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REPORT TO THE TOWN OF PLYMOUTH, MASSACHUSETTS ON FACILITIES PLAN FOR WASTEWATER MANAGEMENT

VOLUME 1 - DRAFT REPORT

March 9, 1984

METCALF & EDDY, INC. / ENGINEERS
BOSTON / NEW YORK / PALO ALTO / CHICAGO

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Metcalf & Eddy, Inc.
Engineers & Planners

50 Staniford Street
Boston, Massachusetts 02114
TWX 710 321 6365
Cable METEDD-Boston
Telex 681 7067 (METED UW)
Telephone (617) 367-4000

March 9, 1984

Mr. Paul Hannigan, Director
Department of Public Works
Water Street
Plymouth, MA 02360

Subject: Town of Plymouth, MA, Wastewater Facilities Plan Draft

Dear Mr. Hannigan:

In accordance with Article 2.1.4 of our agreement dated November 17, 1981, we are submitting the draft report of the Facilities Plan for Wastewater Management. Copies should be displayed for public review in the Town Hall and in the libraries.

The final report will be submitted upon completion of the public participation program after the public hearing.

We wish to thank you and others from the Town of Plymouth for the valuable assistance given to us during the preparation of this report.

This report was prepared in accordance with the requirements of the Federal Water Pollution Control Act Amendments of 1972 and the Clean Water Act of 1977, the United States Environmental Protection Agency, and the Massachusetts Division of Water Pollution Control.

Very truly yours,

METCALF & EDDY, INC.

Eugene S. Grafton

Eugene S. Grafton
Vice President

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

LIST OF TABLES

LIST OF TABLES (Continued)

LIST OF TABLES (Continued)

LIST OF TABLES (Continued)

LIST OF FIGURES

LIST OF FIGURES (Continued)

LIST OF FIGURES (Continued)

ACKNOWLEDGEMENTS

We acknowledge with thanks the helpful cooperation and assistance of Mr. Paul Hannigan, Director, Department of Public Works; Damion Smith, Superintendent of the Wastewater Treatment Plant; Harold Strassel, Chief Operator of the Wastewater Treatment Plant; and Maureen M. Hanna, Public Participation Coordinator, and others in furnishing information for this report.

This report was prepared by Mr. Bradley W. Behrman, Senior Project Engineer; Mr. Paul Trepaney, Project Engineer; Mr. Arthur Knight, Project Engineer; Mr. Joe Federico, Engineer; and Mr. Jon Golden, Engineer; under the direction of Mr. Eugene S. Grafton, Vice President. The land disposal studies were conducted by Mr. Warren F. Diesl, Senior Hydrogeologist and Mr. Clifford E. Stein, Soils Scientist.

Mr. David Bova, Technology Leader for Wastewater Treatment, and other members of Metcalf & Eddy staff also made significant contributions to the development of this report.

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Summary

The North Plymouth area is served by an existing wastewater collection systems and wastewater treatment plant (WWTP). These facilities are unable to accept additional flow because of the limited capacity of the WWTP. Shortcomings in treatment plant performance, due to the combination of flow overloading and the lack of adequate sludge dewatering capabilities, frequently result in effluent suspended solids concentrations in excess of discharge permit requirements. Plymouth's old and deteriorated sewers and interceptors allow large volums of infiltration and inflow (I/I) to enter the system, compounding the problem. Sections of Plymouth Harbor are closed to shellfishing because of occasional bypassing of raw wastewater during wet weather periods, while other sections are closed primarily because of stormwater runoff from the urbanized area.

The portion of West Plymouth north of Route 44 and east of Plympton Road (Route 80) has no existing wastewater-related problems but cannot presently be developed according to its zoning. This is because it overlies the recharge area of the North Plymouth Well and is protected by the Town's Aquifer Protection Bylaw, which limits the on-lot disposal of domestic wastes in the area to 330 gallons per day per 40,000 square feet of area and prohibits the on-lot disposal of nondomestic wastes.

On-lot disposal systems surrounding Bartlett Pond in Manomet have been identified as a potential public health problem. Almost none of the on-lot disposal systems in this area meet current code requirements. A large number of lots have such high groundwater or are located so close to surface water-bodies that the replacement of failed on-lot disposal systems on these lots will likely be prohibited by local and State regulatory agencies.

A detailed analysis of alternatives to provide a long-term solution to the problems in Manomet resulted in the recommendation of the installation of a separate wastewater collection, treatment and disposal system for that area. Because of the desire of the Town to remove restrictions on industrial and commercial development in the portion of West Plymouth east of Plympton Road and north of Route 44, alternative collection systems were also investigated to serve this area. It has been proposed that the expansion of the existing WWTP include capacity to accept wastewater from this area. On-lot disposal systems should be retained in all other areas of the Town, as they are viable in the long term in these areas. Steps to increase long-term viability of these systems are included in this report.

Several alternative treatment and disposal scenarios were evaluated for North Plymouth. A wastewater treatment system utilizing the existing WWTP site and the conventional activated sludge process with the discharge of secondary effluent via an extended ocean outfall and diffuser was determined to be cost effective. The diffuser would be located in Goose Point Channel. Treatment capacity would be increased to 4 mgd, the chlorine disinfection system would be improved, and mechanical sludge dewatering would be provided by installing belt filter presses in a new sludge processing building. Sludge cake would be stabilized with lime and trucked to the Manomet Landfill for disposal.

An I/I reduction program would be undertaken and deteriorated portions of two of the Town's major interceptors would be replaced. Improvement would also be made to the Town's Night Soil Disposal Facility.

