MQP PPM WQ02

MODELING, ANALYSIS AND DESIGN OF EFFLUENT DISPOSAL TO REDUCE ESTUARINE NITROGEN LEVELS IN MASHPEE, MA

A Major Qualifying Project Report:

submitted to the Faculty

of the

WORCESTER POLYTECHNIC INSTITUTE

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by

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Abstract

The Town of Mashpee, Massachusetts, is evaluating wastewater treatment options to reduce nutrient loading from septic system discharge seeping into coastal waters. Several potential treated effluent discharge sites were identified to minimize nutrient loading in estuarine ecosystems. Proposed sites were evaluated using computer modeling to consider current land use, groundwater mounding and the effects of discharge on estuarine nutrient levels and nearby groundwater features. This report concludes with two preliminary design Scenarios for effluent discharge facilities.

Executive Summary

Over the last several decades the towns in Cape Cod, Massachusetts, have experienced substantial population growth and many areas previously dedicated to open space or forests have been developed for residential or light commercial interests. Population growth has resulted in a sudden and severe increase in the levels of nitrogen in runoff from the cape, which enters estuaries – semi-enclosed bodies of water connecting tributary streams and rivers with the ocean – along the coast. Estuaries experience relatively little tidal flushing and high residence times because they are somewhat restricted at the inlet, which in turn leads to a potentially drastic build-up of nutrients and other contaminants in estuarine waters. High nitrogen levels provide food for algae which in turn multiply beyond the carrying capacity of the estuary. Resource-hungry algae populations jeopardize other life forms in the ecosystem and cause increased turbidity. Affected bodies of water must sometimes be closed to swimmers and other recreational activity and often suffer aesthetically. Fortunately, some municipalities, such as the Town of Mashpee, have taken the first steps necessary to ensure the health of the estuaries by confronting the root cause of the problem: increased nitrogen loading from land sources.

Nitrogen comes from several sources, including rainfall and fertilizer used on lawns and golf courses, but the principle source of nitrogen on the cape is septic systems in residential areas, which leach nitrogen-rich wastewater into the aquifer where it is transported by groundwater flow to the estuaries or their tributary waters. Wastewater from septic systems has a typical nitrogen concentration of 35 or 34 mg/L, over one hundred times the maximum estuarine safe level of 0.3 mg/L recommended by the Massachusetts Estuaries Project. One potential solution to this problem is to redirect all of the wastewater from the town to a central location – a wastewater treatment facility (WWTF) – for processing and "denitrification," where the wastewater can be treated to a nitrogen concentration of 3.0 mg/L. However, the product of the wastewater treatment process (known as "effluent") must still be discharged somewhere. The Ocean Sanctuaries Act prohibits discharging any effluent whatsoever directly into the Nantucket Sound, which means it must be discharged back into the aquifer and allowed to flow naturally toward the coast.

The Town of Mashpee lies inside the watersheds of two major estuarine systems: the Popponesset Bay on the east side of town and the Waquoit Bay on the west side. Any wastewater

effluent discharged within the watershed of either of these bays will eventually reach one of several embayments (smaller, enclosed bays or coves which drain into the larger bay, which in turn drains into the Nantucket Sound). If enough treated wastewater effluent reaches these embayments, they still may be at risk for eutrophication and algae bloom. The challenge facing the Town of Mashpee, and the focus of this report, is finding a suitable location to discharge an effluent load of up to 3.3 million gallons per day (MGD) during peak season into the aquifer while ensuring that it doesn't accumulate in any particular estuary.

The Town of Mashpee is currently developing a Watershed Nitrogen Management Plan, and part of that plan will be the identification and screening of alternative discharge sites. As part of this Report, several previously identified sites were provided to the WPI team to analyze and model as part of this "Major Qualifying Project Report". These sites were originally selected on the basis of availability and suitability for the purpose: they must be relatively unobstructed, somewhat removed from residential areas, and located in an area of the aquifer which allows for infiltration. In turn, the focus of this report is the use of a computer modeling package called Visual MODFLOW and a numerical model of the entire western half of Cape Cod (prepared by the USGS in 2004) to test the effects of discharging wastewater effluent at each of the identified sites. The USGS Regional Model is a numerical representation of groundwater flow conditions for the Sagamore Lens, a groundwater system that lies under the western half of the Cape. It takes the form of a three-dimensional grid of "cells," each of which contains data about hydraulic conductivity, water table elevation, recharge from surrounding cells, and other hydrologic properties of the aquifer. Visual MODFLOW is a suite of computer programs which, when used in conjunction with the regional model, is capable of predicting groundwater flow conditions and tracing theoretical inputs to the aquifer (such as effluent discharge) to their final destination.

Using these powerful tools it was possible to determine the outcome of discharging wastewater effluent in each of the potential discharge sites including the amount of nitrogen-rich effluent that will eventually travel into each embayment of the Popponesset and Waquoit Bay systems, the effects of increased discharge on the height of the water table near the site (known as "mounding"), and several other characteristics. To simulate the effects of discharging wastewater effluent into the ground at each site several changes were made the base model. First, the recharge value was modified at each discharge site to reflect the increase in the amount of liquid entering the ground. The original model accounts only for recharge from rainwater and septic system leaching, and the recharge rate at discharge sites can be as much as 100 times as much. Increased recharge has the effect of raising the groundwater table around the discharge site, and if raised too high this would have severe adverse effects on surrounding buildings and infrastructure. Second, "particles" of groundwater were added at each site. Particles are theoretical inputs to the aquifer which are traced from their starting point to whatever point they exit from the groundwater system. They provide a valuable graphical representation of groundwater flow – a brightly colored line stretching from the discharge site to the coast.

When multiple particles are placed at a discharge site which is located along a divide in the water table, some of the particles flow to different embayments than others. This division of particles provides the key data on which the recommendations of this report are based. By measuring the proportion of particles which flow to each separate embayment, it is possible to determine what the nutrient loads for each embayment will be for different discharge scenarios and, by extension, to determine the discharge scenario which most effectively protects the estuaries from eutrophication by distributing the load as widely as possible. Two major numerical engines were used to run the model: MODFLOW2000, which calculates changes in the water table and groundwater flow, and MODPATH, which calculates the particle paths.

The first step in modeling and analysis was to model each of seven proposed discharge sites individually. The results from these sites were varied and many of the discharge scenarios are complicated by factors such as high groundwater mounding, proximity to contaminant plumes currently being remediated by the United States Air Force, or the property ownership. Additionally, many of the sites drain directly into a single embayment, which would result in unacceptable nitrogen concentrations in the affected embayment. However, several discharge sites were identified with enough collective capacity to handle the town's needs, while still spreading effluent between sufficient embayments (and, through subsurface leaching, the Atlantic Ocean) to protect the estuaries from adverse effects.

From the results of the computer modeling, the best possible distribution of wastewater effluent between the seven sites was determined, and the conclusions are outlined in this report.

Capstone Design

This project satisfies the capstone design requirement for a Bachelor of Science degree in Civil & Environmental Engineering at Worcester Polytechnic Institute. As mentioned in the methodology section, this project includes preliminary design work for a discharge facility to be located at one of several possible sites. This facility must be designed to distribute wastewater effluent in a manner that minimizes nitrogen loading in surrounding water bodies under several constraints. The constraints considered include but are not limited to: cost, the size of the site, availability of technology (nitrogen concentration of treated effluent), and impact on the surrounding area including residential areas, buildings and infrastructure, and current land uses. The design also depends on factors such as hydraulic conductivity of the soils at the chosen site, as well other physical parameters that must be modeled using Visual MODFLOW. These factors are considered in light of existing designs and technologies for solutions to similar problems. This design problem satisfies the requirements for capstone design experience for several reasons. First, it includes both an analytical component (modeling and evaluating potential sites) and a synthetic component (designing and locating discharge facilities with sufficient capacity to meet the requirements of the Town of Mashpee). This preliminary design is the result of iterative conceptualization, testing, and redesign and it presents a solution to an open-ended problem. Lastly, this design process is dependent on significant decision-making from beginning to end in creating a finished project.

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1 Introduction

Over the last several decades, the peninsula of Cape Cod, Massachusetts, has experienced sudden and rapid population growth. This growth, attributed by many to the pristine beauty of Cape Cod's many coastal waters and unspoiled natural features, jump-started residential and commercial development in the area. Ironically, increased development has reduced the area devoted to wild spaces and raised growing concerns about the health of water bodies that make up the coast of Cape Cod. Development has a two-fold effect on local ecosystems: buildings and infrastructure devoted to human activity put a direct strain on the surrounding area because they contribute to higher runoff, erosion, and increased levels of contaminants. Additionally, once an area is developed it loses the ability to remediate the effects of other nearby developed areas. Thus, the conversion of open spaces and wilderness area to developed residential and commercial areas sets in motion a positive-feedback loop that can have serious consequences for coastal waters and other natural features.

The Woods Hole Research Center estimates that 70% of the cape was forested in 1951; by 1990, this forest cover had been reduced to 45% (a loss of about 100 square miles), mostly due to increased residential and commercial development (Woods Hole Research Center 1). Additionally, increased development on Cape Cod has affected the precious coastal resources which characterize a distinct way of life. According to the Massachusetts DEP Estuaries Project, septic systems on residential properties are the primary source of nitrate entering the groundwater (Howes et al.). Nitrate is a chemical found in human waste and is a critical nutrient for plants and animals of all kinds. At normal concentrations it is good and necessary, but at excessive levels it tends to encourage the growth of aggressive species of algae, which experience sudden and massive population blooms. Nitrate from septic systems travels through the groundwater and eventually seeps into rivers, coves, and bays along the coast. Measured as "total nitrogen," it collects in these water bodies, often at rates higher than normal attenuation and flushing can disperse. Thus, algae choke out other forms of life and then die, destroying ecosystems and turning formerly clear, blue, lively waters into murky, green, dead swamps.

In order to accommodate the wastewater from a large population on the Cape without having an adverse effect on coastal waters, the level of nitrogen entering estuarine systems from wastewater must be decreased. Modern wastewater treatment facilities can reduce the nutrient concentrations to less than 10% of that of untreated sewage, but the products of the treatment process must still be disposed somewhere near the facility. Currently, treated sewage – known as effluent – from wastewater treatment facilities around the United States is discharged into many different types of water bodies including rivers, lakes, estuaries, coastal waters, and groundwater (Arceivala 892; Metcalf & Eddy, Inc. 1334). The choice of disposal method is based on engineering factors such as geography, water quality and water volume, as well as legal and political factors. In southeast Massachusetts geography is the critical factor. All the rivers in this area are small and flow directly to the Atlantic Ocean. The only surface water bodies that could potentially absorb effluent discharge are the estuaries and the ocean itself. However, The Massachusetts Ocean Sanctuaries Act prohibits the discharging of domestic or municipal waste directly into ocean sanctuaries, so on Cape Cod, Massachusetts, the most viable option for disposing of wastewater is to discharge it into the ground.

The process of returning wastewater effluent to the ground is complicated by the dynamic properties of groundwater. Over long periods of time, certain contaminants and nutrients flow with groundwater, and can either remain in the flow or be absorbed, filtered, and transformed by the biological, chemical, and physical nature of the soil through which the groundwater passes. Eventually, groundwater returns to the surface through springs or seeps into the bottom of lakes, rivers, estuaries, and coastal waters. This process often supplies a significant fraction of the nutrient loading to the surface water bodies. By treating wastewater before it is discharged into the groundwater, a great portion of nutrient loading into surface water bodies will decrease.

The Town of Mashpee, MA is currently evaluating the impacts of nitrogen on its estuaries and has begun the process of identifying potential solutions through the development of a Watershed Nitrogen Management Plan. As part of this process, seven sites have been identified, each with unique physical and geological characteristics, which could be suitable for the construction of a treated effluent discharge facility.

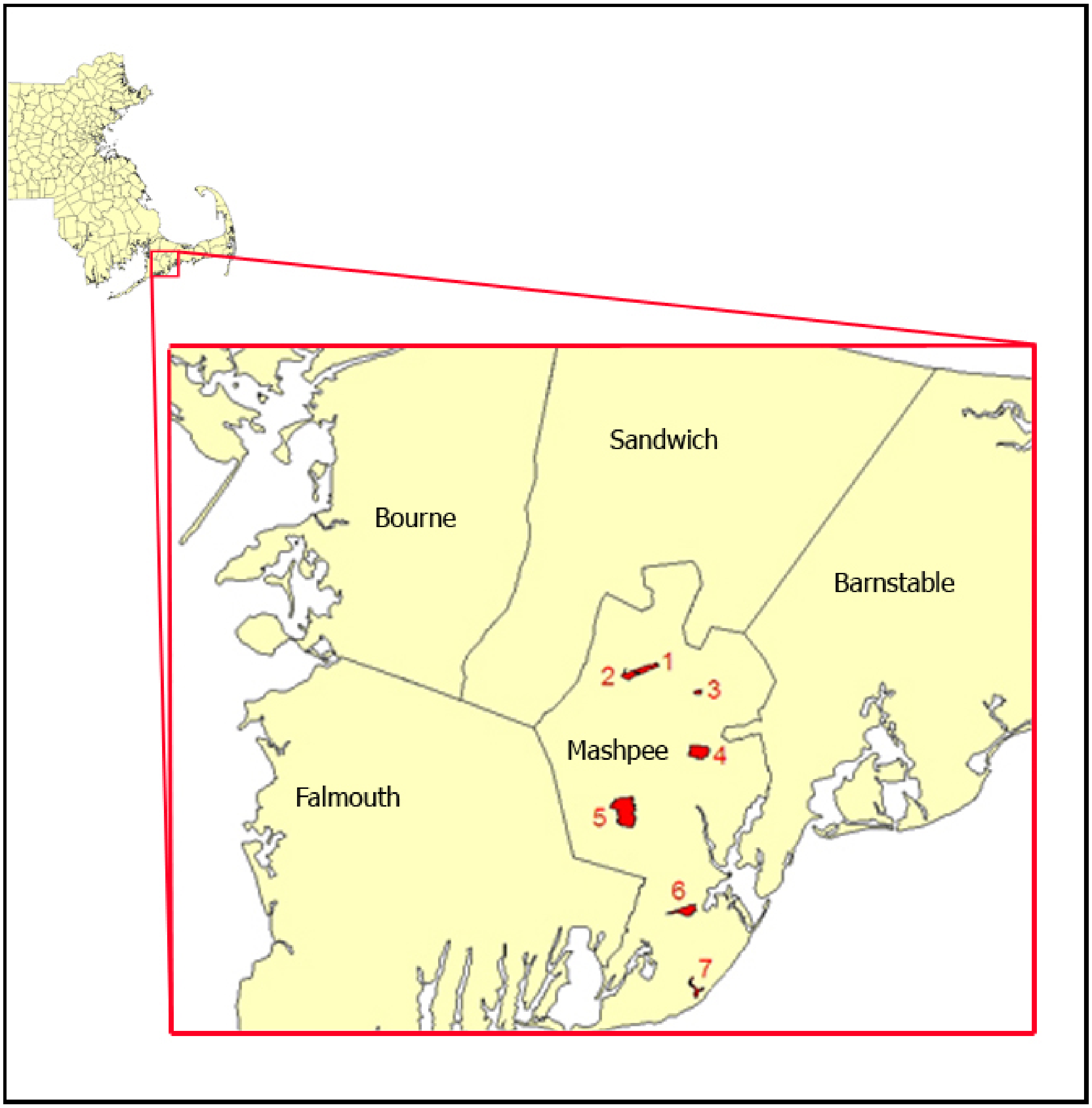

Fig. 1-1 – Proposed Discharge Sites Near Mashpee, Massachusetts

The seven sites are shown in Fig. 1-1. Site 1, the Ball Fields, contains a cluster of baseball fields and a parking lot located across the road from Site 2, Ashumet Road, a large forested area. Site 3, the Old Town Dump, is just southeast of the first two sites. Site 4, the Transfer Station for the town, includes a large undeveloped area nearby. Site 5 is a public High School and the surrounding area. Site 6, known as the Keeter Property, is a large forested area in the south end of town, directly west of Popponesset Bay. Site 7 consists of three holes in the New Seabury Country Club golf course a few hundred feet from the coast.

Each potential discharge site varies in surface area size, location, infiltration capabilities, and groundwater flow patterns. The size of a site determines how much wastewater can be discharged into the ground. The location of the site is important when considering proximity to wells, surface waters, buildings, and other land features. The infiltration capability of each site depends on whether the site can have open bed infiltration basins or if it must have subsurface leaching trenches, which depends on current land use. The groundwater flow patterns beneath these sites must be determined through computer modeling. It is important to determine the features associated with each site and consider them as components of a complex dynamic system in order to decide on an ideal site or set of sites that have favorable discharge conditions.

The goal of this project is to help protect the health of the estuaries near Mashpee, Massachusetts by developing plans that will reduce nitrogen levels to below the maximum safe levels recommended by the Massachusetts Department of Environmental Protection (MassDEP) and outlined in their Total Maximum Daily Load (TMDL) reports. In order to achieve this goal, two objectives have been identified for this report: The first objective is to evaluate these seven potential discharge locations based on a set of objective criteria and determine which site (or sites) represents the best option for reducing estuarine nitrogen levels. The second objective is to provide a preliminary design for treated-effluent disposal facilities at the site or sites.

To address the first objective computer modeling software is used to evaluate the effects of discharging effluent at each site, taking into account the properties of each site and the required daily wastewater generation of the Town of Mashpee. A computer model is used to trace effluent through the aquifer from the discharge site to its destination, following groundwater flow patterns. The results of this modeling indicate total nitrogen load from each discharge site to each coastal water body. To address the second objective a system of discharge locations is chosen based on the modeling results of the first objective. Discharge sites will be designed to minimize estuarine nitrogen loads, while considering constructability, and the impact on the local water table. The results and recommendations from this project are intended to help the Town of Mashpee as it continues its efforts to protect its coastal resources.

2 Background

There are many factors that distinguish Cape Cod from other regions of New England, North America, and the world. A thorough understanding of the unique geology of the region is vital to any investigation of local hydrogeological conditions. Other topics that must be included are the effects of nutrient loading, the science of nitrogen, the current state of the water bodies under consideration (notably Popponesset and Waquoit Bay) regarding nitrogen, and the technology behind the nitrogen removal process, including the discharge of wastewater effluent. This section contains relevant background information on which the methods, results, and conclusions of the report are based, as well as a description of the software tools used to reach these conclusions.

2.1 Environmental and Geological History of Cape Cod

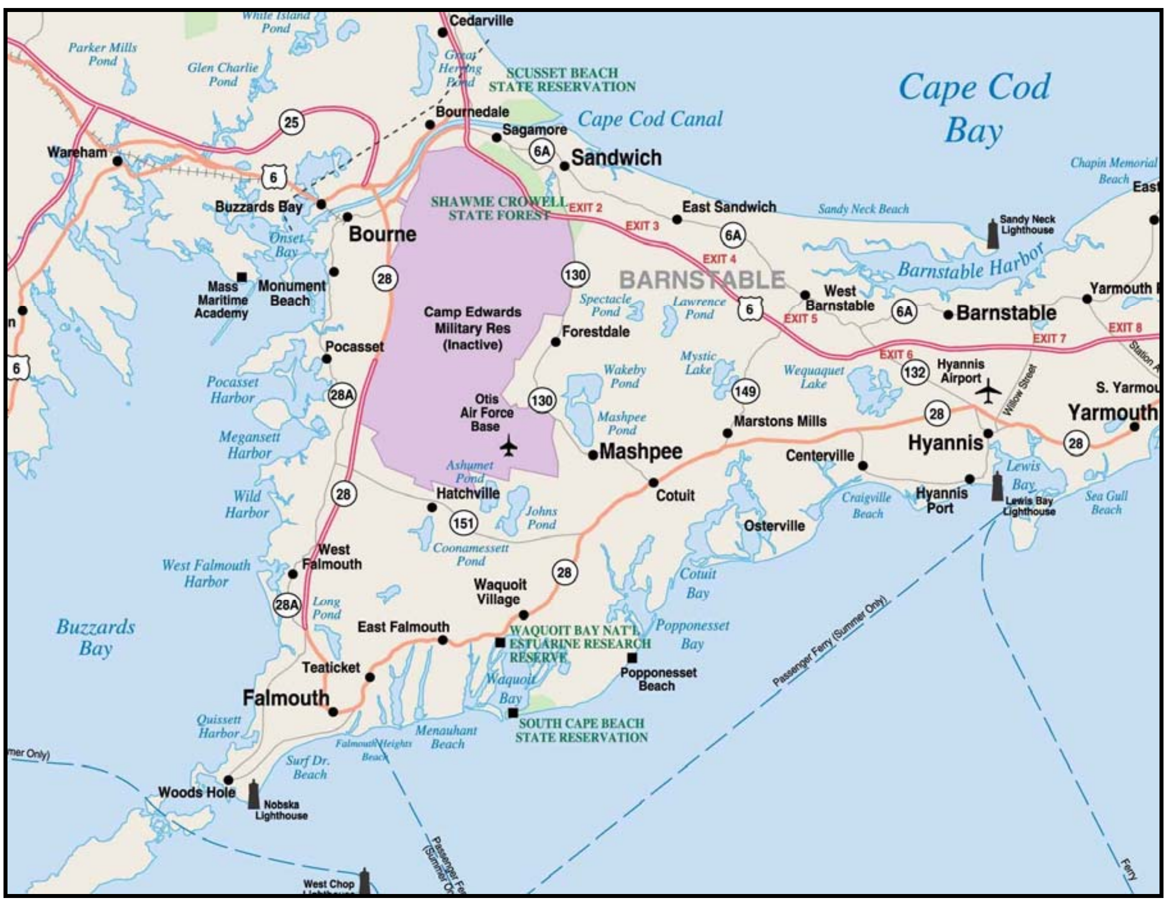

Fig. 2-1 – Major Cities, Water bodies, and Features of Western Cape Cod (CapeCodTravel.com)

The Peninsula of Barnstable County, Massachusetts – also known as Cape Cod – is essentially a large deposit of primarily loose, sandy soils extending out into the Atlantic Ocean. According to the USDA Natural Resources Conservation Service's Cape Cod Water Resources Restoration Project, the Cape area consists of 263,988 acres which is broken down into roughly 42% developed land, 32% forests, and 25% wetlands, open spaces, and other uses (USDA Natural Resources Conservation Service). The bedrock under the Cape region is entirely below sea level, so the groundwater table consists entirely of soil (Maiorano et al.). When colonists first arrived on Cape Cod the entire peninsula was heavily forested, but due to early development the region was almost completely cleared by the early 1800s. Although forest land made a brief resurgence in the 1950s, it has again declined in the face of commercial and residential development (Woods Hole Research Center 1). This project will concentrate on Western Cape Cod, the area portrayed in Fig. 2-1. Some features of note: The estuaries with which this project is concerned are located on the southern shore of the peninsula, near Waquoit and Popponesset Bays. Ashumet and Johns Ponds are large kettle ponds formed by glacial activity, discussed later in this section. The municipalities located within the watersheds of interest for this project include Falmouth, Sandwich, Barnstable and Mashpee.

During the late Pleistocene era approximately 25,000 years ago, a glacier known as the Laurentide ice sheet advanced south from Canada and covered what is now known as New England and Cape Cod. The actions of this glacier, including its eventual retreat and the fluctuations in sea level associated with it, are responsible for the particular geological character of Cape Cod. By about 23,000 years ago, the glacier had advanced to its southernmost point in the vicinity of Martha's Vineyard (See Fig. 2-2). Between 23,000 and 18,000 years ago, the climate warmed somewhat and the glacier began to retreat northward towards the Gulf of Maine (Oldale). During this retreat, a series of warm spells caused a great deal of glacial meltwater to flow off the southern edge of the receding ice sheet and into the ocean. This water carried a great deal of sediment out of the ice sheet and deposited it in wide, sandy areas known as glacial outwash plains which are instrumental to understanding the hydrogeology of Cape Cod (National Park Service).

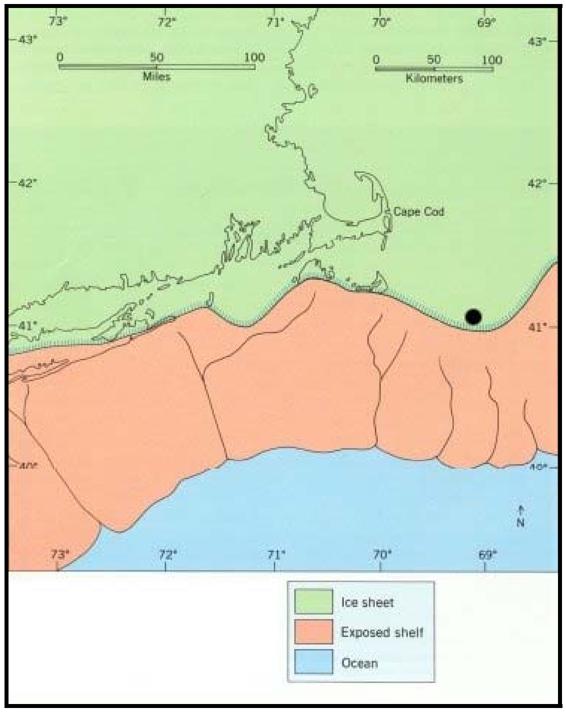

Fig. 2-2 ─ Southernmost Extent of Laurentide Ice Sheet (Oldale)

When the glaciers retreated, they left behind a layer of bedrock like that under the rest of New England, as well as a layer of glacial drift deposit consisting of between 200 and 600 feet of very fine- to very coarse-grained rock debris (Oldale). On top of that, Cape Cod consists mostly of glacial outwash deposits – stratified layers of fine- to medium-grained sand and gravel and some fine layers of silt deposited in broad, flat plains sloping down slightly toward the coast in every direction (National Park Service). According to Barnstable County Soil Survey, soils characteristic of glacial outwash deposits are poor filters for septic systems, sewage lagoons, and sanitary landfills (U.S. Soil Conservation Service). In fact, according to A.H. Brownlow of the Boston University Dept. of Geology:

> "Soils which are coarse and sandy are highly permeable and allow effluent waters to travel quickly over large distances. Low organic matter and clay content provide little contaminant removal through soil sorption or cation exchange. Low organic content of the soils also decreases bacterial immobilization of nutrients as well as denitrification of nitrate-nitrogen. As a result, Cape Cod ground water is susceptible to contamination." (Brownlow)

Due in part to this susceptibility, there have been several incidents of major soil and groundwater contamination on the Cape in the last 100 years, many as a result of the development of the Massachusetts Military Reservation (MMR) in 1911 (Maiorano et al.). During the early years of the MMR, the effects of hazardous materials on the groundwater system were poorly understood and many contaminants including solvents, volatile organic carbons (VOCs), and jet fuel were dumped directly into the ground or into surface water bodies. These contaminants spread out from the MMR following the prevailing subsurface water flows in the region (see Section Hydrogeology of Cape Cod and the Town of Mashpee). The heaviest dumping at the site began in 1955, and by 1979 high levels of detergents, VOC, and other contaminants were discovered in sites all over the peninsula. Contaminant “plumes” were discovered in the area where pollutants in the groundwater exceeded the levels for safe drinking water. Since that time, the EPA, USAF, and other organizations have undertaken major operations to reduce point source pollution on the MMR and remediate existing contaminant plumes (Maiorano et al.).

2.2 Hydrogeology of Cape Cod and the Town of Mashpee

Cape Cod is a peninsula that is separated from the rest of Massachusetts by the Cape Cod Canal. Therefore, all of the surface and ground water is precipitated locally, and has a short path to the ocean. In addition, due to topography, land use, and soil types, roughly half of the precipitation is percolated into the groundwater (U.S. Geological Survey). The soils in this area are primarily coarse-grained, and have high levels of sand, especially near the surface (Howes B., S.W. Kelley, J.S. Ramsey, R. Samimy, D. Schlezinger, T. Ruthven, and E. Eichner). These soils exist primarily in glacial outwash plains formed by glacial activity thousands of years ago. The Natural Resources Conservation Service estimates that up to 94% of soil in the area are “deep, excessively drained or well-drained sands formed primarily in outwash plains” (USDA Natural Resources Conservation Service). This loose soil base facilitates contaminant transport through the water table.

As seen in the accompanying figures, the high point in the water table of Barnstable County is located in the northern region of the Massachusetts Military Reservation, causing contaminants to radiate outward from the reservation, following the hydraulic gradient of approximately 0.002 ft/ft (Maiorano et al.). Within the Town of Mashpee, groundwater generally travels from the north to the south, toward Nantucket Sound. There are two major estuaries and accompanying watersheds that dominate the coast of Mashpee. Popponesset Bay is on the eastern border, while the eastern portion of Waquoit Bay is on the west. The Popponesset Bay estuary and the East Waquoit Bay estuary have been studied and a formal hydrogeological report has been written through the Massachusetts Estuaries Project. Work has yet to be completed on the entire Waquoit Bay estuary system, which extends into Falmouth.

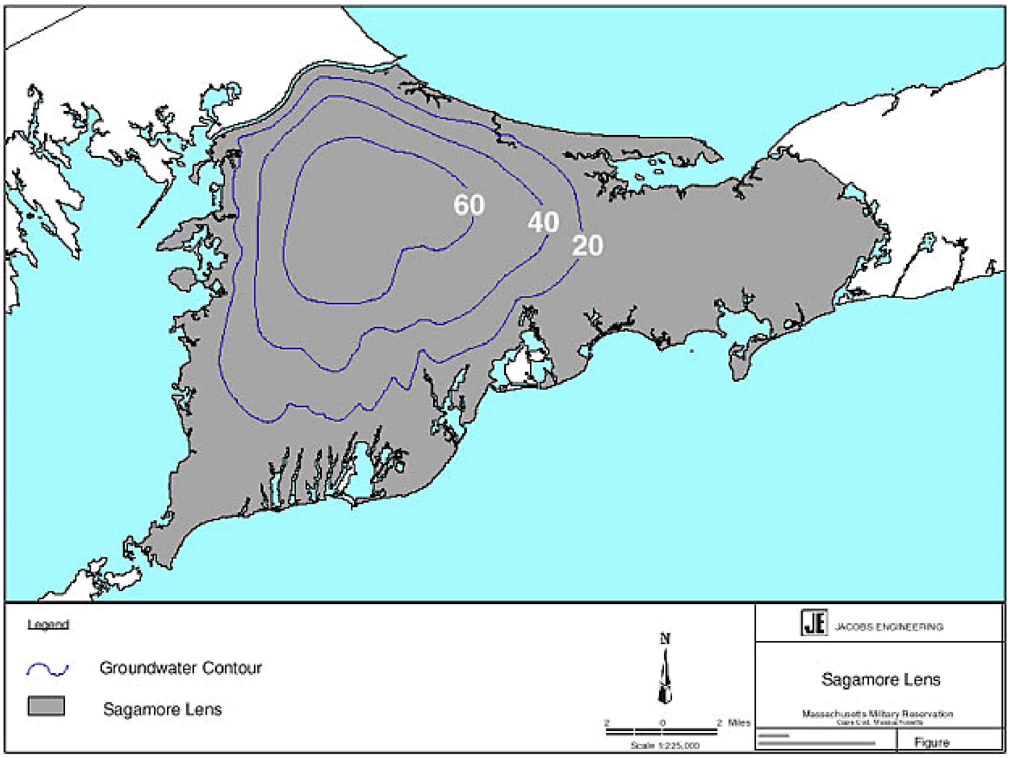

Fig. 2-3 ─ Map of groundwater contours over the Sagamore Lens (Massachusetts Military Reservation)

The Western Cape Cod region is also pockmarked with kettle ponds, characteristic of regions shaped by glacial movement. These ponds form as a result of impressions made in the land when ice sheets advance across a particular region. After the glacier has retreated out of the area, the ponds fill with groundwater; as a result, the surface of the kettle ponds actually represents the height of the water table in the region. These ponds, along with other surface water bodies, affect the flow of groundwater in the entire region, often increasing water transmission in the surrounding area. Some kettle ponds, such as Ashumet Pond along the border between Mashpee and Falmouth or Johns pond at the heads of the Quashnet and Childs Rivers, facilitate groundwater transport because of their large size relative to the thickness of the surrounding aquifer and, in the case of Johns Pond, their direct connection to flowing surface water bodies (Maiorano et al.). Other surface water bodies in the region include small streams, marshes (salt and freshwater) and bogs. There is little substantial river activity on the cape owing to the small watershed and highly permeable soil systems. In addition, there are 109,000 acres of floodplains (USDA Natural Resources Conservation Service).

Barnstable County, and especially the Western Cape Cod region, is served by a "sole source" aquifer, extending up to 400 feet below the surface in some areas. The "sole source" designation, monitored by the EPA, means that the aquifer supplies at least 50% of the drinking water for the area and all activities which could affect the aquifer are subject to EPA review, such as agricultural runoff, road runoff, or nutrient leaching from poorly-designed septic tank systems (USDA Natural Resources Conservation Service).

2.3 Current Contamination Issues in Surface Water Bodies

The Massachusetts Department of Environmental Protection (MDEP) regularly prepares water quality reports for all surface water bodies in the state, assigning each body a ranking from Category 1 ("unimpaired," or known to be unpolluted) to Category 5 ("impaired in one or more categories," or known to have levels of at least one contaminant exceeding state maximums). For each body of water in Category 5, the state is required to write a Total Maximum Daily Load (TMDL) report detailing the problem, suspected source of contamination, maximum acceptable contaminant levels and a plan to reduce observed levels to below the limits. In the most recent report, released in 2004, 80 water bodies or segments of water bodies on Cape Cod were listed in Category 5. Of those 80 bodies, 30 have been listed in Category 5 due to nutrient overloading and risk for eutrophication (USDA Natural Resources Conservation Service). Two of these bodies are of particular interest for this project- Waquoit Bay and Popponesset Bay. The TMDL reports prepared for Popponesset Bay and Eastern Waquoit Bay will be discussed in detail in the following sections. A TMDL has not yet been prepared for Western Waquoit Bay.

2.4 The Nitrogen Cycle

Nitrogen is a nutrient element that is required for all life on Earth because it is a vital building block of all amino acids and proteins as well as the nucleic acids in DNA and RNA. It is found naturally in all ecosystems, and is often a limiting factor in plant growth due to the fact that it is required in chlorophyll molecules used in photosynthesis. Nitrogen moves between the atmosphere, plant crops, soil, groundwater, and surface water in a complex cycle known appropriately as the nitrogen cycle. The basic structure of the nitrogen cycle is as follows: gaseous atmospheric nitrogen enters the soil as organic nitrogen which is subsequently decomposed into ammonium by microorganisms. Further microorganisms convert this ammonia into nitrates, which can be leached into groundwater or converted back into atmospheric nitrogen, completing the cycle (Davis and Masten).

Nitrogen exists in a concentration of about 78% in the Earth's atmosphere, in a form known as "diatomic nitrogen" or N2. This form of nitrogen is chemically inert and not biologically reactive, so in order to have an impact on the ecosystem it must be converted to one of several forms. The first major pathway is called "nitrogen fixation." Many species of bacteria, some free-living and some symbiotic with the roots of certain plants (particularly legumes such as soy or alfalfa), are capable of converting atmospheric nitrogen into an organic form and "fixing" it in the soil. Additionally, all living matter contains nitrogen. When living matter dies, the nitrogen it contains is returned to the soil as organic nitrogen (Smil 221). Once this nitrogen is in the soil, many different species of bacteria turn it into ammonium ions (NH4+) in a process known as "mineralization." In living material, this is a part of the process of decomposition and occurs whenever dead plant or animal matter or animal waste, including human waste, enters the soil (Davis and Masten).

Another major pathway is the industrial production of ammonia fertilizers from nitrogen and hydrogen gases, known as the "Haber-Bosch" process. When these fertilizers are spread on soil, the ammonium ions are transferred to the ground. However, this form of nitrogen is still not biologically useful as a nutrient- it must be "nitrified" by several species of bacteria which will convert it first into NO2 and then NO3, which is called nitrate, and is a valuable nutrient for all forms of life. Unfortunately, unlike ammonium ions that bind tightly to soil particles, nitrate molecules are highly mobile and leach easily into groundwater. This is also the principle method by which nitrogen from human waste enters groundwater (Davis and Masten).

Typically the nitrogen cycle is completed two ways: either nitrates are consumed by plant matter as nutrients or they are "denitrified" by other species of bacteria and returned to the atmosphere as diatomic nitrogen once more. However, in areas with downward groundwater flow mobility a great deal of nitrate matter is leached into groundwater and, eventually, into surface water bodies near the source of the nitrogen (Davis and Masten).

2.5 The Effects of Nitrogen in Surface Water Bodies

Nitrogen from human sources, such as fertilizer or septic tanks, ends up in surface water bodies near the source including estuarine surface water bodies in Cape Cod. Although a certain level of nitrogen is necessary to maintain the health of an ecosystem, excess nitrogen has many ill effects. The growth of algae and other microorganisms in coastal waters and estuaries is usually limited by the availability of nitrogen. When additional nitrogen is introduced, algae will begin growing at a much higher rate than usual. This is known as "algae bloom" or "eutrophication." The increased population of algae will absorb more dissolved oxygen than the water body can provide, resulting in a condition of hypoxia, which means that the dissolved oxygen (DO) concentration is less than 1.0 mg/L (Davis and Masten).

Low DO concentrations put extreme stress on all other forms of life in the water body, often reducing populations of other species to very low levels. Additionally, the presence of algae causes drastically increased turbidity in the water, which reduces the amount of available sunlight to subsurface aquatic vegetation. In many water bodies, submerged vegetation provides food and shelter to a significant portion of the species. During eutrophication, the only biologically productive life is algae, which is undesirable. In addition to the effects listed above, algae cause unwanted tastes and odors and reduced water clarity. During hypoxia, there is a significant reduction in the biological productivity of a body of water. Hypoxic zones are often known as "dead zones." Unfortunately, after a period of increased Nitrogen loading, many water bodies oscillate between eutrophication and hypoxia and are unable to return to normal productivity until the source of nitrogen has been removed (Davis and Masten).

2.6 The Massachusetts Estuaries Project

The Massachusetts Estuaries Project (MEP) was formed as a collaborative effort between the MassDEP and the School of Marine Science and Technology (SMAST) at the University of Massachusetts Dartmouth. The MEP was chartered with the responsibility of investigating the current water quality situation in estuaries along the Southern coast of Massachusetts, specifically with regards to Nitrogen overloading, to determine the current concentrations of nutrients in estuarine waters and the vulnerability of these waters to eutrophication and algae bloom. The goal of the MEP is to determine what level of nutrient loading each estuary can handle without suffering permanent damage and to identify strategies to permanently reduce nutrient levels to below that threshold (Massachusetts Estuaries Project 1).

Estuaries are bodies of water that reach inward from the coast, connecting to the mouth of a river or a stream at some point inland. As these streams or rivers are usually the outlet of a watershed, estuaries often receive the entire contaminant load in the runoff and groundwater flow from that watershed. This can be disastrous to the health of marine ecosystems in the estuary. Because they are relatively calm waters protected from seaborne predators and coastal weather patterns, estuaries are often fragile habitats and can be permanently disrupted by the introduction of excessive nutrient loads or other contaminants. In order to protect estuarine habitats along the southern coast of Massachusetts, the MEP must trace inland from each estuary to establish the watershed that feeds it. One the source of contamination has been identified, a TMDL report is prepared outlining the capacity of the estuary to handle nutrient loading- in most cases, nitrogen runoff from agricultural activity or human septic systems- and how to reduce nutrient load to reasonable levels (Massachusetts Estuaries Project 1).

There are two major estuarine systems along the coast of Mashpee- Popponesset Bay to the east and Waquoit Bay to the west. The MEP has prepared a TMDL report for each of these systems, including the bay and major embayments (smaller bays or features of the estuarine system, linked inextricably to the bay but separate entities for the sake of analysis).

2.7 Summary of Eastern Waquoit Bay TMDL Report

Waquoit Bay is the westernmost of the two major estuarine systems in the Mashpee region of Western Cape Cod. Located on the south side of the peninsula, it is bordered by Mashpee to the east and Falmouth to the north and west. The bay itself is a large expanse of open water which flows into the Nantucket Sound to the south through a single direct outlet, and to the west through Eel Pond, a separate body of water. Immediately to the east are two large ponds known as Hamblin Pond and Jehu Pond which are connected to the Bay through the Little River and Great River respectively. The Quashnet River flows into the northern part of the bay and extends inland for about a mile before reducing to a stream that continues inland for several miles. Collectively, the ponds and the river are referred to as "embayments" of the bay. These embayments exhibit many characteristics typical of estuaries including substantial salt marsh area, relatively low tidal volume and salinity variations. Conversely, the bay itself exhibits none of these characteristics but is an open water system with only fringing salt marshes and a "large basin volume relative to tidal prism" (Howes B., S.W. Kelley, J.S. Ramsey, R. Samimy, D. Schlezinger, T. Ruthven, E. Eichner).

Fig. 2-4 – Western Cape Cod, Waquoit Bay Highlighted

The TMDL Report prepared by the MEP is the result of an ambitious project undertaken to understand how contaminants into the bay, specifically the nutrient nitrogen. In order to determine the source of contaminants entering the area, the authors of the TMDL report first defined the entire eastern portion of the watershed for Waquoit Bay, then examined the activities for which that land was being used and generated predicted nutrient loads based on land use and existing nutrient loading data for various activity types. Additionally, the authors examined the roles of surface water bodies in contaminant transport and nitrogen attenuation between the source and the bay. Finally, by applying sampled nutrient levels to measured groundwater and stream flow, they determined a “budget” of nitrogen entering the bay and established a threshold for healthy bay activity based on several biological signals such as dissolved oxygen concentrations, eelgrass fields, and chlorophyll concentrations (indicating phytoplankton growth) (Howes B., S.W. Kelley, J.S. Ramsey, R. Samimy, D. Schlezinger, T. Ruthven, E. Eichner).

Fig. 2-5 ─ Major Components of the Waquoit Bay Estuarine System (Howes B., et al.)

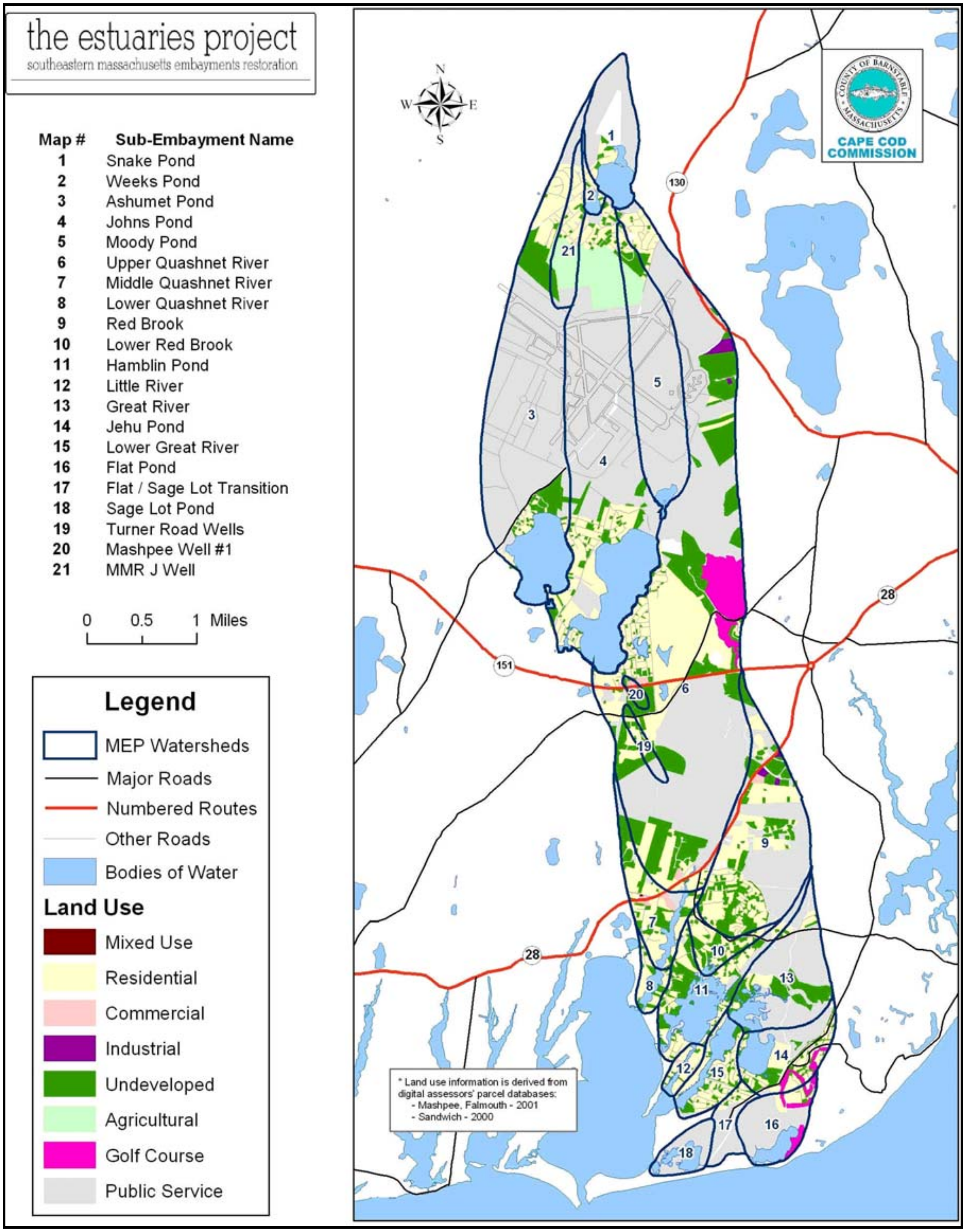

Fig. 2-6 ─ Land Use Coverage in the Waquoit Bay Watershed (Howes B., et al.)

Before land use can be examined the watershed for the eastern portion of Waquoit Bay must be clearly delineated. Once the watershed is well-understood, the sources of nutrient loading can be easily identified- nitrogen originating from any point within the watershed will eventually make its way to the bay. More importantly, by dividing the watershed into several "sub-watersheds," nutrient loads can be budgeted to different areas that have different land use patterns. To this end, the MEP used Visual MODFLOW and the USGS regional model for Cape Cod – the same model and software package used in this report – to predict groundwater flow from areas surrounding the bay. The watershed is defined as any area from which groundwater eventually enters Waquoit Bay or its embayments, either by direct groundwater discharge or by seeping into surface water bodies that then flow above ground into the estuary. It was determined that the watershed for this portion of Waquoit Bay extends northward through Falmouth and Mashpee, with the very tip in Sandwich (Howes B., S.W. Kelley, J.S. Ramsey, R. Samimy, D. Schlezinger, T. Ruthven, E. Eichner).

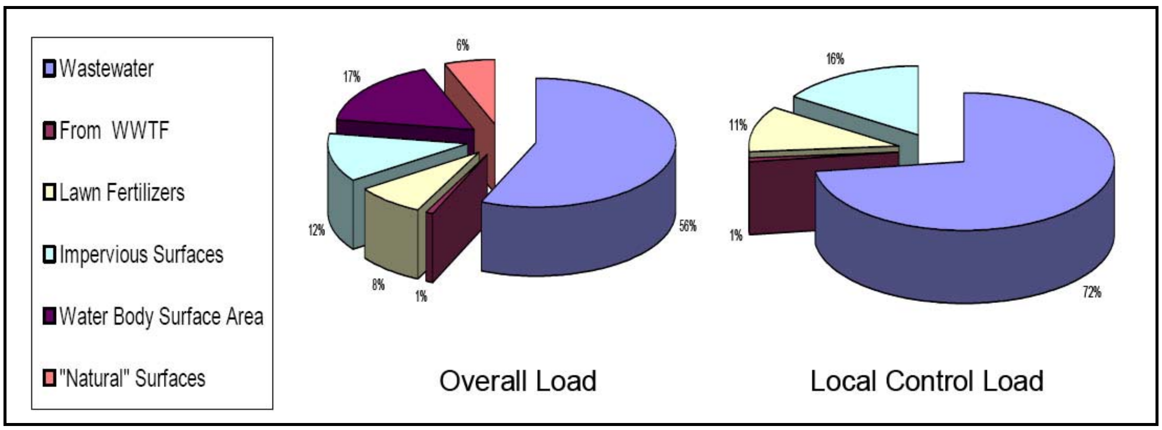

Fig. 2-7 – Nitrogen Loading by Land Use in Waquoit Bay (Howes B., et al.)

