SOURCE CONTAINMENT AT THE MASSACHUSETTS MILITARY RESERVATION MAIN BASE LANDFILL: DESIGN OF A HAZARDOUS WASTE LANDFILL COVER SYSTEM

by

KARL G. ELIAS

B.S.C.E. University of Massachusetts at Amherst
(1989)

Submitted in partial fulfillment of the requirements for the degree of

MASTER OF ENGINEERING IN
CIVIL AND ENVIRONMENTAL ENGINEERING

at the

MASSACHUSETTS INSTITUTE OF TECHNOLOGY
June, 1996

Signature of Author
Department of Civil and Environmental Engineering
May 10, 1996

Certified by
Patricia Culligan Hensley
Assistant Professor
Department of Civil and Environmental Engineering
Thesis Supervisor

Accepted by
Joseph Sussman
Chairman, Departmental Graduate Committee
Department of Civil and Environmental Engineering

Source Containment at the Massachusetts Military Reservation Main Base Landfill: Design of a Hazardous Waste Landfill Cover System

by

Karl G. Elias

Submitted to the Department of Civil and Environmental Engineering on May 10, 1996 in partial fulfillment of the requirements for the degree of Master of Engineering in Civil and Environmental Engineering

ABSTRACT

The Massachusetts Military Reservation on Cape Cod, Massachusetts was placed on the Superfund National Priorities List in 1989. One of the areas of concern at the base is the Main Base Landfill. The Main Base Landfill is an uncontrolled hazardous waste landfill that is serving as a source of contamination for the underlying sole-source groundwater aquifer. As part of Institute thesis requirements, a group of graduate students from the Department of Civil and Environmental Engineering at the Massachusetts Institute of Technology undertook the task of assessing the potential environmental impacts of the contaminant plume originating from this landfill, and proposing possible schemes for its remediation. This report is a detailed description of one facet of that group project. This contribution to the group project addresses the issue of source containment through the design of a landfill final cover system for a portion of the landfill that has yet to be capped.

The landfill final closure requirements of the Resource Conservation and Recovery Act (RCRA) and Massachusetts Solid Waste Management Regulations are examined and adapted to the site specific conditions of the Main Base Landfill. Material and design options for the components of the cover system are examined and choices are made according to performance, availability, and relative cost, as applicable to site-specific conditions.

The proposed cover system design provides a nearly impermeable barrier while also controlling lateral drainage flow, surface runoff, and decomposition gases with a stable, durable design that will maintain its integrity for decades.

Thesis Supervisor: Professor Patricia Culligan Hensley
Title: Assistant Professor of Civil and Environmental Engineering

ACKNOWLEDGMENTS

I would like to first thank my loving wife for her support and encouragement throughout the ordeal called graduate school. I also wish to express my sincere gratitude to Professor Patricia Culligan Henley for her extremely generous giving of time, guidance, and quality advice. Her refreshingly realistic outlook helped make the painful process of thesis production much more tolerable. I must also acknowledge the priceless advice and mentoring provided by Professor David H. Marks. I would also like to thank Professor Lynn W. Gelhar who spent a great deal of his valuable time advising, administering, and generally orchestrating the Master of Engineering thesis process. Finally, I wish to recognize the team of classmates and close friends with whom I worked to complete the group project portion of our thesis requirements: Dan Alden, Kishan Amarasekera, Michael Collins, Karl G. Elias, Jim Hines, Benjamin R. Jordan, and Robert F. Lee.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1.0 ***INTRODUCTION***

The Massachusetts Military Reservation on Cape Cod, Massachusetts was placed on the Superfund National Priorities List in 1989. One of the areas of concern at the base is the Main Base Landfill. The Main Base Landfill is an uncontrolled hazardous waste landfill and is serving as a source of contamination for the underlying sole-source groundwater aquifer. As part of Institute thesis requirements, a group of graduate students from the Department of Civil and Environmental Engineering at the Massachusetts Institute of Technology undertook the task of assessing the potential environmental impacts of the contaminant plume originating from this landfill, and proposing possible schemes for its remediation. This report is a detailed description of one facet of that group project. This contribution to the group project addresses the issue of source containment through the design of a landfill final cover system for a portion of the landfill that has yet to be capped.

An extensive amount of data on contamination at the MMR has been collected and is maintained by the MMR Installation Restoration Program (IRP). The IRP acts as principal agent for the US government on behalf of the MMR. Numerous engineering reports, including data observations and professional opinions, have been produced for the IRP. These reports are available for public review and served as a principal source of data for the project team.

1.1 Group Project: Objectives and Scope

The group project report examines and offers opinions on the potential impacts of the MMR LF-1 on human health and the environment, and proposes potential methods to mitigate these effects. The scope of the project includes: study of source containment, site characterization, groundwater modeling, risk assessment, and management of public interaction. In addition, bioremediation technology is explored as a means for groundwater remediation. The underlying objectives of the report are:

- Characterization of the site through evaluation of subsurface hydraulic conductivity
- Characterization of the landfill plume chemistry, dimensions, and movement through use of existing data and groundwater modeling
- Protection of the Cape Cod groundwater aquifer from further contamination by containing the source with a landfill final cover system
- Evaluation of the potential cancer risk posed to people located near the landfill plume by materials identified in the groundwater, as well as risks associated with ingestion of potentially contaminated shellfish
- Evaluation of ecological risk through study of a limited number of indicator species affected by plume contaminants
- Design of a bioremediation scheme to remediate contaminated groundwater
- Characterization of the management of public interaction surrounding base cleanup activities

The results of the group project are provided in Appendix B.

1.2 Individual Project Objectives

As shown in Figure 1-1, the MMR landfill is composed of six cells termed the 1941, 1947, 1951, 1970, post-1970 and kettle hole cells. As part of remediation operations at MMR, the 1970, post-1970, and the kettle hole cells have recently been secured with a final cover system. Remedial investigation with respect to the necessity of a final closure system for the remaining cells (1941, 1947, and 1951) is ongoing. These cells have been collectively termed the Northwest Operable Unit (NOU).

The primary objective of this contribution to the LF-1 group project is to protect the underlying Cape Cod groundwater aquifer from further contamination by containing the source of contamination. Containment of the source will be accomplished through the design of a landfill final cover system. The specific objectives of such a cover system are:

- To prevent / minimize leachate production by preventing / minimizing percolation into the waste.
- Given the site-specific conditions, to also satisfy the following criteria:
    * isolate the waste from humans, vectors and other animals, and other components of the surrounding ecosystem
    * control gases generated within the waste fill
    * be resistant to erosion by wind and water
    * be resistant to static and seismic slope failures

* be durable, maintaining its design performance level for 30 years (regulatory) or the life of the waste fill (prudent)
* control surface water runoff and lateral drainage flow in a manner which does not promote erosion and does not adversely impact the surrounding environment

1.3 **Individual Project Scope**

The landfill final closure requirements of the Resource Conservation and Recovery Act (RCRA) and Massachusetts Solid Waste Management Regulations are examined and adapted to the site specific conditions of the Main Base Landfill (LF-1) at the Massachusetts Military Reservation. Material and design options for the components of the cover system are examined and choices made according to performance, availability, and relative cost, as applicable to site-specific conditions. The design proposal is limited in areal extent to the 1951 Cell portion of the Northwest Operable Unit (NOU), however, the methodologies used, and recommendations presented, are directly applicable to the NOU in its entirety. Topics which are not be specifically addressed are: (1) the development of construction specifications, and (2) an overall cost-benefit analysis for the project.

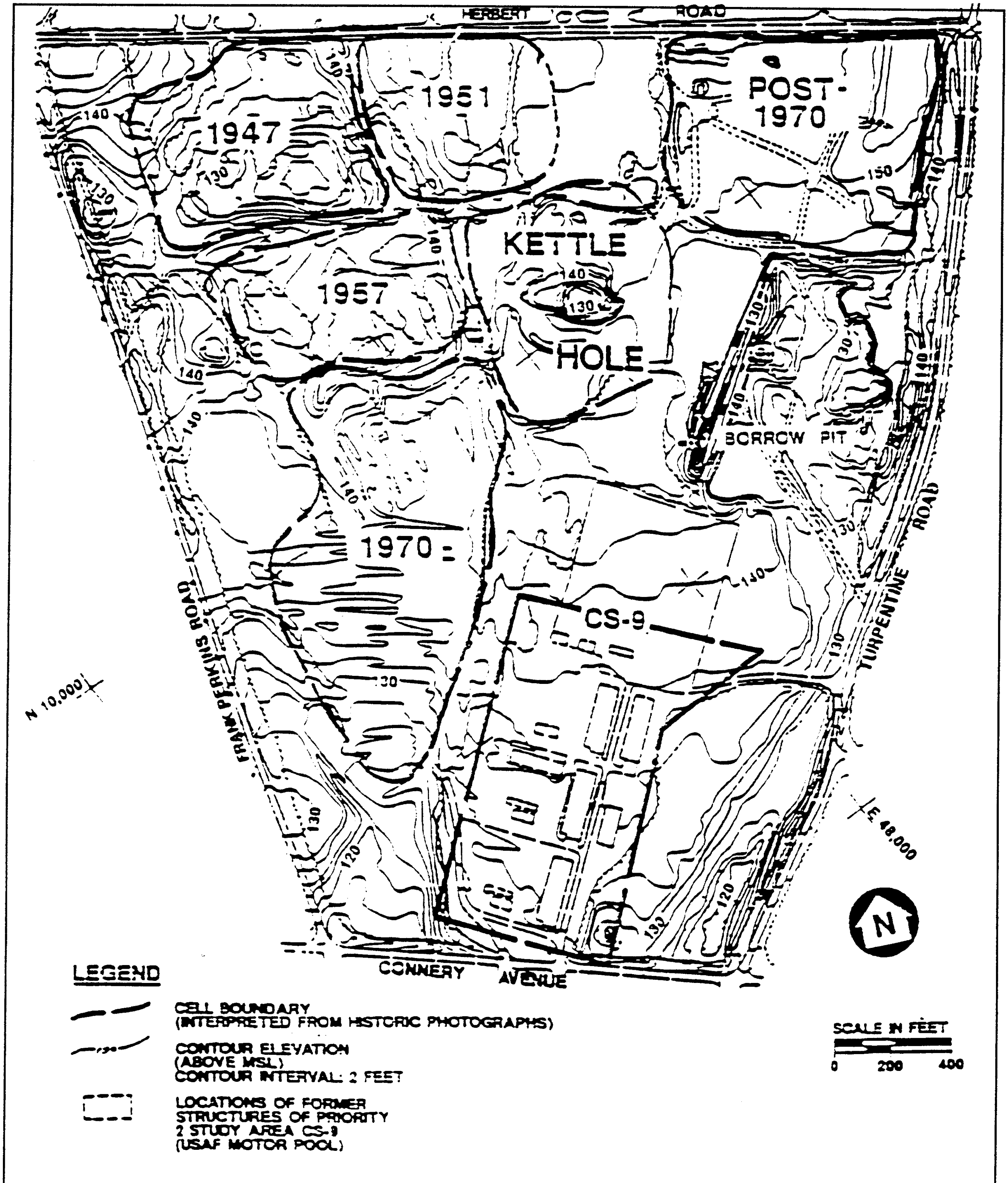


Figure 1-1: Photogrammetric Map Showing Disposal Cells
(ABB, June 1992)

2.0 *BACKGROUND AND SITE DESCRIPTION*

2.1 Geography and Land Use

Massachusetts Military Reservation (MMR) is located in the northwestern portion of Cape Cod, Massachusetts, covering an area of approximately 30 square miles (ABB, June 1992). The site location is illustrated in Figure 2-1. Towns adjacent to the MMR include Bourne, Falmouth, Mashpee, and Sandwich. These towns house both year-round and seasonal residents. Land uses in these areas include residential, recreational, and agricultural uses. The area also supports a large tourist population in the summer season.

2.2 Geology

The geology of the western Cape Cod region near the landfill site consists of glacial drift sediments, ranging in size from fine sand and clay to boulders. These sediments were deposited during the Pleistocene Epoch (Oldale, 1984). Deposits are the result of a sequence of periods of glacial deposition, erosion, and redeposition, resulting in a heterogeneous, anisotropic layering of sediments. A generalized surficial geologic map of the area is provided as Figure 2-2.

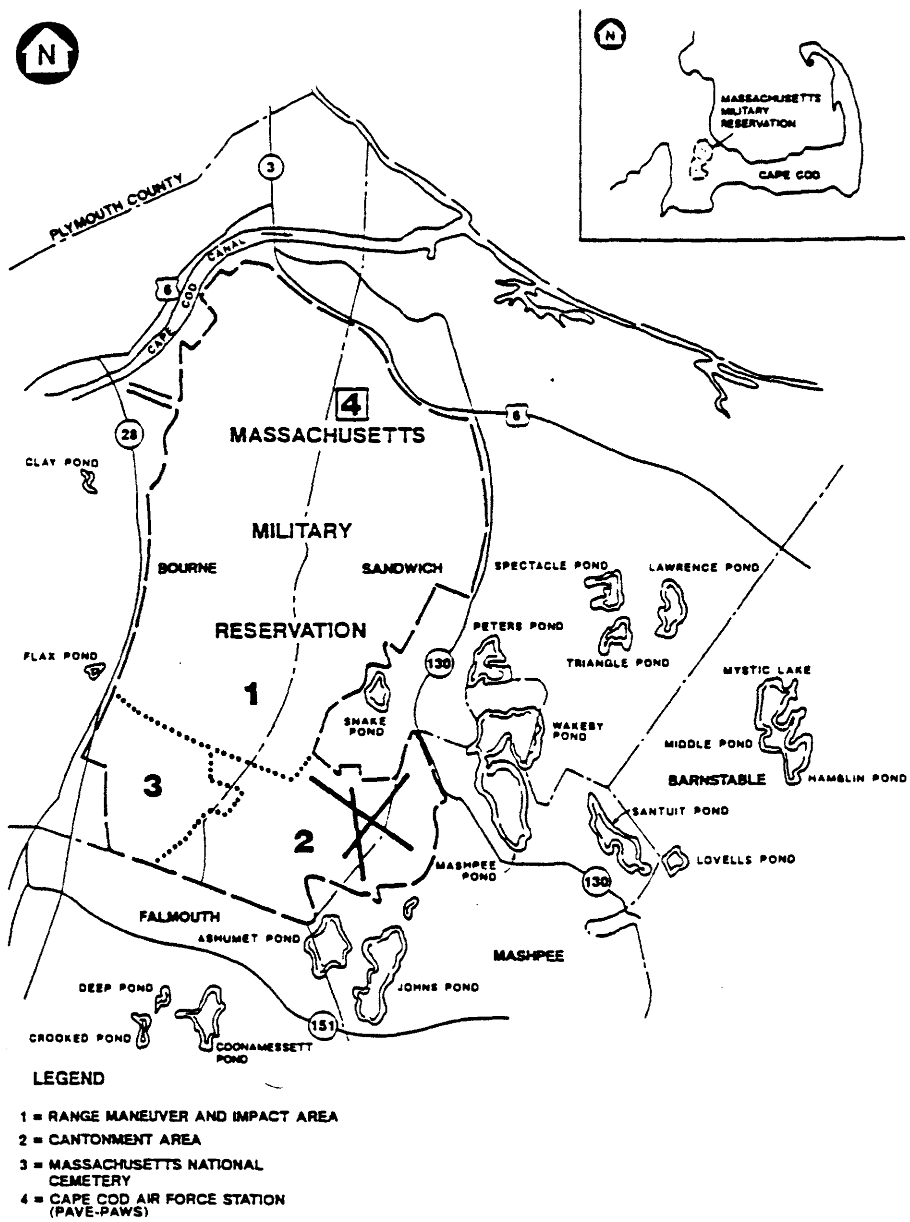


Figure 2-1: Site Location Map (ABB, March 1992)

2.3 **Climate and Hydrology**

The Cape Cod climate is categorized as a humid continental climate (Weston, 1985). Average wind speeds range from 9 mph from July to September to 12 mph from October through March (Weston, 1985). Precipitation is fairly evenly distributed, with an average of approximately 4 inches per month (Weston, 1985). Average annual precipitation is approximately 47 inches (Weston, 1985).

A single groundwater flow system underlies western Cape Cod (CDM Federal, 1995). The aquifer system is unconfined and is recharged by infiltration from precipitation (CDM Federal, 1995). There is very little surface runoff, and approximately 40% of the precipitation infiltrates the ground and enters the groundwater system (CDM Federal, 1995). The groundwater system of the western Cape is characterized by flow which is radially-outward from a mound centered near the western boundary of the MMR. Water for residential use is supplied by either private wells or public water systems (ABB, 1995).

2.4 **Base History**

Military use of the MMR began in the early 1900's, and may be generally categorized as mechanized forces training and military aircraft operations. Since commencement of military operations, the base has seen use by several branches of the armed services, including the United States Air Force, Army, Navy, Coast

Guard, and the Massachusetts Air National Guard. Operations by the Air National Guard and Coast Guard are ongoing.

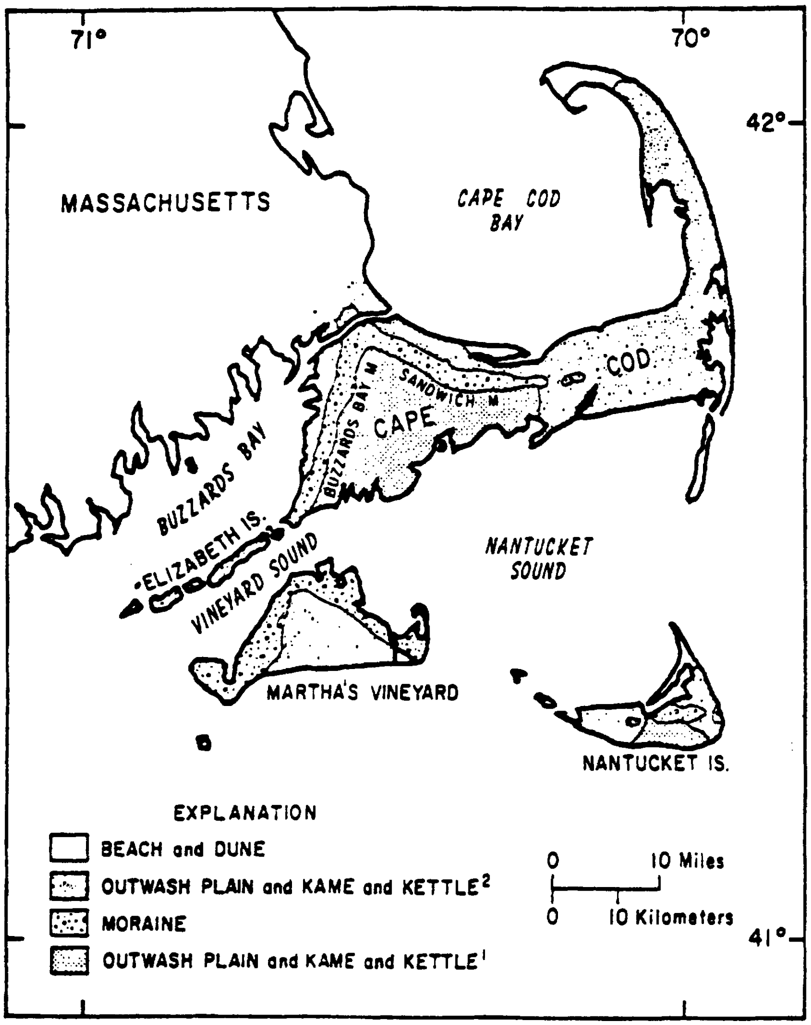


Figure 2-2: Generalized Surficial Geologic Map of MMR Area (USGS, 1995)

2.5 Main Base Landfill (LF-1)

The area of present study is the Main Base Landfill site, termed LF-1 by the MMR Installation Restoration Program (IRP). The landfill is about 10,000 feet from the western and southern MMR boundaries and occupies approximately 100 acres (ABB, 1995). The landfill has operated since the early 1940's as the primary solid waste disposal facility at MMR (ABB, 1995). Unregulated disposal of waste at LF-1 continued until 1980, at which time the Air National Guard began regulating disposal (Metcalf & Eddy, 1983)

Waste disposal operations at LF-1 took place in five distinct disposal cells and a natural kettle hole, respectively (ABB, 1995). These are termed the 1947, 1951, 1957, 1970, post-1970, and kettle hole cells (ABB, 1995). The date designations indicate the year in which disposal operations ceased at that particular cell. The landfill layout is illustrated in Figure 1-1.

As part of remediation operations at MMR, several of the cells have recently been secured with a final cover system. These cells include the 1970 cell, the post-1970 cell, and the kettle hole. The remaining cells (1947, 1951, and 1957) have collectively been termed the Northwest Operable Unit (NOU). Remedial investigations with respect to the necessity of a final closure system for these cells is ongoing (ABB, 1995).

Accurate documentation of the wastes deposited at LF-1 does not exist. The wastes may include some or all of the following: general refuse, fuel tank sludge, herbicides, solvents, transformer oils, fire extinguisher fluids, blank small arms ammunition, paints, paint thinners, batteries, DDT powder, hospital wastes, municipal sewage sludge, coal ash, and possibly live ordnance (ABB, June 1992). Wastes were deposited in linear trenches, and covered with approximately 2 feet of native soil. Waste depth is uncertain, with the maximum depth estimated to be approximately 20 feet below the ground surface (excluding the kettle hole, which is deeper). Waste disposal at the landfill ceased in 1990. As a result of these uncontrolled disposal practices, a plume of dissolved chlorinated volatile organic compounds, primarily tetrachloroethylene (PCE) and dichloroethylene (DCE), has developed downgradient of the landfill.

2.6 1951 Cell

The 1951 cell is one of the three cells which comprise the Northwest Operable Unit. The cell covers approximately 9 acres (ABB, April 1992). The landfill surface in the 1951 cell area is relatively flat, with vegetative cover varying from heavily wooded to bare (ABB, June 1992). The general topography and surface cover are shown in the aerial photo of Figure 2-3 as is the trench-type layout of the cell. Test-pits in the 1951 cell revealed a cell cross-section of approximately 2 feet of native soil overlying approximately 8 feet of burnfill and miscellaneous debris underlain by clean sand (Weston, 1985).

2.7 Present Activity

The MMR is one of 1,236 sites that have been placed on the National Priority List (NPL) by the US Environmental Protection Agency (EPA). NPL sites are those which the EPA has given particularly high human health and environmental risk rankings. Due to the health and environmental risks which have been attributed to activities at the MMR, federal activity is underway to quantify further, and reduce to the extent required, the risk posed to human health and the environment by contamination at this site.

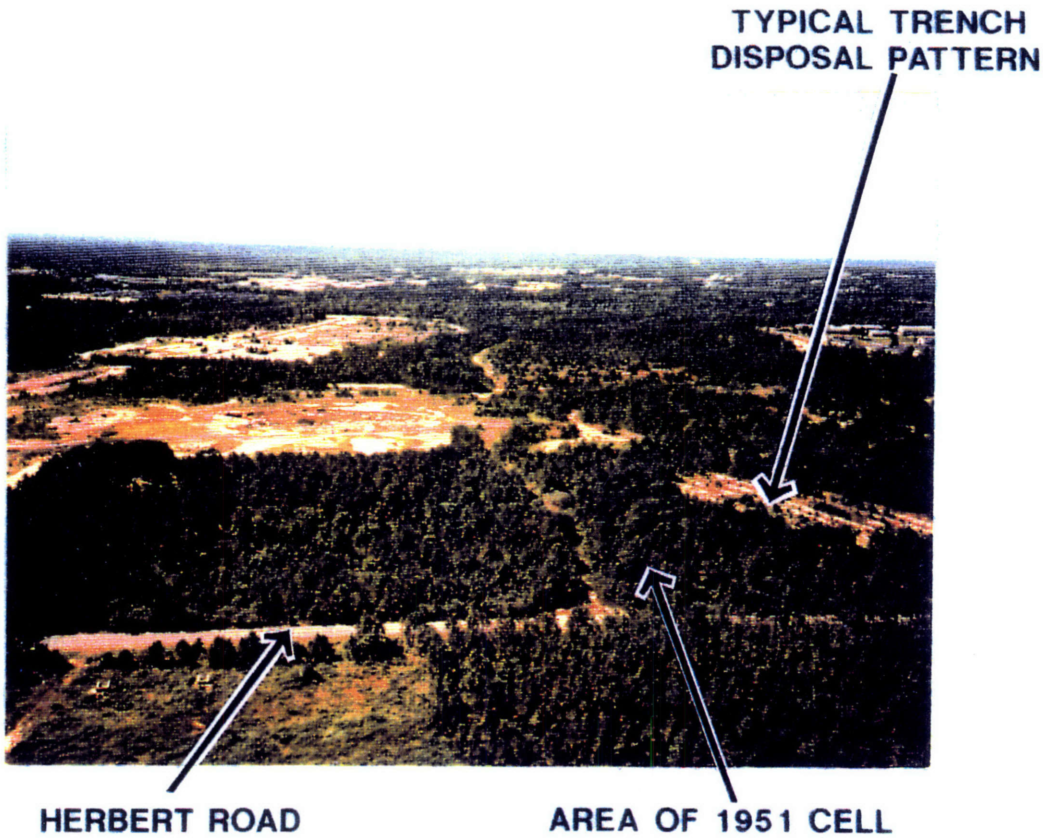


Figure 2-3: Aerial View of 1951 Cell Area (ABB, 1991)

3.0 *REGULATORY REVIEW*

3.1 General Design and Construction Considerations

The following items are of fundamental concern in the design and construction of a landfill closure system (Massachusetts DEP, 1993):

- Prevention of stormwater infiltration into waste fill
- Settlement and differential settlement of waste
- Final cap contour
- Stormwater run-on and run-off controls, particularly erosion control
- Suitable vegetative layer, again important for erosion control
- Prevention of damage to the hydraulic barrier layer from: freeze-thaw cycles, root penetration, and animal penetration
- Control of landfill gases

3.2 Applicable Regulations

The regulations of concern in the design of a final closure system (cap) for a solid waste landfill include, primarily, the Resource Conservation and Recovery Act (40 CFR 264) and Massachusetts Solid Waste Management Regulations (310 CMR 19.000).

3.3 State Regulatory Requirements

Massachusetts regulations specify the following as minimum design components for a landfill final closure system (MA DEP, 1993):

- Subgrade layer
- Venting layer with minimum hydraulic conductivity of $1X10^{-3}$ cm/sec
- Low conductivity layer with minimum thickness of 18 inches (45 cm) and maximum hydraulic conductivity of $1x10^{-7}$ cm/sec, or an approved flexible membrane liner
- Drainage layer with minimum thickness of 6 inches (15 cm) and minimum hydraulic conductivity of $1x10^{-3}$ cm/sec, or a synthetic drainage net
- Combined vegetative support / protection layer of minimum thickness 18 inches (45 cm), with at least 12 inches (30 cm) of soil capable of supporting vegetation.

3.3.1 **Specific Design Considerations** (MA DEP, 1993)

Subgrade

The purpose of the subgrade layer is to act as a foundation for the overlying layers of the cap. This layer is also used as a contouring layer to create the appropriate final slope of the cover system. In designing the subgrade layer, there are several factors which need be considered. The material used should be sufficiently clean of objects that could damage (e.g., puncture) the low permeability layer. The layer must be of 12 inch (30 cm) minimum thickness, 6 inches (15 cm) of which may comprise the soil gas venting layer. The subgrade layer must be sufficiently thick to ensure the long-term integrity of the cap, and create the required slope of the cover system final grade while accounting for settlement of the underlying waste.

Gas Venting Layer

The gas venting layer is a permeable layer containing piping for the collection and venting or recovery of gases produced from waste degradation. This layer should have filter layers above and below if the layer is not self-filtering. Careful consideration should be given to any penetrations of geosynthetic liners by the gas venting piping, so as not to degrade the barrier performance of the low permeability layer. Settlement is also a significant concern, as differential settlement between the venting pipe and the geomembrane can result in damage to the membrane.

Hydraulic Barrier Layer

Many factors must be considered in the design of the hydraulic barrier layer. These include: 1) effects of settlement, 2) effects of freeze-thaw cycles, and 3) slope stability (static and seismic) and shear stability at interfaces. Soil used for this layer must meet the following specifications:

- hydraulic conductivity of 1×10^{-7} cm/sec maximum
- minimum of 40% by weight must pass through #200 sieve
- minimum of 20% by weight < 2 um particle size
- plasticity index : 10% < PI < 40%
- density at least 95% Standard, or 90% Modified Proctor
- material retained on #4 sieve not to exceed 10% by weight

- clod size not to exceed 1/2 of lift thickness
- rock size not to exceed 0.75 to 1 inch in top lift (6 inches), and not to exceed 3 inches in lower lifts

Additional requirements apply if an admixture of native soil and bentonite clay is to be used for the low permeability layer. The bentonite should be added in a powdered form and mixed in a pugmill to produce the best blending.

Drainage Layer

In designing the drainage layer, the points of particular concern are: 1) determination of the need for a filter to prevent migration of fines into the drainage layer from overlying layers, 2) determination of the need for a piping system in the drainage layer to transport water to discharge points, 3) analysis of discharge points, especially with respect to erosion and, 4) an equivalency determination if geosynthetics are to be used as a drainage layer.

Vegetative Support / Protection Layer

Several factors must be considered in the design of the surface layer. Topsoil thickness affects the storage of water which can be used by plants. The physical properties of the topsoil have a direct effect on infiltration rates and consequently runoff and evapotranspiration. It is recommended that the surface be vegetated as soon as possible to minimize erosion. A dense stand of vegetation protects the cover from erosion and maximizes evapotranspiration. DEP recommends that the top 12 inches (30 cm) of the surface layer be capable of supporting vegetation, and that the total thickness be at least 18 inches (45 cm).

3.4 Federal Regulatory Requirements

Subparts G, K, and N of the Resource Conservation and Recovery Act (RCRA) Subtitle C (Hazardous Waste Management) regulations dictate the requirements for hazardous and mixed waste landfill cover systems (US EPA, 1991). The EPA recommends that a final cover system consist of the following (US EPA, 1991):

- A low hydraulic conductivity geomembrane / soil layer consisting of a 24 inch (60 cm) layer of compacted natural or amended soil with a hydraulic conductivity of 1×10^{-7} cm/sec in intimate contact with a geomembrane liner of minimum thickness 0.5 mm (20 mil).
- A drainage layer of 12 inch (30 cm) minimum thickness having a minimum hydraulic conductivity of 1×10^{-2} cm/sec, or a geosynthetic material of equal transmissivity.
- A top vegetative support / soil layer consisting of a top layer with vegetation or an armored surface, and a minimum of 24 inches (60 cm) of soil graded at a slope between 3 and 5 %.

The EPA allows these minimum design recommendations to be altered to meet site specific requirements, provided that the alternative design is equivalent to the EPA recommended design or meets the intent of the regulations. The EPA encourages design innovation, and will accept an alternative design upon a showing of equivalency. A general decision flowchart for cover system design formulation is shown in Figure 3-1 (US EPA, 1985).

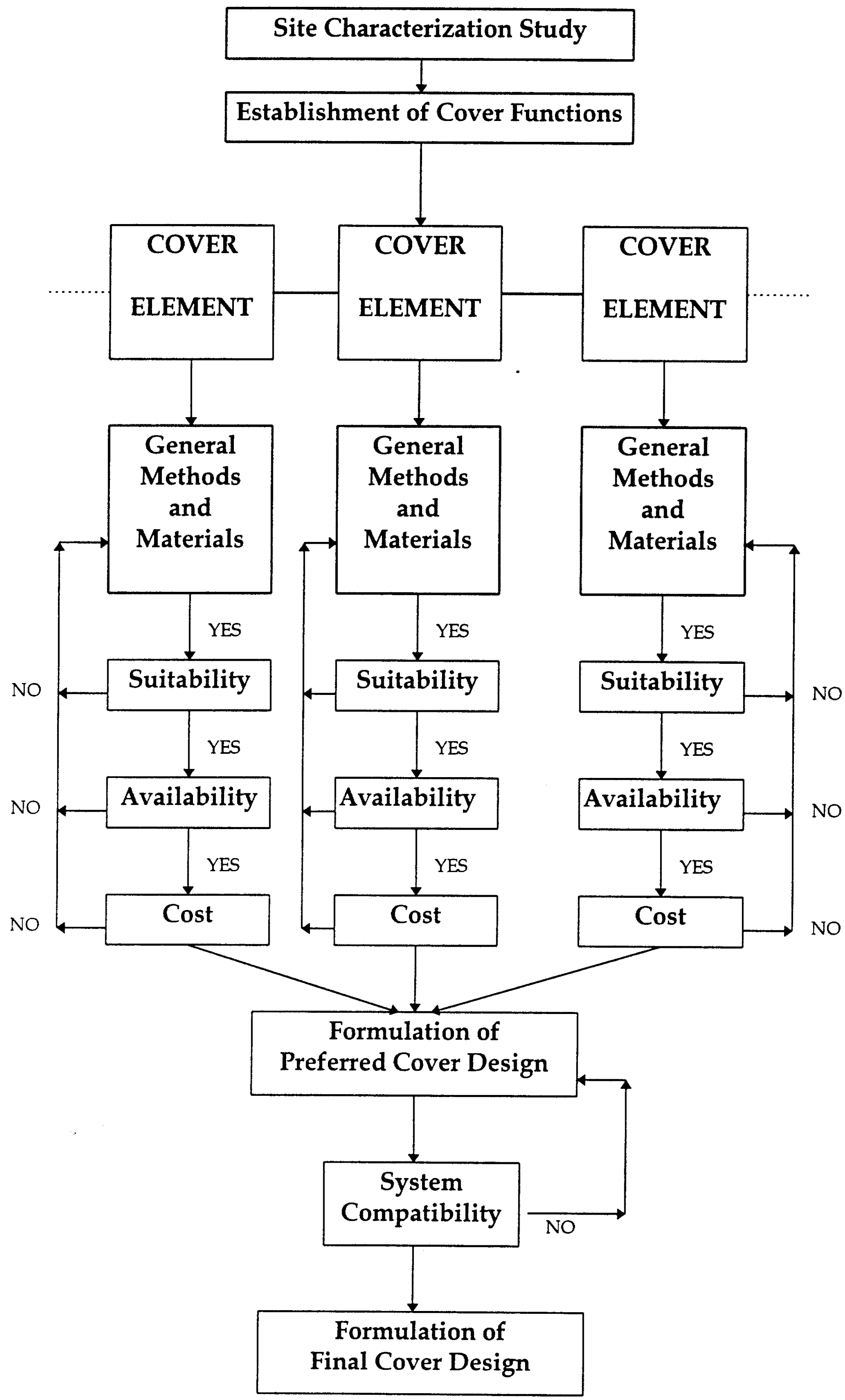
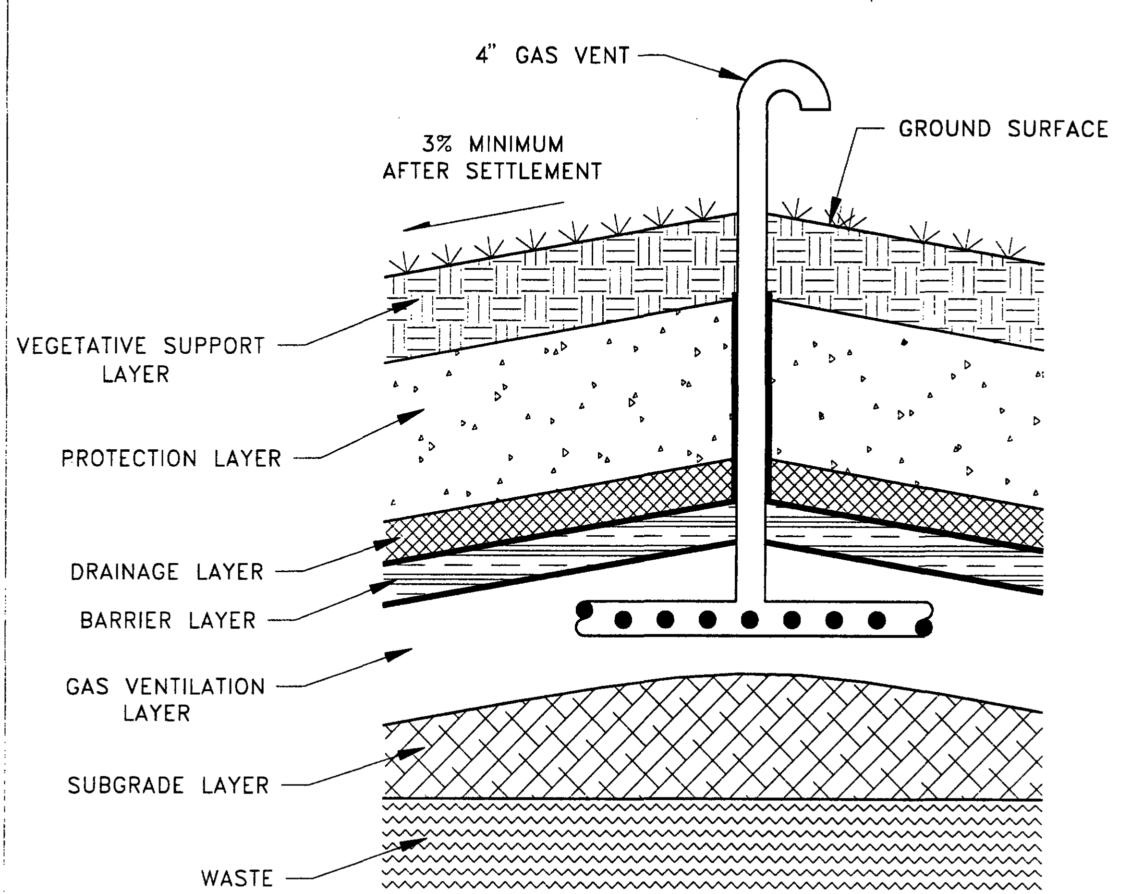


Figure 3-1: General Decision Flowchart for Cover Design Formulation (US EPA, 1985)

4.0 *COVER DESIGN*

This section presents the design process of the cover system layer-by-layer. A cross-sectional drawing of the proposed cover system is shown in Figure 4-1.