A wastewater treatment system using aerated facultative lagoons is recommended for Manomet, with discharge to a rapid infiltration (land disposal) system. The recommended site for this facility is in the buffer zone of Boston Edison's Pilgrim Nuclear Power Station, east of Edison Access Road.

Conclusions

As a result of this study of the wastewater problems and alternative solutions for the Town of Plymouth, we have concluded the following:

1. The existing WWTP is overloaded and has been subject to frequent discharge permit violations.
2. There is inadequate sludge dewatering capacity to handle existing WWTP needs.
3. The existing wastewater collection system is subject to excessive infiltration and inflow.

4. Portions of the Cordage and Harbor Interceptors are in need of replacement.

5. Rag and odor problems have been associated with the Night Soil Disposal Facility on Long Pond Road.

6. Sewers must be extended into the portion of West Plymouth north of Route 44 and east of Route 80 (Plympton Road) to permit the full development of this area allowable under its original zoning.

7. Severe problems have been identified with on-lot wastewater disposal systems in the portion of Manomet surrounding Bartlett Pond. The problems include evidence of at least occasional bacterial contamination of Bartlett Pond and White Horse Beach and widespread non-compliance with Title 5 of the State Sanitary Code. Continued long-term use of on-lot systems, even with improved standards and operation practices, is not feasible in these areas, and a phased sewer construction program is recommended

8. Two separate wastewater treatment and disposal facilities are recommended: an expanded 4-mgd facility to serve North and West Plymouth, which would discharge secondary effluent to Plymouth Harbor through an extended outfall, and a new 0.35 mgd facility to serve Manomet, which would discharge to the land at a new site located on the Edison Access Road

Recommendations

It is recommended that:

1. the facilities plan be submitted to the State DWPC for review and concurrence;
2. the Town take appropriate steps to resolve outstanding issues necessary to implement this facilities plan, including the support of legislation which would permit the Town to obtain a variance to the Ocean Sanctuaries Act; and
3. the Town initiate steps to submit a Step 2 grant application for design of the recommended facilities, in accordance with the schedule of compliance contained in the Town's NPDES permit issued in January 1984.

CHAPTER 1

INTRODUCTION

Purpose and Scope

The densely populated northern portion of Plymouth is served by an existing wastewater collection system and a secondary wastewater treatment plant (WWTP). While the plant is a relatively new facility (placed into operation during 1969), the collection system is, in most locations, very old and is subject to infiltration/inflow (I/I) problems. Wastewater flow at the WWTP, which currently serves about one third of the Town's residents and commercial and industrial development, exceeds the WWTP design capacity of 1.75 million gallons per day (mgd).

Wastewater disposal for the unsewered portion of the Town is by means of on-lot subsurface disposal systems consisting of cesspools or septic tanks followed by leaching systems. Some areas in Town have experienced on-lot uastewater disposal problems over the years.

This Facilities Plan Report is intended to define the needs for additional wastewater collection and disposal facilities or other measures which uill alleviate existing and potential wastewater disposal problems. The studies and evalutions are in accordance with the requirements of the Massachusetts Department of Environmental Quality Engineering (DEQE) and the Federal Water Pollution Control Act Amendments of 1972 and 1977 (Public Laws 92-500 and 95-217).

Under applicable rules and regulations, State and Federal grants are made in three steps as follows:

Step 1 - Facilities Planning Process

Step 2 - Preparation of Construction Documents (Design)

Step 3 - Construction

This report is intended to satisfy the requirements of the Step 1, Facilities Planning Process, and will focus on the following major objectives:

1. Define the nature and extent of wastewater disposal problems, both existing and potential.
2. Develop and evaluate feasible alternative solutions which will alleviate wastewater disposal problems and meet water quality and public health objectives established by local, State and Federal agencies.
3. Select a recommended plan which is cost-effective and compatible with Town objectives.
4. Present estimates of costs associated with the selected project and alternatives.

Due to the controversial nature of the project and the potential adverse environmental impacts, an Environmental Impact Statement (EIS) is being prepared. To minimize the time required for the overall completion of the project, the EIS is being "piggybacked" (prepared interdependently and concurrently with this report). C. E. Maguire, Inc., is preparing the EIS is under contract to the U.S. Environmental Protection Agency (EPA).

The aim of this study is the selection of an environmentally sound, cost-effective plan which will be consistent with local, regional, State and Federal goals. This report will be the basis upon which the Town can make sound decisions regarding future wastewater management. Of major significance in the planning process is the consideration of using the development of wastewater facilities in the management of growth in Plymouth.

Planning Area Description

The Town of Plymouth lies along the coast of Cape Cod Bay approximately 35 miles south of Boston. The Town lies within Plymouth County and encompasses a total land area of nearly 63,000 acres. The Town is bounded on the east by Cape Cod Bay and on the remaining boundaries by the towns of Bourne, Wareham, Carver and Kingston. In terms of land area, it is the largest municipality in Massachusetts and has one of the longest coastlines. The Myles Standish State Forest, containing approximately 10,000 acres, is located in the southwestern part of Plymouth.

The planning area of this study, shown in Figure 1-1, includes the political boundaries of Plymouth and the Rocky Nook section of the Town of Kingston to the north. The area in Kingston was included in the planning area by State and Federal regulatory agencies so that the cost-effectiveness of possible regional facilites could be investigated.