Land use within the eastern Waquoit Bay watershed is substantially different than other watersheds in Cape Cod because the Massachusetts Military Reservation occupies a large portion of the watershed. In fact, land devoted to public or government service occupies 54% of the watershed (as seen in Fig. 2-6 above). The rest is light residential, undeveloped, golf courses, or mixed use, in that order (Howes B., S.W. Kelley, J.S. Ramsey, R. Samimy, D. Schlezinger, T. Ruthven, E. Eichner). Residential areas in the watershed contribute the vast majority of nutrients in the groundwater, either through leaching from septic systems (97% of homes in this watershed use private septic systems) or runoff from lawn fertilizers (the average home lawn in this watershed contributes 0.5 kg of nitrogen directly into the ground each year). Aside from lawns, the source of nutrients in this area is wastewater from residential, commercial, or light industrial uses – see Fig. 2-7 above (Howes B., S.W. Kelley, J.S. Ramsey, R. Samimy, D. Schlezinger, T. Ruthven, E. Eichner).

Wastewater is known to have an average nitrogen concentration of 35 mg/L, and the volume of wastewater can usually be assumed to be 90% of the total water use for an area. From these numbers, a total nutrient load for the area can be calculated. There are only a few exceptions: There are two minor, private wastewater treatment facilities (WWTF) in this watershed, which process 19,400 GPD of wastewater between them and release treated wastewater effluent with a lower nitrogen concentration (5-7 mg/L). More importantly, a single well pumps water from the aquifer for use at the MMR. However, the MMR has its own WWTF which discharges outside of the watershed near the Cape Cod Canal. Therefore, this well represents a significant removal of nitrogen from the watershed. Based on flow rates and sampled nitrogen concentrations of pumped well water, MEP researchers determined that this well removes 1,061 kg of nitrogen per year (Howes B., S.W. Kelley, J.S. Ramsey, R. Samimy, D. Schlezinger, T. Ruthven, E. Eichner).

Aside from the well, anything that enters the groundwater inside this watershed will eventually end up in the bay. However, the amount of nitrogen entering the embayments is significantly lower than the amount generated in the watershed over any period of time. During transport, a great deal of groundwater ends up in streams, kettle ponds, or other surface water bodies. In these water bodies, a bacterial process known as denitrification removes nitrogen from the water and releases it as inert, diatomic nitrogen gas into the atmosphere. This is called "attenuation," and some water bodies with high residence times and active biological populations can remove as much as 89% of nitrogen from the water, although the mean attenuation rate for measured surface water bodies in the Waquoit Bay watershed is actually 63% (Howes B., S.W. Kelley, J.S. Ramsey, R. Samimy, D. Schlezinger, T. Ruthven, E. Eichner). Some groundwater never enters a surface water body but instead seeps directly into the estuary, so the overall attenuation rate of the surface-water-system was found to be 39% (Howes B., S.W. Kelley, J.S. Ramsey, R. Samimy, D. Schlezinger, T. Ruthven, E. Eichner).

Very little from the eastern portion of the watershed flows directly into Waquoit bay itself; most of the nitrogen initially flows into one of the estuaries (Quashnet River, Hamblin Pond/Little River, Jehu Pond/Great River). Thus, there are several interfaces in the bay itself: between the bay and each of its embayments, between the bay and direct groundwater seepage, and between the bay and Nantucket Sound. These interfaces involve not only nitrogen transport, but drastic salinity gradients between inland freshwater systems and the saltwater sound as well as significant differences in depth, tidal fluctuation, etc. Modeling the bay indicated, not surprisingly, that nitrogen concentrations are highest in the more inland sections of the embayments, decreasing steadily out toward the bay. This is unfortunate because the embayments will suffer most from eutrophication and algae bloom due to low tidal flushing volume and high residence times, as well as the close proximity of residential and recreational spaces. Additionally, the embayments are the habitat of choice for many aquatic species such as eelgrass, which are at risk from algae bloom and low dissolved oxygen concentrations (Howes B., S.W. Kelley, J.S. Ramsey, R. Samimy, D. Schlezinger, T. Ruthven, E. Eichner).

Because these species are at risk, they make excellent indicators of the health of estuarine ecosystems. The TMDL report mentions three such indicators of marine health: presence of eelgrass beds, dissolved oxygen concentrations, and chlorophyll concentrations. Reduction of eelgrass vegetation due to nutrient loading has been noticed as long ago as the late 60s, and was confirmed in a 1990 study. A 2001 survey confirmed that eelgrass beds were withdrawn more than ever before seen – see Fig. 2-8. Similarly, dissolved oxygen levels are lower than ever before, and chlorophyll concentrations are higher. These indicators are the basis for the threshold determination – the capacity of the bay to accept, hold, and cycle nitrogen. The report concludes that in order to return eelgrass populations, dissolved oxygen and chlorophyll concentrations to healthy levels (and restore the ecosystem of the bay and embayments), nitrogen concentrations in the bay must be reduced to below 0.38 mg Nitrogen/L (mg N/L). Current nitrogen concentrations in the upper portions of the Quashnet River or Hamblin/Jehu pond embayments is as high as 0.90 mg/L (Howes B., S.W. Kelley, J.S. Ramsey, R. Samimy, D. Schlezinger, T. Ruthven, E. Eichner). The report ends with the recommendation that septic nitrogen loads must be completely eliminated in order to reduce nitrogen loading in the bay to acceptable levels.

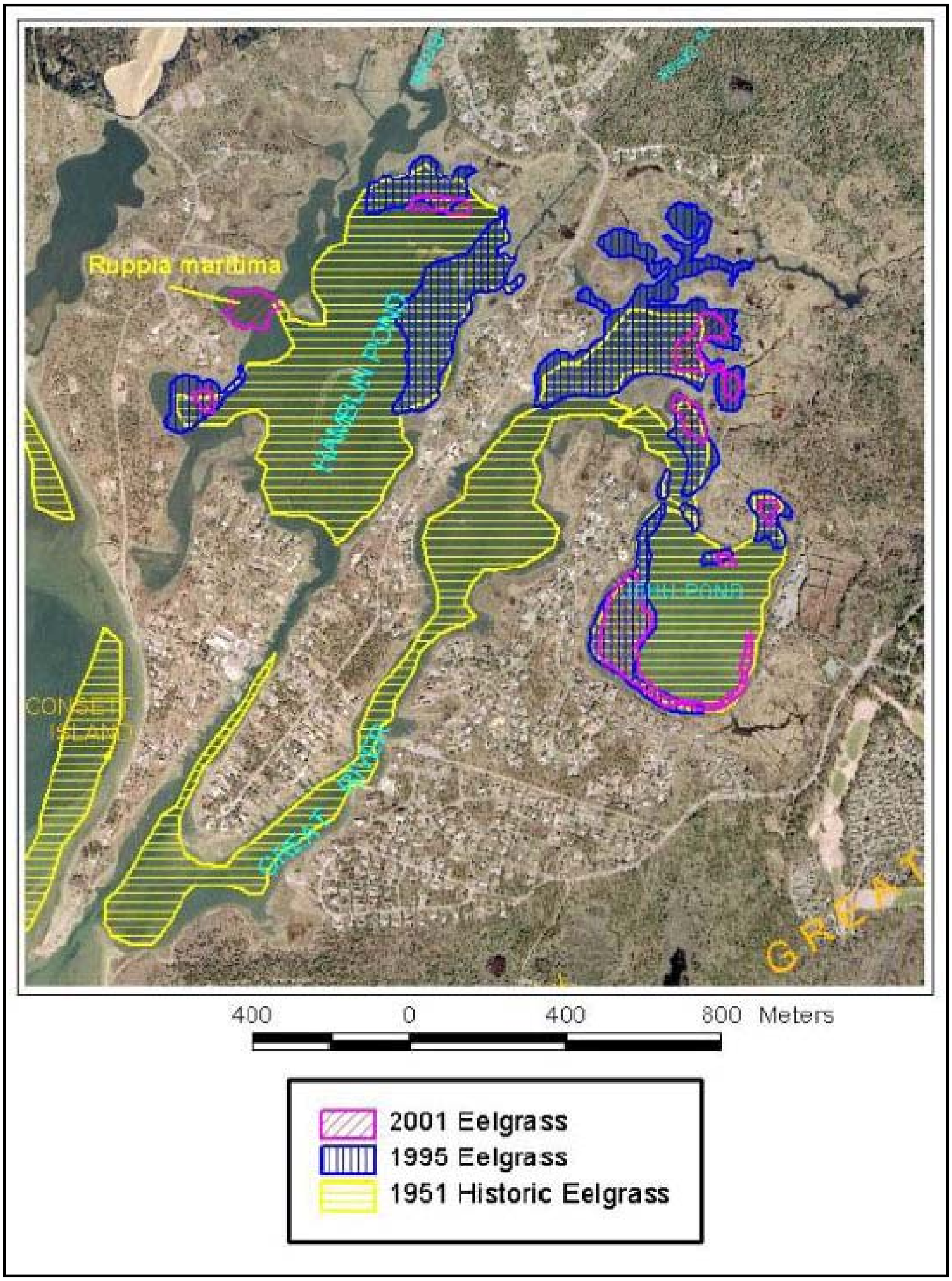

Fig. 2-8 – Reduction of Eelgrass Beds Since 1951 (Howes B., et al.)

2.8 Summary of Popponesset Bay TMDL Report

Popponesset Bay is the easternmost of two large estuarine systems on the south coast of Cape Cod, near Mashpee. Mashpee borders it on the north and west, and Barnstable on the east. The Popponesset estuarine system consists of the bay itself, a large open expanse of water, and several embayments, tributary rivers, coves, and bays that surround the central bay and flow into it. To the southeast, Popponesset Bay is separated from Nantucket Sound by a barrier spit which is a popular beach and an important geological feature of the area. The bay connects to the sound through a single outlet between the northern tip of the spit and the eastern shore of the bay itself.

Fig. 2-9 – Western Cape Cod, Popponesset Bay Highlighted

The embayments surrounding Popponesset Bay include high-volume Shoestring and Ockway bays, the Mashpee and Santuit Rivers which drain into the main bay and Shoestring bay respectively, and a few smaller features: Pinquickset cove is a small feature on the eastern side of the main bay which does not connect to any other body of water, and Popponesset Creek is a small creek which connects the western side of the main bay and Ockway Bay to the Sound. For more information on these embayments, refer to Fig. 2-10 below. The Popponesset Bay system is similar to Waquoit Bay in that the main bay exhibits none of the estuarine characteristics of the surrounding embayments, but acts as a large interface between the freshwater estuaries and the saltwater ocean (Howes et al.).

Fig. 2-10 – Major Components of the Popponesset Bay Estuarine System (Howes B., et al.)

In order to determine the nitrogen threshold for Popponesset Bay, the same procedure as Waquoit Bay was applied to a different watershed: First the watershed was modeled using Visual MODFLOW and the USGS regional model for Cape Cod, then the watershed was divided according to land use and nitrogen was "budgeted" for each type of land use (such as wastewater from residential, commercial and industrial, fertilizer runoff from lawns and golf courses, etc.). From the land use analysis, a total nitrogen load was developed and the surface water system was analyzed for its capacity to attenuate nitrogen during transport. An overall value for nitrogen attenuation between nutrient sources and Popponesset Bay was determined and biological indicators in the bay and embayments were studied to determine an accurate nitrogen threshold for healthy activity in the estuarine system (Howes et al.).

There are several differences between the 13,000-acre Popponesset Bay watershed and the Waquoit Bay system. The land use patterns are drastically different. As opposed to the large amount of land in the Waquoit Bay watershed devoted to the MMR, the majority of developed land in the Popponesset Bay watershed is devoted to residential, and a great deal of nitrogen enters the bay through the Shoestring Bay embayment because residential activity is concentrated in a watershed area which drains into that bay. Nutrients from industrial and commercial uses enter the watershed through northern embayments such as the Mashpee River, because these activities are concentrated in the northern part of the watershed. Unlike the Waquoit watershed, there are no wells in the Popponesset watershed that drain outside of the watershed, so there is no direct nitrogen removal from this system (Howes et al.).

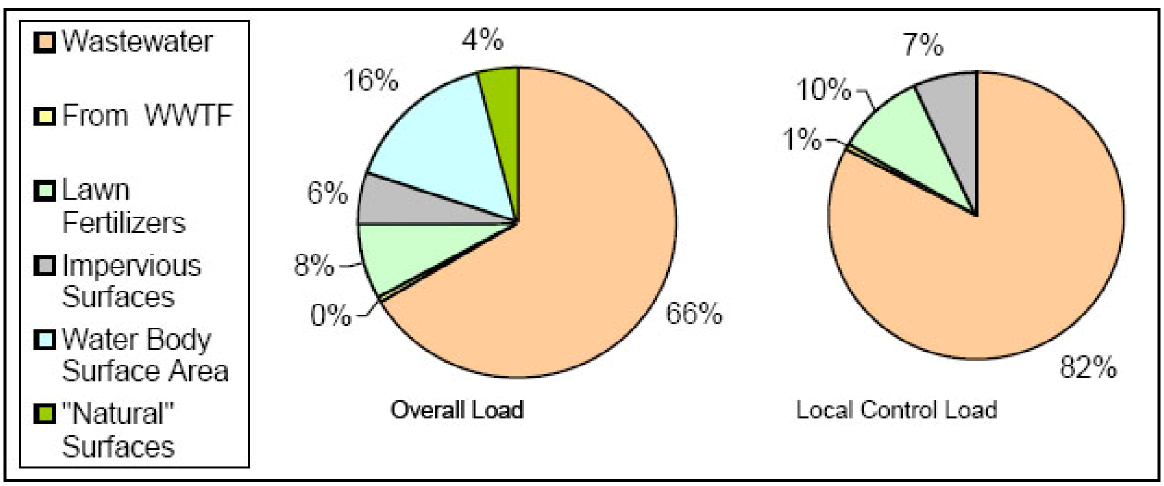

Fig. 2-11 – Nitrogen Loading by Land Use in Popponesset Bay (Howes B., et al.)

Additionally, there are far fewer surface water bodies in the Popponesset watershed. To the west, there are many small kettle ponds and connecting streams that support active nitrogen-remediating bacterial ecosystems. However, in the Popponesset watershed few such ponds exist. Thus, in this watershed most of the nitrogen attenuation takes place in the Mashpee or Santuit rivers, which exhibit attenuation of 71% and 51% respectively. These two rivers and the parts of the bay into which they discharge account for a great deal of the watershed, and so the overall rate for nitrogen attenuation between source and bay in the Popponesset Bay estuarine system is 44% (Howes et al.).

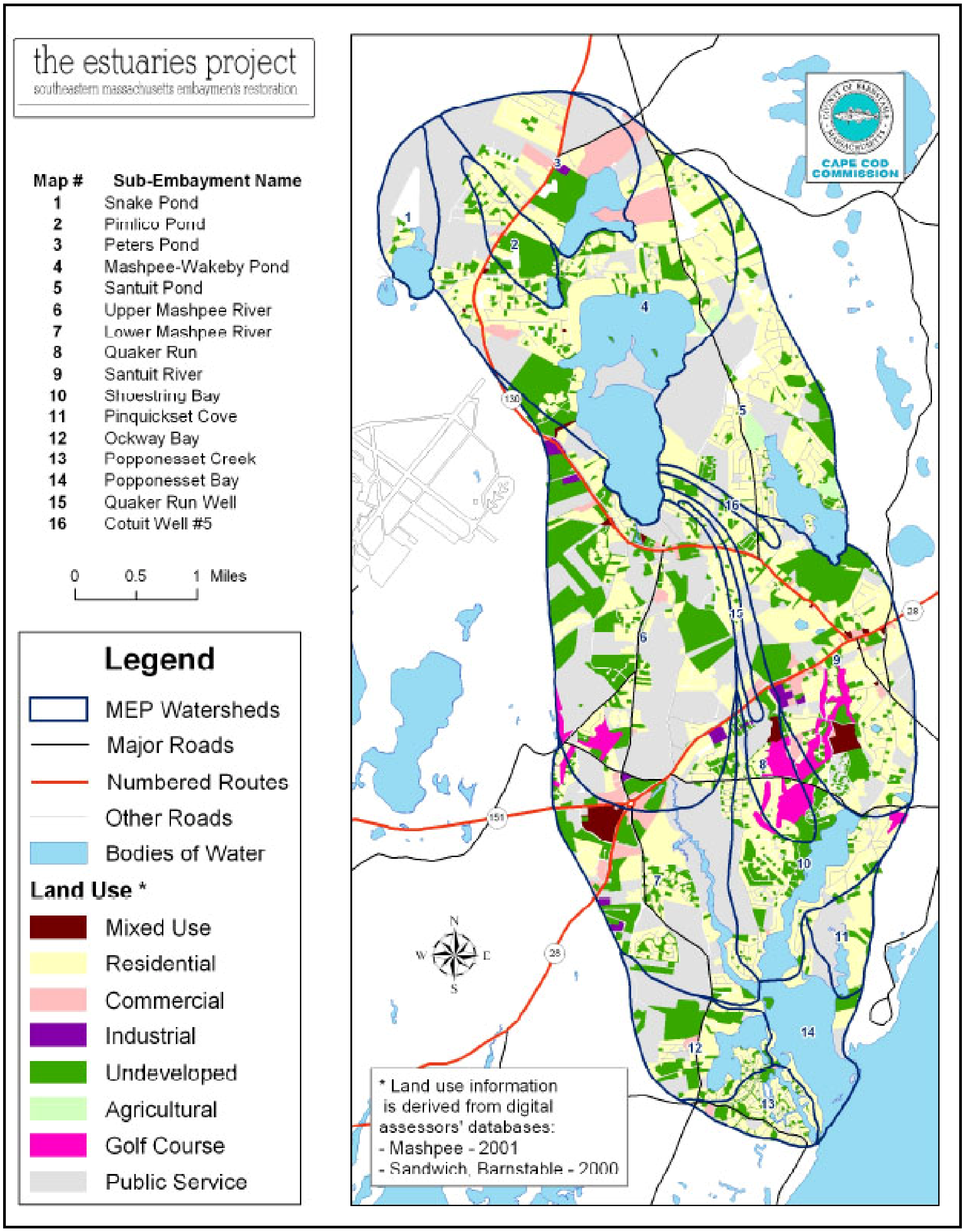

Fig. 2-12 ─ Land Use Coverage in the Popponesset Bay Watershed (Howes B., et al)

The Popponesset and Waquoit systems exhibit an understandable degree of similarity and, in fact, many of the nitrogen loading statistics are the same between the two bays. The ecosystems of both bays are also quite similar and an identical process was used to track the effects of nitrogen loading and determine a threshold in both bays. Thus, it is not surprising that this report concludes that the projected nitrogen capacity of Popponesset bay is also 0.38 mg N/L, nor that the recommended method for reducing nutrient loads to this level is also the same: Completely eliminate residential septic systems within the watershed, opting instead to treat all wastewater and central facilities (Howes et al.).

2.9 Overview of Waste Treatment

Wastewaters are treated with physical, chemical, and/or biological processes. Two basic wastewater treatment plant layouts are activated sludge (biological) plant for domestic wastewater treatment (shown in Fig. 2-13) and physical and chemical plant typically for agricultural and industrial wastewaters. The activated sludge plant is ideal for the residential area of Mashpee. Nutrient Removal technologies are discussed in the next section. The main components for a very general activated sludge plant are (in order of process):

1. Bar rack
2. Grit chamber
3. Primary clarification
4. Aeration basin
5. Secondary clarification
6. Disinfection

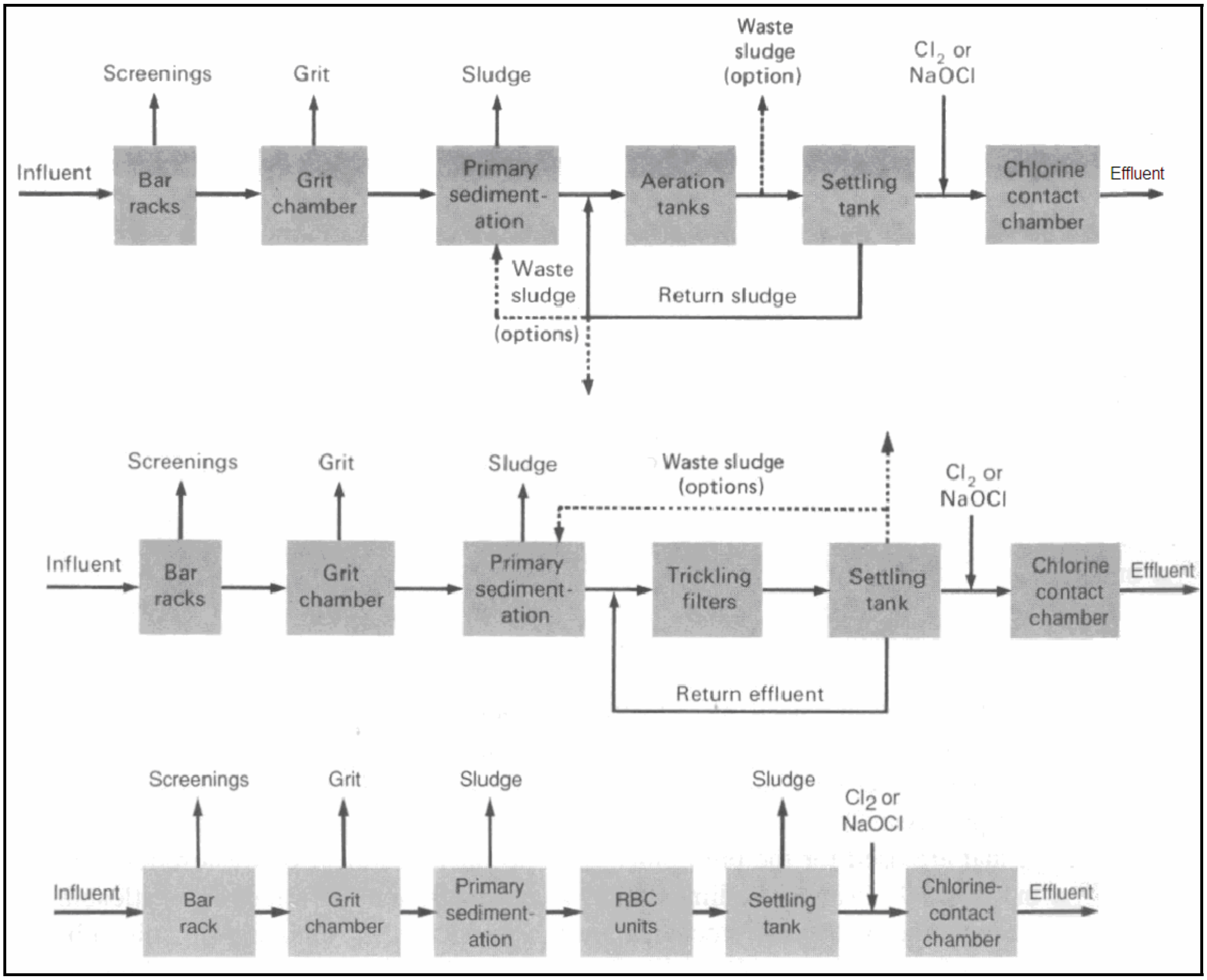

Fig. 2-13 – General Layouts for Biological Treatment of Wastewater (Metcalf & Eddy)

After the UV contact chamber the effluent is then discharged. In between several of these processes, waste products may be treated and re-introduced into the main processes, outlined above, in order to maximize the amount of clean effluent and minimize the amount of waste.

Bar racks are very coarse filters that filter out large debris that may potentially clog the pipes in the wastewater treatment facility. The openings of bar racks can be 5/8 inch or greater (Metcalf & Eddy). Examples of large debris that bar racks may filter out are large kitchen waste, feminine hygiene products, toilet paper, etc. These racks are usually located at the end of the sewage collection system before entering into the wastewater treatment plant. The filters are cleaned regularly throughout the day, most likely mechanically by machines that rake debris from the rack. Debris collected from the rack can be deposited at a landfill or incinerated. Though the water has been filtered of very large debris, the wastewater is still a heterogeneous mixture. In some instances, following the coarse bar rack may be finer screens that can filter out smaller solids. However, sometimes filtering out finer solids is undesirable because these solids are important components in biological treatment.

An alternative to using bar racks or screens is using comminutors. Comminutors grind up, cut up, shred, etc. larger solid waste without removing them from the wastewater flow (Metcalf & Eddy). Leaving solid constituents in the wastewater is an advantage with comminutors because they do not require regular cleaning maintenance the way bar racks do. Also, by leaving solid constituents in the water, the solids have a chance to be treated in the plant rather than wasted, decreasing the amount of waste the plant must produce. However, there are still many disadvantages to comminutors. Even though comminutors grind up solids in the waste into finer particles, it is likely that some particles, especially rags, will recombine later in the wastewater treatment process. Recombined solid particles have the potential to accumulate, clog, and damage parts of the wastewater treatment facility that are not suited for larger solids.

Following the bar racks are grit chambers. Grit chamber are essentially settling tanks (Droste) though grit can also be removed by centrifugal processes (Metcalf & Eddy). However, the difference between grit chambers and the settling tanks further along the wastewater treatment process is that the primary purpose of the grit chamber is to remove non-biodegradable settle-able solids such as silt, sand, etc. that have densities much higher than the organic solids present in wastewater. Because the densities of these particles are greater, their settling velocities are also greater (Metcalf & Eddy). These particles can be abrasive and have the potential to clog and damage pipes, tanks, and other hardware in the wastewater treatment facility. Because the settlement from this process can be mostly non-biodegradable, the waste is likely to be disposed of in landfills. If there is enough organic matter present in the grit, the grit can be incinerated along with the waste sludge of later processes (Metcalf& Eddy). Even though water conditions in settling tanks are usually slow and calm, some circulation is maintained in the grit chamber to keep organic particles in suspension while grit settles because biological treatment of the organic matter is desired later in the wastewater treatment process.

Primary clarification takes place in a settling tank with slow and calm water conditions. The calm water conditions enable large settle-able organic solids (such as fecal matter) to be removed simply by gravity. Large organic solids that are less dense than the wastewater (such as coagulated grease) can also be removed from the surface of the water during primary clarification (removal of coagulated grease can also occur before primary clarification in the grit chamber for example). Both the settled solids and the floating solids are continuously raked from the top and bottoms of the settling tank and the waste sludge is pumped away for either further treatment or disposal. The effluent leaving primary clarification appears mostly homogeneous, but further treatment is necessary especially for nitrogen removal.

The aeration basin can operate using different methods of aeration. Some methods include air or oxygen diffusers at the bottom of the tank, or paddles/mixers or sprinklers at the surface of the tank. Each of these methods increases the amount of dissolved oxygen (DO) in the wastewater. The DO in the wastewater aids in accomplishing several tasks in the aeration basin. Aerating the basin will aid in removing volatile compounds (such as odor causing compounds, CO_2, etc.) present in the wastewater. The DO in the wastewater will also oxidize certain metals (such as iron and manganese) forming precipitates that can be settled out in secondary clarification (Droste). Active microorganisms will also increase its population and metabolism with added DO which will thicken the activated sludge necessary in secondary clarification.

In the secondary clarifier, most of the rest of the remaining organic matter and other solids are removed. The microorganisms in the activated sludge will consume the organic matter as well as further increasing in population. Other solids, such as metals that may have formed precipitates in the aeration process, can also be settled out in secondary clarification. Remaining sludge from this process is pumped away and the mostly clean wastewater has to go through one more step before it is allowed to be discharged.

The final step in the wastewater treatment process is disinfection. Disinfection is almost always necessary to ensure that the quality of the effluent is acceptable for discharge. In wastewater treatment the three main methods for disinfection are chlorination, ozone diffusion, and UV radiation. Each of these methods has its costs, its advantages, and its disadvantages. Chlorine remains the most popular method for disinfection because it is the most inexpensive. However, chlorine disinfection can have its risks. Because wastewater at the end of the treatment process can still contain some amount of organic matter, adding chlorine can create chlorinated organic compounds which are disinfection byproducts. Chlorinated compounds can be carcinogens and harmful to the environment. Storing chlorine is also a risk in itself because of the potential for spills and other accidents.

Ozone diffusion is an effective method of disinfection because the ozone molecule O_3 oxidizes and kills the harmful microorganisms. There are still some disinfection byproducts, though these are much less compared to those formed in chlorine disinfection. One drawback, though, is that creating ozone is very expensive, loud, and creates a foul odor. The foul odor from ozone is not present in water because it is not the ozone molecule itself that has an odor, but it is the process of making the ozone that has an odor. Another advantage to ozone, in comparison to chlorination, is that it does not have to be stored in large tanks. Ozone is usually created on-site and enough is made for what needs to be used.

UV radiation is also more expensive than chlorination and requires more maintenance such as cleaning the glass tubes encompassing the UV lights. However, UV radiation does not produce negative disinfection byproducts so it is safer than chlorination. Although UV radiation does not kill all of the microorganisms in the wastewater, the remaining microorganisms become unable to reproduce and are therefore essentially harmless. The effluent after disinfection is safe to be discharged into either surface or groundwater.

After some of the processes, waste is collected. Some of the waste is discarded and some is recycled into the main water treatment process to undergo further treatment. Discarded waste includes waste from the bar racks and grit chamber. Recycled waste includes sludge collected from the primary and secondary clarifiers. The recycled sludge is usually thickened and separated from any remaining wastewater. The separated sludge is disposed of, and the remaining wastewater is reintroduced into the process either before or after primary clarification.

2.10 Nutrient Removal Systems

The process described above is for a very general wastewater treatment plant. Further processes can be performed to ensure maximum nitrogen removal. Table 2-1 shows different constituents and how well certain processes remove these constituents. For the removal of nitrogen in its different forms, the wastewater treatment processes that would most effectively remove nitrogen in its different forms are aeration, biological denitrification, and disinfection.

Table 2-1 – Contaminant Removal in Wastewater Treatment Processes (Droste)

Contaminant Removal in Wastewater Treatment Processes		
Process	**Ammonia Nitrogen**	**Inorganic Nitrogen**
Air Stripping	G-E	-
Conventional Aerobic Treatment	P	P
Biological Denitrification	G-E	G-E
Low-Loading Trickling Filter	G-E	P
High-Loading Trickling Filter	P	P
Anaerobic Treatment	P	P
Disinfection	F-G	-
Carbon Adsorption	F-G	P
P, poor (0-20% removal F, fair (20-60% removal)	G, good (60-90% removal) E, excellent (90-100% removal)	"-", not applicable

The processes of aeration (air stripping) and disinfection have been previously discussed in the overview of wastewater treatment. Biological denitrification is the process in which nitrogen is converted into its different forms until it finally converts to nitrogen gas. Denitrification follows these steps:

$$NO_3^- \longrightarrow NO_2^- \longrightarrow NO \longrightarrow N_2O \longrightarrow N_2$$

The chemical process above is not balanced.

Non-nitrogen products that result from these processes include hydrogen, oxygen, and water.

The driving forces of these chemical conversions are bacteria that use nitrogen instead of oxygen during respiration for the metabolism of substrate. The bacteria, which can include *Achromobacter, Aerobacter, Alcaligenes, Bacillus, Brevibacterium, Flavobacterium, Lactobacillus, Micrococcus, Proteus, Pseudomonas,* and *Spirillum* respire using nitrate and nitrite as electron acceptors because oxygen is not present in this process (Metcalf & Eddy). The presence of dissolved oxygen in the wastewater will cause the denitrification process to fail because the bacteria will more readily use oxygen to respire than nitrate or nitrite. Nitrogen gas is the desired product in wastewater treatment because it cannot be biologically used by plants such as algae, and because it can be easily removed because it is a volatile atmospheric gas.

Biological denitrification can occur in suspended growth or attached growth processes. Denitrification can also occur to a lesser extent in other processes, the primary goals of which are not denitrification. Suspended growth processes usually occur in plug-flow activated-sludge systems (Metcalf & Eddy).

Attached growth processes, also called fixed film processes, can occur in different types of reactors including trickling filters and rotating biological contactors. In attached growth systems, the bacteria are not in suspension as is the case with activated sludge. The bacteria attach to surfaces as a layer of film which comes into contact with the wastewater as the wastewater flows by the attached-growth media. The medium for trickling filters can range from gravel, to sand, to plastic. Rotating biological filters are usually made of large perforated paddles (Droste).

2.11 Overview of Rapid Infiltration Systems

In some cases, it is preferred that wastewater effluent does not discharge directly into surface water. The situation may be that there are constituents in the wastewater effluent that are not desirable. These constituents can range from nutrients, to suspended solids, to biological or chemical oxygen demand. Rapid infiltration systems are ideal for situations where wastewater effluent must not discharge directly to surface water.

Though the objectives of infiltration systems can entail treatment of the wastewater, the applied wastewater in the case for Mashpee will be completely treated with special attention to the treatment and removal of nitrogen. The wastewater effluent is applied over an infiltration basin where it percolates through the soil – there is a negligible amount of evaporation. Fig. 2-14 shows the basic schematic of an infiltration basin.

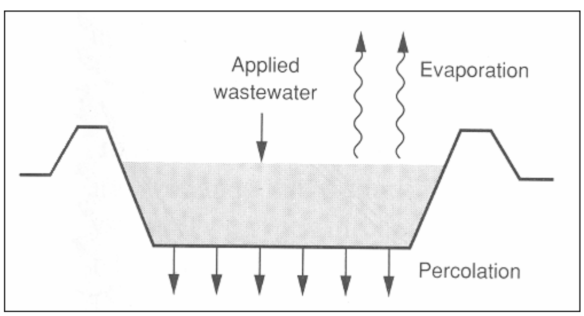

Fig. 2-14 – Rapid Infiltration Hydraulic Pathway (Metcalf & Eddy)

There are several different design objectives for infiltration systems:

1. Treatment during infiltration followed by groundwater recharge (and in some cases prevention of saltwater intrusion)
2. Treatment during infiltration followed by recovery of wastewater through drains or wells (recovered water for use in other processes)

3. Treatment during infiltration followed by groundwater flow then discharge to surface water through aquifer/surface water boundary

Depending on the design objective, there may be different levels of pre-treatment. However, as mentioned above, the wastewater effluent for Mashpee's wastewater will have been fully treated, so the design objectives entailing treatment are unnecessary. Instead, the design objective in this project is to maximize the volume of water infiltrated at each potential site.

Percolation from the infiltration systems will flow through the groundwater and eventually into surface waters, as described in the third objective above. Because the Cape Cod region has a nutrient excess problem in its surface waters, the quality of the effluent must not be overlooked. For maintaining ground- and surface water quality, an average acceptable concentration of nitrogen in effluent for RIBs is 10 mg/L (Metcalf & Eddy, Inc. 1334). However, the design nitrogen concentration of the effluent for this problem will be set at 3 mg/L which is well within acceptable parameters.

Designing a Rapid Infiltration System involves several steps. The basic steps to designing a Rapid Infiltration System (Metcalf & Eddy) follow:

1. Site evaluation & selection
2. Determination of pre-treatment level
3. Selection of distribution method
4. Determination of design hydraulic loading rate
5. Determination of design application rate and operating cycle
6. Determination of land requirements
7. Layout of infiltration area
8. Layout and sizing of effluent recovery system
9. Determination of storage requirements & climatic considerations
10. Determination of monitoring requirements

These steps outlining the process of designing a Rapid Infiltration System should be followed in order. The first step can be the most important because it is a factor in the steps following it.

2.11.1 Site Evaluation & Selection

One of the main factors in selecting an ideal site is evaluating the soil conditions. Soil characteristics to be evaluated are permeability, composition, and uniformity. Other characteristics of the land include depth of the soil from surface to either groundwater or bedrock, and slope of the land.

2.11.2 Determination of Pre-Treatment Level

In general cases, wastewater applied to the infiltration basins should always at least undergo primary treatment. The results of neglecting to settle and remove larger solids from the wastewater are soil clogging and exceeding acceptable nitrogen loads. Secondary treatment can also be carried out to further remove solids and nitrogen. Because the design objective of the infiltration system in Mashpee will be to maximize infiltration, not to treat the water, the pre-treatment level will be fully treatment. Though the wastewater at Mashpee will be fully treated, some aspects of tertiary treatment can and/or should be avoided: to keep expenses down and reduce risks: use of chlorine should be avoided to prevent the presence of chlorinated hydrocarbons which are carcinogenic and can pollute aquifers and; treatment for metals is not necessary because metals are usually removed within the first few feet of soil.

2.11.3 Selection of Distribution Method

There are three distribution methods that can be used in an infiltration system. The methods are sprinkler systems, subsurface leaching trenches, or open bed infiltration basins. For the purposes of this project, only the latter two infiltration methods are considered[1].

Another infiltration method is the utilization of subsurface leaching trenches. It is ideal to use subsurface leaching trenches when the land on the surface of the site is needed for use. Some examples of land usage with leaching trenches are beneath parking lots, parks, athletic fields, lawns, gardens, etc. However, leaching trenches should never be constructed beneath buildings or other heavy structures with foundations. The mechanics of soil when saturated with water cause it to be unstable under high pressures, and the structures make repairs difficult. The subsurface leaching trench is constructed of perforated pipes buried beneath a depth of soil. The pipes discharge the treated wastewater effluent from the wastewater treatment facility. Typically, the pipes would be surrounded by coarse gravel or sand to prevent erosion or clogging of finer soil particles. Although subsurface leaching trenches operate with a higher infiltration rate then

[1] Sprinkler systems are the preferred method if the slope of the land is greater than 5%. Sprinkler systems must be used when the grade is greater than 5% because it is simply not flat enough for open bed infiltration basins – the water in a basin set upon a steep slope will quickly run off before having the time and conditions for infiltrating into the ground. A steeper slope can promote soil erosion when great volumes of water are applied to it so lessening the load of water by method of sprinkling is ideal. Vegetation is also necessary to reduce the effects of erosion in sprinkling infiltration systems. To accommodate large quantities of water, sprinkling systems may not be ideal because of the slow infiltration rate (because of the low discharge rate from the sprinklers themselves).

sprinklers, the capable infiltration rate for subsurface leaching trenches is typically half that of open bed infiltration basins.

The other infiltration method is the utilization of open bed infiltration basins. Open bed infiltration basins are ideally used on flatter topographies where wastewater effluent can percolate vertically downward into the soil. The size and quantity of basins to be used depends on loading rates and soil permeability. The force of gravity by sloping pipes is usually the force that carries the wastewater effluent from the treatment facility to the spreading basins. If gravity is not used, then water is pumped from the facility to the spreading basins at a low pressure. At the ends of the pipes, splash pads are used to prevent soil agitation at the contact area. Vegetation in a spreading basin can be used to intercept suspended solids present in the effluent, and can be used to prevent soil erosion and agitation. However, the presence of vegetation is not necessary for the case in Mashpee because the wastewater will have undergone full treatment.

2.11.4 Determination of Design Hydraulic Loading Rate

The design hydraulic loading rate is expressed in inches per unit time. Usually the hydraulic loading rate is expressed in the annual average hydraulic loading rate. The annual hydraulic loading rate is based on soil permeability or effective vertical hydraulic conductivity. The annual hydraulic loading rate (Metcalf & Eddy) can be determined by:

$$L_w = \left(IR\frac{inches}{hour}\right) \times \left(\frac{1foot}{12inches}\right) \times \left(\frac{24hours}{1day}\right) \times \left(OD\frac{day}{year}\right) \times (F)$$

Where L_w= annual hydraulic loading rate
IR = infiltration rate
OD = operating days
And F = application factor

The infiltration rate IR is determined by field tests, and the application factor F depends on the type of field test used to measure the infiltration rate. Typical value ranges for F are given in Table 2-2.

Table 2-2 – Recommended Application Factor to be Used in Calculating the Average Annual Hydraulic-Loading Rates for Rapid Infiltration Systems (Metcalf & Eddy)

Recommended Application Factors	
Field Measurement	**Application Factor, F**
Basin infiltration test	10-15% of the minimum measured infiltration rate
Cylinder Infiltrometer and air entry permeameter measurements	2-4% of the minimum measured infiltration rate
Vertical hydraulic conductivity measurements	4-10% of the conductivity of the most restrictive soil layer

The operating days per year, OD, is simply the number of operating days per year, including both application and drying periods.

2.11.5 Determination of Design Application Rate and Operating Cycle

The operating cycle is the combination of the drying and application periods for an infiltration basin. Drying periods are necessary between application periods so that the soil can be properly aerated, and so that natural decomposition and other biological conversions can occur. The application rate (Metcalf & Eddy), given a certain operating cycle, is:

$$R_a = \left(\frac{L_w \frac{feet}{year}}{365 \frac{days}{year}} \right) \times \left(\frac{operating cycle time(days)}{application period(days)} \right)$$

Where R_a = design average application rate
L_w = annual hydraulic loading rate
Operating cycle time = application period + drying period
And *application period* = number of consecutive days of application

Table 2-3 shows typical application and drying periods for infiltration systems:

Table 2-3 – Typical Loading Cycles for Rapid Infiltration Systems (Metcalf & Eddy)

Typical Loading Cycles for Rapid Infiltration Systems				
Loading Cycle Objective	Applied Wastewater	Season	Application Period (Days)	Drying Period (Days)
Maximum Nitrogen Removal	Primary	Summer	1-2	10-14
		Winter	1-2	12-16
	Secondary	Summer	7-9	10-15
		Winter	9-12	12-16
Maximum Nitrification	Primary	Summer	1-2	5-7
		Winter	1-2	7-12
	Secondary	Summer	1-3	4-5
		Winter	1-3	5-10

The values for application and drying periods vary depending on the objective of the infiltration basin, the level of wastewater treatment, and the season. These values can be used in the equation determining the average rate of application. Another factor that should be taken into consideration in these calculations is that water depth cannot exceed 12-18 inches.
Because the wastewater in Mashpee will have undergone full treatment, drying days for treatment purposes is not necessary. However, drying days may be accounted for the purposes of allowing the groundwater mound beneath the infiltration site to temporarily decrease. If it is determined that the groundwater mound beneath a site may reach critical heights, then drying periods may alleviate this problem by taking selected basins off-line.

2.11.6 Determination of Land Requirements

The total surface area of land needed for a treatment facility is usually expressed in acres and can be determined when the annual hydraulic loading rate and daily volumetric flow rate are known. The total surface area can be found by (Metcalf & Eddy):

$$A_i = \frac{\left(Q\frac{gallons}{day}\right)\times\left(365\frac{days}{year}\right)}{\left(L_w\frac{feet}{year}\right)\times\left(7.48\frac{gallons}{cu.feet}\right)\times\left(43{,}560\frac{sq.feet}{acre}\right)}$$

Where A_i = total surface area
Q = daily volumetric flow rate
And L_w = annual hydraulic loading rate

It is important to keep in mind that the land area calculation is for determining just the area required for the infiltration basin – it does not include the land area required for buffer zones, the treatment facility, storage area, etc.

The seven sites identified for the project will solely be used for infiltration. The treatment facility will be located on a separate area of land so the area that the wastewater treatment facility uses does not have to be subtracted from the land area calculations.

2.11.7 Layout of Infiltration Area

The layout of the infiltration area is determined by the geometry of the site and the operating cycle. The geometry of the site determines the shape of the infiltration basin, and the operating cycle determines how many basins should be used. For a continuous application of wastewater, multiple basins should be used so that some basins can lay idle for drying while others undergo infiltration without any pause or disruption of the normal operating cycle. Table 2-4 shows the minimum number of basins required for the continuous application of wastewater.

Table 2-4 – Minimum Number of Infiltration Basins Required for a Rapid Infiltration System for the Continuous Application of Wastewater (Metcalf & Eddy)

Loading Application Period (Days)	Cycle Drying Period (Days)	Minimum Number of Basins
1	4-5	5-6
	5-7	6-8
	5-10	6-11
	7-12	8-13
	10-14	11-15
	12-16	13-17
2	4-5	3-4
	5-7	4-5
	5-10	4-6
	7-12	5-7
	10-14	6-8
	12-16	7-9
3	4-5	3
	5-10	3-5
7	10-15	3-4
	12-16	3-4
8	10-15	3
	12-16	3
9	10-15	3
	12-15	3

The minimum number of basins needed varies depending on the application and drying periods. Other considerations in laying out infiltration basins are the basins' effect on groundwater.

When setting up an infiltration basin, the height of the water table below the soil surface will increase once water percolates through the soil. In order for the soil and groundwater quality to be maintained, the soil depth between the soil surface and the groundwater table must not fall below 2 feet (Metcalf & Eddy). However, according to Massachusetts DEP, the depth between the bottom of the infiltration bed and the groundwater table must not fall below 4 feet. So, assuming a uniform infiltration basin depth of 2 feet, the minimum allowable distance from the infiltration basin surface to the top of the groundwater table must be no less than 6 feet. Long and narrow shaped basins, compared to circular or square shaped basins, can be used to avoid a shallow soil depth resulting from groundwater mounding.

If, however, the case is that groundwater under the infiltration basin is to flow into surface water, then the maximum allowable water table height can be found by the following equation (Metcalf & Eddy):

$$WI = \frac{KDH}{L}$$

Where W = width of infiltration area
I = hydraulic loading rate
K = hydraulic conductivity
D = average thickness of zone below water table perpendicular to flow direction
H = elevation difference between water level in stream or lake and maximum allowable water table level below infiltration area
And L = distance of lateral flow

The variable parameters used in the equation above are displayed in Fig. 2-15.

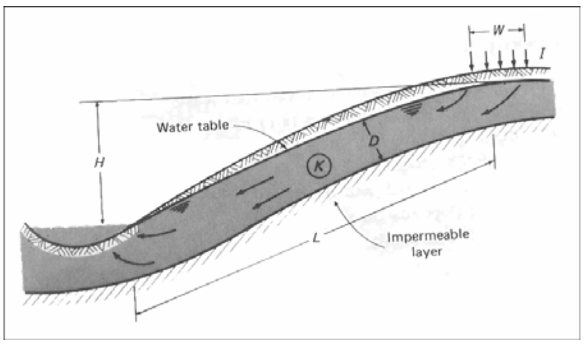

Fig. 2-15 – Natural Drainage from Rapid Infiltration Basin into Surface Water (Metcalf & Eddy)

Another way to avoid shallow soil depth is by controlling groundwater table height with underdrains or wells that recover percolated wastewater.

2.11.8 Layout and Sizing of Effluent Recovery System

Underdrains in infiltration systems can be used to control the development of a groundwater mound. Underdrains are perforated pipes installed beneath the water table. The pipes can be made of plastic, concrete or clay. To prevent excessive debris from entering the perforation of the underdrain pipes, sock filters over the pipe may be used as well as a layer of coarse sand or gravel around the pipe. The following equation (Metcalf & Eddy) can be used to determine the proper distance that an underdrain must be placed away from the infiltration basin:

$$H_c^2 - H_d^2 + \frac{IW(W+2L)}{K}$$

Where H_c = height of water table below outer edge of infiltration area
H_d = drain height above impermeable layer
I = infiltration rate

W = width of basin
L = distance from centerline of mound to underdrain
And K = hydraulic conductivity

The variable parameters used in the equation above are displayed in Fig. 2-16.

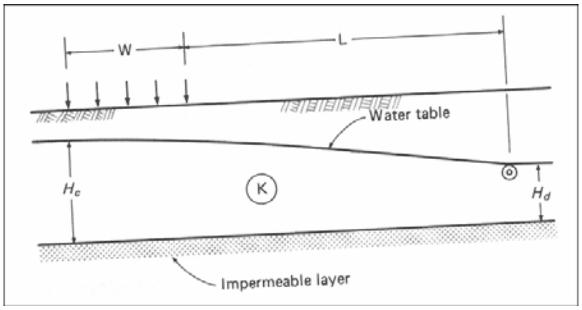

Fig. 2-16 – Collection of Renovated Water by Underdrains (Metcalf & Eddy)

Recovery wells can also be used to control the development of a groundwater mound beneath and infiltration basin. The use of recovery wells is typically only applied to infiltration systems because the wells become ineffective for slower systems. The cone of depression, which can be seen in Fig. 2-17, is the cone-shaped drop in groundwater height in the area surrounding the well. The size of the cone can be measured by test wells on site to determine the proper placement of the wells.