NOT TO SCALE

LAYER	MATERIAL	SPECIFICATIONS
Vegetative Support	Loam	6" thickness
Protection	Borrow Soil	18" thickness
Drainage	Geonet	Transmissivity > 3×10^{-5} m^2/sec
Barrier	Geomembrane over Geosynthetic Clay Liner	Geomembrane: 60mil VLDPE Geosynthetic Clay Liner: Gundseal with 40mil VLDPE Substrate
Gas Ventilation	Screened Borrow Soil	12" thickness Hydraulic conductivity > 1×10^{-3} cm/sec
Subgrade	Borrow Soil	2' minimum thickness

Figure 4-1: Cross-Section of Proposed Cover Design

4.1 *SUBGRADE LAYER*

4.1.1 Site Conditions

The 1951 cell area of LF-1 ranges from heavily wooded to open (ABB, June 1992). Prior to commencement of subgrade work, clearing operations must take place. Once the site has been cleared and stumped, grading procedures can commence. Topographic maps of the area indicate a relatively flat expanse, with a maximum elevation change of approximately 10 feet (ABB, April 1992). Aerial photos (see Figure 2-3) and work on previously covered cells has revealed a clear outline of the disposal trenches. Settlement has resulted in a bathtub-like shape of the trenches, the walls of which must be graded to produce a relatively uniform surface for placement of the subgrade (foundation) layer. The grading of the trenches may also accelerate settlement as the excavation equipment may act to compact the underlying waste.

4.1.2 Design Considerations

Several factors must be considered in the formulation of an appropriate subgrading plan. These include: regulatory requirements, environmental and aesthetic concerns, surface water drainage, soil erosion, cover component limitations, settlement, and stability (Sharma and Lewis, 1994).

4.1.3 Slope and Settlement

One of the critical design features of the foundation layer is the final slope. Because the trench (below-ground) method of disposal was used for the 1951 cell, the cover can be designed with a continuous top slope, as opposed to an above-ground landfill that requires both top and side slopes. This is illustrated in Figure 4.1-1 which shows cross-sectional views of different landfill layouts.

In practice, the recommended grade for a top deck is 3-5% (Sharma and Lewis, 1994), while state regulations recommend a minimum of 5% (MA DEP, 1993) and federal regulations require a minimum of 3% and a maximum of 5% (US EPA, 1989). This slope range has sufficient grade to promote some surface water runoff

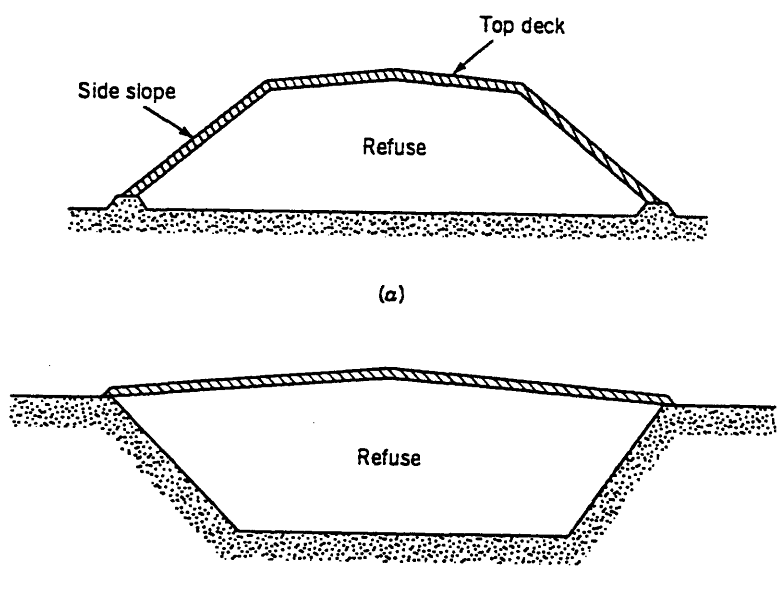


Figure 4.1-1: Landfill final cover configurations: *(a)* above-ground and *(b)* below-ground (Sharma et al.,1994)

while not being so steep as to promote erosion of the surficial soils. The top deck grade must be sufficiently steep to account for future settlement of the underlying waste. Settlement can cause flattening of the top slope which may produce pockets where surface water can accumulate, resulting in degraded performance of the cover system. To account for this, the foundation layer should be placed with a slope steeper than the minimum desired final slope of 3%. A thorough description of settlement mechanisms and calculations is presented in Section 6. The results of the analysis indicate that, by grading to an initial slope of 5%, the anticipated differential settlement can be accommodated (with an adequate factor of safety) without approaching the minimum allowable slope of 3%.

4.1.4 Material Selection

Materials typically utilized for foundation layers include a variety of soils, and some acceptable wastes. Figure 4.1-2 illustrates alternative materials used for foundation layers. The material used should be sufficiently incompressible to withstand the weight of construction equipment and the weight of overlying cover layers. The use of soil as a foundation has several advantages (Geosyntec, 1994). Practically any type of soil, other than a wet clay or high organic content material, will perform acceptably, therefore most locally available materials can be utilized in this role. Soil has a long history of use in the construction industry as a foundation material, therefore its properties and performance are well

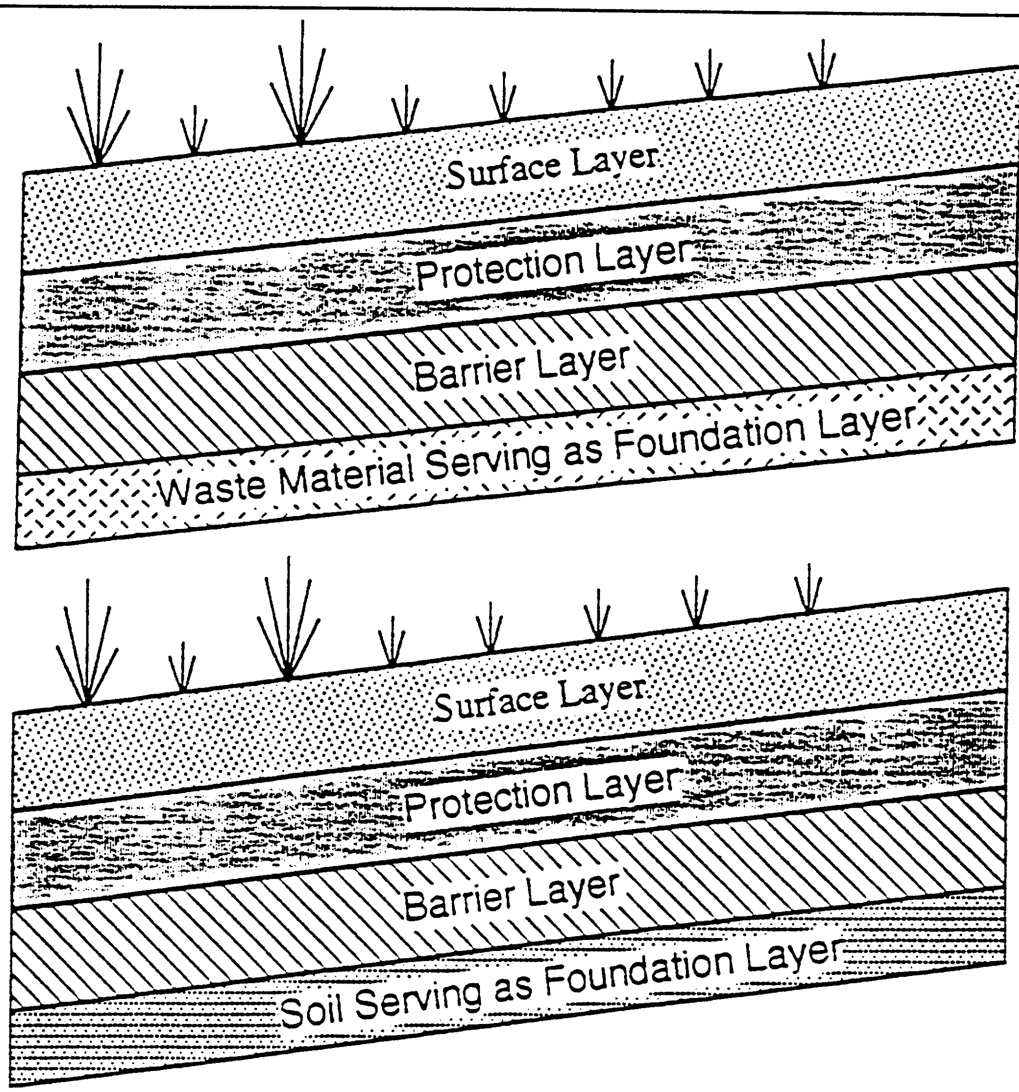


Figure 4.1-2: Alternative Materials for Foundation Layers (Geosyntec, 1994)

understood. At sites such as MMR, where soil borrow volumes are relatively plentiful, soil is the obvious choice for the foundation layer.

Many borings and test pits have been dug to investigate the characteristics of the native soil available in borrow pits surrounding the landfill area (ABB, 1993). The results of this work will be discussed here in the context of determining the applicability of the borrow soil for use in this and other layers of this closure system. Samples were characterized by means of grain size distribution and falling head permeability tests. The results indicate that the borrow area consists generally of two layers. The "upper layer" consists of a reddish-brown silty fine-to-medium sand to silty sand. Hydraulic conductivity results ranged from 3.1×10^{-3} to 1.4×10^{-3} cm/sec. Grain size analysis results ranged from 20 to 77 percent passing the No. 200 sieve. The "lower layer" of the borrow area consists of a fine-to-medium sand with trace to some fine-to-coarse gravel and trace cobbles. Laboratory permeability tests were performed on samples compacted with low to moderate effort, and revealed hydraulic conductivity ranging from 2.9×10^{-2} to 8.9×10^{-2} cm/sec. Based on the results of these borrow characterization tests, either "layer" of the borrow area is suitable for use in the foundation layer, with the lower layer being preferable based on its lower fines content.

4.1.5 **Placement**

Regulations require that the foundation layer be at least 12 inches (30 cm) thick, 6 inches (15 cm) of which may compose the gas drainage layer (MA DEP, 1993). This seems to imply that the minimum coverage provided by the foundation layer is six inches. In practice, it has been recommended (Sharma and Lewis, 1994) that the foundation layer be at least 2 feet (60 cm) thick to provide an adequate foundation for construction of the overlying cover system layers. While regulations contain no restrictions on the placement and compaction of the foundation layer, good engineering practice does. It is recommended that the foundation layer be placed in lifts of approximately 8 inches (20 cm) and compacted by 4 to 6 passes of a typical sheepsfoot roller (Jesionek and Dunn, 1995). This placement procedure should result in compaction to approximately 90% of the maximum dry density.

4.2 *GAS VENTILATION LAYER*

4.2.1 Design Considerations

Gas ventilation is typically not as much of a concern at a hazardous waste disposal facility as it is at a municipal waste facility. The reason for this is that the compounds deposited in a hazardous waste facility do not generally degrade to the extent that municipal waste does, thus they produce less gas. The main base landfill at MMR is a mixed waste facility containing both hazardous and municipal wastes. As a result, a gas ventilation layer is a necessary feature of the final closure system design.

There are several options available in the design of a gas ventilation layer. The fundamental choices are soils versus geosynthetics, and active versus passive systems. State (310 CMR 19) and federal (40 CFR 258 and 264) regulations assign minimum design criteria for the gas collection layer. State and federal regulations both require a minimum hydraulic conductivity of 1×10^{-3} cm/sec. State regulations require a minimum thickness of 6 inches (15 cm) for a soil layer, while federal regulations recommend a minimum 12 inch (30 cm) thickness. State regulations permit the use of synthetic materials upon approval by the DEP. The synthetic must be of sufficient strength to prevent deformation and impairment of function by the weight of vehicles and overlying cover; have sufficient flow capability; and be properly oriented for proper function (310 CMR 19.112). It is also required that, where needed, the gas collection layer (soil or synthetic) be bound on its upper surface with filter material (soil or synthetic) to prevent infiltration of fine material and to maintain the integrity of the layer (310 CMR 19.112).

The choice between active and passive systems is primarily based on the estimated gas generation of the site. An active gas collection system is a major capital investment. The questions that must be asked to determine whether installation of such a system is required are: 1) will a sufficient amount of gas produced to make gas recovery and reuse economically beneficial? 2) will sufficient gas be produced to impact nearby residential or business dwellings? 3) will sufficient gas be produced to result in failure to comply with Clean Air Act standards?

The 1951 cell is composed primarily of burn-fill and has existed for nearly 45 years with only a thin layer of intermediate cover separating waste from atmosphere (Weston, 1985). Two conclusions can be drawn from this information. First, because the refuse has been burned, the majority of organic material has been oxidized by fire precluding further degradation by microbial action. Second, the cell has only a thin intermediate cover of permeable soil, which has allowed relatively large rates of air and water infiltration into the waste. The infiltration of water and air enhances the decomposition process, and thus it seems likely that any material not oxidized by burning has already been biologically degraded. Based on the cell composition (predominantly burn-fill), the moist conditions provided by the intermediate cover, and the time since placement (40^{+} years) it is concluded that gas generation rates at the 1951 Cell will be low. Consequently, a passive gas venting system is recommended.

4.2.2 Material Selection

A passive gas venting system may be constructed of either a permeable soil layer or a geosynthetic layer (typically a geonet). Figure 4.2-1 illustrates material options for the gas ventilation layer. The advantages of using soil are (Geosyntec, 1994):

- long history of use
- sand layer adds to the performance of the foundation layer
- ease of installation

The major disadvantage of using soil is that suitable materials may not be locally available.

The advantages of using a geosynthetic gas collection layer are (Geosyntec, 1994):

- suitable geotextiles are available anywhere
- rapid and easy installation
- specialty materials can be manufactured to meet site specific requirements.

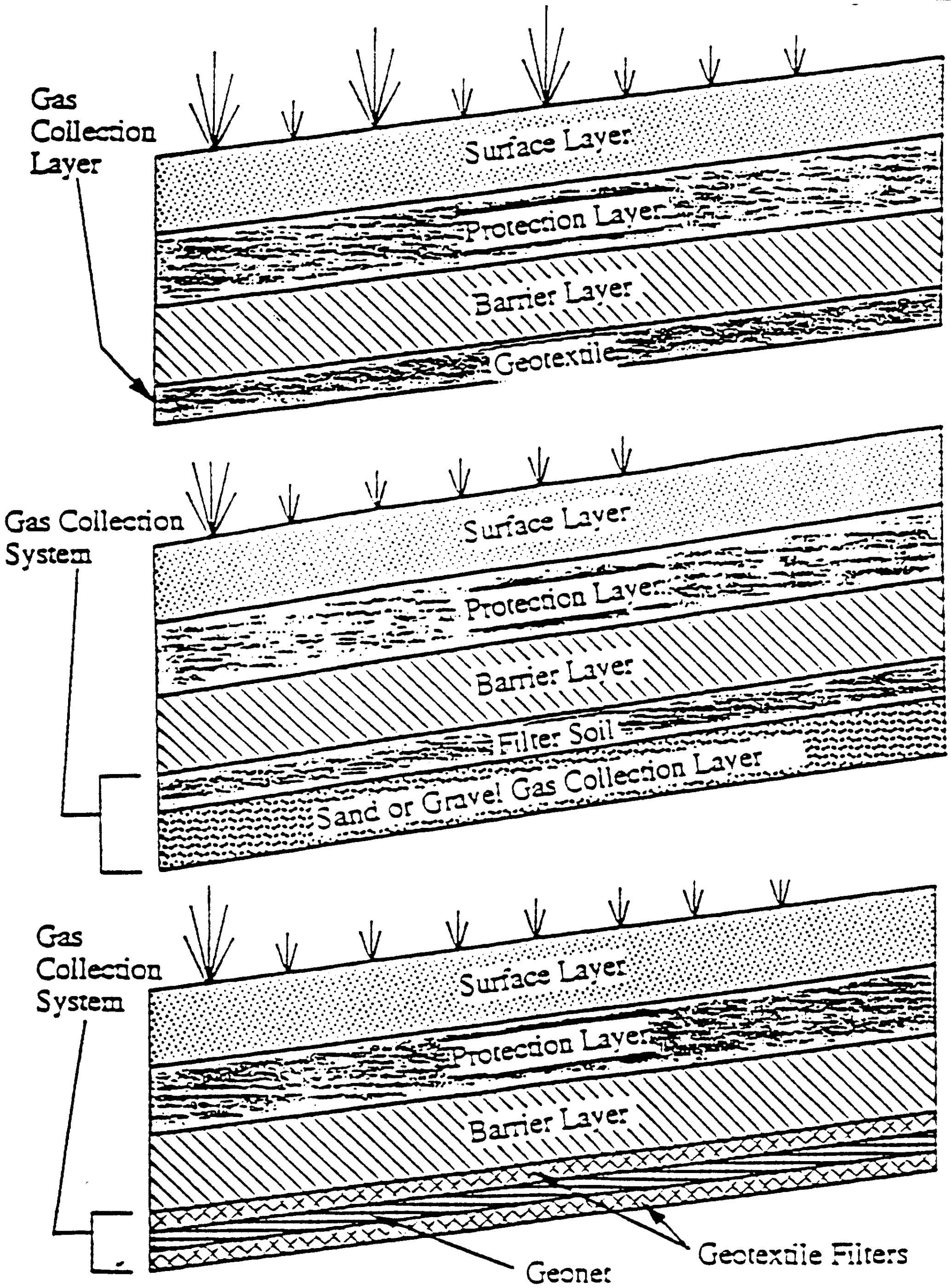


Figure 4.2-1: Alternative Materials for Gas Ventilation Layers (Geosyntec, 1994)

The major disadvantage of using a geosynthetic is the potential for slippage between it and an overlying geomembrane if a geomembrane is used in the hydraulic barrier layer (Geosyntec, 1994).

As discussed in Section 4.1.4, extensive testing and analysis has been performed to characterize the native soils available for borrow. The results of the analyses of the "lower layer" soils indicate hydraulic conductivity ranging from approximately $3x10^{-2}$ to $9x10^{-2}$ cm/sec. Regulations require a minimum conductivity of $1x10^{-3}$ cm/sec, therefore the "lower layer" soils are of acceptable conductivity for this application. The material must be screened to remove stones greater than 3/8 inch. Based on this material's acceptable properties and local availability, it is an excellent, low-cost choice for the gas collection layer.

4.2.3 **Placement**

Once screened, the material should be loosely placed in a single lift of 12 inches (30 cm). Because the layer is designed to be a permeable path for gas migration, compaction should be kept to a minimum. The soil should be placed with a light machine in a single lift with no further compaction efforts.

4.2.4 Gas Ventilation Piping

A passive venting system consists of a permeable soil layer, perforated PVC pipe for gas collection, and PVC risers to vent the collected gas to atmosphere. A

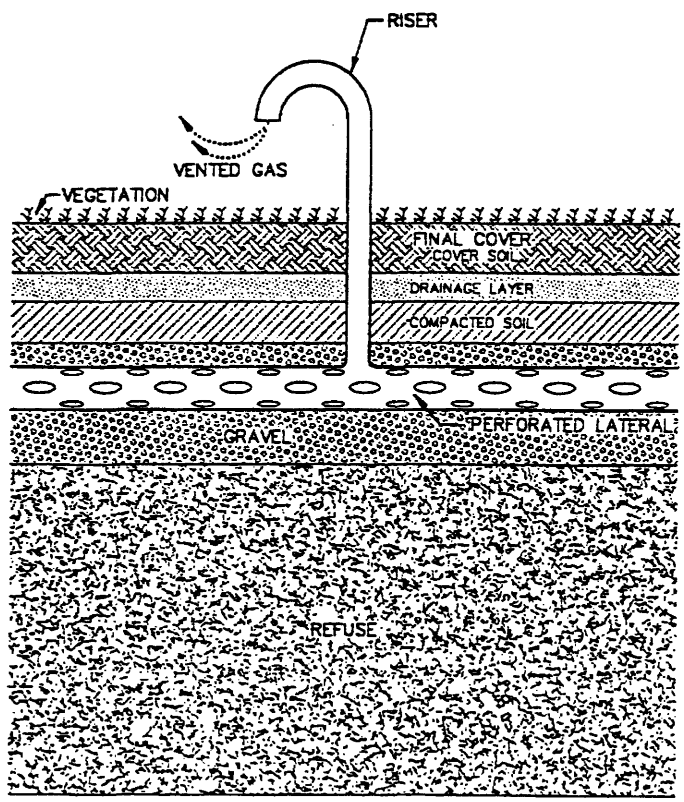


Figure 4.2-2: Typical Passive Gas Venting System
(McBean et al., 1995)

typical passive gas venting system is shown in Figure 4.2-2.

Methane is less dense than air, therefore it will tend to rise. The hydraulic barrier layer will impede the vertical migration of the gas, and the gas will be collected in the perforated piping and vented to atmosphere through the risers. Four inch (10 cm) diameter perforated flexible (to accommodate loading and settlement) PVC is recommended for the collector pipe, and 4 inch (10 cm) diameter non-perforated rigid PVC is recommended for the risers. There is no formal design procedure in practical use to determine the number of vents required for a passive system, but a thumb-rule of one vent per 10,00 yd^3 of waste may be used (Bagchi, 1990).

Using this thumbrule and the following information:

- Waste depth ~ 8 feet (Weston, 1985)
- 1951 cell plan area ~ 9 acres (ABB, April 1992)
- Assume ~ 75% of plan area contains waste
- Waste Volume = $8 \text{ ft.} \times 9 \text{ acres} \times \frac{43{,}560 \text{ ft}^2}{\text{acre}} \times \frac{1 \text{ yd}^3}{27 \text{ ft}^3} \times 0.75 = 87{,}120 \text{ yd}^3$
- # Vents Required = $87{,}120 \text{ yd}^3 \text{ waste} \times \frac{1 \text{ vent}}{10{,}000 \text{ yd}^3 \text{ waste}} = 8.7 \text{ vents}$

$\Rightarrow$ Round to <u>10 Vents</u>

To space the vents equally, a gas collection header will be placed along the crest of the cover, and two collection headers will be placed laterally on both sides of the crest. All headers will be spaced equally. Each of the five headers will include two ventilation risers spaced evenly along the length of the collection header. In order to vent gas to the atmosphere, the risers must penetrate the geomembrane of the overlying hydraulic barrier layer. This penetration must be accomplished while still maintaining the integrity of the membrane. The most common methods of sealing a membrane to a riser pipe are the boot seal and the

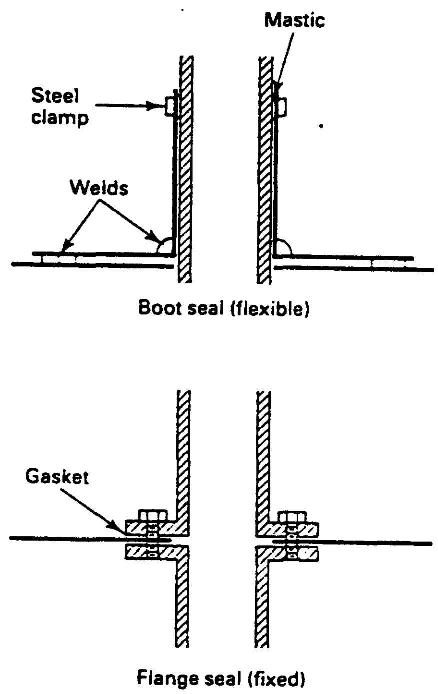


Figure 4.2-3: Membrane Penetration Seals (Koerner, 1994)

flange seal, both of which are shown in Figure 4.2-3. The boot seal is generally preferred over the flange seal because the boot design more easily accommodates cover settlement (Sharma et al., 1994).

4.3 <u>*HYDRAULIC BARRIER LAYER*</u>

4.3.1 Design Considerations

The barrier layer is designed to minimize the percolation of water through the cover system directly by impeding infiltration and indirectly by promoting storage and drainage of water in the overlying layers and eventual removal of water by runoff, evapotranspiration, and internal storage (Geosyntec, 1994). The materials most commonly used for the barrier layer are: a compacted clay liner (CCL), a geomembrane (GM), or a geosynthetic clay liner (GCL). Typically, and as required by regulation, two or more of these materials are used together to form a composite cover system as illustrated in Figure 4.3-1.

Federal regulations require a composite geomembrane / soil layer consisting of a 24 inch (60 cm) layer of compacted natural or amended soil with a hydraulic conductivity of $1x10^{-7}$ cm/sec in intimate contact with a geomembrane liner of minimum thickness 0.5 mm (20 mil), or an approved equivalent composite barrier layer. Thus, the primary decision is between a GM/CCL composite barrier and a GM/GCL composite barrier.

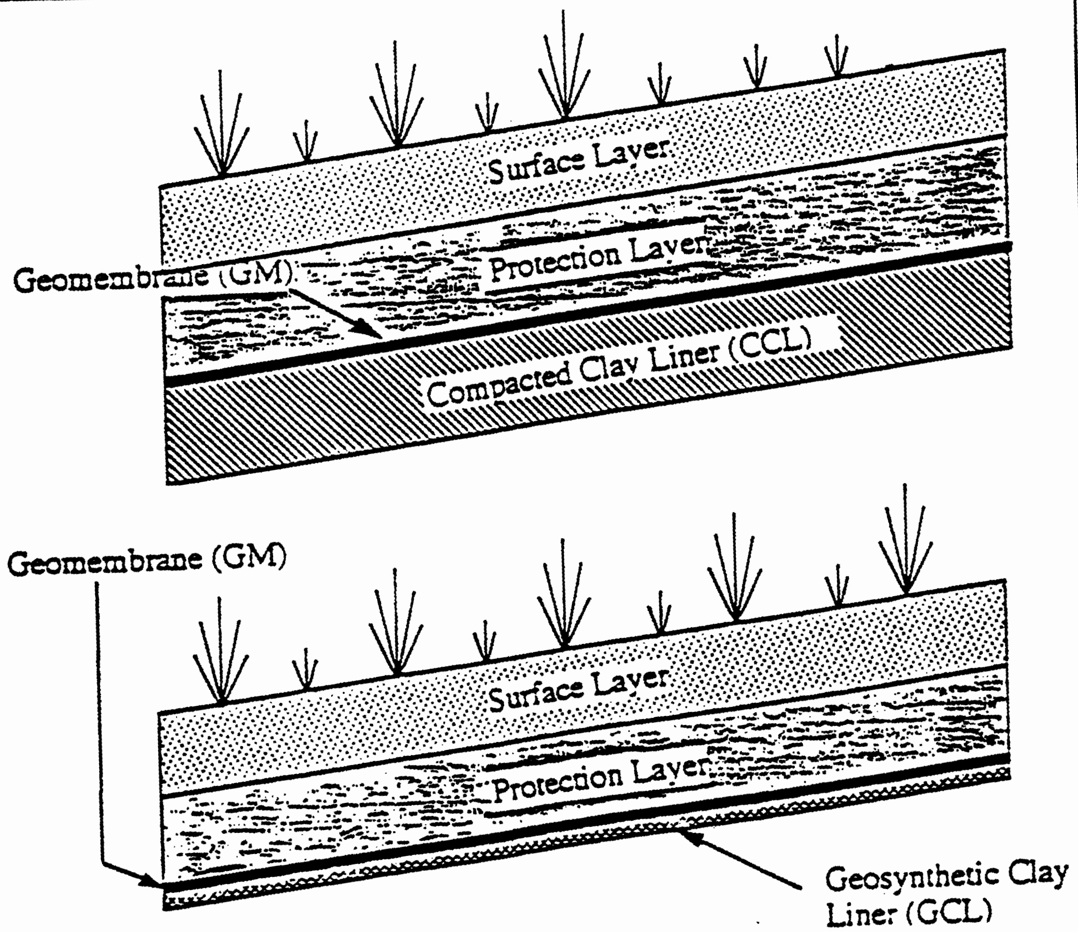


Figure 4.3-1: Alternative Materials for Barrier Layers (Geosyntec, 1994)

4.3.2 Barrier Layer Options

The advantages and disadvantages of the various materials are fairly well known, and tend to dictate which combination of materials is appropriate at a given site. The advantages and disadvantages of a compacted clay liner (CCL) are (Geosyntec, 1994):

<u>Advantages</u>:

- convenient if acceptable clay is locally available
- local clays can be blended with imported processed clay

- long history of use, familiar construction methods

Disadvantages:

- clay may dry out from below, causing cracking
- clay may dry out from above, causing cracking
- differential settlement of waste may result in tension cracks in clay
- freeze/thaw cycles may damage liner
- clay may be difficult to compact over a compressible waste layer
- clay may not be locally available
- liner is difficult to repair if cracks develop

The advantages and disadvantages of a geosynthetic clay liner (GCL) are (Geosyntec, 1994):

Advantages:

- straightforward and rapid installation
- materials are readily available and can be shipped anywhere
- GCL's exhibit some self-healing capability from minor punctures, desiccation, and freeze/thaw
- gas collection system penetrations are relatively easy to construct
- material is dry when placed, and can be installed in dry condition without desiccation damage
- easily repaired by patching

Disadvantages:

- low shear strength of hydrated clay
- potential slope stability problems
- vulnerable to puncture
- choice and placement of cover soil is critical to avoid puncture
- differential settlement can cause shifting that could jeopardize the liner's low conductivity
- dry cycles may cause shrinkage resulting in leakage until rehydration occurs
- long-term performance not well known

4.3.3 Site Specific Conditions

In designing a composite barrier layer, one must focus primarily on tailoring the design to the site specific conditions, while attempting to optimize performance, reliability, and economy by weighing the advantages and disadvantages described above. The critical factors that affect barrier layer selection are climate, differential settlement, erosion, puncture vulnerability, tolerable level of water percolation, need for gas collection, and slope steepness (Koerner and Daniel, 1992). The 1951 cell, and the Northwest Operable Unit as a whole, have many unique characteristics which must be taken into consideration during this design process:

- The cell is of the trench style (see figure 4.1-1), meaning there are no steep side slopes to be designed (top-deck final slope will be 3-5%).

- Maximum frost penetration depth in the region (see Figure 4.3-2) is approximately 27 inches (US EPA, 1990), therefore freeze/thaw protection would be a major concern if a CCL is used.

- Clay of acceptable permeability is not locally available, and would therefore have to be trucked in, adding significantly to the cost of installing a CCL.

- Settlement due to imposed surcharge stresses may be as great as 2 feet. (settlement estimations are presented in Section 6)

- Long-term protection of the underlying groundwater aquifer is the main design criterion, therefore percolation through the barrier must be kept to a practical minimum.

To determine the appropriate composite barrier layer, these site-specific characteristics must be analyzed both independently and collectively.

The lack of steep side slopes results in a generally less complex design for the final cover. The danger with steep side slopes is the tendency of soil to slide downward under the force of gravity. This problem is amplified when a geomembrane is utilized, as the friction angle between soil and a geomembrane is generally less than soil-to-soil friction angles (Koerner, 1995). Current federal regulations require the use of a flexible membrane liner (geomembrane) in conjunction with a compacted clay layer or its geosynthetic equivalent, therefore the issue of the soil-geomembrane interface is one that must be addressed. The shallow slope of this design proposal makes the GM/GCL a more plausible option, as compared to a site with steep side slopes. Static and seismic slope stability is addressed in Section 7. An important result of the stability analyses is that, although a shallow cover slope does not justify the use of one composite barrier (GM/GCL or GM/CCL) over the other, it also does not preclude the use of either barrier system.

Settlement, particularly differential settlement, must be taken into consideration in a cover design. A full discussion of settlement mechanisms and approximations is presented in Section 6. It is anticipated that the 1951 cell will undergo significant consolidation due to the surcharge stresses imposed by the weight of the cover system components. It is not anticipated that settlement due to decomposition of the waste fill will be significant. The cell is composed primarily of burnfill and has existed for nearly 45 years with a thin permeable layer of intermediate cover (Weston, 1985). Therefore, it is expected that any material not oxidized by burning has already been biologically degraded. Differential settlement may result from either variations in surcharge stress or by localized subsidence of the waste fill. The former mechanism of differential settlement is addressed in Section 6, and is not anticipated to be of sufficient magnitude to impact the design of the hydraulic barrier. The latter mechanism of differential settlement is more difficult to quantify, and must be accounted for in the design process with conservative calculations and adequate safety factors.

