing rate of 0.5 mm/min. in their investigation and Eigenbrod et. al (1987) conducted the experiments on clay interfaced with woven and non woven geotextile at a shearing rate of 0.24 mm/min.

### 4.3 Unsaturated Clay Liner/Geomembrane Interfaces

A total of 36 tests were performed to determine the unsaturated interface shear strength for clay liner/smooth HDPE, clay liner/textured HDPE, and clay liner/PVC interfaces. Results from these tests established the baseline interface shear strength to detect the effect of various factors in subsequent experiments. Figure 4.4 presents shear
Figure 4.2-(a) Maximum Shear Stress vs. Normal Stress Plots for Shearing of Compacted Clay at Different Shearing Rates

- 0.778 mm/min
  $c = 14.27$ psi
  $\phi = 13.03$ degrees

- 0.605 mm/min
  $c = 14.51$ psi
  $\phi = 18.03$ degrees

- 0.535 mm/min
  $c = 19.30$ psi
  $\phi = 20.69$ degrees
Figure 4.2-(b) Maximum Shear Stress vs. Normal Stress Plots for Shearing of Compacted Clay at Different Shearing Rates
Figure 4.3 Effect of Shearing Rate on the Frictional Angle of Compacted Clay
Figure 4.4 Shear Stress Versus Horizontal Displacement Plots Under Unsaturated Condition
stress vs. horizontal displacement plots for these interfaces as well as for clay alone under a normal stress of 17.74 psi. Although peak shear stress was reached for the clay/smooth HDPE and clay/PVC interfaces at extremely small horizontal displacements, the clay/textured HDPE required to be displaced almost as much as the clay (in its shear test) to initiate its shear failure. The performance of the clay/smooth HDPE interface was only slightly below that of the clay/PVC interface. It is interesting to note that when these geomembranes are each sheared against a granular soil there will be a much more noticeable difference in performance between the PVC and smooth HDPE geomembranes.

Figure 4.5 shows maximum shear stress vs. normal stress for the three types of interface and clay alone. Resulting cohesion and frictional angle values are indicated on the side of the graphical plots. While the frictional angle for clay/smooth HDPE and clay/PVC interfaces (δ) were almost the same, the clay/textured HDPE interface frictional angle was higher, and it was close to the internal frictional angle of clay (φ). These values compare well with the results reported previously by Koerner et. al (1986). In their experiments, they obtained a frictional angle of 15°, 27° and 16° for smooth HDPE, textured HDPE and PVC against lean clay respectively. They also noted that the performance of clay/PVC was slightly above the clay/smooth HDPE which is very well comparable with the results presented in this investigation. Mitchell et. al (1990) reported a frictional angle of 11° to 14° for clay/smooth HDPE under unsaturated condition. The lower frictional angle in their case can be attributed to the faster shearing rate that was used in the experiment.

Table 4.1 summarizes the efficiency ratio values for these interfaces in terms of cohesion and frictional angle. The efficiency on cohesion, \( E_c \) is defined as

\[
E_c = \frac{C_a}{C} \quad \cdots (4.1)
\]

where \( E_c \) = efficiency on cohesion.
Figure 4.5 Maximum Shear Stress vs. Normal Stress Plots for Clay/Geomembrane Interfaces Under Unsaturated Condition
<table>
<thead>
<tr>
<th>Description</th>
<th>$E_c$ (%)</th>
<th>$E_{\phi}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay/Clay</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Clay/Smooth HDPE</td>
<td>11</td>
<td>59</td>
</tr>
<tr>
<td>Clay/Textured HDPE</td>
<td>17</td>
<td>85</td>
</tr>
<tr>
<td>Clay/PVC</td>
<td>13</td>
<td>60</td>
</tr>
</tbody>
</table>
\[ C_a = \text{adhesion of soil to geomembrane.} \]
\[ C = \text{cohesion of soil to soil.} \]

and efficiency on friction is defined as

\[ E_{\phi} = \frac{\tan \delta}{\tan \phi} \quad \cdots (4.2) \]

where \( E_{\phi} \) = efficiency on friction.
\( \delta \) = friction angle of soil to geomembrane.
\( \phi \) = internal friction angle of soil.

It can be seen from the table that while the clay/smooth HDPE and clay/PVC interface have almost the same efficiency on cohesion and efficiency on friction, the clay/textured HDPE interface has a higher efficiency on cohesion and efficiency on friction. Koerner et. al (1986) reported efficiency on friction of 50\%, 90\%, and 53\% for smooth HDPE, textured HDPE and PVC against a lean clay under the unsaturated condition. These values are in good agreement with the values presented in Table 4.1. However, Koerner et. al reported higher value of efficiency on cohesion for all the three geomembranes.

### 4.4 Clay Liner/Geomembrane Interfaces Saturated with Water

The clay liner/geomembrane interfaces were saturated with tap water and sheared after the expansion rate of the compacted clay reached a limit. In the literature, the time of saturation of the sample varies between as low as 6 hours to as high as 7 days. In this study, a few different samples under the same normal stress were saturated for 1, 2, 3 and 4 days and then sheared. Table 4.2 presents the results of the same. All the samples reached almost the same maximum shear stress at the same horizontal displacement for each of the
TABLE 4.2

Effect of Saturation Time on Clay/Geomembrane Interfaces with Water as Saturating Media

Normal Stress: 17.74 psi

<table>
<thead>
<tr>
<th>Types of Interfaces</th>
<th>Saturation Time (days)</th>
<th>Maximum Shear Stress Reached (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Clay alone</td>
<td>15.033</td>
<td>15.133</td>
</tr>
<tr>
<td>Clay/Smooth HDPE</td>
<td>5.231</td>
<td>5.245</td>
</tr>
<tr>
<td>Clay/Textured HDPE</td>
<td>8.824</td>
<td>8.948</td>
</tr>
<tr>
<td>Clay/PVC</td>
<td>5.686</td>
<td>5.785</td>
</tr>
</tbody>
</table>
conditions. Hence a period of 24 hours was sufficient to saturate the sample and was selected as the time of saturation.

