GROUNDWATER FLOW AND MASS TRANSPORT MODEL
OF THE MAIN BASE LANDFILL SITE AT THE
MASSACHUSETTS MILITARY RESERVATION

by

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ABSTRACT

This study was undertaken to develop a 3-D groundwater flow and mass transport
model of that part of the western Cape Cod aquifer which affects the contaminant
plume emanating from the Main Base Landfill at the Massachusetts Military
Reservation. The modeling effort was part of a larger research project designed to
characterize the environmental impacts of groundwater contamination resulting from
the disposal of organic solvents and other wastes in unlined trenches and a kettle hole
at the landfill site.

A finite element groundwater flow model of the region was first developed using the
DYNFLOW software. The model was calibrated to observed water table elevations in
the study area. The calibrated flow model was then used to run the DYNTRACK
particle tracking procedure and simulate the release of contaminants from the landfill
site. The simulated plume was then calibrated to observations of groundwater
contamination.

Contaminant migration in the study area was found to be highly sensitive to the
geologic properties of the Buzzards Bay Moraine (BBM). The observed divergence of
the plume into two distinct lobes may be the result of differential travel velocities due to
a layer of low permeability material in the BBM. Contaminants migrated to the ocean 50
years after first release. A simulation time of 165 years was required to flush all
contaminants from the aquifer into Buzzards Bay after a successful source remediation
in the absence of groundwater containment and cleanup. Simulating a successful plume
containment and remediation scheme immediately west of Route 28 indicated a
probable flushing time of 12 years for the downgradient portion of the plume that is not
extracted. The possibility of a sinking pool of dense landfill leachate causing the deep
northern plume lobe was found to be unlikely given the manner in which the regional
geologic structure was represented in this model.

Thesis Supervisor: Dr. Lynn W. Gelhar
Title: William E. Leonhard Professor of Civil and Environmental Engineering.
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This thesis would not have seen the light of day if not for Bruce Jacobs, who told me why my plume was hell-bent on heading south and in general showed us how to use the DYN System software. I wish to thank the members of the LF-1 group, Dan Alden, Mike Collins, Karl Elias, Jim Hines, Ben Jordan and Robert Lee, who contributed to writing the group results and background sections of this thesis. Dan Alden, Enrique López-Calva, Alberto Lázaro, Vanessa Riva, and Mitsos Triantopoulos helped me unstintingly each time I had a question or needed assistance on modeling. I also thank Prof. David H. Marks and Shawn Morrisey, who made my Master of Engineering experience a smooth and memorable one. My gratitude extends to Mr. Ed Pesci and others at the MMR who were very generous with their time, data and documentation. I am thoroughly grateful to all the M. Eng. students for their friendship and support, they helped make my time here survivable.

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1. INTRODUCTION

This thesis describes a study undertaken to develop a three dimensional groundwater flow model and particle transport simulation of a plume of contaminants emanating from the Main Base Landfill (LF-1) at the Massachusetts Military Reservation (MMR), Cape Cod, Massachusetts.

1.1 CURRENT SITUATION

The LF-1 site is located about 2 miles from the western and southern borders of the MMR. The site covers approximately 100 acres and has functioned as the primary solid waste disposal facility at the MMR. As a result of this activity, a large plume of contaminants, primarily volatile organic compounds (VOCs), has developed downgradient of the Main Base Landfill in the underlying aquifer. There is considerable concern that this contaminated groundwater may have adverse human and ecological impacts.

The United States Environmental Protection Agency placed the MMR on the Superfund National Priorities List (NPL) in 1989. An extensive amount of data on contamination has been collected and is maintained by the MMR Installation Restoration Program (IRP), which acts as the principal agent of the U.S. government on behalf of the MMR. Numerous reports have been generated for the IRP which include data observations and professional opinions. The most current information on the LF-1 site and related groundwater plume according to the IRP is outlined as follows (Op-Tech, 1996).
I. Three of the six landfill cells have been capped with a final closure system. A similar closure system is being considered for the remaining cells.

II. The LF-1 plume consists of two main sections, referred to as the northern and southern lobes. The northern lobe has advanced further west than the rest of the plume, with the leading edge at Buzzards Bay. The leading edge of the southern plume is located about 600 ft west of Route 28.

III. The northern lobe is thought to be separated vertically into two plumes, with the deeper detached plume section located nearer to Buzzards Bay than the main northern lobe. The detached section is referred to as the deep LF-1 northern lobe.

IV. Primary plume constituents are the volatile organic compounds trichloroethylene (TCE), tetrachloroethylene (PCE), 1,1,2,2-tetrachloroethane (1,1,2,2-PCA), carbon tetrachloride (CCL4), Vinyl Chloride (VC), and ethylene dibromide (EDB).

V. An extraction well system is planned along Route 28, with all plume constituents west of the well fence at the time groundwater extraction begins being allowed to flush from the aquifer into Buzzards Bay.

1.2 THESIS CONTEXT AND SCOPE

The groundwater modeling project discussed in this thesis was conducted as part of a larger research project undertaken to characterize the environmental impacts of groundwater contamination at the LF-1 site. The group project examines the potential impacts of the LF-1 contamination on public health and welfare and how these effects
can be mitigated. The scope of the group research project includes site characterization and groundwater modeling, risk assessment, management of public interaction, study of source containment, and bioremediation technology. The group project results are included as Appendix A.

The DYN System modeling package developed by Camp, Dresser, and McKee, Inc. (1992), is used to devise the groundwater flow and mass transport model. The first step in an undertaking of this type is to determine the scope of the investigation. The goals of this modeling project are as follows,

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

II. Track particles released from a continuous source 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 prominent hydrogeologic 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.

Once all the questions that need to be addressed at the study site are identified, a suitable analytical model can be developed to provide the necessary answers. The actual plume simulation effort can be sub-divided into three distinct parts, as follows.

I. Represent the geologic setting in the model as accurately as possible. This involves identifying and representing the heterogeneity and anisotropy of the regional sediments and accurately delineating the hydrologic boundaries of the study area.
II. Identify the inputs (recharge) and outputs (discharge) of the study area, and then solve the groundwater flow model. Then, iteratively develop solutions that approximate the observed head values in the study region.

III. Use the results of the calibrated flow model to calculate contaminant pathlines and thus the plume migration patterns. This step can be repeated as needed to observe the sensitivity of the model to the location, size and nature of the source.

Chapter 2 of this thesis contains a description of the site and background information on the MMR in general and Main Base Landfill in particular. A detailed discussion of the groundwater flow model and particle tracking procedure developed for the LF-1 site is included in Chapter 3. Chapter 4 contains conclusions and a discussion of the modeling project results. The results of the overall LF-1 group research project are included as Appendix A. Appendix B contains DYN system input and output files from the models described in this thesis in a 3.5" diskette as well as a brief description of each file.
2. HISTORY AND BACKGROUND OF THE LF-1 SITE

2.1 WESTERN CAPE GEOGRAPHY AND LAND USE

The Massachusetts Military Reservation (MMR) is located in the northwestern portion of Cape Cod, Massachusetts, covering an area of approximately 30 square miles (ABB, 1992). See Figure 2-1 for regional and base maps of the area. Military use of the MMR began in the early 1900’s, and may be 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.