Frost penetration is a major concern for design of a final cover system in the Northeast. As shown in Figure 4.3-2, frost depth in this area of Massachusetts is between 27 and 30 inches (US EPA, 1990). The thermal coefficient of contraction

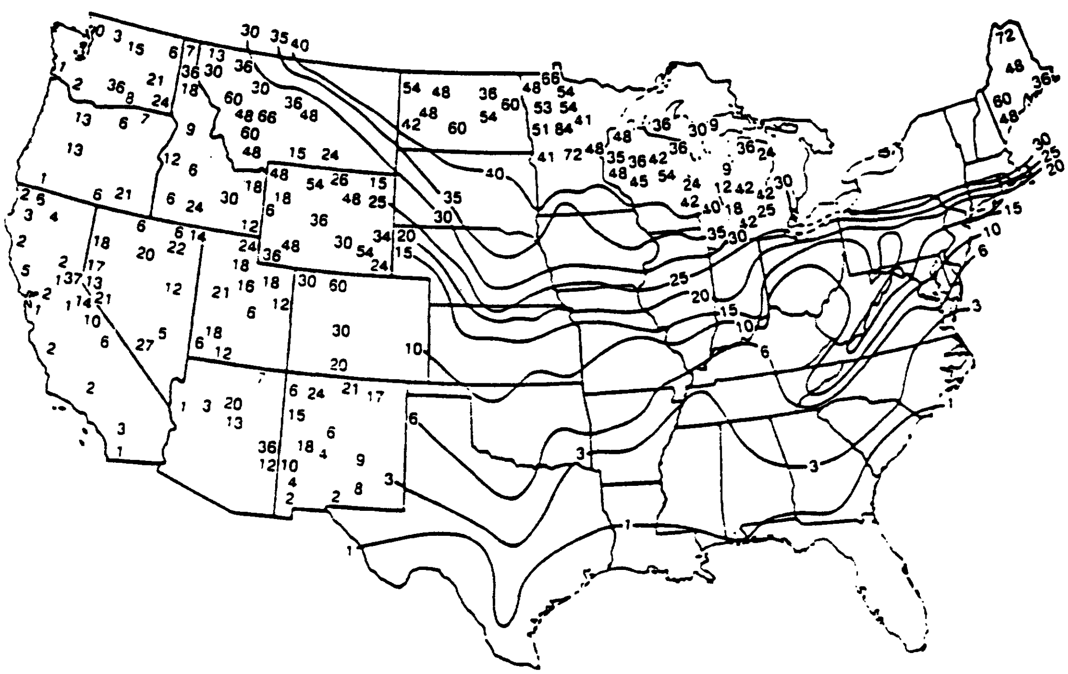


Figure 4.3-2: Regional Depth of Frost Penetration (inches)
(US EPA, 1991)

for soil is nearly three times higher than that of steel (Koerner, 1994). The result is that a small decrease in temperature quickly generates tensile stresses in the soil mass. Frozen ground is weak in tension, and fracturing commences at the ground surface, penetrating the cover soils to the depth required to relieve the stresses (Koerner, 1995). Studies conducted to determine the effect of freeze/thaw cycles on the hydraulic conductivity of fine grained soils (Zimmie and LaPlante, 1990) have found that conductivity increased *one to two orders of magnitude* for all soils tested, and that most of the damage occurred after only one or two freeze/thaw cycles. The implication is, if a compacted clay layer is utilized as a component of a composite hydraulic barrier, sufficient cover soil (27-30 inches) must be placed to protect the CCL from freeze/thaw damage. If sufficient cover is not provided, the CCL may rapidly lose its effectiveness as a barrier layer.

The other option is to replace the CCL with a GCL. Freeze/thaw testing of GCL's is currently limited to one of the four commercially available GCL products. The results of this testing (Eith, Boschuk, and Koerner, 1991) reveal much improved performance in comparison to the above described CCL test results. The testing was performed on several samples of Claymax®, a product of Clem Corporation. The specimens were subjected to laboratory testing conditions that simulated one-dimensional propagation of a freezing front. After subjecting the specimens to 0, 1, 5, and 10 freeze-thaw cycles, the hydraulic conductivity was measured using a falling head permeability apparatus. The results after 0, 1, 5, and 10 cycles were 4.0×10^{-10}, 3.8×10^{-10}, 2.2×10^{-10}, and 1.5×10^{-10} cm/sec respectively (Eith et al., 1991). The conclusion reached was that the tested product was not frost-susceptible for the given test conditions. The results of the CCL and GCL freeze/thaw test indicate that in an area of significant frost penetration, a GCL is probably the better choice (from the perspective of freeze/thaw performance) as the lower layer of a composite hydraulic barrier.

The next site-specific characteristic to be considered is that clay of acceptable conductivity is not locally (i.e., on-site) available. The material would have to be purchased from an off-site supplier and trucked in, adding significantly to the overall cost of barrier layer construction. While an overall cost/benefit analysis will not be considered here, this increased capital expenditure is worthy of consideration in the context of choosing the most appropriate composite barrier system.

The main design criterion for the hydraulic barrier layer is protection of the underlying groundwater aquifer. This area of Cape Cod is supplied by a single source aquifer, therefore percolation through the cover must be minimized to limit further leaching of waste chemicals into the underlying aquifer. From a hydraulic conductivity perspective, a properly placed geosynthetic clay liner is superior to a compacted clay layer. Field performance evaluations (McBean et al., 1995) have revealed that a typical CCL installed with appropriate construction quality assurance exhibited a minimum hydraulic conductivity on the order of $5x10^{-7}$ cm/sec. In comparison, GCL hydraulic conductivity values published in manufacturers' literature range from $5x10^{-8}$ to $\leq 1x10^{-12}$ cm/sec (Eith et al., 1991). When prevention of groundwater from contamination is a primary concern, Danielson and Richardson (1995) strongly support the use of geosynthetics in the barrier layer of a cover system, with the following reasons given:

- Properly installed GMs and GCLs are the least permeable barrier materials available.

- The water infiltration rate through properly installed GMs and GCLs is expected to be several orders of magnitude less than the percolation through a CCL.

- GMs and GCLs are more easily repaired than a CCL if damage should occur, and GCLs exhibit the ability to self-seal small penetrations that would compromise a CCL.

- GCLs are far less vulnerable to damage from differential settlement, desiccation, and freeze/thaw than a CCL, thereby offering better groundwater protection.

- Koerner and Daniel (1992) explain that of the two-layer composite barrier systems, the GM/GCL outperforms the GM/CCL both in cost and performance, and they recommend it unless site-specific conditions preclude its use.

4.3.4 **Material Selection**

Based on the site-specific conditions described above, and the results of the numerous field and laboratory tests, this design proposal recommends that a GM/GCL composite barrier be utilized in this cover system. While it would be feasible to install a CCL rather than a GCL, the increased capital expenditure, the labor-intensive placement, and its relatively low durability and reliability make it an inferior choice in this application.

As described by Cadwallader (1991), landfill closures require a different set of properties from a synthetic membrane than do landfill liner applications. Specifically, the issues of slope stability and the accommodation of differential settlement must be addressed. A product that exhibits many of the physical properties necessary to perform acceptably under these conditions is the textured very low density polyethylene (VLDPE) geomembrane. The attributes and material properties of VLDPE are described extensively by Cadwallader (1991), some of which are summarized here: VLDPE exhibits many of the durability features of HDPE, for example, lack of plasticizers, high strength without reinforcement, low temperature resistance, and resistance to microorganisms and rodents. VLDPE also exhibits excellent inherent (without plasticizers) flexibility, excellent stress crack resistance, and very good performance in puncture and multiaxial elongation testing. Additionally, VLDPE is available with a textured surface which significantly improves the membrane's friction properties. Table 4.3-1 presents a comparison of friction angles from direct shear testing for a textured and a smooth polyethylene membrane in contact with various materials.

A study comparing the performance of smooth and textured geomembranes in landfill covers was performed by Giroud et al. (1990) at a municipal solid waste disposal site in Connecticut. The evaluation consisted of full-scale field testing of three different geosynthetic landfill caps at a large solid waste landfill owned by Waste Management of North America, Inc. in New Milford, Connecticut. Three test pads were constructed with the same dimensions and layout. The test pad slope was 3H:1V (slope angle $\beta = 18.4°$). Design rainfall events were simulated by

	Direct shear friction angle (degrees)	
Sliding Surface	Standard (smooth) Polyethylene	Textured Polyethylene
Polyethylene / clay	16	24
Polyethylene / Ottawa sand	17	26
Polyethylene / concrete sand	23	29
Polyethylene / nonwoven geotextile	11	29

Table 4.3-1: Textured vs. Smooth Geomembrane Friction Angles (Cadwallader, 1991)

a surface irrigation system. All three test pads were constructed with the following basic cross section:

- 0.6 m (2 ft) thick soil cover composed of 0.15 m (0.5 ft) of topsoil and 0.45 m (1.5 ft) of silty sand.
- geosynthetics
- subgrade soil

The geosynthetics of Test Pad A consisted of a geotextile bonded to a geonet, laid on a rough geomembrane (GT/GN/R-GM). The geosynthetics of Test Pad B consisted of a geonet with a geotextile bonded to both faces, laid on a rough geomembrane (GT/GN/GT/R-GM). The geosynthetics of Test Pad C consisted of a geotextile bonded to a geonet, laid on a smooth geomembrane (GT/GN/S-GM). Both the smooth and the rough geomembranes used in the test pads were 1 mm thick HDPE manufactured by Gundle Lining Systems, Inc. The geonet used in the test pads was a 5 mm thick Polynet PN 3000 manufactured by Fluid

Systems, Inc. The geotextile used in the test pads was a 250 g/m^2 polyester needlepunched nonwoven geotextile manufactured by Hoechst Celanese Corporation. All bonding of adjacent geonets and geotextiles was performed at the factory.

Observations at the test pad consisted of monitoring the movement of the geosynthetics and movement of the cover soil by visually noting relative motion between a fixed reference point and a reference point on the layer of interest. The observations taken over approximately a four month period revealed the following:

- Pad A: GT/GN/R-GM; The geomembrane of Pad A was not under tension and exhibited gentle undulations. The GT/GN composite was taut at the top of the slope and there was a gap between it and the underlying GM. At mid-slope, the GT/GN composite was in contact with the GM. In the lower half of the slope, the GT/GN composite exhibited wrinkles, while the underlying GM was flat.

- Pad B: GT/GN/GT/R-GM; After four months, the GM of Pad B did not appear to be under tension, and it exhibited gentle undulations. The GT/GN/GT composite was in contact with the GM and exhibited gentle undulations following those of the GM.

- Pad C: GT/GN/S-GM; The geosynthetics of Pad C exhibited movement during construction, and the GM was partially pulled out of its anchor trench. Four months later, the GM was observed to be under tension in the top half of the slope. The GM was taut and was bridging irregularities of the underlying soil surface. The GT/GN composite was under tension in the top half of the slope. In the bottom half of the slope, the GM and the GT/GN composite exhibited many wrinkles. These observations indicate that the GT/GN composite moved relative to the GM, and the GM moved relative to the underlying soil layer.

To summarize these results, the rough GMs (Pads A and B) did not exhibit tension and wrinkles whereas the smooth GM (Pad C) did. The conclusion to be drawn from these results with respect to geomembrane behavior is that the interface shear strength performance of the rough geomembrane was superior to that of the smooth geomembrane.

These field observations are supported by the stability calculations of Section 7. The textured geomembrane does provide a higher factor of safety against static and seismic slope failure. The results of the stability calculations in Section 7 show that under static conditions, the smooth membrane performs acceptably, with all safety factors above the minimum recommended. However, under seismic conditions, use of the smooth membrane does not provide an acceptable safety factor at the drainage layer interface (0.72), while the textured membrane does (1.0).

The textured VLDPE membrane has the material properties necessary to withstand the unique conditions of a landfill cover in a seismically active region, and it has been deployed successfully at numerous sites including the New Milford site described above. Consequently, textured VLDPE is recommended in this design.

There are various methods of determining the required thickness of a geomembrane in a cover system barrier layer. Regulations require a minimum 20 mil (0.5 mm) thickness (see section 3.2). Koerner has published recommended minimum geomembrane properties based on desired degree of survivability (Koerner, 1994). The recommended minimum thickness for very high survivability is 40 mils (1.0 mm). A third method of determining required thickness is a calculation based on deformations the membrane might experience

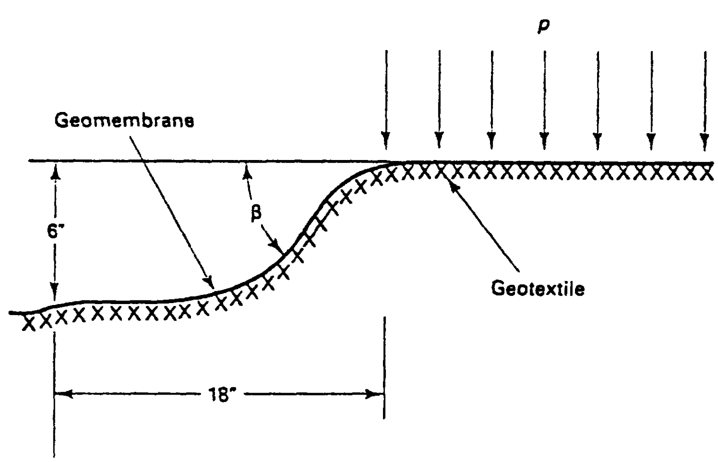


Figure 4.3-3: Design Model for Geomembrane Thickness Calculation (Koerner, 1994)

during its service lifetime (Koerner, 1994). The model used in the calculation is illustrated in Figure 4.3-3. The model addresses the situation where a deformation induced by settlement has occurred. The resulting x-direction forces are summed and equated to zero, resulting in the following equation (Koerner, 1994):

$$\sum F_x = 0$$

$$\Rightarrow F\cos\beta = T_U + T_L$$

$$\Rightarrow (\sigma_{allow} t)\cos\beta = (p\tan\delta_U + p\tan\delta_L)x$$

$$\Rightarrow t_{REQD} = \frac{p}{\cos\beta}\frac{x}{\sigma_{allow}}(\tan\partial_u + \tan\partial_L)$$

where:

ΔH = the settlement mobilizing the stresses
F = the force mobilized in the membrane
t = the membrane thickness
T_L = the shear force below the membrane
T_U = the shear force on top of the membrane
p = applied vertical pressure
β = deformation angle
x = distance of mobilized membrane deformation
σ_{ALLOW} = allowable (yield) stress
δ_U = friction angle between membrane and upper layer
δ_L = friction angle between membrane and lower layer

inserting the following estimated values:

p = 10 lb/in^2 (includes soil and equipment loads)
β = 20° (conservative estimate)
x = 10 in. (Koerner, 1994, Figure 5.10)
δ_U = 20° (Cadwallader, 1991) (conservative estimate)
δ_L = 20° (Cadwallader, 1991) (conservative estimate)

the calculation produces a required thickness:

t_{REQD} = 0.052 in. (52 mils)

To summarize, regulations recommend a 20 mil minimum (EPA, 1991), survivability estimates recommend a 40 mil minimum (Koerner, 1994), and conservative calculations recommend a 52 mil minimum (note: the calculations presented are estimates, an actual design process would include laboratory testing with the materials being used at the site to accurately determine the parameters that have been estimated here). To be conservative, a 60 mil (1.5 mm) thickness will be recommended in this design.

The next step is to choose a geosynthetic clay liner that is appropriate for use in a cover system. As of 1994, there were five commercially available GCL's (Koerner, 1994). Four of the five manufacturers use a geotextile as both the substrate (carrier layer) and the cover layer (Koerner, 1994). The fifth manufacturer uses a polyethylene geomembrane as the substrate and does not utilize a cover layer (Eith et al., 1991). This product, Gundseal,® is manufactured by Gundle Lining Systems, Incorporated. Gundseal® consists of adhesive-bonded bentonite adhered to a geomembrane (Koerner, 1994). The fact that Gundseal® is manufactured with a geomembrane as a carrier layer makes it ideal for use in a landfill liner or cover system. The geomembrane has a much lower vertical hydraulic conductivity than the geotextiles used in the other products, and the geotextiles have the added disadvantage of a much greater in-plane (lateral) conductivity as compared to a geomembrane (Eith et al., 1991; Struve, 1991). Gundseal® can be manufactured with either smooth or textured (rough) HDPE or VLDPE as the substrate with thickness (of the geomembrane) ranging from 20 to 80 mils (0.5 to 2.0 mm) (Koerner, 1994). This GCL can be deployed either clay-side up or clay-side down. The ideal configuration for a landfill cover is to deploy the Gundseal® GCL clay-side up thus sandwiching the bentonite layer between its substrate geomembrane and the overlying geomembrane of the composite barrier system (Koerner, 1994). The material properties and manufacturing options offered by the Gundseal® GCL make it an excellent choice for a landfill cover system. The specifications recommended in this design proposal are a textured VLDPE substrate with a 40 mil thickness, thus providing adequate strength and slope stability performance. A textured substrate is recommended based on the seismic slope stability safety factors calculated in Section 7. The smooth substrate / gas collection layer interface friction angle resulted in a seismic factor of safety of approximately 1.1; the textured substrate resulted in a safety factor of 1.7 at the same interface. See Section 7 for a complete stability discussion.

Another advantage of Gundseal® is that no mechanical joining of the sheets is necessary (Struve, 1991). Struve (1991) describes that adjacent sheets should be overlapped 75 - 150 mm (3 - 6 in.). Bench scale testing evaluating leakage through an overlap seam of a Gundseal® liner is also described by Struve (1991). The testing was conducted at the University of Texas at Austin. The tests measured leakage through overlaps of 75 mm and 37.5 mm (to determine if the minimum recommended overlap of 75 mm provided a factor of safety against leakage). The overlap seams were covered with a one foot layer of gravel and a two foot head of water. No leakage was detected from either overlap during five months of weekly observations (Struve, 1991).

4.4 *DRAINAGE LAYER*

4.4.1 Design Considerations

The basic design considerations summarized here are as described in US EPA, 1989. The drainage layer functions to remove water which infiltrates the vegetative support/protection layer. The two most widely accepted options for the drainage medium of a cover system are a suitable granular layer or a geonet. A geonet is a synthetic drainage net. It is typically a thin (3-8 mm), diamond-shaped, HDPE (high-density polyethylene), extruded sheet with high in-plane transmissivity and high compressive strength (Austin, D., 1991). A typical geonet is shown in Figure 4.4-1.

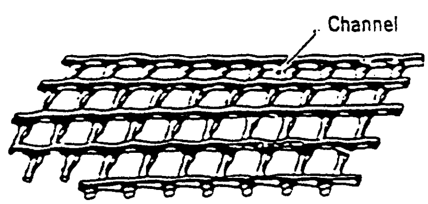


Figure 4.4-1: Typical Geonet
(Koerner, 1994)

A drainage layer should be designed to minimize the standing head and residence time of water on the barrier layer in order to minimize leachate production. Another important consideration is the prevention of physical and biological clogging of the drainage medium. Physical clogging may be caused by the migration of soil particles of surrounding layers or by the intrusion of adjacent geosynthetics into the apertures of a geonet (Koerner, 1994). Physical clogging can be prevented by the installation of a soil or geosynthetic filter layer between the overlying protection layer and the drainage layer. Biological clogging is caused primarily by the intrusion of roots from surface vegetation. Biological clogging can be prevented through the use of shallow-rooted vegetation and/or the use of a biotic barrier layer. The EPA (1989) recommends the following design features for a granular material drainage layer:

- Minimum thickness of 12 inches (30 cm), and minimum slope of 3%.
- Minimum hydraulic conductivity of 1×10^{-2} cm/sec (corresponding to minimum transmissivity of 3×10^{-5} m^2/sec) at the time of installation.
- Granular material no coarser than 3/8 inch, classified as SP with low fines content, grains should be smooth and rounded with no debris that could damage (i.e., puncture) an adjacent geosynthetic layer.
- A soil or geosynthetic filter layer should be installed between the drainage layer and the protection layer to prevent migration of fines.

The EPA (1989) recommends the following design features for a geosynthetic drainage layer:

- Same minimum flow capability (transmissivity) as a soil drainage layer.
- Use of a geosynthetic filter layer above the drainage layer to prevent intrusion and/or clogging.
- As required, the use of a geosynthetic filter layer beneath the drainage layer to increase friction between the drainage layer and the underlying GM, and to prevent intrusion of the GM into the apertures of the drainage layer.

4.4.2 Drainage Layer Options

As mentioned above, the two most widely accepted options for the drainage medium of a cover system are a suitable granular layer or a geonet. Figure 4.4-2 illustrates these options.

The choice between the two options is based primarily on availability of

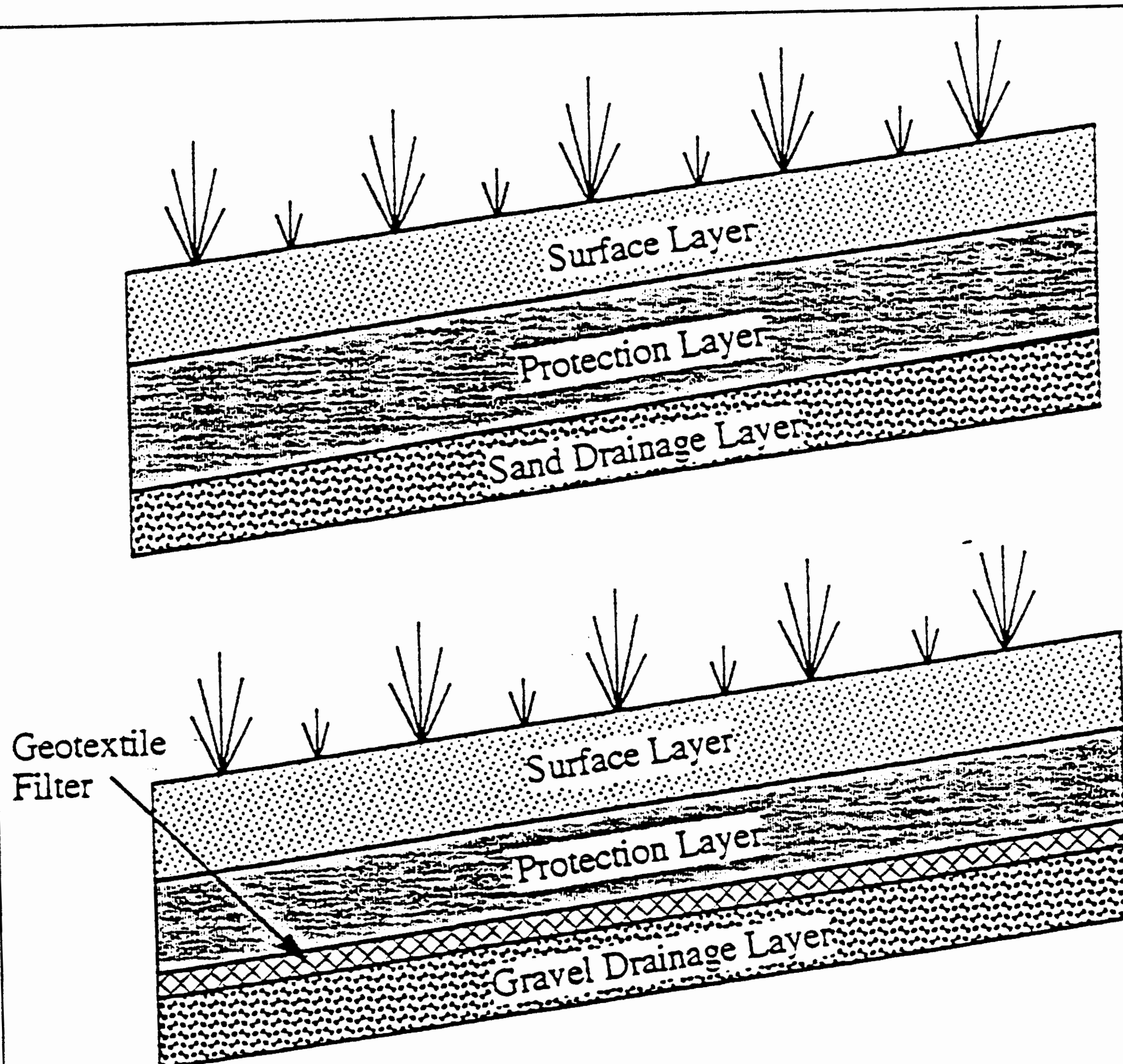


Figure 4.4-2: Alternative Materials for Drainage Layers (Geosyntec, 1994)

considerations, and cost (Geosyntec, 1994). As described in Section 4.1.4, borings and test pits have been used to characterize the soil in the on-site borrow area to determine its applicability for use in the cover system. The "lower layer" soils previously described are the most appropriate for this application. These soils are predominantly of the SP and SM classifications, with hydraulic conductivity on the order of $5x10^{-2}$ cm/sec (ABB, 1993). These soils would be acceptable for use in the drainage layer once screened on a 3/8" sieve. If a granular material is used as a drainage layer, a soil or geosynthetic filter (geotextile) must be installed between the overlying protective soil layer and the drainage layer to prevent clogging from the migration of fines into the drainage layer. The advantages of using a granular material for the drainage layer are (Geosyntec, 1994):

- The thickness of the layer will help to protect underlying layers from puncture, intrusion, and freeze/thaw.
- Sand as a drainage medium has a long history of use in the engineering field.
- Based on the grain size distribution of the drainage layer, a filter may not be necessary between the protection layer and the drainage layer.

The disadvantages of using sand as the drainage layer medium are:

- Fines in the sand may migrate downslope, leading to buildup of pore pressure, and possibly impacting slope stability.
- The use of a 12 inch thick drainage layer greatly increases the cross-sectional height of the cover as compared to using a geonet.
- A filter is required to prevent migration of fines into the drainage sand from the overlying protection layer.

- Placement of a sand layer is much more labor intensive, and the use of machinery increases the probability of damaging the underlying geomembrane.

The most common alternative to sand as the drainage layer medium is the use of a geonet. To prevent intrusion and subsequent clogging of the geonet's apertures, a geotextile filter is required between the geonet and adjacent layers. The primary advantages of using a composite geonet-geotextile for the drainage layer are (Geosyntec, 1994):

- Materials are readily available.
- Lightweight installation equipment can be used, thus reducing the possibility of damaging the underlying liner.
- Simple, rapid installation as compared to a soil layer.
- Geotextiles can be bonded to either or both sides of the geonet by the manufacturer, thus further simplifying the installation process.
- Geonets can be ordered to meet site-specific requirements.

The major disadvantages of using a geosynthetic drainage layer are (Geosyntec, 1994):

- The thin layer does not help much in the protection of the underlying liner from freeze/thaw.
- The use of geotextiles might reduce the interface friction between the drainage layer and the liner (as compared to a soil drainage medium).
- Little data exists as to the long-term durability/survivability of geosynthetics in such an application.

4.4.3 Material Selection

If soil is used as the drainage layer medium, the process will include: excavating and screening the soil, transporting it to the site and stockpiling, then spreading with close construction quality assurance to ensure proper grading and minimum compaction to meet conductivity requirements. This is a very time-intensive process requiring extensive, costly use of heavy equipment. The alternative geosynthetic layer is relatively straightforward to install, and can be factory ordered to meet site-specific design requirements. Based on these considerations, the recommended drainage layer material for this cover system is a geonet.

While there are numerous geonet manufacturers, there are only three basic types of geonets currently available in the United States: geonets with extruded solid ribs, extruded foam ribs, and drawn solid ribs (Koerner, 1994). All three types are formed from high density polyethylene. Of the three types, the extruded solid rib geonet has seen the most use in environmental applications (Koerner, 1994). The drawn solid rib geonet is a fairly new product, and has been shown to have poor performance in compression testing as compared to the two extruded rib types. Based on the results of mechanical and hydraulic testing , and its widespread acceptance for this application (Koerner, 1994), an extruded solid rib geonet with minimum transmissivity of $3x10^{-5}$ m^2/sec is recommended for this design.

When using a geonet as a drainage layer in a cover system, it is strongly recommended that a geotextile filter be placed between the geonet and both adjacent layers of the cover (Koerner, 1994). The upper geotextile serves to minimize extrusion of soil particles from the protection layer, while the lower geotextile serves to minimize intrusion of the adjacent geomembrane (Koerner, 1994). Both geotextiles serve to increase interface friction values. These and other factors regarding geotextile selection are discussed in greater detail in the following paragraphs.

Koerner (1994) strongly emphasizes the importance of choosing the proper geotextile for covering a geonet which has an adjacent soil layer. The following discussion summarizes his recommendations. A geotextile used in a filter application is designed primarily to minimize physical clogging of the geonet by adjacent soil layers. However, the geotextile must also be designed to span the apertures of the geonet without excessively intruding into the geonet's core space. One method of minimizing intrusion of the geotextiles into the geonet's apertures is to use a high-modulus woven monofilament geotextile. In an application such as a landfill cover, the open spaces in the woven fabric will permit extrusion of the overlying soil layer into the geonet openings, which is not acceptable. To prevent the extrusion of overlying soil particles, a needle-punched nonwoven geotextile with multiple layers of continuous fibers may be used. The drawback of a needle-punched nonwoven is that increased intrusion of the fabric into the geonet's aperture must be expected and accounted for. A compromise fabric is a heat-bonded nonwoven geotextile. Heat bonding refers to the process by which the web filaments are bonded together. In the heat bonding process, a web of continuous filaments is melted together at filament crossover points. The result is a fabric that provides both high modulus (to prevent intrusion) and high fiber overlapping (to prevent extrusion). This fabric has seen fairly wide use in environmental applications such as landfill liners and covers. Based on the results of practical applications, a minimum mass per unit area of 8.0 oz/yd^2 (260 g/m^2) is recommended (Koerner, 1994).

Hwu et al. (1990) have performed extensive laboratory testing to examine the issue of geotextile and soil intrusion into geonets. Their studies compared the behavior of geotextiles of varied polymeric material, fiber type, manufacturing method, and fabric weight (mass per unit area) under varying pressures with various overlying soil types. These testing variations produced the following general conclusions (Hwu, Sprague, and Koerner, 1990):

- The geotextile/soil intrusion results in geonet flowrate decreases of 39 to 88% of the geonet's maximum capacity.
- Increasing pressure increases intrusion and thus decreases flowrate.
- Geotextile mass per unit area is not a very sensitive variable with respect to intrusion, as long as the fabric can withstand the imposed stresses.

- Pressures up to 105 kPa (15 psi) did not cause short term failure of the geotextiles tested.
- Sand overlying the geotextile results in less intrusion than clay.
- Continuous filament geotextiles appear to be subject to less intrusion than staple fiber geotextiles.
- Fabric stiffening (resin treating, burnishing, and scrim reinforcing) increases initial modulus thereby reducing intrusion and increasing flowrate - but only to a limited extent.

The following design criteria summarize the recommendations reported by Koerner (1994) and the results of the testing by Hwu et al. (1990):

- Intrusion is a very real concern, and can reduce the flow capacity of a geonet by greater than 50%.
- Because a sandy layer will overlay the upper geotextile in this design, extrusion is not as much of a concern as compared to an overlying silt/clay layer.
- Since mass per unit area is not a very sensitive parameter, a "typical" fabric weight of 260 g/m^2 (8 oz/yd^2) should be sufficient.
- Intrusion and extrusion can be minimized through careful selection of the geotextile.

Another important aspect of geonet/geotextile selection is slope stability. A comprehensive evaluation of this issue was conducted by Giroud et al. (1990) the details of which were presented in Section 4.3. To recap briefly, this evaluation consisted of full-scale field testing of three different geosynthetic landfill caps and laboratory testing of the materials involved. Three test pads were constructed with the same dimensions and layout. The test pad slope was 3H:1V (slope angle $\beta = 18.4°$). All three test pads were constructed with the following basic cross section:

- 0.6 m (2 ft) thick soil cover composed of 0.15 m (0.5 ft) of topsoil and 0.45 m (1.5 ft) of silty sand.
- geosynthetics
- subgrade soil

The geosynthetics of Test Pad A consisted of a geotextile bonded to a geonet, laid on a rough geomembrane (GT/GN/R-GM). The geosynthetics of Test Pad B consisted of a geonet with a geotextile bonded to both faces, laid on a rough geomembrane (GT/GN/GT/R-GM). The geosynthetics of Test Pad C consisted of a geotextile bonded to a geonet, laid on a smooth geomembrane (GT/GN/S-GM). The geonet used in the test pads was a 5 mm thick Polynet PN 3000 manufactured by Fluid Systems, Inc. The geotextile used in the test pads was a 250 g/m^2 polyester needlepunched nonwoven geotextile manufactured by Hoechst Celanese Corporation. All bonding of adjacent geonets and geotextiles was performed at the factory.

Observations at the test pad consisted of monitoring the movement of the geosynthetics and movement of the cover soil by visually noting relative motion between a fixed reference point and a reference point on the layer of interest. The observations with respect to the drainage layers were:

- Pad A: GT/GN/R-GM; The GT/GN composite was taut at the top of the slope and there was a gap between it and the underlying GM. At mid-slope, the GT/GN composite was in contact with the GM. In the lower half of the slope, the GT/GN composite exhibited wrinkles, while the underlying GM was flat.

- Pad B: GT/GN/GT/R-GM; The GT/GN/GT composite was in contact with the GM and exhibited gentle undulations following those of the GM.

- Pad C: GT/GN/S-GM; The GT/GN composite was under tension in the top half of the slope. In the bottom half of the slope, the GM and the GT/GN composite exhibited many wrinkles. These observations indicate that the GT/GN composite moved relative to the GM, and the GM moved relative to the underlying soil layer.

To summarize these results with respect to drainage layer performance, the GT/GN/GT composite (Pad B) stayed in close contact with the rough GM of Pad B, whereas the GT/GN composites of Pads A and C did not remain in close contact with their respective GMs (note that Pads A and C did not have a geotextile between the geonet and the geomembrane as Pad B did). The conclusion to be drawn from these results is that the nonwoven geotextile bonded to the geonet helped prevent movement between the geonet and the rough geomembrane. Direct shear box testing conducted by Giroud et al. (1990) revealed a friction angle of 10° between the rough geomembrane and the geonet. In comparison, the reported friction angle between the nonwoven geotextile and the rough geomembrane was 15° (Giroud et al., 1990). Cadwallader (1991) reported a friction angle of 29° between a textured (rough) polyethylene geomembrane and a nonwoven geotextile. The variation between these results confirms the importance of conducting design-phase testing with the actual soils and geosynthetics intended for use in the specific project. The results do, however, generally support the hypothesis that the interface shear strength between a geotextile and a geomembrane is greater than that of a geonet - geomembrane interface. The issue of static and seismic slope stability is addressed further in Section 7.

Based on the various recommendations and test results cited, a stiffened, nonwoven, continuous fiber, geotextile is the recommended fabric in this design proposal. It is also recommended that this fabric be bonded to both faces of an extruded solid rib geonet by the manufacturer.

4.4.4 Toe Drain

The infiltrated water intercepted by the drainage layer must be collected and transported to a recharge area. The system designed to collect the flow from the drainage layer is termed a toe drain. A toe drain consists of a perforated PVC pipe bedded in a trench of washed crushed stone. The toe drain will be placed along the entire perimeter of the cover, and will be sloped to collect flow at the southeast corner of the cell (see Figure 1-1). From there, the flow will be transported via culvert to the borrow pit area shown in Figure 1-1 which is being utilized as a recharge area (ABB, June 1992).

4.5 *VEGETATIVE SUPPORT/PROTECTION LAYER*

4.5.1 General

The top layer of the cover system is actually comprised of two separate layers; the lower layer is termed the protection layer and the upper layer is termed the vegetative support layer. Federal regulations recommend a 24 inch (60 cm) minimum thickness for the two layers combined (US EPA, 1991). State regulations recommend an 18 inch (45 cm) minimum combined thickness with 12 inches (30 cm) of soil capable of supporting vegetation (MA DEP, 1993).

The vegetative support layer is more appropriately termed the surface layer. This terminology is more appropriate because the surface need not be covered with vegetation. The options for covering the surface include a geosynthetic erosion control material, cobbles, paving material, or vegetation (Geosyntec, 1994). In an area of ample precipitation (such as Cape Cod) where vegetation can be supported, the choice of vegetation as a surface cover has several significant advantages (McBean et al., 1995):

- aesthetically pleasing
- allows for possible recreational use of land in the future
- promotes evapotranspiration
- creates a leaf layer above the soil which reduces the kinetic energy of rainfall thereby decreasing erosion
- decreases surface wind velocity thereby decreasing erosion

- decreases water runoff velocities

The lower of the two layers is termed the protection layer. On-site or local soil is the most commonly used and typically the most suitable material for the protection layer (Geosyntec, 1994). The protection layer serves several functions (Geosyntec, 1994):

- storage of infiltrated water until removal by evapotranspiration
- protection of underlying layers from burrowing animals and plant roots
- minimization of human intrusion
- protection of underlying layers from excessive wetting and drying and from freeze/thaw cycles

4.5.2 **Protection Layer**

Suitable on-site materials are available for use in the protection layer. The on-site borrow materials have been characterized by borings and test pits as described in Section 4.1.4. Both the "upper" and "lower" layers of the borrow area are suitable for use in the protection layer. The "lower" layer soils are perhaps preferred because of the lower fines content as compared to the "upper" layer. A lower fines content reduces the possibility of physical clogging of the upper geotextile of the drainage layer.