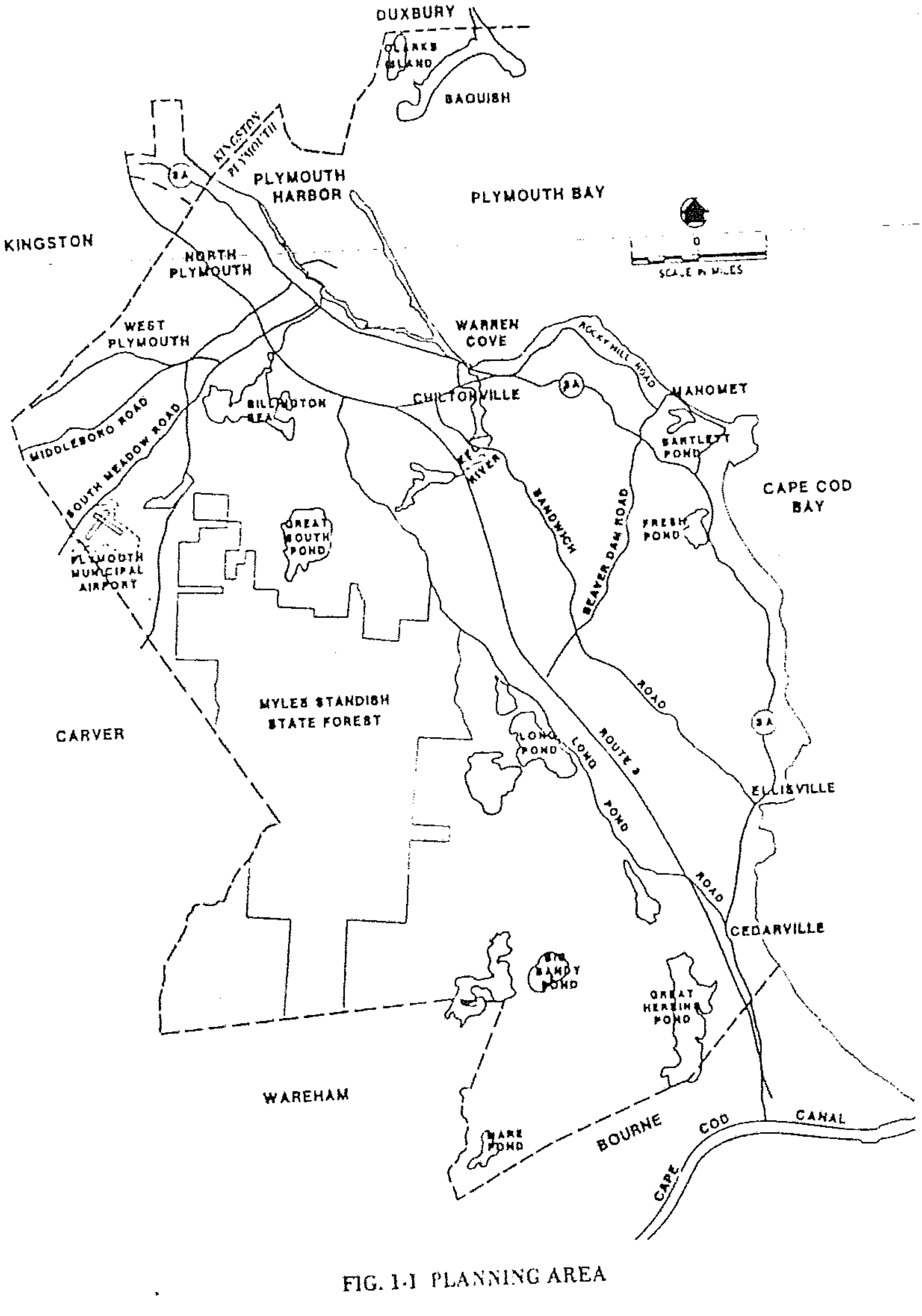

FIG. 1-1 PLANNING AREA

Previous Reports

In 1970, a regional study for water supply, sewage disposal and drainage was prepared for the Southeastern Regional Planning and Economic Development District (SRPEDD) by Tippetts-Abbett-McCarthy-Stratton (TAMS). The report evaluated the wastewater disposal needs of Plymouth and the surrounding communities and concluded that Plymouth and Wareham were building water pollution control facilities which would meet the needs of their communities, Carver would not develop to a future population density to warrant a muncipal wastewater collection system, and Kingston should construct its own secondary wastewater treatment plant.

A sewerage planning study for the Town of Plymouth was prepared by Metcalf & Eddy in a report dated December 1973. While this report focused primarily on collection and treatment of flows from the existing system, it did include a plan for the future expansion of the existing North Plymouth sewer system.

A separate report dated February 23, 1976 was prepared by Metcalf & Eddy for sewerage planning in the Manomet and Cedarville areas of Plymouth. This report recommended that a sewer system be constructed in Manomet along with a wastewater treatment plant and an outfall for discharge to the ocean off White Horse Beach.

In 1979, Metcalf & Eddy prepared a "Town Comprehensive Plan" for the Town's Planning Department, and included planning for the sewerage needs of the Town.

None of the above studies were conducted in accordance with the concept of the Step 1 Facilities Planning process, and each lacks several components which are currently required, including an environmental assessment, cost-effectiveness analyses, infiltration/inflow analysis, and public participation.

Although the Town of Plymouth is currently a member of the Old Colony Planning Council (OCPC), the Town was included in the Areawide Water Quality Management Plan (208 Plan) completed under the direction of SRPEDD. The SRPEDD planning was conducted under Section 208 of the Federal Water Pollution Control Act, and is intended to develop regional strategies for achievement of water quality goals. The draft 208 Plan was published in August 1977 and was reviewed and commented on by the constituent communities. The final report was published in February 1978. Solutions and recommendations developed within this Facilities Plan are intended to be compatible with the overall proposals which have been developed by SRPEDD.

An infiltration/inflow analysis of the existing collection system was initiated by Metcalf & Eddy during 1980. The draft report is included as Appendix J of this Facilities Plan.