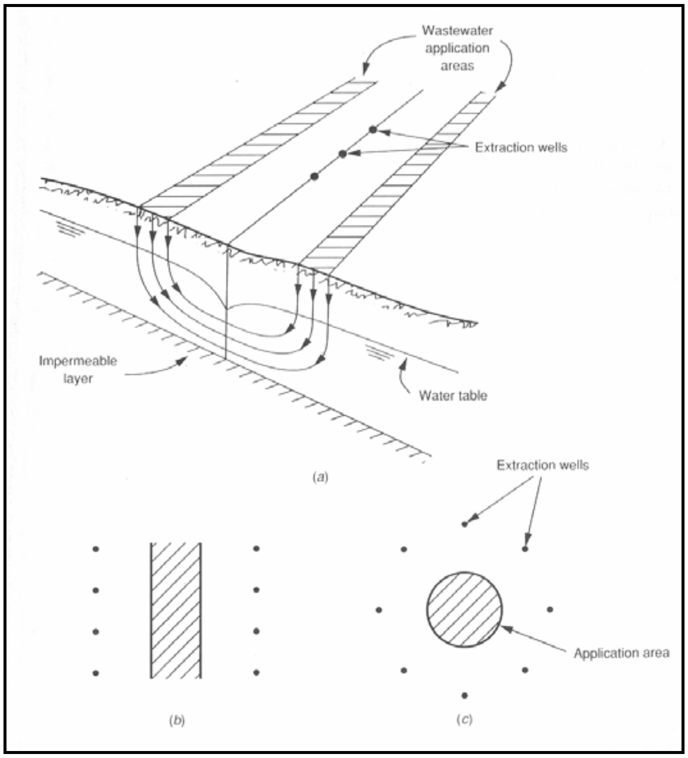

Fig. 2-17 – Recovery Well Configurations (a) cone of depression at wells placed midway between two application areas (b) wells surrounding long rectangular application area (c) wells surrounding circular application area (Metcalf & Eddy)

However, in this project it is desired that the infiltrated water be fully recharged back into Cape Cod, so underdrains or recovery wells are not necessary.

2.11.9 Determination of Storage Requirements & Climatic Considerations

During the winter the colder climatic conditions maybe bring up a few issues. There may need to be on-site storage of the wastewater before infiltration because soil permeability is lower during colder months. The volume of the storage tank can be determined when volumetric flow rate of wastewater in the winter and soil permeability are known.

Other than storage, there are no other special considerations to be taken into account for an infiltration system in the winter. Because infiltration does not depends on vegetation (excluding the case of sprinkler systems), the process still works during cold climates when there is fewer or no vegetation. Also, a flooded infiltration basin is actually less susceptible to freezing than other methods because surface ice helps insulate the water and soil beneath it. It has been found that infiltration systems can still operate at temperatures as low at -37°C (Metcalf & Eddy).

2.12 Visual MODFLOW

The modeling software chosen for use in this project is Visual MODFLOW, which is a graphical interface that allows the user to visually create and analyze groundwater models. The software is divided into three modules: Input, Run, and Output. The Input module allows the user to create and modify models, incorporating all of the data needed to understand groundwater flows. The Run module then translates the data for use by one of several available solvers, which iteratively calculate the water table levels that result when all factors are considered. The solver used for this report is MODFLOW-2000, which was created by the US Geological Survey. The Output module then allows the user to graphically view and interpret the results in 2 or 3 dimensions.

The concept behind MODFLOW is relatively straightforward. The region is divided into rows, columns, and layers, defining a three dimensional grid made up of individual cells. The widths and depths of these cells can be standardized or customized at will. Cells are each assigned relevant properties, such as the conductivity in each direction. Next the boundaries are defined, with certain zones such as the fresh-saltwater interface specified as no-flow zones, other zones such as a large lake or the ocean are assigned a constant head, and others such as the ground surface are assigned a constant input or output flow. Wells can also be added to the model. Finally, the model requires an initial head value to be assigned to each cell, as a starting point for the solver's first iteration. After the first iteration, the solver uses modified head values, and continually calculates new head values for every individual cell until the entire water table is stable to within the desired accuracy.

Visual MODFLOW has the capacity to model for several different situations. The simplest, which is used in this report, is to set up the region and then solve for the steady-state water table level. Additionally, the model can be set up for different time periods, with certain parameters changing from one period to the next. Visual MODFLOW also has the capacity to consider the flow of contaminants within the groundwater, calculating the resulting steady state and variable concentrations. This is one possible method of determining which surface water bodies would be affected by the additional nitrogen loading due to a proposed effluent bed.

Visual MODFLOW also supports the use of MODPATH, which is perhaps a simpler, faster, and more visual method to determine where effluent will travel. Through this package, the user can add particles into the model at desired locations in the Input module. When MODPATH is run, it accesses the MODFLOW 2000 output files to quickly run individual particles as if they were water particles, from their starting location through individual cells until they eventually leave the model. Alternatively, MODPATH could complete the same analysis backwards, tracing water from any point to its starting location. MODPATH is particularly useful when steady state is the only concern, although MODPATH also tracks the time of travel as the particle moves. This package is often used to determine the recharge boundaries for a particular water source. In this report, MODPATH is the method chosen to determine which specific embayments will be affected, and to what extent, by an effluent bed located on each proposed site.

2.13 USGS Regional Model for Sagamore Lens

In 2004, Donald Walter and Ann Whealan wrote a report describing the model which was used as the base model for this project. The report is entitled "Simulated Water Sources and Effects of Pumping on Surface and Ground Water, Sagamore and Monomoy Flow Lenses, Cape Cod, Massachusetts". The purpose of the USGS project was to provide a comprehensive analysis of the Cape Cod water supply. Cape Cod is naturally divided into several distinct underground watersheds, called flow lenses. These lenses provide convenient boundaries for modeling, because each one is self-containing, nullifying the need to carefully define boundary conditions. The report by Walter and Whealan described the two largest lenses, which were each modeled individually. The purpose behind the study was to describe how various changes to the system would affect the entire system, such as changing well pumping rates and natural changes to recharge values. In the process, they created a regional model for each of the two flow lenses under consideration.

This project makes use of the USGS Regional Model, which was created by Donald Walter and Ann Whealan for the Sagamore Lens, as the base model from which all analyses were made.

3 Methodology

In order to determine how to discharge treated wastewater effluent in Mashpee, it was necessary to predict how the distribution of effluent through the watershed would effect nitrogen concentrations along the coast. The only accurate, objective tool for making such predictions in this case is a computer model. For the purposes of this report, Visual MODFLOW 4.2 (Waterloo Hydrogeologic Software, 1995-2006) was selected as an appropriate modeling tool. Thus, the basic methodology used in this report is to obtain and verify a software model of the area under consideration, model the effects of effluent discharge as accurately as possible, and then analyze and interpret the results. This section describes in detail the methods and procedures used in data collection and computer modeling.

3.1 Preparing the Regional Model

This project makes use of the US Geological Survey regional model for the Sagamore Flow Lens, which was created in 2003 by Donald Walter and Ann Whealan. Several steps were required to prepare the model for use in this project: first, the data and software was obtained, second, all of the data was imported into Visual MODFLOW, and finally, the imported model was verified to be consistent with USGS results, known data, and common sense.

To obtain the model data files, a request was sent to Donald Walter. Concurrently, a computer laboratory was equipped with Visual MODFLOW. The regional model consists of a uniform grid of square nodes 400 feet on a side. The grid contains 246 rows, 365 columns, and 20 layers. Separate files specify such data as node elevations for each layer, recharge properties, boundary conditions, well pumping rates, and soil flow properties. The data provided contains two separate models, one for 2003 and the other for 2020. The primary differences between these two models are the well and recharge data, which are projected to change with time due to increased development. For the purposes of this project, the 2003 model was selected.

3.1.1 Importing data into Visual MODFLOW

Once the data and software were available, the data was imported into Visual MODFLOW and checked for completeness. Walter's modeling makes use of a different software package from Visual MODFLOW, so the files had to be imported into a .VMF file, which is the native format. Visual MODFLOW is able to read and automatically import .NAM files, which point toward the other various data files. In one step, nearly everything was successfully imported. However, upon examination, it became apparent that the layer elevations and initial heads were not successfully imported. With the exception of the bottom layer, each layer had entirely uniform levels, extending up to 100 feet above sea level. This unrealistic elevation included the ocean surface, which could be easily distinguished by the boundary of active and inactive cells. A cursory look at the data showed that the layer elevations should vary considerably within each layer. Additionally the data locates the ocean surface at zero elevation, exactly where it logically belongs.

Importing the layer elevations required correct interpretation of the data, modifying the data to make it compatible with Visual MODFLOW, and finally importing it one layer at a time. A separate text file existed for each layer, with a list of 89,790 elevations, which is one elevation for each grid node. Visual MODFLOW requires that imported elevations have coordinates assigned to each value, but the elevations in the data files were not associated with any coordinates. Apparently, the software utilized by the USGS records elevations by systematically specifying the elevation of each grid node. Through trial and error, it was determined that the list begins at the northwest corner of the model, and fills in each row from right to left, moving from the top row down to the bottom row. Microsoft Excel was used to assign each elevation an x and y coordinate, which was then saved as a text file for each layer. These layers were then imported one at a time into the model, through the "import elevations" function. The top of layer 1 is the ground surface. The elevations for this layer seemed logical; the coast marked the boundary within which elevations varied and outside of which the elevations were uniformly at zero. Below the ground within the active regions, layers 1-6 were distributed somewhat evenly, and starting from layer 7, each layer was 10 feet thick, with slight modifications around a few of the estuaries. All of this seemed logical and consistent with the data.

However, there were a few inconsistencies encountered. The bottom of layer 1 assigned a uniform elevation of 60 feet to the inactive region, which describes the ocean. Likewise, the bottom of layer 2 was at 50 ft, layer 3 at 40 ft, layer 4 at 30 ft, layer 5 at 20 ft, and layer 6 at 10 ft. In addition, the bottom of layer 20, which was supposedly bedrock, at times was assigned a value of zero, which pushed it up to sea level. It appeared that these were somehow the default elevations, which were not modified because the regions were marked as inactive and irrelevant.

However, Visual MODFLOW interpolates between data points in an attempt to create continuous surfaces. The result was clearly seen at the coast lines, where the bottom of layer 1 for example would instantly change from 0 to 60 feet. Visual MODFLOW accommodated by spreading this change out over several grid nodes, resulting in sharp peaks and inconsistent data at every coast line and estuary.

Accordingly, the original data was modified to block out all of the default values, and re-imported into the model. Visual MODFLOW requires that every cell have a finite thickness, so a minimum thickness of 0.1 feet was chosen. Without the default values driving the ocean level high, the layer heights became nearly flat crossing the estuaries.

Similar to the layer elevations, the initial heads values were assigned coordinates starting from the top left corner, continuing down row by row. These elevations were then imported into Visual MODFLOW at runtime. At this point, the model was able to successfully run.

3.1.2 Model Verification

After running the model, it was necessary to verify that the model is accurate and useful. Accordingly, it was put through several tests: the model was visually inspected for consistency with the report that was written upon its completion; zone budgets were used to show that the incoming and outgoing steady state water volumes balanced; the model water table contours were checked with known values; and model particle tracking was compared with existing documented contaminant plumes. Upon passing each test, it was concluded that the model was successfully imported, is consistent with reality, and is useful for modeling groundwater flows within the Sagamore Lens.

The first test was to visually inspect the model, checking that it was consistent with the report written by Donald Walter and Ann Whealan. This model was originally developed to model the effects of pumping on surface water bodies and groundwater flows in the Sagamore Lens, which lies under the western part of Barnstable County. A similar model was developed for the Monomoy Lens, directly to the east, but is of no concern for this report. Walter and Whealan developed two models: one to represent pumping conditions current as of the year 2003, and another model to test predicted pumping situations in the year 2020. For the purposes of this report, the 2003 model of the Sagamore Lens was the only model used. (Walter and Whealan)

The model used for this report is divided up into a grid of cells. The grid is 365 cells wide and 246 cells top to bottom, and each cell is a square 400 feet on each side. This yields total model dimensions of 146,000 ft (27.65 mi) by 98,400 ft (18.63 mi) and a total model area of 515.32 square miles. This data is entirely consistent with the report, which also lists that 47% of the cells in the Sagamore lens are active and lists a total active area of 246 square miles (Walter and Whealan). Taking 47% of 515.12 square miles yields a calculated active model area of 242.2, a discrepancy of 3.8 square miles or 1.5% error. It is important to note that the x- and y-coordinate directions in the model should directly correspond to geographical cardinal directions. Based on this data, it was concluded that this model is accurate in terms of size, scale, and position.

The report also describes that this model has been divided into 20 layers. Here, however, some deviation was required from description listed in the report. The report lists that the top 17 layers have uniform thicknesses of 10 ft and a uniform bottom altitude starting at 60 ft above NGVD 29 and extending 10 ft below that for each additional layer (Walter and Whealan). While using Visual MODFLOW, however, it was determined that this situation would require that some cells in layers 1-7 be "empty" near the coast because they only represent areas of higher elevations. Visual MODFLOW represents this by assigning the cell a "0" thickness value and it will not run when any cells have "0" thickness. To account for this discrepancy, any cells which would otherwise be empty were assigned a thickness value of 0.01 feet. Consequently, layers 1-7 each extend all the way to the coast, but are very thin. The bottom of level 7 represents sea level, and below this level all of the layers behave exactly as detailed in the report.

Furthermore, the report includes several graphical representations of ideal, regular flow conditions in the model area. In order to reference the model against the reported conditions, MODFLOW was run under default conditions to determine water table elevations and flow velocities, both from a birds-eye view and from a representative cross section described in the model, coincident with the easternmost edge of Jehu Pond near Waquoit Bay. Fig. 3-1 depicts a top-down view of the Sagamore and Monomoy Lenses, including water table elevations and flow velocities detailing a groundwater "mound" toward the center of the western part of the cape, from which groundwater flows outward toward Nantucket and Vineyard Sounds and Buzzard and Cape Cod Bays. The Line B-B' in this figure is the approximate location of the cross-section shown in Fig. 3-3 and Fig. 3-4. Fig. 3-2 depicts the output of Visual MODFLOW when the

Sagamore Lens 2003 model is run. Obvious similarities can be seen between these two images- the groundwater flow directions are identical (magnitude of groundwater flow is not shown in these images).

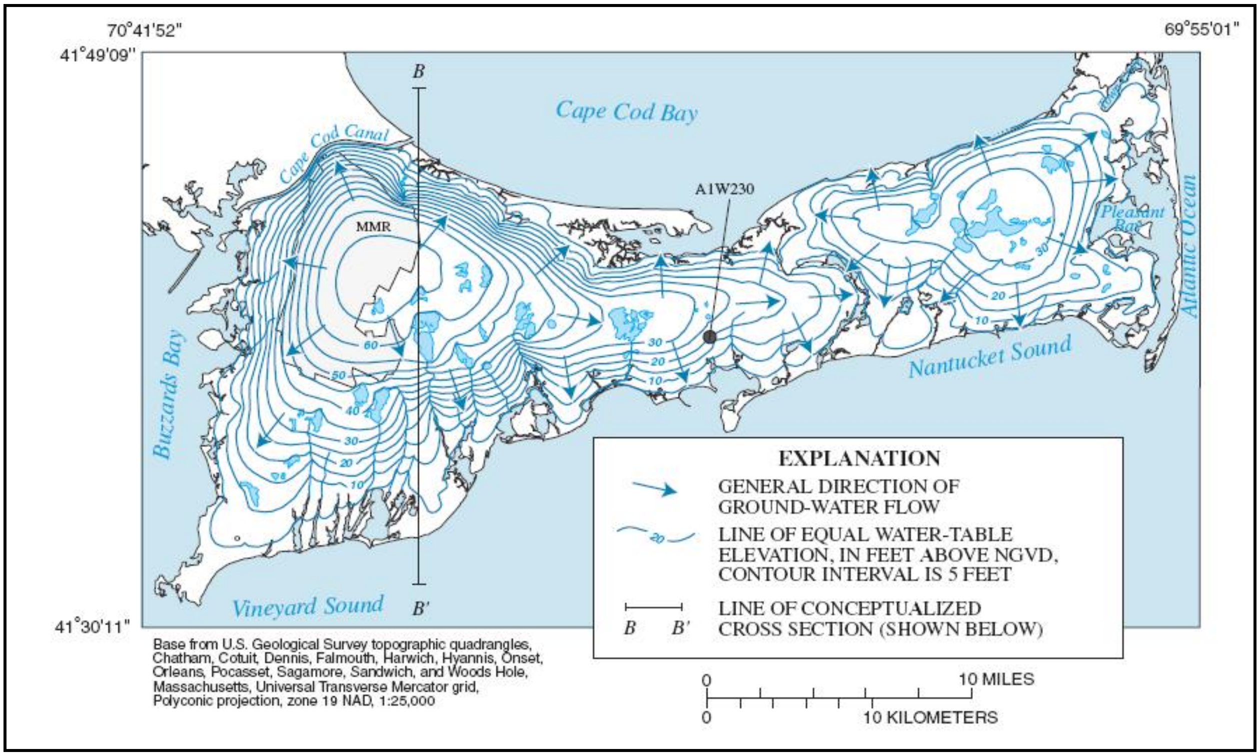

Fig. 3-1 ─ Water Table Elevations and Groundwater Flow Directions, Sagamore Lens (Walter and Whealan)

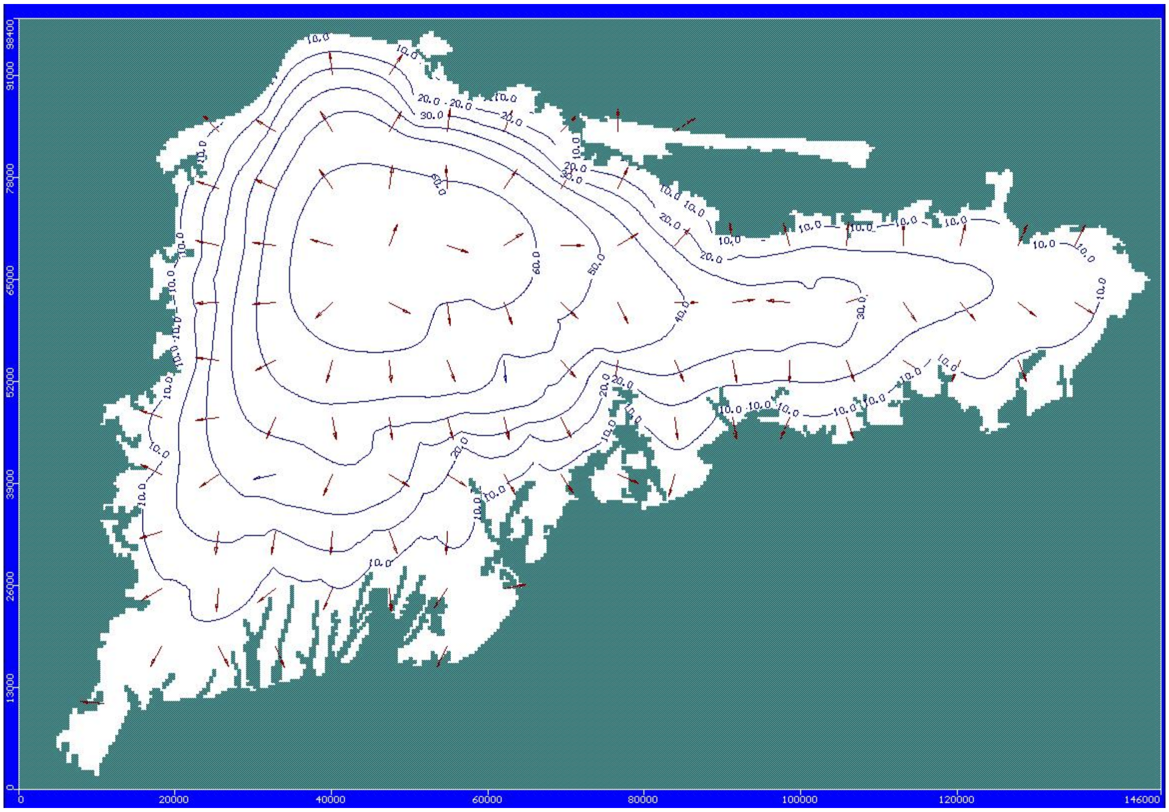

Fig. 3-2 – Water Table Elevations and Groundwater Flow Directions, Sagamore Lens as modeled by Visual MODFLOW

Fig. 3-3 is another chart from the Walter and Whealan report which depicts a cross section from Fig. 3-1. This cross-section is “viewed” from the west coast of the cape, with Cape Cod Bay (north of the cross-section) on the left and Nantucket Sound (south of the cross-section) on the right. A similar cross-section would normally be displayed in Visual MODFLOW in the opposite orientation- viewed from the east, with the north on the right and the south on the left. For the purposes of comparison, the display from Visual MODFLOW (Fig. 3-4) has been horizontally inverted, and will not correspond to any other cross-sections displayed from Visual MODFLOW in this report.

Fig. 3-3 and Fig. 3-4 show one potential discrepancy between the model and real conditions. Bedrock can be clearly seen in Fig. 3-3 sloping down from the left side toward the ocean on the right, and the freshwater-saltwater interface slopes up toward the coast from where it intersects the bedrock. In the model, however, the freshwater-saltwater boundary is vertical up and down. Instead of gently flowing up toward the coast, groundwater must flow all the way to this boundary and then leach directly upward. Although this is not a strict interpretation of real-world conditions, Walter and Whealan explain this anomaly in their report:

> "The interface position is fixed in the models and does not change in response to pumping or recharge stresses. This assumption is based on previous modeling of the salt/freshwater interface indicating that pumping was not sufficient to cause saltwater intrusion into the aquifers beneath the Sagamore and Monomoy flow lenses (Masterson and Barlow, 1997). The interface is truncated from below by bedrock and is represented laterally as a vertical no-flow boundary" (Walter and Whealan)

Aside from the confusion surrounding the vertical freshwater-saltwater interface, comparing these two figures provides additional verification for the model. The effective flow profile is very consistent between the two models- recharge in the upper layers leaches downward toward bedrock and then laterally and upward until it discharges at sea level. Additionally, this view shows the effect of groundwater mounding (the blue dashed line in Fig. 3-3) and a comparison of elevation at the higher points on the cape. After comparing these 4 figures, it was determined that the model was compliant with the report by Walter and Whealan.

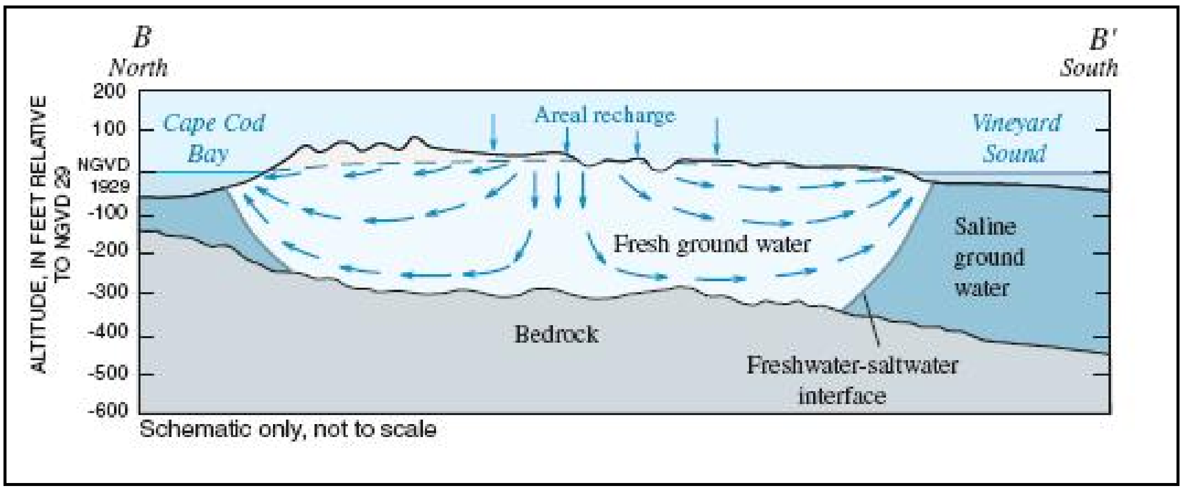

Fig. 3-3 ─ Cross-Sectional Flow Profile of Western Sagamore Lens, Cape Cod, Massachusetts (Walter and Whealan)

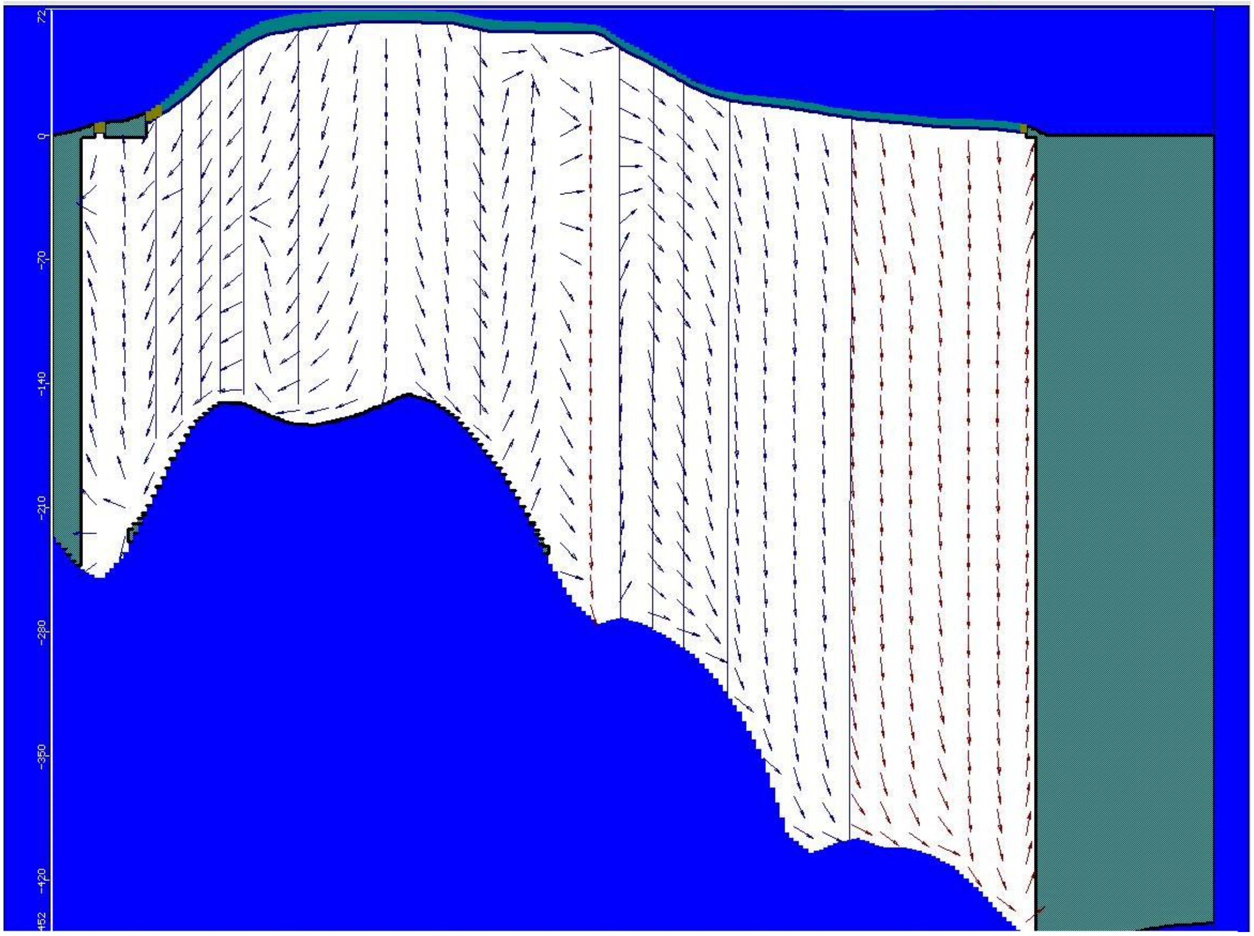

Fig. 3-4 — Cross-Sectional Flow Profile of Western Sagamore Lens, Modeled by Visual MODFLOW, Horizontally Inverted

The second test was to check the model's completeness and internal consistency by running a mass balance of the entire system. Theoretically, once steady state is reached, all the water that enters the system should sum to exactly the amount that exits the system. For this test, it was observed that the LST file, created during model runtime, automatically calculates an overall mass balance. For both the input and output, it considers storage, constant head, wells, drains, head bounds, recharge, and stream leakage. The total difference between the inputs and outputs was calculated, coming to the negligible error of 0.0014%.

At this point it was concluded that the import had been successful, and that the current working model was consistent with the USGS model. The remaining verification tests were designed to conclude that the model corresponds to reality and is useful for modeling groundwater. The first such test was to check the model water table levels against known, measured values. These values were taken from a report by the USGS and US Department of the Interior outlining water quality sampling procedures for samples taken in order to model a phosphorus-rich sewage contaminant plume near Ashumet Pond. Furthermore, this report used water table data taken from a USGS Open File Report on the water table altitude of Western Cape Cod. Data were displayed in the form of a contour map of the area in question, Western Cape Cod, using a 5 ft contour interval starting at sea level. For comparison, Visual MODFLOW was run using standard configuration and a general head contour map was generated upon completion of the model run. This map was generated using a 10 ft contour interval starting at sea level and color shaded for clarity. Although Visual MODFLOW generated a map of the entire Sagamore Lens, only the western portion is relevant to this report and only that section was analyzed. The two maps were compared side by side and were found to correlate to a satisfactory degree. Both the computer-generated model water table elevation map (Fig. 3-5) and the USGS measured water table elevation map (Fig. 3-6) are provided here for comparison.

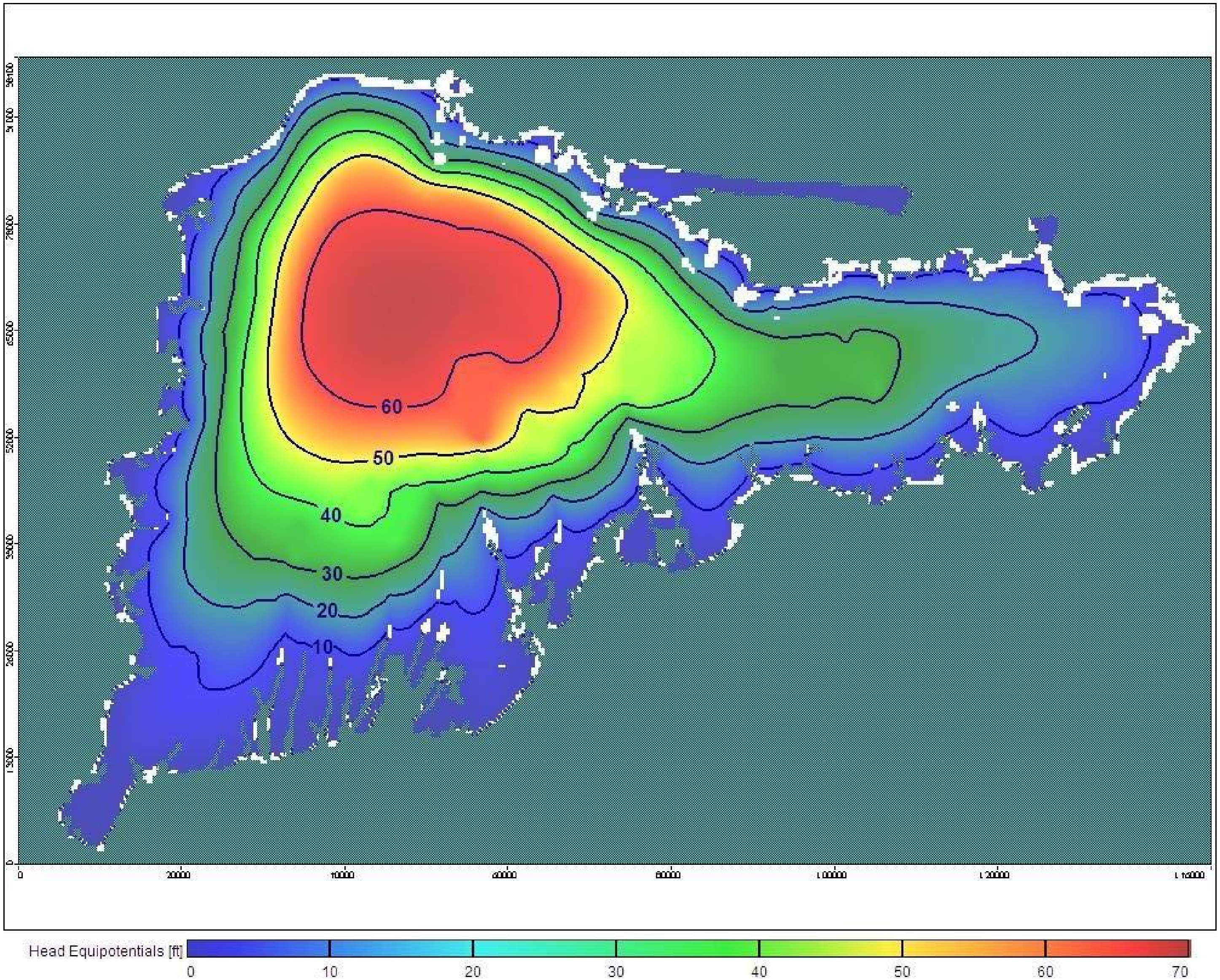

Fig. 3-5 – Computer Generated Model Water Table Elevation (Modified to include contour labels)

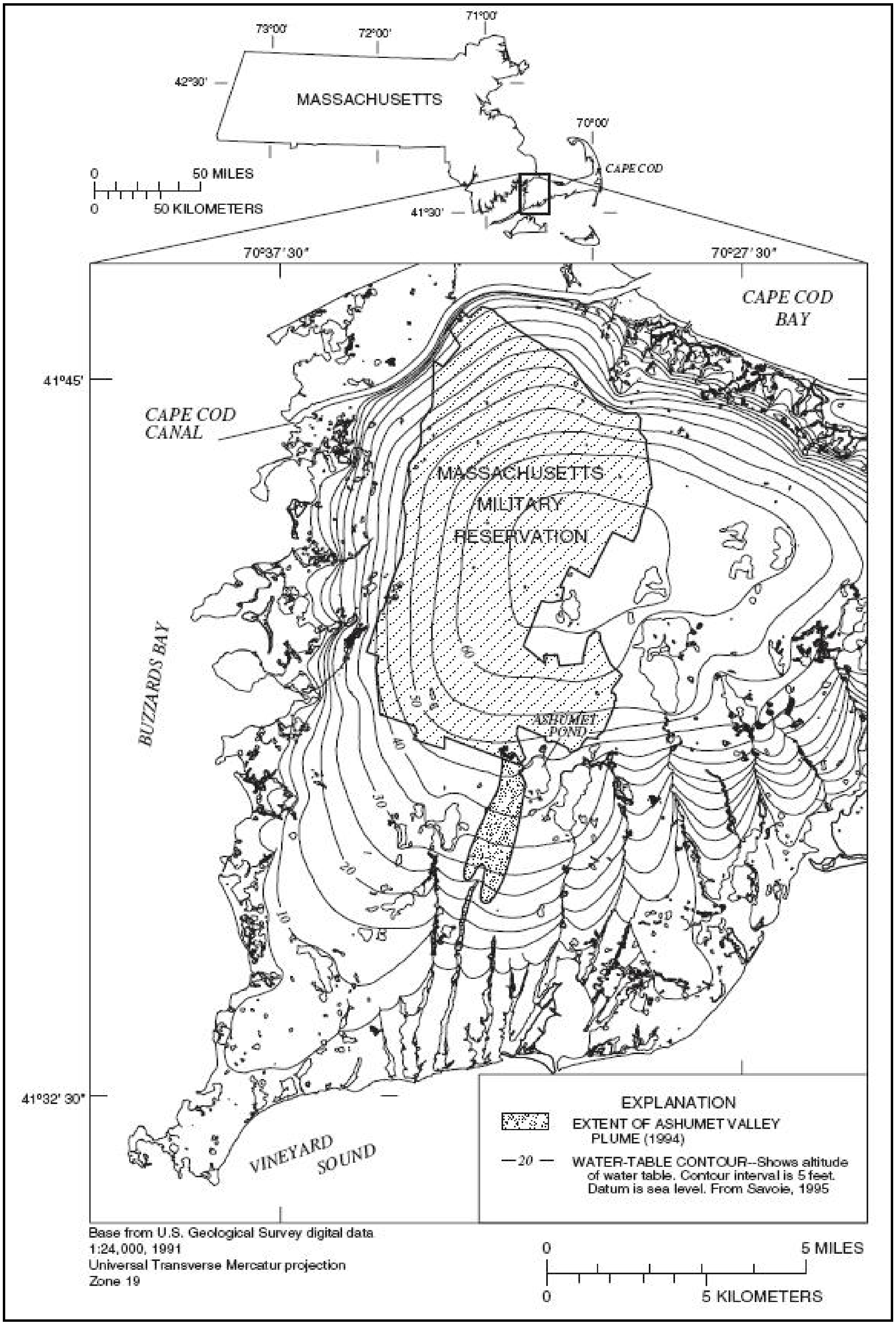

Fig. 3-6 – Measured Water Table Contours of Western Cape Cod, Massachusetts (Savoie and LeBlanc)

The final test was to compare model particle tracking against known, documented contaminant plumes found within the region. The particle tracking module of Visual MODFLOW, known as MODPATH, allows the user to place a theoretical particle of water into the model at any location and trace this particle either forward or backward in time to its ultimate entrance into or exit out of the groundwater system. Therefore, the model could be tested by placing a tracer in the actual ground, and observing where it travels over the years. If the travel path corresponds to the model's particle path when placed in the same location, this verifies that the model is true to reality. Fortunately, this experiment has already been completed. This region contains several contaminant plumes that are known and well documented. It is known precisely where the plumes began, and where they have traveled over the years.

For the purposes of model verification, a plume originating at the Massachusetts Military Reservation near Ashumet Pond was selected on the basis of availability of data and size and clarity of the plume path. Contaminants in the plume include phosphorus from wastewater treatment effluent discharge, volatile organic solvents, waste fuel and oils, and chemicals from a nearby fire response training ground, detailed in a 1995 USGS report on the subject. The report contains several charts which clearly delineate the source area of the plume as well as the current extent of the plume, which reaches south from the MMR toward western Waquoit Bay. Although it is not part of the study area for this report, it can still serve to verify the model. In order to model contaminant flow from the discharge area at the southern end of the MMR, a map of the area was loaded into Visual MODFLOW and referenced to known locations. After this process, Visual MODFLOW displays a transparent map of the discharge site over the corresponding model coordinates so particles can be accurately added to the model at the correct location. Once these particles are added, the model is run and the particle paths can be displayed over a map of the plume to compare the modeled flow to the measured flow.

Fig. 3-8 shows the Ashumet Plume area, including discharge sites at the southern tip of the MMR, Ashumet Pond, and the plume boundary. Fig. 3-7 shows this same map with the particle paths from Visual MODFLOW displayed on top. Based on the high degree of correlation between the particle tracks and the plume boundary, this was considered a successful test of model verification. It is important to note that the recharge values for the discharge areas at the MMR were not increased to represent wastewater effluent discharge. The wastewater discharge beds only operated for a few years and no sufficient value for recharge could be found for those years, so the effect was deemed negligible. If the recharge value was modified, the result would be a slight groundwater mound and minor spreading of the plume path.

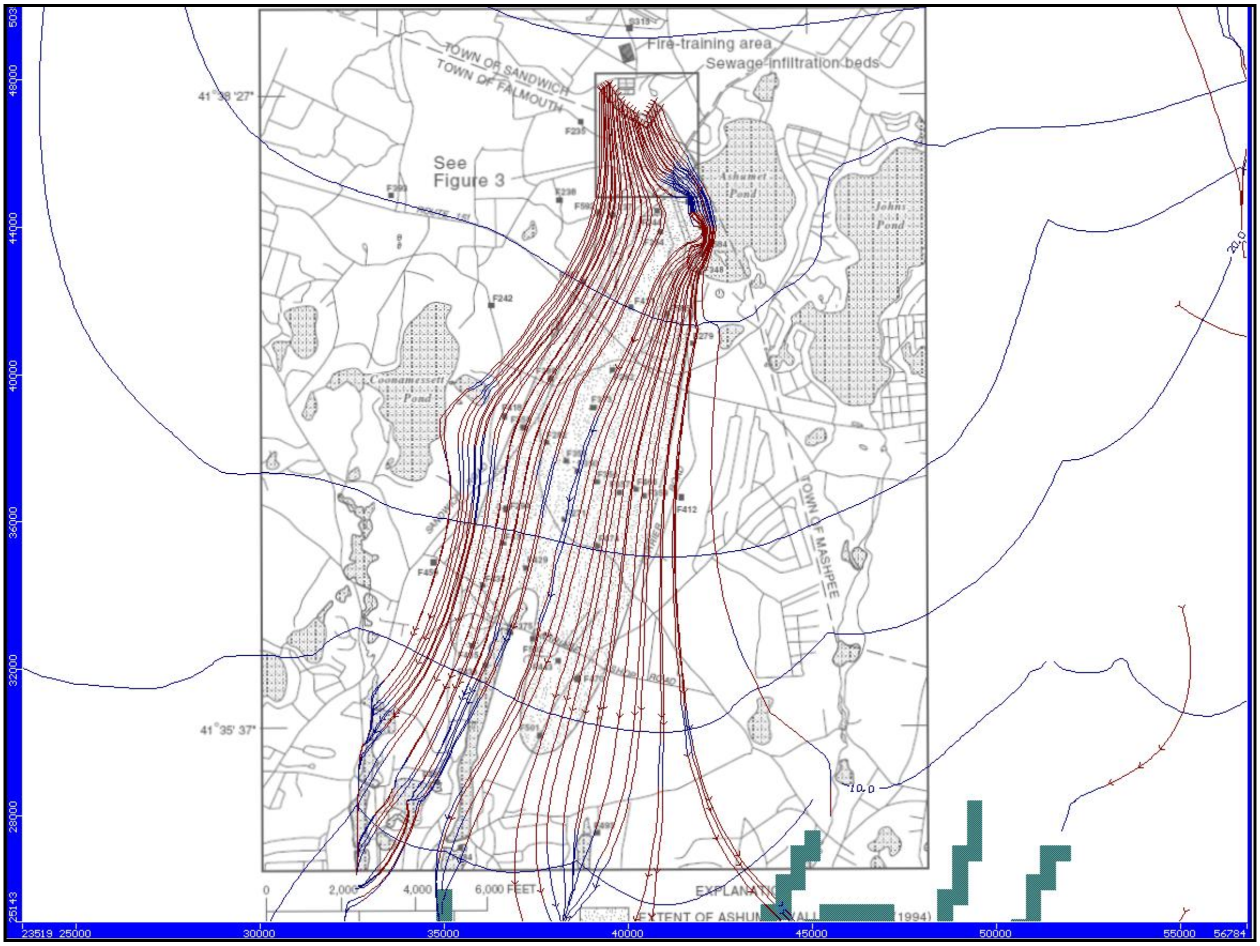

Fig. 3-7 – Particle Tracking Output Window from Visual MODFLOW showing the Ashumet Plume

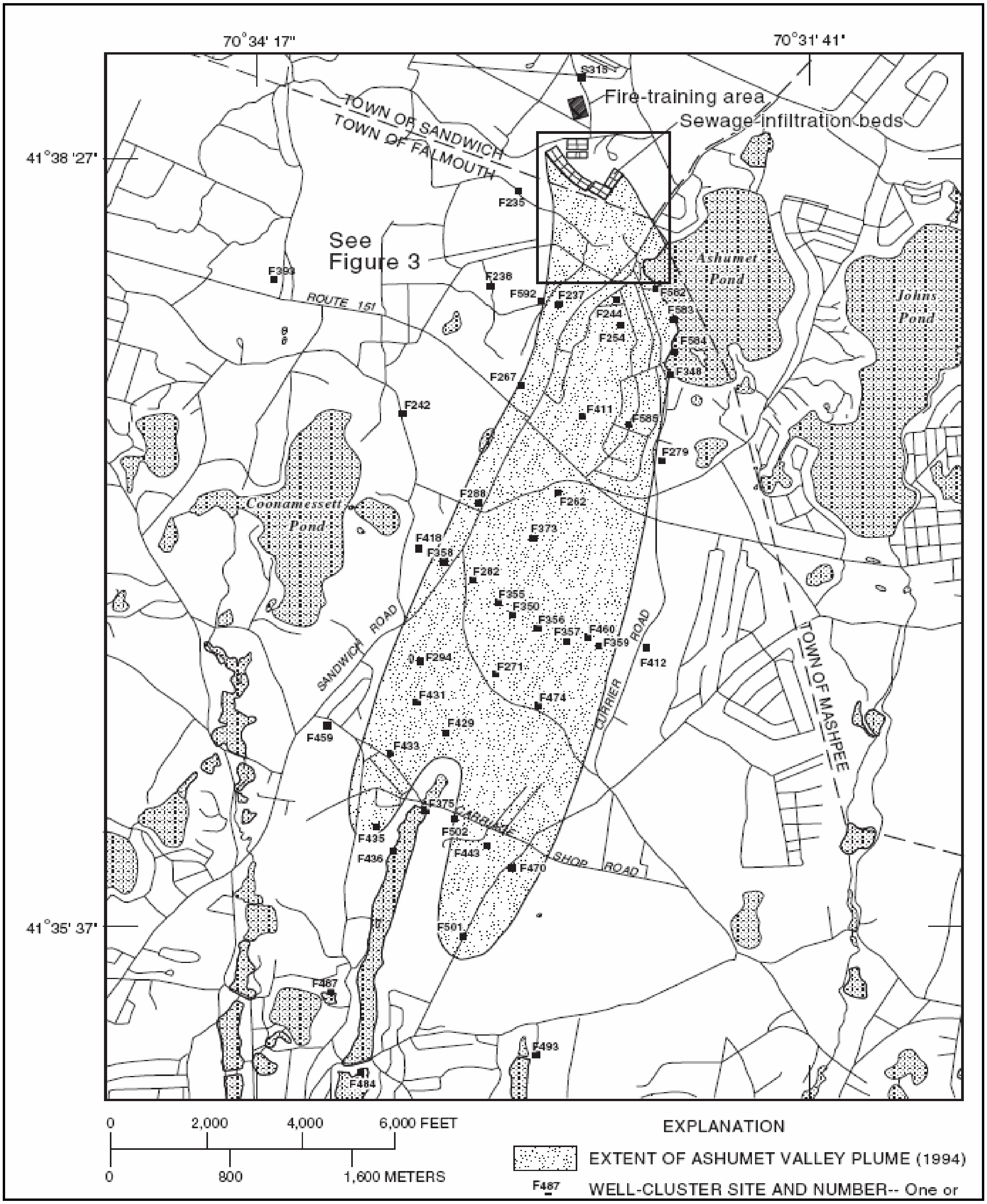

Fig. 3-8 ─ Ashumet Plume Discharge Areas and Plume Boundary, Falmouth, Massachusetts (Savoie and LeBlanc)

After observing the results of the particle tracking verification test and the other verification tests (inspection, comparison to the USGS design report, mass balance, and comparison of water table elevations) it was determined that the model was running correctly and accurately predicting real-world conditions as it was designed to do, including groundwater flow, particle tracking, head elevations, and other hydrologic conditions.

3.1.3 Mapping the Discharge Locations

Before commencing with modeling each site, a general map of all of the sites was imported into the regional model. The environmental engineering firm sponsoring the project, Stearns & Wheler, provided the necessary data including satellite imagery and an ESRI .SHP file containing the approximate boundaries of the discharge sites. A .SHP file contains polygon data defining the border of a region or regions, and usually includes references to a specific geographic coordinate system so that it can be combined with other data using GIS software. After the file was opened with ArcGIS, it was determined that the data was referenced to the Massachusetts State Plane Coordinate System, or SPCS.