Recommendations regarding protection layer thickness are commonly based on providing adequate frost protection for the underlying layers. This is especially important if a compacted clay liner is used as a component of the barrier layer (see Section 4.3.3). Because the barrier in this design proposal is composed of a geomembrane and a GCL, freeze/thaw protection is not a significant issue (Struve, 1991; Eith et al., 1991). Thus, a protection layer thickness of 18 inches (45 cm) is recommended. The borrow material used for the protection layer should be placed using a small dozer with low ground-pressure to protect the underlying cover components. Compaction beyond that which occurs during placement is not necessary.

4.5.3 **Surface Layer**

For the reasons presented in Section 4.5.1, vegetation will be used for surface cover. Consequently, the surface layer will be designed for vegetative support. McBean et al. (1995) describe the primary criterion for choosing a vegetative support topsoil as the ability of the soil to allow sufficient surface water infiltration and subsequent retaining of plant-available water to support plant growth. The factors which affect the ability of a soil to retain water are particle size distribution, structure, and organic content (McBean et al., 1995). The soil that most closely meets these requirements is a mixture of clay, silt, and sand and is termed a loam.

Prior to selection or placement, the topsoil should be tested for pH, Mg, Ca, P, NO_3, NH_4, K, Cu, Fe, Zn, Mn, conductivity, particle size distribution, density, and organic content (McBean et al., 1995). The results of these tests will indicate fertilizer requirements, with the three major fertilizer nutrients being nitrogen, phosphate, and potassium (McBean et al., 1995).

Neither the "upper" nor the "lower" layer soils of the on-site borrow area is well suited to supporting vegetation. The "upper" layer soils are generally silty sands, and the "lower" layer soils are generally sandy gravels (ABB, 1993). It is therefore recommended that loam be imported from an off-base supplier. Loam is an expensive commodity, and it is therefore recommended that it be placed to a thickness of only 6 inches (15 cm). The minimum thickness recommended by the Soil Conservation Service for revegetation is 4 inches (10 cm) (USDA, 1991). The loam should be placed with a light dozer, and it is recommended that the final surface be tracked by the dozer up and down the slope to reduce runoff water velocity.

Selection of proper vegetation is a critical step in the surface layer design process. The factors that should be considered in choosing surface layer vegetation include (US EPA, 1985):

- availability of the seed in the required quantity at the appropriate time of the year

- rapid germination and development
- ability to withstand erosive and traffic stresses
- adaptability to cover soil conditions
- adaptability to regional climatic conditions
- tolerance to landfill gases
- resistance to fire, insect damage, disease, pests
- compatibility with land management goals
- ability to self-propagate
- short and long term maintenance requirements
- depth of root penetration

The Soil Conservation Survey recommends warm season grasses as the best species for revegetation of capped landfill sites in the Northeast because: 1) coarse sandy material is often the growth medium and, 2) droughty conditions often result due to either limited rooting depth to the barrier layer or higher than normal methane concentrations (USDA, 1991). Warm season mixtures should be planted as early in the spring as possible, and before May 1 (USDA, 1991). The seed mixtures recommended for vegetation of capped landfills in the Northeast are shown in Table 4.5-1 (USDA, 1991). The recommended mixture proportions are available from the Soil Conservation Service (USDA,1991).

SEED MIXTURE	SOIL-SITE ADAPTATION		
	Excessively Drained	**Well to Moderately Well Drained**	**Poorly to Very Poorly Drained**
Creeping Red Fescue and either Redtop or Perennial Ryegrass and either Roundhead Bush Clover or Showey Thick Trefoil	-	X	-
Smooth Bromegrass and Perennial Ryegrass and either Roundhead Bush Clover or Showey Thick Trefoil	-	X	-
Switchgrass	X	X	-
Switchgrass, Big Bluestem, Little Bluestem, Sand Lovegrass, and Caucasian Bluestem	X	X	-

Table 4.5-1: Recommended Seed Mixtures (USDA, 1991)

There are various methods of applying seed to the prepared surface. The Soil Conservation Service recommends a grass drill as the most effective (USDA, 1991). The main advantage of using a grass drill is that all of the seed is placed in the soil and covered (US EPA, 1985). The seed is placed in an environment conducive to germination, thus reducing the amount of seed required (US EPA, 1985). The grass drill method of seed placement is limited to relatively smooth shallow slopes on which farm machinery can easily maneuver (US EPA, 1985).

Once seeding is completed, a mulch cover should be placed on the soil. Mulching helps to hold moisture in the soil, protect the soil from erosion, hold the seed in place, and maintain relatively constant soil temperatures (USDA, 1991). Hay or straw is the most commonly used material for mulching of newly seeded areas (USDA, 1991). The mulch should be placed as soon as possible after seeding is complete, and not later than 48 hours after seeding (USDA, 1991). Hay or straw mulch should be anchored to prevent loss by wind. Wood fiber hydromulch (a wood cellulose slurry) is commonly used in this application (USDA, 1991).

4.5.4 Soil Loss Estimation

A common method used to estimate soil loss due to erosion by water is the Universal Soil Loss Equation (USLE). The USLE predicts annual soil loss based on the product of the following factors (McBean et al., 1995):

$$A = RKLSCP$$

where:

A = soil loss (tons / acre-year)
K = soil erodibility factor
R = rainfall factor
L = slope length factor
S = slope gradient factor
C = crop - management factor
P = erosion control practice factor

The following description of the USLE factors is summarized from McBean et al. (1995).

Rainfall Factor, R: The rainfall factor accounts for the fact that soil losses are proportional to the intensity and kinetic energy of the rainfall. The value of R

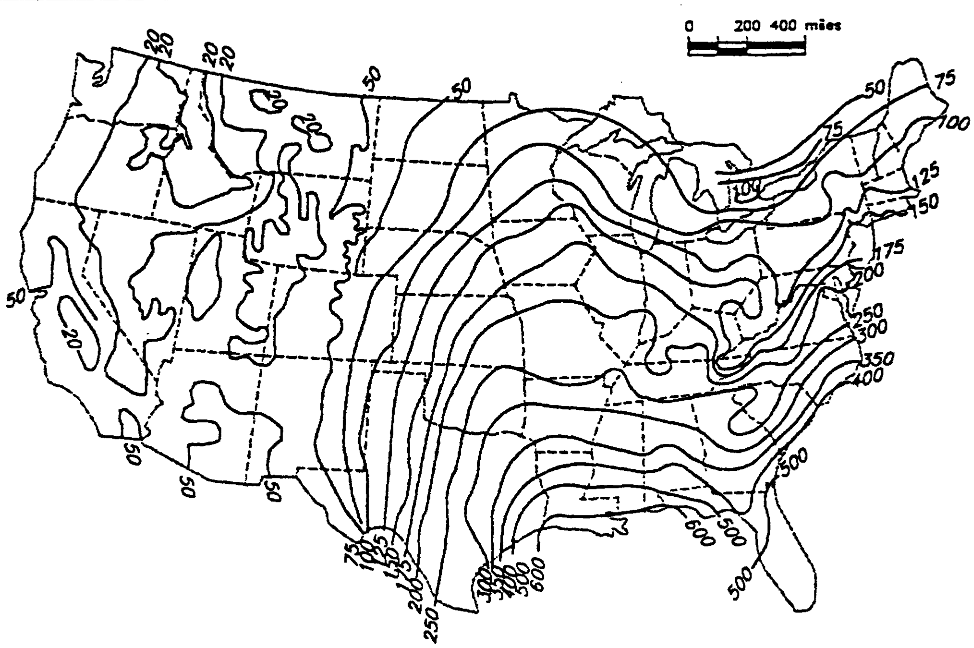


Figure 4.5-1: Average Annual Values of the Rainfall Factor (McBean et al., 1995)

used in the USLE is typically an annual average value. Average annual values of the rainfall factor are shown in Figure 4.5-1.

- Average annual R for Cape Cod ~ 140 (Figure 4.5-1)

Soil Erodibility Factor, K: Soil erodibility depends on the physical and chemical properties of the soil. The value used for this estimation will be an average value for a sandy loam. Values for the soil erodibility factor for various soil types are shown in Table 4.5-2.

- Average K for sandy loam ~ 0.30

Soil	K value	Soil	K value
Dunkirk silt loam	*0.69*	Mexico silt loam	*0.28*
Keene silt loam	*0.48*	Honeoye silt loam	*0.28*
Shelby loam	*0.41*	Cecil sandy loam	*0.28*
Lodi loam	*0.39*	Ontario loam	*0.27*
Fayette silt loam	*0.38*	Cecil clay loam	*0.26*
Cecil sandy clay loam	*0.36*	Boswell fine sandy loam	*0.25*
Marshall silt loam	*0.33*	Zaneis fine sandy loam	*0.22*
Ida silt loam	*0.33*	Tifton loamy sand	*0.10*
Mansic clay loam	*0.32*	Freehold loamy sand	*0.08*
Hagerstown silty clay loam	*0.31*	Bath flaggy silt loam	*0.05*
Austin clay	*0.29*	Albia gravelly loam	*0.03*

Table 4.5-2: Values of Soil Erodibility Factor, ***K*** (McBean et al., 1995)

Soil-Loss Ratio, SL: As slope length increases, soil loss per unit area increases due to runoff accumulation. L is used in conjunction with the slope S to graphically determine a soil-loss ratio SL.

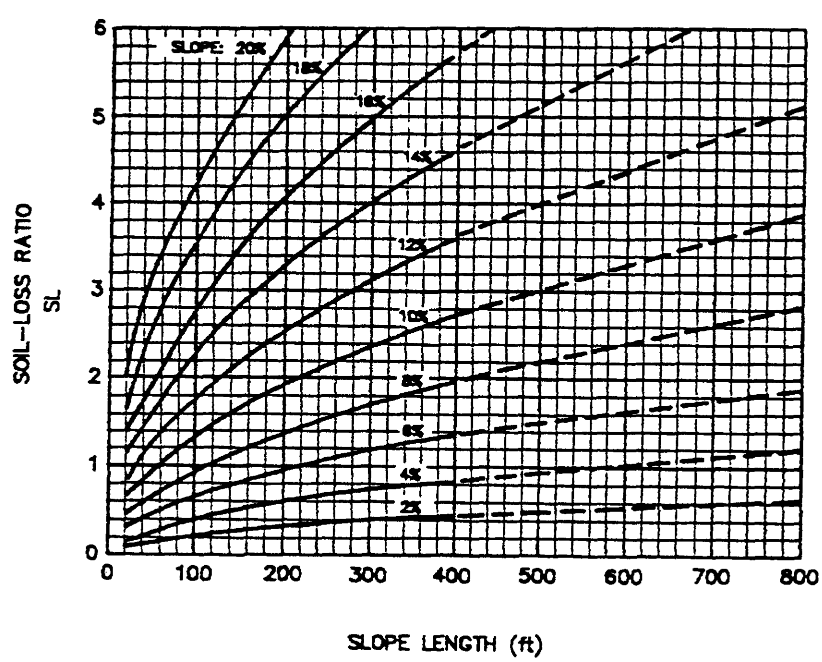


Figure 4.5-2: Soil-Loss Ratio Factor
(McBean et al., 1995)

- Using an approximate slope length of 375 ft. (ABB, April 1992) and a maximum final slope of 5%, a soil-loss ratio is taken from Figure 4.5-2
- SL ~ 1.1

Cropping-Management Factor, C: C is the ratio of soil loss from land under particular conditions relative to that from continuously fallowed land. Because the surface layer will be covered with mulch until the vegetation has germinated, a C value for moderate mulch coverage will be used in the initial calculation to provide a conservative estimate of soil loss. The value for C decreases as the vegetation matures as indicated in Table 4.5-3.

- C for surface with a moderate mulch coverage = 0.4

Land Cover	**C value**	**Land Cover**	**C value**
Continuous fallowed land		Grasses	
Bare soil	*1.0*	Newly seeded, first month	*0.6*
Mulch		Newly seeded, first year	*0.05*
Heavy	*0.2*	95-100% grass cover	*0.003*
Moderate	*0.4*	80% grass cover	*0.01*
Light	*0.6*	60% grass cover	*0.04*

Table 4.5-3: Values for Crop-Management Factor, C (McBean et al., 1995)

Erosion-Control Factor, P: For landfills, the factor P is similar to C except that it accounts for additional land-management practices that are intended to reduce erosion. A value of 1.0 is commonly used for landfill surfaces as shown in Table 4.5-4

- P = 1.0

Erosion Control Practice	P Value
Compact, smooth surface with no cover	*1.30*
Landfill surface	*1.00*
Small sediment basins	*0.90*
Rough, irregular surface	*0.50*

Table 4.5-4: Values of Erosion Control Factor, *P*
(McBean et al., 1995)

SOIL-LOSS CALCULATION:

A_1 = RKLSCP = 140 x 0.30 x 1.1 x 0.4 x 1.0 = 18.5 tons/acre-year

This value is well in excess of the EPA's maximum recommended value of 2 tons/acre-year (US EPA, 1991). The reason the estimation is so high is that it is based on a mulched surface rather than a vegetated surface (see crop-management factor above). As the vegetation is established, the crop-management factor decreases as shown in Table 4.5-3. The factor *C* reduces to 0.6 for the first month after seeding, and 0.05 for later in the first year after seeding. The following calculations estimate soil loss using these values.

A_2 = RKLSCP = 140 x 0.30 x 1.1 x 0.6 x 1.0 = 28 tons/acre-year

A_3 = RKLSCP = 140 x 0.30 x 1.1 x 0.05 x 1.0 = 2.3 tons/acre-year

The empirical estimate from the USLE continues to decrease as the vegetative cover becomes more complete. This is evidenced by the reduction in the factor C of Table 4.5-3. Calculation A_3 nearly meets the EPA recommended maximum annual soil loss using a value of 0.05 for C. Table 4.5-3 indicates a value of 0.01

for *C* with 80% grass coverage. Using this value, the empirical estimate provided by the USLE is well below the EPA recommended maximum. Table 4.5-5 presents the factors and results of the USLE trials. The most accurate calculation is a time weighted sum based on an estimate of the rate of establishment of the vegetated cover.

	USLE FACTORS					
Trial	**R** Rainfall Factor	**K** Soil Erodibility	**SL** Soil-Loss Ratio	**C** Crop-Management	**P** Erosion Control	**A** Soil Loss (tons/acre-yr.)
Mulched surface, no crop	140	0.30	1.1	0.4	1.0	18.5
Newly seeded, first month	140	0.30	1.1	0.6	1.0	28
Newly seeded, first year	140	0.30	1.1	0.05	1.0	2.3
Ground cover 80% grass	140	0.30	1.1	0.01	1.0	0.5

Table 4.5-5: Soil Loss Calculations

5.0 *PERIMETER DRAINAGE*:

A perimeter drainage system is necessary to handle surface water runoff in a controlled manner that does not result in further erosion. The design of a perimeter drainage system requires consideration of the following factors (US EPA, 1985):

- stormwater capacity requirements
- flow velocity
- channel cross section
- land availability
- channel lining
- maintenance requirements
- outlet conditions
- cost

A preliminary step in designing a perimeter drainage system is to estimate the amount of surface water runoff that must be handled. Runoff values from water balance programs such as HELP (see Appendix A) could be used, however, such programs provide runoff values based on daily rainfall levels (McBean et al., 1995). This assumes that the rainfall intensity is constant over a 24 hour period, thus underestimating the intensity of short duration rainfall. Such an assumption will result in an underestimation of runoff.

An alternative means of calculating runoff is the rational method. The rational method is a mathematical formulation commonly used for storm sewer design (McBean et al., 1995). This method is based on the following equation (McBean et al., 1995).:

$$Q = CiA$$

where:

Q = flow in ft^3/sec

C = dimensionless runoff coefficient (See Table 5-1)

i = rainfall intensity (in./hr.)

A = contributing drainage area (acres)

	Surface Soil Type		
Topography and Vegetation	**Open Sand Loam**	**Clay and Silt Loam**	**Tight Clay**
Woodland			
Flat, 0 - 5% slope	*0.10*	*0.30*	*0.40*
Rolling, 5 - 10% slope	*0.25*	*0.35*	*0.50*
Hilly, 0 - 30% slope	*0.30*	*0.50*	*0.60*
Pasture			
Flat	*0.10*	*0.30*	*0.40*
Rolling	*0.16*	*0.36*	*0.55*
Hilly	*0.22*	*0.42*	*0.60*
Cultivated			
Flat	*0.30*	*0.50*	*0.60*
Rolling	*0.40*	*0.60*	*0.70*
Hilly	*0.52*	*0.72*	*0.82*

Table 5-1: Runoff Coefficients for the Rational Formula (McBean et al., 1995)

inserting the following values:

C ~ 0.40 (conservative estimate from Table 5-1)

i = 4 in./hr. (50 yr. 1 hr. duration storm from Northeast Regional Climate Center, 1995.)

A ~ 9 acres (ABB, April 1992)

$\Rightarrow$ Q = 0.40 x 4 x 9 = 14 cfs

The most common channel cross sections for relatively large volumes and relatively high velocities are trapezoidal and rectangular (US EPA, 1985). Trapezoidal is typically preferred over rectangular because of its increased sidewall stability (US EPA, 1985). Channels may be lined with various materials to mitigate erosion. The estimated maximum permissible velocities for vegetated channels is presented in Table 5-2.

COVER	PERMISSIBLE VELOCITY (fps) Erosion Resistant Soils (% slope)			Easily Eroded Soils (% slope)		
	0-5	5-10	>10	0-5	5-10	>10
Bermuda grass	8	7	6	6	5	4
Buffalo grass Kentucky bluegrass Smooth brome Blue grama Tall fescue Reed canarygrass	7	6	5	5	4	3
Lespedeza sericea Weeping lovegrass Alfalfa Crabgrass Redtop Red fescue	3.5	NR[1]	NR	2.5	NR	NR
Grass Mixture	5	4	NR	4	3	NR

Notes: 1. NR = Not Recommended

Table 5-2: Permissible Velocities for Vegetated Channels (US EPA, 1985)

The design of a perimeter drain is very dependent on actual site construction details which are not being addressed here. However, an example of a feasible design procedure is provided for illustrative purposes. The design procedure and tables are drawn from US Department of Agriculture, 1986.

Using Q ~ 15 cfs (from rational method) and the channel details:

- trapezoidal cross section
- side-slopes = 3:1
- channel grade = 2%
- grass mixture lined, good stand 6-12 inches high = Retardance C (See Table 5-3)
- maximum permissible velocity ~ 3.5 fps (See Table 5-2)

  ⇒ Entering Figure 5-1, with velocity = 3.5 fps and a slope of 2%, a hydraulic radius of 0.72 ft results.

  ⇒ Entering Figure 5-2 with a hydraulic radius of 0.72 ft, and an area A = Q/V = 15 cfs/3.5 fps = 4.3 ft^2, a bottom width of approximately 1.5 ft and a depth of approximately 1.3 ft result.

To summarize, it is estimated that a trapezoidal channel with 3:1 side slopes, a 1.5 foot bottom width, and a 2% bottom grade, lined with a good stand of grass, will transport 15 cfs of runoff at a velocity of 3.5 fps and a depth of 1.3 feet.

Cover	Stand	Condition	Retardance
Reed canarygrass	Excellent	Tall (avg. 36 in.)	*A*
Kentucky 31 tall fescue	Excellent	Tall (avg. 36 in.)	
Tufcote, Midland and Coastal bermudagrass	Good	Tall (avg. 12 in.)	
Reed canarygrass	Good	Mowed (avg. 12-15 in.)	
Kentucky 31 tall fescue	Good	Unmowed (avg. 18 in.)	*B*
Red fescue	Good	Unmowed (avg. 16 in.)	
Kentucky bluegrass	Good	Unmowed (avg. 16 in.)	
Redtop	Good	Average	
Kentucky bluegrass	Good	Headed (6 to 12 in.)	
Red fescue	Good	Headed (6 to 12 in.)	
Tufcote, Midland and Coastal bermudagrass	Good	Mowed (avg. 6 in.)	C
Redtop	Good	Headed (15 to 20 in.)	
Tufcote, Midland and Coastal bermudagrass	Good	Mowed (2.5 in.)	*D*
Red fescue	Good	Mowed (2.5 in.)	
Kentucky bluegrass	Good	Mowed (2 - 5 in.)	

Table 5-3: Classification of Vegetative Cover in Waterways Based on Degree of Flow Retardance by the Vegetation (US EPA, 1985)

ABB Environmental Services (1993) explains that the runoff from the three capped cells (1970, post-1970, and Kettle Hole) is diverted to two separate recharge areas. The first recharge area is a detention basin in the southwest corner of LF-1 (see Figure 1-1). The second recharge area was previously a gravel pit, and is located just south of the Post-1970 cell (see Figure 1-1). ABB Environmental Services (1993) reports that the ditches and culverts currently in place were designed and sized to handle future flows from NOU cells if capped in the future. Additionally, runoff from a portion of the NOU cells was modeled to ensure that the borrow-pit recharge area had sufficient capacity to handle this flow if the cells were capped in the future (ABB, 1993). Referring to Figure 1-1 (ABB, June 1992), it seems most reasonable that runoff flow from the 1951 Cell perimeter drainage system be diverted to the culvert between the Kettle Hole and the Post-1970 Cell which discharges into the borrow-pit recharge area.

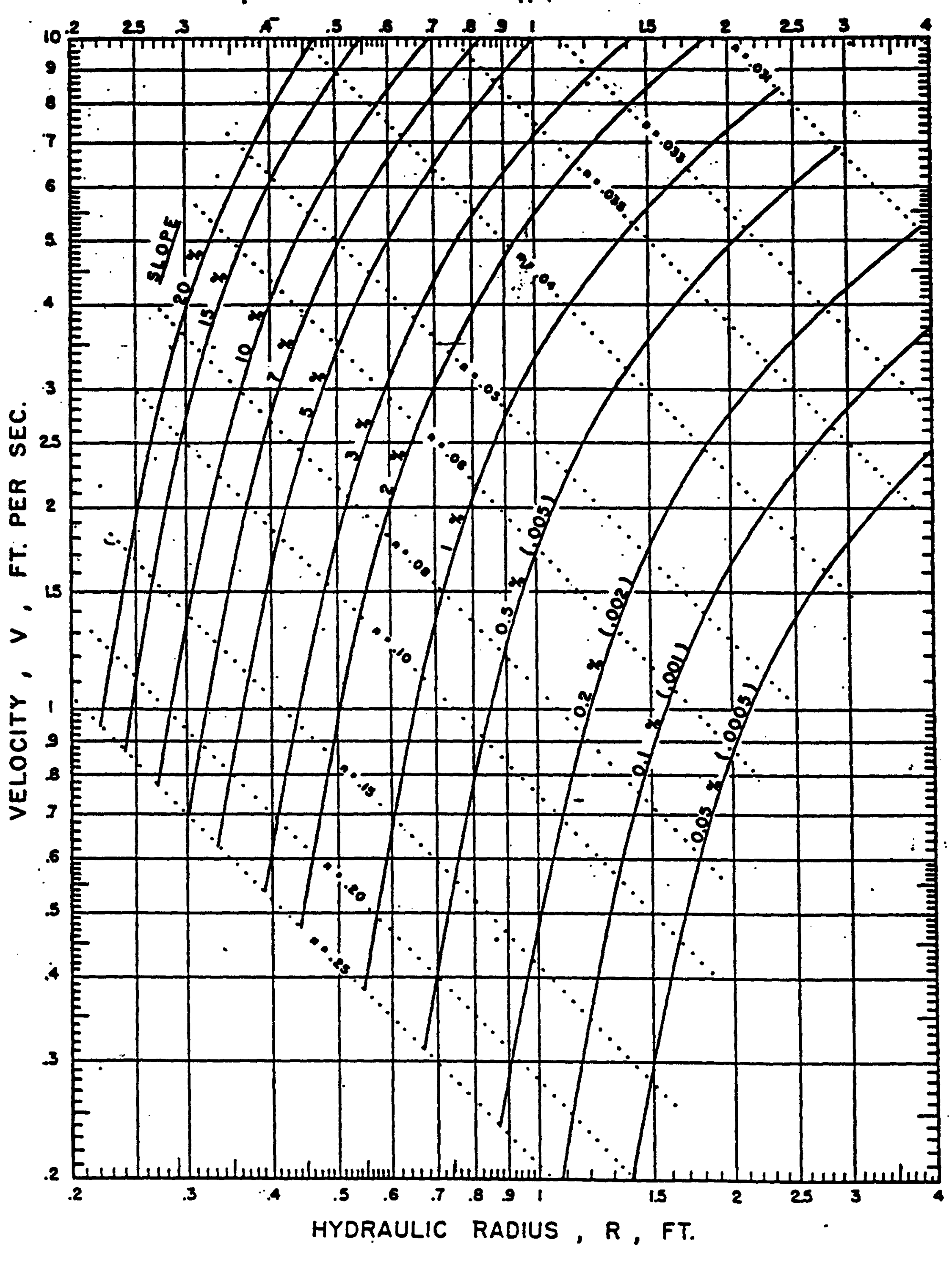


Figure 5-1: Solution of the Manning Formula for Retardance C (USDA, 1986)

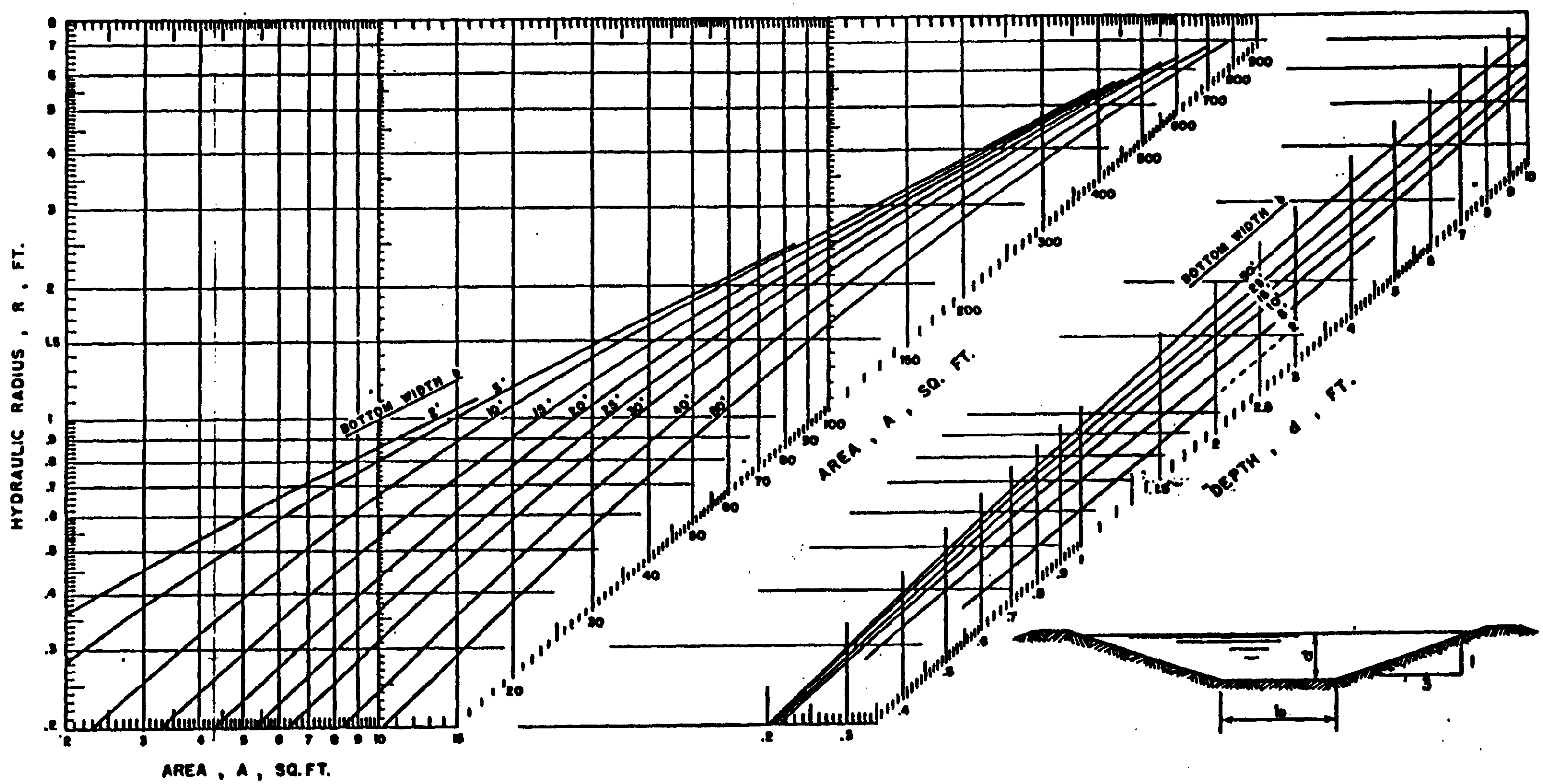


Figure 5-2: Dimensions of Trapezoidal Channel with 3 to 1 Side Slopes (USDA, 1986)

6.0 *SETTLEMENT*

The total settlement of solid waste landfills may be as much as 25 to 50 percent of the original thickness of the waste (Bjangard and Edgers, 1990). Several factors affect the magnitude of settlement, including: the composition of the refuse, refuse density, refuse layer thickness, overburden weight, the amount of moisture and oxygen that reach the waste, and the temperature within the waste layer (Bjangard et al., 1990).

6.1 Settlement Mechanisms

As described by Murphy and Gilbert (1985), there are three basic mechanisms of settlement in a landfill:

1. Mechanical Compression: compression caused by the self-weight of the waste and surcharge loads.
2. Ravelling: movement of fines into the larger voids of the waste fills.
3. Decomposition: deterioration of waste by corrosion, oxidation, combustion, or decay.

These mechanisms are described in detail by Bjangard and Edgers (1990). The following descriptions are summarized from that report.

Mechanical compression occurs in stages. The first stage, initial compression, begins upon loading. Initial compression is the result of the reduction of void space in the waste, compression of loose materials, and waste reorientation and lateral expansion. The next phase of mechanical compression is termed primary consolidation. Primary consolidation is caused by the squeezing out of moisture from void spaces in the waste. The magnitude of, and time frame for, primary consolidation is dependent on the void ratio and degree of saturation of the waste fill. The final phase of mechanical compression occurs due to long-term reorientation of particles and delayed compression of waste materials. This phase is termed secondary compression (Bjangard and Edgers, 1990).

Ravelling is the migration of fine particles into larger voids within the waste fill. This process can occur suddenly or over an extended period of time, and can result in localized, irregular settlement. Ravelling is difficult to distinguish from the other settlement mechanisms (Bjangard and Edgers, 1990).

Decomposition of organic wastes in landfills may occur by either biological or chemical processes. The decomposition typically begins as an aerobic process, but quickly turns anaerobic as oxygen is depleted. Decomposition processes may continue in excess of forty years, and are strongly dependent on the conditions within the waste fill. The factors which most strongly affect the rate of decomposition are: moisture content, waste composition, pH, temperature, nutrient content, and toxic substance content (Bjangard and Edgers, 1990).

Having analyzed settlement data from numerous landfill sites, Bjangard and Edgers (1990) describe the settlement process as occurring in three phases. In the initial phase, compression is thought to occur mainly from compression of the refuse and reduction of gas void spaces. The parameter used to quantify initial compression is termed the compression ratio (CR). The two subsequent phases are collectively termed delayed compression. In the early stage of delayed compression, settlement is thought to be dominated by mechanical mechanisms including reorientation and slippage. In the later stage of delayed compression, the settlement rates are believed to be higher because of the effects of decomposition. The parameters used to quantify the two stages of delayed compression are $C_{\alpha min}$ and $C_{\alpha max}$, the minimum and maximum delayed (secondary) compression coefficients (Bjangard and Edgers, 1990). These concepts are presented graphically in Figure 6-1.

6.2 **Site-Specific Conditions**

In applying these concepts to the 1951 cell, several factors must be considered. The 1951 cell is composed primarily of burn-fill and has existed for nearly 45 years with only a thin layer of intermediate cover separating waste from atmosphere (Weston, 1985). Two conclusions can be drawn from this information. First, because the refuse has been burned, the majority of organic material has been oxidized by fire precluding further degradation by microbial action. Second, the cell has only a thin intermediate cover of permeable soil, which has allowed relatively large amounts of air and water to infiltrate into the waste. The infiltration of water and air enhances the decomposition process, and

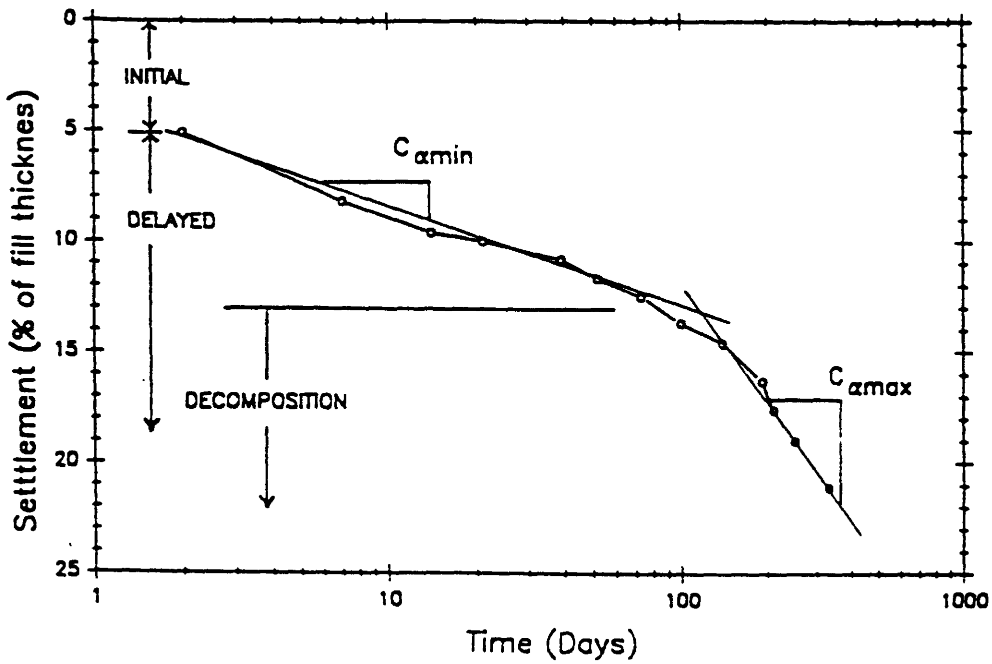


Figure 6-1: Idealized Plot of Landfill Settlement Versus Log-Time (Bjangard et al., 1990)

thus it seems likely that any material not oxidized by burning has already been biologically degraded. Based on the cell composition (predominantly burn-fill), the moist, aerobic conditions provided by the intermediate cover, and the time since placement (over 40 years), it is anticipated that the waste of the 1951 cell will undergo very little degradation by biological or chemical means.

6.3 Settlement Calculations

As mentioned previously, the difference between the minimum and maximum delayed compression coefficients is the added settlement resulting from decomposition. Because little decomposition is anticipated in the 1951 cell, it is appropriate to combine the two delayed compression coefficients ($C_{\alpha min}$ and $C_{\alpha max}$) into one (C_{α}). This modified version of the Bjangard and

Edgers (1990) settlement model was proposed by Fasset et al. (1994) and applied in a case study by Stulgis et al. (1995):

$$\Delta H = H \cdot CR \cdot \log \frac{\overline{P_0} + \Delta P}{\overline{P_0}} + H \cdot C_\alpha \cdot \log \frac{t_2}{t_1} \quad \text{(Stulgis et al., 1995)}$$

where:

ΔH ≡ settlement
H ≡ initial waste layer thickness
CR ≡ compression ratio
$\overline{P_0}$ ≡ initial average vertical stress (at center of waste layer)
ΔP ≡ average induced vertical stress
C_α ≡ delayed (secondary) compression coefficient
t_1 ≡ time for completion of initial compression
t_2 ≡ period of time for prediction of settlement

The use of a single delayed (secondary) compression coefficient in certain cases is supported by the data collected by Bjangard and Edgers (1990) for older (i.e., 1935 to 1955) landfills. These older landfills did not exhibit the pronounced slope discontinuity in the delayed compression portion of the settlement curve as shown in Figure 6-1. The data collected by Bjangard and Edgers (1990) for three older landfill sites in New England are presented in Table 6-1. The first two of these landfills had only one listed delayed compression coefficient, and the third exhibited only a slight difference between the minimum and maximum coefficients, hence only one delayed compression coefficient is presented, with the third case presented as a range.

