REGIONAL FLOW MODEL OF WEST CAPE COD AND OPTIMIZATION ANALYSIS

by

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Abstract

The West Cape Cod aquifer is shallow, unconfined and particularly vulnerable to contamination because of its high permeability. Past activities have caused the contamination of part of the aquifer and the limited amount of high quality drinking water is one of the current major problems, despite the plentiful natural supply of water. Focus of this work is a water supply analysis for the town of Falmouth, which has the most critical situation among the West Cape Cod towns. A groundwater hydraulic management model has been used to evaluate an optimized pumping scheme that maximizes the total water supplied to Falmouth, without drawing upon polluted water. The pumping rates from proposed wells have been maximized with a linear programming algorithm. The problem's constraints have been determined through a groundwater model. The regional flow is described with a 3-D finite element code, and is directly incorporated into the optimization with a response matrix approach. The constraints require that no water is drawn from the portion of the aquifer that could be polluted because of the presence, at the mound of the aquifer, of the plumes emanated from the Massachusetts Military Reservation. Different pumping strategies have been evaluated; the overall results show that enough drinking water could be available to supply the water demand estimated by the Water Resources Office, Cape Cod Commission, for the year 2020.

Thesis Supervisor: Dennis B. McLaughlin
Title: Professor of Civil and Environmental Engineering

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Chapter 1

Introduction

1.1 Background

Although the Cape Cod aquifer has a plentiful supply of water, one of the major problems in the area is its limited amount of high quality drinking water. The activities of the Massachusetts Military Reservation (MMR), hazardous waste disposal, and landfills and sewage-treatment facilities have caused the development of 11 distinct plumes and the contamination of over 65 billion gallons of water. Contaminants in the aquifer include chlorinated solvents (mainly tetrachloroethylene (PCE), trichloroethylene (TCE), 1,2-dichloroethylene (DCE), and carbon tetrachloride (CCl_4)) and fuel components, such as benzene, ethylbenzene, toluene and xylenes (Harris and Steeves, 1994). Some of these plumes do not represent a public or health risk; one has a containment system in operation, two are not yet completely defined, and five of them are objects of MMR's Strategic Plan, July 1996. The intent of this plan is to cease the movement of the Main Base Landfill plume (LF-1), the Chemical Spill 10 plume (CS-10), the Ashumet Valley plume, the Storm Drain 5 plume (SD-5) and the Fuel Spill 12 plume (CS-12).

In 1978 the Massachusetts Department of Environmental Protection (DEP) decided to close a drinking water supply well in the town of Falmouth on Cape Cod because of the presence of detergents coming from the Massachusetts Military Reservation's wastewater treatment plant. In 1982 the Department of Defense established the Installation Restoration Program (IRP) to investigate contaminated areas on the Cape and to propose clean up plans for sites having the potential for environmental problems (currently there are 79 of these sites). Since the program was founded over $165 million have been spent by the Department of Defense. Numerous investigations have been conducted by the IRP and by the U.S. Geological Survey (USGS) to study the groundwater system near the military area and across the Cape, to determine the plumes' movements and to evaluate local and regional water supply issues. The first specific characterization of the plume caused by the sewage treatment facility at the Massachusetts Military Reservation was provided by LeBlanc in 1984. Since then many others studies of the Cape situation have been conducted. The more recent investigations on the Cape Cod flow system have been provided by Masterson and Barlow (1994) and by Masterson, Walter and Savoie (1996). Although these studies have contributed considerably to understand the natural situation, some issues are still unresolved. Present concerns are the future path of the plumes, the determination of critical hydraulic properties that affect the regional flow and the plumes' movements, and finally a global analysis of the drinking water supply development, the groundwater flow, the remediation schemes and the contaminant migration.

The Massachusetts Water Resources Commission (WRC) is in charge of evaluating and planning the water resources in Cape Cod with the technical support of the Office of Water Resources (OWR), in the Department of Environmental Management, which develops the basin plans for each of the 27 major river basins in the United States. The Cape population is growing, and water requests are increasing sharply (especially during the summer months). Different water supply/demand projections have been formulated for the year 2020 and, even though the problem is still under study, it is clear that water needs are increasing particularly fast. Groundwater is abundant, but, since the aquifer is shallow and permeable, it is highly vulnerable to contamination. Therefore, present and future drinking water needs and groundwater resources protection are currently important concerns.

1.2 Analysis Description

The main objective of this study is a water supply analysis for the town of Falmouth and the formulation of an optimization model to evaluate a pumping scheme for proposed wells that will yield the maximum rate without drawing upon polluted water. The optimization was performed with a hydraulic management model. As it is better described in Section 5.2 in this type of model the response of the natural flow system to pumping rates is incorporated into a linear optimization algorithm. In order to evaluate the aquifer response to pumping stress a computer representation of West Cape Cod was created with a numerical code. Therefore, the comprehension of the natural flow in the West Cape Cod aquifer represents an important aspect of the conducted analysis. Essential to the understanding of this flow was the investigation of the geology, hydrogeology and hydrology of the aquifer.

The site under study is described in Sections 2.2, 2.3 and 2.4; particular effort was put into understanding the geology and the hydrogeology of the aquifer following the publications of the United States Geologic Survey (USGS). The comprehension of these characteristics was fundamental to develop an adequate conceptual flow model, that is described in Section 4.1. Section 3.1 and Section 3.2 describe the fundamentals of groundwater computer modeling and the DYN code that was used to model the West Cape Cod flow. The results of the model simulation are presented in Section 4.2.

Section 5.1 describes the water supply problem in the town of Falmouth. A general introduction about water resources planning is presented in Section 5.2 while Section 5.2.1 describes the linear optimization model formulation. The water supply analysis conducted and the obtained results are the topic of Section 5.2.2.

Finally the conclusions that can be inferred both from the groundwater flow model and from the optimization analysis are presented in Chapter 6, along with comments on this work and proposal for future water management investigations.

While the site description and the geology, hydrogeology and hydrology of the aquifer have been deducted from available literature, the development of the groundwater model, its calibration and the management analysis are my personal contributions to this work.

Chapter 2

West Cape Cod: Site Description

2.1 Location

The peninsula of Cape Cod extends into the Atlantic Ocean from the southeastern part of Massachusetts (Figure 1). Cape Cod includes 15 towns in Barntsable County within an area of 440 square miles. From east to west it measures about 40 miles and from north to south about 25 miles, the maximum surface altitude is 309 feet above sea

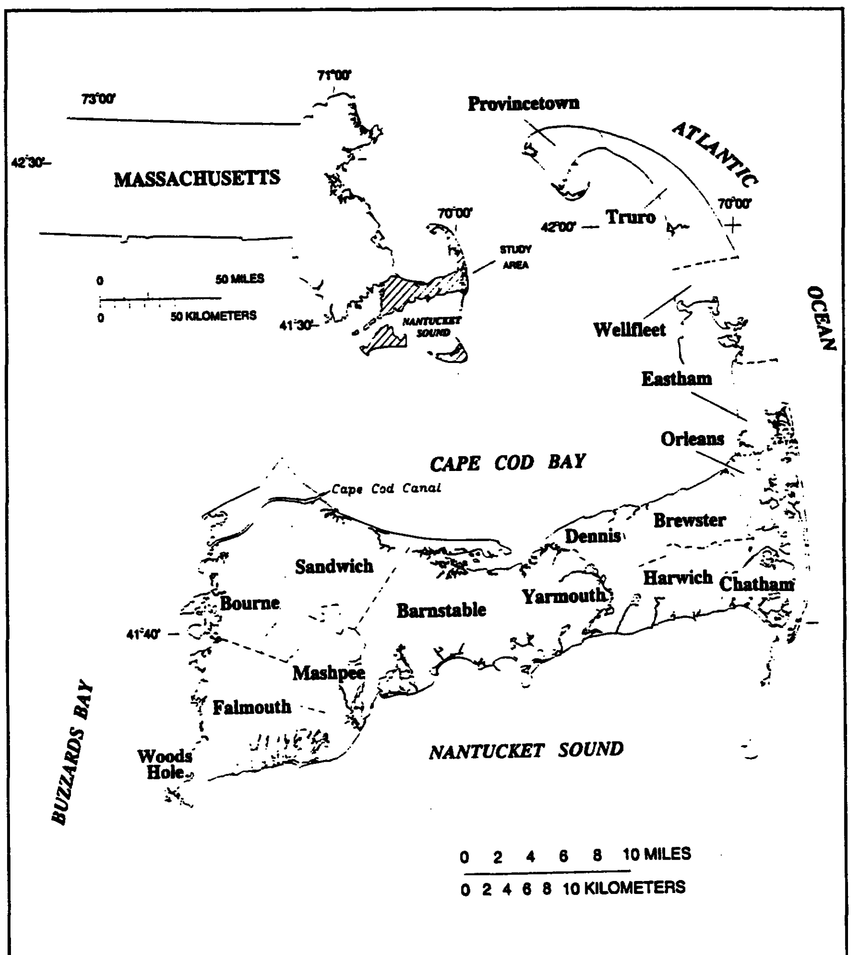

Figure 1
Location of Cape Cod and identification of the towns included in the area (Masterson and Barlow, 1994).

level. A sea-level canal (Cape Cod Canal) separates the peninsula from the mainland, and therefore precipitation is the only source of water to the aquifer. Cape Cod, according to the USGS definition, consists of six groundwater flow cells that are hydrogeologically different and hydraulically independent (Figure 2). These cells are: West Cape, East Cape, Eastwam, Wellfleet, Truro and Provincetown. The West Cape cell extends from the Cape Cod Canal to the Bass River and includes the towns of Bourne, Sandwich, Falmouth, Mashpee, Barnstable and the most of Yarmouth. The town of Falmouth is in the northeastern-most part of the flow cell (Figure 1).

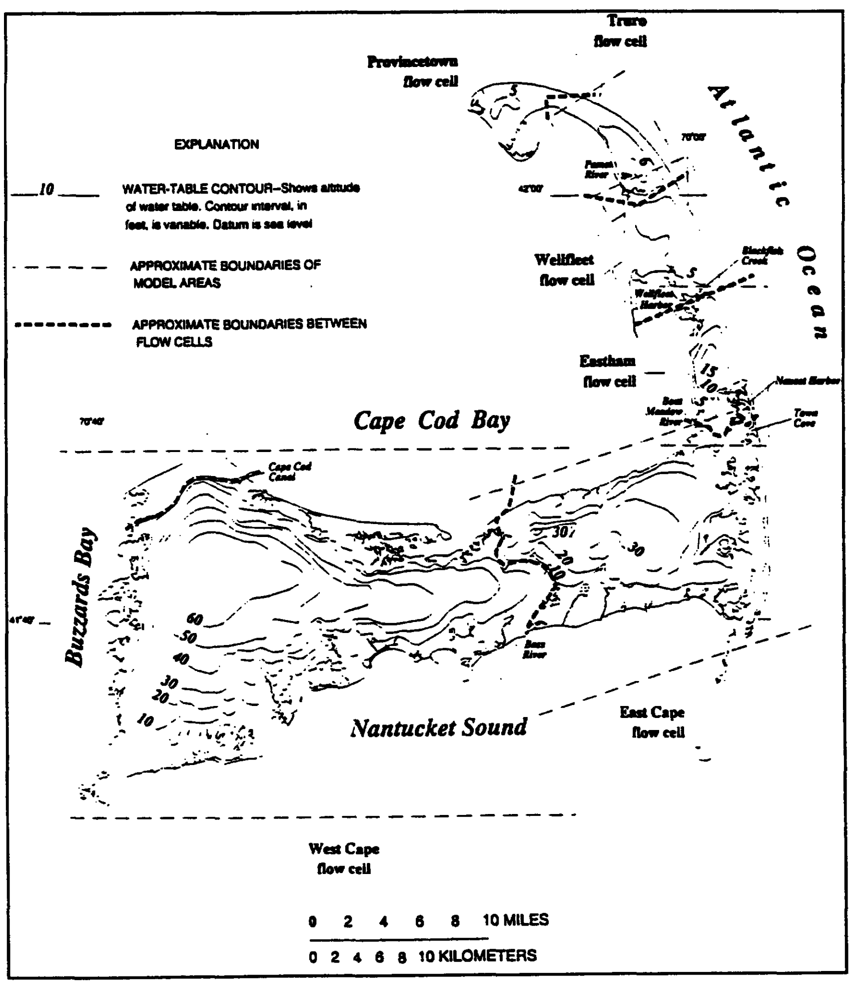

Figure 2
Identification of flow cells in Cape Cod and water table configuration in May 25-27, 1976 (Masterson and Barlow, 1994).