The SPCS is a system of referencing separate pieces of geographic data to a common point so that they can be accurately combined. Since most geographic data readily available is two-dimensional data representing a three-dimensional system, there is inevitably some error associated with geographically projecting two-dimensional data onto the earth’s surface. The error increases with the distance from the reference point. Therefore, if all of the geographic data under consideration can be referenced to a single, nearby point the error will be minimized. For this reason, the State Plane Coordinate System was created by the USGS in 1938. The SPCS consists of a collection of reference points for different zones around the United States, loosely correlating to the borders of states but including multiple zones for larger states. (Jim Riesterer 1) Since the entire extent of this project takes place in Massachusetts, all of the relevant data are referenced to the Massachusetts SPCS.

The Massachusetts Geographic Information System, or MassGIS, is a state office that offers free geographic data for the entire state, all referenced to the MA SPCS. For the purposes of this project, Digital Orthophoto Quadrangles (DOQs) of the entire project area (roughly corresponding to the Popponesset and East Waquoit Bay Watersheds) were downloaded from MassGIS. DOQs are high quality scans of USGS topographic maps that are referenced to the SPCS. These DOQs were added to ArcGIS along with the .SHP file provided by Stearns and Wheler, as well as data layers for the border of the Town of Mashpee and surface water bodies in the area (provided by MassGIS) and the borders of the Popponesset Bay and East Waquoit Bay watersheds (provided by Stearns & Wheler). Using ArcGIS, these layers were laid over each other, resulting in a composite image that was exported to .jpg in order to be imported into Visual MODFLOW. This composite map is shown in Fig. 3-9 below. The composite map shows the Town of Mashpee in purple, water bodies in light blue, and potential discharge sites in yellow.

Unfortunately, Visual MODFLOW does not support State Plane Coordinates, but instead all points in the model are described by x- and y-coordinates from an arbitrary origin listed as 71° 41' 52" W, 41° 30' 11" N by Walter and Whealan (Walter and Whealan). Since the SPCS uses an entirely different origin than traditional latitude and longitude coordinates, no conversion is available between these two systems. Fortunately, Visual MODFLOW can import map data into the model by manually specifying known model coordinates on the map in question. This process, referred to as "georeferencing," was used to import the composite GIS map with discharge locations into Visual MODFLOW. After the map was imported and referenced to the model coordinates, particles were added at regular intervals within the proposed discharge site areas, visible in the model as yellow areas. An example of this practice, the Transfer Station site, can be seen in Fig. 3-10.

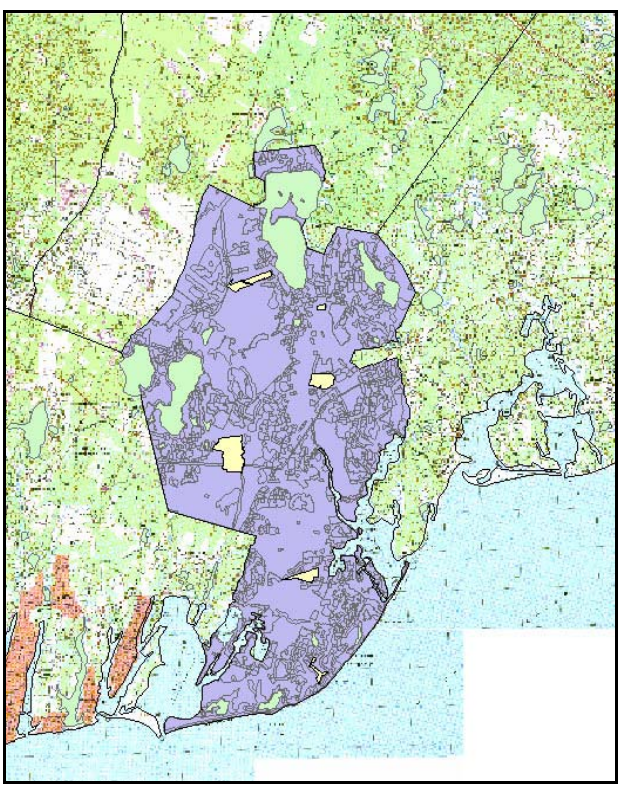

Fig. 3-9 – Composite Map of Mashpee Area exported from ArcGIS

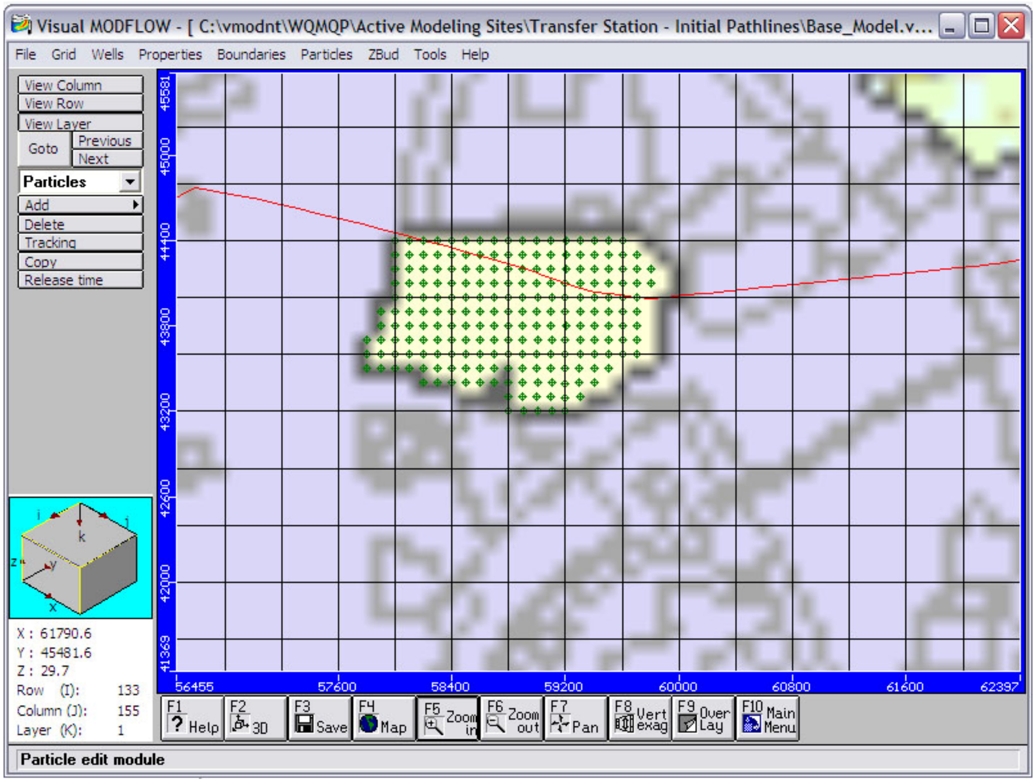

Fig. 3-10 ─ Visual MODFLOW Program Window Showing Particle Tracks at the Transfer Station, 100x100ft spacing

3.2 Site Modeling & Analysis

After fully preparing the regional model, each individual site was modeled and analyzed as a potential location for effluent disposal. Each site was considered individually based on several criteria, such as the area available for effluent beds, and the type of effluent bed technology applicable at the site. This was used to model the addition of an effluent bed to the regional model at each location individually. The results of this model were used to analyze the impacts on the surrounding area, such as estuarine nitrogen loading, localized mounding, and the affects on nearby subsurface features, such as public water supply wells or contaminant plumes.

The locations of the seven proposed discharge sites were provided by Stearns & Wheler in two forms: a .SHP file which was added to the site map, and highlighted satellite map data. An example of the latter, the satellite map of site 4, is shown in Fig. 3-11. Using the satellite images, the sites were located on Google Earth and the approximate available areas of the effluent beds were determined. The area of an effluent bed is critical, because it determines the flow capacity. At this initial phase of the design process, approximate areas were calculated by dividing the area into large shapes which fit correspond to the usable boundaries of the site. These shapes were then measured using Google Earth's "ruler" feature. Once the total available area of the site was determined, it was assumed that 90% of this area would be used for actual infiltration facilities, assuming 10% use for access roads, borders, overhead, etc.

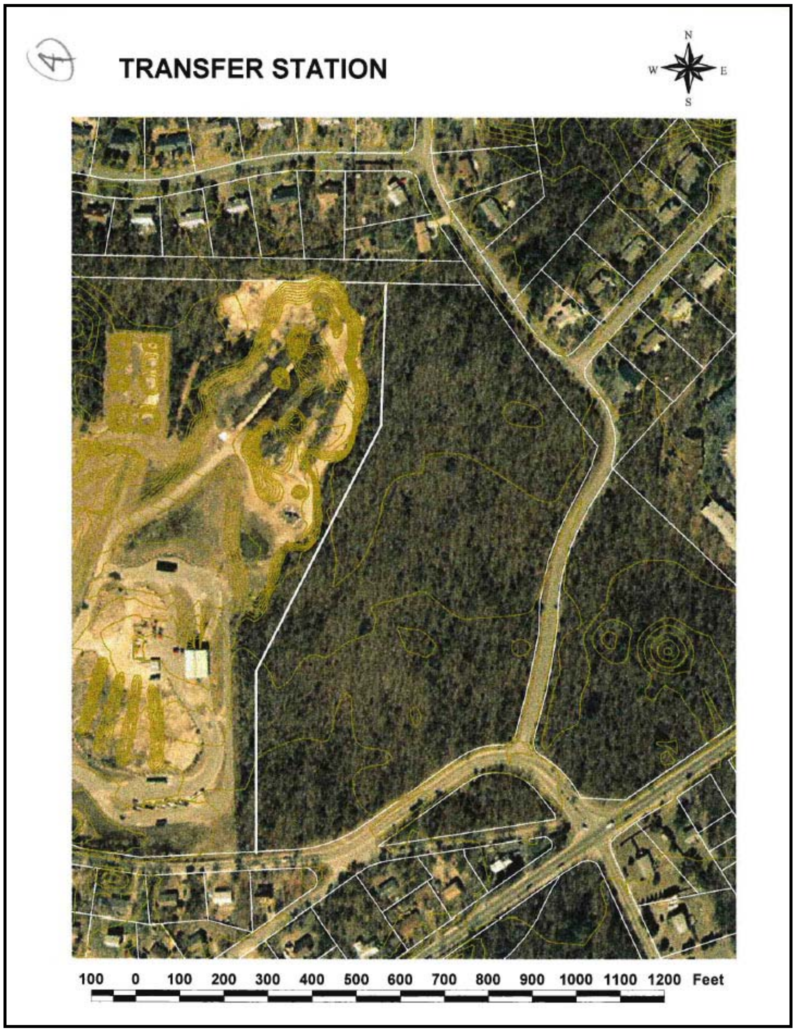

Fig. 3-11 – Satellite Map of Site 4, Transfer Station, Provided by Stearns & Wheler

Next, the site land use was used to determine which technology of infiltration bed was to be used. Two technologies were considered: open bed infiltration basins, and subsurface leach fields. Open bed infiltration basins were assumed to have a standard infiltration rate of 5.0 gallons per day per square foot (GPD/ft^2), while subsurface leach fields have a capacity of 2.5 GPD/ft^2. At this phase of the project, it is assumed that wherever possible, open bed infiltration basins are preferable due to smaller basin area required, and lower construction costs. Subsurface leach fields, on the other hand, incur the extra cost of site restoration when these systems are located on sites where they would be installed under parking areas, parks, golf courses or playing fields. However, once they are built, they have the advantage of being able to exist quietly underground. Accordingly, open bed infiltration basins were selected for currently undeveloped or forested sites. Subsurface leach fields were selected for sites where parking lots, baseball fields, and golf courses currently exist.

The approximate area of the effluent bed was multiplied by the flow rate for the chosen technology to determine a volumetric flow rate, in million gallons per day (MGD), which is the capacity of the site to absorb wastewater effluent. This value was later used to both model the site and to compare the sites against one another.

As discussed above in the Model Verification section, Visual MODFLOW includes a particle-tracking module called MODPATH, which allows users to add discrete “particles” to the model at any coordinate and track their movement through the aquifer. These particles are extremely useful as a visual representation of groundwater flow through the system, as they represent a complete path between any point and its eventual outlet. Furthermore, once the MODFLOW2000 numeric engine has been run, MODPATH can complete the calculations necessary to model thousands of particle tracks with run times on the order of seconds, which makes it a very valuable tool. For these reasons, MODPATH was selected as the tool for the purposes of tracking wastewater effluent through the Sagamore Lens aquifer.

In order to use MODPATH effectively, particles must be entered at the precise location of proposed effluent discharge sites on the cape. Separate copies of the model files were made for each potential discharge scenario so that each scenario could run without interference. At each of the sites, particles were placed in a regular grid within the discharge site, as well as along the outer edge of the site. Depending on the size of the site, these particles were placed either 100 ft or 200 ft apart, as shown in

Fig. 3-10. The MODFLOW2000 and MODPATH solvers were run for each site, and the output was recorded. Individual outputs are listed in the section for each site, and an example is shown here. These results are referred to as the “Initial Pathlines” results, because they show where particles travel in the unaltered model.

The purpose of introducing the particles into the model was to determine the impacts of adding an effluent bed at each location, but the true addition to each model was found by adding the flow due to the effluent bed into the model. For the initial pathline models the recharge values at the discharge sites were not changed from their original model value of 692.46

mm/year. This value represents an average value for non-specific land use on Cape Cod, including rainwater recharge, residential wastewater flow from septic tanks, and many other uses. Actual recharge at the discharge sites would be a much higher value, representing the millions of gallons of effluent entering the site in a small area. This increased recharge would certainly have an effect on the particle tracking and other hydrologic conditions in the area, including groundwater mounding and facilitated contaminant transport. This increased rate of recharge would be equal to the maximum daily loading rate per area as identified by MassDEP guidelines for design loading rates, as 5 GPD/ft^2 for open bed infiltration basins and 2.5 GPD/ft^{2}for subsurface leaching trenches. These figures were converted into the recharge units used by Visual MODFLOW (mm/year) through the unit conversion shown in Equation 1.

$$1\frac{\text{gal}}{\text{ft}^2 \bullet \text{day}} \bullet \frac{3.785 \bullet 10^{-3}\text{m}^3}{\text{gal}} \bullet \frac{10.764\ \text{ft}^2}{\text{m}^2} \bullet \frac{1000\text{mm}}{\text{m}} \bullet \frac{365\text{day}}{\text{year}} = 14\ 882.1\frac{\text{mm}}{\text{year}}$$

Equation 1 – Conversion between recharge units

Using this conversion factor, the modified recharge rates were calculated and an additional 700mm/year was added to account for the normal annual recharge for that area (rainwater, runoff, etc), yielding final values of 75,100 mm/year for open bed infiltration basins and 37,900 mm/year for subsurface leaching trenches.

The model grid for the regional model used in this report uses 400ft x 400ft spacing, as discussed in the Model Verification Section above. However, this grid spacing has insufficient resolution to adequately fit the borders of the proposed discharge zones. Visual MODFLOW does not allow the recharge value to be changed for part of a cell; it must be changed for the entire cell. However, if all of the cells which are part of the proposed discharge zones were changed to the design recharge rate, the result would be extremely inaccurate, and the higher recharge value would be applied outside the feasible recharge zone. Several approaches were attempted in the pursuit of accuracy.

Initially, the model grid was refined in the area of interest by adding additional rows and columns along the edges of the proposed discharge site and recharge values were modified for all of the internal cells. This approach was scrapped when it was found that the wildly varied cell sizing and arbitrarily placed cell boundaries caused the numeric engine used by MODFLOW 2000 to fail. The iterative calculations which form the core of MODFLOW simply did not converge.

To remedy this problem, the grid was refined within the site area to a regular grid with 100ft by 100ft cell. This time the model converged, but with strange results: the water table seemed to sink at random places across the Sagamore Lens, even miles away from any affected cells. It was clear that this model could not be trusted, since the results changed without changing the input data. It was hypothesized that perhaps the problem was due to the fact that, by adding rows and columns, the initial heads data from the last MODFLOW 2000 run was invalidated, so the solver was forced to rely on the model's set heads. However, when the base model was verified, it was found that these initial heads were off by up to 73 feet from the calculated values. Perhaps, it was supposed, Visual MODFLOW was unable to recover from such inaccurate starting data. To test this hypothesis, new initial head zones were defined which placed every cell within five feet of its theoretical value. However, when the grid was modified and the model was run, the same bad results were produced. Other methods of modifying the numbers of columns and rows were attempted, all without success. In one attempt, the entire model was refined uniformly, creating a model with 487 rows and 492 columns (500 is the max allowed). In another, grid lines were removed from the edges and added to the center, refining the grid without changing the number of rows and columns. No such attempts had results that matched the verified base model.

As an inferior but workable solution, existing rows and columns were slightly adjusted locally to align more carefully with the borders of the proposed discharge sites. The discharge sites were approximately located using Google Earth to determine areas and relative distances from identifiable landmark features. MODFLOW 2000 and MODPATH were run successfully for each of the seven proposed discharge sites. A sample modified recharge zone can be seen in Fig. 3-12. Between modifying the grid and activating the new recharge zone at each site, the water table was confirmed by comparing it with the verified base model.

Once the modified recharge was activated, the model was run again, with results which varied from the initial pathline models as expected. Increased groundwater mounding and particle track spreading were observed, substantial in some cases. After modeling was complete, the output was recorded for each site. These are referred to as the “Modified Recharge” results.

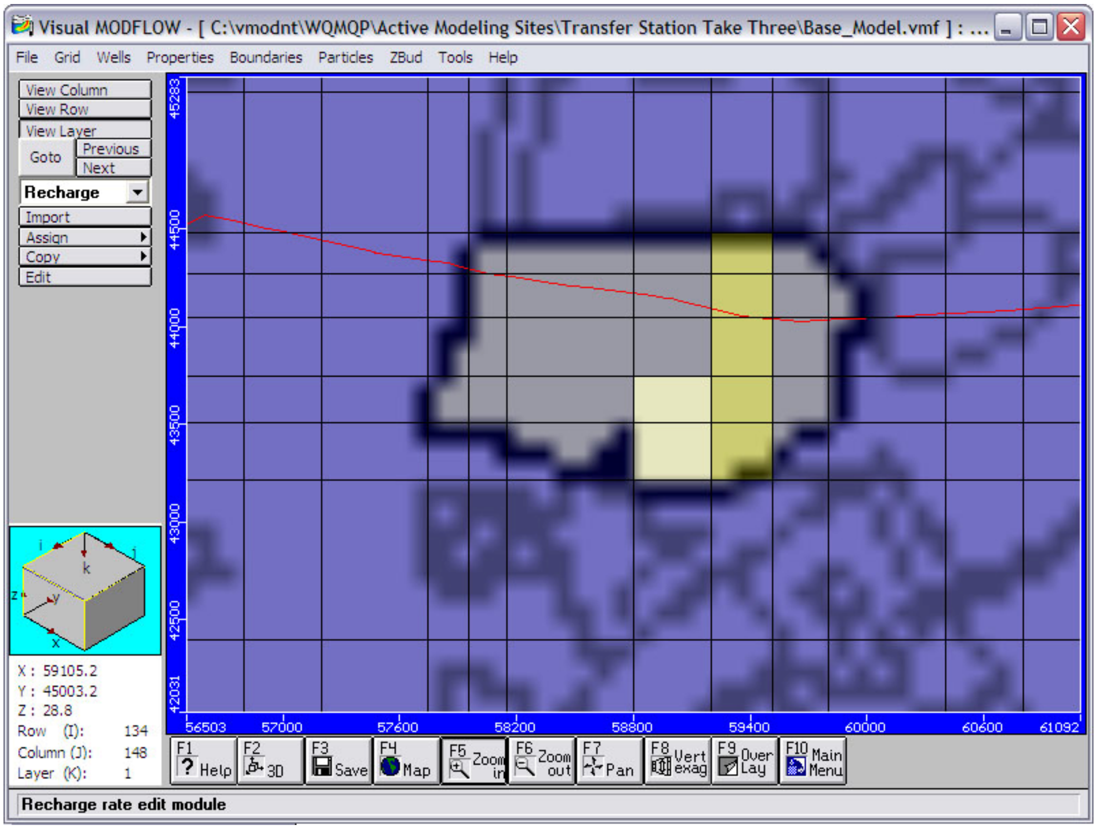

Fig. 3-12 – Visual MODFLOW Program Window Showing Modified Recharge Zones

The modified recharge results show a wide variation in the behavior of particle tracks and in hydrologic conditions resulting from increased recharge. Additionally, there are several other factors to consider for each site, such as mounding, the impact on local wells, and the impact on local known contaminant plumes, all of which are listed in the detailed explanation of each site in the Results section below. The results of the modified recharge modeling were analyzed to determine several different discharge scenarios designed to handle the wastewater treatment load requirements of the Town of Mashpee. These scenarios were subsequently modeled using the same modeling procedure, and the results are discussed below.

4 Results

The results of computer modeling with Visual MODFLOW comprise the bulk of the data on which the conclusions of this report are based. Results are generally taken directly from the output of the computer program, mostly in the form of maps generated from the "Export Image" function. Important images shown in this section include the modeling conditions and the results of particle tracking for each site and the predicted groundwater conditions during discharge. These results are analyzed to determine the value of each site for effluent discharge based on a set of criteria outlined in the beginning of this chapter, and the analysis serves as a basis for the discharge design outlined in the next chapter.

4.1 Design and Analysis Criteria

Prior to modeling and examining each site, it was necessary to consider two sets of criteria: design criteria and analysis criteria. Design criteria consist of the desired features which define a satisfactory design. This set served to guide the project into a successful solution. The design criteria were provided by Stearns & Wheler, an environmental engineering firm hired by the Town of Mashpee. The design criteria are as follows:

Design Criteria

- Yearly Average Discharge of 1.65 MGD
- Peak Factor of 2.0
- Goal is to minimize nitrogen load on estuarine systems
- Open Bed Infiltration Basins can accept discharge of 5 GPD/ft^2
- Subsurface Leaching Trenches can accept discharge of 2.5 GPD/ft^2
- Effluent treated to 3 mg/L if groundwater flows to a sensitive area
- Effluent treated to 10 mg/L if all groundwater does not flow to a sensitive area
- Groundwater mounding resulting from discharge is not to endanger existing structures

It would be difficult to objectively quantify and compare several proposed discharge options strictly based on the design criteria. Accordingly, these criteria were distilled into a list of quantifiable results which could be obtained for any proposed discharge scenario and directly compared for objective analysis. The analysis criteria below are the guidelines by which each discharge location was analyzed in order to determine which sites were optimal, and what restrictions were to be placed upon them. These same criteria were used in considering each individual site as well as each Scenario for a complete design.

Analysis Criteria

- Usable area
- Appropriate infiltration technology
- Discharge capacity
- Maximum percentage nitrogen load to any 1 embayment (excluding main bays)
- Total percentage nitrogen load to embayments (excluding main bays)
- Total number of embayments among which nitrogen load is distributed
- Initial depth to water table
- Depth to mounded water table
- Other considerations: contaminant plume, supply wells

The usable area of the site and the appropriate infiltration technology are both required to calculate the discharge capacity. The area of the site depends on how much land is available; available land must not be occupied by any permanent buildings or infrastructure and it must be sufficiently far from residential and other high-use areas that the discharge facility would not have a severe impact on daily life in the area. The Massachusetts DEP defines this "setback distance" as 25 feet (Massachusetts Department of Environmental Protection). The appropriate infiltration technology depends on the current land use at the site: undeveloped areas can support open bed infiltration basins, but any area that requires redevelopment (the Ball Fields or New Seabury sites) can only support subsurface leaching trenches. Recommendations for technologies to be considered for each site were provided to the team by Stearns & Wheler during a meeting and are listed in the results section for the individual sites. Based on the available discharge area and the daily capacity of the discharge technologies, the discharge capacity of the entire site can be calculated.

The particle tracking analysis will determine how nitrogen will load the various embayments. In the case of discharge sites that drain to several embayments, the particle tracks clearly represent what proportion of effluent will drain to each embayment, and based on this proportion and the total nitrogen discharge at the site it is possible to calculate nitrogen loads separately for every area of interest. TMDL reports for Eastern Waquoit and Popponesset Bay state that any effluent which seeps into the main open bay area will be quickly swept out into the Nantucket Sound due to high tidal flushing volume in the bays (Howes et al.), so any particles which trace to the open bay areas or directly into Nantucket Sound are discounted when calculating embayment nitrogen loading.

Loading analysis is used to determine how nitrogen loads correspond to the nutrient capacity of the embayments: If the maximum percentage to any embayment is high, the health of that embayment will suffer unacceptably. The ideal embayment loading by percentage is equal to 100% divided by the total number of embayments. For example, if a discharge site drains to two embayments the ideal distribution is 50-50 between the two, in which case the maximum embayment loading by percentage would be 50%. If the distribution was 80-20, a less desirable scenario, the maximum embayment loading by percentage would be 80%. Ideally, some percentage of the nitrogen will seep into the Nantucket Sound or into the open areas of Waquoit or Popponesset Bay and be swept away by tidal flushing. If 20% of the discharge flowed out into the ocean, the ideal breakdown would be 40-40 and the maximum load by percentage would be 40%. Sites with a lower maximum embayment loading by percentage are more valuable as discharge locations.

If the number of embayments is high the nitrogen load in each will be low. If the number is low, the nitrogen load will be higher. If a site drains entirely into Nantucket Sound, the number will be zero because the maximum load entering any actual embayment is zero.

It is important to note that many other factors must be taken into consideration before planning for this project will be complete. Some factors not considered by this report include: The location and design of wastewater treatment facilities (as opposed to discharge facilities), the cost of sewering the entire Town of Mashpee (including the cost of pumping effluent from source to treatment to discharge sites), political impact, the effect on local property values, attenuation of nitrogen in surface water bodies, the cost of decommissioning existing septic systems, the reduction of recharge in the Town of Mashpee due to the decommissioning of septic systems, and other factors. If a factor is not specifically listed in this report, it should be assumed that it was not considered in analysis.

4.2 Individual Site Modeling Results

In order to determine how to best distribute treated wastewater effluent between the seven proposed discharge sites in Mashpee, it was first necessary to understand the hydrologic conditions at each site. The effluent discharge facilities under consideration for the Town of Mashpee would put millions of gallons per day into an extremely small area, increasing the local recharge by as much as one hundredfold in some cases. Although the Cape Cod aquifer is understood to be highly transmissive, this increase in recharge will inevitably affect the water table and groundwater flow conditions in the area immediately surrounding the site, and potentially elsewhere within the Sagamore Lens. Nitrogen loads in coastal embayments correlate directly to nitrogen loading in the aquifer. In order to understand these relationships, each proposed discharge site was individually modeled under theoretical effluent discharge conditions.

4.2.1 Site 1 – Ball Fields

Fig. 4-1 – Site 1, the Ball Fields

Proposed discharge site 1 is a parcel of land owned by the Town of Mashpee, between Ashumet Road and Main St in the north of Mashpee. This site, referred to as the “Ball Fields,” currently contains several baseball diamonds of varying sizes and a parking lot. The southwestern corner of the site was under development at the time of the satellite image above, and its current status is unknown. No image of the site was available at the time of this project, and the image shown in Fig. 4-1 was taken from Google Earth. Subsurface leaching trenches could be constructed under the entire site, but the baseball diamonds must be rebuilt later.

Fig. 4-1 shows the polygons used to approximately measure the available infiltration area at the ball fields, outlined in yellow. The total calculated available area of the site is 881,500 sq ft and the available infiltration bed area (assuming 90% of available site area) is 793,350 sq ft. Using a discharge value of 2.5 GPD/ft^2, the total potential discharge for the site is 1.98 MGD. However, nitrogen loading in the embayments must be calculated using average conditions, so the particle tracking model was run with a total site discharge of 1.65MGD as per the design criteria, which equates to a modified recharge value of 21,760 mm/year. The modified recharge zone for the Ball Fields site is shown in Fig. 4-2. The Ashumet Road Site can be seen directly to the southwest of the discharge zone, and the green recharge zone to the northeast is Mashpee Pond. The particle tracking results for the Modified Recharge scenario is shown in Fig. 4-3; 134 particles were used in 100ft x 100ft spacing.

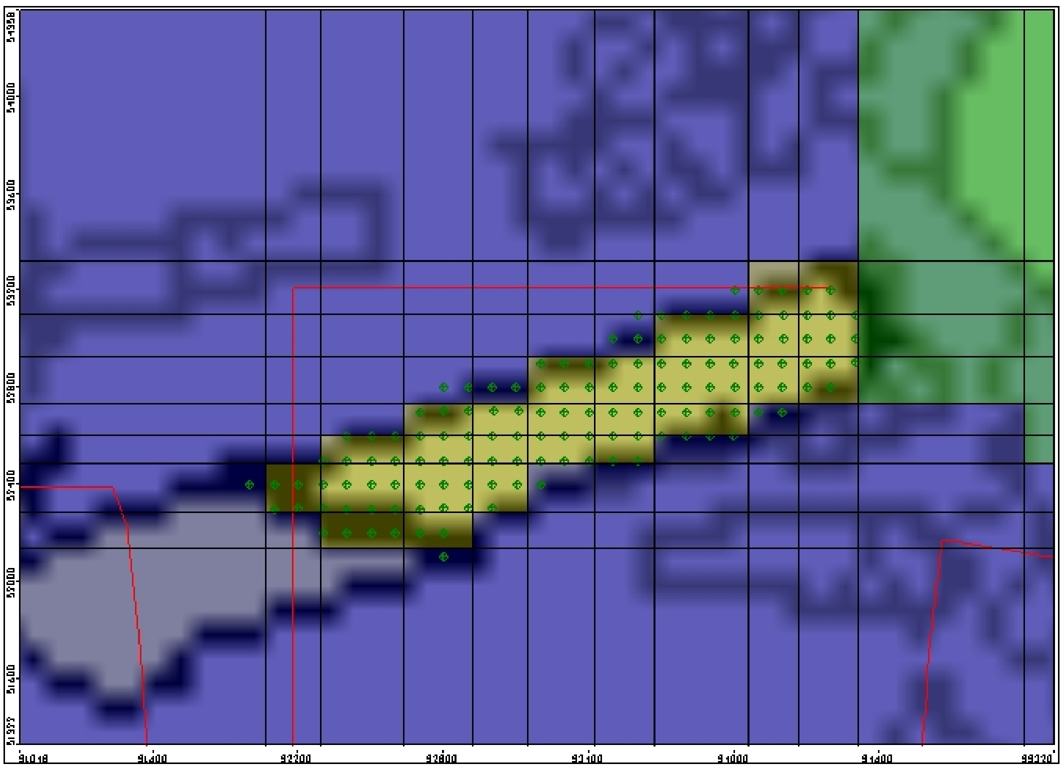

Fig. 4-2 – Ball Fields Modified Recharge Zone

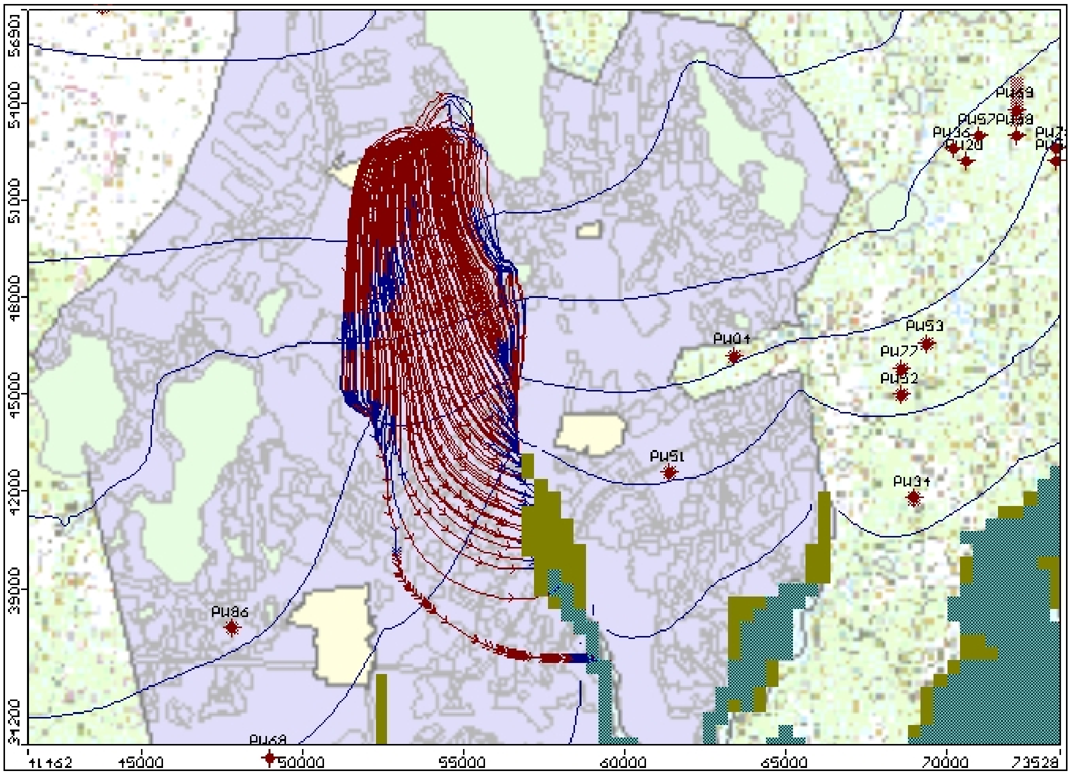

Fig. 4-3 – Particle Tracks From Ball Fields Modified Recharge Zone

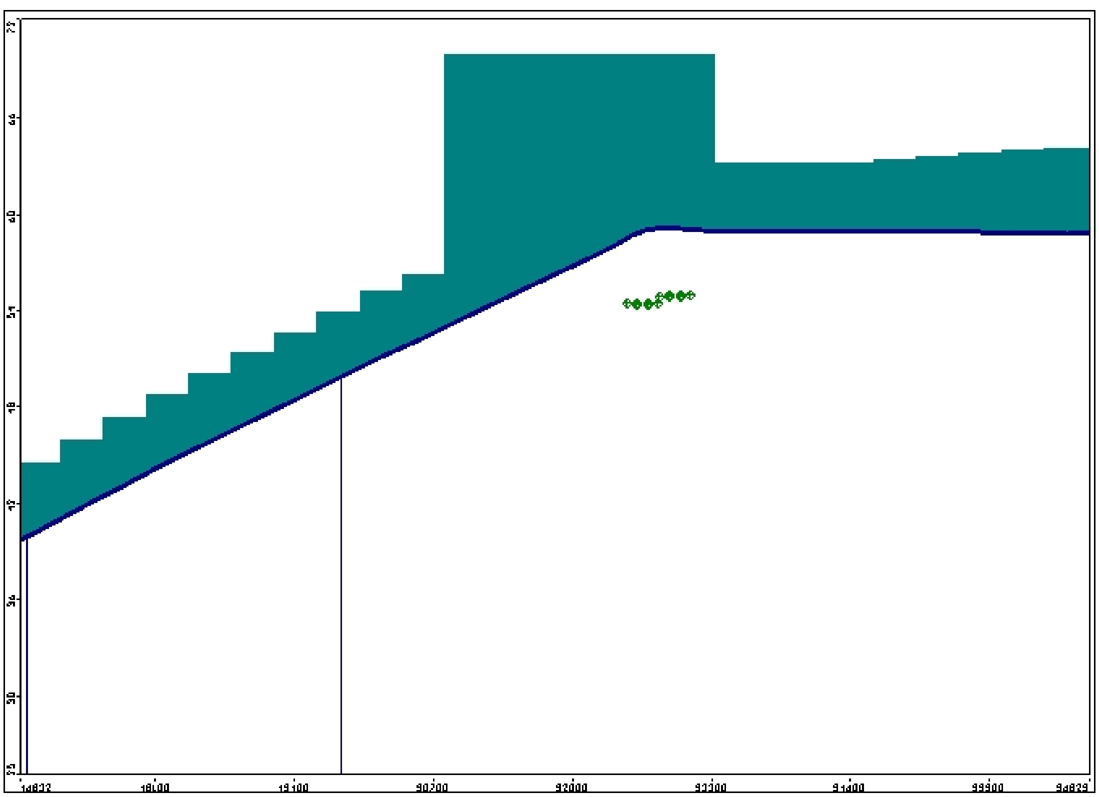
Fig. 4-4 – Groundwater Table Mounding at the Ball Fields Site

Fig. 4-4 is a cross-section the Cape Cod water table at the discharge site. The white area represents the contour of the top of the water table and the green area represents the highest possible elevation programmed into the model. The green dots represent the discharge site. In this figure (and all other water table cross-sections) the view is from east to west, the right side of the picture is the north and the water table slopes down to the south on the left side. This figure was modeled at peak site discharge of 3.3MGD; the modified recharge zone uses a recharge value of 26,016 mm/year. These results are discussed further in Section 4.3.1.

4.2.2 Site 2 – Ashumet Road

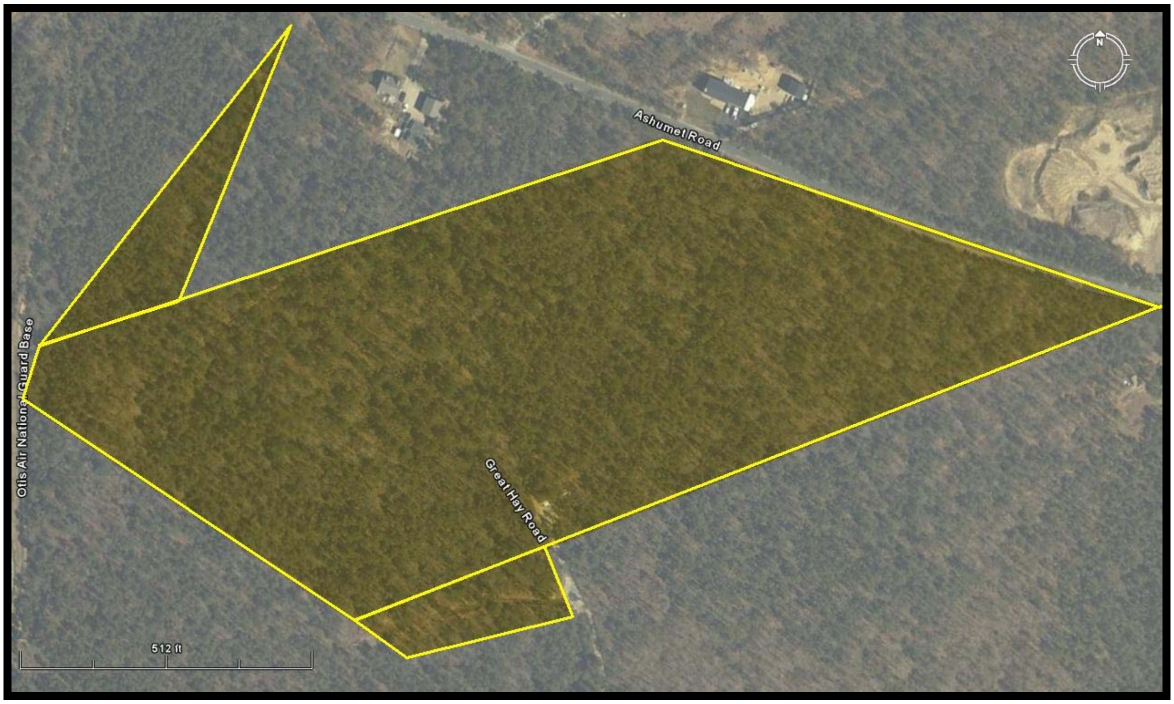

Fig. 4-5 – Site 2, Ashumet Road

Proposed discharge site 2 is a large parcel of forested land, currently undeveloped, owned by the Town of Mashpee. It is located between Ashumet and Great Hay Roads in northern Mashpee, directly across from the Ball Fields, and is known as the Ashumet Road Site. There is currently no development on the site, so the entire site area is considered available for open bed infiltration basins. Fig. 4-5 shows infiltration area measured for the area at Ashumet Road. Areas are taken as one triangle, one parallelogram, and one trapezoid. The total calculated available area of the site is 1,166,550 sq ft and the available infiltration bed area (assuming 90% of available site area) is 1,049,895 sq ft. Using a discharge value of 5 GPD/ft^2, the total potential discharge for the site is 5.25 MGD. However, nitrogen loading in embayments is calculated using average loads of 1.65MGD which equates to 27,568 mm/year. The modified recharge zone for the Ashumet Road site is shown in Fig. 4-6. The Ball Fields Site can be seen directly to the northeast of the discharge zone, and Mashpee Pond can be seen as a green recharge zone further to the northeast. The particle tracking results for the Modified Recharge scenario is shown in Fig. 4-7; 128 particles were used in 100ft x100ft spacing.

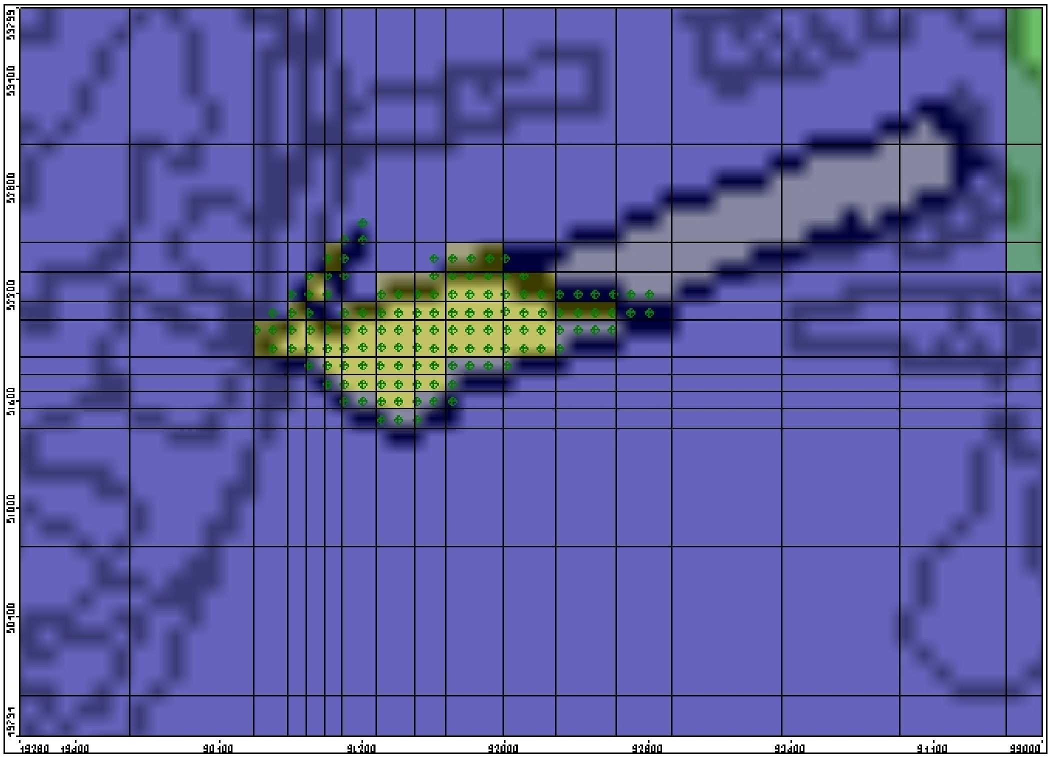

Fig. 4-6 – Ashumet Road Modified Recharge Zone

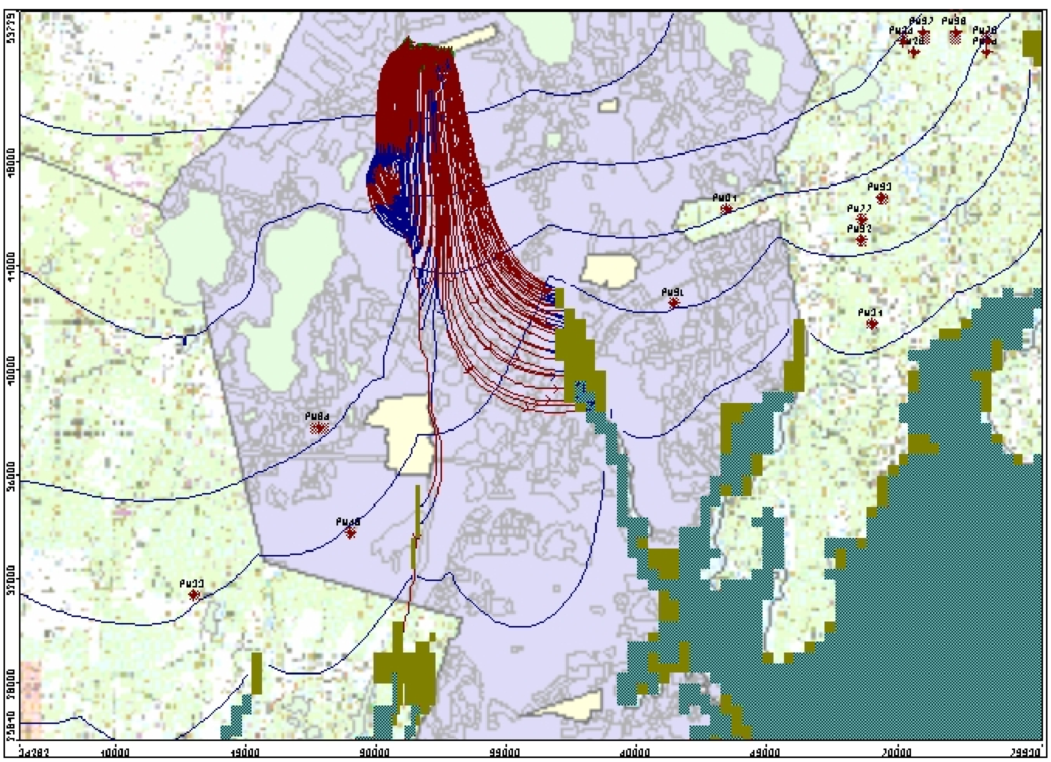

Fig. 4-7 – Particle Tracks from Ashumet Road Modified Recharge Zone

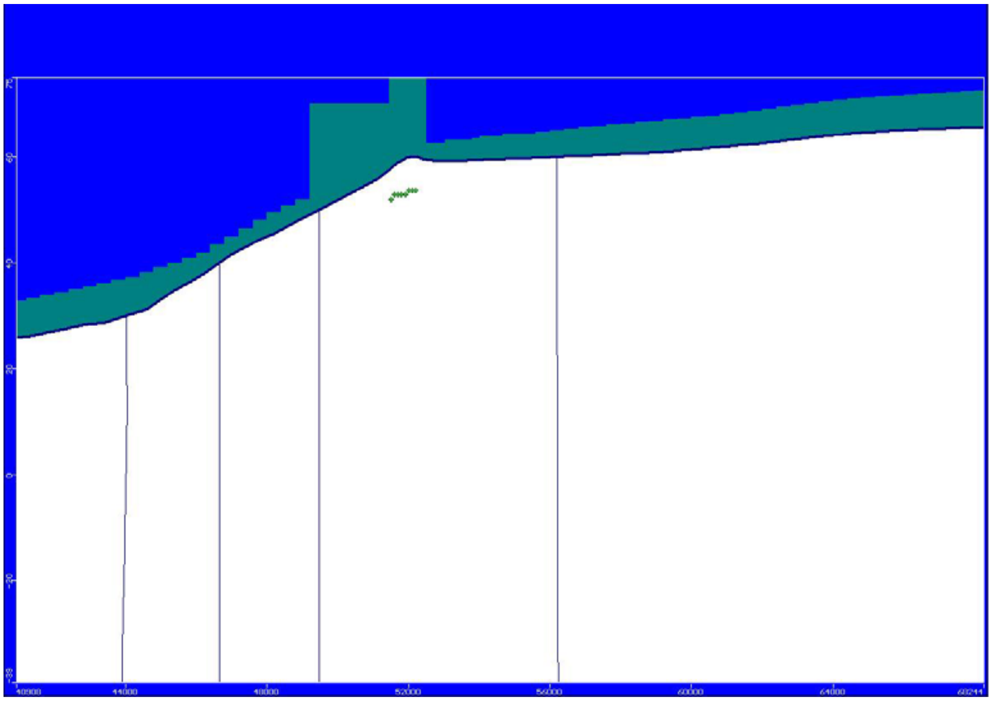

Fig. 4-8 – Groundwater Table Mounding at the Ashumet Road Site

Fig. 4-8 is a cross-section of the Cape Cod water table at the discharge site. This figure was modeled at peak discharge conditions (3.3MGD, 42,768 mm/year) to represent water table mounding due to effluent discharge. The mound is visible as a bump in the water table elevation between the center-most blue lines. These results are discussed further in Section 4.3.2.