Figure 4.6 presents shear stress vs. horizontal displacement for the three types of saturated clay/geomembrane interfaces and saturated clay under a normal stress of 13.63 psi. Contrary to the unsaturated case, a more noticeable difference in peak performance appeared between the clay/smooth HDPE and clay/PVC interfaces. For all the interfaces, the amount of horizontal shear displacement at failure initiation was reduced to some degree.

As for the clay liner alone, as shown in Figure 4.7, it lost about 23.8% of its internal frictional angle and 57.1% of cohesion upon saturation with water. Interface saturation with water reduced both cohesion and frictional angle for all the interfaces (Figure 4.8 to Figure 4.10). Among the three types of clay/geomembrane interfaces, influence of water saturation was least pronounced for the clay/textured HDPE interface (Figure 4.9) and most significant for the clay/smooth HDPE interface (figure 4.8). Maximum reduction of about 25.7% in the frictional angle and about 74% in cohesion was observed for the clay/smooth HDPE interface. Reduction of about 18.5% in the frictional angle and about 35.1% in the cohesion was observed for the clay/textured HDPE interface. Similar results were reported by Carey et. al (1991) and Seed et. al (1991) for clay/smooth HDPE interface. Carey et. al (1991) reported a frictional angle of 14° upon saturation with water for clay/smooth HDPE interface against 12.74° obtained in this investigation. Seed et. al (1991) reported a value of 11.5° to 12.5° for clay/smooth HDPE interface saturated with water.

Table 4.3 summarizes efficiency ratio values for these interfaces saturated with water. It is interesting to note that efficiency on friction is almost the same for both the unsaturated (Table 4.1) and saturated case with water. While the efficiency on cohesion for clay/smooth HDPE and clay/PVC interfaces decreased upon saturation with water, it
Figure 4.6 Shear Stress Versus Horizontal Displacement Plots for Clay/Geomembrane Interfaces Saturated with Water
Clay alone (unsaturated)
\[ c = 16.49 \text{ psi} \]
\[ \varphi = 27.82 \text{ degrees} \]

Clay alone (saturated)
\[ c = 7.08 \text{ psi} \]
\[ \varphi = 21.21 \text{ degrees} \]

Figure 4.7 Maximum Shear Stress vs. Normal Stress for Clay alone Saturated with Water
Figure 4.8 Maximum Shear Stress vs. Normal Stress for Clay/smooth HDPE Interface Saturated with Water

- Clay/smooth HDPE (unsaturated)
  - $c = 1.81$ psi
  - $\delta = 17.14$ degrees

- Clay/smooth HDPE (saturated with water)
  - $c = 0.47$ psi
  - $\delta = 12.74$ degrees
Clay/textured HDPE (unsaturated)
ca = 2.82 psi
δ = 24.21 degrees

Clay/textured HDPE (saturated with water)
ca = 1.83 psi
δ = 19.74 degrees

Figure 4.9 Maximum Shear Stress vs. Normal Stress for Clay/textured HDPE Interface Saturated with Water
Clay/PVC
(unsaturated)
\( c_a = 2.07 \) psi
\( \delta = 17.54 \) degrees

Clay/PVC
(saturated with water)
\( c_a = 0.73 \) psi
\( \delta = 13.66 \) degrees

Figure 4.10 Maximum Shear Stress vs. Normal Stress for Clay/PVC Interface Saturated with Water
TABLE 4.3

Efficiency Ratios for Clay/Geomembrane Interfaces Saturated With Water

<table>
<thead>
<tr>
<th>Description</th>
<th>$E_c$ (%)</th>
<th>$E_{\Phi}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay/Clay</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Clay/Smooth HDPE</td>
<td>7</td>
<td>58</td>
</tr>
<tr>
<td>Clay/Textured HDPE</td>
<td>26</td>
<td>93</td>
</tr>
<tr>
<td>Clay/PVC</td>
<td>10</td>
<td>63</td>
</tr>
</tbody>
</table>
increased for clay/textured HDPE interface upon saturation with water when compared to unsaturated case.

4.5 Clay Liner/Geomembrane Interfaces Saturated with Leachate I

The next stage of the experimental program was to saturate the clay liner/geomembrane interfaces with Leachate I and perform direct shear tests when the expansion rate of the compacted clay becomes negligible. As no work has been done on clay/geomembrane interface saturated with leachate, there is no literature available regarding the saturation time required for the clay sample to attain equilibrium. Hence a few different samples under the same normal stress were saturated with leachate for 1, 2, 3 and 4 days. Table 4.4 presents the results of the same. It was interesting to note that just like the water saturation case, all the samples reached almost the same maximum shear stress at the same horizontal displacement for each of the conditions. Hence a period of 24 hours was sufficient to saturate the sample and was selected as the time of saturation.