Table 6-1: Landfill Case Histories (Bjangard and Edgers, 1990)

State	Period of Operation	Refuse Type	Waste Thickness	Surcharge (lb/ft^2)	CR	C_α
E-MA	Mid 1950's	Misc.[1]	15 ft.	720	0.26	0.024
E-MA	1935 - 1945	Misc.	20 ft.	2000	0.15	0.10
CT	1930 - ?	MSW[2]	38 ft.	750	---[3]	0.014 - 0.019

Notes: 1. Misc. = Miscellaneous solid waste
2. MSW = Municipal solid waste
3. Insufficient data

The sites presented in Table 6-1 operated during the same time period as the 1951 cell, and are located in the same region of the country, therefore it is assumed that the waste disposal practices were similar to those at the 1951 cell. Thus, approximate averages of the compression ratios and delayed compression coefficients in Table 6-1 will be used for the 1951 cell settlement calculations.

In the following calculations, the settlement equation is separated into initial (ΔH_1) and delayed (ΔH_2) components for clarity. Calculations will be made at the center of the cover (where the surcharge stress is greatest) and at the edge of the cover (where the surcharge stress is lowest). The difference between these calculated settlements will provide an estimate of anticipated differential settlement. The estimated time period for completion of initial compression (t_1) is 10 days (Bjangard and Edgers, 1990) and the time period for prediction of settlement (t_2) is 10,000 days (Bjangard and Edgers, 1990).

I. Calculation of Settlement at Center of Cover

1. Calculate existing stress at center of waste layer $\equiv \overline{P_0}$

Assumptions:

- Waste density = 65 lb/ft^3 (Sharma, 1994)
- Average waste layer (trench) thickness = 8 ft. (Metcalf and Eddy, 1983)
- 2 ft. of soil cover currently over waste (Metcalf and Eddy, 1983)
- Soil density = 110 lb/ft^3

$$\Rightarrow \overline{P_0} \approx \left(4 \text{ ft} \times \frac{65 \text{ lb}}{\text{ft}^3}\right) + \left(2 \text{ ft} \times \frac{110 \text{ lb}}{\text{ft}^3}\right) = 480 \frac{\text{lb}}{\text{ft}^3}$$

2. Calculate surcharge stress at center of waste layer $\equiv \Delta P$

surcharge = foundation layer + gas venting layer + surface layer

Assumptions:

- Weight of geosynthetics is negligible
- Soil density = 110 lb/ft^3
- Estimated longest slope = 375 ft. (ABB, April 1992)
- Foundation layer thickness ~ 20 ft. (375 ft long slope at 5% = 19 ft vertical drop)

$$\Rightarrow \Delta P \approx \left(20 \text{ ft} \times \frac{110 \text{ lb}}{\text{ft}^3}\right) + \left(1 \text{ ft} \times \frac{110 \text{ lb}}{\text{ft}^3}\right) + \left(2 \text{ ft} \times \frac{110 \text{ lb}}{\text{ft}^3}\right) = 2530 \frac{\text{lb}}{\text{ft}^3}$$

3. Calculate initial compression:

Assumptions:

- CR = 0.20 (Bjangard and Edgers, 1990)

$$\Rightarrow \Delta H_1 = H \cdot CR \cdot \log \frac{\overline{P_0} + \Delta P}{\overline{P_0}}$$

$$\Rightarrow \Delta H_1 = 8 \text{ ft} \times 0.20 \times \log \frac{480 + 2530}{480} = 1.3 \text{ ft.}$$

4. Calculate delayed compression:

Assumptions:

- $C_\alpha = 0.017$ (Bjangard and Edgers, 1990)
- $t_1 = 10$ days (Bjangard and Edgers, 1990)
- $t_2 = 10{,}000$ days (Bjangard and Edgers, 1990)

$$\Rightarrow \Delta H_2 = H \cdot C_\alpha \cdot \log \frac{t_2}{t_1}$$

$$\Rightarrow \Delta H_2 = 8 \text{ ft} \times 0.017 \times \log \frac{10{,}000 \text{ days}}{10 \text{ days}} = 0.4 \text{ ft.}$$

$$\Rightarrow \Delta H_{total} \text{ at center of cover} = \Delta H_1 + \Delta H_2 = 1.3 \text{ ft} + 0.4 \text{ ft} = 1.7 \text{ ft.} \approx 2 \text{ ft.}$$

II. Calculation of Settlement at Edge of Cover

1. Existing stress at center of waste layer is same as above = 480 lb/ft^3

2. Calculate surcharge stress at center of waste layer $\equiv \Delta P$

surcharge = foundation layer + gas venting layer + surface layer

Assumptions:

- Weight of geosynthetics is negligible
- Soil density = 110 lb/ft^3

- minimum foundation layer thickness = 2 ft. (existing soil cover)

$$\Rightarrow \Delta P \approx \left(2\text{ ft} \times \frac{110\text{ lb}}{\text{ft}^3}\right) + \left(1\text{ ft} \times \frac{110\text{ lb}}{\text{ft}^3}\right) + \left(2\text{ ft} \times \frac{110\text{ lb}}{\text{ft}^3}\right) = 550\ \frac{\text{lb}}{\text{ft}^3}$$

3. Calculate initial compression:

Assumptions:

- CR = 0.20 (Bjangard and Edgers, 1990)

$$\Rightarrow \Delta H_1 = H \cdot CR \cdot \log \frac{\overline{P_0} + \Delta P}{\overline{P_0}}$$

$$\Rightarrow \Delta H_1 = 8\text{ ft} \times 0.20 \times \log \frac{480 + 550}{480} = 0.5\text{ ft.}$$

4. Calculate delayed compression:

Assumptions:

- C_α = 0.017 (Bjangard and Edgers, 1990)
- t_1 = 10 days (Bjangard and Edgers, 1990)
- t_2 = 10,000 days (Bjangard and Edgers, 1990)

$$\Rightarrow \Delta H_2 = H \cdot C_\alpha \cdot \log \frac{t_2}{t_1}$$

$$\Rightarrow \Delta H_2 = 8\text{ ft} \times 0.017 \times \log \frac{10{,}000\text{ days}}{10\text{ days}} = 0.4\text{ ft.}$$

$\Rightarrow \Delta H_{total}$ at edge of cover = $\Delta H_1 + \Delta H_2 = 0.5\text{ ft} + 0.4\text{ ft} = 0.9\text{ ft.} \approx 1\text{ ft.}$

III. <u>Estimation of Anticipated Differential Settlement</u>

Estimated differential settlement = Δ H at center of cover - Δ H at edge of cover

Estimated differential settlement ≈ 2 ft. - 1 ft. = 1 ft.

Estimated differential settlement ≈ 2 ft. - 1 ft. = 1 ft.

Resulting slope decrease:

- Estimated shortest slope = 275 ft. (ABB, April 1992)
- Differential settlement = 1 ft.

$$\Rightarrow \Delta\,\text{Slope} = \frac{1\text{ ft.}}{275\text{ ft.}} \times 100 = 0.4\%$$

$$\Rightarrow \text{Factor of Safety} = \frac{\text{anticipated minimum slope}}{\text{min imum allowable slope}} = \frac{5\% - 0.4\%}{3\%} = 1.53 \Rightarrow \text{O.K.}$$

It should be noted that localized differential settlement may exceed the calculated value of 1 foot. Localized differential settlement, which might result from events such as the collapse of a metal drum due to corrosion or compression, could cause a relatively large differential settlement over a short distance.

7.0 *STATIC AND SEISMIC SLOPE STABILITY*

7.1 Static Stability Analysis

The method used to analyze the static stability of this cover system is a limit equilibrium method termed infinite slope analysis. Infinite slope analyses are one-directional and consider movement parallel to the slope (Sharma et al, 1994). This situation arises when a thin soil veneer is placed on a slope, such is the case for a landfill cover system. The infinite slope analysis assumes the slope is of infinite length, and that the width normal to the cross section is much wider than the thickness (Sharma et al, 1994). In the following analysis, it is also assumed that the cover soil is cohesionless (an accurate assumption for dry sand) and is of uniform thickness (an accurate assumption for this cover system). Applying these assumptions, a force summation along the slope can be written, and the following factor of safety derived (Koerner, 1995):

$$FS = \frac{\Sigma \text{ resisting forces}}{\Sigma \text{ driving forces}}$$

The forces resisting motion are due to the material strength, while the forces driving movement are due to the weight of the materials (see Figure 7-1) (Sharma et al, 1994). Thus, the factor of safety may be written as follows (Sharma et al, 1994):

$$FS = \frac{W \cos\beta \ \tan\delta}{W \sin\beta} = \frac{\tan\delta}{\tan\beta}$$

where:

W = weight (stress) of soil = soil density x thickness

β = the slope angle

δ = the interface friction angle

The static slope stability safety factors are presented in Table 7-1.

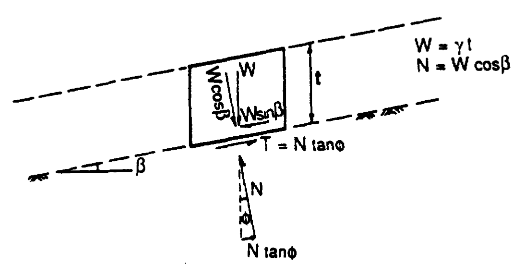


Figure 7-1: Infinite Slope Force Summation (Sharma et al., 1994)

7.2 Seismic Stability Analysis

The method presented here for seismic slope stability analysis is a pseudo-static method. This method utilizes an empirical seismic coefficient, k_s (Richardson et al., 1994). Richardson et al. report that, based upon extensive research on the topic, it has been determined that k_s is most reasonably calculated as:

$$k_s = 0.5\ a_{max}$$

where: a_{max} = peak horizontal acceleration in bedrock expressed as a percentage of *g* (acceleration due to gravity

The factor a_{max} may be determined using charts of seismic impact zones as shown in Figure 7-2 (Sharma et al., 1994). "Seismic impact zones refer to areas with a 10 percent or greater probability that the maximum horizontal acceleration in lithified earth material, expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10*g* in 250 years" (Sharma et al., 1994, p. 560). The seismic impact zone chart provides an a_{max} value for bedrock, which must then be converted to an a_{max} value for soft soil. This conversion may be accomplished graphically using Figure 7-3 (Richardson et al., 1994). Figure 7-3 provides a_{max} values for soft soil as a fraction of *g*, this value is then divided by two to determine k_s as described above. To summarize:

1. Determine a_{max} geographically from seismic impact zone chart (Figure 7-2)

2. Convert a_{max} for bedrock from step 1. into a_{max} for soft soil using Figure 7-3

3. Calculate $k_s = 0.5 \times a_{max}$ (soft soil)

4. Use k_s in the following equation to determine seismic safety factor (Richardson et al., 1994):

$$FS = \frac{\tan\delta \left(1 - k_s \cdot \tan\beta\right)}{k_s + \tan\beta}$$

where:

β = the slope angle
δ = the interface friction angle
k_s = seismic stability coefficient

Seismic Factor of Safety Calculations:

1. a_{max} for bedrock on Cape Cod ~ $0.30g$ (Figure 7-2)
2. a_{max} for soft soil ~ $0.36g$ (Figure 7-3)
3. $k_s = 0.5(0.36g) = 0.18g$
4. FS calculation results presented in Table 7-1

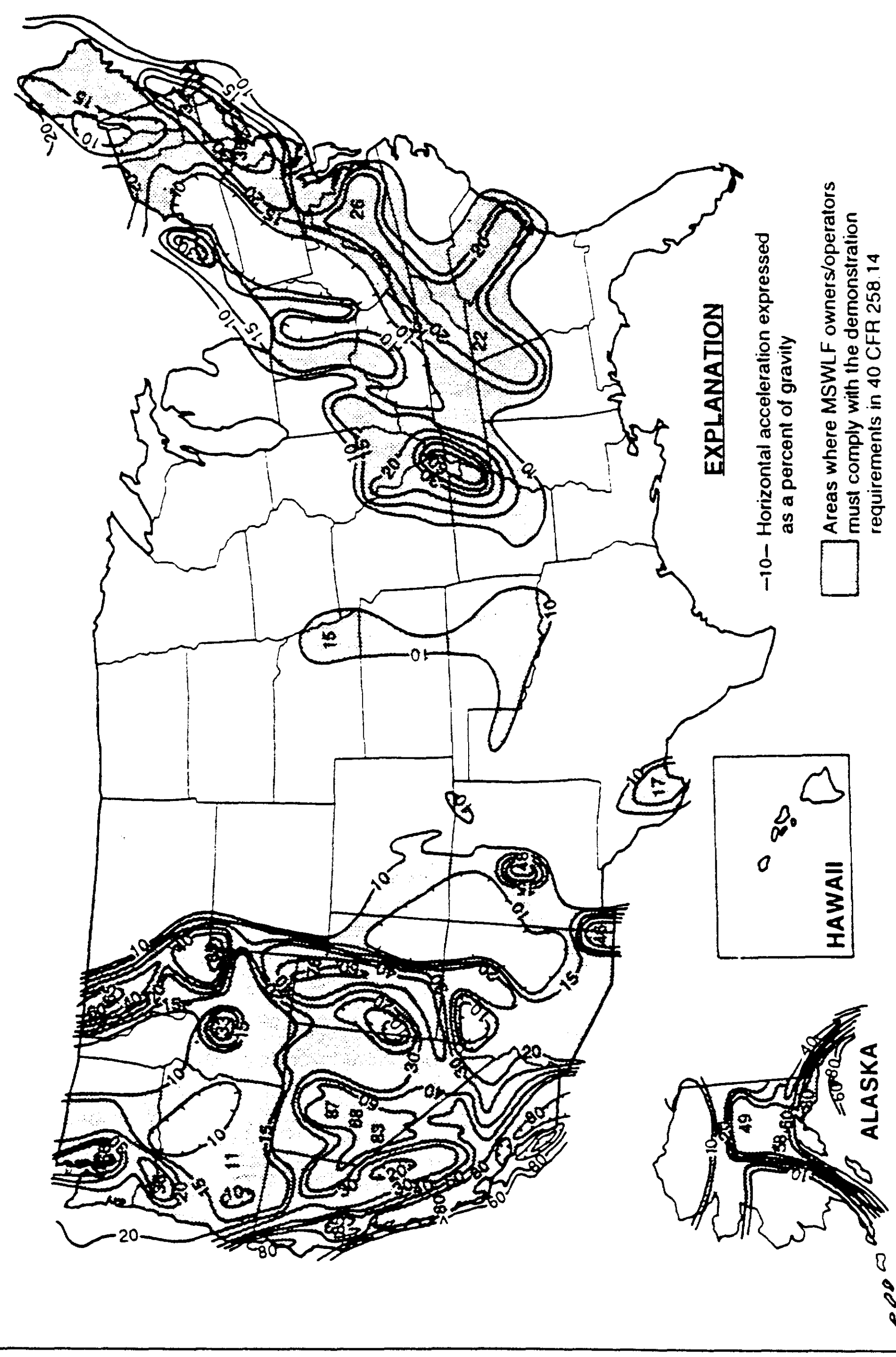


Figure 7.2: Seismic Impact Zones (Sharma et al., 1994)

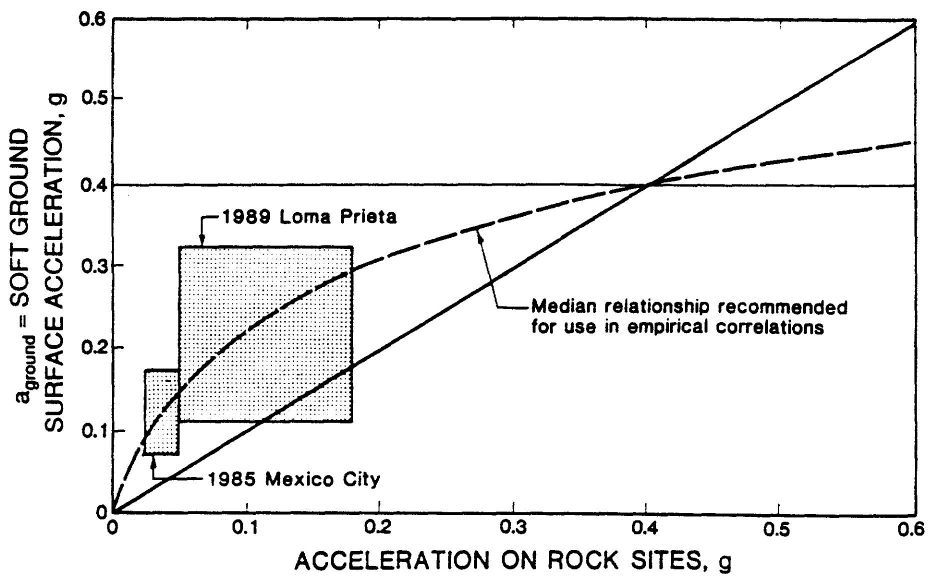


Figure 7.3: Variations of Acceleration on Soft Soil Sites vs. Rock Sites (Sharma et al., 1994)

Table 7-1: Static and Seismic Safety Factors

Interface	Interface Friction Angle (δ)	FS_{static}	k_s	$FS_{seismic}$
sand/GT[1]	26° (Koerner, 1990)	9.8	0.18	1.8
GT/T-GM[2]	15° (Giroud, 1990)	3.1	0.18	1.0
GT/S-GM[3]	11° (Cadwallader, 1991)	2.22	0.18	0.72
S-GM/GCL[4]	16° (US EPA, 1993)	5.5	0.18	1.06
T-GM/GCL	32° (US EPA, 1993)	11.9	0.18	2.3
S-GCL[5]/sand	16° (US EPA, 1993)	5.5	0.18	1.06
T-GCL[6]/sand	25° (US EPA, 1993)	9.3	0.18	1.72

Notes:

1. GT = nonwoven geotextile
2. T-GM = textured (rough) geomembrane
3. S-GM = smooth geomembrane
4. GCL = geosynthetic clay liner
5. S-GCL = GCL with smooth polyethylene substrate
6. T-GCL = GCL with textured (rough) polyethylene substrate

The interface friction angles quoted above are conservative in that they represent the low end of the widely varying range found in the literature. It is highly recommended that shear tests be performed with the actual materials to be used for more accurate stability calculations. The typical recommended range for static safety factors is 1.3 to 1.5 (Sharma et al., 1994). The recommended safety factor for seismic analysis is 1.0 (proof of survivability) (US EPA, 1993). The results in Table 7-1 show that, while a smooth membrane and GCL substrate provide a sufficient safety factor against slope failure under static conditions, the textured membrane and GCL substrate provide more acceptable safety factors under seismic conditions.

8.0 *POSTCLOSURE PROGRAMS*

8.1 Postclosure Monitoring

The key parameters that must be monitored after completion of cover construction are (US EPA, 1991):

- groundwater quality
- gas concentrations (air quality)
- differential settlement (see Section 8.2)
- surface erosion (see Section 8.2)

The regulatory requirements for postclosure monitoring vary from 30 years for RCRA wastes to 500 years for mixed wastes (US EPA, 1991). The actual monitoring period should be influenced by the stability and toxicity characteristics of the waste and by the stability of the cover system (US EPA, 1991).

8.1.1 Groundwater Monitoring

Groundwater monitoring should include monitoring of water quality and the groundwater potentiometric surface (US EPA, 1991). It is critical that background quality and potentiometric data be collected prior to closure. This data will serve as a reference against which postclosure water quality and potentiometric data can be compared. This comparison will serve as an indicator of cover system performance and changes in the groundwater flow regime. As part of remedial investigation work, a well fence has already been installed downgradient of the

Northwest Operable Unit of the landfill (Stone & Webster, 1996). It is recommended that these wells serve as the sampling points for postclosure groundwater monitoring. An initial postclosure sampling frequency of quarterly is recommended. This frequency can be adjusted as necessary depending on parameter trends and data requirements (US EPA, 1991).

8.1.2 **Air Quality Monitoring**

A passive gas venting system has been recommended in this design (see Section 4.2). Such a system vents any gas collected by the gas ventilation layer to the atmosphere. It is therefore important that gas levels near the cover surface and around the cell perimeter be monitored to ensure that gas levels are not exceeding Clean Air Act maximums and to ensure that gas is not migrating off-site. Sampling techniques may include passive samples obtained using collection media, grab samples in evacuated vessels, or active pump and filter samples (US EPA, 1991). An explosion meter is commonly used for methane detection, while more accurate analyses require laboratory analysis of samples (McBean et al., 1995). Gas sampling frequency is dependent on: (1) estimated generation rates, (2) cover system characteristics, (3) the proximity of structures, and (4) the levels of gas found during sampling (McBean et al., 1995).

8.2 Postclosure Maintenance

The primary postclosure maintenance requirement is the repair and upkeep of the vegetative cover and the surface layer. All cover systems that use a vegetative cover must have an annual inspection and repair program (US EPA, 1991). Until the vegetation is well established, the focus of the inspection and repair program is likely to be the repair of erosion damage and revegetation of affected areas. Erosion repair may include replacement of cover soil, regrading of rutted areas, reseeding and mulching, and removal of eroded soil from the perimeter drainage ditches. Once the vegetative cover is well established, the focus of the program will shift to mowing requirements and suppression of undesirable (especially woody or deep-rooted) vegetation species.

Another important aspect of postclosure maintenance is the repair of damage caused by differential settlement. If actual differential settlement exceeds the design value by a large enough margin, cover system damage could occur. The damage may be so severe that it results in damage to the barrier layer components, or it may be so slight that it simply results in small depressions on the cover surface. Damage from differential settlement should be investigated during the annual inspections mentioned previously. A recommended method for detecting damage due to differential settlement is to inspect the cover surface after a rainfall noting any puddles or pooling of water (US EPA, 1991).

9.0 *CONCLUSIONS*

It is concluded that this cover system, if constructed with appropriate construction quality assurance / quality control, will satisfy the primary objective of containing the source of pollution, thereby minimizing further contamination of groundwater by the waste fill. The composite geomembrane / geosynthetic clay liner barrier layer is theoretically very-nearly impermeable. Estimates of the hydraulic conductivity of VLDPE geomembranes are on the order of 1×10^{-10} cm/sec (Koerner, 1994), and estimates of the hydraulic conductivity of Gundseal® GCLs are on the order of 1×10^{-12} cm/sec (Eith et al., 1991). Essentially all infiltration that does occur through such a composite barrier is the result of defects from manufacturing and / or construction processes. Theoretical performance of the cover was evaluated using the Hydrologic Evaluation of Landfill Performance (HELP) computer model (Schroeder et al., 1994), the results of the simulation are included as Appendix A. HELP is a quasi-two-dimensional, deterministic, water-routing model for determining water balances (Schroeder et al., 1994). HELP predicted 0.000000 inches of annual percolation through the barrier layer. Clearly, this prediction is unrealistic as no cover is absolutely impermeable. Because the performance of the cover system is so closely linked to construction QA/QC, it is very difficult to make an accurate estimate of anticipated infiltration through the barrier layer into the underlying waste. It is accurate to state, however, that if this proposed cover system is constructed with appropriate QA/QC, it will meet and exceed the regulatory performance specifications. To accurately monitor the performance of the cover system, it is recommended that the downgradient groundwater quality be closely monitored before and after cover construction to reveal contaminant concentration trends that will be indicative of the cover system effectiveness.

While the primary objective of the cover system is to minimize infiltration into the waste fill, there are several other significant performance criteria which must be satisfied. Given the site-specific conditions, the cover system must also:

- isolate the waste from humans, vectors and other animals, and other components of the surrounding ecosystem
- control gases generated within the waste fill
- be resistant to erosion by wind and water
- be resistant to static and seismic slope failures
- be durable, maintaining its design performance level for 30 years (regulatory) or the life of the waste fill (prudent)
- control surface water runoff and lateral drainage flow in a manner which does not promote erosion and does not adversely impact the surrounding environment

These criteria are satisfied by the proposed cover design. The waste is well isolated from the surrounding ecosystem by a minimum total of over 5 feet of soil. Any gases produced by the waste will be vented to atmosphere to prevent explosive conditions from occurring within the waste layer. Additionally, atmospheric monitoring is included as part of the post-closure program to ensure that vented gases do not violate Clean Air Act standards, and to ensure that no gas migrates off-site. The cover is designed to be erosion-resistant. The surface is graded to a moderate slope, seeded with an appropriate grass mixture, and covered with straw mulch. Surface water runoff and lateral drainage flow are handled by a network of open channels and culverts which divert flow to specified recharge areas in a controlled manner which also assists in erosion control. The cover system is also resistant to static and seismic slope failure. The minimum static factor of safety of the proposed cover system is 3.1, the minimum seismic factor of safety is 1.0. The recommended minimum factors of safety are 1.5 and 1.0, respectively. It should be noted that it is relatively rare to have a cover design satisfy the seismic stability safety factor in a seismically active area such as Cape Cod.

The issue of durability is not so clearly satisfied, in the author's opinion. Relatively little research on the long-term durability of geosynthetics in landfill covers has been performed, and since the history of geosynthetics in cover systems is fairly short, there are few, if any, case studies of sufficient length (e.g., over 30 years) to fill the data gap (Koerner et al., 1991). However, research that has been performed to date indicates that a cover system is an environment which is relatively conducive to geosynthetic survivability (Koerner et al., 1991). In a cover, the geosynthetics are not exposed to toxic chemicals, they are isolated from ultraviolet radiation, and they are fairly well protected from the effects of freeze/thaw cycles. Thus, it seems likely that the cover system will maintain its integrity well into the future.

In summary, it is the author's contention that the proposed cover system will adequately contain the source of the LF-1 plume. If constructed with appropriate construction QA/QC, the proposed cover system design will provide a nearly impermeable barrier while also controlling lateral drainage flow, surface runoff, and decomposition gases with a stable, durable design that should maintain its integrity for decades.

REFERENCES

ABB Environmental Services, Inc., "Closure Plan for Area of Contamination LF-1 1970 Cell, Post-1970 Cell and Kettle Hole," Installation Restoration Program, Massachusetts Military Reservation Cape Cod, Massachusetts, April 1992.

ABB Environmental Services, Inc., "Draft Interim Remedial Investigation Main Base Landfill (AOC LF-1)," Installation Restoration Program, Massachusetts Military Reservation Cape Cod, Massachusetts, March 1992.

ABB Environmental Services, Inc., "Focused Feasibility Study Main Base Landfill (AOC LF-1) Source Operable Unit," Installation Restoration Program, Massachusetts Military Reservation, prepared for HAZWRAP, Portland, Maine, June 1992.

ABB Environmental Services, Inc., "Technical Support for Closure Plan for Study Area LF-1," Installation Restoration Program, Massachusetts Military Reservation Cape Cod, Massachusetts, January 1993.

Austin, D., "Geonets in Landfill Closures - A Case History," *Proceedings of the 4th GRI Seminar on the Topic of Landfill Closures - Geosynthetics, Interface Friction and New Developments*, Geosynthetic Research Institute, Philadelphia, 1991.

Austin, T., "Landfill-Cover Conflict," *Civil Engineering*, Vol. 62, No. 12, December 1992, pp. 70-72.

Bagchi, A., *Design, Construction, and Monitoring of Sanitary Landfills*, John Wiley & Sons, Inc., New York, 1990.

Bjangard, A., and Edgers, L., "Settlement of Municipal Solid Waste Landfills," in *Proceedings of the Thirteenth Annual Madison Waste Conference*, University of Wisconsin - Madison, September 1990.

Cadwallader, M. W., "Addressing the Special Concerns of Landfill Closures: VLDPE and Textured Geomembranes," in *Proceedings of the 4th GRI Seminar on the Topic of Landfill Closures - Geosynthetics, Interface Friction and New Developments*, Geosynthetic Research Institute, Philadelphia, 1991.

CDM Federal Programs Corporation, "Remedial Investigation Main Base Landfill (AOC LF-1) and Hydrogeologic Region I Study Volume I," Installation Restoration Program, Massachusetts Military Reservation Cape Cod, Massachusetts, April 1995.

Code of Federal Regulations, 40 CFR, "Protection of Environment," The Office of the Federal Register National Archives and records Administration, U.S. Printing Office, Washington, D.C., 1992, 40 CFR 264 Subpart G and Subpart N.

Danielson, D. E., and Richardson, G. N., "The Role of Geomembranes and GCL's in Landfill Covers," *Geotechnical Fabrics Report*, Vol. 13 No. 1, January/February 1995, pp. 44-56.

Eith, A. W., Boschuk, J., and Koerner, R. M., "Prefabricated Bentonite Clay Liners," in *Proceedings of the 4th GRI Seminar on the Topic of Landfill Closures - Geosynthetics, Interface Friction and New Developments*, Geosynthetic Research Institute, Philadelphia, 1991.

Fassett, J. B., Leonards, G. A., and Repetto, P. C., "Geotechnical Properties of Municipal Solid Wastes and Their Use in Landfill Design," *Proceedings of the WasteTech Conference*, Charleston, SC, January 1994.

Geosyntec Consultants, *Landfill Cover Material Identification and Evaluation*, Report for the California Integrated Waste Management Board, California, March 1994.

Giroud, J. P., Swan, R. H., and Spooner, P. R., "Geosynthetic Landfill Cap: Laboratory and Field Tests, Design and Construction," *Proceedings of the 4th International Conference on Geotextiles, Geomembranes and Related Products*, The Hague, 1990.

Hwu, B.-L., Sprague, C. J., and Koerner, R. M., "Geotextile Intrusion into Geonets," *Proceedings of the 4th International Conference on Geotextiles, Geomembranes and Related Products*, The Hague, 1990.

Jaros, D. L., "Closing Hazardous Landfills," *Civil Engineering*, Vol. 61, No. 4, April 1991, pp. 64-68.

Jesionek, K. S., and Dunn, R. J., "Landfill Final Covers: A Review of California Practice," in *Landfill Closures... Environmental Protection and Land Recovery*, Geotechnical Special; Publication No. 53, R. J. Dunn and U. P. Singh, Eds., ASCE, New York, 1995.

Koerner, R. M., *"Designing with Geosynthetics,"* Third Edition, Prentice Hall, Englewood Cliffs, New Jersey, 1994.

Koerner, R. M., Halse-Hsuan, Y., and Lord, A. E., Jr., "Long-Term Durability of Geomebranes," *Civil Engineering,* Vol. 61, No. 4, April 1991, pp. 56-60.

Koerner, R. M., and Daniel, D. E., "Better Cover-ups," *Civil Engineering,* Vol. 62, No. 5, May 1992, pp. 55-58.

Massachusetts Department of Environmental Protection (MA DEP), *Landfill Technical Guidance Manual,* MA DEP, Boston, Massachusetts, September 1993.

McBean, E. A., Rovers, F. A., and Farquhar, G. J., *Solid Waste Landfill Engineering and Design,* Prentice-Hall, Inc., New Jersey, 1995.

Metcalf and Eddy, Inc., "Installation Restoration Program Phase I - Records Search Otis Air National Guard Base Massachusetts," Prepared for Otis Air National Guard Base Massachusetts, January 1983.

Murphy, W. L., and Gilbert, P. A., *Settlement and Cover Subsidence of Hazardous Waste Landfills,"* EPA-600/2-85-035, Final Report to Municipal Environmental Research Laboratory, Office of Research and Development, U. S. Environmental Protection Agency, Cincinnati, Ohio, 1985.

Northeast Regional Climate Center, Data Summary for Hatchville Station, March 1996.

Oldale, R.N. "Glaciotectonic Origin of the Massachusetts Coastal End Moraines and a Fluctuating Late Wisconsinian Ice Margin," Geological Society of America Bulletin, v. 95, 1984.

Peyton, R. L., and Schroeder, P. R., "Field Verification of HELP Model for Landfills," *Journal of Environmental Engineering,* vol. 114, No. 2, April, 1988.

Richardson, G. N., Kavazanjian, E., and Matasovic, N., *RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities,* Draft Report to the Risk Reduction Engineering Laboratory, Office of Research and Development, U.S. Environmental Protection Agency, Cincinnati, Ohio, February 1994.

Schroeder, P. R., Dozier, T. S., Zappi, P. A., McEnroe, B. M., Sjostrom, J. W., and Peyton, R. L., "The Hydrologic Evaluation of Landfill Performance (HELP) Model: Engineering Documentation for Version 3," EPA/600/9-94/xxx, U.S. Environmental Protection Agency Risk Reduction Engineering Laboratory, Cincinnati, Ohio, 1994.

Sharma, H. D. and Lewis, S. P., *Waste Containment Systems, Waste Stabilization, and Landfills*, John Wiley & Sons, Inc., New York, 1994.

Stone & Webster Environmental Technology & Services, "Main Base Landfill (LF-1) Data Gap Supplemental Remedial Investigation Work Plan Massachusetts Military Reservation Cape Cod, Massachusetts," Installation Restoration Program, Massachusetts Military Reservation Cape Cod, Massachusetts, January 1996.

Struve, F., "Geomembrane - Clay Composites," in *Proceedings of the 4th GRI Seminar on the Topic of Landfill Closures - Geosynthetics, Interface Friction and New Developments*, Geosynthetic Research Institute, Philadelphia, 1991.

Stulgis, R. P., Soydemir, C., Telgener, R. J., "Predicting Landfill Settlement," in *Geoenvironment 2000*, Geotechnical Special Publication No. 46, Y. B. Acar and D. E. Daniel, Eds., ASCE, New York, 1995.

U.S. Department of Agriculture, Soil Conservation Service (USDA SCS), *Guidelines for Soil Erosion and Sediment Control*, USDA SCS, February 1986.

U.S. Department of Agriculture, Soil Conservation Service (USDA SCS), *A Guide to: Conservation Plantings on Critical Areas for New York*, USDA SCS, Syracuse, New York, June 1991.

U.S. Environmental Protection Agency (USEPA), *Covers for Uncontrolled Hazardous Waste Sites*, EPA/540/2-85/002, USEPA, Washington, D.C., September, 1985.

U.S. Environmental Protection Agency (USEPA), *Design and Construction of RCRA/CERCLA Final Covers*, EPA/625/4-91/025, USEPA, Washington, D.C., May 1991.

U.S. Environmental Protection Agency (USEPA), *Final Covers on Hazardous Waste Surface Impoundments*, EPA/530-SW-89-047, USEPA, Washington, D.C., July 1989.

U.S. Environmental Protection Agency (USEPA), *How to Meet requirements for Hazardous Waste Landfill Design, Construction, and Closure*, Pollution Technology Review No. 185, Noyes Data Corporation, New Jersey, 1990.

U.S. Environmental Protection Agency (USEPA), *Report of Workshop on Geosynthetic Clay Liners*, EPA/600/R-93/171, USEPA, Washington, D.C., August 1993.

U.S. Geologic Survey Toxic Substances Hydrology Program, *Cape Cod Research Site,* Water Resources Division Massachusetts - Rhode Island District, October, 1995

Weston, Inc., "Installation Restoration Program, Phase II - Confirmation / Quantification Stage I, Volumes I and II," Final Report for Otis Air National Guard Base, Massachusetts; Prepared for the United States Air Force Occupational and Environmental Health Laboratory, Brooks Air Force Base, Texas, October 1985.

Zimmie, T. F., and La Plante, C., "The Effect of Freeze-Thaw Cycles on the Permeability of a Fine-Grained Soil," in *Proceedings, 22nd Mid-Atlantic Industrial Waste Conference,* Drexel University, Philadelphia, 1990.