In addition to the above studies, several studies of the water quality in Plymouth's ponds have been conducted during the past few years. These studies are identified in Chapter 4.

Public Participation

Consistent with Section 101(e) of the Clean Water Act and 40 CFR (Code of Federal Register) Part 25 and Part 35, Subpart E, the 201 Consultant Engineer, the EPA, the State Agency, EIS Contractor and the grantee (Town) shall provide for, encourage, and assist public participation in the facilities planning process. This facilities plan has followed a "full-scale" participation program in accordance with the regulatory requirements as hereinbefore stated. A program for public awareness and input has been conducted during this study. The definition of "public" includes Town boards and officials as well as State and Federal agencies. Meetings with local groups have been held throughout the course of the study. The program included the following:

1. Establishing a core group of citizens, the Citizens Advisory Committee (CAC), to help initiate the facilities planning process.
2. Selection of a Public Participation Coordinator (PPC).
3. The distribution of a Needs Survey (questionnaire) to the public in order to obtain their input on wastewater disposal problems.
4. Coverage in the area newspapers regarding the status and findings during the study.
5. Public meetings (workshops) designed to inform the public for the project's status and findings and to obtain citizen input.
6. A formal public hearing where concerned citizens could obtain a summary of the project and have their input entered into the project formal record.

This program is highlighted in greater detail in Appendix I of this report.

CHAPTER 2

WATER QUALITY AND EFFLUENT STANDARDS

General

After collection, wastewater must be treated and disposed of using methods which satisfy standards put forth in State and Federal water quality legislation. The standards, in essence, establish minimum treatment requirements which must be satisfied by all treatment and disposal alternatives.

Massachusetts Water Quality Standards

In 1978, the Division of Water Pollution Control (DWPC) of the Massachusetts Department of Environmental Quality Engineering adopted the Massachusetts Water Quality Standards. In these standards, the waters of the State are designated under three classes each for fresh waters and marine waters. The classification consists of designated uses which the waters are to serve, and the standards specify water quality criteria which are to be met for each class of waters.

Freshwater or inland waters are defined as rivers, streams, ponds, lakes, impoundments, but not groundwater; and include the following classifications:

Class A - Waters assigned to this class are designated for use as a source of public water supply.

Class B - Waters assigned to this class are designated for the uses of protection and propagation of fish, other aquatic life and wildlife; and for primary (e.g., swimming and water skiing) and secondary (e.g., fishing and boating) contact recreation.

Class C - Waters assigned to this class are designated for the uses of protection and propagation of fish, other aquatic life and wildlife; and for secondary contact recreation.

Within the Town of Plymouth, Great South Pond and Little South Pond are classified as Class A The remaining surface waters are Class B.

Coastal and Marine waters include the following classifications:

Class SA - Waters assigned to this class are designated for the uses and protection and propagation of fish, other aquatic life and wildlife; for primary and secondary contact recreation; and for shellfish harvesting without depuration in approved areas.

Class SB - Waters assigned to this class are designated for the use of protection and propagation of fish, other aquatic life and wildlife; for primary and secondary contact recreation; and for shellfish harvesting with depuration (Restricted Shellfish Areas).

Class SC - Waters assigned to this class are designated for the protection and propagation of fish, other aquatic life and wildlife; and for secondary contact recreation.

Surface waters in Plymouth subject to the rise and fall of the tide are classified SA. The coastal waters along Plymouth's shoreline, Plymouth Bay and Plymouth Harbor are also designated Class SA. However, the Harbor falls short of SA quality, due to excessive bacterial contamination. Shellfishing has thus been closed in this area (as discussed in Appendix B).

Associated with these classifications are minimum numerical and narrative limits on pollutants. Numerical criteria are used wherever it is reasonable to do so. However, narrative criteria are also necessary in some cases, particularly with respect to aesthetic considerations. The following criteria are applicable to waters in Plymouth:

Narrative Requirements for All Waters

Parameter	Criteria
1. Aesthetics	All waters shall be free from pollutants in concentrations or combinations that: a. Settle to form objectionable deposits; b. Float as debris, scum or other matter to form nuisances; c. Produce objectionable odor, color, taste or turbidity; or d. Result in the dominance of nuisance species.
2. Radioactive Substances	Shall not exceed the recommended limits of the United States Environmental Protection Agency's National Drinking Water Regulations.
3. Tainting Substances	Shall not be in concentrations or combinations that produce undesirable flavors in the edible portions of aquatic organisms.
4. Color, Turbidity, Total Suspended Solids	Shall not be in concentrations or combinations that would exceed the recommended limits on the most sensitive receiving water use.
5. Oil and Grease	The water surface shall be free from floating oils, grease and petrochemicals, and any concentrations or combinations in the water column or sediments that are aesthetically objectionable or deleterious to the biota are prohibited. For oil and grease of petroleum origin the maximum allowable discharge concentration is 15 mg/l.

6. Nutrients	Shall not exceed the site-specific limits necessary to control accelerated or cultural eutrophication.
7. Other Constituents	Waters shall be free from pollutants in concentrations or combinations that:

a. Exceed the recommended limits on the most sensitive receiving water use;

b. Injure, are toxic to, or produce adverse physiological or behavioral responses in humans or aquatic life; or

c. Exceed site-specific safe exposure levels determined by bioassay using sensitive resident species.