The area of the present study is the Main Base Landfill site, termed LF-1 by the MMR Installation Restoration Program. The landfill is about 10,000 feet from the western and southern MMR boundaries and occupies approximately 100 acres. The landfill has operated since the early 1940’s as the primary waste disposal facility at MMR (CDM Federal, 1995). Unregulated disposal of waste at LF-1 continued until 1984, at which time disposal began to be regulated by the Air National Guard.

Waste disposal operations at LF-1 took place in five distinct disposal cells and a natural kettle hole, as shown in Figure 2-2. These are termed the 1947, 1951, 1957, 1970, post-
Figure 2-1 Site location map of MMR and western Cape Cod (From ABB, 1992).
Figure 2-2 Photogrammetric Map of the Landfill Layout (From ABB, 1993).
1970, and kettle hole cells. The date designations indicate the year in which disposal operations ceased at each particular cell. Accurate documentation of the wastes deposited at LF-1 does not exist. The wastes may include any 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, 1993).

Wastes were deposited in linear trenches, and covered with approximately 2 feet of native soil. Waste depth is uncertain, but estimated to be approximately 20 feet below the ground surface on average. Waste disposal at the landfill ceased in 1990. A plume of dissolved chlorinated volatile organic compounds, primarily tetrachloroethylene (PCE) and trichloroethylene (TCE), has developed in the aquifer downgradient of the landfill. This contaminant plume, referred to as the LF-1 plume, and has caused widespread concern due to its potential for impacting public and private water supply as well as the surrounding ecological system.

2.2 MMR'S LISTING ON THE NATIONAL PRIORITIES LIST

The MMR is one of 1,236 sites that have been placed on the National Priority List (NPL) by the U.S. Environmental Protection Agency (EPA). NPL sites are those to which the EPA has given particularly high human health and environmental risk ranking. NPL
Rankings are determined from an evaluation of the relative risk to public health and the environment from hazardous substances identified in the air, water and geologic surroundings local to a site. Once placed on the NPL, sites are targeted for remedial clean-up financed by the Superfund, which is the federal government's fiduciary and political device for remediating hazardous waste sites. Additional funding for cleanup is provided by potentially responsible parties (PRPs), those individuals and organizations whose activities have resulted in contamination.

2.3 PRESENT ACTIVITY AT THE LF-1 SITE

Due to the health and environmental risks which have been attributed to the contamination of soils and groundwater at the MMR, federal activity is underway to further quantify and reduce, to the extent required, the risk imposed upon human health and the environment. As part of remediation operations at MMR, three of the landfill cells have recently been secured with a final cover system. These cells are 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 as to the necessity of a final closure system for these cells is ongoing. Other IRP activities associated with the LF-1 site include design of a plume containment system and further plume delineation and groundwater modeling.
2.4 POPULATION AND ECONOMY OF THE REGION

The four towns of interest with regard to the LF-1 plume on the western Cape are Bourne, Sandwich, Mashpee, and Falmouth. The total population of this area, according to the 1994 census, is 67,400. The area is mostly residential, with some small industry. A significant amount of economic activity is associated with restaurants, shops, and other tourism related industries. The total population of Cape Cod is estimated to triple in summer, when summer residents and tourists swell the total population. The total population on the Massachusetts Military Reservation has doubled in the last twenty years. Overall, Cape Cod has been one of the fastest growing areas in New England. In 1986, 27% of economic activity was attributed to retirees; tourism accounted for 26%; seasonal residents, 22%; manufacturing, 10%; and business services (fishing, agriculture, and other), 15%. The economy is currently experiencing a shift from seasonal to year-round jobs. (Cape Cod Commission, 1996). Cape Cod is also a site of intense recreational activity, particularly in the summer months. Such activities include boating, biking, fishing, hunting, hiking, and water sports.

Since there are no other significant sources of water supply on Cape Cod other than the groundwater system, the Cape aquifer has been designated a sole source aquifer by the United States Environmental Protection Agency. Prior to the discovery of groundwater contamination at the MMR, most towns on the Cape depended on town groundwater wells for their water supply. Furthermore, households not on public water supply use private wells to obtain water from the aquifer. The growing population of Cape Cod
and increasing recreational activity in turn has lead to a rising demand for water supply and extensive human exposure to surface and aquifer water. Therefore, any contamination of the Cape Cod aquifer system and related surface water bodies is cause for grave concern.

2.5 CLIMATE

The Cape Cod climate is categorized as humid and continental. Average wind speeds range from 9 mph from July through September to 12 mph from October through March. Precipitation is fairly evenly distributed, with an average of approximately 4 inches per month. Average annual precipitation is approximately 47 inches. There is very little surface runoff. Approximately 40% of the precipitation infiltrates the soil and enters the groundwater system (CDM Federal, 1995).

2.6 REGIONAL GEOLOGY AND GROUNDWATER SYSTEM

2.6.1 Geology

The Cape Cod Basin consists of material deposited as a result of glacial action during the Wisconsinian stage between 7000 and 80,000 years ago. Advancing glaciers from the north transported rock debris gouged from the underlying bedrock until reaching the southernmost point of advance at Martha’s Vineyard and Nantucket Island. The glacial action also resulted in a thin layer of basal till being deposited over the bedrock. The entire sedimentation process occurred as a sequence of glacial deposition, erosion and re-deposition. In later periods, the glaciers melted, receded, and reached a stagnation
point near the western and northern shores of Cape Cod. The remaining glacial till was
deposited there and formed the Buzzards Bay and Sandwich moraines. The present day
Sandwich Moraine is thought to be of glacio-tectonic origin, due to pro-glacial
sediments being thrust over older morainal deposits during a readvance of the Cape
Cod Bay glacier (Oldale, 1984).

The regional geology of the LF-1 study area can be classified into three main lithological
facies. These are the Buzzards Bay and Sandwich Moraines (BBM and SM), the Mashpee
Pitted Plain (MPP) and the Buzzards Bay Outwash (BBO). The geographic distribution
of these materials is shown in Figure 2-3. The Mashpee Pitted Plain consists of stratified
coarse to fine grained sands that were transported from the melting Buzzards Bay and
Cape Cod Bay ice sheets, and deposited over a bed of fine-grained glacio-lacustrine
sediments and basal till. The general trends in the glacial outwash deposits in terms of
grain size are coarsening upwards and fining north to south. The thickness of the coarse
material decreases north to south, as the distance from the outwash source increases
(Oldale, 1984).

The morainal sediments were deposited directly as the ice-sheets melted. Thus, these
deposits are not stratified like the MPP glacial outwash and are thought to occur in
layers of poorly sorted sediment-flow deposits and finer till material. These sandy
sediments overlie a fining sequence of sand, silt, clay and basal till. The unsorted glacial
till that comprise the BBM ranges in size from boulders to clays. This complex
heterogeneity leads to wide variations in observed hydrogeological parameters in the
moraine. A general trend of fining downwards in material size and attendant lower hydraulic conductivities is generally assumed to occur in the moraine sediments (Masterson and Barlow, 1994; Oldale, 1984).

The Buzzards Bay Outwash was deposited as a result of sedimentation between the retreating ice sheets and the newly deposited Buzzards Bay Moraine. BBO sediments are generally sand and gravel, and are considered to be stratified in the same manner as the MMP outwash, with a general trend of fining downwards.

The geologic structure described above lies atop a Paleozoic crystalline bedrock and thin layer of basal till. The bedrock contours in western Cape Cod range in depth from 70 to 500 feet below sea level (Oldale, 1969). The bedrock is of a much lower hydraulic conductivity than the surrounding sediments, and therefore acts as an impermeable barrier to groundwater flow and forms the bottom boundary of the Cape Cod aquifer. The thin basal till layer is assumed to exert an insignificant influence on the regional flow regime and is considered to act as bedrock in this model.