Figure 4.11 presents shear stress vs. horizontal displacement for the three types of leachate saturated clay/geomembrane interfaces and leachate saturated clay under a normal stress of 17.74 psi. Contrary to the unsaturated case and water saturation case, the clay/PVC interface exhibited the poorest frictional characteristics among the three interfaces. For all the interfaces, the amount of horizontal shear displacement at failure initiation was further reduced from those observed for the water saturated interface (Figure 4.6). As for the compacted clay alone, it lost about 8.5% internal frictional angle and 58.8% cohesion upon saturation with Leachate I (Figure 4.12 and 4.13). Figures 4.14, 4.16 and 4.18 shows shear stress vs. horizontal displacement for clay/smooth HDPE, clay/textured HDPE and clay/PVC interfaces under three different conditions namely unsaturated, water saturation and leachate I saturation cases. While for clay/smooth HDPE and clay/PVC interfaces, maximum shear stress is reached at lower horizontal displacement, it is reached at higher displacement for clay/textured HDPE interface. There was an increase in cohesion
TABLE 4.4

Effect of Saturation Time on Clay/Geomembrane Interfaces with Leachate I as Saturating Media

Normal Stress : 13.63 psi

<table>
<thead>
<tr>
<th>Types of Interfaces</th>
<th>Saturation Time (days)</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay alone</td>
<td>Maximum Shear Stress Reached (psi)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay/Smooth HDPE</td>
<td>5.458</td>
<td>5.550</td>
<td>5.570</td>
<td>5.455</td>
<td></td>
</tr>
<tr>
<td>Clay/Textured HDPE</td>
<td>7.846</td>
<td>7.955</td>
<td>7.900</td>
<td>7.835</td>
<td></td>
</tr>
<tr>
<td>Clay/PVC</td>
<td>5.458</td>
<td>5.569</td>
<td>5.650</td>
<td>5.490</td>
<td></td>
</tr>
</tbody>
</table>
Figure 4.11 Shear Stress Versus Horizontal Displacement Plots for Clay/Geomembrane Interfaces Saturated with Leachate
Figure 4.12 Shear Stress Versus Horizontal Displacement Plots for Clay alone under Different Conditions
Clay alone (unsaturated)

\[ c = 16.49 \text{ psi} \]
\[ \phi = 27.82 \text{ degrees} \]

Clay alone (water saturation)

\[ c = 7.08 \text{ psi} \]
\[ \phi = 21.21 \text{ degrees} \]

Clay alone (Leachate saturation)

\[ c = 6.791 \text{ psi} \]
\[ \phi = 25.46 \text{ degrees} \]

Figure 4.13 Maximum Shear Stress vs. Normal Stress for Clay alone Under different Conditions
Figure 4.14 Shear Stress Versus Horizontal Displacement
Plots for Clay/smooth HDPE Interface
Under Different Conditions
Figure 4.15 Maximum Shear Stress vs. Normal Stress for Clay/smooth HDPE Interface Under Different Conditions
Figure 4.16 Shear Stress Versus Horizontal Displacement Plots for Clay/textured HDPE Interface Under Different Conditions
Figure 4.17 Maximum Shear Stress vs. Normal Stress for Clay/textured HDPE Interface Under Different Conditions
Figure 4.18 Shear Stress Versus Horizontal Displacement Plots for Clay/PVC Interface Under Different Conditions
for clay/ smooth HDPE and clay/PVC interfaces and a decrease for clay/textured HDPE upon saturation with leachate I (Figures 4.15, 4.17 and 4.19). While there was a reduction of about 22% in frictional angle and an increase of 11.4% in cohesion for clay/smooth HDPE interface, a reduction of about 6.5% in frictional angle and 38.3% reduction in cohesion was observed for clay/textured HDPE interface.

Table 4.5 summarizes the results of the tests conducted on clay alone under different conditions. It can be seen that the friction angle for saturation with leachate I is higher than the case of saturation with water. This was true for the three clay/geomembrane interfaces also. The probable reason for this may be that the density of leachate is slightly higher than water, and, hence, the shear stress required to shear the sample is more, and, thus a higher frictional angle results than that for water.

Table 4.6 summarizes efficiency ratio values for the interfaces saturated with Leachate I. The efficiency on cohesion increased upon saturation with leachate I for clay/smooth HDPE and clay/PVC interfaces and was constant for clay/textured HDPE when compared to the water saturation case. The efficiency on friction was almost the same for all the three cases irrespective of unsaturated, water saturation and leachate I saturation.

4.6 Aging of Geomembranes (Due to Leachate I)

As described in Section 3.4, U.S. EPA Method 9090 test was applied using Leachate I for the selected geomembrane specimens of smooth HDPE, textured HDPE, and PVC material. After 7 day and 30 day periods, each type of geomembrane specimen was removed from the test tank, and its physical characteristics were evaluated. Table 4.7 summarizes the data obtained through this “aging” test. It was observed that the smooth HDPE and textured HDPE geomembranes probably absorbed some moisture and slightly increased in mass after submersion in the leachate. The volume change for both was negligible.
Normal Stress (psi)
\[ \sigma = 2.07 \text{ psi} \]
\[ \delta = 17.54 \text{ degrees} \]

Clay/PVC (saturated with water)
\[ \sigma = 0.73 \text{ psi} \]
\[ \delta = 13.66 \text{ degrees} \]

Clay/PVC (saturated with leachate)
\[ \sigma = 2.38 \text{ psi} \]
\[ \delta = 14.45 \text{ degrees} \]

Figure 4.19 Maximum Shear Stress vs. Normal Stress for Clay/PVC Interface Under Different Conditions
TABLE 4.5

Frictional Characteristics for Clay Alone Under Different Conditions

<table>
<thead>
<tr>
<th>Description</th>
<th>C (psi)</th>
<th>$\phi$ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unsaturated</td>
<td>16.49</td>
<td>27.82</td>
</tr>
<tr>
<td>Saturated with water</td>
<td>7.08</td>
<td>21.21</td>
</tr>
<tr>
<td>Saturated with Leachate I</td>
<td>6.79</td>
<td>25.46</td>
</tr>
</tbody>
</table>
**TABLE 4.6**