2.2 Geology

The western Cape Cod aquifer consists of several different geologic units produced by periods of glacial deposition, erosion and redeposition. The sediments were deposited during the late Wisconsinan glaciation and range in size from clay to boulders. The movements of the ice recession that formed most of the Cape deposits are showed in Figure 3. The Buzzards Bay moraine and the Sandwich moraine were deposited later, during minor advances of the ice lobes (Masterson et al., 1996).

The basement beneath the Cape is probably composed of metamorphic and igneous rock. The depth of the basement ranges from 80 feet to more than 800 feet below sea level (but in the west area to only 500 feet, Oldale, 1969). Above the bedrock there is a relatively thin layer of till, a poorly sorted mixture of sand, gravel, silt, clay and boulders.

The most important geologic units in western Cape Cod are: Mashpee pitted plain

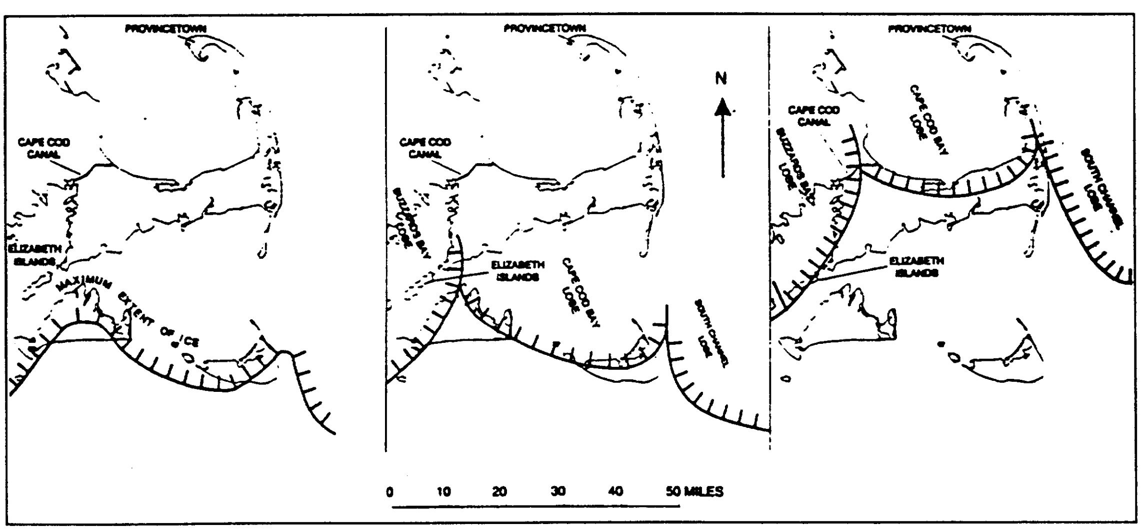

Figure 3 - Ice recession and lobe formation in the Cape Cod area (Masterson and Barlow, 1994).

deposits, Barnstable plain deposits, Harwich outwash plain deposits, Buzzards Bay moraine deposits, Sandwich moraine deposits, Buzzards Bay outwash, Cape Cod Bay lake deposits and marsh and swamp deposits (Oldale and Barlow, 1986 and LeBlanc et al., 1986).

The Mashpee pitted plain is the geologic unit that extends most widely across the west Cape Cod area. The sediments were deposited as stratified drift by meltwater; they are coarsening upward and become finer in texture as one moves north to south. These deposits can be subdivided into three types of sedimentary facies: topset, foreset and bottomset (Figure 4). Facies refers to "bodies of sediments with similar lithologic characteristics that were deposited contemporaneously in similar depositional environments deposit" (Masterson et al., 1996). The topset deposits, consisting of fine-to-coarse sand and gravel, range in thickness from 0 feet at the moraine/outwash contact to about 200 feet near the Nantucket Sound ice contact. The sediments of the foreset

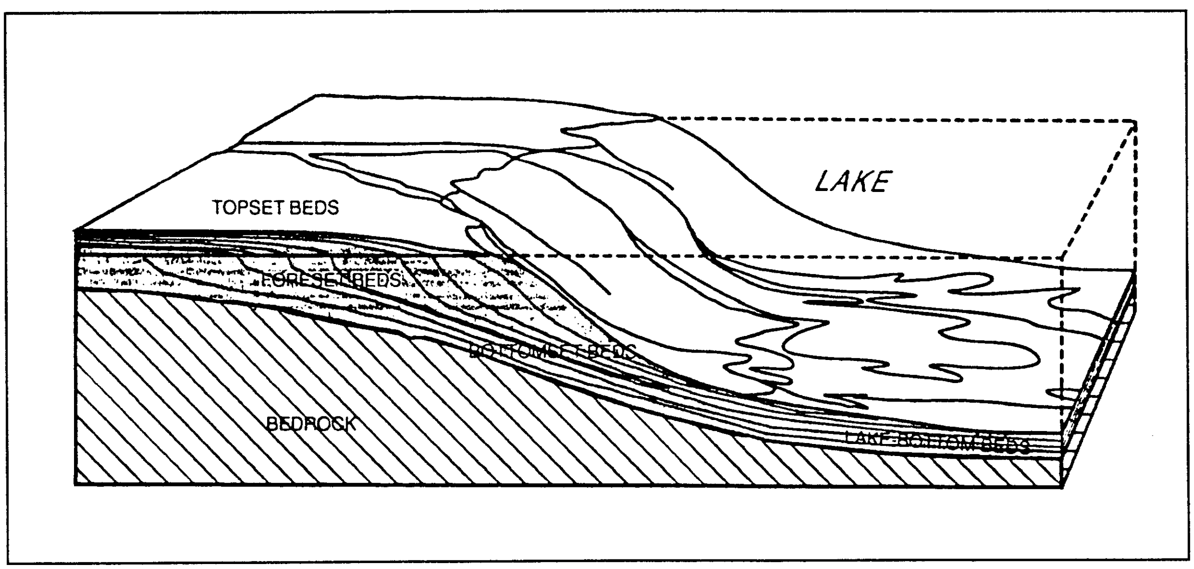

Figure 4 - Generalized sediments sequence in meltwater deposits (Masterson and Barlow, 1994).

facies are mainly medium and fine sand with some silt, whereas the bottomset sedimentary facies sediments consist of horizontal layers of fine sand and silt. The deposits of medium and fine sand and silt range from 0 feet (at the source of the Mashpee pitted plain) to about 150 feet thick in the zone south of Mashpee Pond. Between the bottomset sedimentary facies and the basal till there are the lacustrine lake-bottomdeposits that consist of horizontal depositions of silt and clay with minor amounts of fine sand. The thickness of these deposits ranges from 0 feet at the contact between the moraine and the deltaic deposits to 200 feet near the Nantucket Sound ice-contact deposits (a minor formation on the Nantucket Sound shore).

Barnstable plain deposits and Harwich outwash plain deposits consist mainly of gravelly sand with local intrusions of silt, clay and boulders and are structurally analogous to the Mashpee pitted plain deposits.

In contrast to the depositional origin of the plain deposits, the Buzzards Bay moraine and Sandwich moraine are constituted by sequences of gravel, clay and poorly sorted sand and by abundant, very fine, sand deposits that overlie the basal till. According to Oldale (1986) these two deposits have mainly the same characteristics except for the presence of some silt in the Sandwich moraine. Since the moraines are less permeable than the deltaic deposits and therefore not interesting as water supply sources, few investigations have been made in these sediments.

The Buzzards Bay outwash deposits occur along Buzzards Bay shore west of the moraine and consist mostly of sand and gravel. They have characteristics similar to the foreset deposits formed in the Mashpee pitted plain.

Finally, along the northern shore, in contact with the Sandwich moraine, there are the Cape Cod Bay lake deposits and sand and swamp deposits. The first consist of sand and gravel interbedded with very fine sand, silt and clay and scattered boulders, whereas the second are formed by decaying plants mixed with varying amount of silt, sand and clay and include also some fresh water.

2.3 Hydrogeology

As can be inferred from the previous chapter on the West Cape Cod there are two different main types of sediments: the moraines and the deltaic and lacustrine deposits that can be grouped as meltwater deposits (Masterson et al., 1996). The subdivision between topset, foreset and bottomset facies that have been introduced for the Mashpee deposits can be extended to all the meltwater deposits. Several studies conducted on the glacial deposits indicate a strong correlation between depositional origin and lithology and hydraulic conductivity, therefore the hydraulic properties of the meltwater deposits can be inferred from the sedimentary facies to which they belong. As shown in Table 1 the hydraulic conductivity decreases with the distance from the sediment source (three main areas have been defined: proximal, the closest to the deposit source, mid and distal) and with the depth, following the general rule that K decreases with decreasing grain size. The hydraulic conductivity ranges from 350 feet/day (for topset beds, proximal) to 30 feet/day (for bottomset beds, distal). A very low conductivity (10 feet/day) is assigned to the lake-bottom sediments because of the consistent presence of silt.

[ft/d, foot per day]				
Depositional origin	Sedimentary facies	Lithology	Hydraulic conductivity	
			Horizontal (ft/d)	Ratio of horizontal to vertical
Deltaic				
Glaciofluvial	Topset beds			
	Proximal	Sand and coarse gravel	350	3:1
	Mid	Sand and medium gravel	290	3:1
	Distal	Sand and fine gravel	240	3:1
Glaciolacustrine (near shore)	Foreset beds			
	Proximal	Sand, medium to coarse	280	3:1
	Mid	Sand, fine to medium	200	5:1
	Distal	Sand, fine; some silt	150	10:1
Glaciolacustrine (offshore)	Bottomset beds			
	Proximal	Sand, fine	150	10:1
	Mid	Sand, fine; some silt	70	30:1
	Distal	Sand, fine and silt	30	100:1
Lacustrine	Lake-bottom beds	Silt and some clay	10	100:1
Glacial	Moraine	Gravel, sand, silt, and clay, poorly to moderately sorted	10–150	10:1–100:1
	Till	Sand, silt, and clay, unsorted	1	1:1

Table 1 - Depositional origin, lithology and hydraulic conductivity of sedimentary facies in western Cape Cod (Masterson et al., 1996).

Since the moraines did not have a depositional history to produce an evident differentiation of the stratigrafy, for these sediments there is only a range of values for the hydraulic conductivity. However, from experiments conducted in the Buzzards Bay moraine to identify a MMR plume position, it seems that there is a general trend of decreasing hydraulic conductivity with the depth. In general, in all the Cape area the vertical conductivity is less then the horizontal conductivity because the sediments were deposited in layers and the ratio of horizontal to vertical conductivity depends on the sedimentary facies.

Even if there is no data about the till hydraulic conductivity in Cape Cod, the value of 1 feet/day can be inferred from estimates made for analogous till sediments in other areas.