2.6.2 Groundwater System
Cape Cod is underlain by a large, unconfined groundwater flow system. This phreatic aquifer is divided into six flow cells according to the hydraulic boundaries of the flow system. The Massachusetts Military Reservation and LF-1 plume are located in the West Cape flow cell, the largest of the six flow cells. The aquifer system and water table contours in the western Cape region are depicted in Figure 2-4.
The water table in this region occurs at a depth of 40-80 feet below the ground surface. Surface water is also present in the study area as intermittent streams in drainage swales and more importantly as ponds in kettle holes on the Mashpee Pitted Plain. However, there are no large kettle ponds that can significantly influence the flow regime near the LF-1 site and plume. Cranberry bogs can also act as surface discharge points of groundwater, but it is thought that the cranberry bogs west of the LF-1 site are underlain by localized perched water tables, and thus hydrologically disconnected from the larger aquifer system (CDM Federal, 1995). This fact, and the depth of the contaminant plume near these cranberry bogs, make it unlikely that contaminants from LF-1 will discharge into these important agricultural areas.

2.6.2.1 Vertical Hydraulic Gradients

Vertical gradients that have been calculated for the LF-1 site are very small. Most gradients calculated in the IRP hydrologic investigations were below the survey accuracy threshold. Significant upward vertical gradients do exist where groundwater discharges into large ponds and near coastal areas where the aquifer discharges into the ocean. Small downward gradients of about $10^{-3}$ to $10^{-4}$ ft/ft are observed throughout the rest of the study area (CDM Federal, 1995). Such vertical gradients generally indicate upward flow near the shoreline and surface water bodies and downward flow due to infiltration elsewhere.
2.6.2.2 Horizontal Hydraulic Gradients

Groundwater flow in the region is driven mostly by horizontal gradients. These can be measured by dividing a groundwater elevation contour interval by the horizontal distance between the contours. The latter value can be estimated from a contour map similar to Figure 2-4. Horizontal gradients calculated for the LF-1 study area using February 1994 water levels range from $1.3 \times 10^{-3}$ to $6.8 \times 10^{-3}$ ft/ft (CDM Federal, 1995). These gradients are observed to steepen from the LF-1 source area westwards.

2.6.2.3 Seepage Velocity

Calculated seepage velocities in the LF-1 study area indicate that advective contaminant transport takes place at velocities ranging from 0.10 ft/day to over 3 ft/day. Since seepage velocity is a function of hydraulic conductivity, the differential permeabilities of the various sediment types strongly influence calculation of seepage velocities at this site. An estimate of the seepage velocity of contaminants made using observed LF-1 plume migration distance and time yielded an average seepage velocity of 0.9 ft/day (CDM Federal, 1995).
Figure 2-3 West Cape Cod Glacial Deposits (From CDM Federal, 1995).
Figure 2-4 Western Cape Cod Water Table Contour Map (From CDM Federal, 1995).
3. GROUNDWATER FLOW AND MASS TRANSPORT MODEL

3.1 REPRESENTING THE PHYSICAL SYSTEM

3.1.1 General Assumptions
The models in this study were developed under the broad assumption that the aquifer system was in steady state. Thus, seasonal changes in recharge, water table elevations, pond elevations and aquifer discharge are neglected. Transient properties of the system, such as storativity and specific yield, are also not considered.

3.1.2 Assigned Geologic Materials
The geologic structure of the LF-1 study area was represented as depicted in Figures 3-1 to 3-4. The geographic locations of the materials were assigned according to USGS maps of the region (Oldale, 1986). The Mashpee Pitted Plain was represented vertically as two material types and two horizontal sections, to accurately represent the upward coarsening and north-south fining that is observed. The Buzzards Bay Moraine was defined vertically as four different material of increasing permeability upwards and two horizontal divisions. The Buzzards Bay Outwash was depicted as two vertical materials, with the coarser material on top. All three deposit types were underlain by a layer of Glacio-Lacustrine deposits (GLS) of varying thickness atop the contoured bedrock. This vertical geologic representation was made according to the classification adopted by Masterson and Barlow (1994).
Figure 3-1 Plan view of LF-1 study area with assigned geologic materials.
Figure 3-2 North-South cross-section of Buzzards Bay Moraine.
Figure 3-3  East-West cross-section of study area near Buzzards Bay.
Figure 3-4 East-West cross-section of study area near Nantucket Sound.
3.1.3 Source
The LF-1 site was depicted as six distinct sources within the larger source area, representing the six landfill cells. 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. The DYNTRACK software locates each source at the user defined X-Y coordinates but assigns a Z coordinate at the water table. Thus, flow in the unsaturated zone is not accounted for in this model.

3.1.4 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 designed to extend from the observed surface elevation (from USGS topographic maps) to the 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 uniform horizontal head and acts as a sink for groundwater upgradient of the pond and a source of groundwater to sections of the grid downgradient of it. This formulation was considered to most closely approximate the behavior of ponds in the Cape Cod region. The above method of representing a pond prevents it from acting as an infinite source of water in the event of aquifer pumping nearby, a situation that may arise if ponds are defined as constant head nodes.
3.1.5 **Hydraulic Properties**

3.1.5.1 **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 site characterization section of the group report in Appendix A carries a more detailed discussion of these empirical findings. For the purposes of the groundwater model, hydraulic conductivities proved to be the parameter that the flow model was most sensitive to. Horizontal hydraulic conductivity values of each sediment type were considered a variable input in calibrating the flow model, and were assigned values within an empirically determined range described by Masterson and Barlow (1994). The ranges of hydraulic conductivities observed for each geologic material are included in Table 3-1.

3.1.5.2 **Anisotropy Ratio**

The anisotropy ratio, $K_H/K_V$, assumed for Cape Cod sediments usually ranges from 1:1 to 50:1 (Masterson and Barlow, 1994). An anisotropy ratio of 3:1 was initially assumed for all material types in this model. This particular parameter is important in determining vertical gradients and vertical migration of the simulated particles in the mass transport model. Thus, a final anisotropy ratio would be assigned for the materials during the calibration process for the models.
3.1.5.3 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., 1988; Barlow, 1989). It was decided to use an effective porosity value of 39% throughout the model.

<table>
<thead>
<tr>
<th>Geologic Material</th>
<th>$K_H$ ft/day</th>
<th>Anisotropy Ratio $(K_H/K_V)$</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>LeBlanc et al., 1988.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Barlow and Hess, 1993.</td>
</tr>
</tbody>
</table>

*Table 3-1 Ranges of observed hydraulic conductivities in Cape Cod soils (Masterson and Barlow, 1994).*

3.1.5.4 Dispersivity

The DYNTRACK mass transport software simulates the dispersion process by adding random dispersive displacements to each particle. The input requirements to account for dispersion in the particle tracking procedure program are the longitudinal dispersivity ($\alpha_L$, ft), the transverse dispersivity ($\alpha_T$, ft) and the dimensionless vertical dispersion anisotropy ratio ($S_a$) for each soil type.
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). Therefore, the fact that the LF-1 site is a large one, on the scale of about a kilometer, must be taken into account when assuming values of dispersivities to use in the particle tracking procedure. Ideally, dispersivity values obtained from a site similar in geology to the western Cape and a source similar in scale to the LF-1 site should be used. 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 (Gelhar et al., 1992). The dispersivity values adopted in the model are included in Table 3-3.