The following additional minimum criteria are applicable to freshwater classifications:

Numerical Requirements for Class A Waters

Parameter	Criteria
1. Dissolved Oxygen	Shall be a minimum of 5.0 mg/l in warm water fisheries and a minimum of 6.0 mg/l in cold water fisheries.
2. Temperature	Shall not exceed 83 deg. F (28.3 deg C) in warm water fisheries or 68 deg. F (20 deg. C) in cold water fisheries nor shall the rise resulting from artificial origin exceed 4.0 deg. F (2.2 deg. C).
3. pH	As naturally occurs.
4. Total Coliform Bacteria	Shall not exceed a log mean for a set of samples of 50 per 100 ml during any monthly sampling period.
5. Turbidity	None other than of natural origin.

6.	Total Dissolved Solids	Shall not exceed 500 mg/l.
7.	Chlorides	Shall not exceed 250 mg/l.
8.	Sulfates	Shall not exceed 250 mg/l.
9.	Nitrate	Shall not exceed 10 mg/l as nitrogen.

Numerical Requirements for Class B. Waters

1.	Dissolved Oxygen	Shall be a minimum of 5.0 mg/l in warm water fisheries and a minimum of 6.0 mg/l in cold water fisheries.
2.	Temperature	Shall not exceed 83 deg. F (28.3 deg. C) in warm water fisheries or 68 deg. F (20 deg. C) in cold water fisheries nor shall the rise resulting from artificial origin exceed 4.0 deg. F (2.2 deg. C).
3.	pH	Shall be in the range of 6.5 - 8.0 standard units and not more than 0.2 units outside of the naturally occurring range.
4.	Fecal Coliform Bacteria	Shall not exceed a log mean for a set of samples of 200 per 100 ml, nor shall more than 10 percent of the total samples exceed 400 per 100 ml during any monthly sampling period, except as provided in 314 CMR 4.02(1).

The following additional minimum criteria are applicable to coastal and marine waters:

Numerical Requirements for Class SA Waters

Parameter		Criteria
1.	Dissolved Oxygen	Shall be a minimum of 6.0 mg/l.
2.	Temperature	None except where the increase will not exceed the recommended limits on the most sensitive water use.

3.	pH	Shall be in the range of 6.5-8.5 standard units and not more than 0.2 units outside of the naturally occuring range.
4.	Total Coliform Bacteria	Shall not exceed a median value of 70 MPN per 100 ml and not more than 10% of the samples shall exceed 230 MPN per 100 ml in any monthly sampling period.

<u>Numerical Requirements for Class SB Waters</u>

1.	Dissolved Oxygen	Shall be a minimum of 6.0 mg/1.
2.	Temperature	None except where the increase will not exceed the recommended limits on the most sensitive water use.
3.	pH	Shall be in the range of 6.5 - 8.5 and not more than 0.2 units outside of the naturally occurring range.
4.	Total Coliform Bacteria	Shall not exceed a median value of of 700 MPN per 100 ml and not more than 20% of the samples shall exceed 1000 MPN per 100 ml during any monthly sampling period, except as provided in 314 CMR 4.02(1).

The following antidegradation provisions of the State Water Quality Standards (314 CMR 4.00) are relevant to the waters of Plymouth:

<u>Section 4.04(2) Protection of High Quality Waters</u>. From and after the date these regulations become effective, waters designated by the Division in 314 CMR 4.05(5) whose quality is or becomes consistently higher than that quality necessary to sustain the national goal uses shall be maintained at that higher level of quality unless limited degradation is authorized by the Division. Limited degradation may be allowed by the Division as a variance from this regulation as provided in Section 4.04(6).

<u>Section 4.04(3) Protection of Low Flow Waters</u>. Certain waters will be designated by the Division in Section 4.05(5) of these standards for protection under this section due to their inability to accept pollution

discharges. New or increased discharges of pollutants to waters so designated are prohibited unless a variance is granted by the Division as provided in Section 4.04(6).

Section 4.04(5) Control of Eutrophication. The discharge of nutrients, primarily phosphorus or nitrogen, to waters of the Commonwealth will be limited or prohibited by the Division as necessary to prevent excessive eutrophication of such waters. There shall be no new or increased discharges of nutrients into lakes and ponds, or tributaries thereto. Existing discharges containing nutrients which encourage eutrophication or growth of weeds or algae shall be treated. Activities which may result in non-point discharges of nutrients shall be conducted in accordance with the best management practices reasonably determined by the Division to be necessary to preclude or minimize such discharges of nutrients.

Section 4.04(6) Variances. A variance to authorize a discharge in water designated for protection under Section 4.04(2) may be allowed by the Division where the applicant demonstrates that:

a. The proposed degradation will not result in water quality less than specified for the class; and

b. The adverse economic and social impacts specifically resulting from imposition of controls more stringent than secondary treatment to maintain the higher water quality are substantial and widespread in comparison to other economic factors and are not warranted by a comparison of the economic, social and other benefits to the public resulting from maintenance of the higher quality water.

In addition to the above, the applicant for a variance to authorize a discharge into waters designated for protection under Section 4.04(3) must demonstrate that:

c. Alternative means of disposal are not reasonably available or feasible.

Massachusetts Ocean Santuaries Act

Plymouth Harbor and Plymouth Bay are included in the Cape Cod Bay Ocean Sanctuary under the Massachusetts Ocean Sanctuaries Act, M.G.L. ch. 132A, ss. 14 to 16 and 18. This law prohibits any new discharges of wastewater into the sanctuary and also prohibits any increase to a licensed discharge existing as of December 8, 1971. The implications of this law are discussed in Chapter 8.