Efficiency Ratios for Clay/Geomembrane Interfaces Saturated With Leachate I

<table>
<thead>
<tr>
<th>Description</th>
<th>$E_C$ (%)</th>
<th>$E_\Phi$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay/Clay</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Clay/Smooth HDPE</td>
<td>31</td>
<td>50</td>
</tr>
<tr>
<td>Clay/Textured HDPE</td>
<td>26</td>
<td>88</td>
</tr>
<tr>
<td>Clay/PVC</td>
<td>35</td>
<td>54</td>
</tr>
</tbody>
</table>
TABLE 4.7
Physical properties of Aged Geomembranes Immersed in Leachate I

<table>
<thead>
<tr>
<th>Number of days immersion in Leachate I</th>
<th>Mass (gm)</th>
<th>Length (mm)</th>
<th>Width (mm)</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M</td>
<td>L</td>
<td>W</td>
<td>T</td>
</tr>
<tr>
<td></td>
<td>M'</td>
<td>L'</td>
<td>W'</td>
<td>T'</td>
</tr>
<tr>
<td>Smooth HDPE</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7 days</td>
<td>30.0</td>
<td>178</td>
<td>178</td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>30.2</td>
<td>178</td>
<td>178</td>
<td>0.67</td>
</tr>
<tr>
<td>30 days</td>
<td>29.2</td>
<td>177</td>
<td>179</td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>29.6</td>
<td>176</td>
<td>179</td>
<td>0.66</td>
</tr>
<tr>
<td>Textured HDPE</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7 days</td>
<td>32.6</td>
<td>179</td>
<td>179</td>
<td>1.28</td>
</tr>
<tr>
<td></td>
<td>32.6</td>
<td>178</td>
<td>178</td>
<td>1.27</td>
</tr>
<tr>
<td>30 days</td>
<td>39.6</td>
<td>184</td>
<td>177</td>
<td>1.30</td>
</tr>
<tr>
<td></td>
<td>40.1</td>
<td>183</td>
<td>176</td>
<td>1.29</td>
</tr>
<tr>
<td>PVC</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7 days</td>
<td>29.7</td>
<td>205</td>
<td>205</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>28.4</td>
<td>196</td>
<td>203</td>
<td>0.62</td>
</tr>
<tr>
<td>30 days</td>
<td>31.5</td>
<td>205</td>
<td>205</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>29.3</td>
<td>190</td>
<td>204</td>
<td>0.61</td>
</tr>
<tr>
<td>60 days</td>
<td>30.0</td>
<td>205</td>
<td>205</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>27.3</td>
<td>187</td>
<td>203</td>
<td>0.60</td>
</tr>
</tbody>
</table>

M = Mass of the specimen before immersion
M' = Mass of the specimen after immersion
L = Length of the specimen before immersion
L' = Length of the specimen after immersion
W = Width of the specimen before immersion
W' = Width of the specimen after immersion
T = Thickness of the specimen before immersion
T' = Thickness of the specimen after immersion
The aging effect on the PVC geomembrane was the most significant among the three types of geomembranes. It lost 4.38% of its original mass by the 7th day and 6.98% by the 30th day. Its volume reduction was 9.69% on the 7th day and 13.44% on the 30th day. This pronounced degradation of the PVC geomembrane is considered to be caused by extraction of plasticizers contained inside the PVC material as indicated by Randolph et.al (1991). This led to a decrease in mass as well as length, width and thickness.

Since there was drastic change in the physical properties for PVC geomembrane, an additional specimen was removed after 60 days, and its physical properties were evaluated. The mass and length of the PVC specimen further decreased due to prolonged immersion. Figures 4.20 and 4.21 show the effect of leachate I on the length and the mass of the PVC specimen with time. It was observed that the mass and length continued to decrease as the number of days the geomembrane immersed in leachate I increased. Considering the fact that the chemical nature of leachate used in this investigation was relatively mild (comparing Tables 3.2 and 2.1) and in the actual landfill, the geomembrane will be in contact with leachate for a longer period of time, the results observed here are significant.

Similar studies were conducted by Randolph et. al (1991). In their investigation, smooth HDPE and PVC geomembranes were exposed to different chemicals namely kerosene, ethanol, turpentine and nitric acid solution. While, the HDPE geomembrane exhibited very little change in physical properties over time, the PVC specimens underwent significant change. The smooth HDPE specimens slightly increased in mass and PVC specimens decreased in mass with time.

Leachate from the different immersion tanks after a period of 30 days was tested for various characteristics. Leachate samples were taken for suspended solids determination by stirring the contents of the tank well. While collecting the leachate samples, it was observed that a lot of iron had precipitated at the bottom of the tank. After sampling for suspended solids, the leachate in the tank was treated with concentrated nitric acid so that all the matter
Figure 4.20 Effect of Leachate on the Length of the PVC Specimen with Time.
Figure 4.21 Effect of Leachate on the Mass of the PVC Specimen with Time.
in the tank dissolved and the pH was lowered to 2.0. This was done to determine COD, TOC, and metals.

Table 4.8 presents the characteristics of leachate in the different tanks in which different geomembrane materials were immersed. When these characteristics are compared with the initial characteristics presented in Table 3.2, there is reduction in all the characteristics upon immersion of the geomembranes except for pH which increased upon immersion of the geomembranes. The chemical oxygen demand, (COD) reduced from 619 mg/L to 181 mg/L for the leachate in which the PVC geomembranes were immersed. Reduction in the calcium was significantly greater for the leachate in which PVC geomembranes were immersed when compared to the leachates in which smooth HDPE and textured HDPE geomembranes were immersed. Surprisingly, the reduction in the iron concentration was almost the same for both the leachates in which textured HDPE and PVC geomembranes were immersed. The probable reason is that the material composition present in the PVC could have reacted with the surrounding leachate and absorbed metals present in leachate and hence the reduction noticed. Iron could have precipitated or adhered to the textured HDPE material. Also, the characteristics are compared with a composite leachate sample rather than that of the individual leachate samples that were poured in the containers.