4.2.3 Site 3 – Old Town Dump

Although seven sites were initially taken into consideration, Site 3 – "Old Town Dump" has been omitted from consideration. During a meeting with Stearns & Wheler, it was determined that this site might not be favorable due to its location and potential for undesirable soil characteristics for an effluent discharge facility. This site received no further consideration.

4.2.4 Site 4 – Transfer Station

Fig. 4-9 – Site 4, Transfer Station

Proposed Discharge Site 4 is a site currently used by the Town of Mashpee for a transfer station for the town's municipal waste disposal service, located near the intersection of Beacon Way and Meetinghouse Road in east-central Mashpee. The site is located due north of the head of the Mashpee River. The site initially seems enormous, although upon inspection it turns out that the majority of the site is occupied by waste handling facilities and a capped landfill that could not be easily or cheaply rebuilt if they were destroyed for the purpose of installing subsurface leach fields. There is, however, a large forested area on the eastern corner of the site which is available for discharge and, since it currently isn't being used, is appropriate for open bed infiltration basins. Additionally, there is a large parking lot at the southern end of the site.

Although the availability of this site could not be easily determined, it was considered prudent to include this area in the discharge modeling in order to account for the maximum capacity of the site. The parking lot area was modeled as subterranean leaching trenches. Fig. 4-9 shows infiltration area measured for the area at the Transfer Station. Areas are taken as two trapezoids and one parallelogram. The total calculated available area of the forest site is 477,100 sq ft and the available infiltration bed area (assuming 90% of available site area) is 429,390 sq ft. Using a discharge value of 5 GPD/ft^2, the potential discharge for the forest site is 2.15 MGD. The total available area of the parking lot site is 210,000 sq ft, and using a discharge value of 2.5 GPD/ft^2 the potential discharge for the parking site is 0.525MGD. The total potential discharge for the Transfer Station is 2.675MGD. However, embayment nitrogen loading must be calculated at the average load of 1.65MGD, which equates to recharge levels of 46,323 mm/year for the forested area and 23,377 mm/year for the parking lot. The modified recharge zones for the Transfer Station site are shown in Fig. 4-10. Fig. 4-11 shows the particle tracks from the modified recharge zone, modeling potential effluent flows in the aquifer. 70 particles were used in 100ft x 100ft spacing.

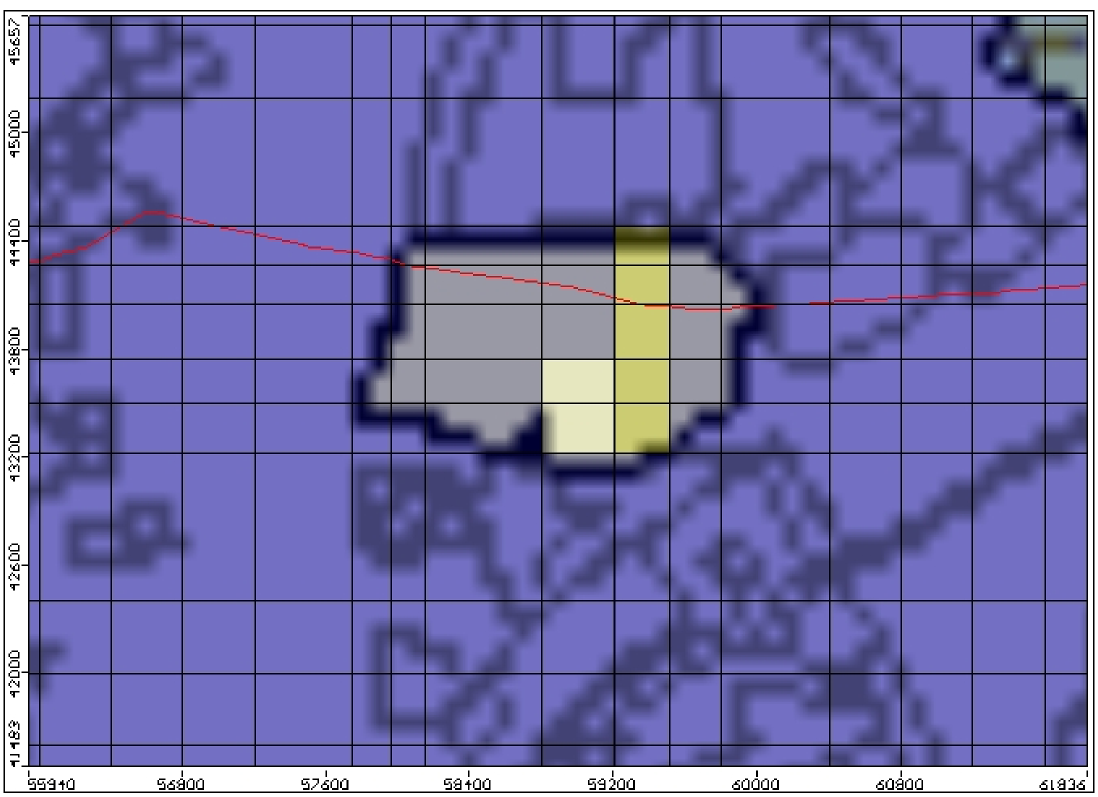

Fig. 4-10 – Transfer Station Modified Recharge Zones

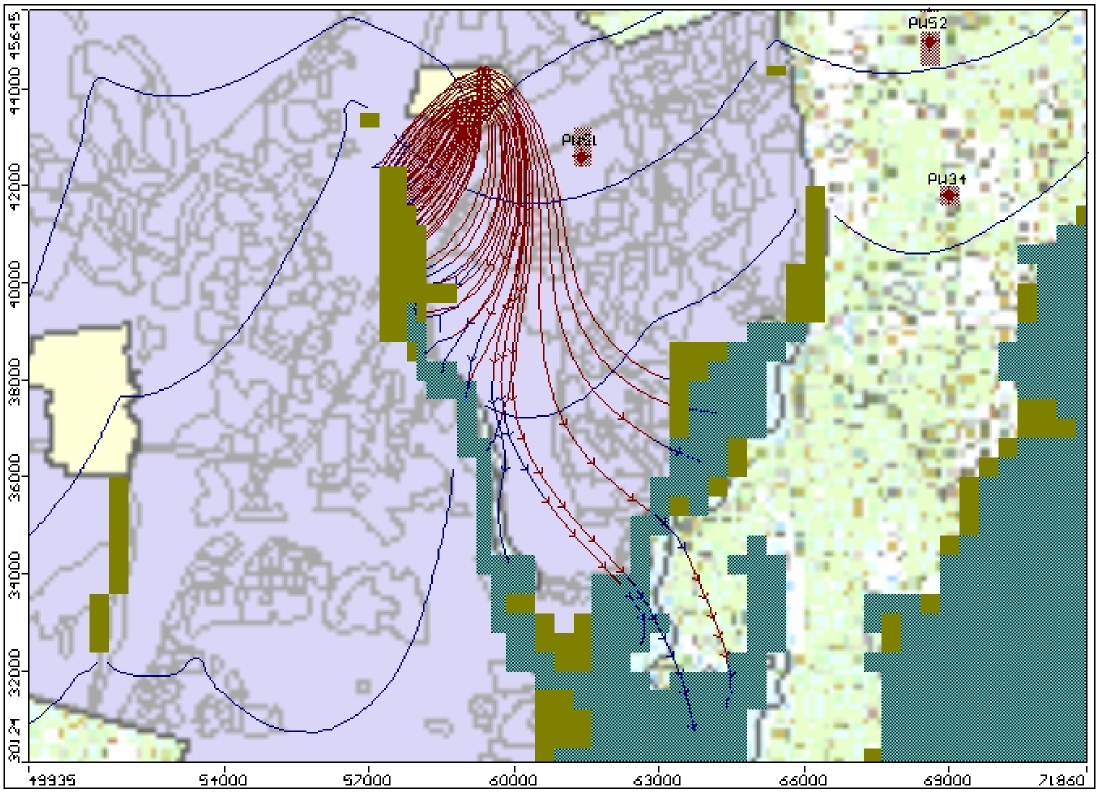

Fig. 4-11 – Tracks from Transfer Station Modified Recharge Zones

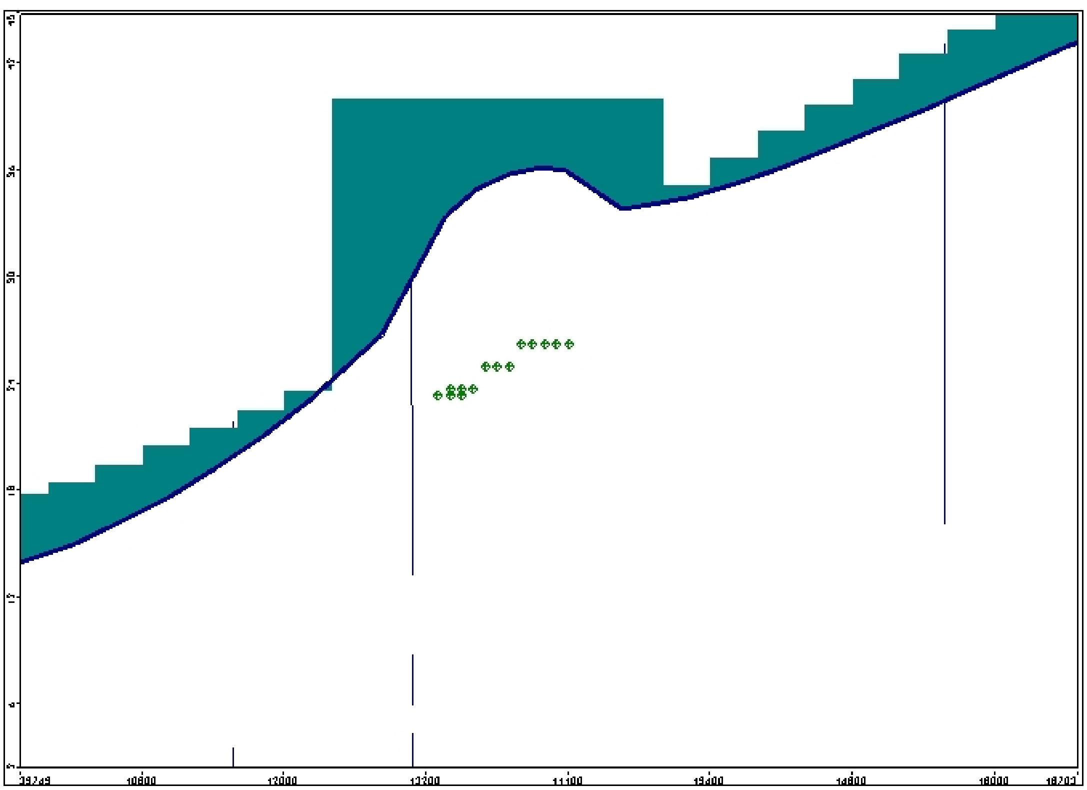

Fig. 4-12 – Groundwater Mounding at the Transfer Station Site

Fig. 4-12 is a cross-section of the discharge site modeled at peak conditions (2.675MGD, 37,900 mm/year at the parking lot and 75,100 mm/year at the forest area) to represent water table mounding. These results are discussed further in Section 4.3.3.

4.2.5 Site 5 – High School

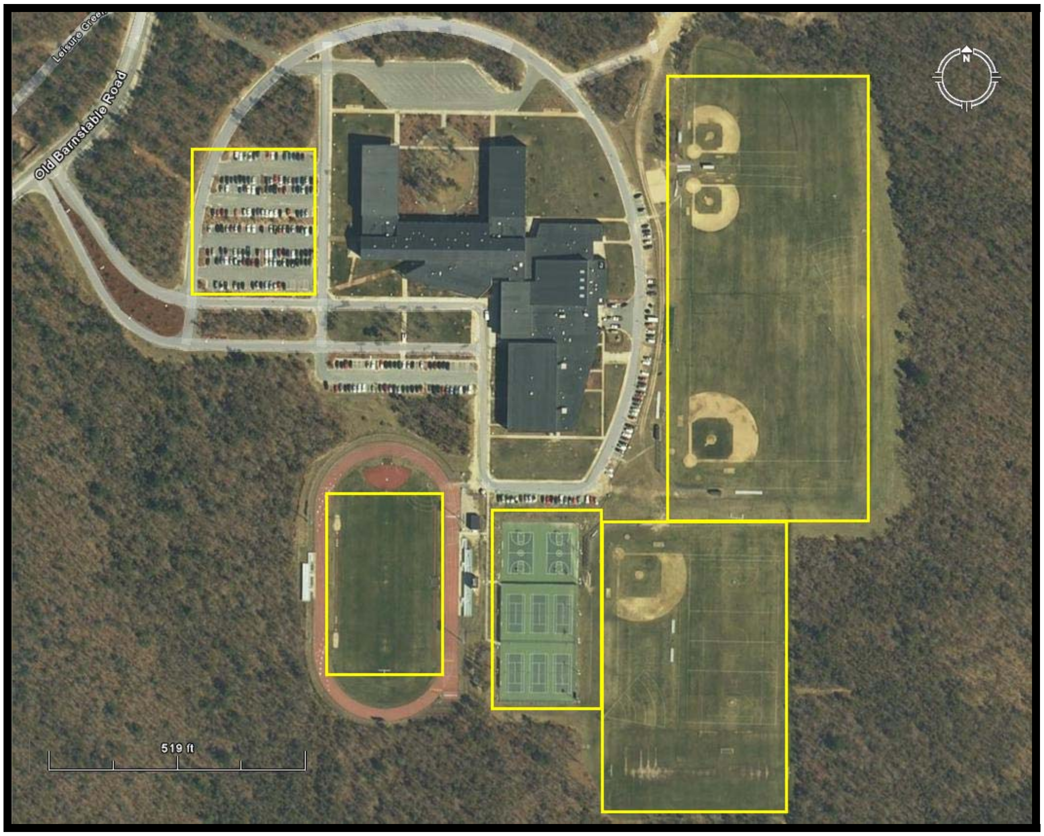

Fig. 4-13 – Site 5, High School

Site 5 is a high school located at the intersection of Old Barnstable Road and the Nathan Ellis Highway in central Mashpee. The entire property is currently used for various support facilities for the high school including buildings, sports fields, roads, and parking lots. Due to the expense of rebuilding the site, the only subsurface leaching trenches were considered. Potential discharge areas include the current baseball fields, tennis courts, track, and a parking lot. Fig. 4-13 shows infiltration area measured for the area at the High School. Areas are taken as five rectangles. The total calculated available area of the forest site is 795,900 sq ft and the available infiltration bed area (assuming 90% of available site area) is 716,310 sq ft. Using a discharge value of 2.5 GPD/ft^2, the potential discharge for the forest site is 1.79 MGD. However, embayment nitrogen loading must be calculated at the average load of 1.65MGD, which equates to recharge levels of

31386 mm/year for the High School. The modified recharge zone for the High School site is shown in Fig. 4-14. Fig. 4-15 shows the particle tracks from the modified recharge zone. 113 particles were used in 100ft x100ft spacing.

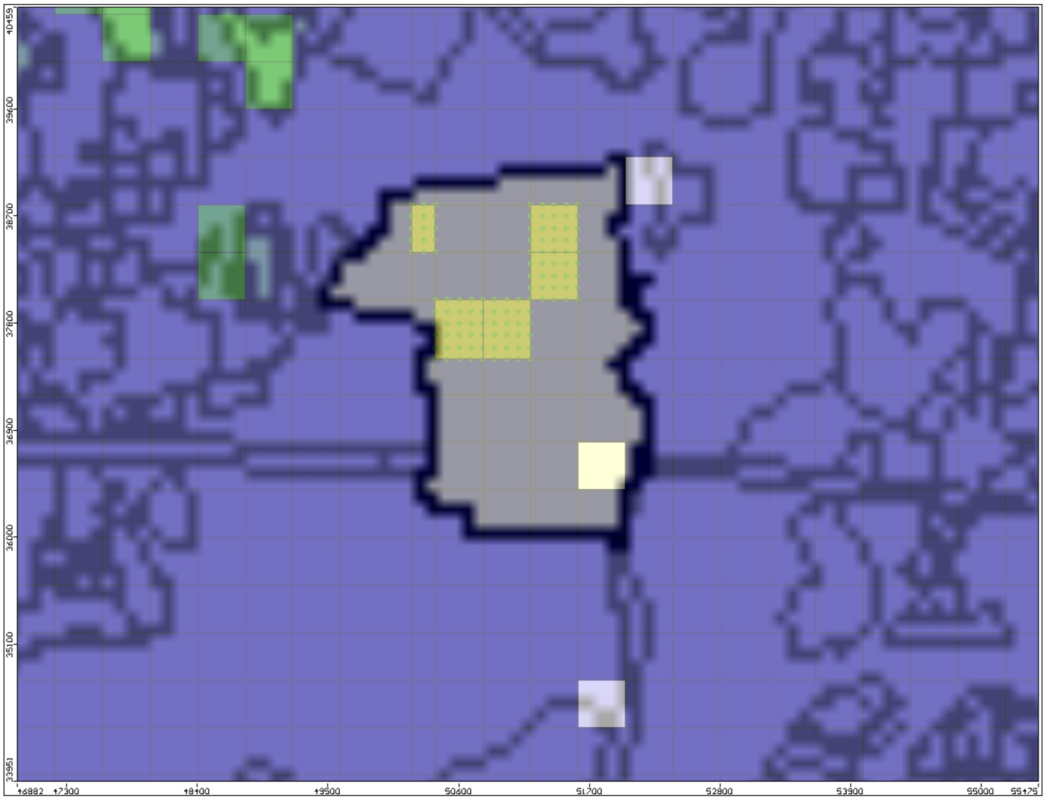

Fig. 4-14 – Modified Recharge Zone for the High School

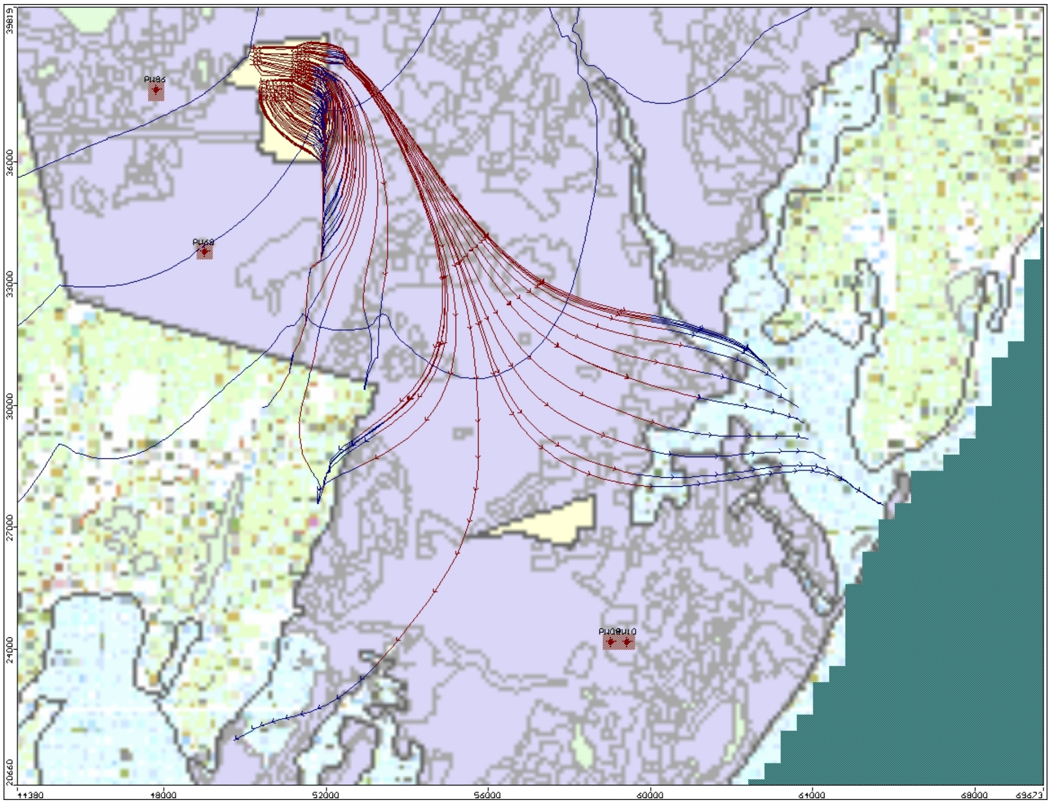

Fig. 4-15 – Particle Tracks from High School Modified Recharge Zone

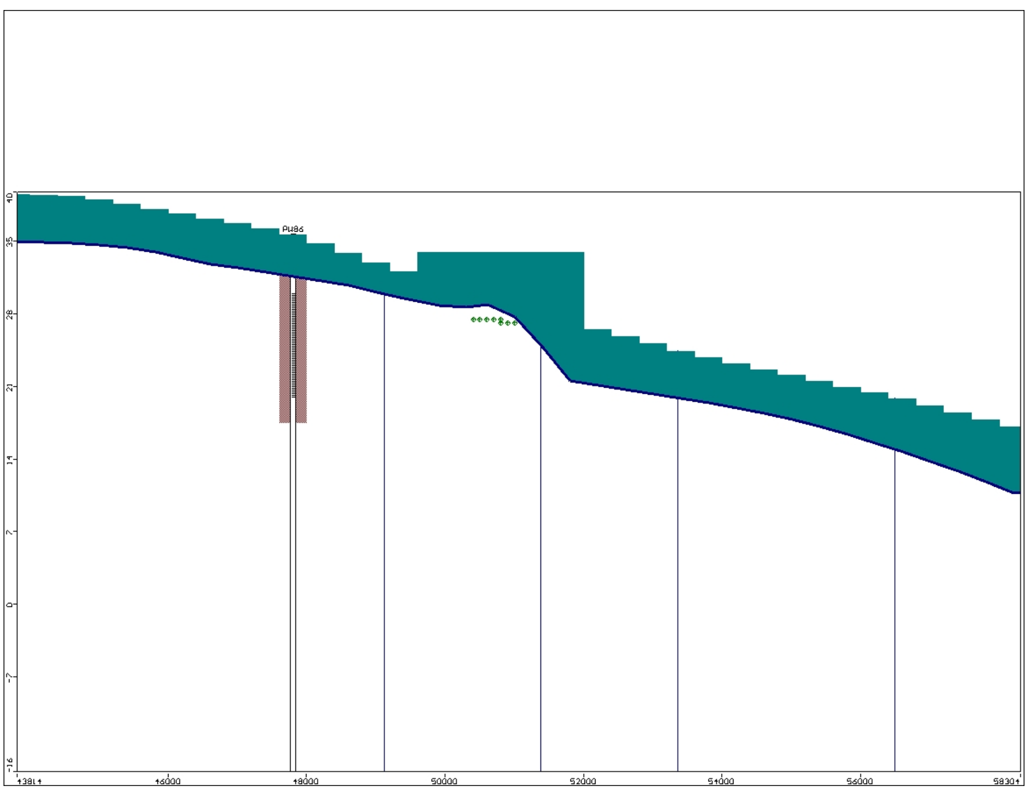

Fig. 4-16 – Groundwater Table Mounding at the High School Site

Fig. 4-16 is a cross-section of the discharge site modeled at peak conditions (1.79MGD, 34,005 mm/year) to represent water table mounding at the site. This figure is viewed from west to east, the reverse of other mounding results figures. These results are discussed further in Section 4.3.4

4.2.6 Site 6 – Keeter Property

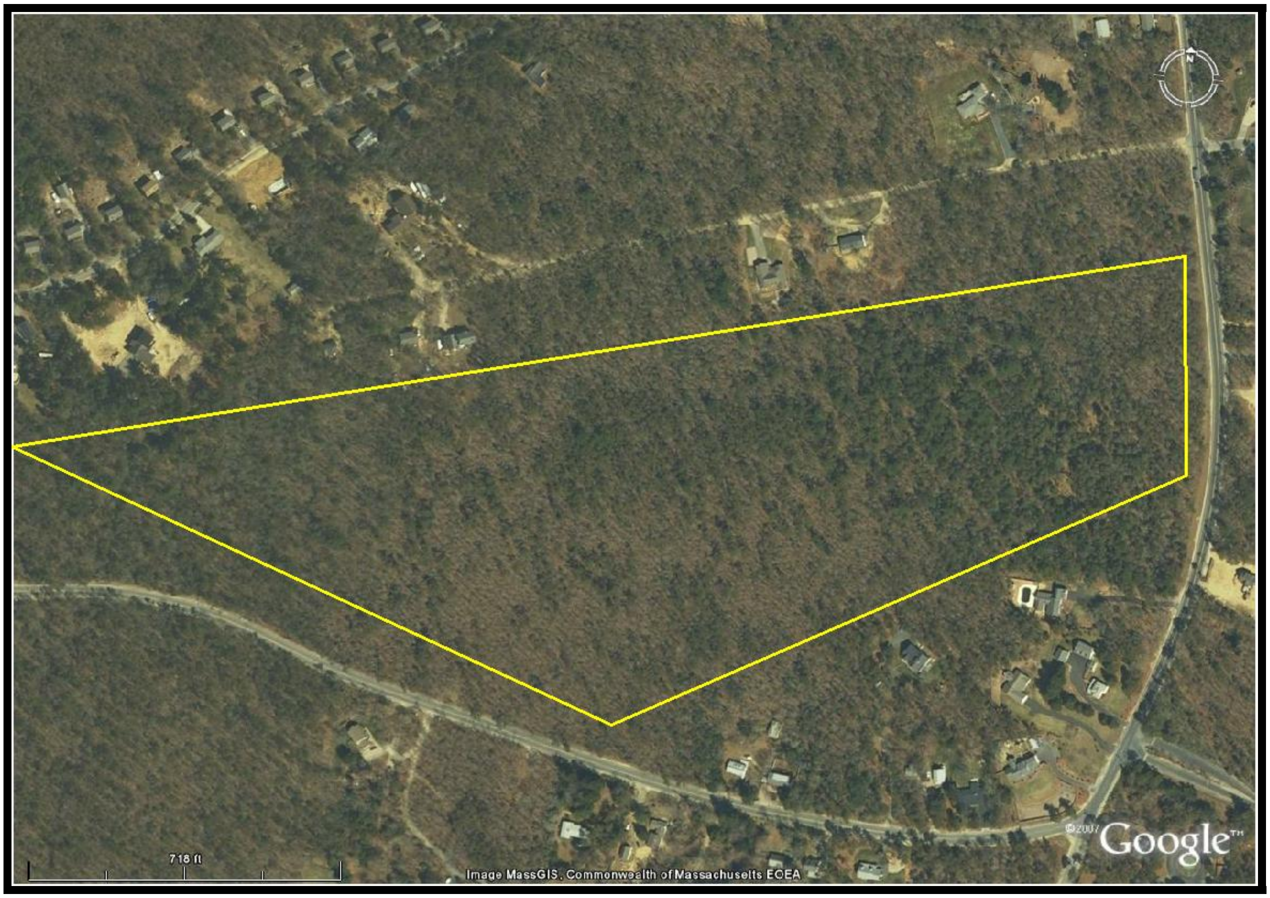

Fig. 4-17 – Site 6, Keeter Property

The Keeter Property is a large, forested, undeveloped area located at the intersection of Great Neck Road South and Red Brook Road in central Mashpee directly west of the Ockway Bay embayment of Popponesset Bay. It is currently owned by the Town of Mashpee and, because it is undeveloped, the entire site area is available for effluent discharge facilities. The town has tentative plans for a fire response station which would not affect discharge site operations. In this case, open bed infiltration basins were chosen. Similar to Ashumet Road, this site is large enough to handle the entire wastewater effluent load for the Town of Mashpee. Regardless, this site was initially modeled at 4.00 MGD, which was determined to be the maximum possible daily effluent load plus a safety factor plus the existing recharge from rainwater. The particle tracks shown in Fig. 4-19 were modeled with a recharge of 4.00MGD. Fig. 4-17 shows the infiltration area measured which corresponds to the modeled flow rate for the Keeter Property.

The total calculated available area of the forest site is 1,480,000 sq ft and the available infiltration bed area (assuming 90% of available site area) is 1,332,000 sq ft. Using a discharge value of 5 GPD/ft^2 the potential discharge for the forest site is 6.66 MGD which is far greater than the site will ever need to accommodate. However, estuarine nitrogen loading must be modeled at average conditions, 1.65MGD, which equates to a recharge value of 31,386 mm/year. The modified recharge zone for the Keeter Property site is shown in Fig. 4-18. Although the modified recharge zone does not correlate exactly to the proposed discharge area, the total amount of recharge is the same. Fig. 4-19 shows the particle tracks from the modified recharge zone, modeling potential effluent flows in the aquifer. 101 particles were used in 100ft x 100ft spacing. Fig. 4-20 is a cross-section of the discharge site modeled at peak discharge conditions (3.3MGD, 62,081.11 mm/year) to represent groundwater mounding. These results are discussed further in Section 4.3.5.

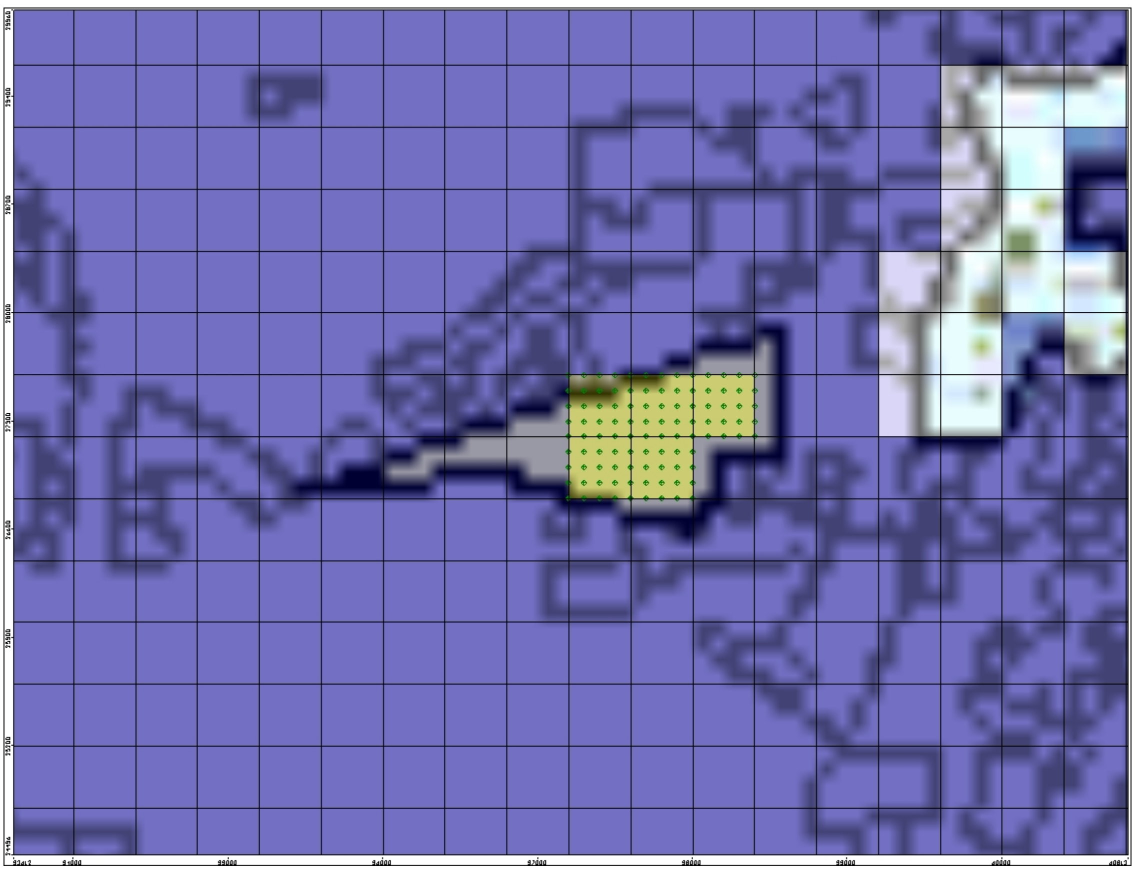

Fig. 4-18 – Keeter Property Modified Recharge Zone

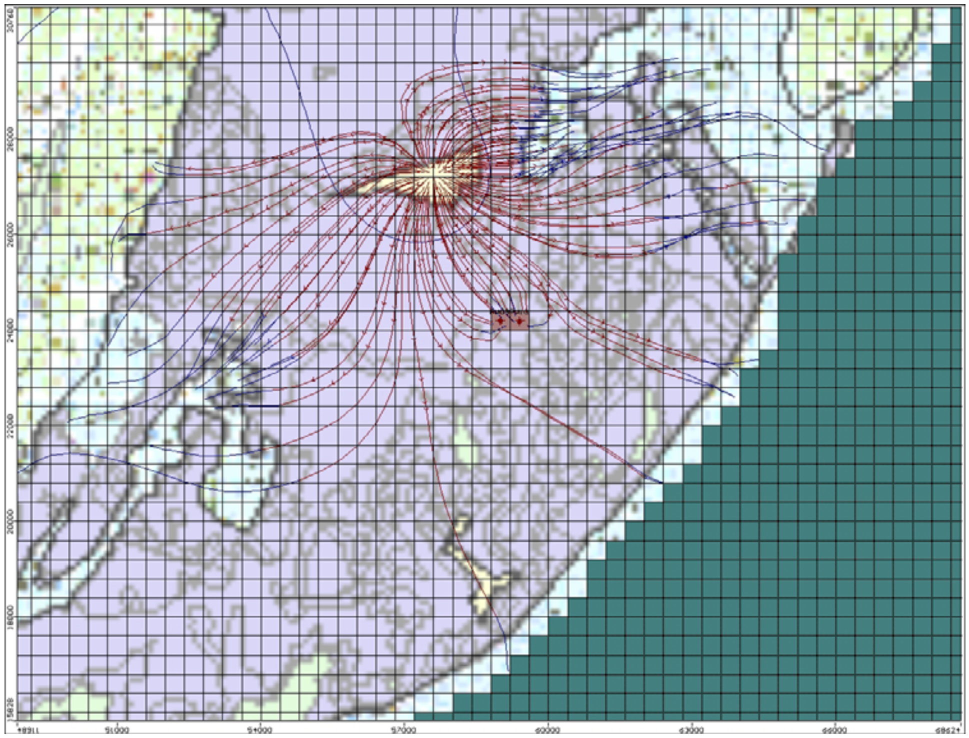

Fig. 4-19 – Particle Tracks from the Keeter Property Modified Recharge Zone

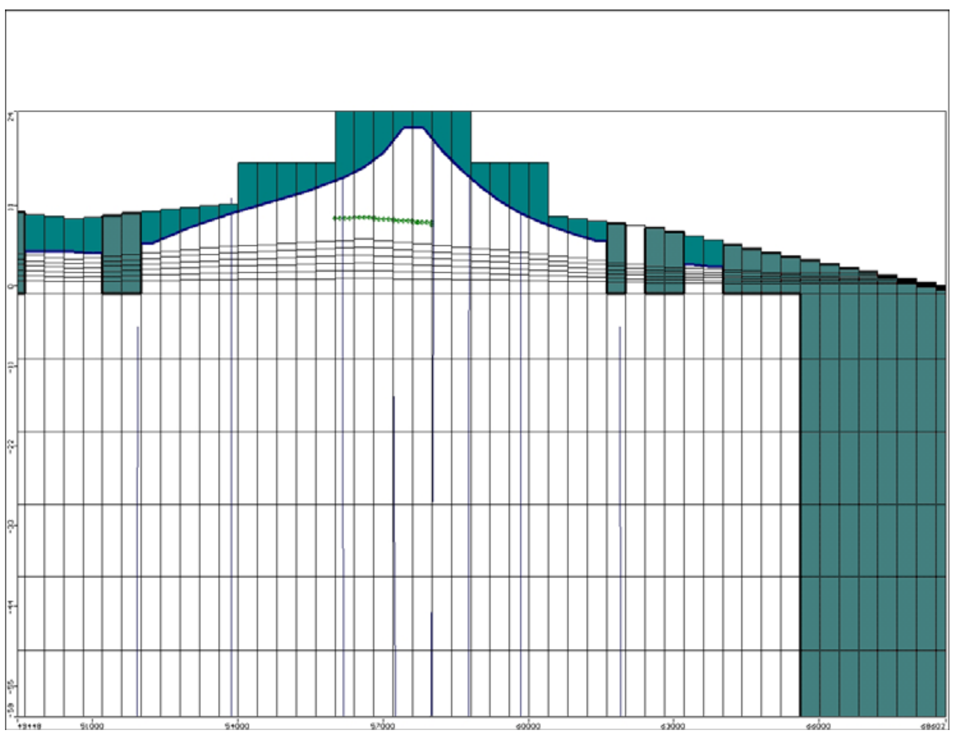

Fig. 4-20 – Groundwater Table Mounding at the Keeter Property

4.2.7 Site 7 – New Seabury Country Club

Fig. 4-21 – Site 7, New Seabury Country Club

The New Seabury Country Club Site is the only proposed discharge site that is not owned by the Town of Mashpee. It is a privately-owned property including a golf course and support infrastructure (buildings, roads, parking lots) along the southern coast of Mashpee, less than 1,000 feet from the coast at some points. The only areas available for effluent discharge are three holes of the golf course itself, which would only be available for subterranean leaching trenches. Fig. 4-21 shows infiltration area measured for the area at the New Seabury Country Club. The total calculated area of the site is 313,000 sq ft and the available infiltration area (assuming 90% of total) is 281,710 sq ft. Using a discharge value of 2.5 GPD/ft^2 the potential discharge for the New Seabury site is 0.70MGD, which equates to 17,069 mm/year.

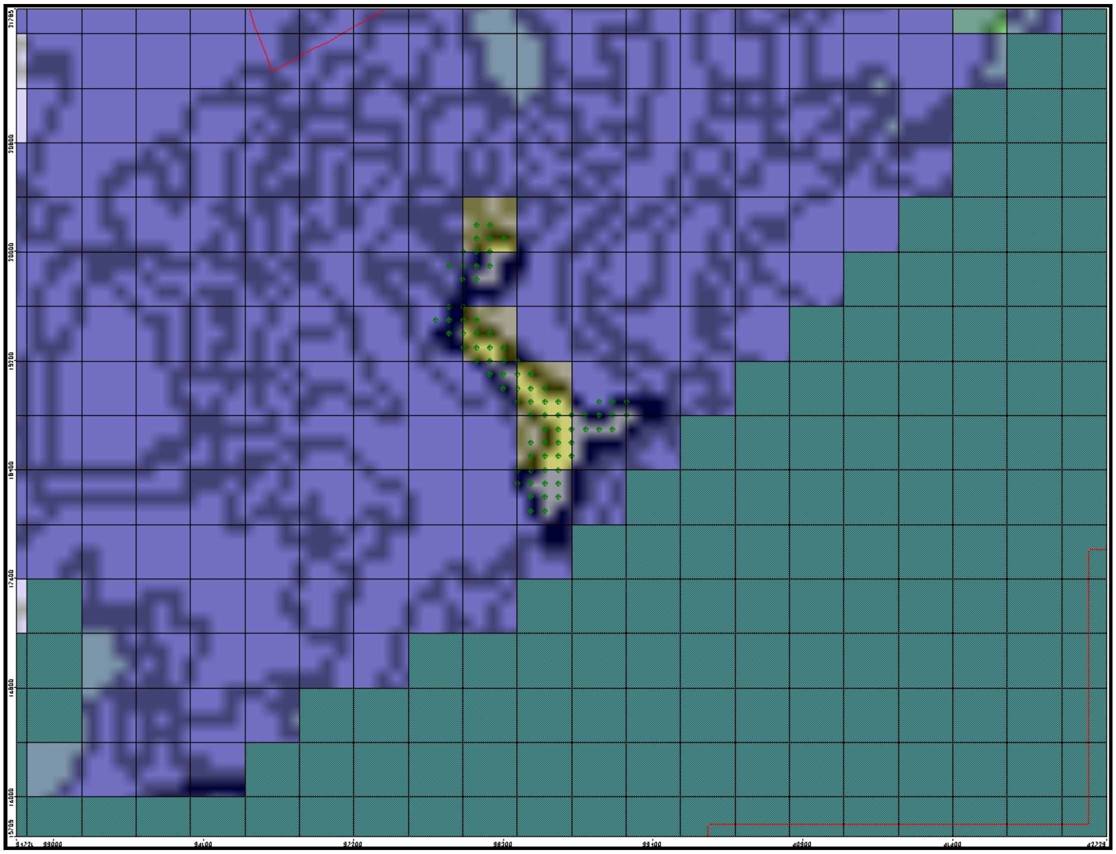
Fig. 4-22 – Modified Recharge Zone for the New Seabury Country Club Site

Fig. 4-22 shows the modified recharge zone for the New Seabury Country Club site. Unfortunately, the grid spacing could not be modified at the New Seabury site the same way it could be modified at the other sites to bring the rows and columns into alignment with the edges of the site because of the proximity to the ocean – moving the rows or columns would seriously disrupt the shape of the coastline. Instead, cells of regular size and spacing were selected that matched the general shape of the discharge site and the true discharge rate was modified to accommodate for the unrealistically large area. In this way, the correct amount of recharge was added to the model even though it was added in an inaccurately large area. It was determined that this inconsistency would have a negligible effect on model accuracy Fig. 4-23 shows the particle tracks from the modified recharge zone at the New Seabury site.

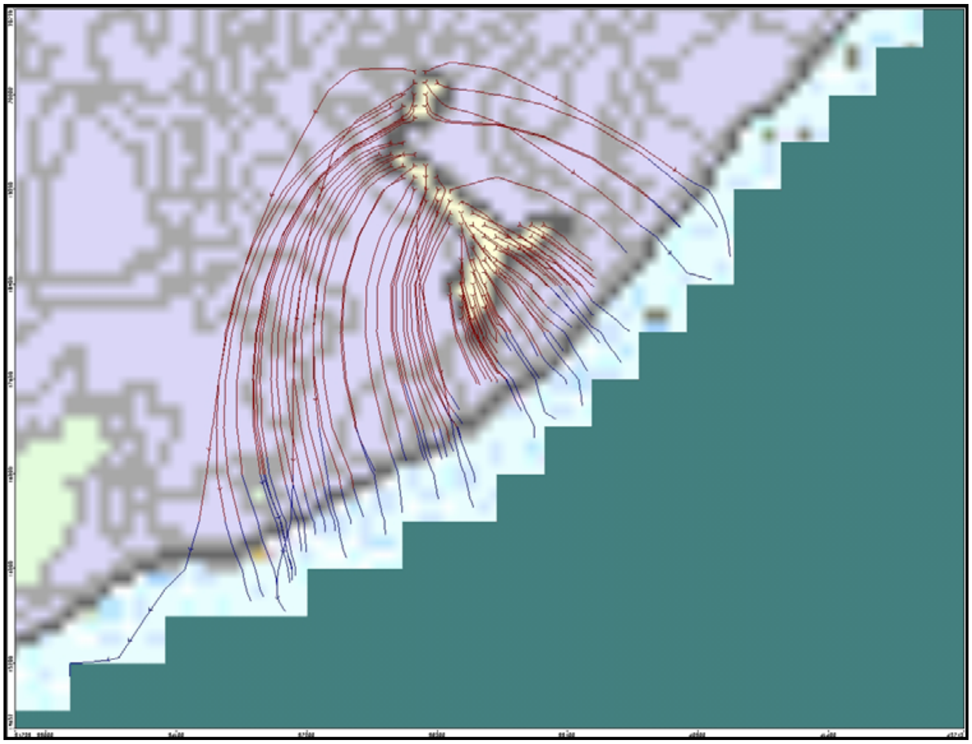

Fig. 4-23 – Particle Tracks from the New Seabury Modified Discharge Zone

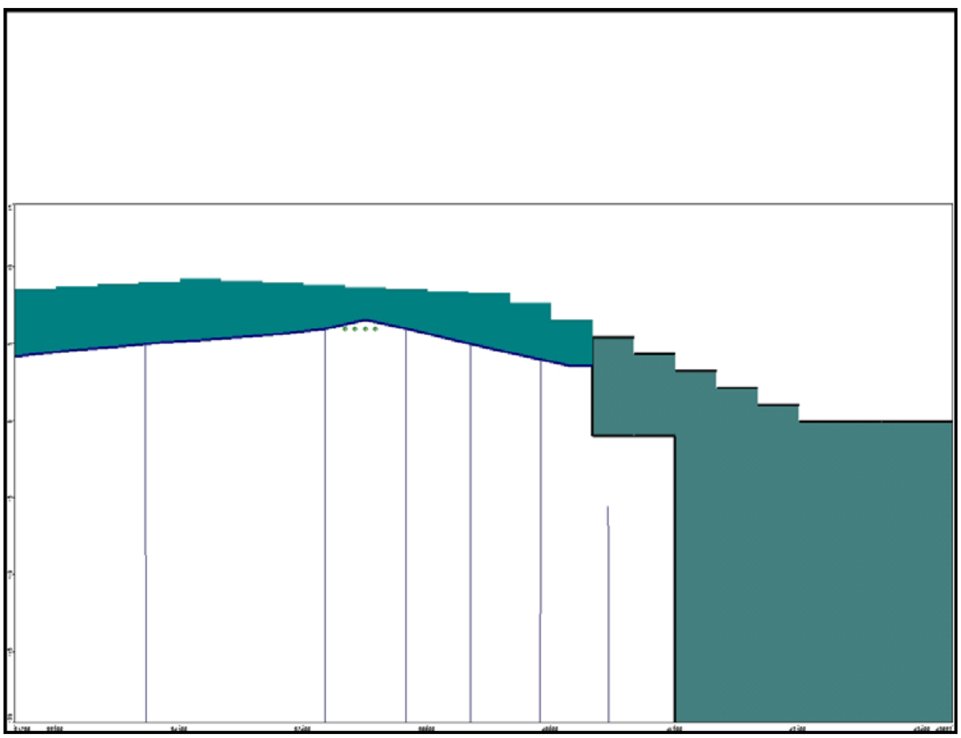

Fig. 4-24 – Groundwater Table Mounding at the New Seabury Site

Fig. 4-24 is a cross-section of the discharge site to represent water table mounding. These results are discussed further in Section 4.3.6.

4.3 *Nitrogen Loading Analysis*

This section contains the analysis of the results from the previous section, based on the criteria outlined in Section 4.1.

4.3.1 Site 1 – The Ball Fields

Table 4-1 – Embayment Loading Results for Site 1, the Ball Fields

Site Particle Tracks & Nitrogen Loads - From Average Flow				
	Particles	Percentage	Total Flow (MGD)	Nitrogen Load (KG/Day)
Total Site	134	100%	1.65	18.74
Nitrogen Concentration: 3 mg/L				
Waquoit (main) Bay	0	0%	0.00	0.00
Quashnet River	0	0%	0.00	0.00
Hamblin Pond	0	0%	0.00	0.00
Jehu Pond	0	0%	0.00	0.00
Little River	0	0%	0.00	0.00
Great River	0	0%	0.00	0.00
Total Waquoit System	0	0%	0.00	0.00
Popponesset (main) Bay	0	0%	0.00	0.00
Mashpee River	134	100%	1.65	18.74
Shoestring Bay	0	0%	0.00	0.00
Ockway Bay	0	0%	0.00	0.00
Popponesset Creek	0	0%	0.00	0.00
Pinquickset Cove	0	0%	0.00	0.00
Total Popponesset System	134	100%	1.65	18.74
Nantucket Bay	0	0%	0.00	0.00
Mashpee Supply Wells	0	0%	0.00	0.00
Total Embayment Loading	134	100%	1.65	18.74
Total Non-Embayment Loading	0	0%	0.00	0.00

The particle tracks displayed in Fig. 4-3 represent wastewater effluent traveling through the aquifer. Table 4-1 lists the destinations of these particles and the nitrogen loads they represent by embayment. For this site, the total number of affected embayments is 1; None of the effluent seeps into non-embayment locations (such as the open areas of Popponesset or Nantucket Bays), so the maximum embayment load by percentage is 100%. It is clear that the entire effluent load from this discharge site enters the Mashpee River toward the head. This will lead to a startling concentration of nitrogen in a part of the river which is not adequately flushed to handle such a load. Since there is no nitrogen entering any of the other embayments, every part of Popponesset Bay other than the Mashpee River will experience relief from nitrogen overloading and nutrient-related eutrophication, but this would be at the cost of the health of the Mashpee River. For this reason the Ball Fields site was determined to be a poor choice for wastewater effluent discharge.