3.1.5.5 Retardation

Sorption driven retardation factors ranging from 1.82 (TCE) to 4.19 (CCL₄) were considered to apply for western Cape Cod soils in the IRP analysis of a containment system for LF-1 (Op-Tech, 1996). However, laboratory analysis of sorption of volatile organic compounds on to the soils in this region indicated retardation factors to range from 1 to 1.25 (Khachikian, 1996). Hence, retardation of contaminant particles in the Cape Cod aquifer due to sorption on to soils was considered negligible in formulating the transport model. The particle tracking procedure was run assuming a retardation factor of 1.0.
3.2 MODEL FORMULATION

3.2.1 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 and grid are depicted in Figure 3-5. 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 since it determines how far out at sea the LF-1 plume will discharge if it is not fully contained.

The grid covering the LF-1 study area was generated in DYNPLOT. 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. The grid was generated with smaller elements in the source area and presently observed plume locations and progressively coarser grid elements moving away from these locations. The study grid is composed of 3156 triangular elements and 1652 nodes, covering an area of approximately 58 square miles. The grid discretizes the vertical dimension of the study area in 8 layers (9 levels). The bottom (1st) level follows the bedrock contours (Oldale, 1969), while the top (9th) level approximates the surface topography (USGS Topographic Maps).
Figure 3-5 Plan view of LF-1 study area and finite element grid.
3.2.2 Boundary Conditions

3.2.2.1 Saltwater-Freshwater Interface

The saltwater-freshwater interface determines where the landfill plume, if not fully contained, will discharge into Megansett, Red Brook and Squeteague harbors. The interface defines a thin zone of mixing between salt and fresh water. It can be assumed that the interface acts as a impermeable barrier to flow, causing the aquifer to discharge into the sea along a discharge face on the ocean floor extending from where the interface intersects the seabed, to the shore. 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 shoreline are determined entirely by the discharge and hydraulic conductivity of the aquifer. The following equation approximates the saltwater-freshwater interface in a phreatic aquifer (Bear, 1972).

\[ X_0 = \frac{\delta Q}{2K_z} \]

where \( \delta = \frac{\rho_{\text{fresh}}}{(\rho_{\text{salt}} - \rho_{\text{fresh}})} \), \( X_0 \) is the horizontal distance from shore to the point where the interface intersects the seabed, and \( K_z \) is the vertical hydraulic conductivity of the aquifer near shore. This is assuming that flow near the interface is vertical. The distance \( X_0 \) was calculated to be approximately 500 ft. Therefore, for the purposes of this model,
the salt-fresh interface is assumed to be a vertical, no-flux boundary located approximately 500 ft offshore.

3.2.2.2 No-Flux Boundaries

No-flux boundaries are modeled in DYNFLOW by assigning all such nodes, for instance 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 unduly influence the calculated values of head and flow velocity.

3.2.3 Aquifer Pumping and Artificial Recharge

A total of 11 public water supply wells were located in the LF-1 study area in 1989. These wells are estimated to pump on average a total of approximately 8.0 cubic feet per second from the aquifer (Masterson and Barlow, 1994). This translates to roughly 1.9 inches of aquifer water pumped annually out of the study area. However, since the discovery of the contaminant plume emanating from the Massachusetts Military Reservation, the general trend has been to shut down public and private wells that are seen to be at risk of being impacted by contaminated groundwater. Even when the aquifer is pumped, it is estimated that 85% of this pumped volume is infiltrated back into the system (Masterson and Barlow, 1994). Therefore, the effect of aquifer pumping is not considered to be a major impact on the Cape Cod aquifer system for the purposes of this study.
Artificial recharge constitutes only a small portion of the annual flow into the Cape Cod aquifer system. Table 3-2 contains the average annual recharge and pumping from all sources for the LF-1 study area defined in this model. This data demonstrates that net loss from aquifer pumping is only 1.3% of the total natural recharge, and therefore not very significant. It is also estimated that the effect of human demand of aquifer water, if assumed to increase at a rate similar to that seen from 1978 to 1986, will be a decrease in water table elevation of only 0.8 ft in the year 2020 from the steady state level (Masterson and Barlow, 1994). This again highlights the minimal effect of aquifer pumping for domestic consumption on the Cape Cod aquifer.

The effects of artificial pumping and recharge on the West Cape flow cell is thus seen to be minimal. Therefore, recharge and pumping due to human activity is ignored in this flow model of the western Cape aquifer. This assumption is even more applicable under the current circumstances when most public and private water supply wells in the region are being taken out of service, and most consumers being put on public water imported from outside the LF-1 study area.

3.2.4 Natural Recharge

Natural recharge is the largest source of replenishment of the West Cape aquifer system. This natural recharge is composed entirely of rainfall infiltrating through the surface layer. Cape Cod on average receives 46 inches of rainfall annually (CDM Federal, 1995). Nearly half of this precipitation, or 23 inches/year, infiltrates to the
groundwater system through the highly permeable top soil (LeBlanc, 1986). The rest of the precipitation is lost through evaporation and transpiration. There is little or no surface runoff due to the permeable nature of the soils and the small topographic gradients present in this region.

<table>
<thead>
<tr>
<th>Hydraulic Stress</th>
<th>Recharge/Pumping, in/yr</th>
<th>Percent of Natural Recharge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aquifer Pumping</td>
<td>1.9</td>
<td>8.3</td>
</tr>
<tr>
<td>Artificial Recharge</td>
<td>1.6</td>
<td>7.0</td>
</tr>
<tr>
<td>Net Loss From Aquifer</td>
<td>0.3</td>
<td>1.3</td>
</tr>
<tr>
<td>Natural Recharge</td>
<td>23</td>
<td>100</td>
</tr>
</tbody>
</table>

*Table 3-2 Comparison of aquifer pumping, artificial recharge and natural recharge from precipitation in 1989 (From Masterson and Barlow, 1994).*

### 3.2.5 Recharge and Discharge Specified in Models

Since aquifer pumping and artificial recharge is negligible in this region, only natural recharge is considered to be a user defined input/output to the model. 23 inches/year of natural recharge from precipitation is applied uniformly to the model across all elements.
3.3 FLOW MODEL

3.3.1 DYNFLOW Groundwater Flow Model

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 high spatial resolution without having to implement the same degree of resolution throughout the model and suffer significant penalties in terms of extended computation time.

DYNFLOW solves linear (confined aquifers) and non-linear (unconfined aquifers) flow equations. This is done using linear finite elements, induced infiltration from streams, artificial and natural recharge/discharge, and non-homogeneous and anisotropic aquifer hydraulic properties. The modeling package employs a “rising water” scheme to handle drainage to local streams when the piezometric head in a phreatic aquifer rises to the elevation of the streambed. The governing equation for three dimensional groundwater flow solved by DYNFLOW is

\[
S_\phi \frac{\partial \phi}{\partial t} = \frac{\partial}{\partial x}[K_x \frac{\partial \phi}{\partial x}] + \frac{\partial}{\partial y}[K_y \frac{\partial \phi}{\partial y}] + \frac{\partial}{\partial z}[K_z \frac{\partial \phi}{\partial z}]
\]
The variable $\phi$ is the hydraulic potential; $K_x$, $K_y$ and $K_z$ are the hydraulic conductivities along the principal axis of the orthogonal coordinate system; $S_s$ is the specific storativity; and $t$ is time (Camp, Dresser, and McKee, 1992).