APPENDIX A
THE HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE (HELP)

The Hydrologic Evaluation of Landfill Performance (HELP) is a computer model that was developed to assist landfill designers and evaluators in estimating the components of a landfill water balance (Peyton and Schroeder, 1988). The model computes daily runoff, evapotranspiration, lateral drainage, and percolation to obtain daily, monthly, and annual water balances (Peyton and Schroeder, 1988).

Field verification work performed by Peyton and Schroeder (1988) found that the HELP model produced reasonable water balance results. However, it is noted that no model can be expected to exactly reproduce field results because of the great variability of field sites. The results of the field work verified the utility of the HELP model for estimating general landfill performance (Peyton and Schroeder, 1988).

The results of the simulation performed on the proposed cover design are not entirely realistic. The model predicted zero infiltration through the barrier layer for each of the five years of simulation performed. It is well accepted that no cover system is completely impermeable (Koerner, 1994), therefore the results provided by the model must be taken as approximate indicators of cover system performance.

```
******************************************************************************
******************************************************************************
**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.04    (13 MARCH 1995)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:     C:\HELP3\MMRP.D4
TEMPERATURE DATA FILE:       C:\HELP3\MMRT.D7
SOLAR RADIATION DATA FILE:   C:\HELP3\MMRS.D13
EVAPOTRANSPIRATION DATA:     C:\HELP3\MMRE.D11
SOIL AND DESIGN DATA FILE:   C:\HELP3\MMRSOIL.D10
OUTPUT DATA FILE:            C:\HELP3\MMR2OUT.OUT


TIME:  14:58      DATE:   4/26/1996


******************************************************************************

     TITLE:  MMR MAIN BASE LANDFILL

******************************************************************************


     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
              COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                   LAYER  1
                                   --------

                      TYPE 1 - VERTICAL PERCOLATION LAYER
                          MATERIAL TEXTURE NUMBER   8
           THICKNESS                  =        6.00   INCHES
           POROSITY                   =        0.4630 VOL/VOL
           FIELD CAPACITY             =        0.2320 VOL/VOL
           WILTING POINT              =        0.1160 VOL/VOL
           INITIAL SOIL WATER CONTENT =        0.2755 VOL/VOL
           EFFECTIVE SAT. HYD. COND.  =  0.369999994000E-03 CM/SEC
         NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  4.20
                  FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.
```

LAYER 2

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 6

THICKNESS	=	18.00 INCHES
POROSITY	=	0.4530 VOL/VOL
FIELD CAPACITY	=	0.1900 VOL/VOL
WILTING POINT	=	0.0850 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.3105 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.720000011000E-03 CM/SEC

LAYER 3

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 20

THICKNESS	=	0.20 INCHES
POROSITY	=	0.8500 VOL/VOL
FIELD CAPACITY	=	0.0100 VOL/VOL
WILTING POINT	=	0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.1982 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	10.0000000000 CM/SEC
SLOPE	=	0.00 PERCENT
DRAINAGE LENGTH	=	0.0 FEET

LAYER 4

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 36

THICKNESS	=	0.06 INCHES
POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.399999993000E-12 CM/SEC
FML PINHOLE DENSITY	=	4.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	4.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD

LAYER 5

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.24 INCHES
POROSITY	=	0.7500 VOL/VOL
FIELD CAPACITY	=	0.7470 VOL/VOL
WILTING POINT	=	0.4000 VOL/VOL

```
INITIAL SOIL WATER CONTENT  =      0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.300000003000E-08 CM/SEC
```

LAYER 6

```
            TYPE 1 - VERTICAL PERCOLATION LAYER
                MATERIAL TEXTURE NUMBER   2
THICKNESS                   =      12.00   INCHES
POROSITY                    =      0.4370 VOL/VOL
FIELD CAPACITY              =      0.0620 VOL/VOL
WILTING POINT               =      0.0240 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.0617 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.579999993000E-02 CM/SEC
```

LAYER 7

```
            TYPE 1 - VERTICAL PERCOLATION LAYER
                MATERIAL TEXTURE NUMBER   4
THICKNESS                   =      24.00   INCHES
POROSITY                    =      0.4370 VOL/VOL
FIELD CAPACITY              =      0.1050 VOL/VOL
WILTING POINT               =      0.0470 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.1046 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.170000002000E-02 CM/SEC
```

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

```
NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
         SOIL DATA BASE USING SOIL TEXTURE # 8 WITH A
         FAIR STAND OF GRASS, A SURFACE SLOPE OF  5.%
         AND A SLOPE LENGTH OF  400. FEET.

SCS RUNOFF CURVE NUMBER             =     79.40
FRACTION OF AREA ALLOWING RUNOFF    =    100.0     PERCENT
AREA PROJECTED ON HORIZONTAL PLANE  =      9.000   ACRES
EVAPORATIVE ZONE DEPTH              =     15.0     INCHES
INITIAL WATER IN EVAPORATIVE ZONE   =      4.286   INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE  =      6.855   INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE  =      1.461   INCHES
INITIAL SNOW WATER                  =      0.000   INCHES
INITIAL WATER IN LAYER MATERIALS    =     10.711   INCHES
TOTAL INITIAL WATER                 =     10.711   INCHES
TOTAL SUBSURFACE INFLOW             =      0.00    INCHES/YEAR
```

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
NANTUCKET MASSACHUSETTS

STATION LATITUDE	=	41.15 DEGREES
MAXIMUM LEAF AREA INDEX	=	3.00
START OF GROWING SEASON (JULIAN DATE)	=	129
END OF GROWING SEASON (JULIAN DATE)	=	295
EVAPORATIVE ZONE DEPTH	=	15.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	12.50 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	74.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	81.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	86.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	76.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR NANTUCKET MASSACHUSETTS

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.02	3.93	4.17	3.64	3.41	2.32
2.87	3.89	3.34	3.26	4.34	4.16

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR NANTUCKET MASSACHUSETTS

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
30.00	30.00	38.00	45.00	55.00	65.00
70.00	70.00	65.00	55.00	45.00	35.00

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR NANTUCKET MASSACHUSETTS
AND STATION LATITUDE = 41.15 DEGREES

****

ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	44.06	1439440.250	100.00
RUNOFF	0.372	12160.365	0.84

EVAPOTRANSPIRATION	27.451	896836.750	62.30
DRAINAGE COLLECTED FROM LAYER 3	16.2373	530473.875	36.85
PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0166		
PERC./LEAKAGE THROUGH LAYER 7	0.012918	422.046	0.03
CHANGE IN WATER STORAGE	-0.014	-452.424	-0.03
SOIL WATER AT START OF YEAR	10.711	349942.031	
SOIL WATER AT END OF YEAR	10.698	349489.594	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.372	0.00

****

****

ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	49.16	1606057.120	100.00
RUNOFF	10.349	338091.469	21.05
EVAPOTRANSPIRATION	19.705	643774.375	40.08
DRAINAGE COLLECTED FROM LAYER 3	19.4450	635268.625	39.55
PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0399		
PERC./LEAKAGE THROUGH LAYER 7	0.012372	404.194	0.03
CHANGE IN WATER STORAGE	-0.376	-12289.666	-0.77
SOIL WATER AT START OF YEAR	10.698	349489.594	
SOIL WATER AT END OF YEAR	10.321	337199.937	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0247	808.076	0.05

****

****

ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	45.04	1471456.500	100.00
RUNOFF	8.366	273309.125	18.57
EVAPOTRANSPIRATION	25.192	823031.687	55.93
DRAINAGE COLLECTED FROM LAYER 3	11.6231	379725.344	25.81
PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0161		
PERC./LEAKAGE THROUGH LAYER 7	0.011859	387.433	0.03
CHANGE IN WATER STORAGE	-0.153	-4996.476	-0.34
SOIL WATER AT START OF YEAR	10.321	337199.937	
SOIL WATER AT END OF YEAR	10.067	328893.719	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.101	3309.733	0.22
ANNUAL WATER BUDGET BALANCE	0.0000	-0.630	0.00

****

****

ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT
PRECIPITATION	38.09	1244400.620	100.00
RUNOFF	7.726	252408.125	20.28
EVAPOTRANSPIRATION	18.835	615348.625	49.45
DRAINAGE COLLECTED FROM LAYER 3	12.0882	394922.219	31.74
PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0288		
PERC./LEAKAGE THROUGH LAYER 7	0.011436	373.623	0.03

CHANGE IN WATER STORAGE	-0.577	-18864.014	-1.52
SOIL WATER AT START OF YEAR	10.067	328893.719	
SOIL WATER AT END OF YEAR	9.591	313339.437	
SNOW WATER AT START OF YEAR	0.101	3309.733	0.27
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0065	212.078	0.02

****

****

ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	41.58	1358418.750	100.00
RUNOFF	5.366	175316.562	12.91
EVAPOTRANSPIRATION	24.636	804849.187	59.25
DRAINAGE COLLECTED FROM LAYER 3	11.0996	362623.375	26.69
PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0166		
PERC./LEAKAGE THROUGH LAYER 7	0.010958	357.983	0.03
CHANGE IN WATER STORAGE	0.465	15188.034	1.12
SOIL WATER AT START OF YEAR	9.591	313339.437	
SOIL WATER AT END OF YEAR	10.056	328527.500	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0026	83.645	0.01

****

****

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 5

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	3.69	4.50	3.11	5.47	4.35	2.91
	1.94	3.83	1.99	2.42	3.11	6.27
STD. DEVIATIONS	1.20	2.01	1.18	2.65	2.06	0.48
	1.06	1.25	1.10	1.08	1.49	2.43
RUNOFF						
TOTALS	1.305	2.378	1.924	0.163	0.163	0.000
	0.022	0.016	0.030	0.003	0.031	0.401
STD. DEVIATIONS	1.775	1.659	2.315	0.204	0.364	0.000
	0.048	0.025	0.046	0.007	0.055	0.485
EVAPOTRANSPIRATION						
TOTALS	0.585	0.616	1.126	2.795	3.661	3.360
	1.863	2.570	2.532	2.018	1.214	0.824
STD. DEVIATIONS	0.374	0.374	0.726	0.359	0.912	0.584
	0.902	1.269	0.454	0.642	0.279	0.163
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	1.9979	0.6709	2.0597	2.7219	1.5875	0.2844
	0.0768	0.0060	0.2515	0.2195	0.6258	3.5967
STD. DEVIATIONS	1.7321	0.9008	0.8734	1.3834	1.7336	0.2753
	0.0818	0.0081	0.5477	0.4014	0.9429	1.8072
PERCOLATION/LEAKAGE THROUGH LAYER 5						
TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
PERCOLATION/LEAKAGE THROUGH LAYER 7						
TOTALS	0.0010	0.0009	0.0010	0.0010	0.0010	0.0010
	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010
STD. DEVIATIONS	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	0.0213	0.0146	0.0441	0.0642	0.0229	0.0014
	0.0004	0.0000	0.0013	0.0011	0.0033	0.1086
STD. DEVIATIONS	0.0225	0.0292	0.0246	0.0592	0.0290	0.0014
	0.0004	0.0000	0.0028	0.0020	0.0050	0.0716

****

****

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 5

	INCHES		CU. FEET	PERCENT
PRECIPITATION	43.59	(4.112)	1423954.7	100.00
RUNOFF	6.436	(3.8279)	210257.12	14.766
EVAPOTRANSPIRATION	23.164	(3.7203)	756768.12	53.146
LATERAL DRAINAGE COLLECTED FROM LAYER 3	14.09861	(3.61671)	460602.687	32.34672
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.00000	(0.00000)	0.000	0.00000
AVERAGE HEAD ON TOP OF LAYER 4	0.024	(0.011)		
PERCOLATION/LEAKAGE THROUGH LAYER 7	0.01191	(0.00077)	389.056	0.02732
CHANGE IN WATER STORAGE	-0.131	(0.3964)	-4282.91	-0.301

****

****

PEAK DAILY VALUES FOR YEARS 1 THROUGH 5

	(INCHES)	(CU. FT.)
PRECIPITATION	3.76	122839.203
RUNOFF	3.154	103030.5940
DRAINAGE COLLECTED FROM LAYER 3	1.75306	57272.50000
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.000000	0.00000
AVERAGE HEAD ON TOP OF LAYER 4	2.316	
MAXIMUM HEAD ON TOP OF LAYER 4	0.000	
PERCOLATION/LEAKAGE THROUGH LAYER 7	0.000042	1.37193
SNOW WATER	3.65	119219.1020

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.4223
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0974

*** MAXIMUM HEADS ARE COMPUTED USING THE MOUND EQUATION. ***

****

****

FINAL WATER STORAGE AT END OF YEAR 5

LAYER	(INCHES)	(VOL/VOL)
1	1.6764	0.2794
2	5.0066	0.2781
3	0.0020	0.0100
4	0.0000	0.0000
5	0.1800	0.7500
6	0.7245	0.0604
7	2.4664	0.1028
SNOW WATER	0.000	

****
****

APPENDIX B
LF-1 GROUP PROJECT RESULTS

This Appendix provides the results of the investigations conducted for the LF-1 group report. The results are divided into two sections. Section A covers site characterization, groundwater modeling, and risk analysis. Section B examines source containment and bioremediation as potential remedial actions.

A. THE MMR LF-1 PLUME

SITE CHARACTERIZATION

Site characterization investigations followed two main topics with respect to this report. The first involved describing the nature and extent of the chemical contamination in the groundwater. The second involved analyzing tests for hydraulic conductivity to determine parameters that could be used for modeling contaminant migration.

Groundwater Contamination

As part of the Superfund Remedial Investigation process, 73 wells at different locations and different depths were tested for 34 of the most likely compounds. The EPA standard for drinking water sets individual maximum contamination levels (MCLs) for most of these compounds. 28 out of the 73 wells had at least one contaminant which exceeded the MCL. 7 out of the 34 possible contaminants were at levels which exceeded the MCL. These contaminants are vinyl chloride (VC), carbon tetrachloride (CT), trichloroethene (TCE), tetrachloroethene (PCE), 1,4 dichlorobenzene (1,4 DCB), benzene (B), and chloroform (CF). All of these compounds have an MCL of 5 ppb, except for vinyl chloride which has an MCL of 2 ppb. The highest total of all 7 of these contaminants at any one well was 168 ppb.

The highest total of all contaminants sampled at any one well was 236 ppb. (Some of these contaminants have an MCL much higher than 5 ppb.) The highest three individual contaminant readings were CT at 60 ppb, TCE at 64 ppb, and PCE at 65 ppb. One ppb by volume is equivalent to one drop in 15,000 gallons. 168 ppb is equivalent to about 1/3 ounce per 15,000 gallons. At 60 gallons per day of individual water use, 15,000 gallons are used in 250 days. At 236 ppb, the highest total concentration sampled, this works out to about 1 drop of exposure per person per day. The risk assessment section of this report discusses the danger to humans from possible exposure.

Looking at two dimensional log-linear contours of the contamination data points and vertical section filtered contours (see Figure A-1), a very rough estimate of the total volume of contamination can be made. This is estimated to be about 103 cubic feet or 14 - 55 gallon drums. This mass is distributed over approximately 4.5 square miles. The area where any single MCL level is exceeded is about 2 square miles.

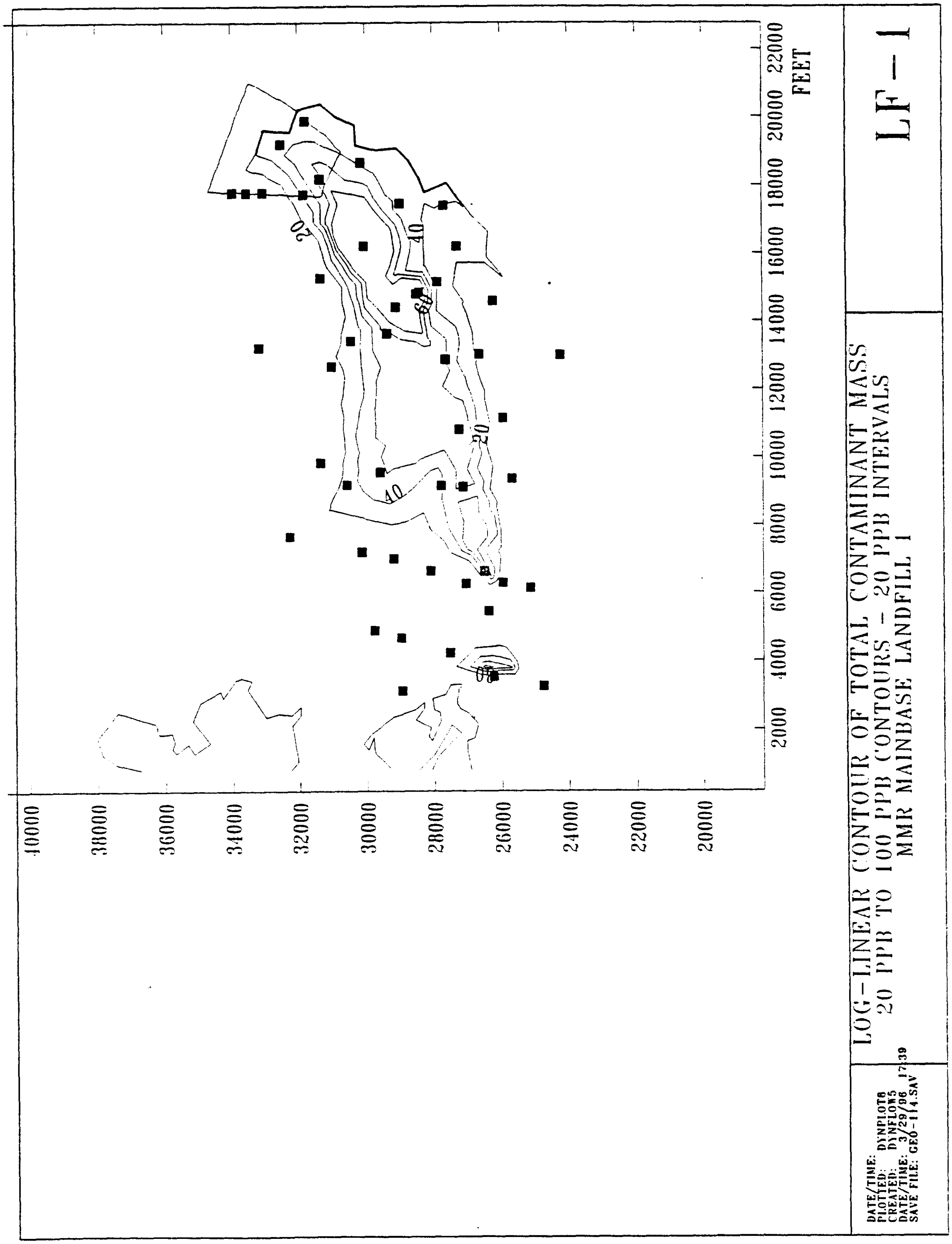

LOG-LINEAR CONTOUR OF TOTAL CONTAMINANT MASS
20 PPB TO 100 PPB CONTOURS - 20 PPB INTERVALS
MMR MAINBASE LANDFILL 1
LF-1
FEET

Figure A-1

Contamination contours show that little degradation of PCE is occurring. TCE is the degraded product of PCE. The contours show the center of PCE concentration to be downgradient from the center of TCE concentration, therefore the TCE could not be the result of PCE degradation. Instead, this indicates that TCE must be one of the originally dumped contaminants.

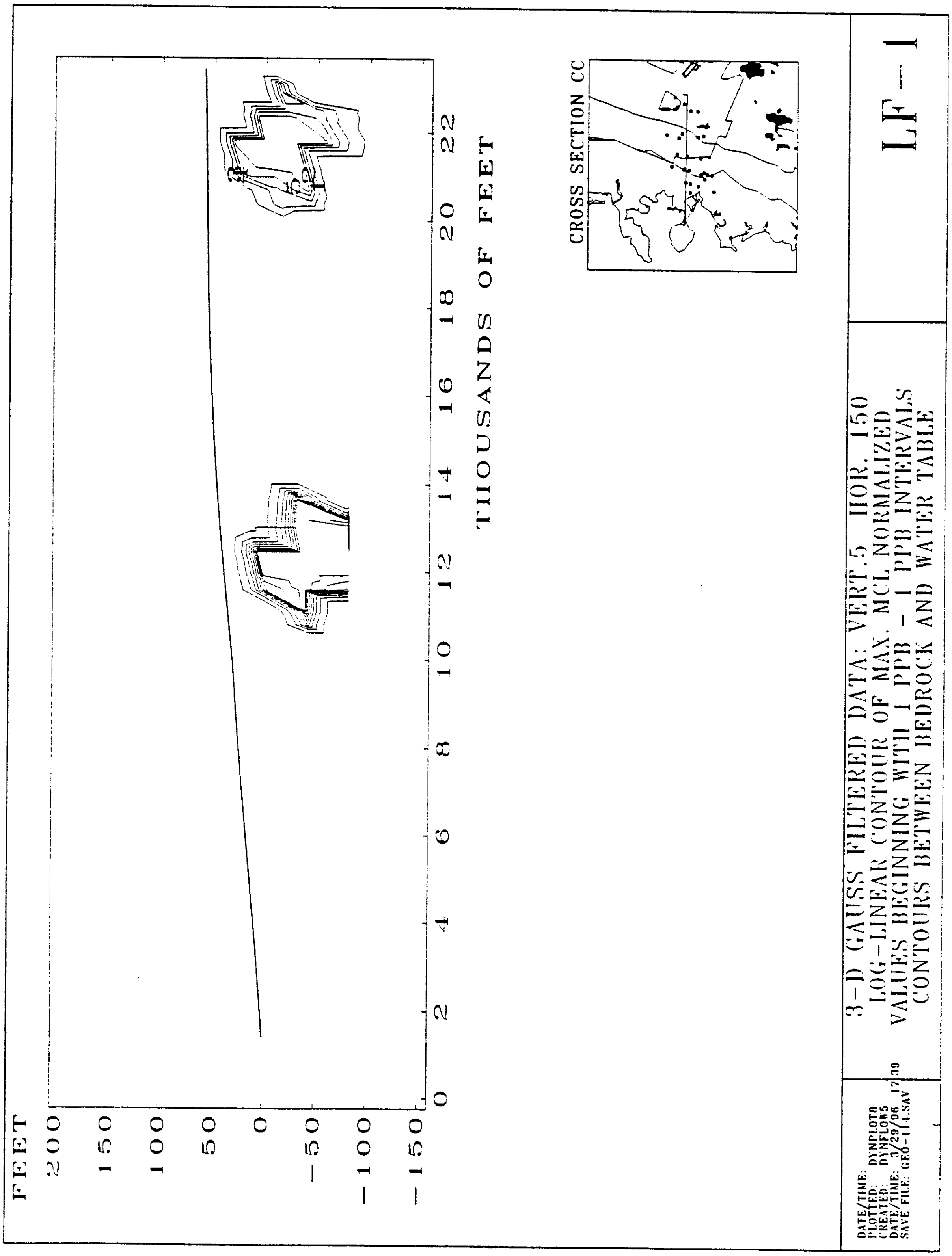
A comparison can be made between possible contaminant discharge to the ocean through groundwater migration versus the same discharge through a pipe from a hypothetical industrial source. If the contaminant front is considered to be 50 feet thick by 5000 feet wide and moving at a rate of 1 foot per day, this equates to an outfall pipe 2 feet in diameter with a flow rate of 1 foot per second. (A fast walk is about 5 feet per second.) In addition to drinking water standards, the EPA publishes guidelines for allowable contaminant marine discharge beyond the mean low water mark. These standards are considerably higher than those for drinking water. If the landfill plume were being discharged from a single pipe, the EPA would have to decide whether to permit such a discharge. From the given guideline values, and the known contamination levels, it is difficult to say whether a permit would be granted. However, the discharge is, in effect, put through a diffuser over an area 2500 times as large as the hypothetical pipe.

Examining cross sectional contours of contamination (see Figure A-2), it is seen that a contamination level exceeding the MCL comes within 10 feet of the top of the aquifer. It is estimated that the withdrawal depth of a hypothetical private well pulling 1000 gallons per day to be 13 feet, given a conservative figure for hydraulic conductivity (50 ft/day) and hydraulic gradient (1/100). Therefore, it is possible that private wells located directly over the uppermost levels of contamination could draw in water exceeding the MCL levels for drinking water.

Hydraulic Conductivity

Hydraulic conductivity (K) was determined using 140 grain size samples from 21 well locations and 79 slug test well locations. A comparison of values from these two different tests generally shows very poor correlation. However, a good correlation was seen between the Alyamani/Sen (Alyamani, et al, 1993) and Bedinger (Bradbury, et al, 1990) grain size methods. This is due to the fact that both depend on the grain size fraction d_{50}. Both grain size and slug test data were put through a 3-D gauss filtering process. The resulting data and corresponding contours exhibit a significant correlation between the Hazen and slug methods. However, the Hazen values are much lower.

The filtered slug contours match the general geology of the area, showing a decline in conductivity from north to south and with depth. In addition, the Buzzard's Bay Moraine is clearly seen (see Figure A-3). The contours also point



FEET
200
150
100
50
0
-50
-100
-150
0 2 4 6 8 10 12 14 16 18 20 22
THOUSANDS OF FEET
CROSS SECTION CC
3-D GAUSS FILTERED DATA: VERT.5 HOR. 150
LOG-LINEAR CONTOUR OF MAX. MCL NORMALIZED
VALUES BEGINNING WITH 1 PPB - 1 PPB INTERVALS
CONTOURS BETWEEN BEDROCK AND WATER TABLE
LF-1

Figure A-2

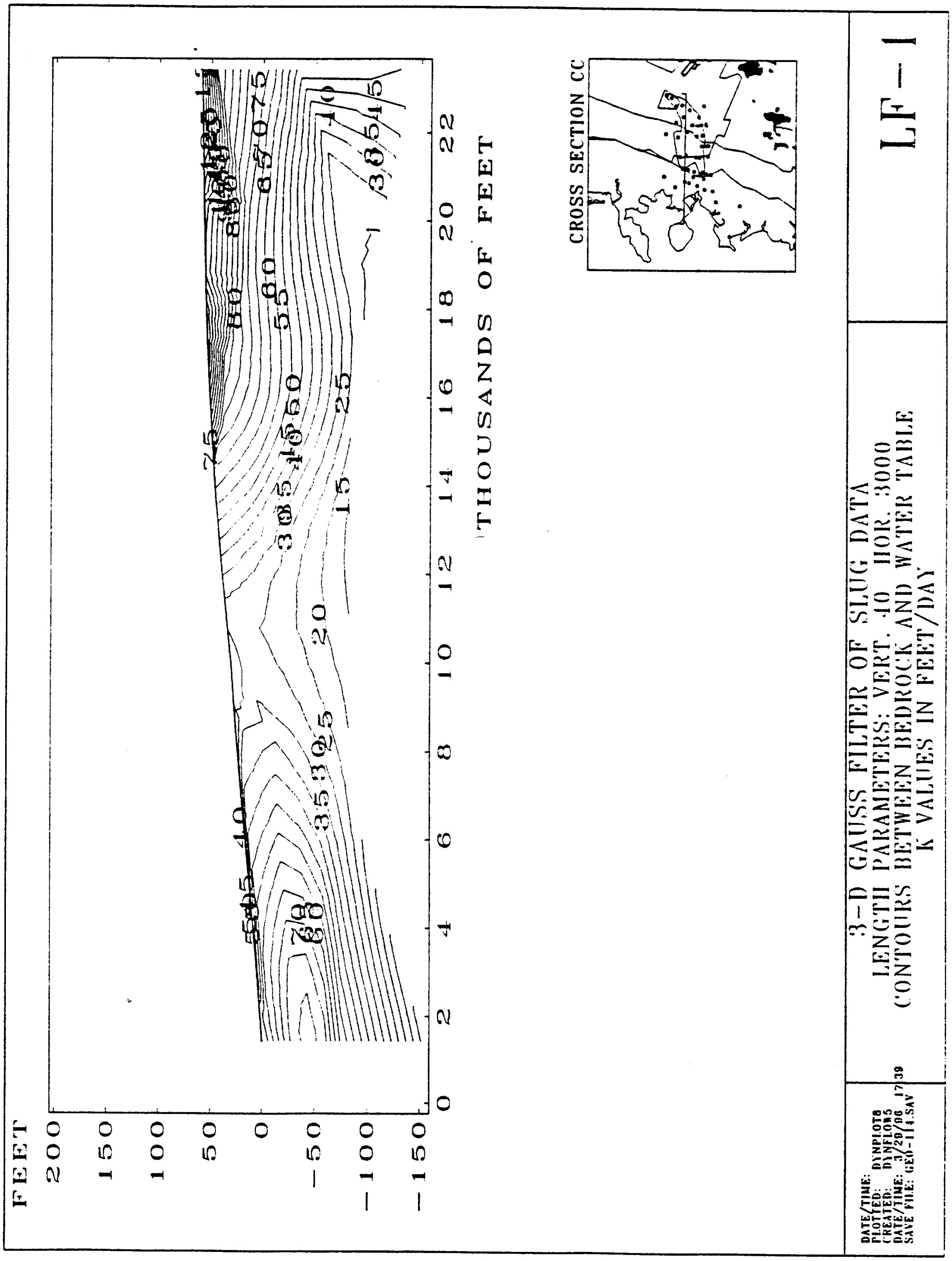

FEET
200
150
100
50
0
-50
-100
-150
0 2 4 6 8 10 12 14 16 18 20 22
THOUSANDS OF FEET
CROSS SECTION CC
LF-1
3-D GAUSS FILTER OF SLUG DATA
LENGTH PARAMETERS: VERT. 40 HOR. 3000
CONTOURS BETWEEN BEDROCK AND WATER TABLE
K VALUES IN FEET/DAY

Figure A-3

out a zone of lower conductivity in a region where the contaminant plume appears to be dividing. This finding may provide part of the explanation for the observed migration path. The arithmetic mean of the unfiltered slug test data was 75 feet/day, ranging from less than 1 ft/day to 316 feet/day. The calculated horizontal conductivity from the filtered slug test data had a mean of 85 feet/day and a maximum of 272 feet/day. In addition to hydraulic conductivity, a determination of overall hydraulic anisotropy was made using the filtered slug K values. The number was approximately 3.4. It is very similar to the value of 3.2 determined by Springer for the Mashpee Pitted Plain (Springer, 1991).

Summary

In summary, a large area of groundwater has been contaminated by the MMR Main Base Landfill 1 with halogenated volatile organic compounds. The contaminant plume is heading west through the Buzzards Bay Moraine. Public and private drinking supply wells are in danger of drawing water with concentration levels exceeding EPA drinking water standards. Hydraulic conductivity trends can be ascertained using gaussian filtered slug test data. Values for horizontal and vertical hydraulic conductivity may be calculated from the filtered data. These values may be used to model migration of the plume. The next section describes the groundwater modeling process.

Groundwater Modeling and Particle Tracking Simulation

Objectives and Scope

This section of the report describes a three dimensional groundwater model and particle tracking simulation of the portion of the aquifer that is deemed to affect the spatial characteristics and migration pathlines of the LF-1 plume. The DYN System modeling package developed by CDM, Inc., is utilized for this purpose. The goals of the modeling effort are as follows,

I. Develop a steady state flow model for the study area.

II. Track particles released from a continuous source area and observe migration patterns.

III. Determine flushing time and plume migration with source removed.

IV. Determine sensitivity of model results (plume migration) to the Buzzards Bay Moraine and other geologic features and characteristics of the region.

V. Explore the possibility that the deep plume observed in advance of the main plume is caused by a pool of dense leachate from the landfill sinking below the source area.

DYNFLOW, DYNTRACK and DYNPLOT Systems

The groundwater flow system of the Western Cape is modeled with the DYNFLOW groundwater modeling package. DYNFLOW is a FORTRAN based program that simulates three-dimensional flow using a finite element formulation. A distinct advantage of the finite element based model over a finite difference model like MODFLOW is that the former allows the user the flexibility to use variable sized grid elements. Thus, in regions of interest, the user can obtain higher resolution without having to implement the same degree of resolution throughout the model and obtain significant advantages in terms of computational time and complexity.

DYNTRACK simulates three-dimensional contaminant mass transport and uses the same finite element grid, flow field and aquifer properties that were used in and derived from DYNFLOW. DYNTRACK models either single particle tracking or 3-d transport of conservative or first-order decay contaminants with or without adsorption and dispersion.

DYNPLOT is a graphical pre- and post-processor that can create full color displays in plan view or cross-section of observed data, DYN system calculated data and simulated results. DYNPLOT is also capable of generating the finite element grid used by the flow and tracking models.

Study Area and Grid

The roughly triangular study area of the model was chosen to be large enough to ensure that boundary effects did not unduly influence the calculated flow and head values in the area of concern. The study area is depicted in Figure A-4. The northern and eastern boundaries of the model are streamlines (no-flux boundaries). The western part of the grid area is bounded by the ocean. The ocean-aquifer interface is of particular interest because it determines how far out at sea the LF-1 plume will discharge if it is not completely contained.

The grid covering the LF-1 study area was generated in DYNPLOT, with smaller grid elements in the sources area and presently observed plume locations and progressively coarser grid elements moving away from these locations. The study grid is composed of 3401 triangular elements and 1281 nodes. The grid discretizes the vertical dimension of the study area in 8 layers (9 levels). The bottom (1st) level follows the bedrock contours, while the top (9th) level approximates the surface topography.

Model Formulation

Assigned Geologic Materials

The geologic structure of the LF-1 study area was represented as depicted in Figures A-5, A-6, A-7 and A-8. The geographic locations of the material were assigned according to USGS maps of the region. The Mashpee Pitted Plain (MPP) was represented vertically as two material types and two horizontal sections. This was done to accurately represent the upward coarsening and north-south fining that is observed (LeBlanc, 1986) The Buzzards Bay Moraine (BBM) was defined vertically as four different material of increasing permeability upwards and two horizontal divisions. The Buzzards Bay Outwash (BBO) was depicted by two vertical materials, coarsening upwards. All three deposit types were underlain by a layer of Glacio-Lacustrine deposits (GLS) of varying thickness and bedrock.

Source

The LF-1 source was represented by six distinct cells within the source area. In the particle tracking simulation, three cells were defined as being non-sources after 1994. This was done to simulate a successful capping of part of the landfill in 1994 by the IRP.

Ponds

Ponds were modeled as a layer of material that was almost infinitely permeable horizontally and with a high vertical conductivity of the order of 500 ft/day. The pond material layer was extended to the observed depth of the each pond. These pond nodes were then assigned a rising head boundary condition. With this method, the material defined as the pond displays a consistent horizontal head and acts as a sink for groundwater upgradient of the pond and a source of groundwater to sections of the grid downgradient. This formulation was considered to most closely approximate the behavior of ponds in the Cape Cod region.

Hydraulic Properties

Hydraulic Conductivity

Estimates of hydraulic conductivity for the LF-1 region have been made through field investigations. Many slug tests, and laboratory tests of soil samples have been carried out for the sediments found in the Cape Cod region. The previous section on site characterization carries a full discussion of these empirical findings. For the purposes of the groundwater model, hydraulic conductivities proved to be the parameter to which the flow model was most sensitive. Hydraulic conductivity values of each sediment type were considered a variable input, and were assigned values within an empirically determined range obtained from literature in calibrating the flow model. The final values of hydraulic conductivities assigned to each geologic material are included in Table A-1.

Material	K_x, K_y ft.day	K_z ft/day	Long. Disp ft.	Trans. Disp ft	Disp Ratio vert./horiz
Lacustrine	15	5	90.0	3.3	0.03
Fine Sand West	80	27	90.0	3.3	0.03
Coarse Sand West	180	60	90.0	3.3	0.03
Fine Sand South	135	45	90.0	3.3	0.03
Coarse Sand South	210	70	90.0	3.3	0.03
BBM Low -North	30	10	90.0	3.3	0.03
BBM Med Low-North	110	33	90.0	3.3	0.03
BBM Med High-North	150	50	90.0	3.3	0.03
BBM High-North	170	57	90.0	3.3	0.03
BBM Low -South	15	5	90.0	3.3	0.03
BBM Med Low-South	60	20	90.0	3.3	0.03
BBM Med High-South	100	33	90.0	3.3	0.03
BBM High-South	135	45	90.0	3.3	0.03
Nant. Ice Deposits	190	63	90.0	3.3	0.03
Pond Material	10^{-5}	10	90.0	3.3	0.03
Fine Sand North	140	47	90.0	3.3	0.03
Coarse Sand North	270	90	90.0	3.3	0.03
Fine Lacustrine	10	3	90.0	3.3	0.03

Table A-1 Hydraulic Conductivities and Dispersivities used in flow and mass transport models.