4.7 Clay Liner/Aged Geomembrane Interfaces (Unsaturated)

A total of 36 direct shear tests were performed to evaluate the “aging” effect of the geomembranes on the unsaturated interface shear strength for clay liner/smooth HDPE, clay liner/textured HDPE and clay liner/PVC interfaces. Figures 4.22 through 4.24 present the shear stress versus horizontal displacement for clay/aged smooth HDPE, clay/aged textured HDPE and clay/aged PVC interfaces under a normal stress of 17.74 psi under unsaturated conditions. Although peak shear stress was reached for the clay/aged smooth HDPE and clay/aged PVC interfaces at extremely small horizontal displacements, the clay/aged
**TABLE 4.8**

Characteristics of the Leachate I in Which Geomembranes Were Immersed for a Period of 30 Days

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Leachate with Smooth HDPE</th>
<th>Leachate with Textured HDPE</th>
<th>Leachate with PVC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total suspended solids, TSS (mg/L)</td>
<td>14</td>
<td>12</td>
<td>24</td>
</tr>
<tr>
<td>Total dissolved solids, TDS (mg/L)</td>
<td>1926</td>
<td>1650</td>
<td>2160</td>
</tr>
<tr>
<td>Total solids, TS (mg/L)</td>
<td>2102</td>
<td>1667</td>
<td>2192</td>
</tr>
<tr>
<td>Chemical oxygen demand, COD (mg/L)</td>
<td>300</td>
<td>304</td>
<td>181</td>
</tr>
<tr>
<td>Total organic carbon, TOC (mg/L)</td>
<td>57.0</td>
<td>43.0</td>
<td>60.2</td>
</tr>
<tr>
<td>Hydrogen ion concentration, pH</td>
<td>8.8</td>
<td>8.7</td>
<td>9.0</td>
</tr>
<tr>
<td><strong>Metals</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Calcium, Ca (mg/L)</td>
<td>41.3</td>
<td>42.3</td>
<td>8.0</td>
</tr>
<tr>
<td>Magnesium, Mg (mg/L)</td>
<td>930</td>
<td>860</td>
<td>940</td>
</tr>
<tr>
<td>Iron, Fe (mg/L)</td>
<td>3.54</td>
<td>1.71</td>
<td>1.19</td>
</tr>
<tr>
<td>Manganese, Mn (mg/L)</td>
<td>0.15</td>
<td>0.02</td>
<td>0.13</td>
</tr>
<tr>
<td>Copper, Cu (mg/L)</td>
<td>0.15</td>
<td>0.15</td>
<td>0.11</td>
</tr>
<tr>
<td>Chromium, Cr (mg/L)</td>
<td>0.01</td>
<td>0.04</td>
<td>0.00</td>
</tr>
</tbody>
</table>
Figure 4.22 Shear Stress Versus Horizontal Displacement
Plots for Clay-aged smooth HDPE Interface
Under Unsaturated condition
Figure 4.23 Shear Stress Versus Horizontal Displacement Plots for Clay/aged textured HDPE Interface Under Unsaturated Condition
Figure 4.24 Shear Stress Versus Horizontal Displacement Plots for Clay/aged PVC Interface Under Unsaturated Condition
textured HDPE reached a peak almost at the end of the test. Maximum reduction in the frictional angle occurred for clay/aged PVC interface (Figures 4.25 through 4.27). In the case of clay/aged smooth HDPE and clay/aged textured HDPE interface, the frictional angle was slightly higher for the 7 days degraded geomembrane, but decreased for the 30 days degraded when compared to the fresh geomembrane. The increase in the frictional angle in the case of smooth HDPE was negligible.

The frictional angle in the case of clay/aged PVC decreased as the aging increased, and it was reduced by about 43% of the original frictional angle for the 30 day aged PVC geomembrane. These results are comparable with those reported by Randolph et al. (1991). They conducted direct shear tests for undegraded and degraded geomembranes against sand. The degraded geomembranes were tested after a period of 120 days. In their study, there was no change in the frictional angle for both undegraded and degraded smooth HDPE against sand. The frictional angle of 32.9° for sand/undegraded PVC was reduced by about 70% for the sand/degraded PVC interface. In this investigation the direct shear tests were conducted on aged geomembranes after a period of 30 days, hence less reduction would be expected. Moreover, Randolph et al. (1990) conducted tests on sand against geomembranes that were exposed to chemicals instead of leachate. The frictional angles however, were reduced in both the cases.

**4.8 Clay Liner/Aged Geomembrane Interface Saturated with Leachate II**

The last stage in the experimental program was to saturate the clay liner/geomembrane interfaces with leachate and conduct direct shear tests when the expansion rate of the compacted clay became negligible. Since the leachate for this phase of the study was obtained at a different time, some of the experiments of Section 4.5 were repeated by saturating with this Leachate II. Table 4.9 presents the results of the same. It was observed that the maximum shear stress reached was the same as obtained by saturating
Figure 4.25 Maximum Shear Stress Vs. Normal Stress for Clay/aged smooth HDPE Interface Under Unsaturated Condition

- Fresh geomembrane
  \( c_a = 1.81 \text{ psi} \)
  \( \delta = 17.14 \text{ degrees} \)

- 7 days degraded
  \( c_a = 1.81 \text{ psi} \)
  \( \delta = 17.82 \text{ psi} \)

- 30 days degraded
  \( c_a = 2.31 \text{ psi} \)
  \( \delta = 13.81 \text{ degrees} \)
Figure 4.26 Maximum Shear Stress vs. Normal Stress for Clay/aged textured HDPE Interface Under Unsaturated Condition
Figure 4.27 Maximum Shear Stress vs. Normal Stress for Clay-aged PVC Interface Under Unsaturated Condition
TABLE 4.9