Federal Effluent Quality Standards

The basic Federal water pollution control requirements and objectives are embodied in the 1972 Water Pollution Control Act Amendments (Public Law 92-500). For municipal treatment facilities, three goals are associated with these requirements:

> Sections 301(b)(1)(B) and (C) of the Act require "secondary treatment" by July 1, 1977 for existing plants or June 1, 1978 for new construction, and "any more stringent limitation, including those necessary to meet water quality standards, or schedules of compliance, established pursuant to any State Law or regulations...or any other Federal law or regulation, or required to implement any applicable water quality standard established pursuant to this Act."
>
> Section 301(b)(2)(B) and 201(g)(2)(A) of the Act require BPWTT (best practicable wastewater treatment technology) by July 1, 1983.
>
> Section 101 of the Act states that "...it is the national goal that the discharge of pollutants into the navigable waters be eliminated by 1985."

These goals, as amplified in guidelines issued by the EPA, are discussed further below.

Secondary Treatment. As defined in guidelines issued by the EPA in August of 1973, and as modified in July of 1976, secondary treatment generally means greater than 85 percent removal of BOD5 (5-day biochemical oxygen demand) and suspended solids (SS). The following specific criteria must be met:

Parameter	Weekly Average	Monthly Average
BOD5	45 mg/l	30 mg/l
SS	45 mg/l	30 mg/l
pH	6.0 - 9.0	6.0 - 9.0

Monthly average for BOD5 and SS shall not exceed 15 percent of the mean of influent samples collected at the same time (85 percent removal).

The above criteria apply to all direct discharges to receiving waters. In the case of BOD5 and SS, monthly and weekly averages are defined as the arithmetic mean of values for effluent samples collected over a period of 30 and 7 consecutive days, respectively. The limitation for fecal coliform bacteria has been deleted from EPA's definition of secondary treatment. The State will determine disinfection requirements based on water quality and local needs.

<u>BPWTT - General</u>. In its guidelines for BPWTT, PRM 79-3 Attachment F, the EPA has defined three wastewater management techniques, and developed criteria defining the effluent characteristics to be attained for each. The three techniques are: (1) treatment and discharge, (2) land application of wastewater, and (3) treatment and reuse of wastewater.

<u>BBWTT for Treatment and Discharge</u>. This guideline defines BPWTT for publicly-owned treatment works employing treatment and discharge into navigable waters as achieving, as a minimum, the treatment attainable by the application of secondary treatment as

defined in 40 CFR (Code of Federal Requirements) 133, Appendix C. Requirements for additional treatment, or alternative management techniques, will depend on several factors including availability of cost-effective technology, cost and the specific characteristics of the affected receiving water body.

BPWTT for Land Application Techniques. Criteria are defined for BPWTT for land application techniques such as spray irrigation, rapid infiltration (infiltration-percolation), overland flow, and evaporation ponds. Where the wastewater returns via an underdrain system to a point-source discharge into navigable waters, the criteria are the same as those for treatment and discharge as given above.

For purposes of establishing eligibility for grant funding, the discharge of pollutants onto the land should not degrade the air, land or water (navigable or groundwater), and should not interfere with:

1. The attainment or maintenance of public health;
2. State and local land use policies, water quality, protection of public water supplies, agriculture, and industrial users;
3. Propagation of a balanced population of aquatic and land flora and fauna; and
4. The recreational activities in the area.

According to EPA Regulations, the groundwater resulting from the land application of wastewater, including the affected native groundwater, shall meet the following criteria:

Case 1. The groundwater can potentially be used for drinking water supply

1. The maximum contaminant levels for inorganic chemicals and organic chemicals specified in the National Interim Primary Drinking Water Regulations (40 CFR 141) (Appendix D) for drinking water supply systems should not be exceeded except as indicated below (see Note 1).

2. If the existing concentration of a parameter exceeds the maximum contaminant levels for inorganic chemicals or organic chemicals, there should not be an increase in the concentration of the parameters due to land application of wastewater.

Case 2. The groundwater is used for drinking water supply.

1. The criteria for Case 1 should be met.
2. The maximum microbiological contaminant levels for drinking water supply systems specified in the National Interim Primary Drinking Water Regulations (40 CFR 141) (Appendix D) should not be exceeded in cases where the groundwater is used without disinfection (see Note 1).

Case 3. Uses other than drinking water supply.

1. Groundwater criteria should be established by the Regional Administrator based on the present or potential use of groundwater.

The Regional Administrator in conjunction with the appropriate State officials and the grantee shall determine on a site-by-site basis the areas in the vicinity of a specific land application site where the criteria in Case 1, 2 and 3 shall apply. Specifically determined shall be the monitoring requirement appropriate for the project site. This determination shall be made with the objective of protecting the groundwater for use as a drinking water supply and/or other designated uses as appropriate and preventing irrevocable damage to groundwater. Requirements shall include provisions for monitoring the effect on the native groundwater.

Note 1. Any amendments of the National Interim Primary Drinking Water Regulations and any National Revised Primary Drinking Water Regulations hereafter issued by EPA prescribing standards for public water systems relating to inorganic chemicals, organic chemicals or microbiological contamination shall automatically apply in the same manner as the National Interim Primary Drinking Water Regulations".

The maximum contaminant levels in the National Interim Primary Drinking Water Regulations (40 CFR 141) were published in final form in the FEDERAL REGISTER on December 24, 1975. In accordance with the criteria for best practicable waste treatment, 40 CFR 141 should be consulted in its entirety when applying the standards contained therein to wastewater treatment systems employing land application techniques and land utilization practices.

<u>BBWTT for Treatment and Reuse</u>. Under this alternative the total quantity of any pollutant in the effluent from a reuse project which is directly attributable to the effluent from a publicly-owned treatment works must not exceed that which would have been allowed under the other two alternatives previously discussed.