2.4 Hydrology

The aquifer in Cape Cod is shallow and unconfined; the flow is radial and mostly horizontal, even if near streams, ponds, pumping wells and at the salt-water interface there are discharge zones where the hydraulic gradient has a vertical component, from the groundwater mound, approximately in the middle of the area, into the ocean. The slope of the water table is greater to the west and the north sides because of several reasons. The first is that along these sides there are the moraine formations, whose permeability is lower than the one of the deltaic deposits. The second reason is that the bedrock depth decreases in these directions and finally the water table position in the south is determined by elevation of streams. The altitude of the water table ranges from 0 feet to about 69 feet (LeBlanc et al., 1986) and defines the top of the aquifer. Seasonal fluctuations are between 1 and 4 feet (Barlow and Hess, 1993) and they are lowest near the coast because of the sea-level constraint and are highest near the aquifer mound and near pumping wells. In general the highest levels are in early spring and the lowest in the fall (LeBlanc et al., 1986). The lower boundary of the system is the bedrock (or the poorly permeable till layer) and the lateral boundaries are the zones of transition between salt and fresh water.

The aquifer is the principal source of water supply for all the peninsula and is fed by precipitation that, in the West Cape, has been estimated to be 45 in/year. Since the soil is highly permeable most of the precipitation infiltrates and the runoff represents less than 1% of the total. The recharge to the aquifer varies between 18 and 22 in/year (Masterson et al., 1996) and part of this amount (about 5%) is due to the wastewater return flow from domestic septic systems and from sewage-treatment facilities (at MMR, in northwestern

Falmouth and in Barnstable). The precipitation that does not recharge the aquifer is "lost" to evapotranspiration.

On Cape Cod there are about 350 ponds (LeBlanc et al., 1986) that are, like the streams, hydraulically connected to the ground water flow system. In most of the ponds in the western Cape the aquifer discharges to their upgradient side and is recharged from the downgradient side. The net effect is that ponds recharge the aquifer because the precipitation rate is higher than the potential evaporation rate. The seepage amount that recharges the streams is basically unknown because most of them are ungaged, but groundwater flow is definitely their primary source of water.

Most of the groundwater discharge is into the ocean, the fresh flow prevents any saltwater intrusion landward. Any change in recharge or pumping rates of wells could cause a movement of the interface between salt and fresh water. From 1975 the movement of this boundary has been observed through a network of monitoring wells.

Chapter 3

Groundwater Computer Modeling

3.1 Numerical Models for Groundwater Flow

Groundwater hydrologists are often requested to predict the behavior of groundwater systems to evaluate the consequences of proposed actions or to optimize the resources of aquifers. Groundwater models are mostly formulated for projects involving remediation, investigation, feasibility analysis, supply, and contamination problems. Models can be used also in an interpretive sense to represent in a relatively simple way the characteristics of complex natural systems, and provide a tool to compare different management schemes. Models can assist in siting of monitoring wells and in designing and evaluating pump tests, can determine travel time and directions of contamination migration, can provide a better understanding of contamination source history and compare different water supply plans. The magnitude of the modeling effort depends on the characteristics of the site, on the availability of data and on the objectives of the investigation. The first step in a typical model application (Figure 5) should be to establish the purpose of the study, keeping in mind that the model is only one component in a hydrological assessment and that its role is to produce information and not precise answers. The second step is data collection and review of all the information available; the data quality is extremely important because it affects the results of the model. The third step is the development of a conceptual model: an abstract description of the natural

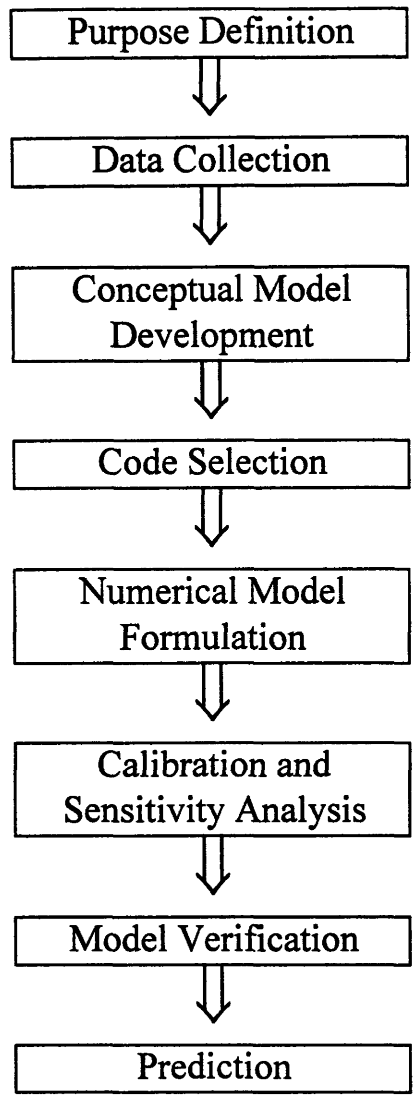

Figure 5 - Illustration of the steps involved in groundwater modeling procedure.

system. It includes the evaluation of geologic and hydrologic properties and of boundary and initial conditions. The conditions that are identified during the formulation of the conceptual model determine the choice of an adequate code (fourth step) for simulating the behavior of the aquifer. Next, the model has to be designed and calibrated within the numerical code. Once the adjustment of parameters reaches a satisfactory level, a sensitivity analysis should be performed to check the effects of uncertainty in the model. If a set of data, different from the one used for the calibration, are available the model can be verified and validated by proving that the measured and calculated values are reasonably close.

When the model is ready it can be used to quantify the response of the system to applied stresses and can be used to predict future situations. However, the results are inevitably affected by uncertainties due to simplifications necessary for building the conceptual model and to uncertainties due to evaluation of parameter distribution, boundary conditions and stresses.

3.2 DYN Modeling System

The computer modeling system DYN, was developed at Camp Dresser & McKee (CDM) for displaying and mapping groundwater data and for displaying groundwater flow and contaminant transport simulations. The DYNSYSTEM includes DYNFLOW, DYNTRACK and DYNPLOT. DYNPLOT is a graphic display package for the DYNFLOW and DYNTRACK computer programs that allows plotting and displaying data and simulated results. DYNFLOW is a three-dimensional groundwater flow simulation program for confined or phreatic aquifers or a mix of both . DYNTRACK takes the flow field produced by DYNFLOW to simulate the movement of particles. The code can perform simple particle tracking or three-dimensional mass transport for conservative or first order decay substances with dispersion and retardation.

DYNFLOW (DYNamic groundwater FLOW simulation model) is written in FORTRAN and solves the conventional equations of flow in porous media through linear finite elements (Galerkin finite element formulation). Either equilibrium or transient response flows can be simulated to represent different types of stresses such as infiltration from streams, artificial and natural recharge or discharge, pumping and evapotranspiration.

From Darcy's law:

$$q_i = -K_{ij} \frac{\partial H}{\partial x_j} \quad ; \quad (i,j = 1,2,3)$$

(where q_i is the specific discharge) and from the continuity equation:

$$\frac{\partial q_i}{\partial x_i} = -S_s \frac{\partial H}{\partial t} \quad ; \quad (i,j = 1,2,3)$$

can be derived the governing equations that DYNFLOW solves to describe the three-dimensional groundwater flow:

$$S_s \frac{\partial H}{\partial t} = \frac{\partial}{\partial x_i}\left(K_{ij} \frac{\partial H}{\partial x_j} \right) \quad ; \quad (i,j = 1,2,3)$$

where:

H is the piezometric head, in units of length [L],

S_s is the specific storativity, i.e. the volume of water released (or stored) per unit volume of aquifer per unit change in head, in units of one over length [1/L],

t is time, in units of time [T],

i and j represent the three principal orthogonal coordinate directions (x,y,z),

K_{ij} is the hydraulic conductivity, i.e. the flow per unit area, in units of length per unit time [L/T].

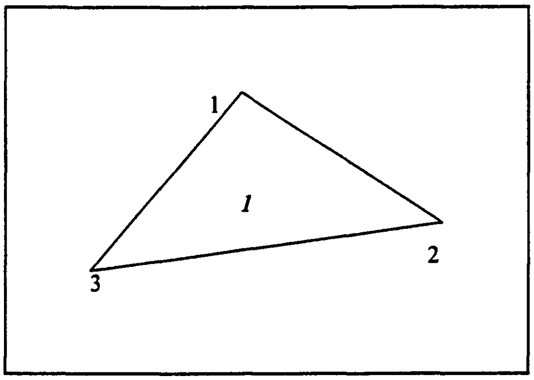

Figure 6 - DYN Single triangular element.

For complex flow systems there is no analytical solution and the governing equations can be solved only with the use of numerical methods like finite difference and finite element. DYNFLOW uses the finite element method that has proven to be robust and flexible. The region of interest is divided into working elements that are subdivided by the code into computational elements with tetrahedral shapes (Figure 6 and Figure 7). A more detailed description of the computational issues related to the solution of the governing flow equations are included in *DYNFLOW. A Finite Element Groundwater Flow Code. Description and Users Manual, Version 5,* produced by Camp Dresser and McKee.

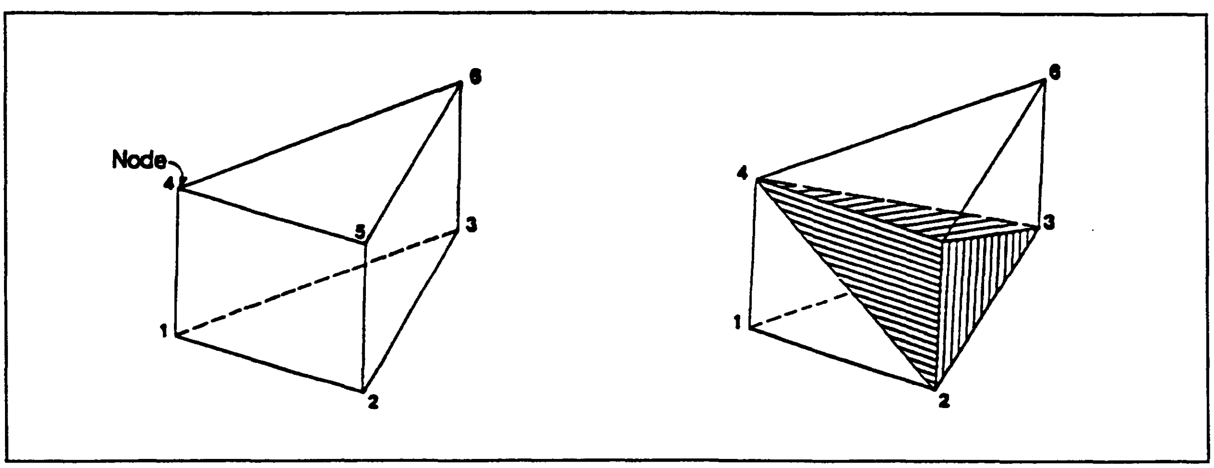

Figure 7 - The basic element and its subdivision in three tetrahedral computational elements.

Chapter 4

Regional Flow Model

4.1 Conceptual Model

A groundwater flow model for the aquifer in West Cape Cod was developed with the numerical code DYNFLOW. The system was assumed to be in steady state conditions and therefore seasonal variations in recharge, water table and ponds elevations were not considered. The model includes all West Cape Cod area as shown in Figure 8 and it covers an area of about 226 square miles.

4.1.1. Discretization

The finite element grid generated by DYNPLOT to cover West Cape Cod consists in 1092 nodes and 2004 triangular elements. The elements are smaller in the western area and coarser moving in the eastern direction as shown in Figure 8.