3.3.2 Flow Model Calibration

The flow model was calibrated using water table elevations measured at 106 observation wells in March 1993 (Savoie, 1993). Calibration was achieved by iteratively comparing the results of each DYNFLOW simulation with observed water table elevations until the model results were reasonably consistent with the empirical data. The method of adjusting flow model results was changing the hydraulic conductivities of the layer materials. The final calibrated head distribution displayed a mean difference of +0.044 ft and a standard deviation of 2.159 ft from the observed water table elevations. Calibrated water table contours are shown in Figure 3-6.

Pond, cranberry bog and stream elevations were set in the model formulation to observed values from USGS topographic maps of the region. Streams and ponds were modeled by the rising water boundary condition. This ensured that where the calculated water table intersected the surface, that point would function as a discharge point for groundwater. An inspection of the final groundwater model showed that at ponds in the southern part of the study area, the invoked rising boundary condition was indicated upgradient of the ponds (Figure 3-7). An invoked rising boundary condition was also observed at the inlets and streams on the southern boundary of the
model. This result is again consistent with field observations, where many streams, rivers and inlets act as discharge points for the Cape Cod aquifer near Nantucket Sound. Observations of flow patterns near cranberry bogs west of the LF-1 site indicated no large vertical gradients or groundwater discharges from the main aquifer into these areas.

Thus, it is seen that in the model, major surface water bodies function as discharge points for the modeled aquifer. This result is consistent with field observations of the region, where many ponds receive groundwater discharges at the upgradient location and then act as aquifer recharge locations at the downgradient location. Local streams near the shore line in the Cape, such as Red Brook, are recipients of groundwater from the aquifer system, and the calibrated groundwater model accurately reflects this situation. Therefore, the model was considered to be calibrated and representative of the West Cape cell of the Cape Cod aquifer.
Figure 3-6 Calculated water table elevation contours and flow model calibration results.
3.4 MASS TRANSPORT MODEL

3.4.1 DYNTRACK Particle Tracking Model
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.

3.4.2 Source Definition
Figure 2-2 is a plan view of the LF-1 source area, and show the six cells that contain wastes. Each of these landfill cells may contain the contaminants present in the LF-1 plume, and the lack of documentation about the nature and amount of waste in the cells makes exact source definition difficult. It was assumed that all six cells were sources of contaminants. It was also assumed that the volatile organic contaminants that are of concern in the LF-1 plume were not widely used until 1945. Thus, the sources were defined to begin discharging contaminants to the aquifer in 1945. The fact that three cells of the landfill were capped in 1994 was also taken into consideration in the particle tracking model.

The LF-1 site was modeled as six particle sources, similar in area and location to the six landfill cells. Simulation was defined to begin in 1945. A 51 year simulation was first carried out (to 1996), with three of the sources being shut off in the 49th year to
approximate a successful capping of the three cells in 1994. The simulation was then extended to see the effects of successfully capping the entire landfill by year 2000 using a design similar to that discussed by Elias (1996).

3.4.3 Particle Tracking Model Calibration
It was decided to calibrate the particle tracking model using observed contamination locations. Thus, the vertical location and thickness of the simulated plume was compared to discrete points of contamination measured in the LF-1 area. This was done to avoid the introduction of any bias due to a contouring method if comparison was made to concentration contours, instead of individual observation points. The general direction of migration and horizontal width east of the Buzzards Bay Moraine of the simulated plume were also compared to observations to check for model accuracy. Since the data on the longitudinal extent of the LF-1 plume available at the time of model formulation were considered to be inadequate to accurately define the leading edge of the plume, no effort was made to calibrate simulated plume length to observed length. The recently released Op-Tech Technical Memorandum on containing the LF-1 plume, however, which indicated that the LF-1 plume had reached Red Brook Harbor by 1996, was consistent with results of the simulation (Op-Tech, 1996). Figures 3-8 and 3-9, which show the simulated plume in plan view and cross-section as well as measured contaminant locations, indicate that the modeled plume is consistent with observations.
Figure 3-8 Plan view of simulated LF-1 plume. Buzzards Bay Moraine also shown.
Figure 3-9 Cross-Section of simulated LF-1 plume with observed contaminant locations.
3.5 RESULTS

3.5.1 Steady State Flow Simulation
Once the geological setting of the study area was defined and instituted, the DYNFLOW modeling software was used to solve for heads and flow velocities throughout the grid. The method of calibration used was to match observed water table elevations at discrete data points with heads calculated by DYNFLOW at the same points. This was considered sufficient since there were no appreciable vertical head gradients observed or calculated in the main study area.

Prior to obtaining the final flow regime solution, a run of the particle tracking procedure was made with a preliminary flow model result that matched observed and calculated heads within a mean difference of 0.050 ft and standard deviation of 1.716 ft. The results of the DYNTRACK run, however, indicated that flow contours obtained from the preliminary DYNFLOW run resulted in particles released from the landfill site migrating directly south towards Coonamasset Pond. It was concluded that this unexpected and improbable result was caused by the hydraulic properties of the Buzzards Bay Moraine.

It was then decided alter the properties of the moraine, particularly the northern portion of the moraine such that the calculated head contours would not bend as sharply as before. The adjustments made consisted of horizontally subdividing the
moraine into north and south sections, in addition to the vertical layers previously defined. The northern morainal deposits were now defined as being somewhat more permeable than the southern deposits. DYNFLOW was used to recalculate heads, which were calibrated as before, by changing conductivities of the materials.