<u>National Pollutant Discharge Elinination System</u>. The NPDES was established under Section 402 of the Federal Water Pollution Control Act Amendments of 1972 to provide a national system for issuance of permits to control the discharge of pollutants into the nation's waters. The existing treatment plant has a permit for the discharge of 1.75 mgd of secondary effluent to Plymouth Harbor, a copy of which is included in Appendix N. New or expanded wastewater treatment facilities would require the issuance of a new NPDES permit.

CHAPTER 3

EXISTING WASTEWATER FACILITIES

General

This chapter describes Plymouth's planning area in terms of its existing wastewater collection and treatment facilities. Included is the nature and extent of current flows and wasteloads and the performance of pertinent facilities.

The densely developed North Plymouth area, which includes approximately 2000 acres, is served by an existing wastewater collection system. Wastewater is conveyed to the Town's wastewater treatment plant located on the waterfront for treatment and discharge to Plymouth Harbor through an outfall. Approximately 11,240 people, or about thirty five percent of the town's population, are served by these facilities.

On-lot wastewater disposal systems are utilized by the remaining 65 percent of the population of the Town and consist mainly of septic tanks with leaching areas or cesspools.

Wastewater Collection System

General Description. Plymouth's existing wastewater collection system contains about 215,000 lineal feet (41 miles) of separate sewer pipe (excluding building connections), two conventional wastewater pump (lift) stations, two package pump stations, three small wastewater lift stations and associated force mains. Approximately 75 percent of the collection sewers were constructed before 1918. About 90 percent of the sewers are vitrified-clay with either cement-mortar or bituminous hot-poured joints. PVC sewer pipe and reinforced-concrete interceptor pipe with rubber gasket joints have been used for construction during the past 20 years. During the past 10 years, the Town has completed many sewer separation projects. However, it is known that some roof leaders and catch basins are still connected to the Town's sanitary sewers. Further detailed description of the Town's collection system can be found in the I/I Analysis included in Appendix J of this report.

The major components of the existing wastewater collection system, including collection sewers, pump stations, and service area limits are shown in Figure 3-1. The system has been divided into 14 subareas which correspond to natural drainage areas. The outlets of these drainage areas are spread out along the shoreline and necessitate having a wastewater interceptor system that is comprised of large diameter sewers at minimum slopes built along the seashore to the north and south. The three major interceptors along the shoreline which collect wastewater from the 14 subareas are:

1. The Cordage Interceptor
2. The Knapp Terrace Interceptor
3. The Harbor Interceptor

The Cordage Interceptor. This interceptor is an 18-inch reinforced concrete pipe (RCP) with a 15-inch plastic liner in some sections and extends from the Knapp Terrace Pump Station northerly along the shoreline to the Cordage Industrial Park.