Reproducibility of results on Clay/Geomembrane Interfaces with Leachate I and Leachate II as Saturating Media

<table>
<thead>
<tr>
<th>Type of Interface</th>
<th>Leachate I</th>
<th>Leachate II</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Normal Stress (psi)</td>
<td>Normal Stress (psi)</td>
</tr>
<tr>
<td>13.63</td>
<td>35.48</td>
<td>13.63</td>
</tr>
<tr>
<td>Clay/Clay</td>
<td>14.100</td>
<td>22.583</td>
</tr>
<tr>
<td>Clay/Smooth HDPE</td>
<td>5.458</td>
<td>10.439</td>
</tr>
<tr>
<td>Clay/PVC</td>
<td>5.458</td>
<td>11.235</td>
</tr>
</tbody>
</table>
with Leachate I. Hence it was appropriate to compare the results to the previous case.

Figures 4.28 to 4.30 show shear stress vs. horizontal displacement for the three interfaces for fresh, 7 days aged and 30 days aged geomembranes saturated with leachate. Unlike the unsaturated case (Section 4.7) the maximum shear stress was reached at a higher horizontal displacement for all the three interfaces.

Figures 4.31 through 4.36 shows the maximum shear stress vs. normal stress for all interfaces for 7 days aged and 30 days aged geomembranes. There was a reduction in cohesion and frictional angle for clay/7 days aged smooth HDPE and clay/7 days aged textured HDPE saturated with leachate. For 30 days degraded geomembranes, in the case of clay/aged smooth HDPE and clay/aged textured HDPE, even though the shear stress under each normal stress level was lower corresponding to the shear stress under the unsaturated degraded case, the frictional angle increased slightly. Clay/aged PVC interface frictional angle, which was the lowest among all the interfaces under unsaturated condition, decreased further upon saturation with leachate II and was about 9.99 degrees and 8.18 degrees after 7 and 30 days degradation.

Tables 4.10 and 4.11 summarize efficiency ratio values for the interfaces saturated with Leachate II for 7 days aged and 30 days aged geomembranes. The efficiency on cohesion decreased more for clay/aged smooth HDPE and clay/aged textured HDPE when compared to fresh smooth HDPE and textured HDPE geomembrane saturated with leachate (comparing to Table 4.6) and was almost the same for clay/aged PVC when compared to fresh PVC saturated with leachate.

Tables 4.12 through 4.14 summarize the results of the interfaces tested under different conditions investigated in this study. In the case of clay/smooth HDPE and clay/textured HDPE interfaces, leachate has little effect on the frictional angle of the aged geomembranes and saturation with water forms the weakest plane (Figure 4.37 and 4.38),
Figure 4.28 Shear Stress Versus Horizontal Displacement
Plots for Clay/aged smooth HDPE Interface
Saturated with Leachate

Normal Stress = 9.08 psi

- Fresh geomembrane
- 7 days degraded
- 30 days degraded
Figure 4.29 Shear Stress Versus Horizontal Displacement Plots for Clay-AGED textured HDPE Interface Saturated with Leachate
Figure 4.30 Shear Stress Versus Horizontal Displacement Plots for Clay-aged PVC Interface Saturated with Leachate
Figure 4.31 Maximum Shear Stress vs. Normal Stress for Clay/7 days aged smooth HDPE Interface Saturated with Leachate
Figure 4.32 Maximum Shear Stress vs. Normal Stress for Clay/30 days aged smooth HDPE Interface Saturated with Leachate

- Unsaturated
  $c_a = 2.31$ psi
  $\phi = 13.81$ degrees

- Saturated with leachate
  $c_a = 0.26$ psi
  $\phi = 15.19$ degrees
Figure 4.33 Maximum Shear Stress vs. Normal Stress for Clay/7 days aged textured HDPE Interface Saturated with Leachate

- 7 days degraded (unsaturated)  
  \( \sigma_a = 3.97 \text{ psi} \)  
  \( \phi = 25.75 \text{ degrees} \)

- 7 days degraded (saturated with leachate)  
  \( \sigma_a = 1.13 \text{ psi} \)  
  \( \phi = 21.58 \text{ degrees} \)
Figure 4.34 Maximum Shear Stress vs. Normal Stress for Clay/30 days aged textured HDPE Interface Saturated with Leachate

unsaturated
\( c_a = 3.63 \text{ psi} \)
\( \delta = 20.66 \text{ degrees} \)

saturated with leachate
\( c_a = 0.62 \text{ psi} \)
\( \delta = 29.77 \text{ degrees} \)
Figure 4.35 Maximum Shear Stress vs. Normal Stress for Clay/7 days aged PVC Interface Saturated with Leachate

- unsaturated
  $c_a = 2.64$ psi
  $\delta = 12.26$ degrees
- saturated with leachate
  $c_a = 2.04$ psi
  $\delta = 9.99$ degrees
Figure 4.36 Maximum Shear Stress vs. Normal Stress for Clay/30 days aged PVC Interface Saturated with Leachate

- unsaturated
  \( \text{ca} = 2.76 \text{ psi} \)
  \( \delta = 9.99 \text{ degrees} \)

- saturated with leachate
  \( \text{ca} = 1.95 \text{ psi} \)
  \( \delta = 8.18 \text{ degrees} \)
**TABLE 4.10**

Efficiency Ratios for Clay/7 Days Aged Geomembrane Interfaces Saturated With Leachate II