The final DYNFLOW solution resulted in a mean difference of +0.044 ft and a standard deviation of +2.159 ft between calculated heads and observed heads, as shown in Figure 3-6. Calculated and observed head contours (contouring done with the procedure available in DYNPLOT) were also seen to match well, as can be seen in that figure. Table 3-3 contains a comparison of the hydraulic properties defined in the preliminary and final versions the flow model. The close agreement in calculated and observed heads for both versions of the model indicate that calculated heads over the entire study area are not very sensitive to changes in the hydraulic conductivities of the geologic materials. The shape of head contours near the moraine is, however, strongly dependent on the contrast in hydraulic conductivities between the moraine and outwash deposits, and it is this difference in permeabilities between the moraine and its surroundings that dictate the path of the LF-1 contaminant plume.
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Lacustrine</td>
<td>90.0</td>
<td>3.3</td>
<td>0.03</td>
<td>20</td>
<td>15</td>
<td>3:1</td>
</tr>
<tr>
<td>Fine Sand West</td>
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<td>80</td>
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<tr>
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<td>160</td>
<td>180</td>
<td>3:1</td>
</tr>
<tr>
<td>Fine Sand South</td>
<td>90.0</td>
<td>3.3</td>
<td>0.03</td>
<td>160</td>
<td>135</td>
<td>3:1</td>
</tr>
<tr>
<td>Coarse Sand South</td>
<td>90.0</td>
<td>3.3</td>
<td>0.03</td>
<td>300</td>
<td>210</td>
<td>3:1</td>
</tr>
<tr>
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<td>90.0</td>
<td>3.3</td>
<td>0.03</td>
<td>30</td>
<td>30</td>
<td>3:1</td>
</tr>
<tr>
<td>BBM Med Low-N</td>
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<td>3.3</td>
<td>0.03</td>
<td>60</td>
<td>110</td>
<td>3:1</td>
</tr>
<tr>
<td>BBM Med High-N</td>
<td>90.0</td>
<td>3.3</td>
<td>0.03</td>
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<td>150</td>
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<tr>
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<td>0.03</td>
<td>130</td>
<td>170</td>
<td>3:1</td>
</tr>
<tr>
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<td>3.3</td>
<td>0.03</td>
<td>30</td>
<td>15</td>
<td>3:1</td>
</tr>
<tr>
<td>BBM Med Low-S</td>
<td>90.0</td>
<td>3.3</td>
<td>0.03</td>
<td>60</td>
<td>60</td>
<td>3:1</td>
</tr>
<tr>
<td>BBM Med High-S</td>
<td>90.0</td>
<td>3.3</td>
<td>0.03</td>
<td>110</td>
<td>100</td>
<td>3:1</td>
</tr>
<tr>
<td>BBM High-S</td>
<td>90.0</td>
<td>3.3</td>
<td>0.03</td>
<td>130</td>
<td>135</td>
<td>3:1</td>
</tr>
<tr>
<td>Nant. Ice Deposits</td>
<td>90.0</td>
<td>3.3</td>
<td>0.03</td>
<td>190</td>
<td>190</td>
<td>3:1</td>
</tr>
<tr>
<td>Pond Material</td>
<td>90.0</td>
<td>3.3</td>
<td>0.03</td>
<td>$10^{-5}$</td>
<td>$10^{-5}$</td>
<td>3:1</td>
</tr>
<tr>
<td>Fine Sand North</td>
<td>90.0</td>
<td>3.3</td>
<td>0.03</td>
<td>210</td>
<td>140</td>
<td>3:1</td>
</tr>
<tr>
<td>Coarse Sand North</td>
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<td>3.3</td>
<td>0.03</td>
<td>320</td>
<td>270</td>
<td>3:1</td>
</tr>
<tr>
<td>Fine Lacustrine</td>
<td>90.0</td>
<td>3.3</td>
<td>0.03</td>
<td>10</td>
<td>10</td>
<td>3:1</td>
</tr>
</tbody>
</table>

*Table 3-3 Dispersivities used in model and comparison of hydraulic conductivities in preliminary and final models*

### 3.5.2 Particle Tracking

Once the groundwater model was calibrated, the next step of the project was to use these calibrated results and observe the behavior of contaminant particles released from the known source location. The initial run of DYNTRACK was made with a preliminary
DYNFLOW groundwater flow model result. The resultant plume path for particles released at the LF-1 site was diametrically opposite to the expected plume location.

It was concluded that the calculated head contours, which in the absence of significant vertical gradients determine the direction of particle transport in the aquifer, were being unduly affected by the contrast in hydraulic conductivity between the Buzzards Bay Moraine deposits and the surrounding material. Therefore, the DYNFLOW model was calibrated to both observed head levels at specific monitoring well locations and observed water table contours. The DYNTRACK particle tracking procedure was then run for particles released at the LF-1 site. This process was carried out for several source formulations as described below.

3.5.2.1 Continuous Source

The most simple definition of the landfill is depicting the site as a continuous source. This is consistent with an assumption that the landfill leaches contaminant into the groundwater system at a constant rate. Such an assumption would be an accurate representation of the situation until the capping of a part of the site in 1994 and even beyond this point if the Northwest Operable Unit (NOU) contains the same wastes that are present in the capped portion of the landfill. All available data points to the conclusion that the NOU does indeed contain volatile organic compounds, and is therefore a potential source of the LF-1 plume (Elias, 1996).
The results of releasing particles at the six landfill cells, with the three cells that were capped in 1994 defined to be non-sources after that year, show that the predicted horizontal plume path and vertical location closely agrees with the observed plume locations, as shown in Figures 3-8 and 3-9. The simulated plume is close to the wells at which contamination was observed, and is generally surrounded by the non-detect wells (Figure 3-9). The simulation was carried out assuming that the landfill began to leach VOCs into the aquifer in 1945. This assumption was made because the chlorinated compounds that are considered to be the main contaminants in the LF-1 plume were not widely used until the end of the Second World War, around 1945. The results of the simulation depicted in Figure 3-9 indicate that 50 years after source inception, contamination has just begun to reach the ocean discharge point. This prediction is consistent with the results of the MMR Data Gap Field Investigation (Op-Tech, 1996).

The relative patterns of the particle pathlines are also of interest. As Figure 3-8 shows, the particles travel in two distinct lobes, a northern and southern lobe. The northern lobe of the plume travels much faster than the southern lobe. This predicted behavior is again consistent with observed data (Op-Tech, 1996). Figure 3-10, a cross-section of the simulated plume at the point of entry into the moraine, provides an explanation of the differential plume travel velocities.

When the plume enters the moraine, the southern portion of the plume is at a lower elevation than the northern portion. This difference is due to the southern portion of the plume traveling a longer route to reach the moraine along the outside curvature of the
plume path, thereby being forced down more by the infiltrating recharge. The difference in vertical location causes the southern portion of the plume to enter the lowest conductivity layer of the moraine, whereas the northern plume lobe travels through a higher conductivity layer. Thus, the velocity of the southern lobe is retarded in comparison to the northern lobe. In 51 years of simulation, the southern lobe does not reach the Buzzards Bay Outwash, while the northern lobe has reached the Red Brook Harbor saltwater-freshwater interface. Table 3-4 compares the average travel velocities of the simulated southern and northern plume lobes during the 51 years from 1945 to 1996.

<table>
<thead>
<tr>
<th>Plume Lobe</th>
<th>Average Velocity, ft/day</th>
</tr>
</thead>
<tbody>
<tr>
<td>Southern</td>
<td>0.55</td>
</tr>
<tr>
<td>Northern</td>
<td>1.10</td>
</tr>
</tbody>
</table>

*Table 3-4 Calculated travel velocities of the northern and southern lobes of the LF-1 plume.*

### 3.5.2.2 Total Source Removal in 2000

The removal of the contaminant source can be accounted for in DYNTRACK, and the behavior of the discontinuous plume that results can then be observed. This is similar to implementing a successful cap for the entire landfill. All particle sources were removed from the simulation after 55 years (Year 2000) and the simulation carried on further. It was seen that under steady state flow conditions it would take over 165 years after total source remediation to fully flush the West Cape aquifer of LF-1 derived contaminants if the entire landfill was capped successfully and no other remedial action was taken.
3.5.2.3 Effect of Extraction Well Fence at Route 28

The Installation Restoration Program, mainly due to problems with land acquisition and proximity to the salt-fresh interface, is considering a pump and treat system along Route 28 despite the fact that the LF-1 plume has now migrated beyond this point. The portion of the LF-1 plume west of the extraction well system at the time the pump and treat system goes into operation will then be allowed to discharge into Buzzards Bay. The flushing time of contaminants due to the successful implementation of this pump and treat a plan were considered by locating sources along Route 28. It was observed that the simulated particles required a time of 12 years to fully discharge from the system.