Dispersivity

Accurately characterizing the dispersivity at a field site is essential in predicting the transport and spreading of a contaminant plume. Due to natural heterogeneities in the field that cause irregular flow patterns, field-scale dispersivities are several orders of magnitude larger than laboratory scale values (Gelhar et al., 1992). In this model, a tabulation of field-scale dispersivity data is used to obtain suitable values of the dispersivity coefficients while taking into account the scale of the LF-1 source. These values are also included in Table A-1.

Effective Porosity

Porosity estimates for the outwash in the LF-1 study area range from less than 1% to over 30% (CDM Federal, 1995). These values are somewhat lower than expected from tracer tests of Cape Cod, which range from 38-42% (Garabedian et. al, 1988; LeBlanc et al., 1986). It was decided to use an effective porosity value of 39% throughout the model.

Boundary Conditions

Saltwater-Freshwater Interface

The saltwater-freshwater interface determines where the landfill plume, if not fully contained, will discharge in to Megansett, Red Brook and Squeteague harbors. The steepness and the distance from shore of the interface depends on the aquifer discharge and geologic characteristics of the coastal region. Available geologic information does not indicate the existence of low permeability layers above the aquifer near the shore that will force the salt-fresh interface further into the ocean. Therefore, for the purposes of this report, it is assumed that the location and shape of the salt-fresh interface along the Western Cape Cod shore are determined entirely by the discharge and hydraulic conductivity of the aquifer. The distance from the shore to the salt-fresh interface was calculated to be approximately 500 ft.

No-Flux Boundaries

No-flux boundaries are modeled in DYNFLOW by assigning all nodes on streamlines at the edge of the study area a "free head" boundary condition. It is assumed that the no-flux boundaries are far enough from the areas of the model we wish to observe that they do not influence the calculated values of head and velocity.

Recharge

Natural recharge is the largest source of replenishment of the West Cape aquifer system. This natural recharge is composed entirely of rainfall infiltrate through the surface layer. Cape Cod on average receives 46 inches of rainfall annually (source). Nearly half of this precipitation, or 46-50%, infiltrates to the groundwater system through the highly permeable top soil (LeBlanc et al., 1986). There is little or no surface runoff due to the permeable nature of the soils and the small topographic gradients present in this region. Artificial recharge and pumping is considered to be negligible in this region in comparison with the natural recharge.

Results

The calibrated flow model agreed with observed water table measurements at 106 wells within 0.044 ft mean difference and 2.159 ft standard deviation. Figure A-9 shows the calibrated model results and calculated water table contours. The calculated contours are also consistent with observed water table contours in the region.

The flow model was found to be very sensitive to the difference in permeability between the moraine and surrounding deposits. This sensitivity is highlighted by the curvature of the model calculated head contours, which in turn significantly influence the migration pathlines of a contaminant released at the LF-1 site. The sensitivity of the particle paths to head contours is enhanced by the fact that the LF-1 source area is located close to the point where north south head contours change to an east-west orientation.

The first particles released at the LF-1 site will migrate to the ocean in 50 years. Figure A-10 shows a 51 year mass transport simulation in plan view, with particles reaching the ocean interface. Figure 3-11 is a cross section of the simulated plume. Thus, assuming that the volatile organic compounds of concern at this site were released in 1945, the predicted extent of the plume reaches the ocean discharge face by 1996. The initial discharge point is at Red Brook Harbor. This finding is in agreement with the Op-Tech Data Gap Report which concludes that the LF-1 plume has now reached Red Brook Harbor (Op-Tech, 1996).

If the entire landfill is successfully capped by the year 2000, and the contaminated groundwater is allowed to flush unmitigated into the ocean, the DYNTRACK simulation time of 110 additional years is required for all LF-1 derived contaminants in the aquifer to travel beyond the Buzzards Bay Moraine. A further 55 years is required for all the contaminant particles to be discharged from the aquifer.

The predicted plume exhibits the same differential North and South Lobe travel times observed in the field. In the model, the presence of a low-permeability layer in the moraine causes the southern part of the plume to be retarded. The northern section, by virtue of having to travel a shorter distance to the moraine, is at a higher elevation than the southern part of the plume and thus travels through a higher permeability layer of the moraine. These differential travel velocities through the moraine cause the distinct northern and southern lobes observed in the simulated plume. Figure A-12 is a north-south cross-section of the plume at the point of entry into the moraine, showing the differential elevations of the particles from north to south.

The previous finding that the portion of the plume at a lower elevation is retarded by the presence of a lower conductivity layer of moraine deposits indicates that the deep plume observed near the shoreline cannot be simulated by a sinking source of contaminant in this model formulation. A tenable explanation for the observed deep northern plume is that the down-sloping bedrock surface near the shoreline causes the faster moving simulated northern lobe to sink further due to infiltration as it traverses the Buzzards Bay Outwash towards the shoreline. Since the slower moving southern lobe is still in the moraine, the leading edges of the northern lobe near Red Brook Harbor now appear to be a northern plume lobe at a lower elevation.

If an extraction well system is constructed along Route 28, and it is assumed that the extraction pumping and infiltration are carried out so that the hydraulic system is relatively unchanged, the uncaptured section of the LF-1 plume will take a further 12 years to completely discharge into the ocean. This result was obtained assuming that the portion of the plume upgradient of the extraction well fence is fully captured.

In summary, the groundwater flow and particle transport model provides results that are similar to field observations. The Buzzards Bay Moraine exerts a great deal of influence on the regional hydrologic system. The geologic characteristics assigned in the flow model to the BBM defines the shape of the regional head contours and thus the travel path and velocity of the simulated plume. Therefore, it is essential that the geology of this moraine be properly identified if a flow and particle tracking model that can accurately represent the region is to be formulated. In the absence of such data, any groundwater flow model of the LF-1 region will contain a significant degree of uncertainty and error. The models developed in this study can be used to determine the effects of an extraction system to contain or capture the LF-1 plume and also as a means of designing an efficient capture system for this contaminated site. The following section addresses the risks associated with the LF-1 plume and how these risks can be managed.

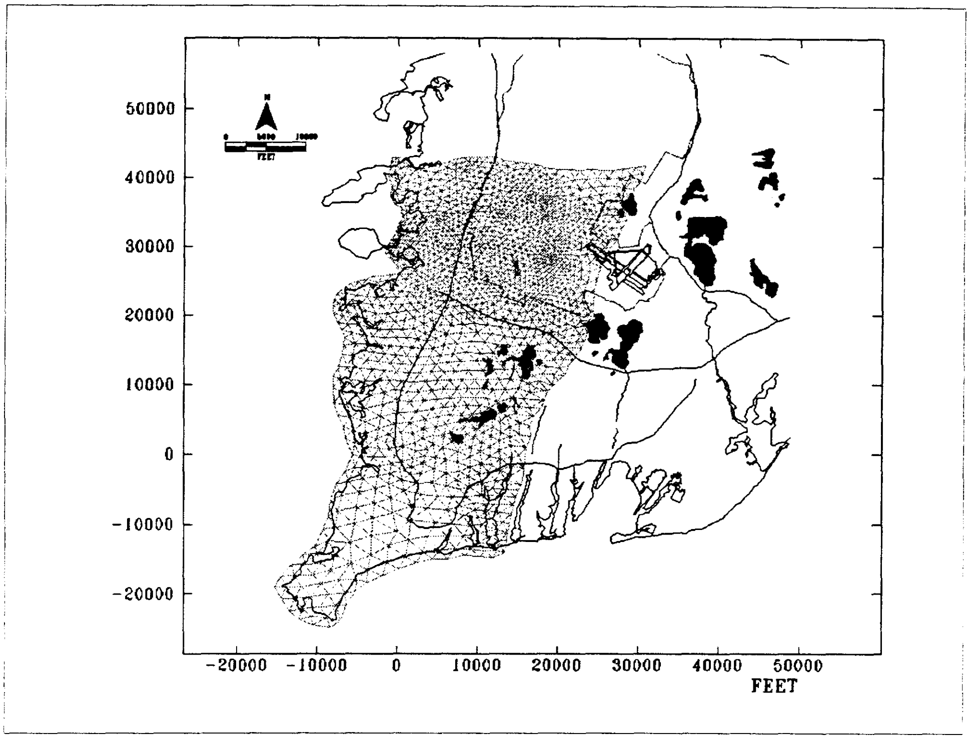


Figure A-4 LF-1 study area and finite element grid.

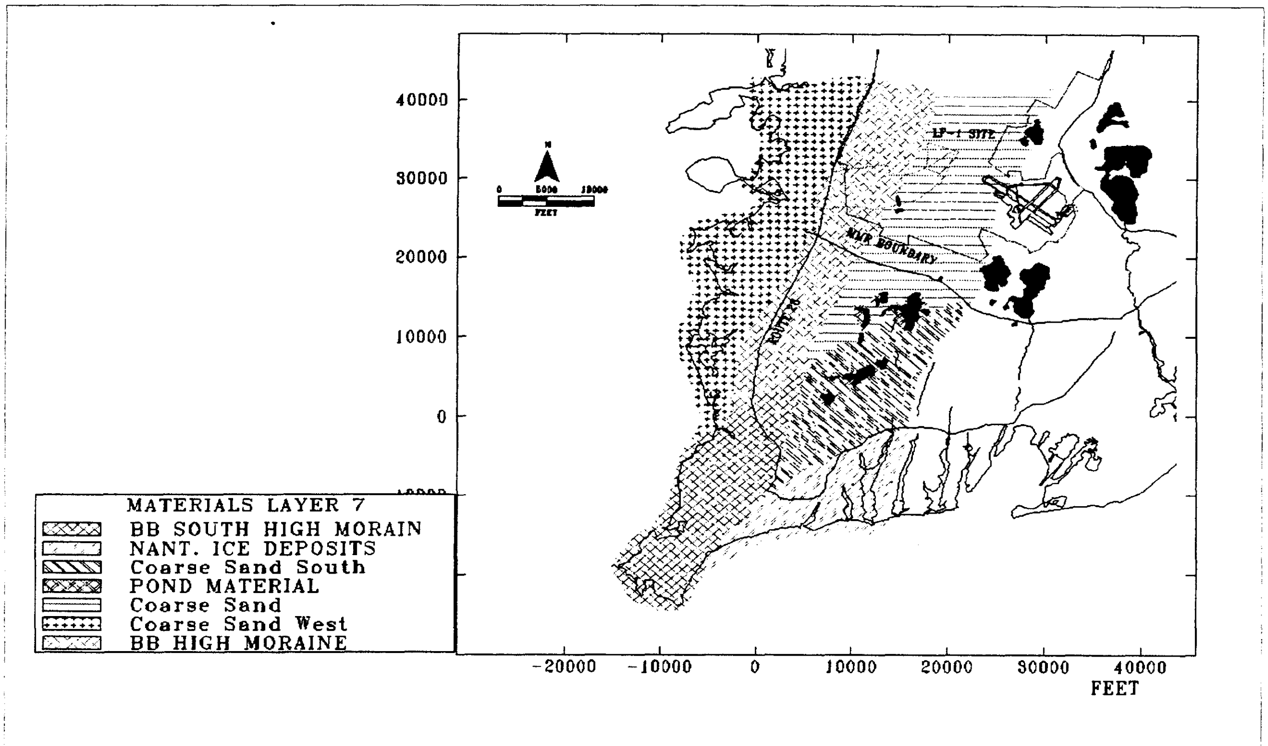


Figure A-5 Plan view of LF-1 study area with assigned geologic materials.

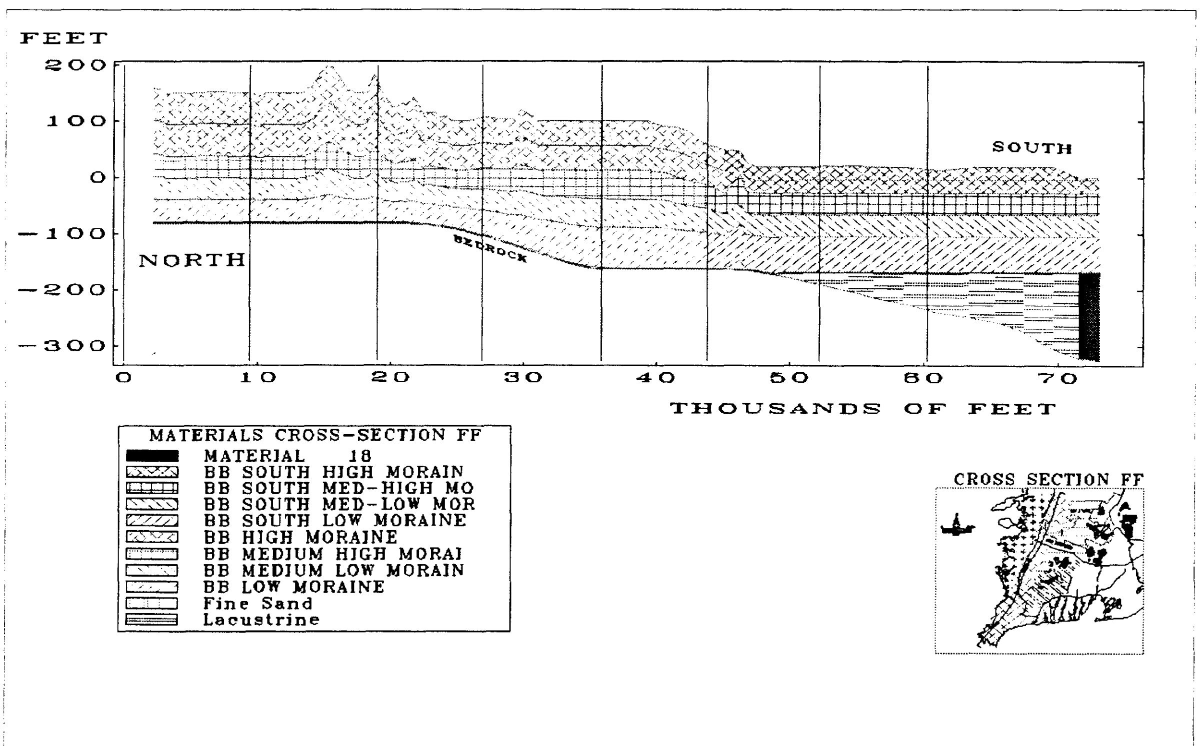


Figure A-6 Cross-sectional view of Buzzards Bay Moraine deposits.

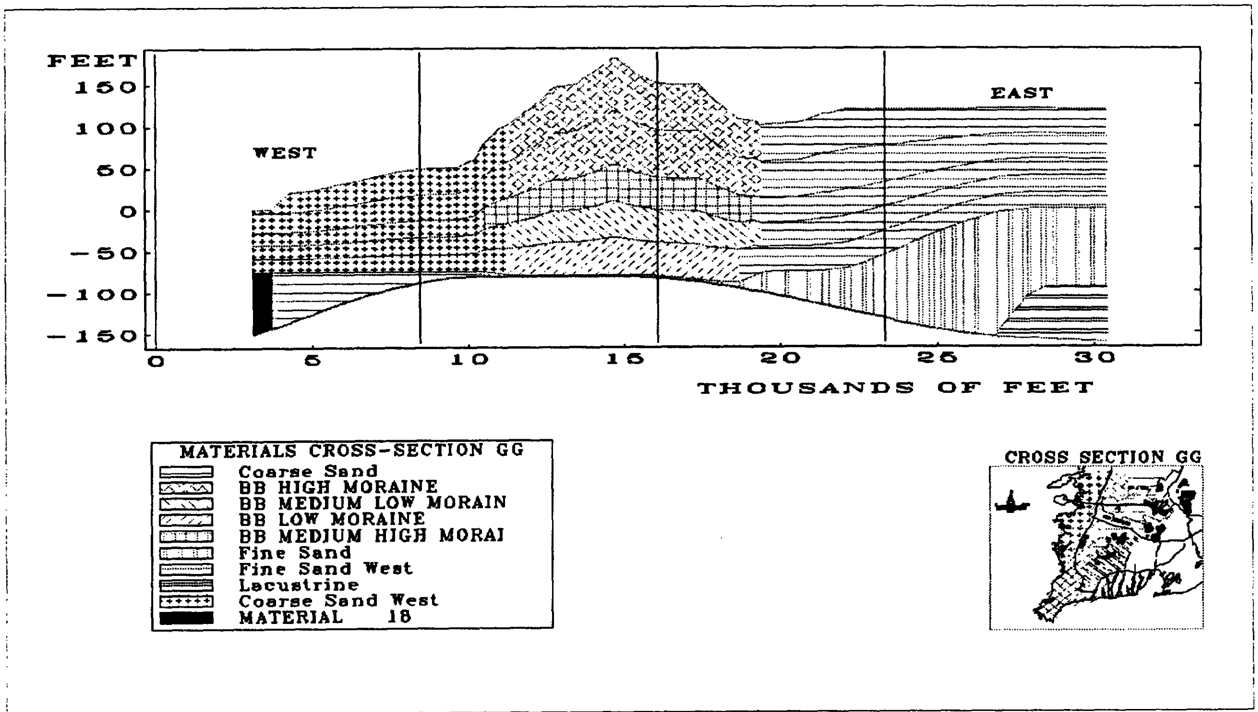


Figure A-7 East-West cross-section of study area near Buzzards Bay.

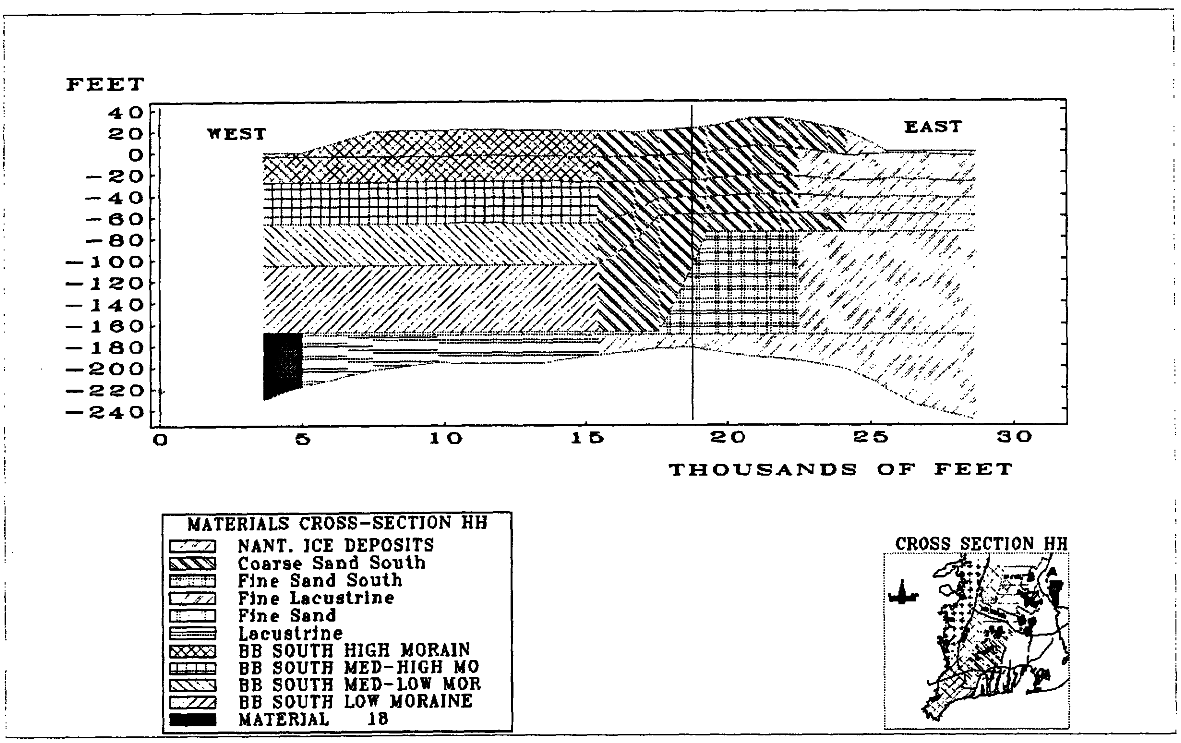


Figure A-8 East-West cross-section of study area near Nantucket Sound.

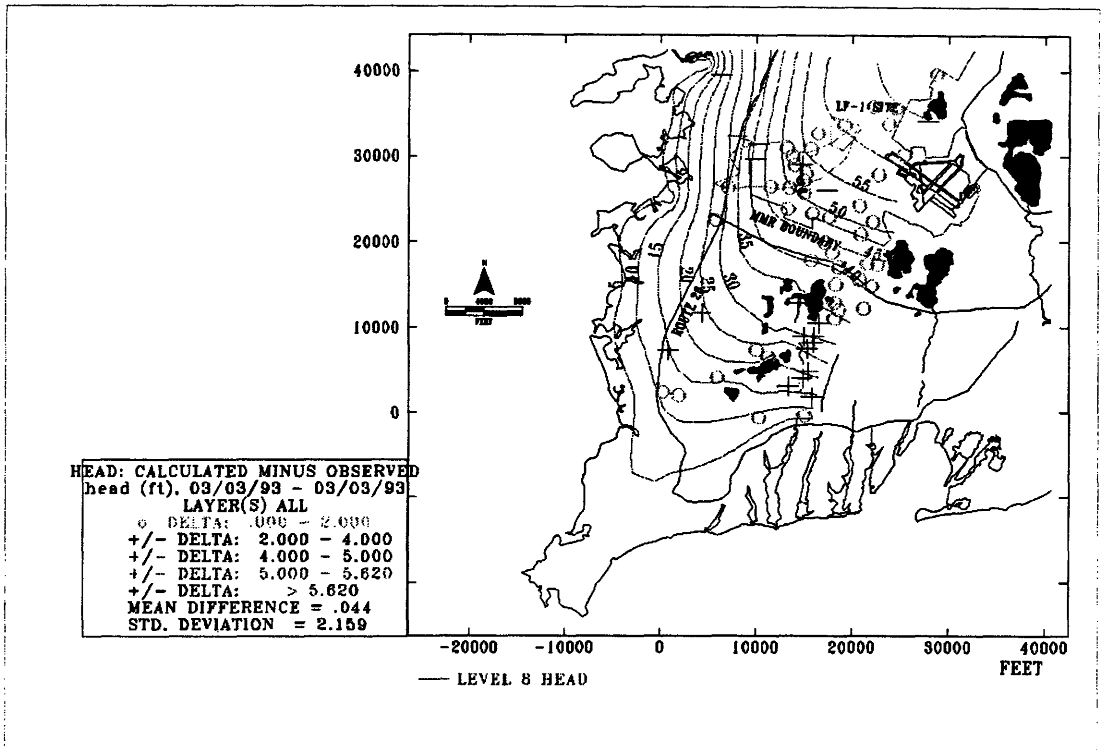


Figure A-9 Calculated water table elevation contours and flow model calibration results.

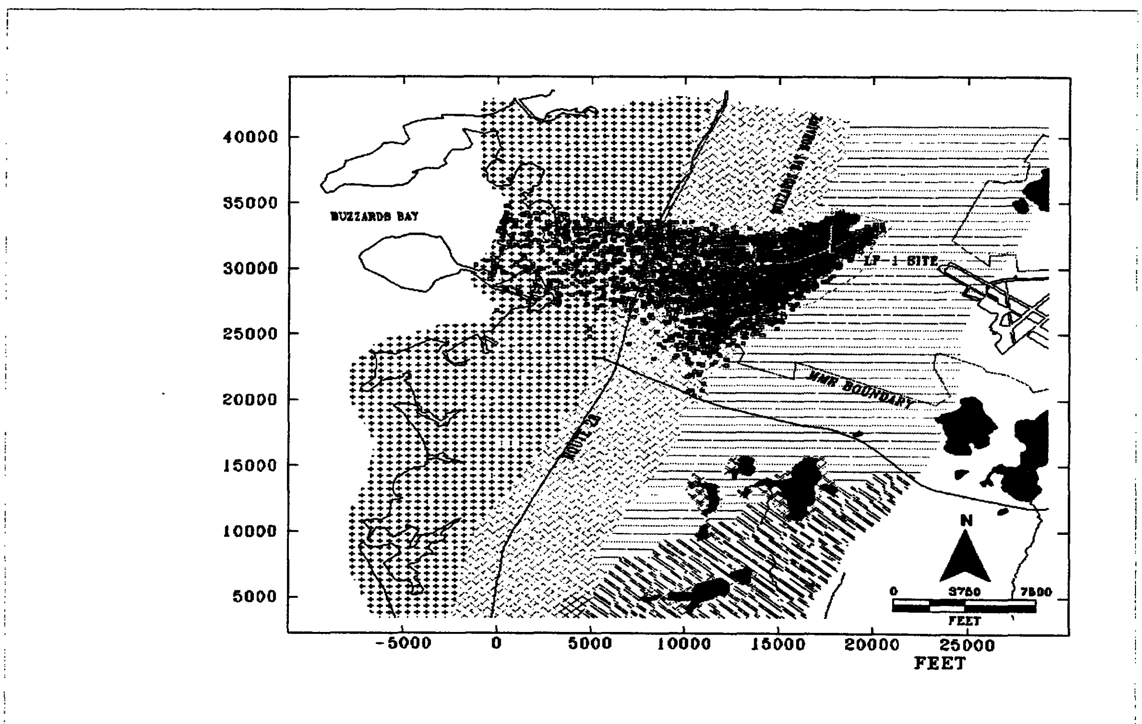


Figure A-10 Plan view of simulated LF-1 plume. Buzzards Bay Moraine is also shown.

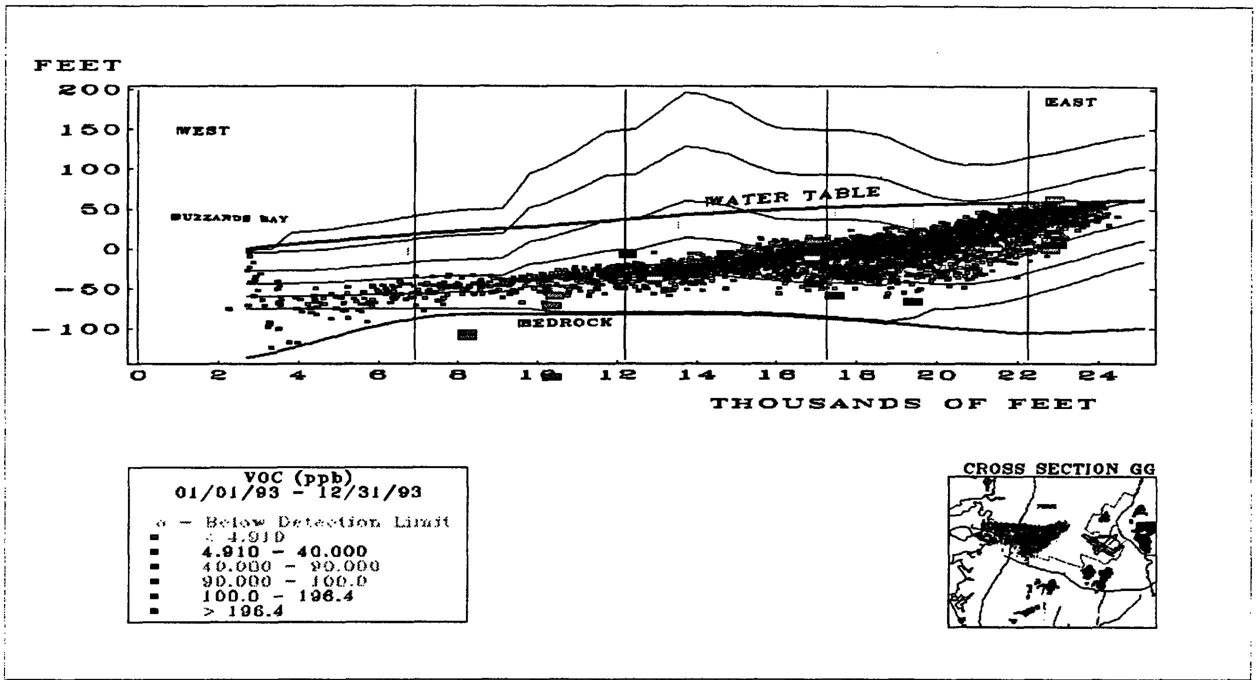


Figure A-11 Cross-Section of simulated LF-1 plume and observed contamination locations.

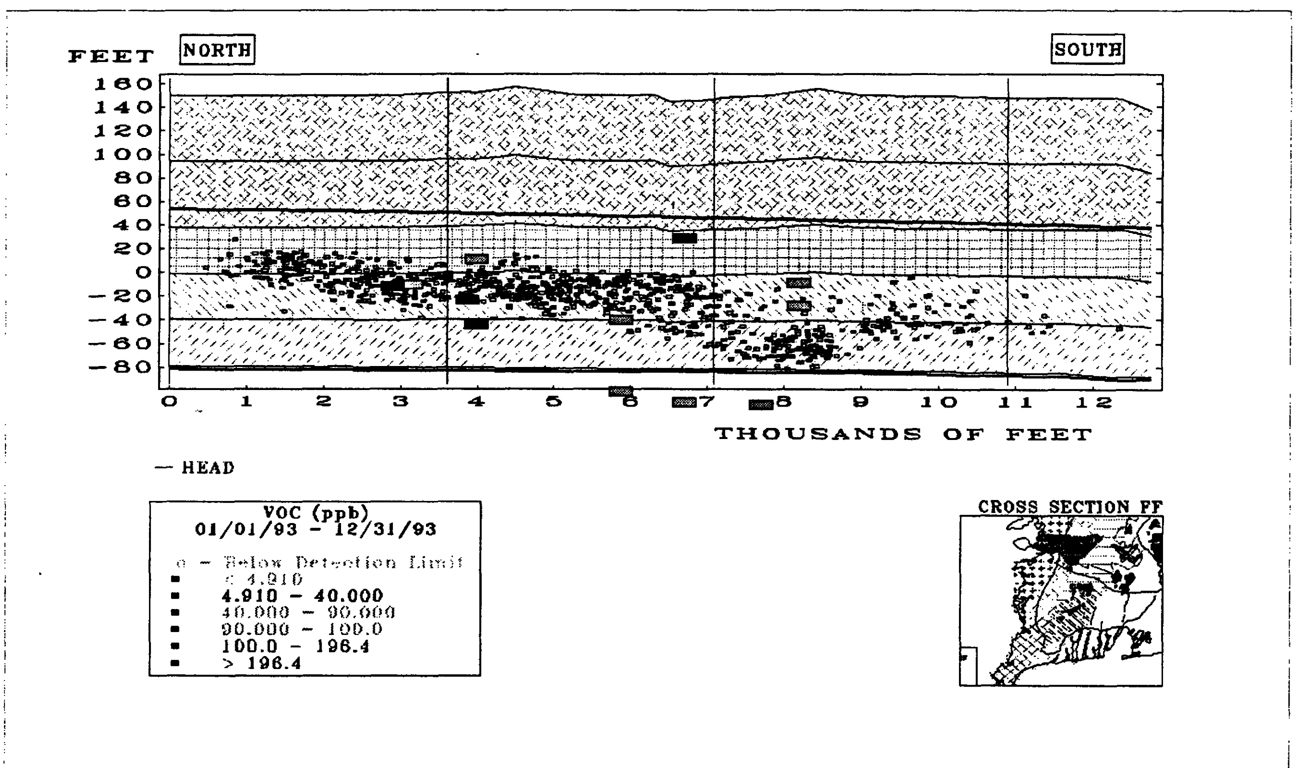


Figure A-12 Cross sectional view of LF-1 plume as it enters the Buzzards Bay Moraine.

Risk Assessment & Management of Risks

The IRP's Remedial Investigation (RI) Report and their Final Risk Assessment Handbook (RAH) present an evaluation of potential adverse effects to human health from materials identified in the MMR LF-1. The MMR site has been classified using EPA guidelines which were not specifically developed for the MMR site. The accuracy of the health and environmental risk scores are limited by the constraints of the EPA's deterministic risk assessment model.

Cancer risk is the statistical increase in mortality rate for a member of the local community who has been exposed to carcinogenic materials identified in the MMR LF-1 as compared to the rate for a member of the local community if the MMR LF-1 did not exist. It is the probability of an event occurring and the magnitude of the effect which an event will likely produce. More simply, cancer risk is the product of the probability of dying from cancer because of exposure to carcinogens and the probability of exposure to carcinogens.

Toxicology

According to the EPA guidelines (cited in both the RAH, 1994 and LaGrega et. al., 1994), toxicology and dose are to be calculated by following specific protocols. In terms of toxicology, carcinogens are considered to vary greatly in their potency. "When considering lifetime cancer risk to humans, it is widely accepted that carcinogenesis works in a manner such that it is possible, however remote, that exposure to a single molecule of a genotoxic carcinogen could result in one of the two mutations necessary to initiate cancer". (LaGrega et. al., 1994, p. 277). Therefore, the calculation of carcinogenic risk from toxicology involves the use of cancer potency factors which are basically the slopes of the dose-response curves for carcinogens which are extrapolated to zero for extremely small doses. These extrapolated slopes are commonly referred to as cancer slope factors (CSFs) and they are used for the toxicological component of the EPA's acceptable risk calculations. CSFs are maintained in the EPA's Integrated Risk Information System (IRIS) database.

Many papers have been published which comment upon the uncertainty of the EPA's CSFs. In addition, "the EPA is well aware of the problems associated with overly conservative risk estimates and has repeatedly stressed that the unit cancer risk estimate only provides a plausible upper limit for a risk that can very well be much lower. The problem is that, in reality, official EPA unit risk estimates are widely used , more or less, as absolute standards." (LaGrega et. al., 1994, p.280). Due to insufficient expertise in toxicology, this report will not offer an opinion concerning specific toxicological uncertainty of the EPA's CSFs.

Dose

In terms of dose calculations, it is important to understand the environmental pathway. Therefore, for this cancer risk evaluation it is important to identify the following:

- carcinogens
- source of carcinogens
- release mechanisms
- transport mechanisms
- transfer mechanisms
- transformation mechanisms
- exposure paths
- exposure point concentrations
- receptors

However, it is interesting to note that in performing an EPA risk assessment, only the carcinogens and the exposure point concentrations are used to calculate risk. Although the other seven above-referenced factors are essential for developing spatially distributed exposure point concentrations, EPA protocol requires maximum detect concentrations for maximum or upper bound risk calculations. In addition, EPA protocol requires arithmetic averaging of detect concentrations for mean risk calculations. That is to say, two sites with hazardous materials at similar concentrations with entirely different hydrogeologic conditions, would have the same risk according to EPA guidelines. However, at their discretion, EPA will review risk assessments which incorporate site-specific conditions into their calculations.

Identification of Hazardous Materials

Hazardous materials are broadly defined as non-carcinogens which are known to have harmful systemic effects upon humans, and carcinogens which have a propensity to initiate and promote cancer. Both terminal and "quality of life" health problems from exposure to hazardous materials are primary human health concerns. Because of these health concerns, human exposure to hazardous materials, especially carcinogens, is a source of risk and is of primary concern for risk assessment and management. However, for this report, only the carcinogenic materials identified in the MMR LF-1 are being evaluated for potential risk.