<table>
<thead>
<tr>
<th>Description</th>
<th>$E_c$ (%)</th>
<th>$E_\phi$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay/Clay</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Clay/Smooth HDPE</td>
<td>11</td>
<td>56</td>
</tr>
<tr>
<td>Clay/Textured HDPE</td>
<td>17</td>
<td>83</td>
</tr>
<tr>
<td>Clay/PVC</td>
<td>30</td>
<td>37</td>
</tr>
</tbody>
</table>
### TABLE 4.11

Efficiency Ratios for Clay/30 Days Aged Geomembrane Interfaces

Saturated With Leachate II

<table>
<thead>
<tr>
<th>Description</th>
<th>$E_c$ (%)</th>
<th>$E_{\phi}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay/Clay</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Clay/Smooth HDPE</td>
<td>4</td>
<td>57</td>
</tr>
<tr>
<td>Clay/Textured HDPE</td>
<td>9</td>
<td>80</td>
</tr>
<tr>
<td>Clay/PVC</td>
<td>29</td>
<td>30</td>
</tr>
</tbody>
</table>
TABLE 4.12

Frictional Characteristics for Clay/smooth HDPE Interface Under Different Conditions

<table>
<thead>
<tr>
<th>Description</th>
<th>C_a (psi)</th>
<th>δ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unsaturated Interface</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fresh geomembrane</td>
<td>1.81</td>
<td>17.14</td>
</tr>
<tr>
<td>7 days aged</td>
<td>1.81</td>
<td>17.82</td>
</tr>
<tr>
<td>30 days aged</td>
<td>2.31</td>
<td>13.81</td>
</tr>
<tr>
<td>Interface Saturated with water</td>
<td></td>
<td></td>
</tr>
<tr>
<td>After 24 hours</td>
<td>0.47</td>
<td>12.74</td>
</tr>
<tr>
<td>Interface Saturated with Leachate</td>
<td></td>
<td></td>
</tr>
<tr>
<td>After 24 hours</td>
<td>2.07</td>
<td>13.37</td>
</tr>
<tr>
<td>7 days aged</td>
<td>0.74</td>
<td>14.99</td>
</tr>
<tr>
<td>30 days aged</td>
<td>0.26</td>
<td>15.19</td>
</tr>
</tbody>
</table>
**TABLE 4.13**

Frictional Characteristics for Clay/textured HDPE Interface Under Different Conditions

<table>
<thead>
<tr>
<th>Description</th>
<th>$c_a$ (psi)</th>
<th>$\delta$ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unsaturated Interface</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fresh geomembrane</td>
<td>2.82</td>
<td>24.21</td>
</tr>
<tr>
<td>7 days aged</td>
<td>3.97</td>
<td>25.75</td>
</tr>
<tr>
<td>30 days aged</td>
<td>3.63</td>
<td>20.66</td>
</tr>
<tr>
<td>Interface Saturated with water</td>
<td></td>
<td></td>
</tr>
<tr>
<td>After 24 hours</td>
<td>1.83</td>
<td>19.74</td>
</tr>
<tr>
<td>Interface Saturated with Leachate</td>
<td></td>
<td></td>
</tr>
<tr>
<td>After 24 hours</td>
<td>1.74</td>
<td>22.63</td>
</tr>
<tr>
<td>7 days aged</td>
<td>1.13</td>
<td>21.58</td>
</tr>
<tr>
<td>30 days aged</td>
<td>0.62</td>
<td>20.77</td>
</tr>
</tbody>
</table>
### Table 4.14

Frictional Characteristics for Clay/PVC Interface Under Different Conditions

<table>
<thead>
<tr>
<th>Description</th>
<th>$C_a$ (psi)</th>
<th>$\delta$ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unsaturated Interface</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fresh geomembrane</td>
<td>2.07</td>
<td>17.54</td>
</tr>
<tr>
<td>7 days aged</td>
<td>2.64</td>
<td>12.26</td>
</tr>
<tr>
<td>30 days aged</td>
<td>2.76</td>
<td>9.99</td>
</tr>
<tr>
<td>Interface Saturated with water</td>
<td></td>
<td></td>
</tr>
<tr>
<td>After 24 hours</td>
<td>0.73</td>
<td>13.66</td>
</tr>
<tr>
<td>Interface Saturated with Leachate</td>
<td></td>
<td></td>
</tr>
<tr>
<td>After 24 hours</td>
<td>2.38</td>
<td>14.45</td>
</tr>
<tr>
<td>7 days aged</td>
<td>2.04</td>
<td>9.99</td>
</tr>
<tr>
<td>30 days aged</td>
<td>1.95</td>
<td>8.18</td>
</tr>
</tbody>
</table>
Fresh geomembrane (unsaturated)  
\( \text{ca} = 1.81 \text{ psi} \)  
\( \delta = 17.14 \text{ degrees} \)

Fresh geomembrane (saturated with leachate)  
\( \text{ca} = 2.07 \text{ psi} \)  
\( \delta = 13.37 \text{ degrees} \)

7 days degraded (saturated with leachate)  
\( \text{ca} = 0.74 \text{ psi} \)  
\( \delta = 14.99 \text{ degrees} \)

30 days degraded (saturated with leachate)  
\( \text{ca} = 0.26 \text{ psi} \)  
\( \delta = 15.19 \text{ degrees} \)

Fresh geomembrane (saturated with water)  
\( \text{ca} = 0.47 \text{ psi} \)  
\( \delta = 12.74 \text{ degrees} \)

Figure 4.37 Maximum Shear Stress vs. Normal Stress for Clay/Fresh and Aged smooth HDPE Interface Under Different Conditions
Figure 4.38 Maximum Shear Stress vs. Normal Stress for Clay/Fresh and Aged textured HDPE Interface Under Different Conditions
whereas in the case of clay/PVC interface, saturation of leachate for the 30 days degraded geomembrane forms the weakest plane (Figure 4.39). These aspects have to be given due importance in design of landfills depending upon the type of geomembrane material used.
Figure 4.39 Maximum Shear Stress vs. Normal Stress for Clay/Fresh and aged PVC Interface Under Different Conditions
5.1 Conclusions

The following conclusions can be drawn from the current investigation:

(1) Shearing rate has significant effect on frictional angle. As the shearing rate decreases, the frictional angle increases. However, there is a maximum friction the shear plane can attain. A shearing rate of 0.425 mm/min. is recommended for the direct shear tests on clay alone and clay/geomembrane interfaces. These results are in good agreement with the shearing rates utilized by other researchers in the past.