The stratigraphy was modeled with 6 different layers (7 levels). The lowest level elevations were extrapolated from the bedrock contours reported by Oldale (1969) and represent the impermeable bottom boundary of the aquifer. The top level, the seventh, approximates the surface topography that was derived from Brownlow (1979). The moraines and the meltwater deposits are the main lithologic units as described in Section 2.2; zones of lithologically similar deposits were defined following available maps and sections (Oldale and Barlow, 1986 and Masterson et al., 1996).

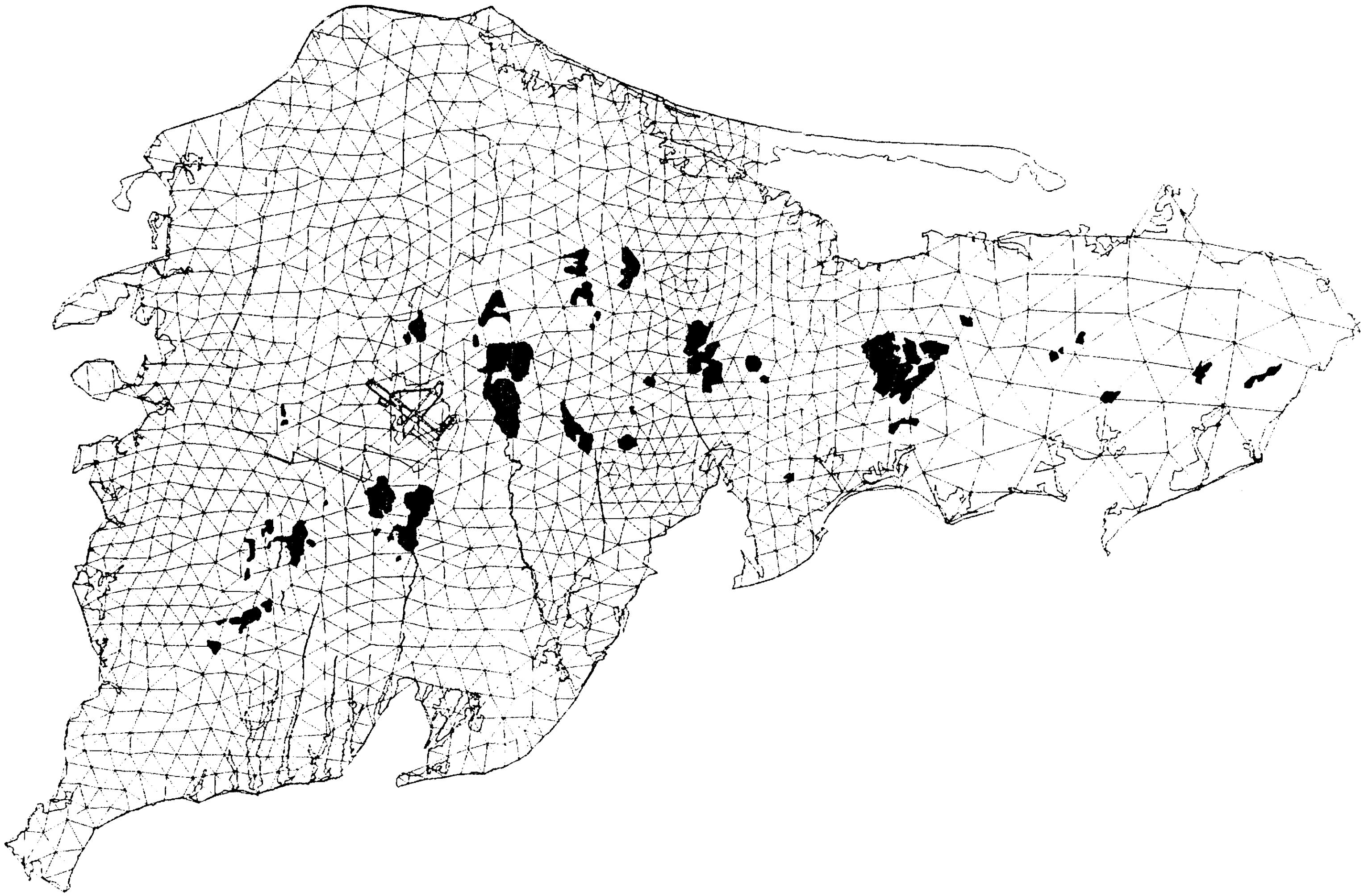

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Figure 8

Plan view of West Cape Cod and finite element grid

In the horizontal direction no differentiation between the Sandwich and the Buzzards Bay moraines were made, while the pitted plain deposits were divided into proximal, mid and distal sedimentary facies (Figure 9). In the vertical direction the horizontal and vertical hydraulic conductivity in the moraine area decreased moving from the top layer to the bottom layer.

In the meltwater deposit area the first layer (from the bottom) represents lacustrine deposits, the second and third the bottomset sedimentary facies, the fourth the foreset, and the fifth layer describes the transition between the foreset and the topset sedimentary facies, which constitutes the last layer, i.e. the surface layer (Figure 10).

4.1.2 Boundary Conditions

The boundaries of the model correspond to the boundaries of the West Cape Cod flow system. The saltwater-freshwater interface represents the flow boundary along the northern shore (Cape Cod Bay and Cape Cod Canal), the western shore (Buzzards Bay) and the southern shore (Nantucket Sound). This condition was simulated in the DYNFLOW model by fixing the head to zero at the nodes on the boundary at the two upper levels (consequently a non-horizontal flow was induced). This representation, even if is a rough approximation, does not effect the model results for the purposes of this analysis.

The few nodes that lie along the eastern boundary were considered as "rising water" nodes. With this condition the program fixes the head at the elevation of the node if the head tends to rise above the node elevation. The "rising water" condition is the