Although there appear to be no extenuating political considerations at the Ball Fields Site and the land use issues are relatively straightforward, there is another factor which could prove critical. The border of the Massachusetts Military Reservation/Otis AFB is about a half a mile to the west. According to the Air Force Center for Engineering Excellence (AFCEE), between 1955 and 1970 pilots taking off from the base were instructed to dump fuel from their planes as part of a procedure to test emergency fuel release valves. The released fuel fell on a small patch of land just across the border from the Ball Fields and Ashumet Road sites. Although many constituents of jet fuel degrade over time, one additive – Ethylene Dibromide (EDB) – leached into the aquifer and has formed a large plume over the last 40 years. The AFCEE began efforts to remediate this plume in 1998, installing extraction wells, a treatment plant, and effluent return bubblers at the southern extent of the plume, as seen in Fig. 4-25.

The Ball Fields and Ashumet Road discharge sites are close enough to this plume that any increase in recharge will affect the plume itself and potentially disrupt remediation efforts already underway. In order to predict the effect of increased recharge on the plume, 31 particles were placed in a circular pattern near the current head of the plume just to the west of the Ashumet Road site and the model was run with a recharge value of 37900 mm/year in the Ball Fields Modified Recharge Zone. The output of the model run was recorded and the results are displayed in Fig. 4-26.

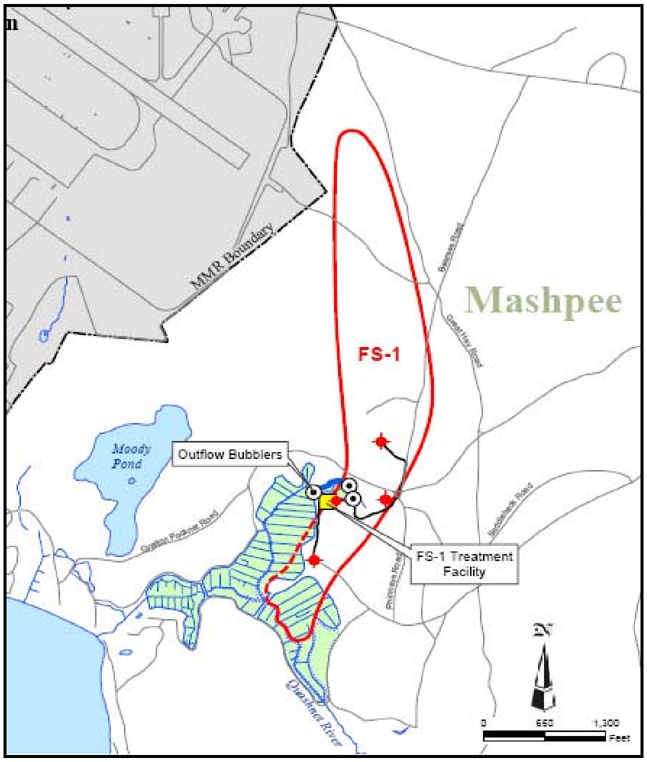

Fig. 4-25 – Current Plume Area

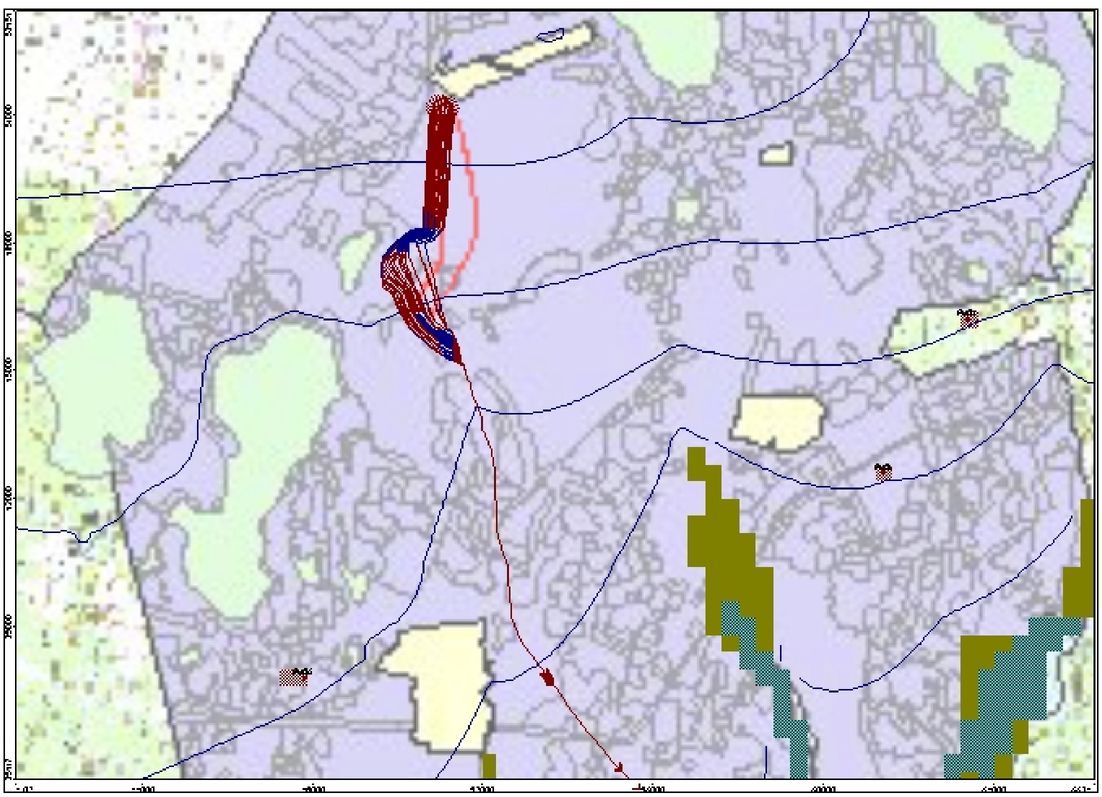

Fig. 4-26 – Modified Plume Path from Increased Recharge at the Ball Fields Site

The light red outline in Fig. 4-26 above delineates the current extent of the plume. It is obvious from the particle tracks that increased recharge at the Ball Fields site causes the plume conditions to change rather drastically. In this case it is apparent that the new path of the plume avoids the extraction wells constructed by the Air Force to remediate the plume, circumventing clean-up attempts. Although this alone does not preclude the Ball Fields site from being used, it is certainly another aspect to consider. If recharge is going to be increased at the Ball Fields site, it must be done in cooperation with the relevant Air Force authorities to assure that remediation efforts for the plume can continue. It is likely that this could substantially increase the cost of discharging effluent at the Ball Fields site.

4.3.2 Site 2 – Ashumet Road

Table 4-2 – Embayment Loading Results for Site 2, Ashumet Road

Site Particle Tracks & Nitrogen Loads - From Average Flow				
	Particles	Percentage	Total Flow (MGD)	Nitrogen Load (KG/Day)
Total Site	127	100%	1.65	18.74
Nitrogen Concentration: 3 mg/L				
Waquoit (main) Bay	0	0%	0.00	0.00
Quashnet River	90	71%	1.17	13.28
Hamblin Pond	0	0%	0.00	0.00
Jehu Pond	0	0%	0.00	0.00
Little River	0	0%	0.00	0.00
Great River	0	0%	0.00	0.00
Total Waquoit System	90	71%	1.17	13.28
Popponesset (main) Bay	0	0%	0.00	0.00
Mashpee River	37	29%	0.48	5.46
Shoestring Bay	0	0%	0.00	0.00
Ockway Bay	0	0%	0.00	0.00
Popponesset Creek	0	0%	0.00	0.00
Pinquickset Cove	0	0%	0.00	0.00
Total Popponesset System	37	29%	0.48	5.46
Nantucket Bay	0	0%	0.00	0.00
Mashpee Supply Wells	0	0%	0.00	0.00
Total Embayment Loading	127	100%	1.65	18.74
Total Non-Embayment Loading	0	0%	0.00	0.00

Table 4-2 lists the destinations of the particle tracks and the nitrogen loads they represent by embayment. For this site, the total number of embayments affected is two: the Quashnet River of Waquoit Bay, and the Mashpee River of Popponesset Bay. In this case, the maximum embayment load by percentage is 71%. This value is much lower than that at the Ball Fields and indeed the effect on each embayment will be less apparent than the previous modeling scenario in which the entire effluent load entered the Mashpee River. However, the Ashumet Road site is bigger and is operating at a higher effluent capacity, and so the total nitrogen load entering the Mashpee River during peak season is still significant.

Overall, substantial effluent discharge at the Ashumet Road site would result in significantly reduced nitrogen loads in every embayment of Waquoit and Popponesset Bays except the Quashnet and Mashpee Rivers, respectively, where nitrogen loading would continue to pose a problem. Even so the Ashumet Road site remains a potentially very useful discharge site because it is currently not being used or under any development, which would reduce construction costs and avoid any political complications.

However, the Ashumet Road site is the closest of any site to the Massachusetts Military Reserve/Otis AFB. Specifically, the site is about a third of a mile away from the source area for the ethylene dibromide/jet fuel contaminant plume described above. The plume currently flows directly past the southern end of the Ashumet Road site, and any increase in recharge at the site would affect the path of the contaminant plume. As described above, this could have a potentially serious impact on the efforts already underway to remediate the plume.

In order to predict the effect of effluent discharge at Ashumet Road on the contaminant plume, an additional model was constructed including the Ashumet Road Modified Recharge Zone (using the increased recharge rate of 42,786 mm/year to accommodate for peak season) and 31 particles placed in a circular pattern at the head of the current contaminant plume area. The model was run and the output was recorded. The particle tracks from this model run (shown in Fig. 4-27) represent the path that the contaminant plume would take if the recharge at the Ashumet Road site was increased to meet demand.

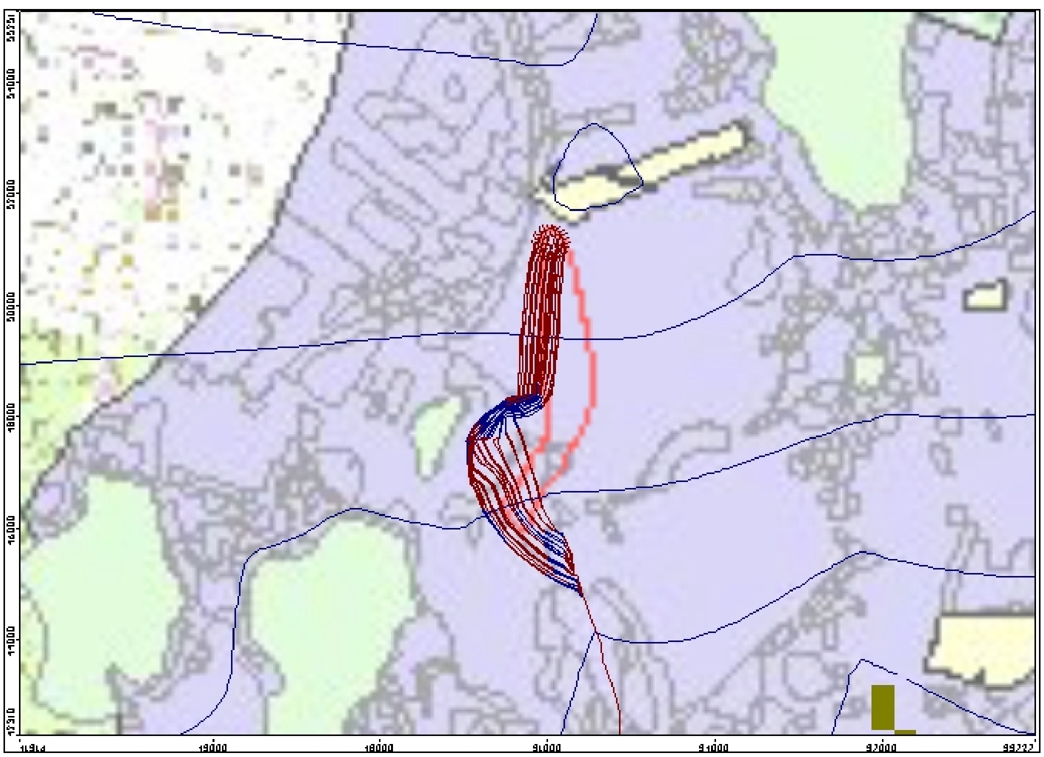

Fig. 4-27 – Modified Plume Path from Increased Recharge at the Ashumet Road Site

Again, in this situation the light red outline represents the current plume area. It is clear from these particle tracks that the plume path would be modified somewhat by increased recharge at the Ashumet Road site. Before any discharge activity can be conducted at the site, the AFCEE must be contacted to determine what changes would have to be made in remediation efforts at the plume. Although this does not disqualify Ashumet Road, it would likely increase the expense and difficulty of discharging any substantial amount of wastewater effluent there.

4.3.3 Site 4 – the Transfer Station

Table 4-3 – Embayment Loading Results for Site 4, the Transfer Station

	Particles	Percentage	Total Flow (MGD)	Nitrogen Load (KG/Day)
Total Site 4	70	100%	1.65	18.74
Nitrogen Concentration: 3 mg/L				
Quashnet River	0	0%	0.00	0.00
Hamblin Pond	0	0%	0.00	0.00
Jehu Pond	0	0%	0.00	0.00
Little River	0	0%	0.00	0.00
Great River	0	0%	0.00	0.00
Total Waquoit System	0	0%	0.00	0.00
Mashpee River	64	91%	1.51	17.13
Shoestring Bay	4	6%	0.09	1.07
Ockway Bay	0	0%	0.00	0.00
Popponesset Creek	0	0%	0.00	0.00
Pinquickset Cove	0	0%	0.00	0.00
Total Popponesset System	68	97%	1.60	18.20
Waquoit (main) Bay	0	0%	0.00	0.00
Popponesset (main) Bay	2	3%	0.05	0.54
Nantucket Bay	0	0%	0.00	0.00
Mashpee Supply Wells	0	0%	0.00	0.00
Total Embayment Loading	68	97%	1.60	18.20
Total Non-Embayment Loading	2	3%	0.05	0.54

Particles from the Transfer Station, representing wastewater effluent in the aquifer, terminate eventually at two major locations- the head of the Mashpee River and the central part of Shoestring Bay. The total number of embayments affected is two. Small amounts of effluent, represented by a single particle, terminate in the center of Popponesset Bay (traveling down through the aquifer and resurfacing through the bottom of the bay) and the mouth of Shoestring Bay (same conditions). Although these two flows are depicted by discrete particles they actually represent a gradient in effluent concentration between the head of the Mashpee River (very dense), the middle of Shoestring Bay (somewhat dense) and Popponesset Bay itself (very sparse). Based on particle tracks, the maximum embayment loading by percentage for the Transfer Station is 91%, in the Mashpee River. Although this site does drain to two embayments and the open bay area of Popponesset Bay, a sufficiently high percentage drains to a single destination that the site is still a very poor candidate for wastewater effluent discharge. As with the two sites above, if this site is chosen for effluent discharge special attention should be paid to monitoring nitrogen concentrations in the Mashpee River.

4.3.4 Site 5 – the High School

Table 4-4 – Embayment Loading Results for Site 5, the High School

Site 5 Particle Tracks & Nitrogen Loads - From Average Flow				
	Particles	Percentage	Total Flow (MGD)	Nitrogen Load (KG/Day)
Total Site 5	113	100%	1.65	18.74
Nitrogen Concentration: 3 mg/L				
Quashnet River	95	84%	1.39	15.75
Hamblin Pond	6	5%	0.09	0.99
Jehu Pond	0	0%	0.00	0.00
Little River	0	0%	0.00	0.00
Great River	0	0%	0.00	0.00
Total Waquoit System	101	89%	1.47	16.75
Mashpee River	0	0%	0.00	0.00
Shoestring Bay	0	0%	0.00	0.00
Ockway Bay	0	0%	0.00	0.00
Popponesset Creek	0	0%	0.00	0.00
Pinquickset Cove	0	0%	0.00	0.00
Total Popponesset System	0	0%	0.00	0.00
Waquoit (main) Bay	0	0%	0.00	0.00
Popponesset (main) Bay	12	11%	0.18	1.99
Nantucket Bay	0	0%	0.00	0.00
Mashpee Supply Wells	0	0%	0.00	0.00
Total Embayment Loading	101	89%	1.47	16.75
Total Non-Embayment Loading	12	11%	0.18	1.99

It is apparent from the particle tracking results that effluent flow from the high school takes two major paths to two separate destinations. The total number of embayments affected is two, Hamblin Pond and the Quashnet River. Additionally, some effluent travels deep into the ground and seeps into the center of Popponesset Bay. The maximum embayment load by percentage is 84% in the Quashnet River. Having a great deal of effluent flowing through the tributary streams to the Quashnet River is not an optimal situation because, although it is possible that the nitrogen load will be attenuated somewhat by biological activity in the stream, it may also have adverse affects on the health of tributary streams. Additionally, the stream draws in effluent from the surrounding area and concentrates it at the head of the Quashnet River, where nitrogen concentrations would soon grow to even higher than current levels. Although nitrogen concentrations would decrease in all other embayments, special attention would have to be paid to indications of estuarine health (such as dissolved oxygen concentrations and algae growth) over the next several years.

Some of the effluent from the High School site flows east across the divide between the Popponesset and Waquoit Bay watersheds and into Popponesset Bay. This effluent flows down very deep into the ground before percolating back up under the bay itself and leaching through the surface of the bay into the open area at the center. This area of the bay exhibits a much higher tidal volume and more tidal flushing, which results in lower residence times for water in the bay and, by extension, lower concentrations of nitrogen from sources in the aquifer. Any effluent which winds up in the central part of Popponesset Bay would be almost immediately flushed out into Nantucket Sound. This is, in fact, an ideal solution, and the effectiveness of the High School as a discharge site is tempered only by the fact that only the very northern reaches of the site drain into Popponesset Bay and there is no area on the northern part of the site which is available for effluent discharge, with the single exception of the northernmost baseball diamond which has too small an area to be of much use.

Overall, because the High School site discharges almost entirely into one embayment (max. load 84%) and only a small corner drains into Popponesset Bay, it is still a poor choice for effluent discharge.

4.3.5 Site 6 – Keeter Property

Table 4-5 – Embayment Loading Results for Site 6, the Keeter Property

Site 6 Particle Tracks & Nitrogen Loads - From Average Flow				
	Particles	Percentage	Total Flow (MGD)	Nitrogen Load (KG/Day)
Total Site 6	101	100%	1.65	18.74
Nitrogen Concentration: 3 mg/L				
Quashnet River	0	0%	0.00	0.00
Hamblin Pond	8	8%	0.13	1.48
Jehu Pond	14	14%	0.23	2.60
Little River	0	0%	0.00	0.00
Great River	0	0%	0.00	0.00
Total Waquoit System	22	22%	0.36	4.08
Mashpee River	0	0%	0.00	0.00
Shoestring Bay	0	0%	0.00	0.00
Ockway Bay	36	36%	0.59	6.68
Popponesset Creek	6	6%	0.10	1.11
Pinquickset Cove	0	0%	0.00	0.00
Total Popponesset System	42	42%	0.69	7.79
Waquoit (main) Bay	1	1%	0.02	0.19
Popponesset (main) Bay	14	14%	0.23	2.60
Nantucket Bay	7	7%	0.11	1.30
Mashpee Supply Wells	15	15%	0.25	2.78
Total Embayment Loading	64	63%	1.05	11.87
Total Non-Embayment Loading	37	37%	0.60	6.86

The Keeter Property is unique among all of the sites described in this report because of its location adjacent to embayments of both Popponesset and Waquoit Bays. Particle tracking results show that wastewater effluent from the Keeter property travels outward in all directions and enters Ockway Bay, Popponesset Creek, Popponesset Bay, Hamblin and Jehu Ponds and Little and Great Rivers as well as leaching directly into Nantucket Sound directly to the south. Due to its centralized location, effluent discharged at the Keeter Property is distributed between embayments in a manner unparalleled by any other potential discharge site. Additionally, any effluent which leaches directly into the Atlantic Ocean will be immediately swept away by tidal flushing action. This diversity of drainage destinations is represented numerically by the total number of embayments affected – five including Popponesset and Waquoit Bays – and the maximum embayment load by percentage – 36% in Ockway Bay. This loading factor is significantly lower at the Keeter Property than at any other site.

There is one potential complication. A drinking water supply well for the Town of Mashpee is located just south of the site, visible in Fig. 4-19 above as a red square with a red dot in the center. This well supplies drinking water to a portion of the town and operates year-round, and if dumping effluent at this site causes undesirable contaminant concentrations at the well (specifically, concentrations in excess of Massachusetts standards for safe drinking water) the Keeter Property will have to be abandoned as a potential discharge site. Fortunately, the Massachusetts Department of Environmental Protection standards for drinking water specify in the section titled "Maximum Contaminant Levels for Inorganic Chemicals" that the MCL for total nitrogen is 10 mg/L (Massachusetts Department of Environmental Protection 22.06). Again, the wastewater effluent discharged at the Keeter property will be treated to a concentration of 3 mg/L, less than a third of the MCL for nitrogen. Although the concentration of total nitrogen in the well should remain substantially below the MCL for the duration of discharge activity at the Keeter Property site, it would be prudent to monitor nitrogen levels in the well in order to reassure the public about the safety of their drinking water supply.

4.3.6 Site 7 – New Seabury

Table 4-6 – Embayment Loading Results for Site 7, New Seabury

Site 7 Particle Tracks & Nitrogen Loads - From Average Flow				
	Particles	Percentage	Total Flow (MGD)	Nitrogen Load (KG/Day)
Total Site 7	82	100%	0.70	26.50
Nitrogen Concentration: 10 mg/L				
Quashnet River	0	0%	0.00	0.00
Hamblin Pond	0	0%	0.00	0.00
Jehu Pond	0	0%	0.00	0.00
Little River	0	0%	0.00	0.00
Great River	0	0%	0.00	0.00
Total Waquoit System	0	0%	0.00	0.00
Mashpee River	0	0%	0.00	0.00
Shoestring Bay	0	0%	0.00	0.00
Ockway Bay	0	0%	0.00	0.00
Popponesset Creek	0	0%	0.00	0.00
Pinquickset Cove	0	0%	0.00	0.00
Total Popponesset System	0	0%	0.00	0.00
Waquoit (main) Bay	0	0%	0.00	0.00
Popponesset (main) Bay	0	0%	0.00	0.00
Nantucket Bay	82	100%	0.70	26.50
Mashpee Supply Wells	0	0%	0.00	0.00
Total Embayment Loading	0	0%	0.00	0.00
Total Non-Embayment Loading	82	100%	0.70	26.50

All of the particle tracks (and therefore all of the discharged effluent) from the New Seabury site eventually seep into Nantucket Sound along a several-mile stretch of coastline. This is actually a very desirable result- this effluent will leach into the ocean and disappear, carried away from the cape by tidal flushing. Residence times along exposed beaches are completely negligible, and nitrogen concentrations along the coast are comparable to the concentration throughout the ocean. Any nitrogen released into the Atlantic Ocean will no longer contribute to eutrophication in any of the coastal zones or estuaries in the Town of Mashpee or anywhere on Cape Cod. In terms used elsewhere in this report, the number of embayments affected is zero and the maximum embayment loading by percentage is 0%. In terms of nitrogen loading in embayments, New Seabury is the perfect site.

Unfortunately, despite the fact that this site offers the best possible option for discharging effluent, it is not large enough to accommodate the wastewater effluent for the entire Town of Mashpee – a liberal estimate of the capacity of this site 1.0 MGD, whereas the town produces 1.65MGD on average, with twice as much during the peak season. Any comprehensive discharge scenario that includes the New Seabury site would have to include at least one additional discharge site. The biggest complication at the New Seabury site may be politics. New Seabury is currently an active country club and the golf course in question is currently in use. Installing effluent discharge facilities at the club would require that the town get permission to use the site, possibly in the form of an easement. Then the proposed three holes would be shut down for several months during leach field construction as well as reconstruction of the course on top. Even if permission is secured, the course is surrounded by residential development, and local residents may oppose the installation of discharge facilities in their neighborhood. It is worth noting that, even though the New Seabury site represents the best possible discharge solution, it may not be possible to install facilities there without a resolution to these possible issues.

4.4 *Mounding Analysis*

The last consideration at the discharge sites is the potential that the increase in the groundwater mound would result in undesirable effects on the surrounding area: if the water table mound rises higher than basements, foundations, or other structures in the area there could be potentially serious consequences that would disqualify any affected site as a useful discharge area. The initial model runs for most sites did not conclusively show the behavior of water table mounding because the mound rose higher than the maximum water-table elevation allowed in the model. To address this concern, the maximum water-table elevation was raised to 70 or 75 feet (40 feet for the Keeter Property) in the vicinity of the discharge site and the model was run again. The results are shown in Table 4-7.

Table 4-7 – Mounding Results for all discharge sites

Mounding Analysis - From Peak Flow	Site 1	Site 2	Site 4	Site 5	Site 6	Site 7
Minimum Ground Elevation	105	89	62	60	26	33
Maximum Initial Water Table Elevation	59	55	26	30	8	4.5
Peak Mounding Water Table Elevation	60.5	60	35	30	20	6.6
Water Table Increase	1.5	5	9	0	12	2.1
Depth to Mounded Water Table	44.5	29	27	30	6	26.4

According to MassDEP regulations governing discharge facilities for small WWTFs, the minimum depth to mounded water table is 6 feet at the discharge site (Massachusetts Department of Environmental Protection). As demonstrated in the table above, the groundwater table does not mound enough to be a concern at any site except the Keeter Property. Mounding was modeled at peak discharge for each site, and it was concluded that (with the exception of Keeter Property) there will be no problems with mounding at any site at any point.

Unfortunately, there is not adequate clearance for the Keeter Property site. If the modeled effluent volume was actually discharged, nearby homes and structures could be substantially affected by the increased groundwater table. This could lead to treated effluent leaking into the basements of nearby homes, or permanent damage to the substructure of roads in the area. Thus, if the Keeter Property is to be used for effluent discharge, it must be used for a lower volume than the volume used in the model runs described in this section. All of the recommended discharge scenarios, described further in Section 5.1 below, include substantially reduced recharge at the Keeter Property. Further mounding results for the proposed discharge scenarios will demonstrate increased safety margins and adequate clearance, and can be found in the sections below.

4.5 Cost Analysis

As with any project, the overall cost is a critical factor which must be taken into consideration. Building infrastructure is an expensive undertaking, and is compounded by the more stringent requirements which must be maintained for the sake of preserving the environment in order to sustain economical and ecological health. This section compares the estimated costs of the two infiltration technologies under consideration for use in Mashpee. The RS Means 1999 Building Construction Cost Data manual was used to establish average costs for the various items necessary. Adjustments were made to the price to account for 3% yearly inflation, and a local adjustment factor of 1.07 was used. These costs are based on several assumptions, stated below, the modification of which would require revisions to the given prices. Subsurface leaching trenches are considered first, with open bed infiltration basins following.

4.5.1 Subsurface Leaching Trenches

To estimate the cost of subsurface leaching trenches, first the amount of area necessary to accommodate the entire design peak flow of 3.3 MGD was calculated. The standard design capacity of 1 sq ft of leaching trench is 2.5 GPD. Next, the major items necessary to build a standard leaching trench were listed (see Table 4-8), and the quantity of each necessary for the design area were calculated. Then the Building Construction Cost Data book was referenced to determine standard prices for each item, and the price was adjusted for construction in Mashpee in 2009.

Table 4-8 – Subsurface Leaching Trenches Construction Cost

Items	**Unit**	**Quantity**	**Unit Cost**	**1999 Cost**	**Local Cost**
Leaching Field area	SF	1,320,000			
Leaching Field Chambers (20'x4')	EA	16,500	$1,350.00	$22,275,000	$32,031,239
Excavation (4' wide trenches)	LF	330,000	$3.41	$1,125,300	$1,618,171
Backfill	CY	98,000	$1.32	$129,360	$186,018
Total	SF	1,320,000	**$25.63**	(Local unit)	**$33,835,428**
Direct Comparison Price	Gal/Day	1	**$10.25**	Price for 1 GPD capacity	

Clearly, the most expensive item is to furnish and install the leaching field chambers. It is assumed that the chamber unit cost does not include excavation and backfill, but does not require any additional piping within the network of trenches. Additionally, since the existing and final surface features present at potential leaching field locations varies from site to site, the cost of clearing and rebuilding is not included within the price of the leaching field. For the proposed design in Chapter 5, this cost is considered separately. The conclusion of these calculations is that, to build a typical subsurface leaching field in Mashpee, MA in 2009, the cost is roughly $10.25 for each gallon per day capacity required. If this technology were to be used for the entire peak flow capacity, the overall cost would be about $34 million.

4.5.2 Open Bed Infiltration Basins

Similarly to subsurface leaching trenches, the entire peak flow design capacity of 3.3 MGD was used in this calculation. The same method for estimating the cost was followed: listing the necessary items; calculating quantities needed; looking up unit costs; and extrapolating the local 2009 cost.

Table 4-9 – Open Bed Infiltration Basin Construction Cost

Items	Unit	Quantity	Unit Cost	1999 Cost	Local Cost
Open Bed Total Area (5gal/day/SF)	CY	660,000			
Clearing & Grubbing	ACRE	15	$6,925.00	$105,260	$151,363
Excavation to subgrade	CY	49,000	$3.45	$169,050	$243,092
On-site Piping (18" D avg)	LF	4,000	$63.00	$252,000	$362,374
3.5'x2' Earth Berm	LF	11,000	$50.00	$550,000	$790,895
2' sand bed	CY	49,000	$17.55	$859,950	$1,236,600
5' Perimeter Fence	LF	6,000	$13.00	$78,000	$112,163
20'x15'x1' CIP concrete splash pads	EA	17	$1,479.70	$25,155	$36,173
1' Splash pad compacted gravel base	CY	220	$26.50	$5,830	$8,383
Total	SF	660,000	**$4.46**	(Local unit)	**$2,941,043**
Direct Comparison Price	Gal/Day	1	**$0.89**	Price for 1 GPD capacity	

Unlike with leaching trenches, it is generally safe to assume that any land within Mashpee which is potentially usable for open bed infiltration basins is currently wooded. Hense the price for clearing and grubbing with trees up to 12 inches in diameter was included. It was assumed that 17 square beds each 200 feet on a side would be used. Several beds were assumed to be adjacent, sharing an earth berm with a fence only around the perimeter. Depending on the actual configuration, these quantities are subject to change. Additionally, it should be noted that there was no listed price for anything similar to an earth berm, so engineering judgment was used to estimate a conservative value, based on other values for backfill, compaction, and earth reinforcement. This value in particular is very likely to change with more information.

According to this estimate, the cost to construct an open bed infiltration basin is roughly $0.89 for each gallon per day capacity required. This can be compared directly with the corresponding value of $10.25 for leaching fields. Clearly, open bed infiltration basins are significantly less expensive for the same disposal capacity. However, this does not rule out the use of subsurface technology. Surface technology can only be used in very limited locations, and could be highly disfavored by the neighbors. On the other hand, subsurface technology can be used in many more locations, and provide a much less obtrusive finished product. The extra cost of leaching fields must be weighed against the benefits of doing so, and considered within the context of the budget that can be allocated toward this project.

4.6 Summary

Upon running the model for each of the six sites, the analysis criteria outlined below were compiled for each site.

Analysis Criteria

- Usable area and existing land use
- Appropriate infiltration technology and infiltration rate correlating to that technology
- Discharge capacity
- Excluding the Main Bays (Main Popponesset Bay, Main Waquoit Bay)
    - total number of embayments among which nitrogen load is distributed
    - maximum percentage nitrogen load to any 1 embayment
    - Total percentage nitrogen load to embayments
- Groundwater mounding: depth to water table before added recharge
- Groundwater mounding: depth to mounded water table
- Other considerations: contaminant plume, supply wells, etc.

The analysis criteria are described in section 4.1 of this report. A side-by-side comparison of the analysis criteria for each of the six sites facilitated the process in determining which of the sites would best fit the needs of serving the city of Mashpee. The side-by-side comparison was carried out in Table 4-10 below.

Table 4-10 – Summary Table of All Seven Sites and Analysis Criteria

Site	Site Name	Approx. Area (1000 sqft)	Existing Land Use	Infiltration Tech-nology	Infiltration Rate (GPD/ sqft)	Approx. Discharge Capacity (MGD)	No. of Embayments Affected	Max. % Nitrogen Load to any Embayment (excluding main bays)	Total % Nitrogen Load to all Embayments (excluding main bays)	Initial Depth to Water Table (ft)	Depth to Mounded Water Table (ft)	Other Considerations
1	Ball Fields	793	Athletic Ball Fields & Parking Lot	subsurface leaching trench	2.5	1.98	1	100.0	100.0	46.0	44.5	interaction with MMR plume
2	Ashumet Road	1,050	Undeveloped	open bed infiltration basins	5	5.25	2	71.0	100.0	34.0	29.0	interaction with MMR plume
3	Old Town Dump	The "Old Town Dump" has been deemed unsuitable for construction of a rapid infiltration system upon inspection of the site - further site analysis will not be conducted										
4	Transfer Station	430	Undeveloped	open bed infiltration basins	5	2.15	2	91.0	97.0	36.0	27.0	landfill adjacent to infiltration site
		210	Parking Lot	subsurface leaching trench	2.5	0.53						
5	High School	716	High School Parking Lot & Athletic Fields	subsurface leaching trench	2.5	1.79	2	84.0	89.0	30.0	30.0	proximity to easement at football field
6	Keeter Property	1,332	Undeveloped	open bed infiltration basins	5	6.66	4	36.0	63.0	18.0	6.0	proximity to drinking wells
7	New Seabury Country Club	368	Golf Course	subsurface leaching trench	2.5	0.92	0	0.0	0.0	28.5	26.4	on private property, golf course has varying elevations, can treat to 10 mg/L

Of the criteria quantified and described in Table 4-10, the critical values are the maximum and total percent of nitrogen affecting the embayments. These values have strong importance in this report because the goal of, not this report, but the greater situation of nitrogen loading in Cape Cod is to minimize the nitrogen loading. Therefore, the ideal value for the percent of nitrogen affecting embayments is the minimum percent. Looking at Table 4-10 it is apparent that Site 7 – New Seabury, with 0% nitrogen loading to all embayments, is the best candidate for reducing nitrogen loads. However, the discharge capability of Site 7 is very low at approximately 0.83MGD.

The design discharge value is 3.3MGD (under maximum monthly conditions); therefore if Site 7 is to be used as an infiltration site, 2.38MGD of effluent must still be accounted for. After Site 7, Site 6 – Keeter Property has the next smallest percent nitrogen loading value of 63%. The discharge capacity of Site 6 also meets, and even greatly exceeds, the remainder discharge of 2.38MGD. Therefore, Sites 6 and 7 are the ideal locations for infiltration basins. Table 4-10 only describes the general analysis criteria for each of the sites. The specific values for Sites 6 and 7, as well as the designs for two different Scenarios for these sites, are described, calculated, and determined in the following sections.

5 Recommendations

5.1 Proposed Discharge Design

Based on the analysis of each site described in the previous section, and the design criteria and objectives described in the introduction, a plausible design is proposed in this section. The design was selected to provide an average flow capacity of 1.65 MGD, with a peak factor of 2.0. Modeling and analysis of the design assumes peak flow for certain short term considerations, such as discharge capacity. The average flow is assumed when examining long term impacts, such as the estuarine nitrogen loads, and the water table steady-state level.

This design is the culmination of careful consideration of the pros and cons of using each site. These are each considered in greater detail in Chapter 4. After analysis, it was determined that Site 7 is ideal because it allows designers to circumnavigate the problem of estuarine nitrogen loading. In fact, effluent disposed at this site only has to be treated to 10 mg/L of total nitrogen as allowed by MassDEP regulations, since the discharge would be seeping directly into the ocean. Every other site would have to be treated to 3 mg/L, which requires more expensive treatment processes. This could save costs at the treatment stage over the years, which could perhaps balance the cost of rebuilding the golf course on top of subsurface leaching trenches. However, Site 7 cannot possibly handle the entire design flow, so another site must be chosen to handle the rest. Site 6 was determined to be nearly ideal. The total embayment loading by percentage is only 63%, so more than a third of the effluent disposed at Site 6 will never reach an estuary. Additionally, the maximum embayment loading by percentage is only 36%, so the nutrient load will be distributed between many embayments and therefore concentrated in none of them. These smaller nutrient loads may be well within the nutrient carrying capacity of the embayment. Additionally, it is sufficiently large that enough open bed infiltration basins can be built to handle the entire remainder of the flow while still being hidden within the woods by at least 150 feet in every direction. Therefore, the New Seabury Country Club was chosen as the primary site, with the remainder being disposed of within the Keeter Property.

Ideally, as much as possible will be discharged into subsurface leach fields that are to be constructed under three existing holes at the New Seabury Country Club, year-round. When this site was initially considered, rough areas were measured and the capacity was calculated as

0.72MGD using subsurface leach fields. However, this area is only approximate, and assumes a factor of safety of 0.9 for inefficiencies in the design. Once it became apparent that this was the ideal site for nitrogen disposal, this area calculation was revisited. The design is constrained by the fact that, once construction is complete, the golf course conditions must be unchanged. Accordingly, trees were avoided by approximately 10 feet. Scenario 1 takes an ideal approach, assuming that every usable square foot would be usable for discharge. The purpose of this is to better understand the maximum capacity of the proposed scenario. Scenario 2 on the other hand, aligns the leaching field chambers in series in several trenches along the contours of the fairways. Site 7 theoretically has the capacity to dispose of 1.20 MGD according to Scenario 1 and 0.83 MGD of that is utilized in the piping design of Scenario 2.

According to the design, the remaining flow is to be disposed of at Site 6 - the Keeter Property, using open infiltration beds, which are less expensive, require less land, and can appropriately be used at this more remote site. The approximate measurement of the effluent disposal capacity of the entire site reported a conservative value of 2.93 MGD, which is well above the capacity necessary to handle the entire remaining peak load, after Site 7 is fully utilized. This Scenario results in minimum total nitrogen load applied to the combined Popponesset and Waquoit Estuarine systems, while distributing the nitrogen as evenly as possible between the two.

5.2 Scenario 1 - Ideal Capacity

Scenario 1 proposes that as much effluent as theoretically possible is discharged at Site 7, and the remainder at the Keeter property. This discharge capacity was calculated by using Google Earth to map a polygon onto each hole. These polygons, seen in Fig. 5-1, were adjusted to fill as large of an area as possible within the site. The areas were calculated using a surveying equation which calculates the area inside any polygon based on the coordinates of the corners. Google Earth was used to measure the Universal Transverse Mercator coordinates of each point to a probable error of 0.02 meters. According to this calculation, assuming 100% of the area can be incorporated into the leach field design, a maximum area of 484,000 sq ft was calculated. This area corresponds to 1.21 MGD. Therefore, 1.20 MGD was taken as the Scenario 1 design flow rate for the New Seabury Country Club.

Fig. 5-1 – Scenario 1 Subsurface Leach Fields for Site 7

After filling Site 7 to capacity, the remaining flow is discharged within Site 6, the Keeter Property, making use of open bed infiltration basins. At this point the design peak factor of 2.0 becomes critical. During the peak season, enough effluent beds must be available to dispose of the yearly average flow at 1.65 MGD as well as the peak flow, which brings the design capacity to 3.3 MGD. However, this assumes that during the entire peak season, no emergency repairs will need to be made to the system, and there will never be a day where the design flow of 3.3 million gallons is exceeded. To create a buffer in case of unexpected circumstances, an additional 10% was added to the design flow, bringing the necessary effluent bed capacity to 3.63 MGD. After subtracting the design flow rate for Site 7, the flow rate for Site 6 is 2.43 MGD for Scenario 1, which requires 486,000 sq ft of effluent bed area.

This design flow rate is well beneath the capacity of the Keeter Property. Since the area of this property is not critical, it was designed in such a way as to minimize cost and neighborhood impact while maximizing the attractive features of the site. Based on the initial modeling of the site, it was determined that the effluent travels to several locations. For analysis purposes, these have been divided into five categories: Waquoit Bay, Inner Popponesset Bay, Outer Popponesset Bay, the Atlantic Ocean, and the two Public Supply wells. As seen in Fig. 4-19, effluent from the east side travels east toward Popponesset, effluent from the west side travels west toward Waquoit and effluent from the central portion travels toward the wells and the Atlantic. In order to maximize the amount of effluent which goes directly to the coast, as well as balance the amount which travels to each of the bays, the Open Bed Infiltration Basins were designed to be grouped toward the center of the site. Additionally, they were shaped and located so that they would be an equal distance from each of the nearby private properties, and the sizes were standardized for simplicity in designing and constructing the site.

For the Scenario 1 design flow at the Keeter property, 486,000 sq ft of effluent bed was required. This was divided into 6 square effluent beds, each 285 feet on a side, shown in. Each effluent bed conveniently absorbs 0.41 MGD, which means that even at 98% of peak capacity, one entire effluent bed or equivalent at Site 7 could be offline for repairs. This extra capacity could also be used in case of wastewater flows larger than the expected peak value.

Scenario 1 was modeled twice, to examine the critical long term and short term impacts caused by this design. First, the average flow was considered. Since Site 7 is the primary location, during less than average flows, site 7 will still be used as much as possible. When the flow is between 1.2 and 3.3 MGD, the discharge at Site 6 varies proportionally while Site 7 stays constant. Once the total flow drops below 1.2 MGD the flow for Site 7 must decreased. To determine the average flow at each site, some assumptions regarding the seasonal flow variations must be made.

Fig. 5-2 – Scenario 1 Open Bed Infiltration Basins for Site 6

A report by Donald Walter and Ann Whealan addresses the seasonal variations of water flow specifically for the Sagamore Lens. On page 35, the report describes the difference in well pumping between the seven off-season months, October through April, and the five in-season months, May through September. The flow of water that is being pumped out of their model represents seasonal anthropogenic water use, which also corresponds to the wastewater flow which must be disposed of within Sites 6 and 7. According to their numbers, the average peak season flow is 2.32 times larger than the average off-season flow. Applying this equation, the off-season flow is found to be 1.07 MGD, which applies for a season of 7 months. This brings the average flow at Site 7 down to 1.12 MGD, with the remaining 0.53 MGD disposed of at Site 6.

Based on the experiences from modeling the individual sites, it was determined that modifying the grid in any way was not the optimal method for adding recharge to the model. Instead, recharge zones were defined to match as closely as possible to the actual locations chosen. The model recharge value was calculated to yield the correct total effluent flow, taking into account the difference in actual verse model areas. Appendix I – Calculations includes all the calculations which were used in the model.

In Fig. 5-3 and Fig. 5-4, the model recharge zones for Sites 6 and 7 are shown in yellow and white, respectively. The yellow area was assigned recharge of 8,892.85 mm/year, and the white area was assigned a recharge of 13,725.91 mm/year, which, when converted, correspond to 0.53 MGD and 1.12 MGD for the areas shown. Particles were placed in a grid every 100 feet in the middle of the first layer. For Site 7, since the entire flow travels directly to the ocean, the particles were simply copied from the initial model of the site. However, Site 6 displayed much more variety in effluent travel locations. In an effort to judge as accurately as possible the ratios of effluent which would travel to each final location, the grid of particles was placed so that particles and effluent bed locations matched with the recharge location as closely as possible. Due to the roughness of the grid, some error will be inevitable.

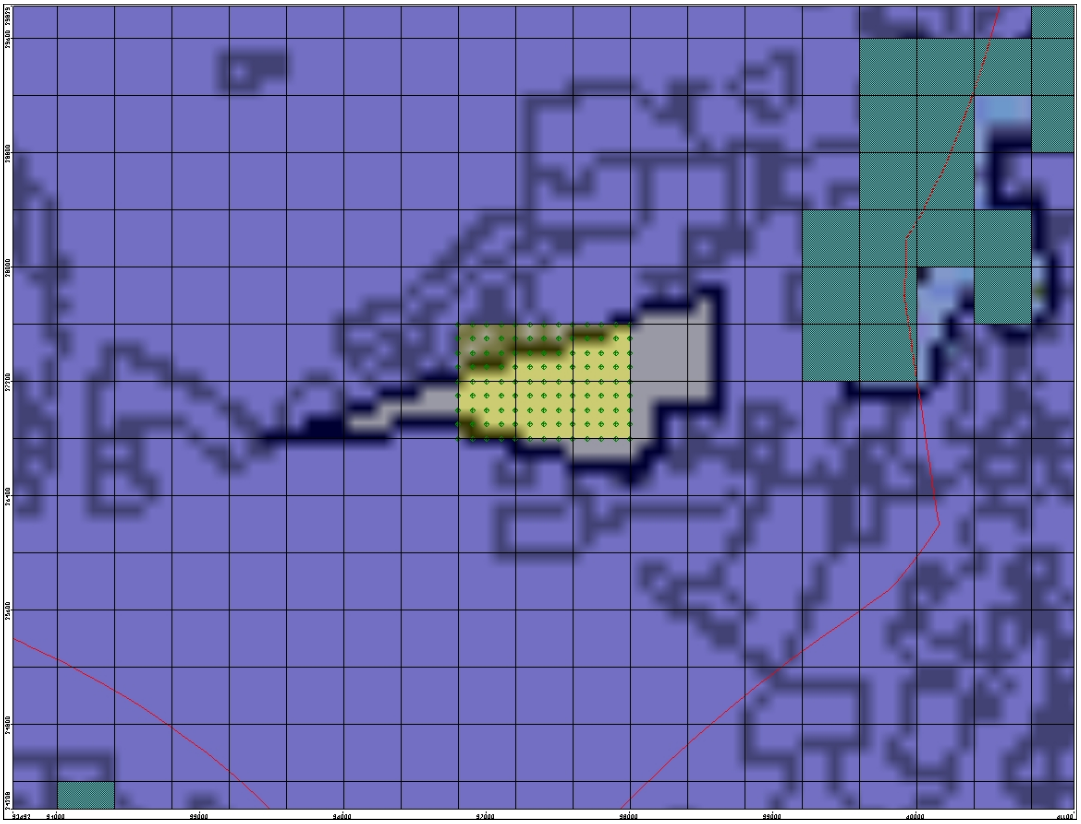

Fig. 5-3 – Scenario 1 Site 6 Recharge Areas

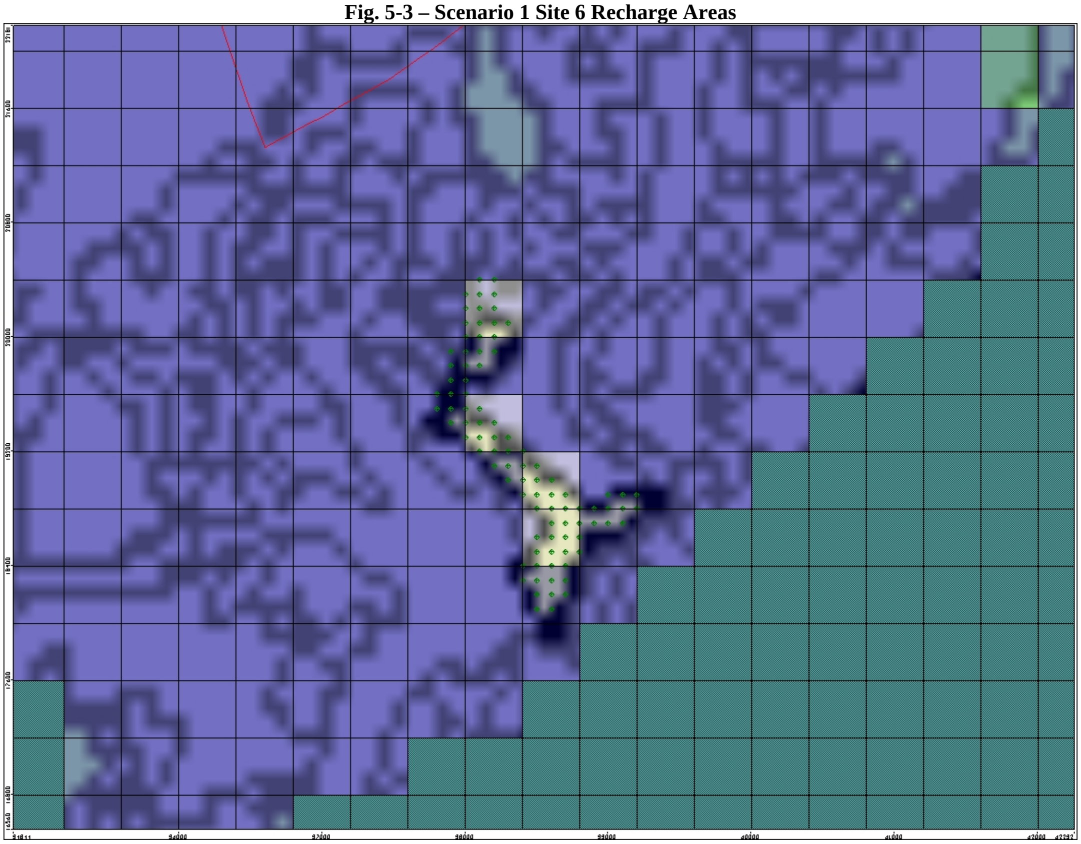

Fig. 5-4 – Scenario 1 Site 7 Recharge Areas

5.2.1 Scenario 1 Nitrogen Loading Results and Analysis

In order to determine the success of the design, Scenario 1 was modeled and analyzed by the same criteria used for individual discharge at Sites 1-7. Fig. 5-5 and Fig. 5-6 show the particle tracks for sites 6 and 7, representing effluent flow at average conditions. 117 particles were placed at Site 6 and 93 particles were placed at Site 7, arranged in 100ft x 100ft spacing. The model was run at average flow conditions in order to predict the long-term effects of effluent discharge at the sites.