3.5.2.4 Sinking Contaminant Source

A possible explanation of the observed detached deep northern lobe of the LF-1 plume could be a sinking pool of landfill leachate. This seems unlikely in light of the fact that the lower layers of the moraine in the model are defined to be of lower conductivities. Thus, it is not possible for a deep source under the LF-1 site to introduce contaminants that would travel farther than contaminants released to the system at the water table, using the current definition of the flow model.
Figure 3-10 Cross-Sectional view of simulated LF-1 plume as it enters the Buzzards Bay Moraine.
4. CONCLUSIONS

A groundwater flow and mass transport model was developed for the Main Base Landfill (LF-1) site at the Massachusetts Military Reservation. This exercise was carried out to model a contaminant plume caused by the disposal of volatile organic solvents in unlined trenches at the landfill. A study area encompassing about 58 square miles was defined and a finite element flow model for this site was obtained. The calculated head values were then used to simulate the migration of particles released from the LF-1 site. The following results were obtained from the modeling project.

The flow model is very sensitive to differences in permeability between the moraine and outwash deposits. This sensitivity is highlighted by the curvature of the 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.

Particles released at the LF-1 site migrate to the ocean near Red Brook Harbor in approximately 50 years. Thus, assuming that the volatile organic solvents of concern at this site were released in 1945, the predicted extent of the plume reaches the ocean discharge face by 1996. This finding is in agreement with the most recent observations at the MMR which indicate 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 unremediated into the ocean, a DYNTRACK simulation of 165 additional years is required for all LF-1 derived contaminants in the aquifer to fully discharge into the ocean.

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 Buzzards Bay Moraine causes the southern part of the plume to be retarded by one half. 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. The field observations of a similar phenomenon may also be due to the presence of a low conductivity layer at depth in the Buzzards Bay Moraine.

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 detached deep LF-1 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 from above as it traverses the Buzzards Bay Outwash towards the shoreline.
Since the slower moving southern lobe is still in the moraine, the leading edge of the northern lobe near Red Brook Harbor is now at a lower elevation.

No plausible explanation of the detached deep northern plume based on the geology of the site can be conceived with the models developed in this thesis as they are defined. It is recommended that further field explorations of the deep northern LF-1 lobe be made to determine if this plume is actually detached from the main northern lobe, and if so, conduct studies to better characterize the Buzzards Bay Moraine to explain this unusual phenomenon.

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 hydrologic system is relatively unchanged, the uncaptured section of the LF-1 plume will take an additional 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.

The groundwater flow model developed in this study can be used to design an efficient extraction well fence system to remediate the LF-1 plume and simulate the effects of such a system on the aquifer. Another conclusion that resulted from this project was that a regional finite element flow model for the entire West Cape Cod flow cell would probably be a more efficient and accurate approach than developing separate localized flow models for each plume, as was done in the Master of Engineering thesis projects (Riva, 1996; Triantopoulos, 1996; Lázaro, 1996; López-Calva, 1996). This is because the
dominant geologic characteristics of the Cape, such as the Sandwich and Buzzards Bay moraines, influence aquifer flow patterns on a regional scale, and thus must be modeled and calibrated on a similar scale. Furthermore, the complex hydrologic interactions caused by locating several high capacity pump and treat systems in close proximity to each other can be better addressed with an overall regional model.

The finite element method, which has not yet been used to develop a regional Cape Cod flow model, would allow for greater resolution in all the contaminated sites and contaminant plumes at the MMR without unduly increasing computational complexity. However, the lack of extensive hydrogeologic data on the northern and eastern portions of the West Cape flow cell makes accurate calibration of a regional model difficult. This situation highlights the need for greater hydrogeologic exploration in the northern and eastern regions of Cape Cod.
References


CDM Federal Programs Corporation, *Remedial Investigation Report, Main Base Landfill (AOC LF-1) and Hydrogeological Region 1 Study*, Boston, MA, 1995.


Appendix A: LF-1 Group Project Results

This appendix contains the results of the Master of Engineering (M. Eng.) group project. According to the goals and directives of the M. Eng. program, each of the following sections of Appendix A was written by a different member of the LF-1 group. These group results are divided into two sections. Section A covers site characterization, groundwater modeling, and risk analysis. Section B looks at possible source containment and bioremediation actions. Chapter 3 of this thesis contains a more detailed report of the groundwater modeling project, which is the subject of this author's individual thesis.

A The MMR LF-1 Plume

Site Characterization
Site characterization investigations were undertaken with two main goals in mind. 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
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 Appendix A 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.
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, implying that the TCE could not be the result of PCE degradation. Instead, this indicates that TCE must be one of the contaminants originally disposed of in the landfill.

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 pumping 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 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. A detailed analysis of the site characterization results discussed herein can be found in Alden (1996). The next section describes the groundwater modeling process.
3-D GAUSS FILTERED DATA: VERT. 5  HOR. 150
LOG-LINEAR CONTOUR OF MAX. MCL NORMALIZED
VALUES BEGINNING WITH 1 P.P.B. - 1 P.P.B. INTERVALS
CONTOURS BETWEEN BEDROCK AND WATER TABLE
Groundwater Modeling and Particle Tracking Simulation

A three dimensional groundwater model and particle tracking simulation of the LF-1 study area was carried out to predict the potential migration paths and travel times of the LF-1 plume. The groundwater flow system of the western Cape was modeled with the DYNFLOW groundwater modeling package, a FORTRAN based program that simulates three-dimensional flow using a finite element formulation. Transport of particles released from the LF-1 source were modeled using DYNTRACK. This mass transport modeling software simulates three-dimensional contaminant mass transport and uses the same finite element grid, aquifer properties, and flow field that were used in and derived from DYNFLOW. DYNPLOT, a graphical pre- and post-processor that can create full color displays, was used to view the study grid and simulated results in plan view or cross-section (Camp, Dresser, and McKee, 1992).

The groundwater flow and particle transport model provided results that were similar to field observations. The Buzzards Bay Moraine was observed to exert a great deal of influence on the regional hydrologic model. The geologic characteristics assigned in the flow model to the BBM were seen to define the shape of the regional head contours and thus the travel path and velocity of the simulated plume. Chapter 3 of this thesis contains a detailed description of the groundwater model and particle tracking procedure and the results of the modeling effort. The following section describes the group project section on risk assessment and risk management.
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 LF-1 plume as compared to the rate for a member of the local community if this contamination did not exist. It is the probability of an event occurring and the magnitude of the effect such 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 the said 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 uncertainties 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, 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 on 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 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 LF-1 plume are being evaluated for potential risk. These materials and risks are identified in the risk spreadsheets presented in Tables A-1 to A-5.

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 (Aschengrau and Ozonoff, 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 (Davis, 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 to the release of carcinogenic materials at the MMR site.
Review Existing Reports

Part of this investigation was a comprehensive review of the RI (CDM Federal, 1995), and the RAH (Automated Sciences Group, 1994) 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 documentation which have been provided to MIT to calculate risk and formulate risk opinions. Independent spreadsheet calculations are included in Tables A1-A5. As the MMR LF-1 is part of an on-going clean-up, new and updated data from the above-referenced reports have 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, 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. Hines (1996) describes a more detailed analysis of the human cancer risks associated with the LF-1 plume.
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\text{TOTAL RISK}_{\text{mean}} = \sum (\text{Concentration} \times ([\text{Inhalation Risk/Exposure}] + [\text{Dermal Risk/Exposure}] + [\text{Ingestion Risk/Exposure}]])
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*Table A-1* Mean Risk Calculations According to EPA Guidelines
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Table A.2 Inhalation Risk Calculations According to EPA Guidelines
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<th>Exp. Dur. years</th>
<th>Exp. Freq. days/yr.</th>
<th>Surface Area cm$^2$</th>
<th>Dermal Perm. cm/hour</th>
<th>Unit Adj. l/cm$^3$</th>
<th>Unit Adj. hours/day</th>
<th>Body Wt. kilograms</th>
<th>Avg. Time years</th>
<th>Unit Adj. days/yr</th>
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<td>0.0248</td>
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<td>Chloroform</td>
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<td>0.0080</td>
<td>0.001</td>
<td>24</td>
<td>70</td>
<td>70</td>
<td>365</td>
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</table>