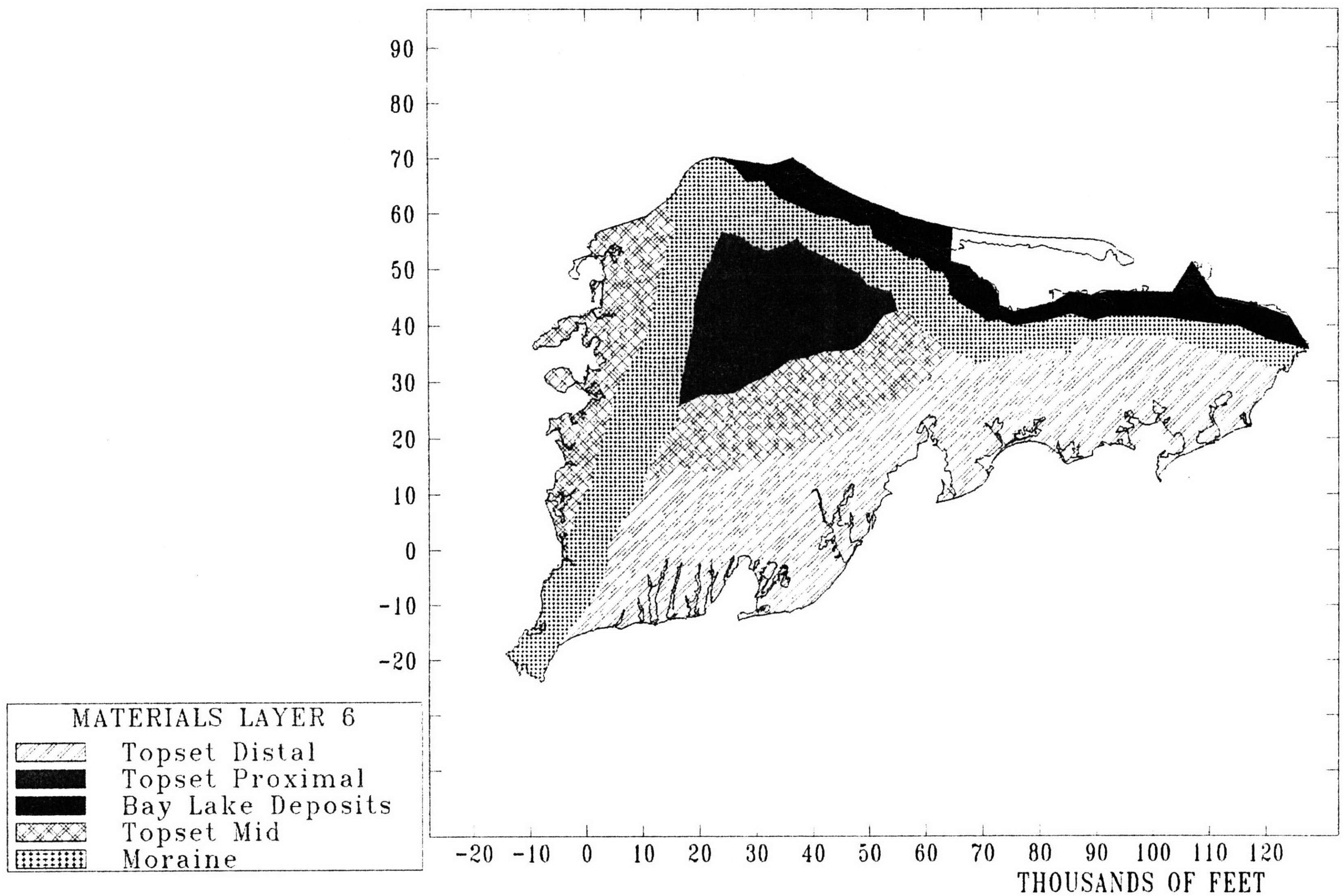

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Figure 9

Plan view of the study area with assigned geologic deposits

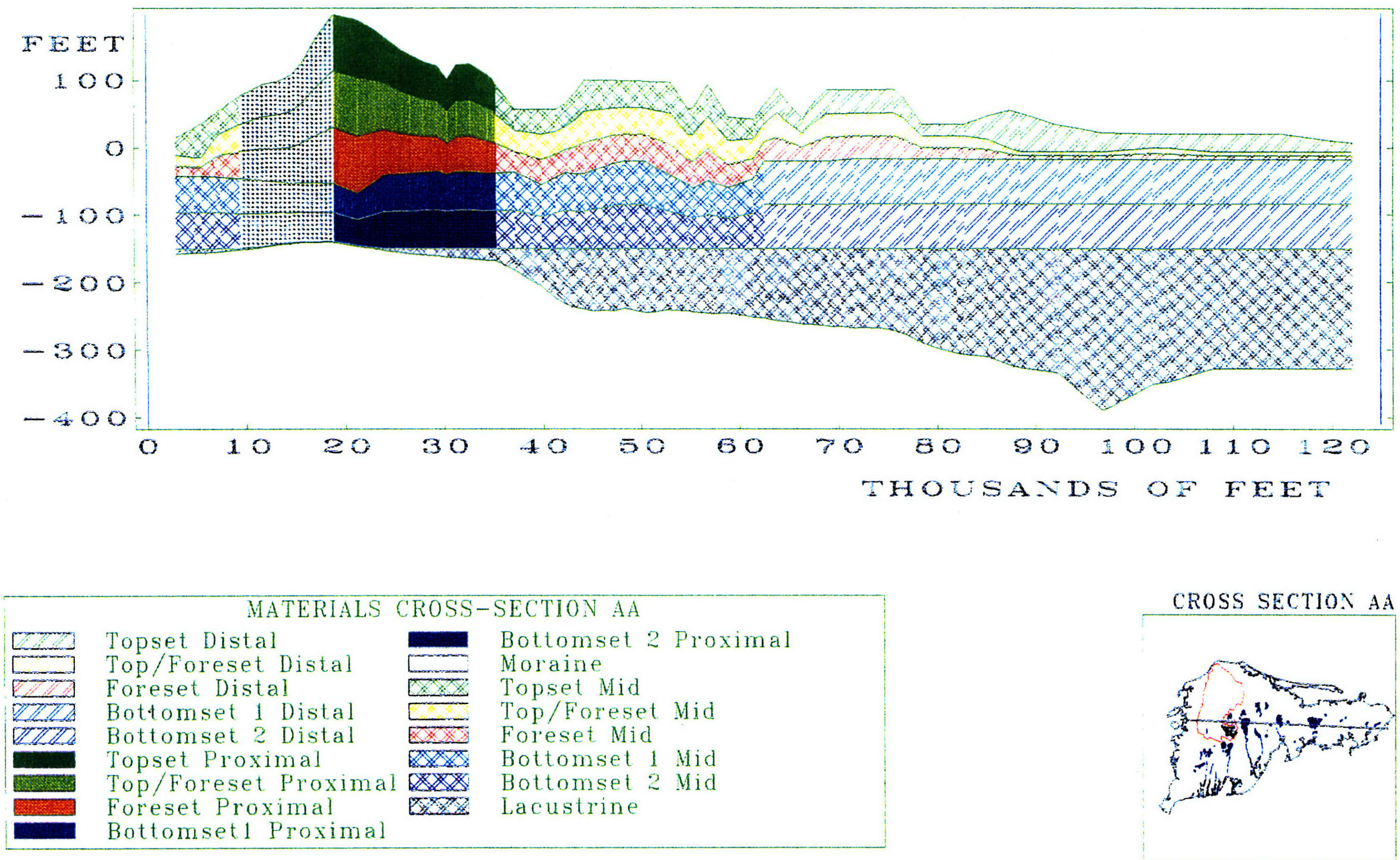

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Figure 10

East-West cross section of the study area default for the nodes in the upper level while the program automatically assigns a "dry" condition to the nodes in the first level, that is the level elevation represents the lower limit for the head. For all the other nodes the default condition is "no flux". If the computed head for a node in the upper level reaches the surface elevation, the program labels this node as "invoked rising water". This is the case of nodes along streams and ponds, into which the aquifer discharges, and of nodes in very flat areas, like in the eastern part of the West Cape model (Figure 11).

4.2 Modeling Results

The model calibration was based on the measured values of the hydraulic head reported by Savoie (1995) for the most western part of the area under study. Data regarding water table position in the rest of the area were not available for the same period (March 1993). Data published by LeBlanc (1986) was used only for a "general control" of the computed values. The recharge was used as calibration parameter only during the first calibration phase, afterwards it was assumed to be 0.00489 feet/day (21.42 inches/year) and only values of hydraulic conductivity were changed to calibrate the model. The calibration results are reported in Table 2. As shown in Figure 12 the mean difference between simulated and measured values is -0.043 feet and the standard deviation, probably due to the presence of local geologic situations that are not significant on the scale of the model, is 2.58 feet. The path of the simulated contours is close enough to the representation available from the literature for the purpose of this study.

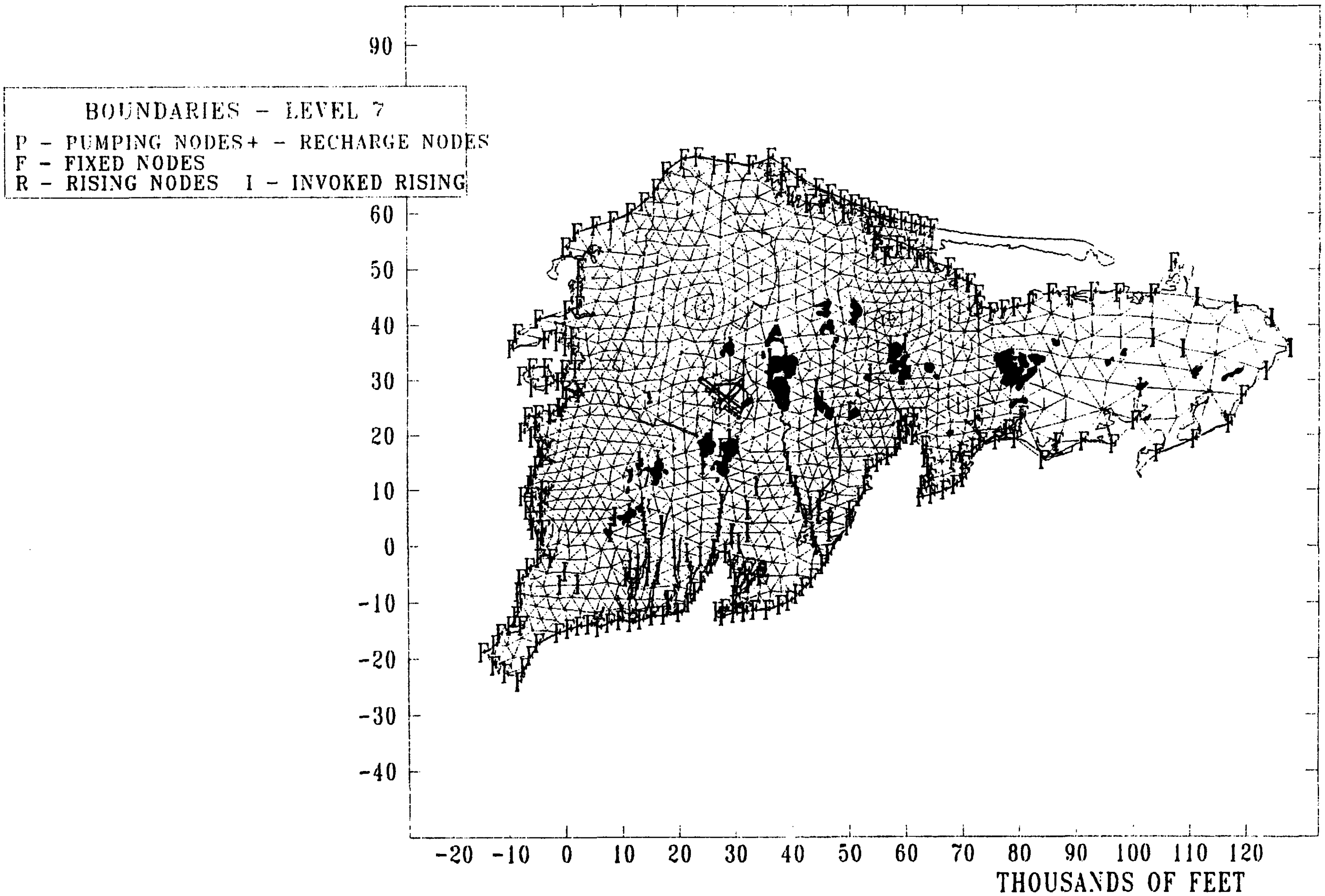

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Figure 11

Boundary conditions
Invoked Rising nodes and Fixed nodes at model level 7

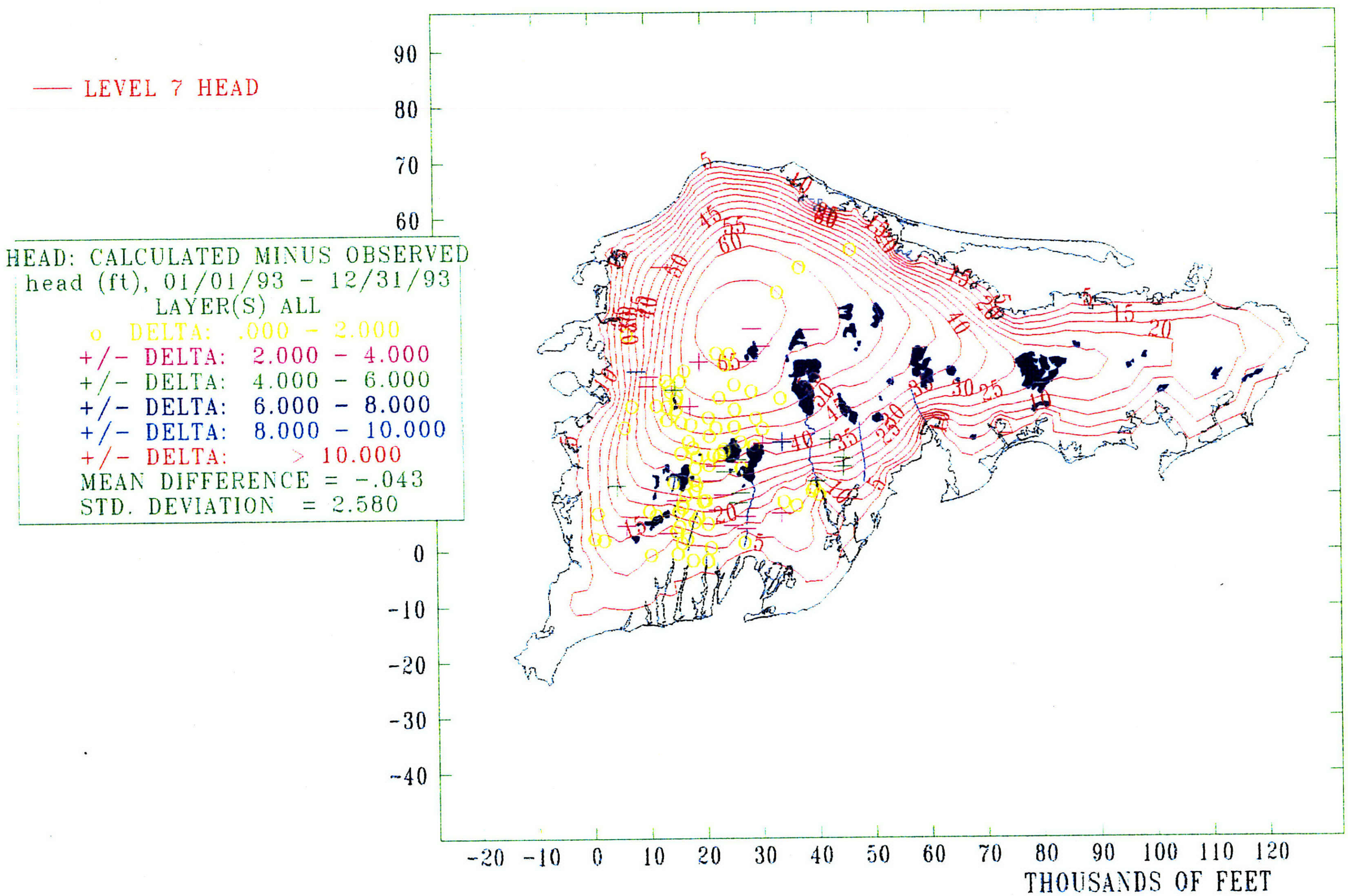

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Figure 12

Calculated water table elevation contours and flow model calibration results

Sedimentary Facies	Material Number	K_x ,K_y (feet/day)	K_z (feet/day)
Topset Beds, Proximal	10	270	85
Top/Foreset Beds Proximal	11	190	70
Foreset Beds Proximal	12	150	60
Topset Beds Mid	20	215	77
Top/Foreset Beds Mid	21	170	61
Foreset Beds Mid	22	130	30
Topset Beds Distal	30	200	80
Top/Foreset Beds Distal	31	160	33
Foreset Beds Distal	32	140	17
Bottomset Beds Proximal (Layer 2)	40	130	13
Bottomset Beds Proximal (Layer 3)	41	120	13
Bottomset Beds Mid	50, 51	70	2
Bottom Beds Distal	60, 61	50	0.3
Moraine (Layer 6)	80	100	57
Moraine (Layer 5)	81	90	57
Moraine (Layer 4)	82	60	30
Moraine (Layer 3)	83	50	30
Moraine (Layer 2)	84	20	10
Moraine (Layer 1)	85	20	10
Bay Lake Deposits	90	60	2
Lacustrine	99	10	0.1

Table 2 - Horizontal and vertical hydraulic conductivity of sedimentary facies used in the flow model. Each material type is identified by DYN through a material number.