(2) The frictional angle for the clay/geomembrane interface is always less than the internal frictional angle of clay alone regardless of the type of geomembrane. Even the clay/textured HDPE interface mobilizes only up to 88% of the internal friction angle of the clay.

(3) Maximum shear stress is reached in the case of clay alone (when compared to clay/geomembrane interfaces) either under the unsaturated condition or saturated condition with water or leachate under any particular normal stress.

(4) Among the clay/geomembrane interfaces, shear stress is maximum for clay/textured HDPE and minimum for clay/smooth HDPE interface for both the unsaturated condition and saturated condition with water under any particular normal stress. For the saturated condition with leachate, while the clay/textured HDPE interface attains maximum shear stress, the clay/PVC interface is minimum.
(5) The frictional angle was reduced significantly upon saturation with water for clay as well as for the clay/geomembrane interfaces. Maximum reduction occurred for the clay/smooth HDPE interface and minimum for the clay/textured HDPE. The frictional angle obtained for the clay/smooth HDPE saturated with water was in agreement with results reported by Carey et. al (1991) and Seed et. al (1991). Water appears to act as an interface lubricant.

(6) The clay/PVC interface exhibited the poorest frictional characteristics among the three interfaces studied upon saturation with leachate. However, the calculated frictional angle was between the unsaturated and water saturation conditions for all the cases.

(7) The aging effect observed by immersion of geomembranes in leachate was the most significant for the PVC geomembrane. Smooth HDPE and textured HDPE geomembranes probably absorbed some moisture and their mass increased slightly, and the mass of the PVC geomembrane decreased through possible leaching out its plasticizer, as reported by Randolph et.al (1991).

(8) It was observed that the mass and length of the PVC specimen continuously decreased as the number of days the geomembrane was immersed in leachate I. Considering the fact that the chemical nature of leachate used in this investigation was relatively mild (comparing Tables 3.2 and 2.2) and in the actual landfill, the geomembrane will be in contact with leachate for a longer period of time, the results observed here are significant.

(9) There was a reduction in the TSS, TDS, TS, COD, TOC, and metals of leachate I in all the three tanks in which geomembranes were immersed when compared with the initial values except for the pH which increased.
(10) The frictional angle at the unsaturated condition in the case of the clay/aged PVC interface decreased as the aging of the geomembrane increased, and it was reduced by about 43% of the original frictional angle for 30 day aged PVC geomembrane. One possible reason for this is that upon immersion of geomembranes in leachate, there was a chemical reaction between the chemical composition of the PVC and the leachate. The other reason may be that the material of the leachate would have filled the surface holes of the PVC geomembrane, and thus made the surface of the geomembrane smoother and hence resulted in a lower frictional angle.

(11) The clay/aged PVC interface frictional angle, which was the lowest among all the interfaces at the unsaturated condition, decreased further upon saturation with leachate II and was 10 degrees and 8.2 degrees after 7 and 30 days degradation.

(12) U.S. EPA Method 9090 requires the geomembranes to be immersed in leachate for a period of 120 days and then tested for various properties. Considering the fact that the leachate used in this investigation was relatively weak, the results obtained on the 30th day may be quite conservative.

5.2 Recommendations

The following summarizes recommendations for further study:

(1) Leachate from different landfills should be utilized in future studies so that a database can be established to draw general conclusions on effect of landfill leachate on the clay/geomembrane interfaces.

(2) Aging tests on the geomembrane should be continued at least till 120 days, and the physical and chemical properties should be tested at least after every 30 day interval. Direct shear tests should then be conducted on the aged geomembranes.
(3) Since temperature tends to be elevated within the landfill, direct shear tests should be conducted on clay/geomembrane interfaces at various temperatures higher than normal room temperature.

(4) Geomembranes should be immersed in water and taken out at specific intervals and tested for their frictional characteristics to see the effects of water immersion contrasted to leachate immersion.

(5) Large scale direct shear tests should be conducted under the conditions simulated in this investigation to verify the small scale direct shear test results.
REFERENCES


ABSTRACT

A comprehensive study of the effect of leachate on landfill composite liners (recompacted clay liner overlain by geomembrane) is investigated in the present research. In the past several studies were conducted on clay/geomembrane interfaces under the unsaturated condition. However, only a few studies have been published on the clay/geomembrane interfaces saturated with water, and no study has been reported with leachate as the saturating media.

Three types of geomembranes were utilized in this investigation. They were smooth HDPE, textured HDPE, and PVC. Modified direct shear tests were conducted on geomembrane interfaces against clay under unsaturated conditions and saturated conditions with water and leachate as the saturating media, and the results were compared. Aging of geomembranes immersed in leachate was determined adopting the U.S. EPA Method 9090, and changes in physical properties of geomembranes were evaluated. The characteristics of leachate in which geomembranes were immersed for a period of 30 days were evaluated. Frictional characteristics were determined by conducting modified direct shear tests on geomembranes aged at 7 days and 30 days, against clay under unsaturated conditions and leachate saturated conditions. A comparison study was made, also.