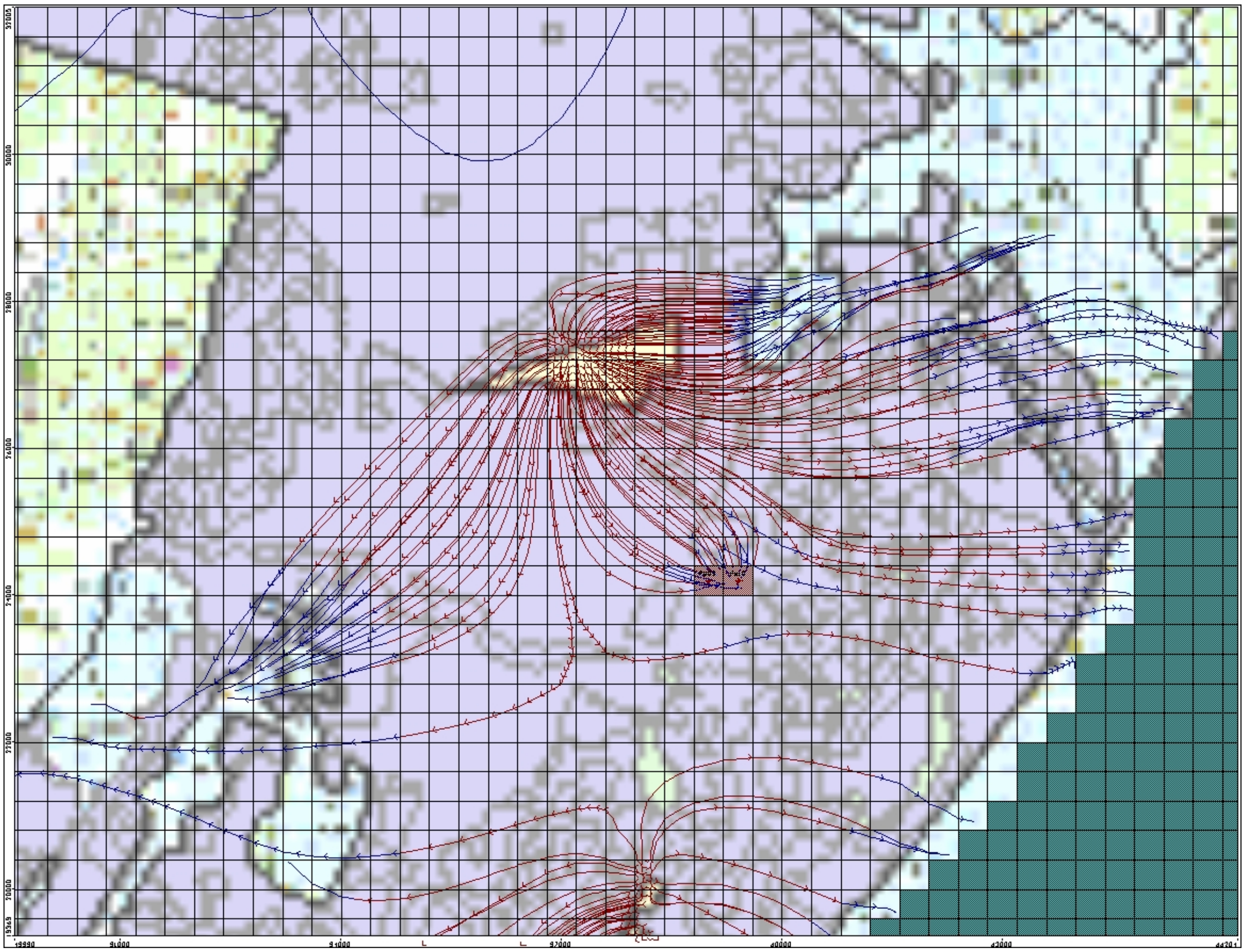

Fig. 5-5 – Scenario 1 Site 6 Average Flow Pathlines

It should be noted that Fig. 5-5 and Fig. 5-6 actually represent the same model results, but two images have been used in order to display the entire system of particle tracks in sufficient detail.

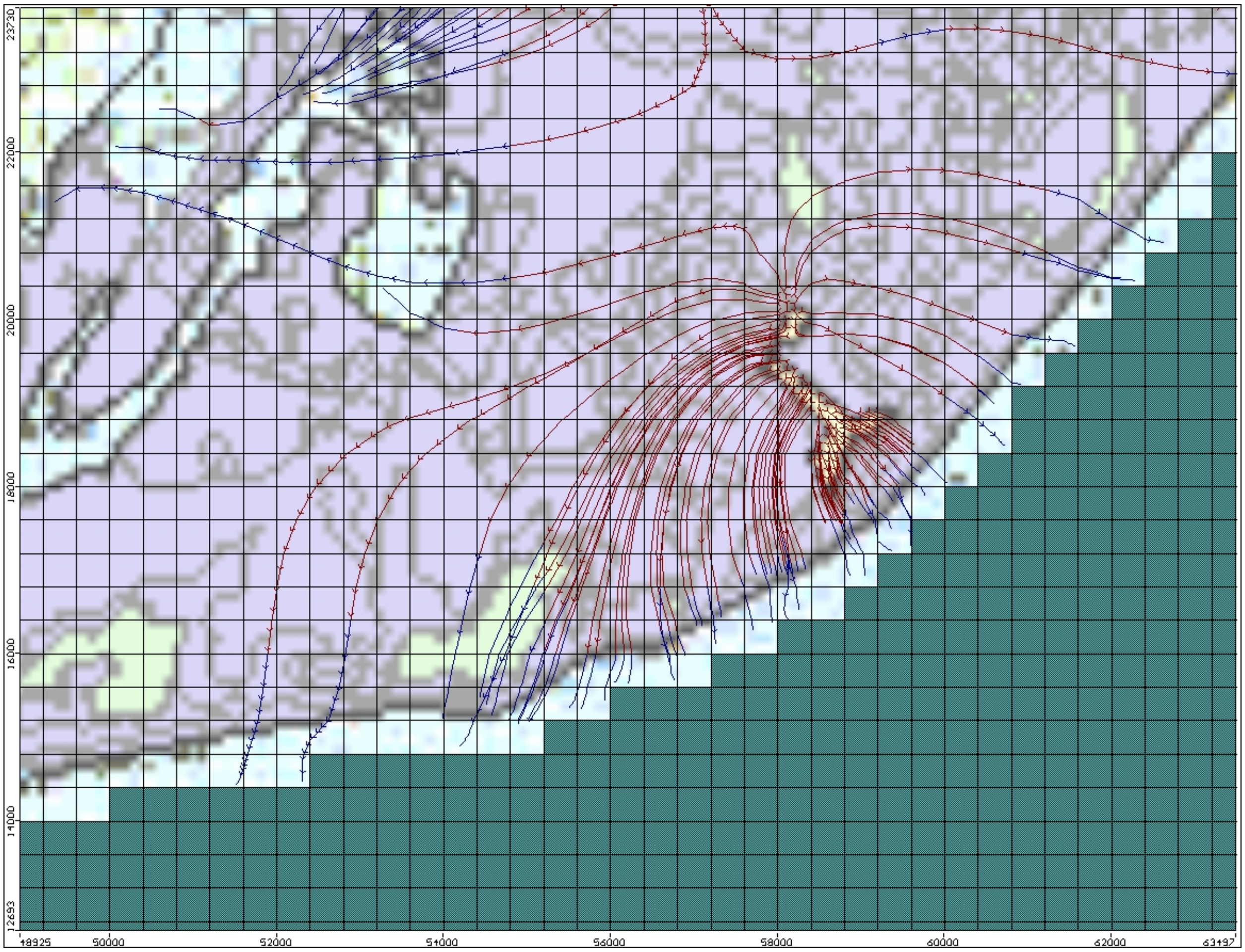

Fig. 5-6 – Scenario 1 Site 7 Average Flow Pathlines

The results of nitrogen loading analysis are summarized in Table 5-1 and Table 5-2. Site 6 displays the same characteristically low total embayment loading (58%) and maximum embayment loading by percentage (36% in Ockway Bay) for this discharge scenario. Site 7 actually deviates from expected results to a very small degree, and a fraction of effluent (2%) ends up in Hamblin and Jehu Ponds. However, Site 7 still displays unbelievably high non-embayment loading (98%) and maximum embayment loading by percentage of 1% for Hamblin and Jehu ponds, leading to the conclusion that if Scenario 1 is adopted by the Town of Mashpee, there is definite hope that all embayments will make a full recovery from nutrient overloading and eutrophication.

Table 5-1 – Scenario 1 Average Flow Particle Tracks

Scenario 1 Site 6 Particle Tracks & Nitrogen Loads - From Average Flow				
	Particles	Percentage	Total Flow (MGD)	Nitrogen Load (KG/Day)
Total Site 6	117	100%	0.53	6.02
Nitrogen Concentration: 3 mg/L	117			
Quashnet River	0	0%	0.00	0.00
Hamblin Pond	2	2%	0.01	0.10
Jehu Pond	19	16%	0.09	0.98
Little River	0	0%	0.00	0.00
Great River	0	0%	0.00	0.00
Total Waquoit System	21	18%	0.10	1.08
Mashpee River	0	0%	0.00	0.00
Shoestring Bay	0	0%	0.00	0.00
Ockway Bay	42	36%	0.19	2.16
Popponesset Creek	5	4%	0.02	0.26
Pinquickset Cove	0	0%	0.00	0.00
Total Popponesset System	47	40%	0.21	2.42
Waquoit (main) Bay	2	2%	0.01	0.10
Popponesset (main) Bay	19	16%	0.09	0.98
Nantucket Bay	7	6%	0.03	0.36
Mashpee Supply Wells	21	18%	0.10	1.08
Total Embayment Loading	68	58%	0.31	3.50
Total Non-Embayment Loading	49	42%	0.22	2.52

When evaluating Table 5-1, care must be taken so that the accuracy of the model is not overstated. Based on the model grid size, which determines the recharge locations, the model cannot say precisely how much effluent will end up in each location. However, the fact that particles travel several distinct directions indicates that this location is on the boundary between the contributory areas of three different watersheds. Additionally, the manner in which particles spread from this location is consistent with the dispersion that results from a local mound underneath an effluent location.

Table 5-2 – Scenario 1 Average Flow Particle Tracks

Scenario 1 Site 7 Particle Tracks & Nitrogen Loads - From Average Flow				
	Particles	Percentage	Total Flow (MGD)	Nitrogen Load (KG/Day)
Total Site 7	93	100%	1.12	42.40
Nitrogen Concentration: 10 mg/L				
Quashnet River	0	0%	0.00	0.00
Hamblin Pond	1	1%	0.01	0.46
Jehu Pond	1	1%	0.01	0.46
Little River	0	0%	0.00	0.00
Great River	0	0%	0.00	0.00
Total Waquoit System	2	2%	0.02	0.91
Mashpee River	0	0%	0.00	0.00
Shoestring Bay	0	0%	0.00	0.00
Ockway Bay	0	0%	0.00	0.00
Popponesset Creek	0	0%	0.00	0.00
Pinquickset Cove	0	0%	0.00	0.00
Total Popponesset System	0	0%	0.00	0.00
Waquoit (main) Bay	0	0%	0.00	0.00
Popponesset (main) Bay	0	0%	0.00	0.00
Nantucket Bay	91	98%	1.10	41.48
Mashpee Supply Wells	0	0%	0.00	0.00
Total Embayment Loading	2	2%	0.02	0.91
Total Non-Embayment Loading	91	98%	1.10	41.48

5.2.2 Scenario 1 Mounding Results and Analysis

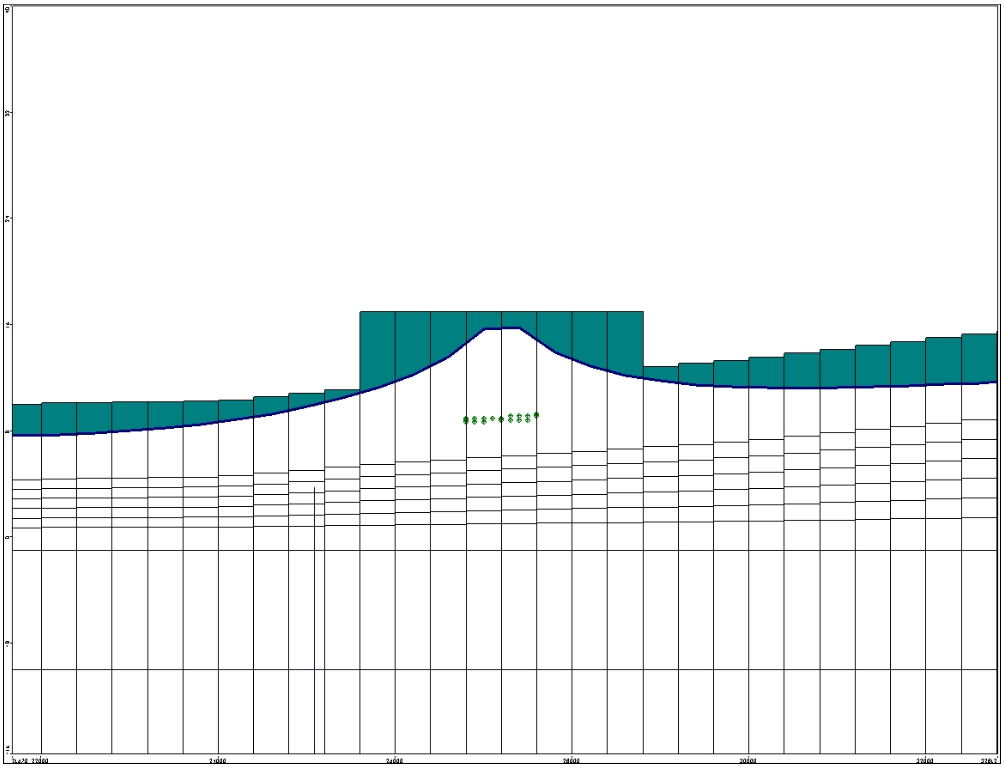

Fig. 5-7 – Scenario 1 Site 6 Mounding

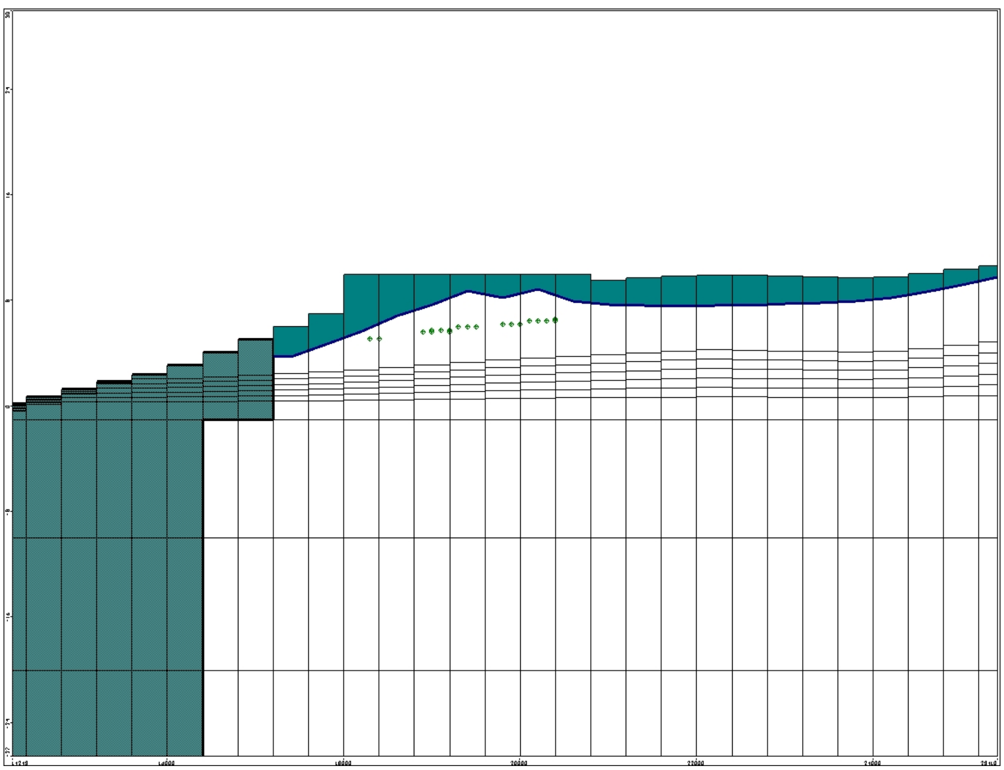

Fig. 5-8 – Scenario 1 Site 7 Mounding

The maximum height of the groundwater mound has been reduced somewhat from the results of the mounding analysis conducted in Section 4.2.7 but the water table under the Keeter Property, modeled at the proposed design flow rate, is still some cause for concern. According to the mounding results in Table 5-3 – Mounding Results for Scenario 1, Site 6, the depth to mounded water table is 10.2 feet at its shallowest point. This is within acceptable limits for discharge facility design, but may impact any structures built over this part of the water table. Fortunately, it is clear from an inspection of Fig. 5-7 that the water table drops off steeply on either side of the Keeter Property discharge site. The site is several hundred feet wide and no buildings are located for another 150 feet on either side. Most likely, this will provide adequate clearance, and again this mound represents the peak load during the year. However, if Scenario 1 is implemented, it will be necessary to monitor groundwater mounding levels during peak season to ensure that no nearby structures are compromised.

The maximum height of the groundwater mound at Site 7 is available from Table 5-4 – Mounding Results for Scenario 1, Site 7, and it is clear that groundwater mounding is not a threat at this site.

Table 5-3 – Mounding Results for Scenario 1, Site 6

Site 6 Mounding Analysis - From Peak Flow	
Minimum Ground Elevation	26
Maximum Initial Water Table Elevation	8
Peak Mounding Water Table Elevation	15.8
Water Table Increase	7.8
Depth to Mounded Water Table	10.2

Table 5-4 – Mounding Results for Scenario 1, Site 7

Site 7 Mounding Analysis - From Peak Flow	
Minimum Ground Elevation	33
Maximum Initial Water Table Elevation	4.5
Peak Mounding Water Table Elevation	8.9
Water Table Increase	4.4
Depth to Mounded Water Table	24.1

5.3 Scenario 2 - Proposed Design

As in the previous design scenario, Scenario 2 requires that as much wastewater effluent as possible be disposed at the New Seabury Country Club, from where it will be directed to Nantucket Sound such that nitrogen impacts on the two estuaries will be minimized. The principle difference between Scenarios 1 and 2 is that in this scenario leaching trenches at the New Seabury Country Club are actually designed in organized sections along the length of the fairways. By placing as many of these series of chambers as possible into the site, and adding up the effective areas of each, a total area of 332,000 sq ft was calculated. This corresponds to 0.83 MGD, which is makes use of 68% of the overall flow capacity calculated for Scenario 1. The disposal basins are shown in Fig. 5-9.

With the area of Site 7 used to full capacity for effluent disposal the remaining flow is discharged within Site 6, the Keeter Property, using open bed infiltration basins. It is recognized that there may be constraints that limit the use of this site. However, it is considered appropriate in this project to assess the maximum potential use of the site. At this point the design peak factor of 2.0 becomes critical. During the peak season, enough effluent beds must be available to dispose of the yearly average flow at 1.65 MGD as well as the peak flow, which brings the design capacity to 3.3 MGD. However, this assumes that during the entire peak season, no emergency repairs will need to be made to the system, and there will never be a day where the design flow of 3.3 MGD is exceeded. To create a buffer in case of unexpected circumstances, an additional 10% was added to the design flow, bringing the necessary effluent bed capacity to 3.63 MGD. After subtracting the design flow rate at Site 7 from the peak design flow rate with the factor of safety, the flow rate for Site 6 is 2.80 MGD for Scenario 2 which requires 560,000 sq ft, of effluent bed area. The Keeter Property site uses 14 disposal basins at 200'x200' each, yielding a total area of 560,000 sq ft which meets the required basin area.

Fig. 5-9 – Scenario 2 Subsurface Leach Fields for Site 7

Fig. 5-10 – Scenario 2 Open Bed Infiltration Basins for Site 6

Based on the results from modeling the individual sites, it was determined that modifying the grid in any way was not the optimal method for adding recharge to the model. Instead, recharge zones were defined to match as closely as possible to the actual locations chosen. The model recharge value was calculated to yield the correct total effluent flow, taking into account the difference between the actual and modeled areas.

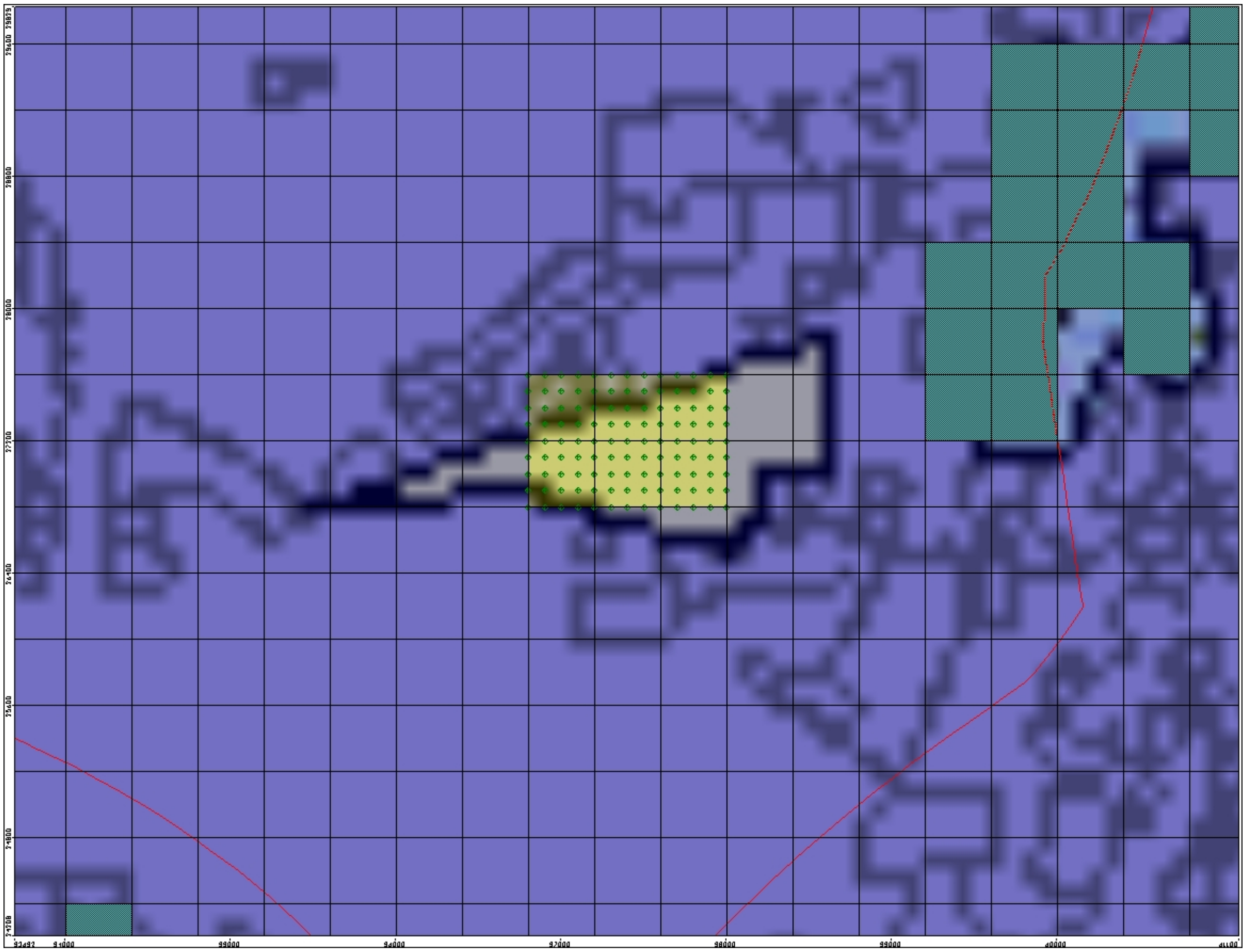

Fig. 5-11 – Scenario 2 Site 6 Recharge Areas

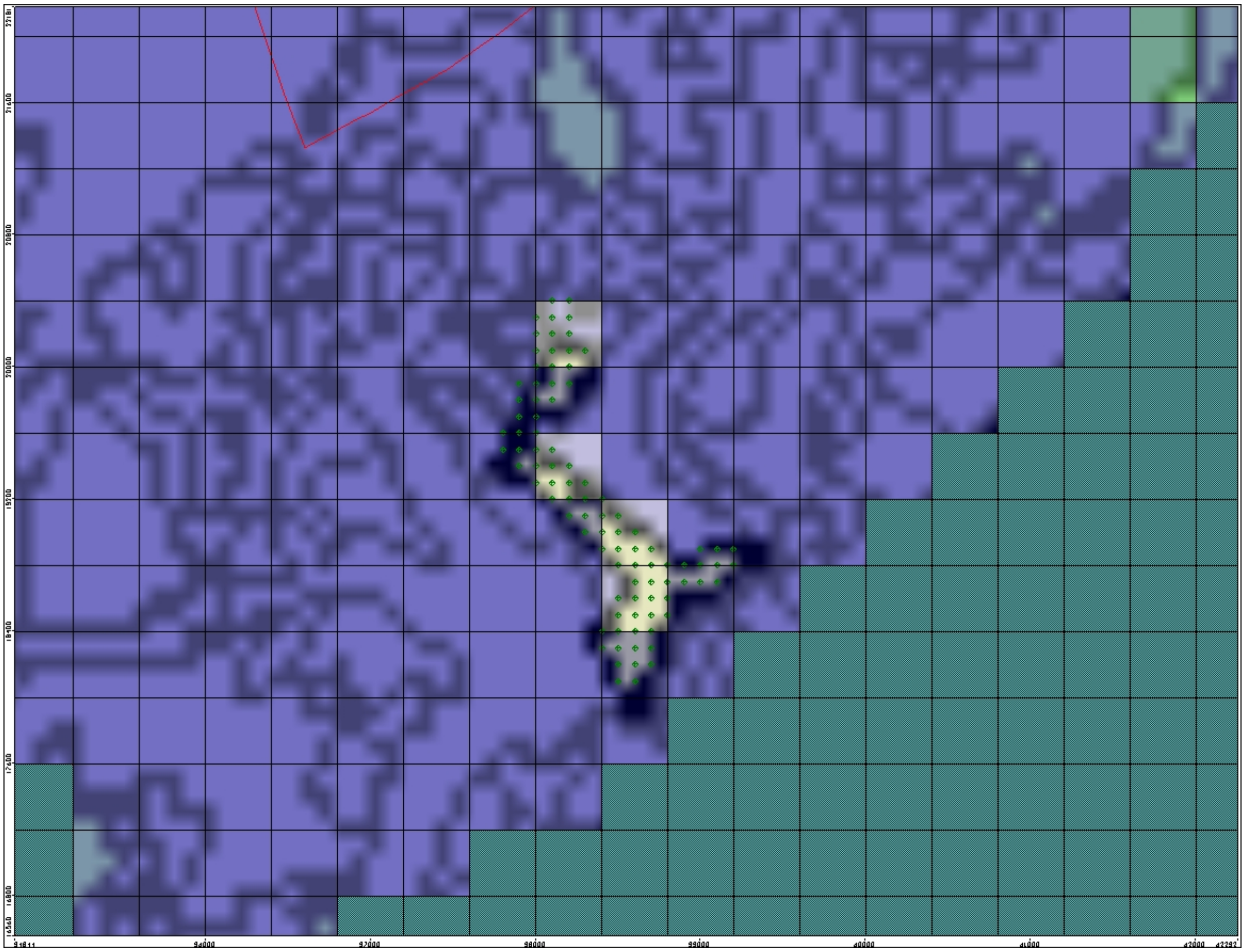

Fig. 5-12 – Scenario 2 Site 7 Recharge Areas

5.3.1 Scenario 2 Nitrogen Loading Results and Analysis

In order to determine the success of the design, Scenario 2 was modeled and analyzed by the same criteria used for individual discharge at Sites 1-7. Fig. 5-13 and Fig. 5-14 show the particle tracks for sites 6 and 7, representing effluent flow at average conditions. 117 particles were placed at Site 6 and 93 particles were placed at Site 7, arranged in 100ft x 100ft spacing. The model was run at average flow conditions in order to predict the long-term effects of effluent discharge at the sites.

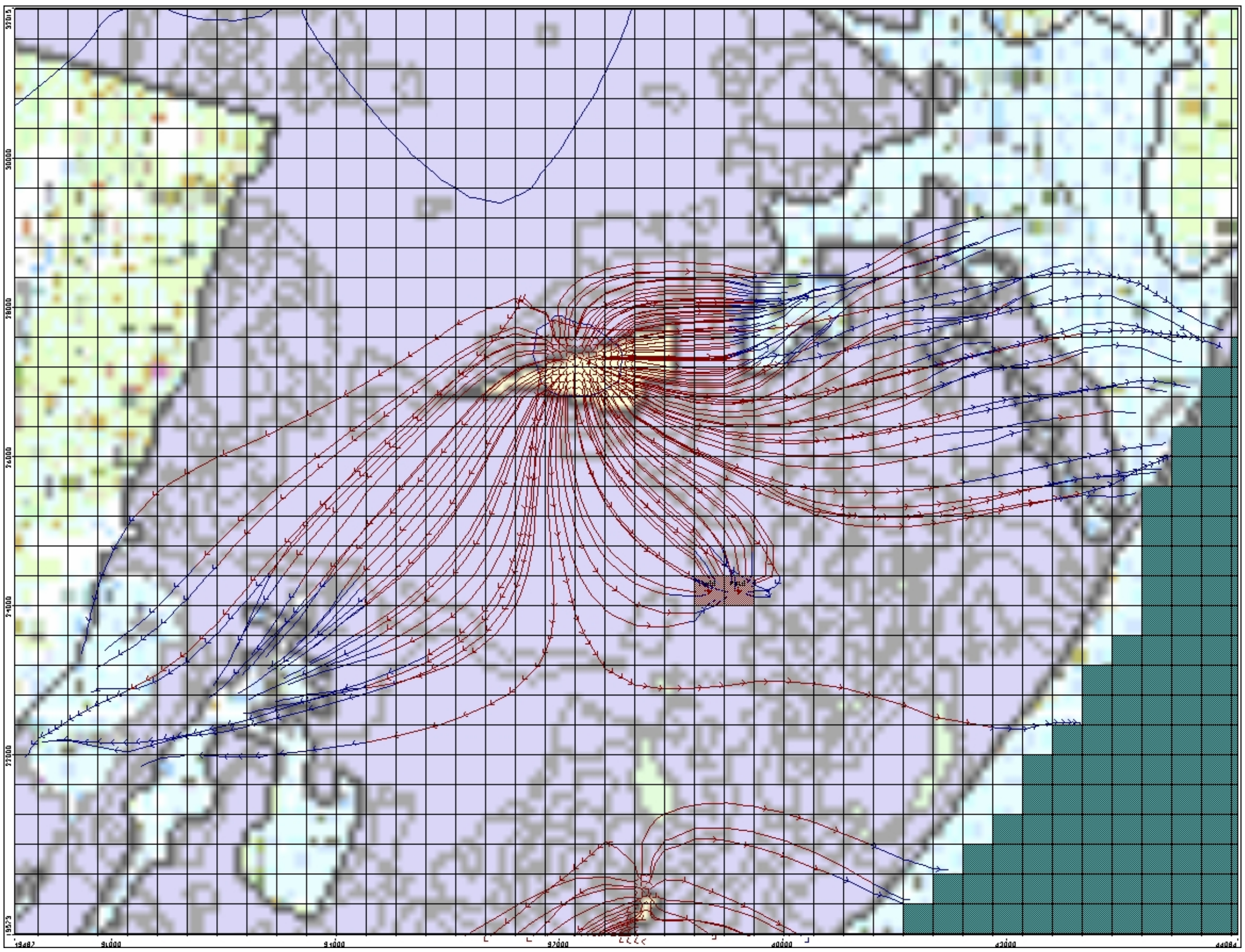

Fig. 5-13 – Scenario 2 Site 6 Average Flow Pathlines

It should be noted that these two figures are showing the same results, but focusing on different areas for clarity.

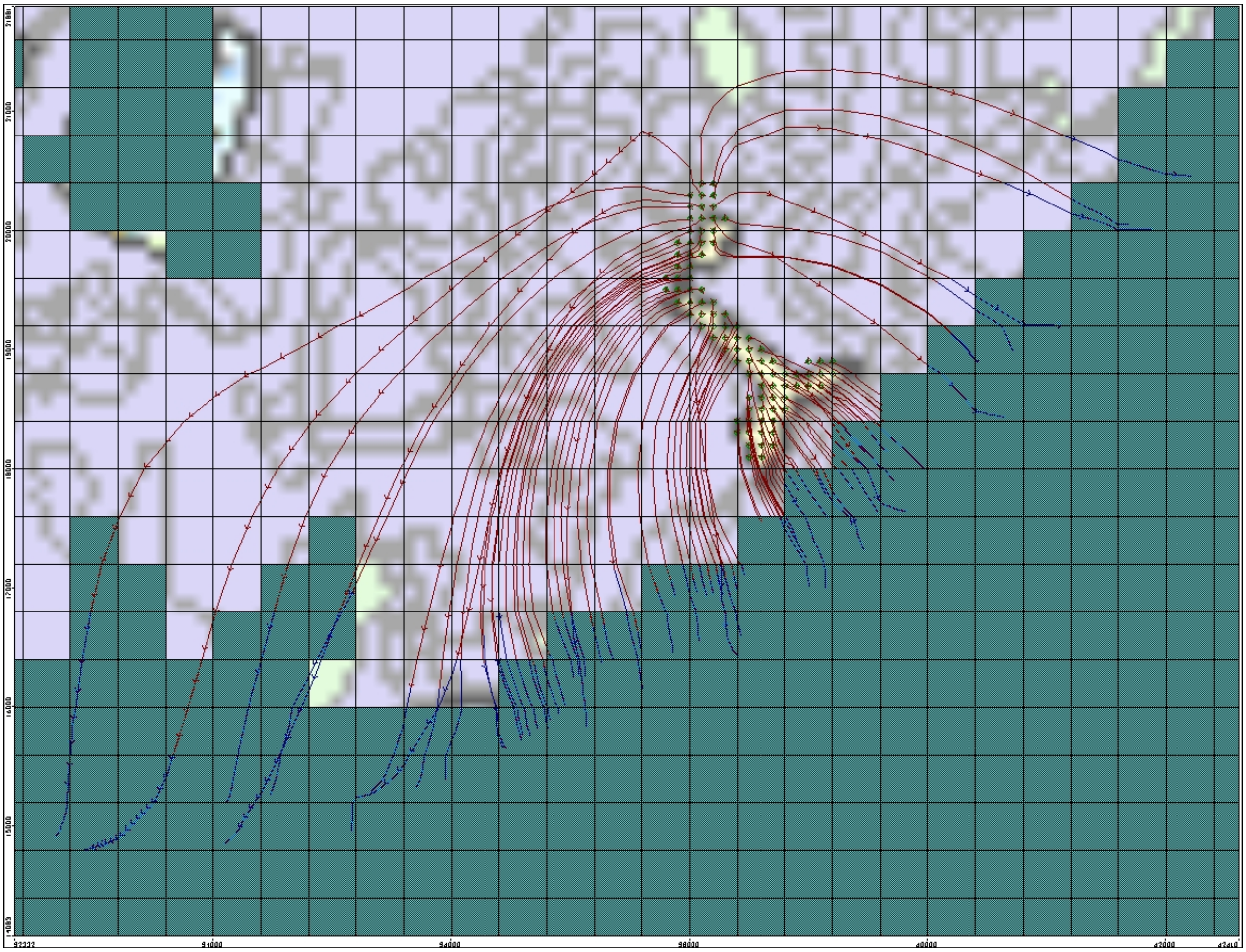

Fig. 5-14 – Scenario 2 Site 7 Average Flow Pathlines

The results of nitrogen loading analysis are summarized in Table 5-5 and

Table 5-6. Site 6 has slightly higher total embayment loading (61%) and slightly lower maximum embayment loading by percentage (35% in Ockway Bay) for this discharge scenario. The model predicts that discharging effluent at Site 7 will lead to 0% total embayment loading, leading to the conclusion that if Scenario 2 is adopted by the Town of Mashpee, there is definite hope that all embayments will make a significant recovery from nutrient overloading and eutrophication.

Table 5-5 – Scenario 2 Average Flow Particle Tracks

Scenario 2 Site 6 Particle Tracks & Nitrogen Loads - From Average Flow				
	Particles	Percentage	Total Flow (MGD)	Nitrogen Load (KG/Day)
Total Site 6	117	100%	0.77[2]	8.74
Nitrogen Concentration: 3 mg/L	117			
Quashnet River	0	0%	0.00	0.00
Hamblin Pond	7	6%	0.05	0.52
Jehu Pond	20	17%	0.13	1.49
Little River	0	0%	0.00	0.00
Great River	0	0%	0.00	0.00
Total Waquoit System	27	23%	0.18	2.02
Mashpee River	0	0%	0.00	0.00
Shoestring Bay	0	0%	0.00	0.00
Ockway Bay	41	35%	0.27	3.06
Popponesset Creek	3	3%	0.02	0.22
Pinquickset Cove	0	0%	0.00	0.00
Total Popponesset System	44	38%	0.29	3.29
Waquoit (main) Bay	0	0%	0.00	0.00
Popponesset (main) Bay	22	19%	0.14	1.64
Nantucket Bay	1	1%	0.01	0.07
Mashpee Supply Wells	23	20%	0.15	1.72
Total Embayment Loading	71	61%	0.47	5.31
Total Non-Embayment Loading	46	39%	0.30	3.44

When evaluating Table 5-5, care must be taken so that the accuracy of the model is not overstated. Based on the model grid size, which determines the modeled recharge locations, the model cannot be used to determine precisely how much effluent will end up in each location. However, the fact that particles travel in several distinct directions indicates that this location is on the boundary between the contributory areas of three different watersheds. Additionally, the manner in which particles spread from this location is consistent with transport that would be expected from a local mound located beneath an effluent location.

[2] When Scenario 2 was first being considered, the design flow was 0.88 MGD at Site 7, with the remaining 0.77 MGD at Site 6. This is the flow that was used in the model. Due to further revisions of the design, these values were changed to 0.82MGD and 0.83MGD in the final design.

Table 5-6 – Scenario 2 Average Flow Particle Tracks

Scenario 2 Site 7 Particle Tracks & Nitrogen Loads - From Average Flow				
	Particles	Percentage	Total Flow (MGD)	Nitrogen Load (KG/Day)
Total Site 7	93	100%	0.88	33.31
Nitrogen Concentration: 10 mg/L	93			
Quashnet River	0	0%	0.00	0.00
Hamblin Pond	0	0%	0.00	0.00
Jehu Pond	0	0%	0.00	0.00
Little River	0	0%	0.00	0.00
Great River	0	0%	0.00	0.00
Total Waquoit System	0	0%	0.00	0.00
Mashpee River	0	0%	0.00	0.00
Shoestring Bay	0	0%	0.00	0.00
Ockway Bay	0	0%	0.00	0.00
Popponesset Creek	0	0%	0.00	0.00
Pinquickset Cove	0	0%	0.00	0.00
Total Popponesset System	0	0%	0.00	0.00
Waquoit (main) Bay	0	0%	0.00	0.00
Popponesset (main) Bay	0	0%	0.00	0.00
Nantucket Bay	93	100%	0.88	33.31
Mashpee Supply Wells	0	0%	0.00	0.00
Total Embayment Loading	0	0%	0.00	0.00
Total Non-Embayment Loading	93	100%	0.88	33.31

5.3.2 Scenario 2 Mounding Results and Analysis

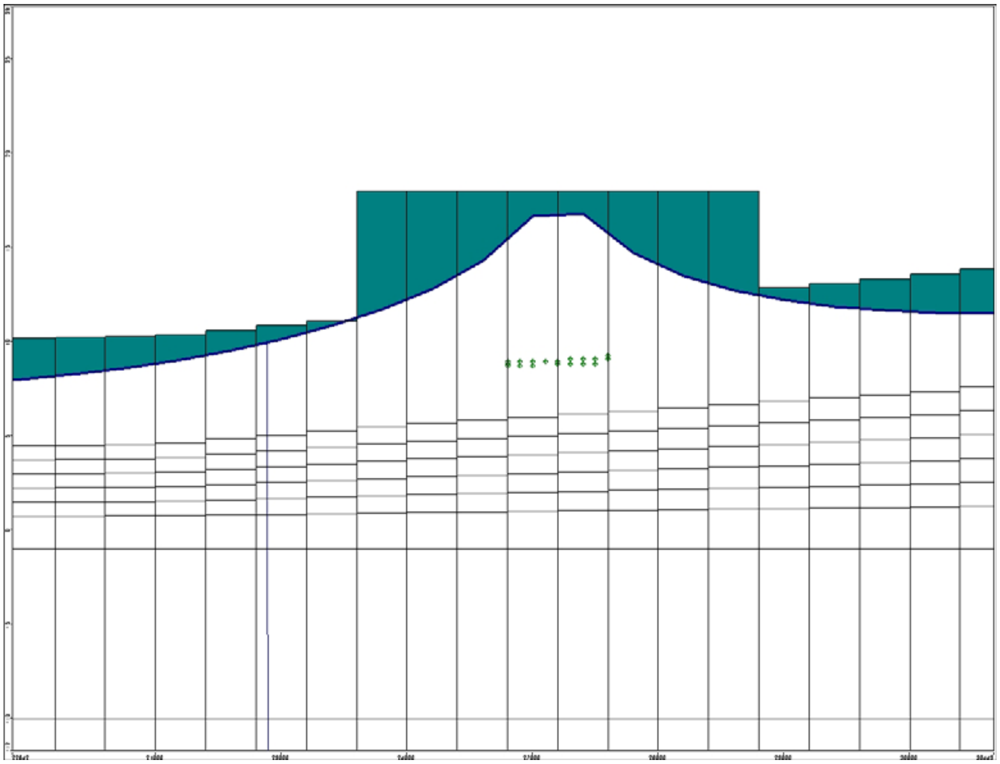

Fig. 5-15 – Scenario 2 Site 6 Mounding

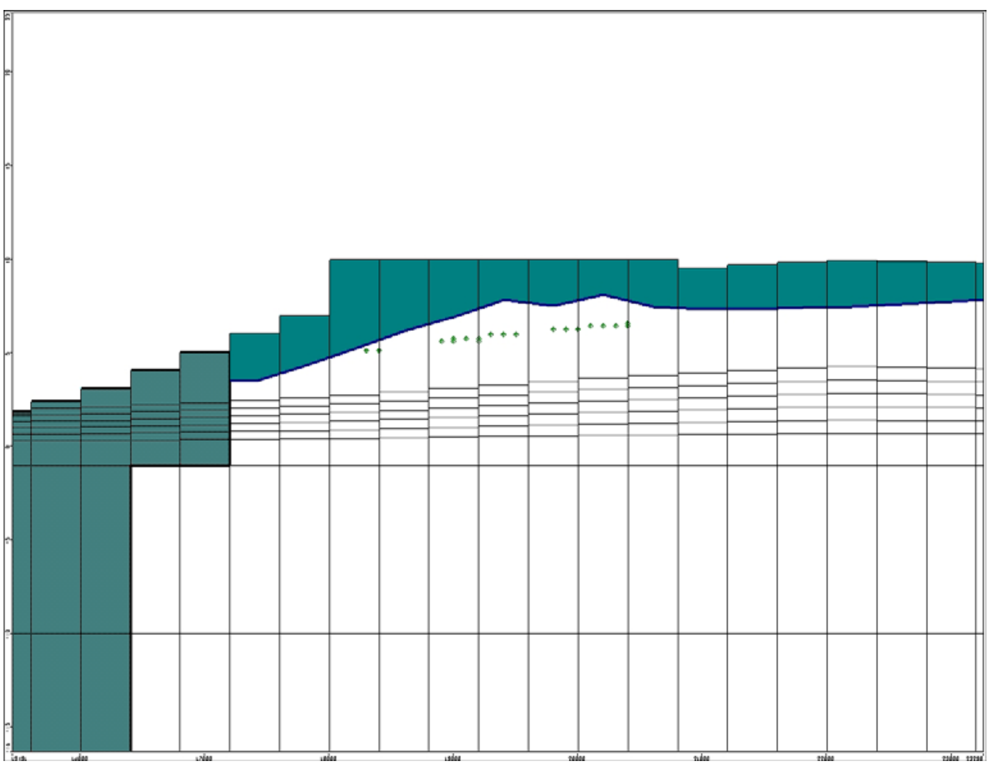

Fig. 5-16 – Scenario 2 Site 7 Mounding

The maximum height of the groundwater mound has been reduced somewhat from the results of the mounding analysis conducted in Section 4.2.6, but the water table under the Keeter Property, modeled at the proposed design flow rate, is still some cause for concern. According to the mounding results in Table 5-7, the depth to mounded water table is 10.2 feet at its shallowest point. This is within acceptable limits for discharge facility design, but may wreak havoc on any structures built over this part of the water table. Fortunately, it is clear from an inspection of Fig. 5-15 that the water table drops off steeply on either side of the Keeter Property discharge sites- the sites themselves are several hundred feet wide and no buildings are located for another 150 feet on either side. Most likely, this will provide adequate clearance, and again this mound represents the peak load during the year. However, if Scenario 2 is implemented, it will be necessary to monitor groundwater mounding levels during peak season to ensure that no nearby structures are compromised.

The maximum height of the groundwater mound at Site 7 is available from Table 5-8, and it is clear that groundwater mounding is not a threat at this site.