4.3 Response of the Flow System to Stress Conditions

Two different pumping situations were simulated with the model. In the first one the pumping rates for 1989 were considered and in the second one existing and proposed wells and projected rates for 2020 were analyzed. Considering the 1989 pumping rates the decline in water table altitude at the well nodes is, on average, 0.8 feet and the model-calculated average head is 25.64 feet (while it is 26.45 feet if no water is pumped). When the pumping rates increase to the 2020 values and the proposed wells are introduced, the average decline in water table altitude is slightly less than 3 feet and the average head is equal to 23.49 feet. The pumping rates that were used are reported in Table 3.

USGS Map Number	DYN Node Number	Average Daily Demand (ft^3/day) 1989	2020	USGS Map Number	DYN Node Number	Average Daily Demand (ft^3/day) 1989	2020
42	1232	-	23069	77	1245	-	24624
41	1230	-	23069	14	1244	-	141869
45	1229	-	23069	13	1243	-	73613
44	1233	-	23069	5	1242	-	19786
43	1234	-	23069	4	1241	-	24970
50	1235	-	38102	55	579	56160	971136
48	711	-	31278	54	582	76205	69811
49	1236	-	38102	56	426	21427	48557
37	1237	-	34215	60	376	13392	20995
30-31-32	1238	-	126490	15	307	40090	82253
53	1233	-	18490	17-18	308	36115	73958
20	1231	-	38102	16-19	100	38794	79488
58	1239	-	48560	27	914	111024	102298
59	1234	-	48560	28	971	49507	72230
29	1240	-	62640	52	446	54778	52013
74-76	1248	-	49248	39	544	98928	56160
81	1224	-	24624	40	464	7883	56160
80	1247	-	24624	46	661	5357	7603.2
79	1249	-	24624	33-34-36	862	36461	49248
78	1246	-	24624	35	817	11059	8208

Table 3 - Pumping rates of public supply wells (existing and proposed) for the year 1989 and for the year 2020 (Masterson and Barlow, 1994).

Chapter 5

Groundwater Management

5.1 Water Supply Demand in the Town of Falmouth

In Falmouth, 85% of the population is served by public water provided by three wells and by Long Pond, while private wells furnish water for the rest of the population. In 1995 4,172,000 gallons/day were pumped in the Falmouth Water Department. Part of this amount is used also to irrigate golf courses and in agriculture. More water supplies (Figure 13) are currently needed, not only because the population is growing but also

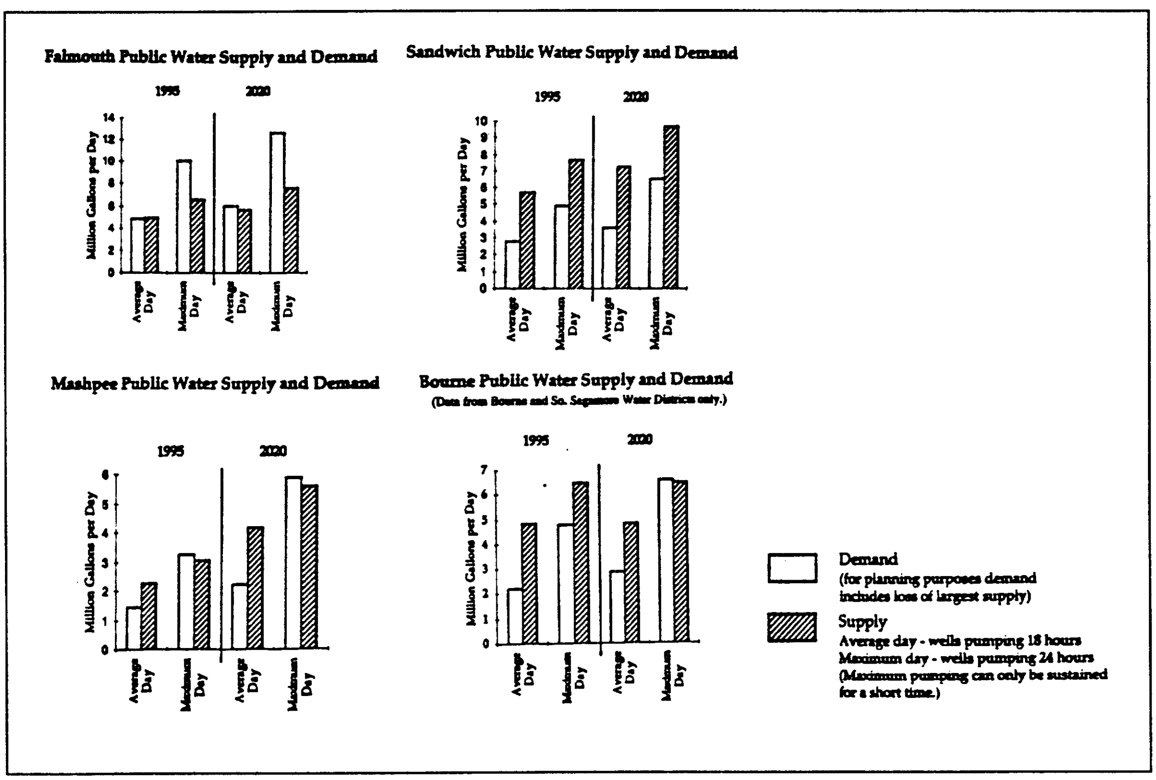

Figure 13 - Public water supply and demand for the town of Falmouth, Sandwich, Bourne and Mashpee.

because emergency well closures are possible. Two wells have already been taken off line, and the pumping rates in Long Pond have been reduced to avoid lowering water levels and pumping from contaminated areas. Two sites have been proposed for the installation of new wells (Figure 14), and protection areas have been delineated, based on a travel time of 50 years.

Scope of the present study is the determination of possible new well sites and the evaluation of pumping schemes that will yield the maximum rate without drawing upon polluted water.

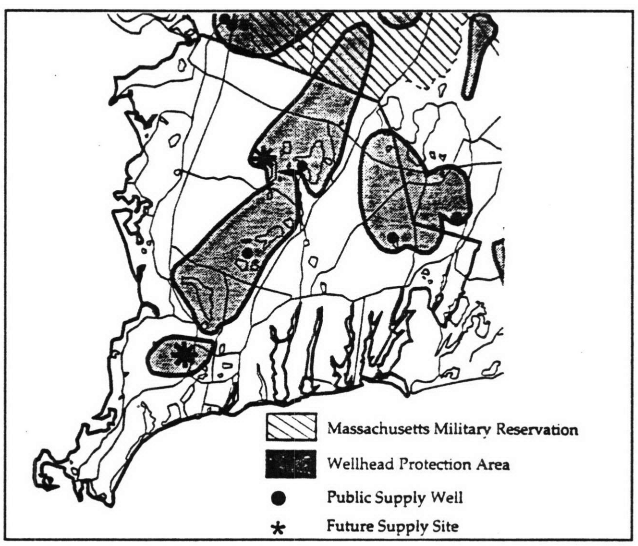

Figure 14 - Proposed wells sites for Falmouth public water supply.

5.2 Water Resources Planning

Historically the main objective of water resources planning has been economic efficiency, but currently other goals have been added at different levels (national, regional, community or individual consumer scale). Often in a project there are different economic, political, social and technological objectives, some of which are quantifiable and some of which are qualitative. The role of the planners is to evaluate alternative solutions obtained considering the quantitative objectives, and to illustrate their cost effectiveness; while the politicians are the real decision makers. The planners must translate the goals into operational objectives and formulate an objective function. This function depends on decision variables that are subject to specified constraints. Decision variables could be spatial and temporal distributions of pumpage (or of artificial recharge), water levels in streams and lakes in contact with the aquifer, quality of pumped water, and others. Examples of constraints are: minimum (or maximum) water levels everywhere or at specified points, chemical concentration of certain species in pumped water, and minimum total pumpage to satisfy water need (Bear, 1979).

An important role in the planning process is played by models; in groundwater management they are especially powerful tools. The design process involves the creation of a model of the aquifer system that is then used to predict the response of the flow regime to proposed pumping strategies. In the past, simulation flow models were used to evaluate groundwater management alternatives by a repeated execution of these models under different design scenarios. Recently the linking of simulation and optimization components has been developed since it is unlikely that optimal alternatives will be discovered through manual repetition of flow model simulations alone (Gorelick, 1983). Groundwater management models can be classified into two categories: hydraulic management models (embedding method and response matrix method) and policy evaluation and allocation models (Figure 15). In this study, simulation and optimization is performed by employing the response matrix approach in a hydraulic management model. While the embedding method uses numerical approximations of the groundwater flow equations as constraints for a linear programming formulation, the response matrix method uses solutions to the flow equations as constraints. The response matrix is an assemblage of the unit responses of an aquifer, caused by a pulse stimulus (such as a unit pumping) and calculated with the groundwater flow model. With this method the dynamics of the physical system is directly incorporated into the linear optimization problem.

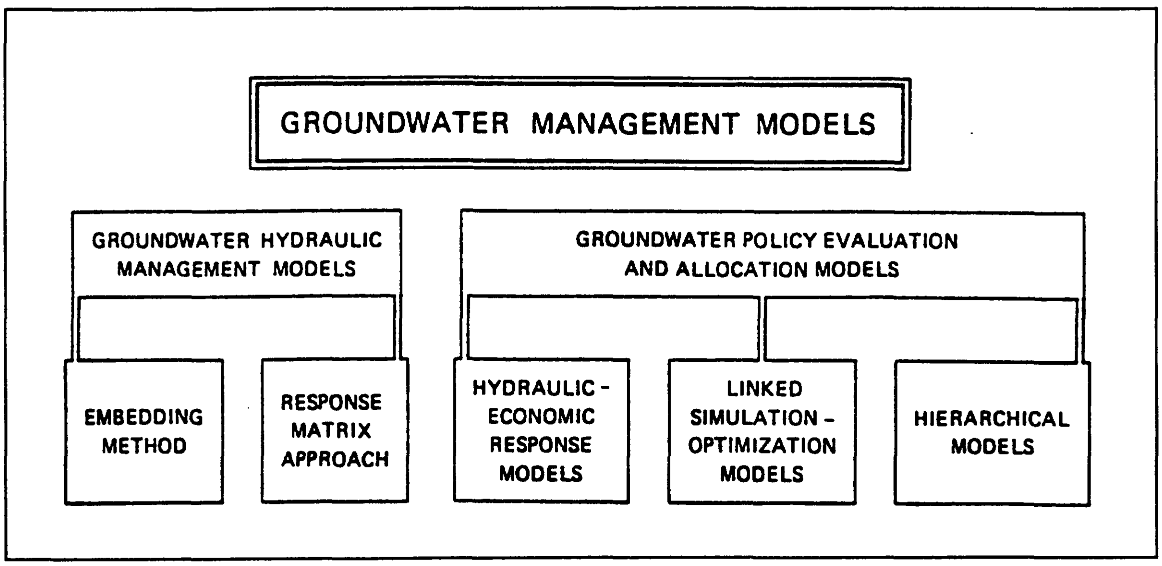

Figure 15 - Classification of groundwater management models.

5.2.1 Optimization Model Formulation

Linear programming (LP) models are characterized by the fact that the objective function (i.e. the function that has to be maximized or minimized) and the constraint set are linear.