Inorganic Carcinogens

| Beryllium                     | 0.0000E+00          | 4.3000               | 30              | 2.9                  | 19,400              | 0.0000               | 0.001             | 24                | 70               | 70             | 365             |

**DERMAL RISK/EXPOSURE =** \[(\text{Exposure Duration} \times \text{Exposure Frequency} \times \text{Dermal CSF} \times \text{Surface Area} \times \text{Dermal Permeation})\] / (\text{Body Weight} \times \text{Averaging Time}) \times \text{(Unit Adjustments)}

*Table A-3* Dermal Risk Calculations According to EPA Guidelines
<table>
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<td>years</td>
<td>days/year</td>
<td>liters/day</td>
<td>kilograms</td>
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INGESTION RISK/EXPOSURE = (Exposure Duration x Exposure Frequency x Ingestion CSF x Water Ingestion Rate) x (Body Weight x Averaging Time) x (Unit Adjustments)

Table A-4 Ingestion Risk Calculations According to EPA Guidelines
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\[
\text{TOTAL RISK}_{\text{max}} = \sum (\text{Concentration} \times (\text{Inhalation Risk/Exposure}) + (\text{Ingestion Risk/Exposure}) + (\text{Dermal Risk/Exposure}))
\]

**Table A-5** Maximum Risk Calculations According to EPA Guidelines
Assessment of Risk from Ingestion of Contaminated Shellfish

From the available data on the LF-1 plume, the contaminants are projected to discharge into Red Brook, Squeteague, and Megansett harbors of Buzzards Bay (Op-Tech, 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.

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-6. The maximum concentration of metals detected in well samples from the LF-1 plume are derived from the reports of CDM Federal (1995) and Op-Tech (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 $1.0\times10^{-6}$ to $1.0\times10^{-4}$. The standard is set independently for each site and case.

The increased risk of $3.3\times10^{-3}$ for each exposed resident is above the highest acceptable USEPA standard. A maximum cancer risk of $3.0\times10^{-3}$ is calculated when maximum concentration of metals from Op-Tech (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 Op-Tech (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-7.
<table>
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<th>Metal</th>
<th>Max. C&lt;sup&gt;1&lt;/sup&gt; (ug/l)</th>
<th>Max. C&lt;sup&gt;2&lt;/sup&gt; (ug/l)</th>
<th>Oral SF</th>
<th>Oral RfD</th>
<th>Cancer Risk&lt;sup&gt;1&lt;/sup&gt;</th>
<th>Cancer Risk&lt;sup&gt;2&lt;/sup&gt;</th>
<th>Cancer Index&lt;sup&gt;1&lt;/sup&gt;</th>
<th>Cancer Index&lt;sup&gt;2&lt;/sup&gt;</th>
</tr>
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<td>Aluminum</td>
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Notes:
1 Derived from CDM Federal (1995)  
2 Derived from Op-Tech (1996)  
@ Maximum total concentration  
# Chromium (VI) values are used  
* Maximum dissolved concentration

**Table A-6.** Maximum cancer and non-cancer risk for each metal

<table>
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<th>Metal</th>
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<th>Maximum Hazard Index</th>
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**Table A-7** 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. The methodology and assumptions used in the current risk calculations are described in detail by Lee (1996). 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 on 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 an entirely different 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 populations of species and the ecosystem as a whole. Lee (1996) describes an in depth analysis of the ecological and human health risks associated with LF-1 plume constituents that discharge into the ocean.
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, the actual risks are often 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 only a perceived one. Public interest groups have fought many a facility siting and won, not due to actual risk, but because of the perception that risk exists. In Superfund cases, unlike potential hazardous waste facility sitings, contamination has already occurred, but there remains 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 feasible extent” 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 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 has also included the creation of many committees that assist 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. 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 remains to be answered.

**Design Of Future Approaches At The MMR And Elsewhere**

There are several factors 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 interest group opinion will rapidly polarize as soon as contamination and a 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 activity will be inadequate to alleviate the problem of contamination.

Several steps 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 all the “right” approaches were apparently taken, public concern remained an issue. This is due to the fact that early on in the MMR IRP process, the public was excluded and seen more as a “problem” than a potential source of solutions. Jordan (1996) further discusses the management public interaction at the MMR in greater depth.
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-4.

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 $1 \times 10^{-3}$ cm/sec
- Low conductivity layer with minimum thickness of 18 inches and maximum hydraulic conductivity of $1 \times 10^{-7}$ cm/sec, or an approved flexible membrane liner (geomembrane)
- Drainage layer with minimum thickness of 6 inches and minimum hydraulic conductivity of $1 \times 10^{-3}$ 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.


- A low hydraulic conductivity geomembrane / soil layer consisting of a 24 inch layer of compacted natural or amended soil with a hydraulic conductivity of $1 \times 10^{-7}$ cm/sec in intimate contact with a geomembrane liner of minimum thickness 0.5 mm (20 mil).
- A drainage layer of 12 inch minimum thickness having a minimum hydraulic conductivity of $1 \times 10^{-2}$ 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 of soil graded at a slope between 3 and 5 %.

The EPA does encourage design innovation, and will accept an alternative design upon a showing of equivalency.
Figure A-4: 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 geosynthetic 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 (USEPA, 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 $3 \times 10^{-5}$ 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 1x10^{-10} cm/sec (Koerner, 1994), and estimates of the
hydraulic conductivity of Gundseal® GCLs are on the order of 1x10^{-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

As described by Elias (1996), 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, according to Elias (1996). 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 contended 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. A more detailed analysis of the landfill cover design discussed above is described by Elias (1996). The next section discusses the use of bioremediation as an in-situ option for cleaning up the LF-1 plume.
**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 supplying 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 (see Figure A-5). Each well would be driven across the width of the plume and have thousands of small injection ports along the top. The ports are used to inject gases into the aquifer in order to stimulate the microbes which will then 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 (Collins, 1996).

Given a PCE migration rate within the plume of 0.9 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 (Collins, 1996). 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 construct, 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. Collins (1996) discusses the bioremediation system
described above in greater detail.
Figure A-5  Conceptual diagram of sequential bioremediation system.
References

ABB Environmental Services, Inc., Technical Support for Closure Plan for Study Area LF-1, Installation Restoration Program, Massachusetts Military Reservation, Portland, Maine, 1993

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


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


APPENDIX B: DYN System Files

The following input and output files from the LF-1 groundwater flow and mass transport model are included in the attached 3.5" IBM formatted diskette. DYNPLOT, DYNFLOW and DYNTRACK programs used in this thesis project were installed in a IBM compatible workstation running Microsoft® Windows NT™.

Files Included:

♦ MMR.STF: DYNPLOT site file.
♦ LF1-SITE.MAP: Map file of western Cape Cod region.
♦ MORAINES.MAP: Map file of Buzzards Bay Moraine.
♦ LF1-FLOW.SAV: Save file containing Calibrated DYNFLOW results, Grid and Material Properties.
♦ LF1-TRAC.CFI: DYNTRACK command file for particle tracking simulation.