Table 5-7 – Mounding Results for Scenario 2, Site 6

Site 6 Mounding Analysis - From Peak Flow	
Minimum Ground Elevation	26
Maximum Initial Water Table Elevation	8
Peak Mounding Water Table Elevation	16.8
Water Table Increase	8.8
Depth to Mounded Water Table	9.2

Table 5-8 – Mounding Results for Scenario 2, Site 7

Site 7 Mounding Analysis - From Peak Flow	
Minimum Ground Elevation	33
Maximum Initial Water Table Elevation	4.5
Peak Mounding Water Table Elevation	8.1
Water Table Increase	3.6
Depth to Mounded Water Table	24.9

5.4 Conceptual Plan for Layout and Piping

5.4.1 Design Considerations

The installation of leaching fields requires considerations of the infrastructure required to deliver flow to the system. The delivery infrastructure should be designed in order to minimize the cost of piping and installation without restricting the volume of effluent flow delivered to the site. For the purpose of this project, effluent force mains have been run underneath roads close in proximity to the disposal locations. The roads were chosen in order to maximize access during construction and maintenance of the system. Other alternatives for piping are available, such as running the force mains down the side of the fairway directly adjacent to the leaching trenches or down the center of the fairway in the 10 feet of space between the trenches. Although these options are available for the final design of the site, they have not been considered here due to the difficulty of laying pipe so close to the trenches and accessing those pipes for maintenance, as well as the necessary reduction in available infiltration area.

At one end of each section of pipe a distribution control method must be implemented. For the purposes of this project, gate valves have been chosen. The final site design may implement distribution boxes at several key points, which would have a reduced impact on the reconstructed golf course and may ease the task of distributing effluent flow evenly between the pipes in any given section. There is an open area at the site, directly adjacent to point D in Fig. 5-17, which could be an appropriate space for a distribution control center.

5.4.2 Site 7 – New Seabury Site

5.4.2.1 Layout

Fig. 5-17 shows Site 7, the New Seabury site, with a proposal for a set of leaching field chambers. Each of the leaching field chambers measure 20ft x 4ft, with an effective area of 5 ft horizontally around the length of each chamber (the 5 ft effective area around the width of each chamber is lost when chambers are placed in series).

Fig. 5-17 – Site 7 – Overhead View of New Seabury Site with Conceptual Piping Design

Site 7 is a golf course with several fairways running in different directions. Leaching field chambers are placed in series down the fairway. The series of leaching field chambers are placed parallel across the width of each fairway. Because the width of each fairway varies over its length, there are different sections of leaching field chambers. As shown in Fig. 5-17, there are different leaching sections: A, B, C, D, E, F, and G. Each section serves different surface areas, and therefore serves different portions of the 0.83 MGD flow distributed to New Seabury.

5.4.2.2 Piping

All pipes used in distributing effluent to the chambers will be self-draining SDR-35 PVC. The main pipe carrying the effluent from the wastewater treatment facility to the discharge sites will be located under streets. The pipes should be located below drinking water distribution pipes, and above any sewage pipes carrying untreated wastewater to avoid contamination of the treated waters (the state typically requires that wastewater pipes be placed at least 18 inches below and 10 feet horizontally separated from the drinking water pipes. Due to the relatively flat topography of the Keeter Property and the varying topography of the New Seabury site it is anticipated that gravity flow would be prohibitively difficult to design. Therefore, it is assumed for this analysis that force mains under pressure will be used to deliver flow to the disposal area.

Table 5-9 below shows each section at New Seabury, the portion each section serves (as a percent of the total area), and the maximum flow capacity for each section. Knowing the capacity and a design flow velocity of 10 feet per second (fps), required pipe diameter can be estimated by solving for the pipe's cross-sectional area using the equation

$$Q = A \cdot V$$

where Q is the flowrate [in cubic feet per second(cfs)], A is the cross-sectional area of the pipe (in sq ft), and V is the velocity of the flow (in fps). The diameter presented here is the diameter of the pipe running from the main distribution pipe (that runs under the street) to each section. Pipe diameters are rounded off to the nearest inch, with a minimum of 3 inches.

As the volumetric flowrate decreases down the length of the main distribution pipe (that runs down, underneath the streets) due to portions of the flow being diverted at each leaching section, so must the size of the main distribution pipe decrease. The main distribution pipe is divided into five sections: Main, I, II, III, IV, and V. Each main pipe section lies between a leaching section. The main pipe is the pipe running from the wastewater treatment facility to

Section A at New Seabury carrying 0.83 MGD, Pipe I is the pipe running from Section A to Section B carrying the remainder, and the other pipes are named respectively. Pipe diameter between each main pipe section is determined by using the same method above to solve for pipe diameter of each leaching section. Adapters should be placed between each main pipe section to accommodate the decreasing diameters. As with the pipes for each leaching section, pipe diameters for the main pipes are rounded off to the nearest inch, with a minimum of 3 inches.

Table 5-9 – Pipe Diameters at New Seabury

New Seabury - 0.83 MGD				
Section (Pipe)	Percent of Total Area (%)	Capacity (MGD)	Flow Velocity (fps)	Pipe Diameter (in)
A	10	0.1	10	3
B	4	0.04	10	3
C	14	0.13	10	3
D	20	0.19	10	3
E	14	0.13	10	3
F	30	0.26	10	3
G	8	0.03	10	3
Main Pipe Section		Capacity (MGD)	10	Pipe Diameter (in)
Main		0.83	10	6
I		0.75	10	5
II		0.71	10	5
III		0.60	10	5
IV		0.43	10	4
V		0.32	10	3

5.4.3 Site 6 – The Keeter Property

5.4.3.1 Layout

Fig. 5-18 – Overhead View of Keeter Property with Designed Sand Beds and Piping

Fig. 5-18 shows the layout of the infiltration beds at the Keeter Property, which includes the width of the base of the berms between each bed, and a suggested layout for the discharge pipes to each infiltration bed. In total, there are fourteen (14) infiltration beds at Site 6. A fence surrounding the infiltration area is not shown in Fig. 5-18 but is required for open infiltration sand beds. The fence must be at least 5 ft in height and include a locking gate. Exact locations of the fence can depend on setback rules for the Town of Mashpee, and possibly also from input of nearby residence that may want to fence set a certain distance from their property line for aesthetic reasons.

Generally speaking, the layout of the Keeter Property is vastly simpler than that at the New Seabury site because there is more open space and the layout is not complicated by existing uses. Because there is more freedom in the placement and orientation of the infiltration basins they can be laid out in a manner that will minimize the difficulty of laying pipe. Indeed, Fig. 5-18 shows that only a single pipe is needed, running the entire length of the discharge site between two rows of infiltration basins.

5.4.3.2 Piping

For the Keeter Property, pipe diameters are determined based on flow capacity. Table 5-10 shows different pipe diameters at different parts of the Keeter property. Each infiltration bed at the Keeter Property can handle 0.2 MGD of treated effluent. Therefore, the pipes leading from the main distribution pipe to each infiltration bed (onto the splash pad) is the minimum of 3 inches in diameter. The main distribution pipe running down the Keeter Property will also decrease in diameter as flowrate decreases in the pipe. Table 5-10 shows the main distribution pipe divided into seven sections (I through VII), each section carrying a different flowrate and a different diameter. As with the pipes at New Seabury, pipe diameters at the Keeter Property are rounded off to the nearest inch, with a minimum of 3 inches.

Table 5-10 – Pipe Diameters at Keeter Property

Keeter Property – 2.38 MGD			
	Capacity (MGD)	Flow Velocity (fps)	Pipe Diameter (in)
Each Bed	0.2	10	3
Section (Pipe)	Capacity (MGD)	Flow Velocity (fps)	Pipe Diameter (in)
I	2.6	10	9
II	2.2	10	9
III	1.8	10	9
IV	1.4	10	9
V	1	10	6
VI	0.6	10	6
VII	0.2	10	3

At the Keeter Property, gate valves will be placed at each infiltration bed, between the main distribution pipe and splash pad. At New Seabury, gate valves will be placed at each series of leaching chambers. Fig. 5-19 shows an example, at Section B of New Seabury, of the location of gate valves at the entrance of each series of chambers. Sections A, C, D, E, F, and G follow the same gate valve layout as Section B. To maintain aesthetic appearance of the golf course, Gate valves will be placed underground contained within a concrete enclosure. When a single chamber series or bed is taken off-line for maintenance or drying purposes, there is sufficient infiltration area at the Keeter Property to make up for the off-line unit. The maximum design peak flow in Mashpee is 3.3 MGD, and the maximum total capacity of the infiltration areas at both sites is 3.63 MGD.

Fig. 5-19 – Leaching Field Chambers with Gate Valves

An alternative to gate valves at the entrance to each leaching trench is to construct distribution boxes. Distribution boxes would control flow for several leaching sections so the boxes could be constructed at several (less than 5) locations at the New Seabury site. The advantage of distribution boxes is that the boxes would be located off of the fairway so that maintenance would not interfere with the main part of the golf course. Another advantage to constructing distribution boxes is that wastewater distribution pipes would not have to be installed under the roads east of the golf course, Troon Way and Shore Drive. The disadvantages are that construction of the distribution boxes may require uprooting existing trees and easements on properties adjacent to the golf course.

5.5 *Recommended Sand Bed and Trench Design*

The infiltration sand beds at Site 6, the Keeter Property, will be square-shaped with 200-ft sides. As shown in Fig. 5-20, each sand bed will have a concrete splash pad where the effluent discharges from the pipe. The splash pads are rectangular, measuring 15ft x 20ft x 2ft. The 2-ft depth of the splash pad includes 1 ft of concrete and 1 ft of base rock (gravel). The purpose of installing a concrete splash pad is to minimize the disturbance that the discharge may cause in the sand by absorbing the kinetic energy from the discharging water before it comes into contact with the sand. In the absence of a splash pad, the discharging water may cause a depression in the sand bed which will lead to uneven distribution of water over the surface of the sand bed. In turn, uneven distribution of water over the surface of the sand bed can lead to unpredicted groundwater mounding.

Figures Fig. 5-21 and Fig. 5-22 show cross-sectional views of berms surrounding sand beds with sand beds on one side and sand beds on either side, respectively. The berms must surround each sand bed and be at least 2 ft in height. The composition of the berm can range from compacted soil, to soil fortified with fly ash, cement (to create soil cement), etc. To incorporate sustainable practice into the construction of this infiltration system, soil excavated from the infiltration bed locations can be recycled and used for the construction of the berms. (Another possible sustainable use for any excess excavated soil would be to re-use the soil for filling any septic tanks that must be taken off-line). However, if there is too much organic content in the soil, then it is not recommended to re-use the soil for berm construction or septic tank filler. When the organic content in soil is high, the volume of the soil can decrease a considerable amount after the organic matter decomposes. The slope of the berm shown in Figures Fig. 5-21 and Fig. 5-22 is arbitrarily chosen at 45 degrees. The 45-degree slope should be sufficient for most berms, however, berm composition determines the actual slope of the berm. The installation of walkways on top of all berms is suggested so that there are pathways to facilitate maintenance and similar activities. The walkways shown in Figures Fig. 5-21 and Fig. 5-22 measure 10 feet wide – this width should be sufficient for a maintenance crew, equipment, machinery, vehicles, etc. The width of the walkways can be increased further to accommodate larger machinery, vehicles, etc.

In each bed, the sand should be at least 2 ft in depth. Assuming a near level soil grade (0-5%) below the sand bed, the depth of the sand should be evenly distributed across the bed which will result in a near level grade of the sand surface. Sand type (size) can vary depending on the level of treatment of the wastewater. At the Keeter property where the effluent is well treated, sand type can range towards the finer sizes. It is imperative that the depth from the soil to the groundwater table (after mounding has occurred) is no less than 4 ft (so for a 2ft deep sand bed, the depth from the sand surface to the groundwater mound would be 6 ft).

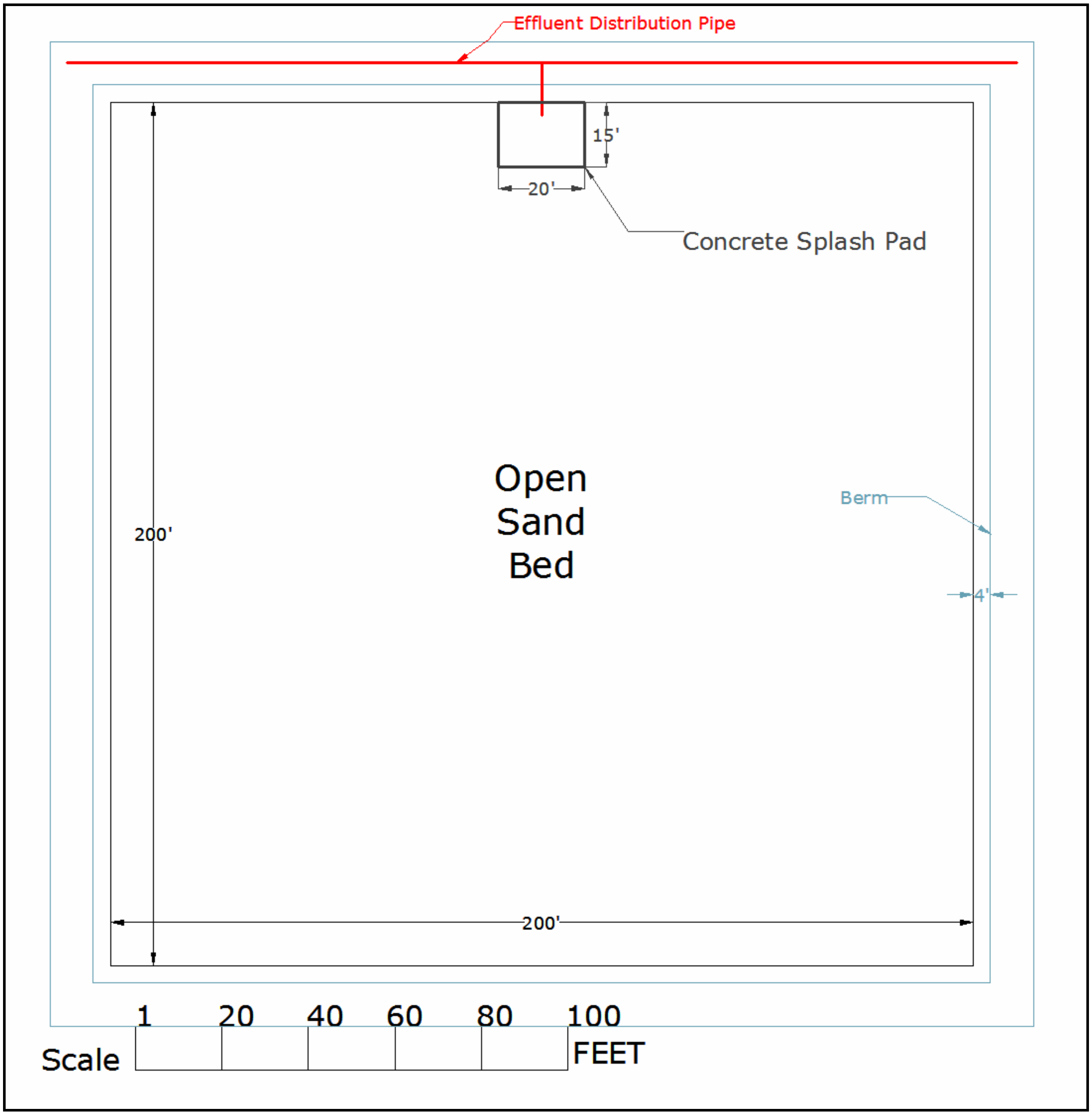

Fig. 5-20 – Plan View of Infiltration Bed

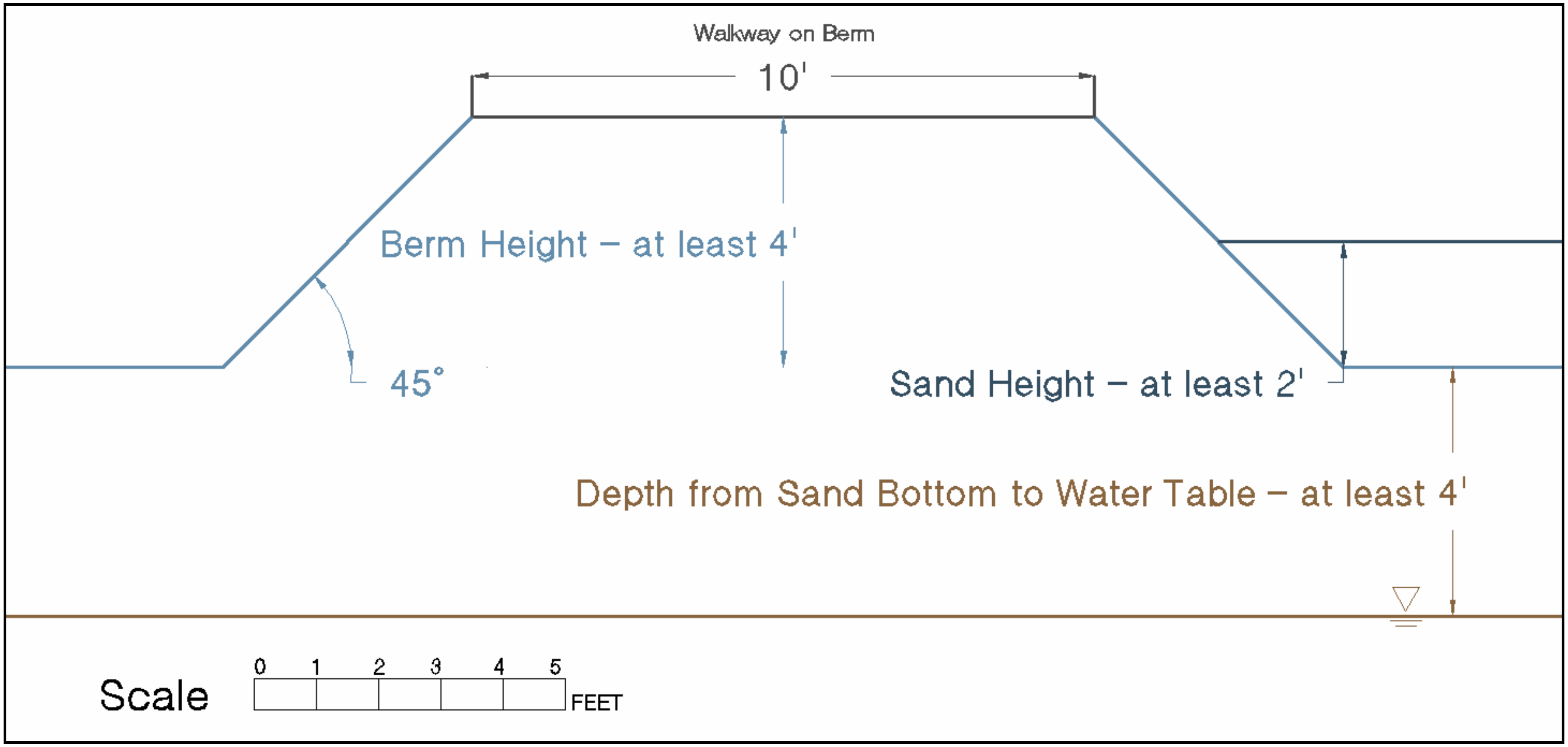

Fig. 5-21 – Berm Cross Section with Sand Bed on One Side

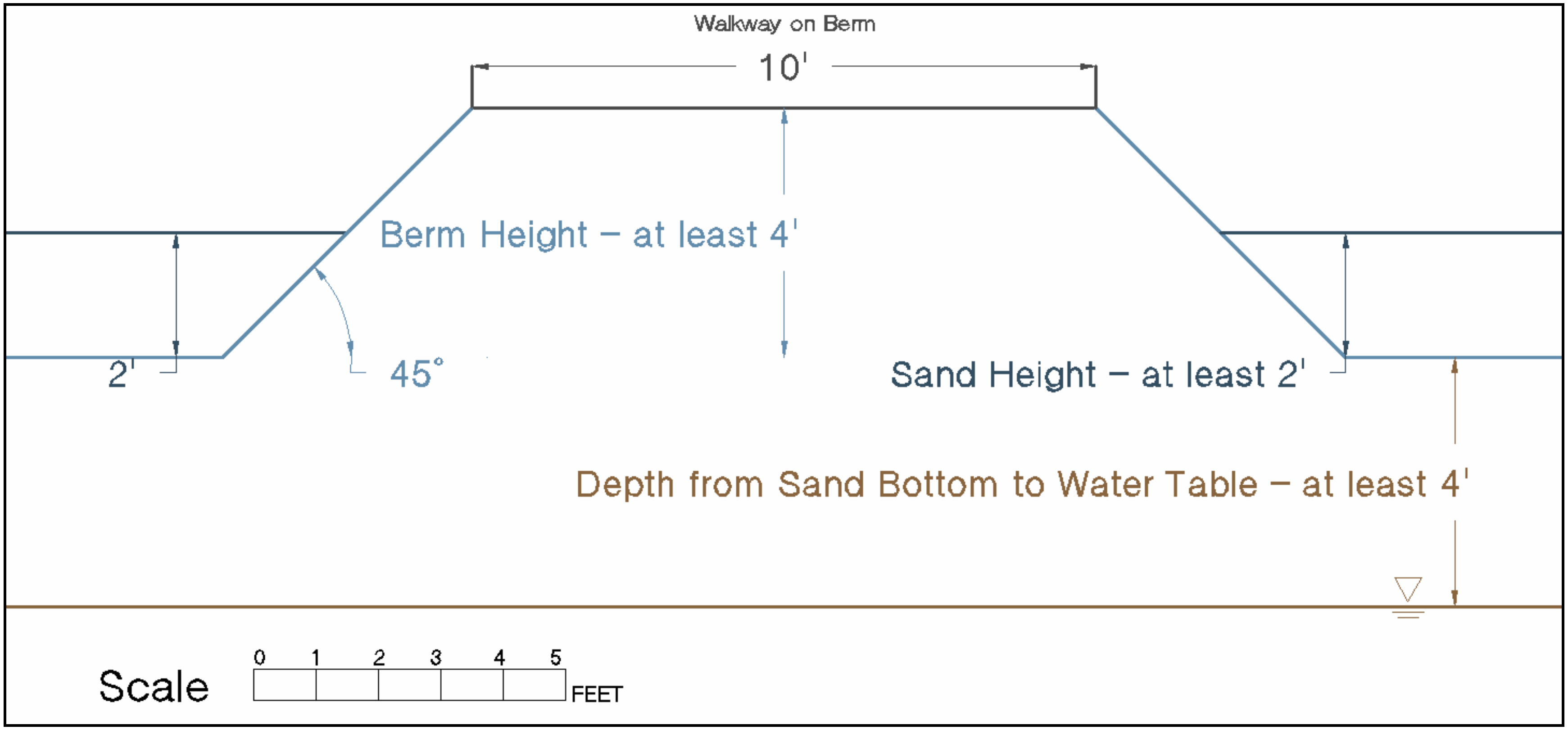

Fig. 5-22 – Berm Cross Section with Sand Bed on Both Sides

The leaching chambers at Site 7 measure 20ft x 4ft with an effective infiltration area of 5 ft around the length of the chamber. Fig. 5-23 below shows two chambers in series, parallel to each other. Because the chambers have an effective infiltration are of 5 ft around the length of the chamber, each series of chambers must be separated by 10 ft.

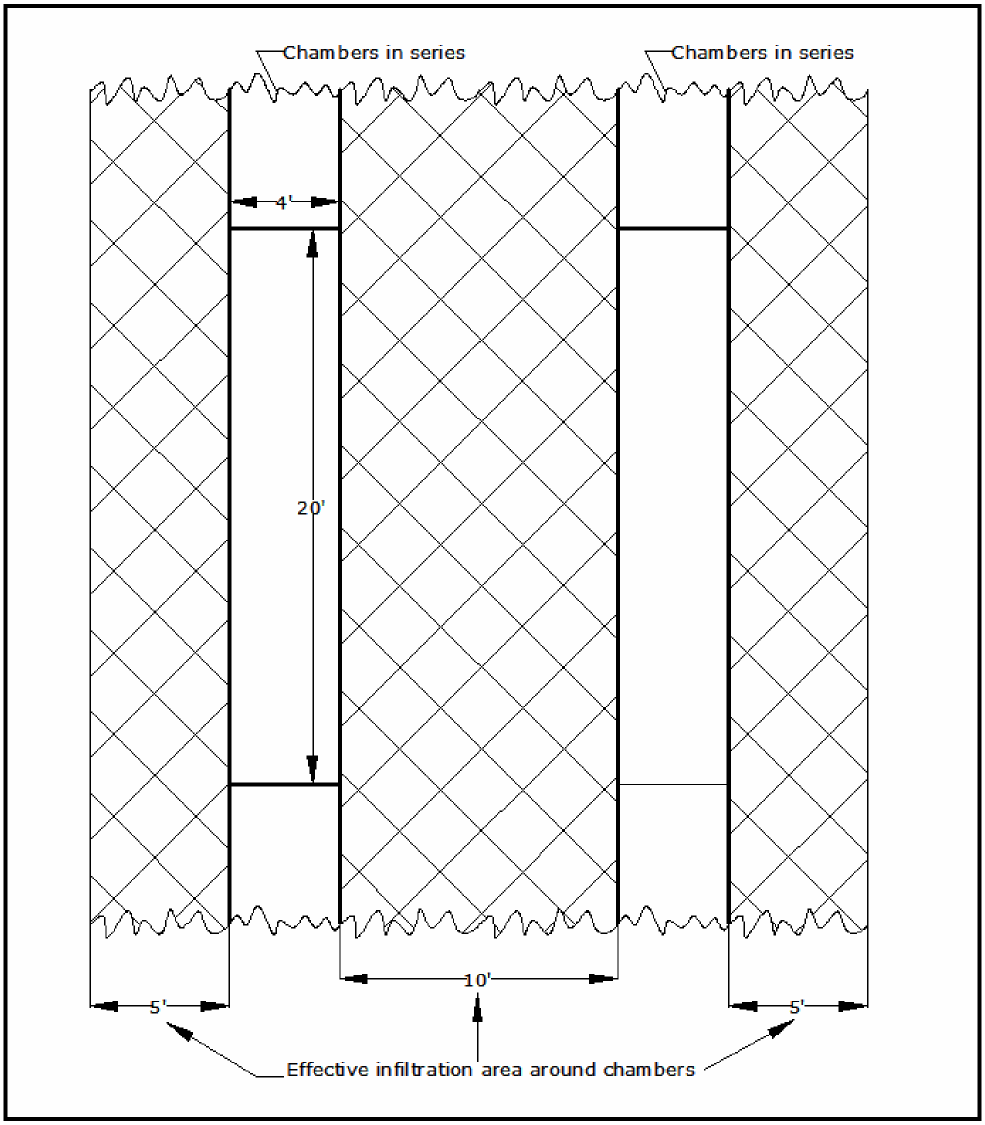

Fig. 5-23 – Plan View of Leaching Chambers

The leaching chambers will be installed in leaching trenches. The trenches will be excavated to a 12-ft depth. To prevent caving-in of vertical excavated trench walls, the trenches will be excavated at a 45-degree angle. Fig. 5-24 shows a cross-sectional view of excavated trenches for leaching chambers. Once leaching chambers are installed in the trenches, the remainder of the trenches will be filled to ground level with the appropriate soil.

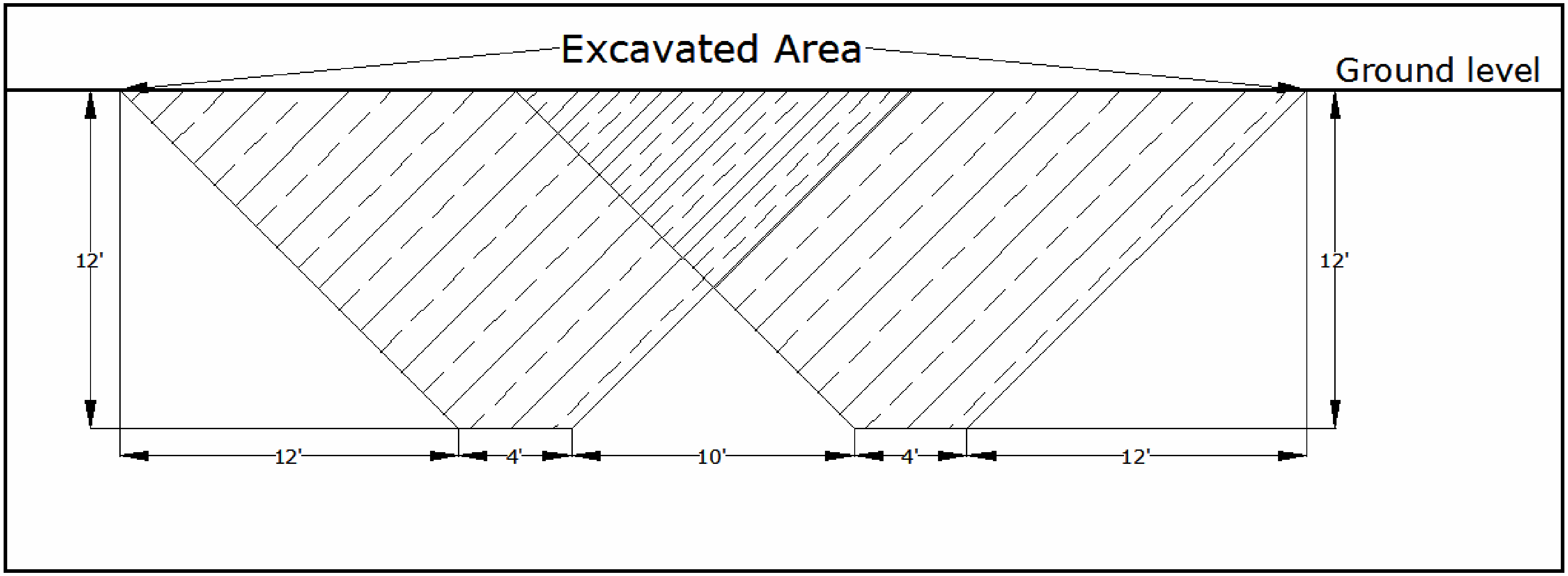

Fig. 5-24 – Cross-sectional View of Trenches

5.6 Preliminary Cost Analysis

An estimate for the New Seabury site and Keeter Property is outlined in a cost analysis in Table 5-11. This estimate, considered to be preliminary, is based on the analysis in Section 4.5, which calculates the approximate on-site cost of creating subsurface leaching trenches and open bed infiltration basins. All of the same items are presented here, with the addition of the major incidental expense of rebuilding the golf course on top of the proposed subsurface leaching trenches. This cost analysis does not include the construction of a treatment facility, pumping stations, a sewer system, connecting all the residential and commercial units to the sewer system, or taking off-line the remaining septic systems in use.

Table 5-11 – Scenario Construction Cost

Scenario II for Site 6 - Keeter Property					
Items	**Unit**	**Quantity**	**1999 Unit**	**1999 Cost**	**Mashpee 2009 Cost**
Open Infiltration Sand Bed	SF	560,000			
Clearing & Grubbing	ACRE	12.9	6925	89000	$ 141,000
Excavation to subgrade	CY	41,500	3.45	143000	$ 227,000
On-site Piping (10" D avg)	LF	2,000	13.65	27000	$ 43,000
Piping accessories	LS	1	40000	40000	$ 63,000
4'x3' Earth Berm	LF	6,000	25.00	150000	$ 238,000
Access Road Earthwork	CY	2,800	3.05	9000	$ 14,000
Access Road gravel base	CY	667	26.50	18000	$ 29,000
Access road asphalt binder & top	SF	16,000	1.64	26000	$ 41,000
Sand Backfill	CY	41,500	17.55	728000	$ 1,153,000
5' Perimeter Fence	LF	4,800	13.00	62000	$ 98,000
20'x15'x1' CIP conc splash pads	EA	14	1479.70	21000	$ 33,000
1' Splash pad comp. gravel base	CY	185	26.50	5000	$ 7,000
Total Site 6	SF	560,000	**4.01**	(Local Unit)	**$ 2,087,000**
Direct Comparison Price	Gal/Day	1	**0.80**		
Total Site 6					
Scenario II for Site 7 - New Seabury CC					
Items	**Unit**	**Quantity**	**1999 Unit**	**1999 Cost**	**Mashpee Local Cost**
Leaching trench effective area	SF	332,640			
Leaching trench Chambers (20'x4')	EA	1,188	1350	1604000	$ 2,540,000
Excavation (12' deep trenches)	CY	168,960	5.70	963000	$ 1,525,000
Backfill	CY	168,960	1.32	223000	$ 353,000
On-site Piping (10" D)	LF	3,700	13.65	51000	$ 80,000
On-site Piping (6" D)	LF	900	11.15	10000	$ 16,000
Piping accessories (valves & wyes)	LS	1	60000	60000	$ 95,000
Total Site 7	SF	332,640	**13.86**	(Local Unit)	**$ 4,609,000**
Direct Comparison Price	Gal/Day	1	**1.40**		
Golf Course Restoration					
Golf Course Irrigation System	9 Holes	1/3	154500	52000	$ 82,000
Fine Grading & sodding	MSF	375	1550	581000	$ 920,000
**Backfill included above					
Total Restoration Cost:	LS	1			**$ 1,000,000**
Grand Total					**$ 8,000,000**

The expense of constructing subsurface leaching trenches will be balanced against the cost savings gained by the ability to treat this 0.83 MGD to a lesser extent (10 mg/L total Nitrogen at site 7, rather than 3 mg/L at site 6). Calculation of this savings, as well as the maintenance and off-site costs associated with the project, are considerations for further study.

The proposed cost of $13.3 million is a balance between cost and effectiveness. Over 80% of the cost is allocated toward Site 7, where only 0.83 MGD capacity is proposed. This is less than 1/3 of the peak flow capacity required of the system. However, this site clearly provides a unique opportunity to circumnavigate the problem of estuarine nutrient overload. Therefore, the scenario is to make use of this site year-round, while the much less expensive Site 6 would be used for the overflow that cannot be discharged at Site 7. Therefore, the majority of the expense is appropriately being allocated to the more critical of the two sites. It would perhaps be inappropriate for the entire peak flow to be discharged through such expensive technology, since a large fraction of this would only be used during a short season. However, this scenario presents a highly utilized, expensive core capacity, to be supplemented seasonally by less expensive, less optimal technology.

5.7 Summary of Proposed Design

The preliminary designs in this report include designs for subsurface infiltration trenches and open infiltration sand beds. These designs follow the guidelines specified in *Guidelines for the Design, Construction, Operation, and Maintenance of Small Wastewater Treatment Facilities with Land Disposal* by the Massachusetts Department of Environmental Protection. In the case where some components of either subsurface or open infiltration systems were not specifically discussed in the *Guidelines for Design*, engineering judgment was used. For example, the *Guidelines for Design* does not specify possible materials for berm construction. With general knowledge about soil stability and the desire to minimize costs, without minimizing soundness, the conclusion that fortified soil, with soil recycled from the soil excavated for the sand beds, should be used as the building material of the berms.

The preliminary pipe and sand bed designs in Sections 5.4 & 5.5 of this report are a good starting point for further development and refinement. It is not expected that construction will be carried out directly from the designs in this report. Rather, the purpose of creating these designs is to show the feasibility of constructing infiltration systems at these sites. Also, from these designs, required construction materials are measured and quantified to produce the preliminary cost analysis in Section 5.6.

6 Conclusions

This project includes an evaluation of candidate sites for substrate disposal for the town of Mashpee, Massachusetts. It is apparent from the results of the computer modeling that no individual site represents a solution to the problems facing the Town of Mashpee. Disposal of the town's average wastewater flow, estimated for this report, at 1.65MGD in many of the sites under consideration would result in an increase in the health of several of the embayments of Popponesset or Waquoit Bays at the cost of the viability of one. As per the requirements of this project, no discharge scenario is acceptable if it sacrifices the natural ecosystem of one of the embayments or deprives the town of the use of these precious natural resources. Furthermore, there are extenuating circumstances at several of the discharge sites, such as the proximity to the jet fuel contaminant plume at the Ashumet Road/Ball Fields properties, the drinking well located just south of the Keeter Property, or the fact that young children will be gathering at the High School property every day. Furthermore, it must be recognized that it may not be possible to build at the New Seabury Country Club due to cost or institutional constraints despite the fact that it offers a very favorable option for effluent disposal.

Regardless, this report demonstrates that, all other obstacles aside, the Town of Mashpee could implement a plan to collect and treat wastewater and discharge treated effluent into the ground. Mashpee can discharge at the Keeter Property and New Seabury sites, which could help to reduce nutrient loading and eliminate eutrophication and algal blooms in the embayments of Popponesset and Eastern Waquoit Bay. When reviewing the costs for the construction (and in the case of New Seabury, also reconstruction) infiltration systems at these two sites, the final cost may seem high. However, there is also a great value to the people of the Town of Mashpee in protecting the quality of water in the embayments of the bays. The Keeter Property and New Seabury are appropriate discharge sites with sufficient capacity to handle the town's wastewater load, and discharging into these sites should, over time, help protect the health of the estuaries.

7 Works Cited

Arceivala, S. J. Wastewater Treatment and Disposal. Ed. Paul N. Cheremisinoff. New York and Basel: Marcel Dekker, Inc., 1981.

Brownlow, A. H. Cape Cod Environmental Atlas. Boston, MA: Department of Geology, Boston University, 1979.

Cape Cod Map, 2007.

Cooper, PF. "Historical Aspects of Wastewater Treatment." Decentralized Sanitation and Reuse (2001): 11-38. . Google.

Craun, M. F., et al. "Waterborne Outbreaks Reported in the United States." Journal of water and health 4 Suppl 2 (2006): 19-30.

Davis, Mackenzie L., and Susan J. Masten. Principles of Environmental Engineering and Science. 1st ed. New York, NY: McGraw-Hill, 2004.

Droste, Ronald L. Theory and Practice of Water and Wastewater Treatment. Canada: John Wiley & Sons, Inc., 1997.

Guidelines for the Design, Construction, Operation, and Maintenance of Small Wastewater Treatment Facilities with Land Disposal., 2004.

Howes B., S.W. Kelley, J.S. Ramsey, R. Samimy, D. Schlezinger, T. Ruthven, E. Eichner. Linked Watershed-Embayment Model to Determine Critical Nitrogen Loading Thresholds for the Quashnet River, Hamblin Pond, and Jehu Pond, in the Waquoit Bay System in the Towns of Mashpee and Falmouth, Massachusetts. Boston, MA: Massachusetts Estuaries Project, Massachusetts Department of Environmental Protection, 2005.

Howes, Brian, et al. Linked Watershed-Embayment Model to Determine Critical Nitrogen Loading Thresholds for Popponesset Bay, Mashpee and Barnstable, Massachusetts. Commonwealth of Massachusetts: Massachusetts Estuaries Project, 2004.

Maiorano, Jaclyn M., et al. Development of a Long Term Plan for Remediation of the SD-5 Plume at the Massachusetts Military Reservation. Ed. Paul H. Mathisen. Worcester, MA: Worcester Polytechnic Institute, 2003.

Mashpee Environmental Coalition. “Comprehensive Watershed Nitrogen and Wastewater Management Plan.” Mashpee Environmental Coalition. <http://www.mashpeemec.us/S%20C%20Plan.htm>.

Massachusetts Department of Environmental Protection. Drinking Water. Trans. Department of Environmental Protection. Water Quality Regulations ed. Vol. 310CMR. Commonwealth of Massachusetts, 2006.

Massachusetts Department of Environmental Protection. Guidelines for the Design, Construction, Operation, and Maintenance of Small Wastewater Treatment Facilities with Land Disposal. Department of Environmental Protection. Commonwealth of Massachusetts Department of Environmental Protection Division of Watershed Permitting, 2004.

Massachusetts Department of Environmental Protection. "Massachusetts Estuaries Project." Massachusetts Department of Environmental Protection and the School for Marine Science and Technology. <http://www.oceanscience.net/estuaries/index.htm>.

Massachusetts Estuaries Project. "About the Massachusetts Estuaries Project." <http://www.oceanscience.net/estuaries/about.htm>.

Massachusetts General Court. Massachusetts Ocean Sanctuaries Act. Trans. Massachusetts Senate. Vol. 132A. Massachusetts.

Massachusetts Military Reservation. “MMR Site Description.” Air Force Center for Engineering and the Environment. <http://www.mmr.org/IRP/about/descrip.htm>.

Metcalf & Eddy, Inc. Wastewater Engineering: Treatment, Disposal, and Reuse. Ed. George Tchobanoglous and Franklin L. Burton. 3rd ed. New York: McGraw-Hill, Inc., 1991.

National Park Service. "Cape Cod Geological Fieldnotes." National Park Service. 04 Jan 2005 2005. <http://www2.nature.nps.gov/geology/parks/caco/>.

Oldale, Robert N. Geological History of Cape Cod, Massachusetts. US Geological Survey, 2001. USGS Publications Database01 Nov 2007.

Jim Riesterer. "State Plane Coordinate System." Sep. 18 2003 2003. <http://geology.isu.edu/geostac/Field_Exercise/topomaps/state_plane.htm>.

Savoie, Jennifer, and Denis R. LeBlanc. Water-Quality Data and Methods of Analysis for Samples Collected Near a Plume of Sewage-Contaminated Ground Water, Ashumet Valley, Cape Cod, Massachusetts. Vol. WRIR 97-4269. Cape Cod, MA: US Geological Survey, 1995.

Smil, Vaclav. Cycles of Life : Civilization and the Biosphere. New York : Scientific American Library: Distributed by W.H. Freeman and Co., 1997.

USDA Natural Resources Conservation Service. Final Watershed Plan and Areawide Environmental Impact Statement. Washington, D.C.:, 2006. USDA NRCS Publications Database.

U.S. Geological Survey. "USGS Learning Web :: Lesson Plans :: Land and People :: Teachers :: Cape Cod." 01/29/2002 2002. <http://interactive2.usgs.gov/learningweb/teachers/landpeople_teachers_cc_3.htm>.

U.S. Soil Conservation Service. Barnstable County Soil Survey. Washington, D.C.: U.S. Department of Agriculture, 1993.

Valiela, I., et al. "Transport of Groundwater-Borne Nutrients from Watersheds and their Effects on Coastal Waters." Biodegradation 10.3 (1990): 177-97. September 24, 2007.

Valiela, I., et al. "Couplings of Watersheds and Coastal Waters: Sources and Consequences of Nutrient Enrichment in Waquoit Bay, Massachusetts." Estuaries 15.4 (1992): 443-57.

Walter, Donald A., and Ann T. Whealan. Simulated Water Sources and Effects of Pumping on Surface and Ground Water, Sagamore and Monomoy Flow Lenses, Cape Cod, Massachusetts. Vol. SIR 2004-5181. US Geological Survey, 2005.

Woods Hole Research Center. "Current Land Cover and Recent Land Cover Change." Woods Hole Research Center. 2005. <http://www.whrc.org/capecod/land_cover_population/land_cover_change.htm>.

8 Appendix

8.1 Appendix I – Calculations

New Seabury Country Club

Hole 1
Universal Transverse Mercator Coordinates

Point	meters E	meters N	Yb*(-Xa+Xc)	
1	376664.43	4601931.12	217809400	
2	376726.52	4602015.77	331759317	
3	376736.52	4602082.46	182472570	
4	376766.17	4602122.11	357999079	
5	376814.31	4602132.91	427354062	
6	376859.03	4602146.59	297160605	
7	376878.88	4602158.27	229647698	
8	376908.93	4602154.82	370381420	
9	376959.36	4602166.11	247320407	
10	376962.67	4602158.75	-685629611	
11	376810.38	4602090.96	-948951156	
12	376756.47	4602025.85	-418140069	
13	376719.52	4601936.9	-142890141	
14	376725.42	4601914.31	-185595204	
15	376679.19	4601899.16	-280669830	
		Double Area	28547	SM
		Area	14274	SM
		Area	153639	SF

Hole 2

Point	Meters E	Meters N	Yb*(-Xa+Xc)
1	376519.09	4602358.14	153534668
2	376552.45	4602338.81	260584423
3	376575.71	4602311.15	271214196
4	376611.38	4602285.61	668067779
5	376720.87	4602181.77	558888954
6	376732.82	4602165.49	17488229
7	376724.67	4602126.88	-92134580
8	376712.80	4602135.93	14956942
9	376727.92	4602143.09	73174075
10	376728.70	4602147.11	111187874
11	376752.08	4602130.65	-46067328
12	376718.69	4602096.77	-205115453
13	376707.51	4602052.12	-125267859
14	376691.47	4602016.87	-74322572
15	376691.36	4601967.81	-124575269
16	376664.40	4601931.16	-245973221
17	376637.91	4601984.68	39116870
18	376672.90	4602018.06	180721249

19	376677.18	4602114.56	-5936728	
20	376671.61	4602145.44	-4187952	
21	376676.27	4602174.81	-52188662	
22	376660.27	4602180.48	-640485457	
23	376537.10	4602291.41	-749575202	
24	376497.40	4602306.53	-263390003	
25	376479.87	4602328.19	99824498	
26	376519.09	4602358.14	180504486	
1	376519.09	4602358.14		
		Double Area	43958	SM
		Area	21979	SM
		Area	236581	SF

Hole 3

Point	Meters E	Meters N	Yb*(-Xa+Xc)	
1	376479.93	4602328.25	-232739740	
2	376468.5	4602347.96	166927161	
3	376516.2	4602440.14	331283641	
4	376540.48	4602464.16	99643349	
5	376537.85	4602478.57	65125072	
6	376554.63	4602502.12	120447480	
7	376564.02	4602494.39	40225801	
8	376563.37	4602512.45	98769917	
9	376585.48	4602532.49	144381444	
10	376594.74	4602526.47	103602871	
11	376607.99	4602536.1	140515427	
12	376625.27	4602522.91	37694663	
13	376616.18	4602494.09	-163848790	
14	376589.67	4602460.96	-244850923	
15	376562.98	4602443.76	-194545298	
16	376547.4	4602411.94	-171808038	
17	376525.65	4602407.15	-171669787	
18	376510.1	4602377.51	-55504673	
19	376513.59	4602364.83	41283213	
20	376519.07	4602358.19	-154915377	
1	376479.93	4602328.25		
		Double Area	17415	SM
		Area	8707	SM
		Area	93725	SF

Keeter Property

Point	Meters E	Meters N	Yb*(-Xa+Xc)	
1	376301	4604700	1984625700	
2	376732	4604792	1970850976	
3	376729	4604713	-1031455712	
4	376508	4604586	-1970762808	
5	376301	4604640	-953160480	
1	376301	4604700		
		Double Area	97676	SM

Area	48838	SM
Area	525688	SF

Scenario 1 Average Flow Calculations
Fill New Seabury Country Club up to capacity year-round
Dispose of remaining effluent in Keeter Property

Variable	Value	Units	Source
Average Flow	1.65	MGD	Stearns & Wheler
Peak Factor	2.00	Ratio	Stearns & Wheler
Peak Flow	3.30	MGD	Calculated
In-season factor	2.320	Ratio	Walter & Whealan Page 35
In-Season length	0.417	Years	Walter & Whealan Page 35
In-Season flow	2.470	MGD	Calculated
Out-of-Season flow	1.065	MGD	Calculated

New Seabury Country Club

Total Area	483,945	SF	Google Earth - Polygons
Flow Capacity	1.210	MGD	Google Earth - Polygons
Design Average Flow	**1.12**	**MGD**	Chosen
Design Area	**448403**	**SF**	Chosen
Infiltration Rate	**2.5**	**Gal/D/SF**	Stearns & Wheler
Infiltration Rate	**37205**	**mm/year**	Converted

Visual Modflow Model

Grid size	160000	SF
Model nodes	8	EA
Model area	1280000	SF
Model recharge	13033.45	mm/year
Zone 2 (existing) recharge	692.46	mm/year
Zone 8 recharge	13725.91	mm/year

Keeter Property

Design Flow	**0.53**	**MGD**	Remaining Flow
Infiltration Rate	5	Gal/D/SF	Stearns & Wheler
Infiltration Rate	74409	mm/year	Converted
Required Area	105798.69	SF	Calculated
Safety Flow	**0.33**	**MGD**	10% capacity above peak flow, for repairs & unexpectedly high volume
Safety Area	66000		
Design Area	**487350**		

Visual Modflow Model

Grid size	160000	SF
Model nodes	6	EA
Model area	960000	SF
Model recharge	8200.39	mm/year
Zone 2 (existing) recharge	692.46	mm/year
Zone 7 recharge	8892.85	mm/year