The general linear programming problem can be mathematically expressed as:

$$\max z = \mathbf{cx}$$

subject to

$$\mathbf{Ax} \leq \mathbf{b}, \quad \mathbf{x} \geq 0$$

where z is the objective function, **c** is a n coefficient row vector, **x** is a n column vector of decision variables, **b** is a m coefficient column vector and **A** is a m x n coefficient matrix (technological matrix) that defines the constraints of the optimization problem. The technological matrix in groundwater management problems is the response matrix described in the previous Section 5.2. The elements (A_{ij}) of the response matrix, that is a symmetric matrix, are computed as follow:

$$A_{ij} = \frac{\partial h_i}{\partial Q_j}$$

where:

h_i is the head at well i,

Q_j is the pumping rate at well j.

5.2.2 Water Supply Analysis for the Town of Falmouth

The first step in the water supply analysis conducted for the town of Falmouth has been the identification of possible well sites. As can be inferred from Figure 16 the entire Falmouth district lies downgradient of the contaminated area. To ensure that no polluted water will be pumped, boundary flow lines have been drawn, such that water coming

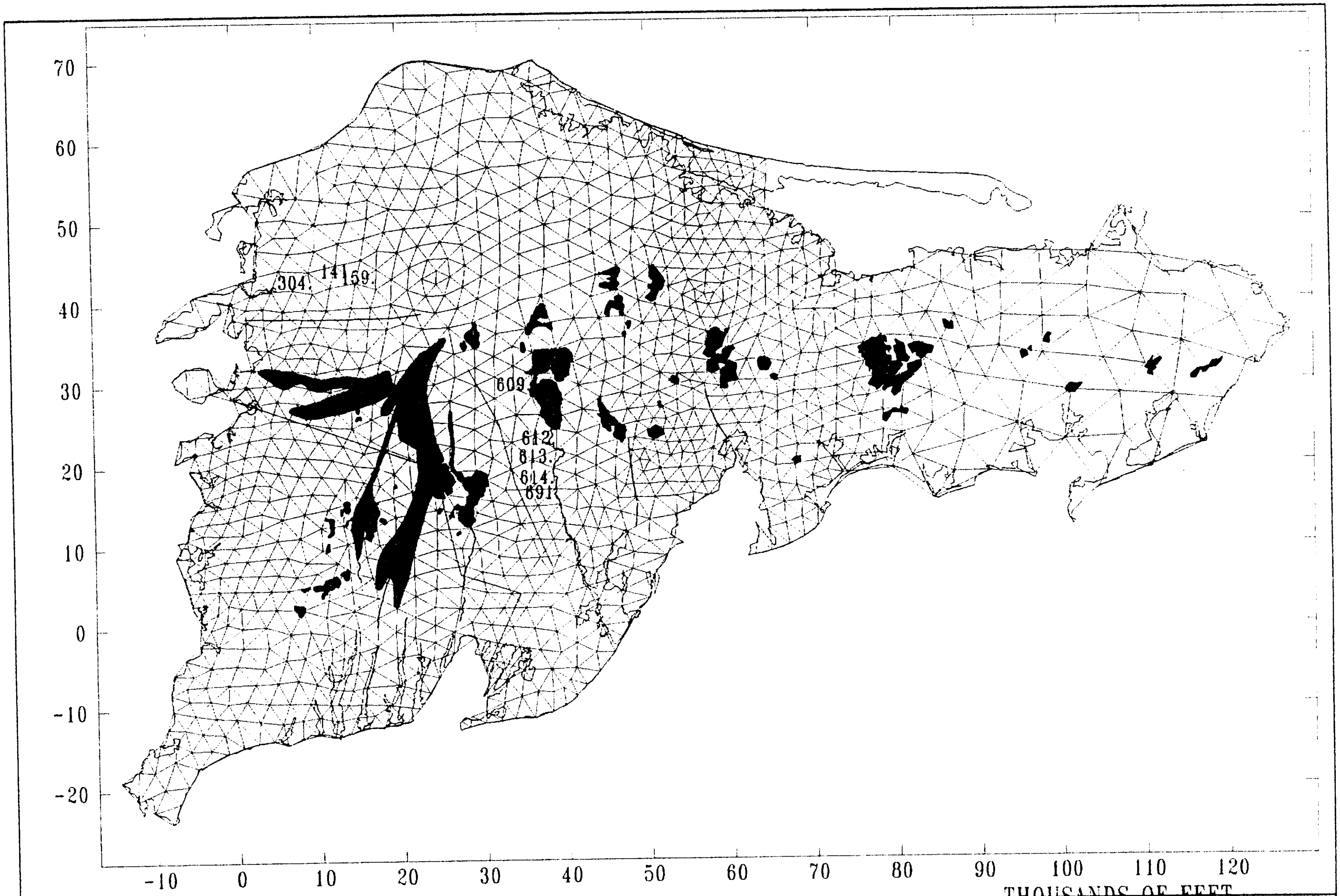

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Figure 16

Position of the plumes, pumping nodes and considered boundary flow lines from areas outside these boundaries can be considered clean. The flow model nodes number 609, 612, 691, 304 and 159 (Figure 16) have been chosen as potential well sites.

The maximization of the sum of the pumping rates available from the five wells represents the objective function. Initially no restrictions were imposed on the capacity of the wells, with the goal of estimating the total amount of water ideally available. Later some restrictions were introduced and other wells' sites proposed to describe a more likely scenario. The groundwater flow model has been used to evaluate the response matrix in pairs of points, and the differences in head at these perimeter points are the constraints on the optimization problem. The requirement that no water is pumped from the area inside the delineated boundaries is met by constraining the gradient at these control points to be inward. It is therefore possible to pump water from the aquifer until when in one of the pairs the difference between the head at the node outside the boundary and the head at the node inside the boundary becomes equal to the natural difference in head. A graphical representation of the constraints used in the optimization to avoid to pump "polluted" water is reported in Figure 17. The first illustration (a) shows a cross section of the aquifer, perpendicular to a boundary, in natural conditions, when the flow is inward and there is a difference in head equal to b_i between the pair of points A-B. Point B is outside the boundary ("clean" part of the aquifer), while point A is inside the boundary ("polluted" part of the aquifer). In the second illustration (b) the extreme allowable condition is represented. It is possible to pump water at rate Q_j from a proposed well only until when the head at node B becomes equal to the head at node A, so that the water supply well provides water only from the "clean" part of the aquifer.

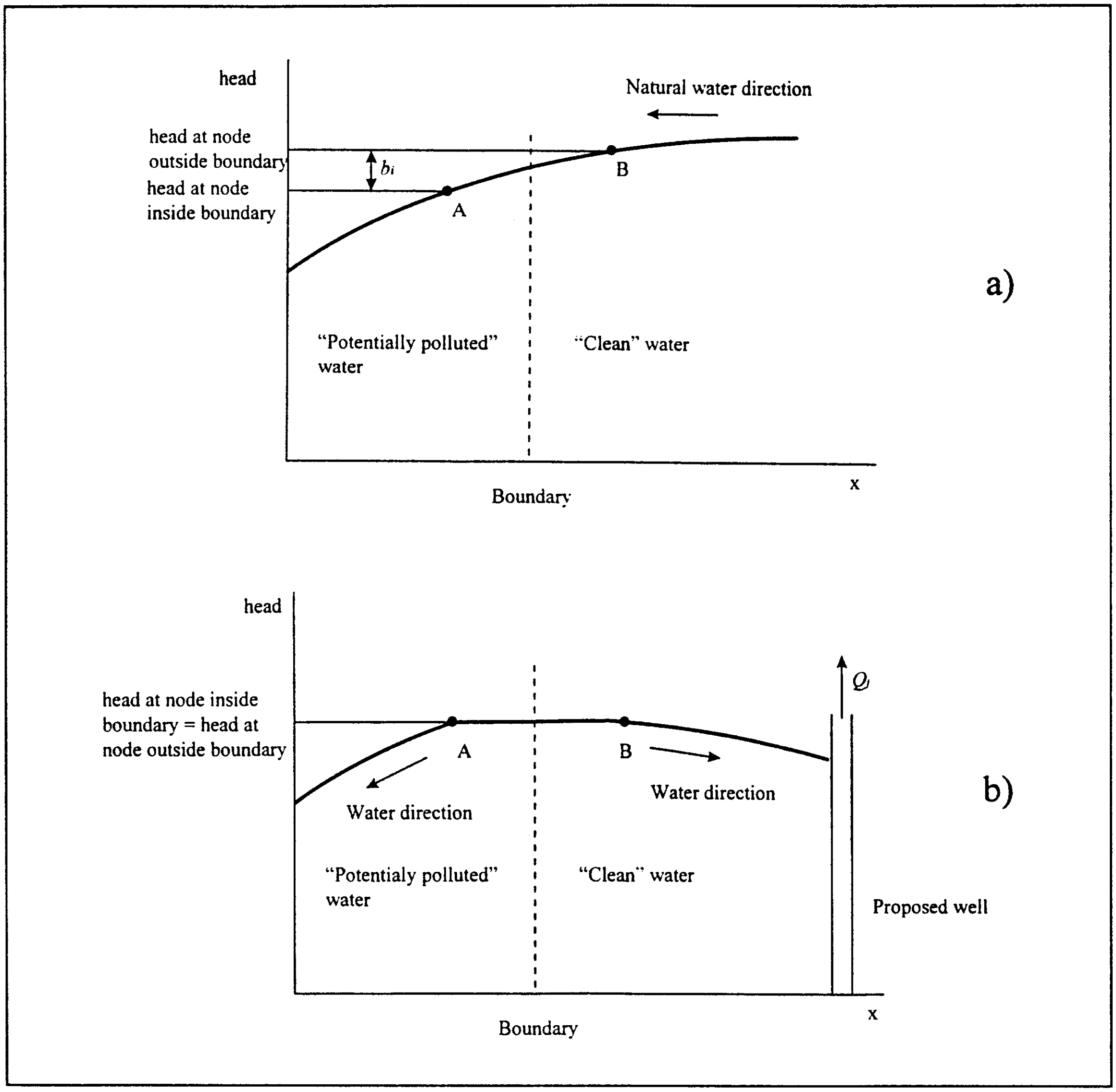

Figure 17 - Illustration of the limitation imposed by the constraines used in the optimization model. Part a) shows the natural situation in cross section, while part b) shows the limit pumping condition.
Clean water is pumped from a proposed well if the difference in head between point B, situated outside the delineated boundary, and point A, inside the boundary, is less or equal to the natural value b_i.

The mathematical representation of the management problem is:

$$\max z = \sum_j Q_j$$

for $j=1...n$, where n is the total number of wells, such that

$$\mathbf{A}_{ij}\mathbf{Q}_j - \mathbf{B}_{ij}\mathbf{Q}_j \leq \boldsymbol{b}_i$$

where:

i represents control points pairs that were considered,

Q_j is the pumping rate (feet^3/day) at the well j,

$\mathbf{A}_{ij}$ is the response matrix for the nodes inside the boundaries (Table 5-3),

$\mathbf{B}_{ij}$ is the response matrix for the nodes outside the boundaries (Table 5-4),

$\boldsymbol{b}_i$ is a known vector representing the natural difference in head at the control points pairs.

The optimization was performed by using a computer program called LINDO (Linear, Interactive aNd Discrete Optimizer). Considering five pairs of points (i.e. there are five equations in the optimization problem, Appendix A), the results showed that, if the pumping rate at each well is not constrained, it is possible to pump about 12.72 MGD without drawing water coming from the "contaminated area". This amount would be enough to supply the 2020 maximum day water demand estimated by the Water Resources Office, Cape Cod Commission, as shown in Figure 13.