According to Boston University's School of Public Health Upper Cape Cancer Incidence Study which was prepared under contract to the Massachusetts Department of Public Health, cancer incidence rates for the MMR regional area have increased at a relative rate of approximately fifty six (56) percent overall (BUSPH, 1992). In addition, according to the Journal of the American Medical Association cancer incident rates are increasing steadily for the United States at a relative rate of approximately forty four (44) percent overall (JAMA, Vol. 271, No. 6, 1994). Furthermore, it is generally accepted that approximately twenty five (25) percent of all annual deaths in the US are caused by cancer. When the uncertainties presented in the above-referenced reports are taken into account, both the MMR cancer rate and the US cancer rate are very similar. Since these cancer rates are so similar, it is difficult to discern if the cancer rate increase at the MMR region is caused on account of reasons which are linked to the background national cancer rate increase, or if cancer rate increase near the MMR is tied to the release of carcinogenic materials at the MMR site.

Review Existing Reports

Part of this investigation was a comprehensive review of the RI, and the RAH which are relevant to risk assessment for the MMR LF-1. An examination of the methodology used, the consistency of the reports with respect to the EPA's regulatory guidelines, and independent spreadsheet calculations using the equations and numerical values which are cited in the above-referenced reports supplied similar results. This three part process confirmed the consistency of the reporting which has been provided to MIT to calculate risk and formulate risk opinions. As the MMR LF-1 is part of an on-going clean-up, new and updated data from the above-referenced reports has been included, as required, to present the most current EPA approved health risk connected with the MMR LF-1.

Uncertainty

In all statistically intensive calculations there are uncertainties specific to the numerical model which is being used. Since the EPA's model is the requisite regulatory guideline for Superfund sites, their model is the one which is being scrutinized. The EPA's deterministic model does not distribute uncertainty uniformly. When combined, concentration uncertainty and cancer slope factor (CSF) uncertainty account for approximately 97% of total risk uncertainty.

Approximately 80% - 95% of the total risk uncertainty is CSF uncertainty. (Hines, J.J. 1996) The EPA understands that their methods are statistically conservative and consequently will tend to overestimate risk, because the EPA incorporates policy constructs into risk quantification calculations. Basically, the EPA uses regulated risk assessment as opposed to probabilistic risk assessment coupled with regulations for risk management. Ultimately, risk regulated by the EPA is as uncertain as the EPA's CSFs. Recently, according to several major journals including the April 17, 1996 issue of the Wall Street Journal, the EPA has proposed policy changes for their assignment of CSFs. This should decrease the statistically localized risk uncertainty inherent within EPA regulated risk assessment calculations.

Results of Human Health Risk Assessment

CDM Federal performed a preliminary risk assessment of the LF-1 plume with no containment system in the *Remedial Investigation (RI): main base landfill and hydrogeologic region I study* (1995). The maximum cancer risk found for adult residents of the towns of Bourne and Falmouth in Cape Cod for future exposure to contaminated groundwater is 1.3E-03. This risk is interpreted as the incremental increase in probability of developing cancer above background level for each exposed resident. The United States Environmental Protection Agency (USEPA) acceptable risk standard ranges from 1.0E-06 to 1.0E-04. The standard is set independently for each site and case. The increased risk of 1.3E-03 for each resident is above the highest acceptable USEPA standard. In addition, the overall maximum Hazard Index (HI) for non-cancer risk from potential exposure to the contaminated groundwater is 39.5. The USEPA's acceptable HI standard for non-cancer risk is 1.0. Calculated HI that are above the USEPA standard pose possible non-cancer deleterious health effects to exposed populations. Thus, the current LF-1 plume poses cancer and non-cancer risks to adult residents of Bourne and Falmouth above the USEPA acceptable standard.

Operational Technologies, the main design contractor to contain contaminated groundwater plumes, has recommended a row of extraction wells along Route 28 of western Cape Cod as the strategy to contain the LF-1 plume (OpTech, 1996). The fence line of wells at Route 28 is designed to capture the landfill contaminated groundwater as the plume migrates westward to Buzzards Bay of Massachusetts. Current plume data which describes the spatial distribution of the contamination indicates that the leading edge of the plume has been detected passed Route 28 (OpTech, 1996). Since the proposed containment strategy will not capture this leading edge termed the "toe" of the plume, the detached plume of contaminated groundwater is expected to continue its migration and discharge into Buzzards Bay untreated. This containment strategy of extraction wells installed along Route 28 was proposed due to potential disturbance to the freshwater-saltwater interface along the coastline if the extraction wells are installed at the leading edge of the plume, and the possible difficulty of private property access.

Operational Technologies also performed a preliminary risk assessment of future potential effects to human health and ecological systems from this recommended plume containment system. The maximum cancer risk for adult residents in the towns of Bourne and Falmouth is 4.7E-04. This increased risk to adult residents from the detached contaminated groundwater plume is also above the USEPA acceptable standard. The overall maximum HI for non-cancer risk from exposure to the detached plume is 3.3. HI above the acceptable USEPA standard of 1.0 poses non-cancer deleterious health effects risk to exposed residents. The cancer and non-cancer risks posed by the detached plume are also above both USEPA standards.

A comparison of the preliminary risk assessment results indicate that the proposed containment system for LF-1 plume will reduce the maximum cancer and non-cancer risks posed by the contaminants of LF-1 plume, but both the cancer and non-cancer risks are still significant and above the acceptable USEPA standards. The results of the risk estimates clearly show that the containment strategy will still pose tangible risk to the potentially exposed population of western Cape Cod. Alternative containment and remediation systems need to be further investigated to reduce the risk to USEPA acceptable standards. The

USEPA sets acceptable risk standards to adequately protect human health and the natural environment.

Assessment of Risk from Ingestion of Contaminated Shellfish

From the current data of the LF-1 plume, the contaminants are projected to discharge into Red Brook, Squeteague, and Megansett harbors of Buzzards Bay (OpTech, 1996, CDM Federal, 1995). The shallow tidal flats of these harbors support a rich population of local shellfish species. Soft shell clams, quahogs (hard clams), oysters, bay scallops, surf clams, mussels, and conch which are harvested by local commercial and recreational fishermen. Since metals are part of the LF-1 plume contaminants and shellfish have been shown to bioaccumulate metals in their body tissue, the potential discharge of the plume into the harbors along the shoreline pose a risk to the coastal marine shellfish population as well as to human health from the consumption of tainted shellfish.

	Max. $C_{@,1}$	Max. $C_{@,2}$	Oral	Oral	Cancer	Cancer	Hazard	Hazard
	(ug/l)	(ug/l)	SF	RfD	Risk[1]	Risk[2]	Index[1]	Index[2]
Aluminum	20900	10200	NA	1	NA	NA	3.18217	1.55302
Antimony		2.6	NA	0.0004	NA	NA	0	0.98967
Arsenic	3.5	8.4	1.75	0.0003	0.00093	0.00224	1.77633	4.2632
Barium	400	107	NA	0.07	NA	NA	0.87004	0.23274
Beryllium	3.6	1.1	4.3	0.005	0.00236	0.00072	0.10963	0.0335
Cadmium	2	2	NA	0.001	NA	NA	0.30451	0.30451
Chromium#	54.2	66.3	NA	0.005	NA	NA	1.65047	2.01893
Copper	48.7	28.2	NA	NA	NA	NA	NA	NA
Cyanide	16.4		NA	0.02	NA	NA	0.12485	0
Iron	134,000	24000	NA	0.5	NA	NA	40.8049	7.30834
Lead	27.8	9.8	NA	NA	NA	NA	NA	NA
Manganese	5040	824	NA	0.14	NA	NA	5.48126	0.89614
Mercury	0.3*	0.3*	NA	0.0003	NA	NA	0.15226	0.15226
Nickel	24.4	184	NA	0.02	NA	NA	0.18575	1.40077
Vanadium	33	41	NA	0.007	NA	NA	0.71778	0.89179
Zinc	262	184	NA	0.3	NA	NA	0.13297	0.09338

Notes:

1 Derived from CDM Federal (1995)
2 Derived from OpTech (1996)
@ Maximum total concentration
# Chromium (VI) values are used
* Maximum dissolved concentration

SF = Cancer slope factor
RfD = Non-cancer reference dose
NA = Not available

Table A-2 Maximum cancer and non-cancer risk for each metal

The results of maximum cancer and non-cancer risk assessment of consuming contaminated quahogs over a life time are calculated for each metal in Table A-2. The maximum concentration of metals detected in well samples from the LF-1 plume are derived from the reports of CDM Federal (1995) and OpTech (1996). The oral cancer slope factors (SF) and non-cancer reference doses (RfD) of the metals are obtained from the Risk Assessment Handbook for MMR published by Automated Sciences Group (1994). Using the CDM Federal (1995) data, the maximum cancer risk from consumption of tainted quahogs is 3.3E-03. This risk is interpreted as the incremental increase in probability of developing cancer above background level for each exposed resident. The United States Environmental Protection Agency (USEPA) acceptable risk standard ranges from 1.0E-06 to 1.0E-04. The standard is set independently for each site and case. The increased risk of 3.3E-03 for each exposed resident is above the highest acceptable USEPA standard. A maximum cancer risk of 3.0E-03 is calculated when maximum concentration of metals from OpTech (1996) data is used in the assessment. The cancer risk for humans from consumption of tainted quahogs are derived from only two metals - arsenic and beryllium - since these are the only metals with published cancer slope factors.

The overall maximum hazard index (HI) for non-cancer risk from potential exposure to the contaminated quahogs are 55.5 and 20.1, when CDM Federal (1995) and OpTech (1996) data, respectively, are used in the assessment. The USEPA's acceptable HI standard for non-cancer risk is 1.0. Calculated HI that are above the USEPA standard pose possible non-cancer deleterious health effects to the exposed population. The maximum cancer and non-cancer risks from contaminated quahogs are summarized in Table A-3.

	Maximum Cancer Risk	Maximum Hazard Index
CDM Federal Data	3.3E-03	55.5
OpTech Data	3.0E-03	20.1

Table A-3 Total maximum cancer and non-cancer risks from consumption of tainted quahogs

The risk assessment results show that both cancer and non-cancer risks are above the USEPA standards. The USEPA risk standards are set at levels that adequately protect human health and the natural environment. The calculated risk results indicate that tainted quahogs from the coastal harbors where LF-1 plume is predicted to discharge pose significant risk to consumers of shellfish from these harbors. The calculated risk estimations are based on worst case assumptions. Thus, the risk is a conservative estimate and indicates the maximum risk posed to human health. From these results, it is recommended that a monitoring program for shellfish harvested from Red Brook, Squeteague, and Megansett harbors be implemented.

Qualitative Assessment of Potential Ecological Risk

Since quahog clams are predicted to bioaccumulate metals, the discharge of the LF-1 groundwater plume into Red Brook and Megansett harbors is likely have detrimental effects to the coastal ecological system. Quahogs are a food source for certain marine species that reside in the coastal harbors of Buzzards Bay. The contamination of the quahog clams can potentially reduce the population thus triggering a decline in the population of marine species that depend on quahogs as their sole food source. The decline of key species in the ecosystem can lead to an overall decline of the whole ecosystem.

The bioaccumulation of metals by the quahog clams can also have detrimental effects on the ecosystem in a separate way. Since quahog clams are not at the top of the shoreline ecosystem food web, they are consumed by higher order food chain species. In this process of nutrient transfer up the food chain, contaminants accumulated within lower food chain organisms are also transferred up the food web. Thus, tainted quahogs clams can potentially transfer toxic metals to higher food chain species. The bioaccumulation of metals in the higher order organisms can also lead to the decline of particular population of species and the ecosystem as a whole.

Public Perception: Management of Public Interaction at the MMR

An analysis of the approaches used to manage public interaction at the Massachusetts Military Reservation was undertaken to characterize the evolution of public perception of risk posed by past activities at the MMR. Public meetings at the MMR between January 15 and March 31, 1996, were attended. In addition, a comparison of management approaches at other bases was carried out. This included interviewing personnel at military bases in California and Arizona. As part of the analysis, suggestions future approaches at IRPs were explored. This included the design of public opinion surveys to be carried out early in the IRP process. Other suggestions for future approaches are also presented.

Public Perception in Superfund Cleanup

In any scenario where pollution is an issue, there is frequently a gap between the perceived risk to human health and the actual risk posed by contamination. Because of scientific uncertainty in risk assessment, often times, the actual risks are not known, and so the perceived level of risk results from speculation by many parties. In the siting of hazardous waste facilities, the potential threat to human health results in the NIMBY ("Not in my backyard") syndrome. Often times this "potential threat" is a perceived one. Public interest groups have fought many a facility siting and won, not due to actual risk, but because of a perceived one. In Superfund cases, unlike potential hazardous waste facility sitings, contamination has already occurred, but there is still a question of whether the contamination poses a real threat to public health. The gap between actual and perceived risks in this case results in the answer to the question of "how clean is clean?" becoming a policy, rather than a scientific, one. Groundwater contamination at the Massachusetts Military Reservation Superfund site is perceived to be a problem, and steps are being taken to remediate this problem to the greatest extent feasible. Public opinion has defined "the greatest extent feasible" as the level to which groundwater is treated to "non-detect" levels for contaminants that pose threats to human health. In private sector cases, economics would figure into the calculation of feasibility of cleanup, but in the case of the MMR, where an entity as large as the federal government is funding the cleanup, the public believes that "anything is affordable" and therefore feasible.

History of Public Involvement at the MMR

The initial approach to management of public interaction surrounding the Installation Restoration at the MMR was similar to the "compliance-based" approach many companies take towards environmental regulation--the National Guard Bureau met only the minimum requirements necessary. Actions taken by the NGB were reactive rather than proactive. The NGB promulgated press releases and sent reports to local libraries, as well as holding news conferences after technical meetings, but any actions beyond that were minimal. Technical meetings concerning IRP activities were closed to the public and media, and virtually no public information meetings were held.

During 1990 and 1991, there was a modest effort to increase public involvement in the cleanup at Otis, as the IRP office at the MMR was created to manage the program locally rather than from far away. The "Joint Public Involvement Community Relations Plan" was presented, bi-monthly public information meetings were initiated, site tours/briefings were made possible, a site mailing list was created, and the IRP office began to print quarterly fact sheets that described the IRP activities. Although these fact sheets were limited in scope, they, along with the public information meetings, represented the first real effort to inform the public about specific activities associated with the IRP.

Late-1991 marked a major change in the way public interaction was managed at the MMR. The IRP office began updating technical reports much more frequently, and progress reports were made available to all interested parties. The local IRP office began educating the public by participating on local radio/cable TV programs as well as taking part in neighborhood association meetings. An educational display was created for to be used at these meetings and at libraries, and detailed bi-monthly fact sheets were developed. In addition, all technical meetings were opened to the public and media.

The post-1991 period also has included the creation of many committees that assist the cleanup activities at the MMR. These committees, called "process action teams", are made up of personnel from the MMR, the relevant regulatory agencies, and the public. These process action teams (or "PATs") report to the senior management board, which was created to oversee the restoration. Presently, a total of 8 community working groups hold regular meetings (Karson, 1995). Although the public is highly involved in the IRP process at this point, how much influence the public actually has in the decision-making process is still a question.

point, how much influence the public actually has in the decision-making process is still a question.

Design Of Future Approaches At The MMR And Elsewhere

There are several things that should be considered before an Installation Restoration Program is initiated at a particular base or military reservation. Not the least of these is the management of public interaction surrounding the restoration. Public and public interest group opinion are very likely to polarize as soon as contamination and threat to public health are made known. Public distrust of government, especially on the federal level, compounds the fear that public health is in danger and contributes to the belief that any cleanup activities will be inadequate to alleviate the problem of contamination.

There are steps that can be taken to minimize the potential for adversarial relationships developing between all interested parties in base cleanup. Since the public has been involved in the restoration process at the MMR, the relationships between all interested parties have become less of a barrier to cleanup as all parties are seen to have input into the process. However, analysis of the approach used to manage public interaction at the MMR shows that, even though outwardly it appears that all the "right" approaches were taken, public concern is still an issue. This is due to the fact that early on in the MMR IRP

B. REMEDIAL APPROACHES

SOURCE CONTAINMENT

Introduction
As part of remediation operations at MMR, several of the cells at the Main Base Landfill have recently been secured with a final cover system. These cells include the 1970 cell, the post-1970 cell, and the kettle hole. The remaining cells (1947, 1951, and 1957) have collectively been termed the Northwest Operable Unit (NOU). Remedial investigation as to the necessity of a final closure system for these cells is ongoing. This proposal is focused on the design of a final closure system for the 1951 cell. The landfill final closure requirements of the Resource Conservation and Recovery Act (RCRA) and Massachusetts Solid Waste Management Regulations will be examined and adapted to site specific conditions. Material and design options for the components of the cover system will be examined and choices made according to performance, availability, and relative cost, as applicable to site-specific conditions. A cross-section of the proposed cover system is provided in Figure A-13.

Regulatory Review
Massachusetts Solid Waste Management regulations specify the following as minimum design requirements for a landfill final closure system (MA DEP, 1993):

- Subgrade layer

- Venting layer with minimum hydraulic conductivity of $1X10^{-3}$ cm/sec

- Low conductivity layer with minimum thickness of 18 inches and maximum hydraulic conductivity of $1x10^{-7}$ cm/sec, or an approved flexible membrane liner (geomembrane)

- Drainage layer with minimum thickness of 6 inches and minimum hydraulic conductivity of $1x10^{-3}$ cm/sec, or a synthetic drainage net (geonet)

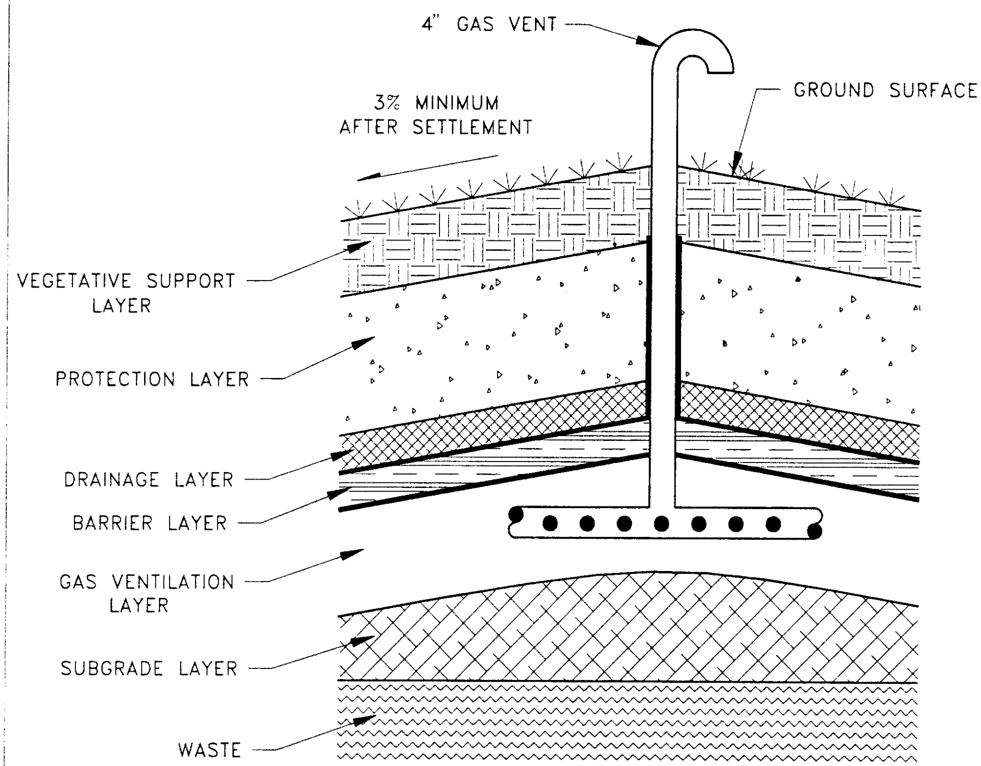
- Combined vegetative support / protection layer of minimum thickness 18 inches, with at least 12 inches of soil capable of supporting vegetation.

Subparts G, K, and N of the Resource Conservation and Recovery Act (RCRA) Subtitle C (Hazardous Waste Management) regulations dictate the requirements for hazardous and mixed waste landfill cover systems (US EPA, 1991). The EPA recommends that a final cover system consist of the following (US EPA, 1991):

- A low hydraulic conductivity geomembrane / soil layer consisting of a 24 inch layer of compacted natural or amended soil with a hydraulic conductivity of $1x10^{-7}$ cm/sec in intimate contact with a geomembrane liner of minimum thickness 0.5 mm (20 mil).

- A drainage layer of 12 inch minimum thickness having a minimum hydraulic conductivity of $1x10^{-2}$ cm/sec, or a geosynthetic material of equal transmissivity.

- A top vegetative support / soil layer consisting of a top layer with vegetation or an armored surface, and a minimum of 24 inches of soil graded at a slope between 3 and 5 %.



NOT TO SCALE

LAYER	MATERIAL	SPECIFICATIONS
Vegetctive Support	Loam	6" thickness
Protection	Borrow Soil	18" thickness
Drainage	Geonet	Transmissivity > 3×10^{-5} m^2/sec
Barrier	Geomembrane over Geosynthetic Clay Liner	Geomembrane: 60mil VLDPE Geosynthetic Clay Liner: Gundseal with 40mil VLDPE Substrate
Gas Ventilation	Screened Borrow Soil	12" thickness Hydraulic conductivity > 1×10^{-3} cm/sec

Figure A-13: Cross-Section of Proposed Cover Design

Subgrade Layer

The subgrade layer acts as a foundation for the overlying layers of the cap, and it is also used as a contouring layer to create the appropriate final slope of the cover system. It is recommended that the foundation layer be placed to provide a final grade (after settlement) no greater than 5% and no less than 3%. This slope range provides sufficient grade to promote some surface water runoff while not being so steep as to produce erosion of the surficial soils. Allowance must be made for waste settlement that will occur as a result of the vertical stresses imposed by the weight of the cover materials.

Materials typically utilized for foundation layers include a variety of soils, and some acceptable wastes. At sites such as MMR where soil borrow volumes are relatively plentiful, soil is the obvious choice for the foundation layer. Results of on-site borrow characterization tests (ABB, 1993) have revealed that this material is acceptable for use in the foundation layer. The material is classified as a fine-to-medium sand with trace-to-some fine-to coarse gravel (ABB, 1993). This material has a relatively low fines content and has acceptable compressibility characteristics, therefore it is recommended for use in this layer. The subgrade should be placed in lifts of approximately 8 inches and compacted by 4 to 6 passes of a typical sheepsfoot roller. This placement procedure should result in compaction to approximately 90% of the maximum dry density.

Gas Ventilation Layer
The gas venting layer is a permeable layer containing piping for the collection and venting or recovery of gases produced from waste degradation. Based on the cell composition (predominantly burn-fill), the moist, aerobic conditions provided by the intermediate cover, and the time since placement (over 40 years) it is concluded that gas generation rates at the 1951 cell will be low. Consequently, a passive gas venting system is recommended. It is recommended that material from the "lower layer" of the borrow area be utilized for the ventilation layer. The soil must be screened on a 3/8 inch sieve prior to placement, and then placed with a light machine in a single lift with no further compaction efforts. To collect the gas, PVC collector pipe is bedded in the sand and run laterally along the slope. To vent the gas to atmosphere, it is recommended that a total of ten ventilation risers be installed and spaced equidistantly. Flexible (to accommodate loading and settlement) 4 inch perforated PVC is recommended for the collector pipe, and 4 inch non-perforated rigid PVC is recommended for the risers.

Hydraulic Barrier Layer
The barrier layer is designed to minimize the percolation of water through the cover system directly by impeding infiltration and indirectly by promoting storage and drainage of water in the overlying layers and eventual removal of water by runoff , evapotranspiration, and internal storage (Geosyntec, 1994). This design proposal recommends a composite geomembrane over geonsynthetic clay

liner (GCL) as the hydraulic barrier layer. The specified geomembrane is a 60 mil (1.5 mm) textured very low density polyethylene (VLDPE), and the specified GCL is a Gundseal® GCL with a 40 mil (1.0 mm) textured VLDPE substrate placed bentonite-side up.

Drainage Layer

The drainage layer functions to remove water which infiltrates the vegetative support/protection layer. It should be designed to minimize the standing head and residence time of water on the barrier layer in order to minimize leachate production (US EPA, 1989). The recommended drainage layer for this design is an extruded solid rib geonet with factory bonded nonwoven, heat-bonded geotextile on both faces. The composite drainage layer must have a minimum transmissivity of 3×10^{-5} m^2/sec.

Surface Layer

The top layer of the cover system is actually comprised of two separate layers; the lower layer termed the protection layer and the upper layer termed the surface layer. On-site or local soil is the most commonly used and typically the most suitable material for the protection layer. Suitable on-site materials are available for use in the protection layer. The on-site borrow materials have been classified as a fine-to-medium sand with trace-to-some fine-to coarse gravel (ABB, 1993). This material has a relatively low fines content and a low organic content, therefore it is acceptable for use in the protection layer. The borrow material should be placed to a thickness of 18 inches using a small dozer with low ground-pressure to protect the underlying cover components. Compaction beyond that which occurs during placement is not necessary.

Vegetation is specified as the surface layer cover, consequently the surface layer will be designed for vegetative support. The on-site borrow material is not well suited to supporting vegetation, therefore it is recommended that loam be imported from an off-base supplier and placed to a thickness of 6 inches. A warm season grass mix is specified as the vegetative cover. Periodic mowing and inspection of the vegetative cover are recommended as part of the Postclosure Program.

Conclusions

It is concluded that this cover system, if constructed with appropriate construction quality assurance / quality control, will satisfy the primary objective of containing the source of pollution, thus minimizing further contamination of groundwater by the waste fill. The composite geomembrane / geosynthetic clay liner barrier layer is theoretically nearly impermeable. Estimates of the hydraulic conductivity of VLDPE geomembranes are on the order of 1×10^{-10} cm/sec (Koerner, 1994), and estimates of the hydraulic conductivity of Gundseal® GCLs are on the order of 1×10^{-12} cm/sec (Eith et al., 1991). Essentially all infiltration that does occur through such a composite barrier is the result of defects from manufacturing and / or construction processes.

Theoretical performance of the cover was evaluated using the Hydrologic Performance of Landfill Performance (HELP) computer model (Schroeder et al., 1994). HELP is a quasi-two-dimensional, deterministic, water-routing model for determining water balances (Schroeder et al., 1994). HELP predicted 0.000000 inches of annual percolation through the barrier layer. Clearly, this prediction is unrealistic as no cover is absolutely impermeable. Because the performance of the cover system is so closely linked to construction QA/QC, it is very difficult to make an accurate estimate of anticipated infiltration through the barrier layer. It is accurate to state, however, that if this proposed cover system is constructed with appropriate QA/QC, it will meet and exceed the regulatory performance specifications. To accurately monitor the performance of the cover system, it is recommended that the downgradient groundwater quality be closely monitored before and after cover construction to reveal contaminant concentration trends indicative of cover system effectiveness.

While the primary objective of the cover system is to minimize infiltration into the waste fill, there are several other significant performance criteria which must be satisfied. Given the site-specific conditions, the cover system must also:

* isolate the waste from humans, vectors and other animals, and other components of the surrounding ecosystem
* control gases generated within the waste fill
* be resistant to erosion by wind and water

* be resistant to static and seismic slope failures

* be durable, maintaining its design performance level for 30 years (regulatory) or the life of the waste fill (prudent)

* control surface water runoff and lateral drainage flow in a manner which does not promote erosion and does not adversely impact the surrounding environment

These criteria are satisfied by the proposed cover design. The waste is well isolated from the surrounding ecosystem by a total of over 5 feet of soil. Any gases produced by the waste will be vented to atmosphere to prevent explosive conditions from occurring within the waste layer. Additionally, atmospheric monitoring is included as part of the post-closure program to ensure that vented gases do not violate Clean Air Act standards and to ensure that no gas migrates off-site. The cover is designed to be erosion-resistant. The surface is graded to a moderate slope, seeded with an appropriate grass mixture, and covered with straw mulch. Surface water runoff and lateral drainage flow are handled by a network of open channels and culverts which divert flow to specified recharge areas in a controlled manner which also assists in erosion control. The cover system is also resistant to static and seismic slope failure. The minimum static factor of safety of the proposed cover system is 3.1, the minimum seismic factor of safety is 1.0. The recommended minimum factors of safety are 1.5 and 1.0 respectively. It should be noted that it is relatively rare to have a cover design satisfy the seismic stability safety factor in a seismically active area such as Cape

Cod. The issue of durability is not so clearly satisfied, in the author's opinion. Relatively little research on the long-term durability of geosynthetics in landfill covers has been performed, and since the history of geosynthetics in cover systems is fairly short, there are few, if any, case studies of sufficient length (e.g., over 30 years) to fill the data gap. However, the research that has been performed indicates that a cover system is an environment which is relatively conducive to geosynthetic survivability (Koerner et al., 1991). In a cover, the geosynthetics are not exposed to toxic chemicals, they are isolated from ultraviolet radiation, and they are fairly well protected from the effects of freeze/thaw cycles. Thus, it seems likely that the cover system will maintain its integrity well into the future.

In summary, it is the author's contention that the proposed cover system will adequately contain the source of the LF-1 plume. If constructed with appropriate construction QA/QC, the proposed cover system design will provide a nearly impermeable barrier while also controlling lateral drainage flow, surface runoff, and decomposition gases with a stable, durable design that should maintain its integrity for decades.

Bioremediation

Bioremediation of the LF-1 plume has been considered as a potential remedial action for the site, but a comprehensive plan has yet to be proposed (ABB Environmental, 1992). Conventional enhanced bioremediation systems stimulate microbial degradation by amending groundwater from the aquifer with oxygen and nutrients and recirculating it through the contaminated area (O'Brien & Gere Engineers Inc., 1995). The immense size of the LF-1 plume would necessitate the pumping and recirculation of hundreds of millions of gallons of water in order to ensure the removal of all of the chlorinated solvents. This plan would not only be prohibitively costly, it would also be ineffective because the plume contains PCE which cannot be aerobically degraded (Pavlostathis and Zhuang, 1993).

In order to solve the technical problems associated with a traditional enhanced bioremediation action, a passive anaerobic/aerobic system can be used. This system would consist of two groups of horizontal injection wells which are driven into the aquifer at a depth just below that of the plume . The wells would be driven across the width of the plume and have thousands of small injection ports along the top of each one. The ports are used to inject gases into the aquifer in order to stimulate the microbes which will degrade the plume contaminants. Each set of wells will form a distinct biozone above it. The first biozone will be anaerobic and will treat the PCE in the plume, while the second biozone will be an aerobic treatment phase which will remove the remaining chlorinated solvents. This system has a significant advantage over traditional systems because it is a flow-through system; the gas is injected below the plume where it can rise up into the contaminated water and stimulate microbial activity as the plume flows over the gas injection wells. This significantly reduces the pumping costs associated with a more traditional bioremediation system.

The LF-1 plume contains significant quantities of PCE which can only be degraded anaerobically because methanotrophic bacteria possess a monooxygenase enzyme which cannot oxidize a fully chlorinated ethene molecule (Semprini, 1995). Therefore, the first stage of the system must be designed to turn the system anaerobic so that anaerobic bacteria can utilize the PCE in the plume in the process of reductive dechlorination. PCE is an oxidized chemical species while organic matter is relatively reduced. Reductive dechlorinating bacteria use the PCE as a chemical oxidant in a redox reaction with organic matter in order to obtain energy to function and grow (Hollinger et al, 1993). In the process, one or more chlorines are removed from the PCE and replaced with hydrogen. This renders the PCE susceptible to aerobic attack.

In order to turn the aquifer anaerobic, methane and air are injected at the first biozone. This injection serves a threefold purpose. Methanotrophs utilize the methane for growth and deplete the oxygen in the plume as it flows past the well. In addition, the methanotrophs will also degrade some of the TCE and

DCE in the plume since their monooxygenase enzymes can degrade the solvents as well as methane (Semprini, 1995). Finally, as methane is utilized by the methanotrophs for growth, biomass will be accumulated in the region above the treatment well. This biomass will then be used by methanogenic bacteria to fuel the process of reductive dechlorination of PCE within the plume.

Once the oxygen is depleted from the plume, the first biozone will be anaerobic. It will remain anaerobic since there will be little or no vertical mixing with oxygenated recharge water (Domenico and Schwartz, 1990). Furthermore, oxygen will be depleted from the plume as it flows into the biozone by periodic injections of methane. Bacteria in this anaerobic zone will utilize the dead biomass and reductively dechlorinate the solvents in the plume. This is a slow biological process; based on laboratory batch studies and the temperature and pH of the aquifer the biozone needs to produce at least five milligrams per liter of biomass and it should take about 540 days to achieve extensive removal (greater than 99 percent) of the PCE in the plume . Given a PCE migration rate within the plume of .9 ft per day and a treatment zone of two hundred feet associated with each horizontal well, three six-thousand foot horizontal wells will need to be installed to create the first biozone. Some of the TCE and DCE in the plume will also be dechlorinated within this area, rendering all of the chlorinated solvents in the LF-1 plume more susceptible to treatment by aerobic degradation.

The second biozone will be an aerobic zone that will be used to degrade the bulk of the chlorinated solvents in the plume. Gaseous methane, air, nitrous oxide, and triethyl phosphate will be injected into the aquifer (Skiadas, 1996). Methanotrophs will feed on this and will also degrade the solvents in a process termed cometabolic oxidation. One horizontal well must be used to produce the aerobic biozone which will achieve a ninety-five percent reduction in the concentration of TCE and ensure total remediation of DCE and VC . This level of remediation is more than sufficient to ensure that federal MCLs for the pollutants in the LF-1 plume are not exceeded in private drinking wells in the path of the plume.

It is apparent that the enhanced bioremediation system proposed above has the potential to effectively remediate the chlorinated solvent plume emanating from the main base landfill at the MMR on Cape Cod. The system would be difficult to manage and expensive to emplace, but it does offer many cost advantages over other remediation or containment schemes because it does not involve pumping large volumes of water or treating contaminated groundwater with granular activated carbon to remove the chlorinated organics. However, this type of system has never been used in the field so a pilot-scale study should be conducted at a smaller site to ensure that the concept works and is cost-effective. If this test produces positive results, then a sequential anaerobic/aerobic enhanced bioremediation system of this nature could be used to clean up the LF-1 plume.

REFERENCES

ABB Environmental Services, Inc., "Installation Restoration Program, Massachusetts Military Reservation, Interim Remedial Investigation, Main Base Landfill (AOC LF-1)," Portland, Maine, March 1992.

ABB Environmental Services, Inc., "Technical Support for Closure Plan for Study Area LF-1," Installation Restoration Program, Massachusetts Military Reservation Cape Cod, Massachusetts, January 1993.

Alyamani, M.S., and Z. Sen, "Determination of Hydraulic Conductivity from Complete Grain-Size Distribution Curves," *Ground Water*, v. 31, no. 4, p. 551-555, 1993.

Automated Sciences Group, Inc., Hazardous Waste Remedial Actions Program, and Oak Ridge National Laboratory, *Risk Assessment Handbook*, Oak Ridge, Tennessee, 1994.

Bradbury and Muldoon, "Hydraulic Conductivity Determinations in Unlithified Glacial and Fluvial Materials," ASTM STP 1053, *Ground Water and Vadose Zone Monitoring*, 1990.

Cape Cod Commission, "Cape Trends, Demographic and Economic Characteristics and Trends, Barnstable County - Cape Cod," 1996.

CDM Federal Programs Corp., "Remedial Investigation Report Main Base Landfill (AOC LF-1) and Hydrogeological Region I Study," Boston, Massachusetts, April 1995.

Domenico, Patrick A., and Franklin W. Schwartz, *Physical and Chemical Hydrogeology*, John Wiley & Sons, New York, NY, 1990.

Eith, A. W., Boschuk, J., and Koerner, R. M., "Prefabricated Bentonite Clay Liners," in *Proceedings of the 4th GRI Seminar on the Topic of Landfill Closures - Geosynthetics, Interface Friction and New Developments*, Geosynthetic Research Institute, Philadelphia, 1991.

Garabedian, S. P., D. R. LeBlanc, L. W. Gelhar, and M. A. Celia. Large Scale Natural Gradient Tracer Test in Sand and Gravel, Cape Cod, Massachusetts, *Water Resources Research*, 27(5), 911-924, 1991.