If the pumping rate in each of the five proposed wells is limited to 50,000 cubic feet per day (0.374 MGD) the objective function becomes equal to about 1.49 MGD. According to the data prepared by the Water Resources Office the 2020 average day water supply shortfall in Falmouth is 1.44 MGD and therefore the simulated situation could reasonably be close to a solution for the average day demand problem. Other optimizations were

	Pumping Node Number							
	609	612	691	304	159	613	614	141
Control Point 1	-4.3E-07	-2E-07	-4.1E-06	-2.6E-08	-6.6E-08	-2.3E-06	-5E-07	-1.7E-07
Control Point 2	-2.5E-06	-1.6E-07	-2.4E-05	-1.4E-07	-4.3E-07	-5.9E-06	-5E-07	-3.2E-07
Control Point 3	-2E-07	-2.6E-07	-2.5E-06	-1.7E-05	-6.9E-06	-6.2E-08	-2.3E-07	-8.9E-06
Control Point 4	-4.9E-07	-5.1E-07	-6.5E-06	-5.1E-06	-1.8E-05	-1.8E-07	-5.2E-07	-1.2E-05
Control Point 5	-4.3E-06	-3.1E-07	-4.4E-05	-4.1E-07	-1.6E-06	-1.4E-06	-4E-07	-1.4E-06

Table 5-3 - Response matrix (feet) for the control points outside the boundaries indicated in Figure 16.

	Pumping Node Number							
	609	612	691	304	159	613	614	141
Control Point 1	-4E-07	-1.3E-07	-3.2E-06	-7.3E-08	-5.7E-08	-1.6E-06	-3.6E-07	-1.8E-07
Control Point 2	-1.9E-06	-5.4E-08	-1.9E-05	-1.9E-07	-4.2E-07	-3.7E-06	-3.1E-07	-3E-07
Control Point 3	-2.2E-07	-2.1E-07	-2.4E-06	-1.4E-05	-6.2E-06	-1.1E-07	-1.9E-07	-7.8E-06
Control Point 4	-5.3E-07	-4.3E-07	-7E-06	-4.8E-06	-1.4E-05	-2.9E-07	-4.5E-07	-9.7E-06
Control Point 5	-2.6E-06	-1.4E-07	-2.8E-05	-5.1E-07	-1.7E-06	-9.8E-07	-2.1E-07	-1.7E-06

Table 5-4 - Response matrix (feet) for the control points inside the boundaries indicated in Figure 16.

performed by adding new wells at the model nodes number 613, 614 and 141 (Figure 16) with the scope of providing enough water to supply also the maximum day water needs.

The calculated amount of water available from all the eight considered wells is about one third of the 2020 maximum day shortfall (that is equal to 6.54 MGD). A better strategy would probably be the addition of more wells in the same site that could be open in case of necessity rather than the addition of new wells' sites.

Chapter 6

Conclusions

From the conducted optimization it may be concluded that there is a plentiful availability of water also in the situation that all the current water supply sources have to be shut down. The optimization conducted with constrained pumping rate showed that enough water would be available from five proposed wells to supply the average day shortfall and that about one third of the maximum day shortfall could be provided by adding other three wells.

The numerical model that was developed with the DYN code for the West Cape Cod basin, based on the USGS hydrogeological characterizations, seems to represent satisfactorily the natural groundwater flow. Limitations in the results are mainly due to the availability of data regarding the hydraulic properties of the materials (in particular in the moraine) that compose the aquifer.

The model can be a starting point for more detailed investigations such as Zone II studies or transport analysis. The determination of the particles pathlines would represent a consistent improvement of the regional model. This improvement would make the flow model a more powerful tool to analyze the groundwater management problem. It would allow to describe the contaminants' movements and therefore to identify constraints different from the ones chosen in this study and new wells sites, possibly inside the Falmouth district.

Further evaluation could consider additional constraints that may be introduced to limit the drawdown in specific positions (such as the head control points or the pumping nodes). Particular attention should also be paid in the evaluation of the drawdown near ponds and streams because an excessive lowering of the water levels would be unacceptable.

Topics of interest for future analysis could be the evaluation and the optimization of more elaborate pumping distributions. For example the presence of groups of wells, placed near the "average-day supply" sites, to be opened only in an emergency situation, could be evaluated. A different analysis could consider the possibility of pumping water from an economic point of view. Wells could be placed in sites further from the contaminated area and costs to deliver water, proportional to the distance of each well from the center of the Falmouth district, should considered to take into account the fact that the cost to deliver water has to be subtracted from the benefits available from pumping more from these wells rather than from closer wells.

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Appendix A

Groundwater Flow Model Input Files

Site File

```
!mmr97.stf
FEET FINISH
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        YMIN  -25000.00  YMAX   70000.00 DELY   10000.00 &
        XLL   .382  XUR   .970  YLL   .177  YUR   .72   FINISH
MATERIALS &
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TYPE 20 NAME "Topset Mid" COLOR 3 PATTERN -1035 &
TYPE 30 NAME "Topset Distal" COLOR 3 PATTERN -1032 &
TYPE 11 NAME "Top/Foreset Proximal" COLOR 9 PATTERN 0 &
TYPE 21 NAME "Top/Foreset Mid" COLOR 9 PATTERN -1035 &
TYPE 31 NAME "Top/Foreset Distal" COLOR 9 PATTERN -1032 &
TYPE 12 NAME "Foreset Proximal" COLOR 8 PATTERN 0 &
TYPE 22 NAME "Foreset Mid" COLOR 8 PATTERN -1035 &
TYPE 32 NAME "Foreset Distal" COLOR 8 PATTERN -1032 &
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TYPE 90 NAME "Bay Lake Deposits" COLOR 2 PATTERN 0 &
TYPE 99 NAME "Lacustrine " COLOR 15 PATTERN -1035 &
FINISH
!
COLOR &
NUMBER 27 HUE  150 LIGHT 60 SATUR 75 &
NUMBER 41 HUE  210 LIGHT 80 SATUR 100 &
FINISH
BASEMAP mmr97.map FINISH
WELLFILE wells.wel FINISH
OBSFILE ex.obs FINISH
QUIT
```

Well File

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!wells.wel
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WELL TYPE
1 PUBLIC SUPPLY
4 TARGETS
FINISH
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Observation File with Water Table Elevations

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1/1/94
FINISH
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"WT-49","head","3/3/93",44.65
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"WT-51","head","3/3/93",44.87
"WT-52","head","3/3/93",44.12
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"WT-56","head","3/3/93",41.76
"WT-57","head","3/3/93",42.22
"WT-58","head","3/3/93",44.38
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"WT-60","head","3/3/93",42.95
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"WT-65","head","3/3/93",41.2
"WT-66","head","3/3/93",40.32
"WT-67","head","3/3/93",32.24
"WT-68","head","3/3/93",36.06
"WT-69","head","3/3/93",37.41
"WT-70","head","3/3/93",35.47
"WT-71","head","3/3/93",33.74
"WT-72","head","3/3/93",31.14
"WT-73","head","3/3/93",31.46
"WT-74","head","3/3/93",32.09
"WT-75","head","3/3/93",33.26
"WT-76","head","3/3/93",34.27
"WT-77","head","3/3/93",25.14
"WT-78","head","3/3/93",26.57
"WT-79","head","3/3/93",25.21
"WT-80","head","3/3/93",22.66
"WT-81","head","3/3/93",30.58
"WT-82","head","3/3/93",28.6
"WT-83","head","3/3/93",12.67
"WT-84","head","3/3/93",23.57
"WT-85","head","3/3/93",21.92
"WT-86","head","3/3/93",24.36
"WT-87","head","3/3/93",21.04
"WT-88","head","3/3/93",22.56
"WT-89","head","3/3/93",20.31
"WT-90","head","3/3/93",20.5
"WT-91","head","3/3/93",15.9
"WT-92","head","3/3/93",7.08
"WT-93","head","3/3/93",15.76
"WT-94","head","3/3/93",17.49
"WT-95","head","3/3/93",15.51
"WT-96","head","3/3/93",15.59

"WT-97","head","3/3/93",13.35
"WT-98","head","3/3/93",11.7
"WT-99","head","3/3/93",10.17
"WT-100","head","3/3/93",12.23
"WT-101","head","3/3/93",12.27
"WT-102","head","3/3/93",7.59
"WT-103","head","3/3/93",5.74
"WT-104","head","3/3/93",5.45
"WT-105","head","3/3/93",13.18
"WT-110","head","3/3/93",8.01
"WT-111","head",'3/3/93",8.63
"WT-112","head","3/3/93",11.85
"WT-113","head","3/3/93",11.86
"WT-115","head","3/3/93",14.28
"WT-117","head","3/3/93",16.36
"WT-119","head","3/3/93",17.62
"WT-125","head","3/3/93",21.31
"WT-129","head","3/3/93",25.42
"WT-142","head","3/3/93",36.16
"PS36","head","3/3/93",6.71
"PS37","head","3/3/93",8.77
"PS38","head","3/3/93",27.70
"PS39","head","3/3/93",28.64
"PS40","head","3/3/93",31.81
"PS41","head","3/3/93",35.40
"PS42","head","3/3/93",55.07
"PS43","head","3/3/93",60.23

LINDO Input Files

File Number 1

```
!Ottimiz1.ltx
!Five pumping wells - no constraints on the maximum pumping rate available from each well
MAX Q1+Q2+Q3+Q4+Q5
ST
+0.3Q1+0.69Q2+8.97Q3-0.47Q4+0.09Q5 < 7564200
+5.6Q1+1.02Q2+57.1Q3-0.46Q4+0.12Q5 < 6996100
-0.25Q1+0.53Q2+0.12Q3+29.17Q4+6.07Q5 < 2721900
-0.32Q1+0.8Q2-4.61Q3+3.09Q4+40.1Q5 < 1292100
+16.82Q1+1.78Q2+162.89Q3-0.93Q4-1.4Q5 < 3939600
END
```

File Number 2

```
!Ottimiz2.ltx
!Five pumping wells -with constraints on the maximum pumping rate available from each well
MAX Q1+Q2+Q3+Q4+Q5
ST
Q1<50000
Q2<50000
Q3<50000
Q4<50000
Q5<50000
+0.3Q1+0.69Q2+8.97Q3-0.47Q4+0.09Q5 < 7564200
+5.6Q1+1.02Q2+57.1Q3-0.46Q4+0.12Q5 < 6996100
-0.25Q1+0.53Q2+0.12Q3+29.17Q4+6.07Q5 < 2721900
-0.32Q1+0.8Q2-4.61Q3+3.09Q4+40.1Q5 < 1292100
+16.82Q1+1.78Q2+162.89Q3-0.93Q4-1.4Q5 < 3939600
END
```

File Number 3

```
!Ottimiz3.ltx
!Eight pumping wells -with constraints on the maximum pumping rate available from each well
MAX Q1+Q2+Q3+Q4+Q5+Q6+Q7+Q8
ST
Q1<50000
Q2<50000
Q3<50000
Q4<50000
Q5<50000
Q6<50000
Q7<50000
Q8<50000
+0.3Q1+0.69Q2+8.97Q3-0.47Q4+0.09Q5+7.17Q6+1.35Q7-0.17Q8 < 7564200
+5.6Q1+1.02Q2+57.1Q3-0.46Q4+0.12Q5+21.24Q6+1.92Q7+0.16Q8 < 6996100
-0.25Q1+0.53Q2+0.12Q3+29.17Q4+6.07Q5-0.48Q6+0.47Q7+10.83Q8 < 2721900
-0.32Q1+0.8Q2-4.61Q3+3.09Q4+40.1Q5-1.01Q6+0.74Q7+20.85Q8 < 1292100
+16.82Q1+1.78Q2+162.89Q3-0.93Q4-1.4Q5+4.07Q6+1.99Q7-2.3Q8 < 3939600
END
```