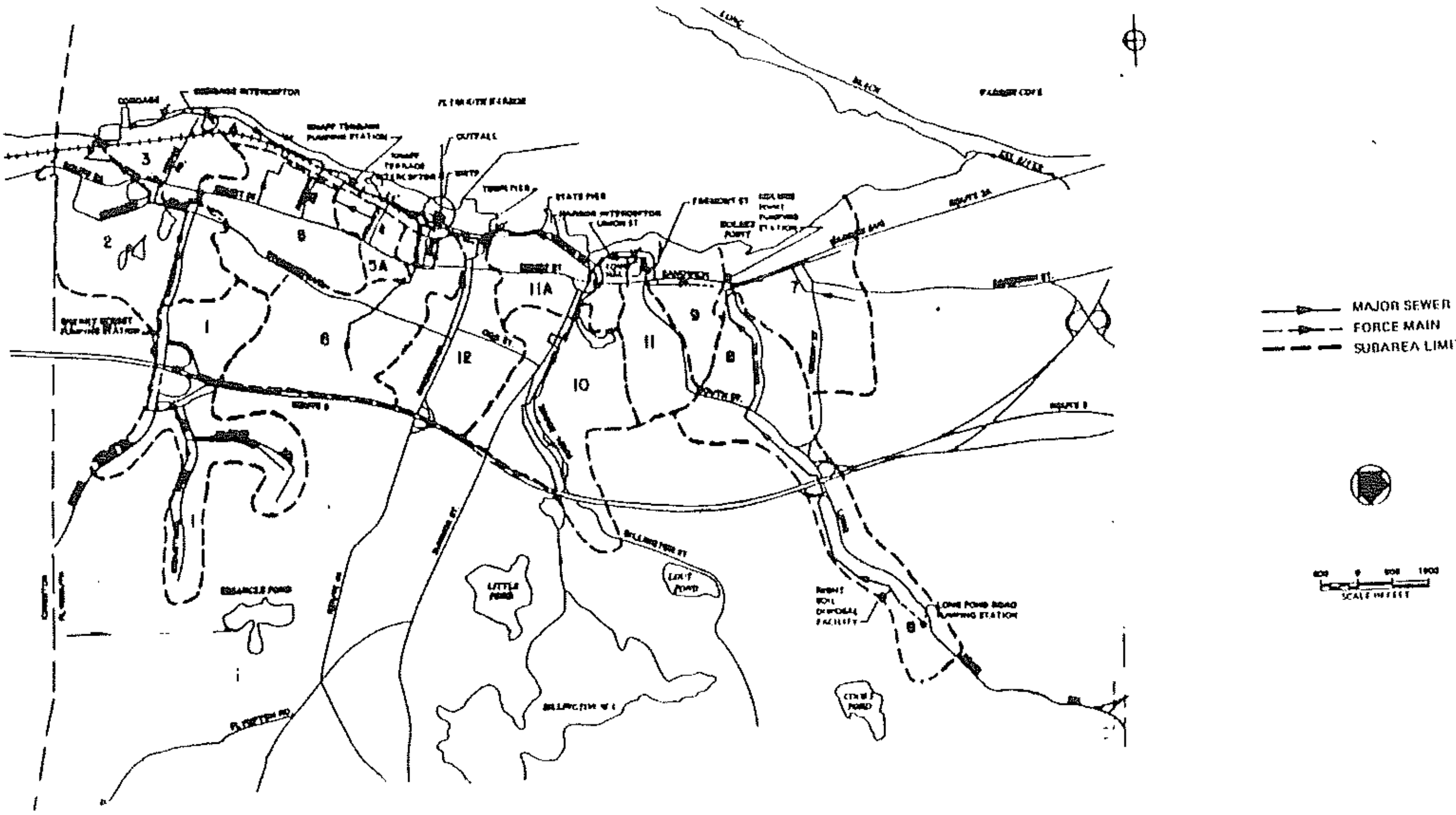

FIG. 3-1 EXISTING WASTEWATER
FACILITIES AND SERVICE SUBARE

This interceptor was constructed in 1940 and placed into service in 1969. Wastewater from Subareas 1, 2, and 3 (Cordage Park) is transported to the Knapp Terrace Pump Station by the Cordage Interceptor. Subarea 1 contains the most recently constructed sewers, including the industrial development to the west of Route 3, and is served by the Cherry Street package pump station. The sewers serving Cordage Park and the area north of Store Pond in Subarea 2 are the oldest, dating back to the early 1900's. Subarea 3 contains a small lift station serving the residential development on Castle Court.

Knapp Terrace Interceptor. This interceptor is a 24-inch RCP which extends northerly from the WWTP to the Knapp Terrace Pump Station. This interceptor conveys wastewater from Subareas 5 and 5A as well as the pumped discharge from Knapp Terrace Pump Station. This sewer was constructed in the late 1960's along with the construction of the WWTP. A private pump station serves the shopping area on Court Street in Subarea 5A.

The Harbor Interceptor. This interceptor includes a 30-inch section of reinforced concrete pipe which was constructed in the late 1960's along the shoreline between the Town Pier and the State Pier, and a section of 18-inch vitrified clay pipe constructed around 1918 and extending South on Water Street to Union Street. The 18-inch interceptor then extends south on Union Street to Fremont Street, in Fremont Street to Sandwich Street and finally southerly in Sandwich Street to a point near Mt. Pleasant Street. The Harbor Interceptor carries the pumped flow from the Holmes Point Pump Station which serves Subareas 7 and 8 and the gravity flow from Subareas 6, 9, 10, 11, 11A and 12. Flow from the Night Soil (septage) Disposal Facility on Long Pond Road in Subarea 8 is conveyed to Holmes Point Pump Station, as well as the flow from the Long Pond Road package pump station which serves the ice skating rink and the Plymouth - Carver Regional Intermediate School. A manual wastewater by-pass is located at Holmes Point Pump Station. An ejector station serving Winter Street is located in Subarea 9.

Capacity Analysis of Existing Interceptors. The existing wastewater collection system has been analyzed in detail to determine the ability of the Town's major interceptor sewers and lift stations to handle existing flows from currently sewered areas. The purpose of this analysis is twofold: first, to identify deficient sewer segments, if any, where only remedial measures such as replacement, relief, or flow diversions can be considered; and secondly, to identify sewers with adequate capacity under present conditions and others with reserve capacity.

Estimated present peak dry-weather flows were determined by adding peak infiltration to the peak wastewater component for the tributary sewered area, as described in Chapter 7. Wet-weather peak flows were then determined by adding inflow rate estimates to the appropriate dry-weather totals.

The results of the capacity analysis are presented in Table 3-1. Estimated present peak flows for selected segments of

TABLE 3-1 CAPACITY ANALYSIS OF EXISTING MAJOR SEWERS

Sewer segment From	To	Location	Existing sewer: Size, in.	Date Constructed	Slope, ft/ft	Capacity, mgd	Present peak flows, in mgd: Dry weather total	Inflow estimate	Wet weather total
Cordage Interceptor									
Railroad Tracks	Hedge Rd.	Cordage Park	18"	1969	.0013	2.40	0.70	0.25	0.95
Hedge Rd. (Knapp Terrace P.S.	Knapp Terrace P.S. 0.86 mgd(1) @ 24' TDH)	Beach	18"	1940	.0013	2.40	1.43	0.25	1.68
Knapp Terrace Interceptor									
Knapp Terrace P.S.	Wastewater Treatment Plant	Water Street	24"	1969	.0012	5.00	1.81	0.8	2.61
Harbor Interceptor									
(Holmes Point P.S.	1.08mgd(1) @ 30'TDH)								
Winter Street	Fremont Street	Sandwich Street	10 12	1918 1969	.0150 .0150	3.8	1.10	0.25	1.35
			15 18	1969 1918	.0022 .0048	5.6	1.11	0.25	1.36
Sandwich Street	Union Street	Fremont Street	18	1918	.0033	3.30	1.11	0.25	1.36
Fremont Street	Water Street	Union Street	18	1918	.0055	4.30	1.28	0.25	1.53
Union Street	State Pier	Water Street	18	1918	.0012	2.00	1.66	0.25	1.91
State Pier	Wastewater Treatment Plant	Water Street	30	1969	.0010	8.40	2.79	0.8	3.59

(1) Design average flow

these sewers are presented in the table together with estimates of sewer capacity for comparison. The theoretical hydraulic capacities of the sewers have been estimated by using Manning's formula with an "n" value of 0.013. The location of each sewer segment is indicated on Figure 3-1.

As can be seen in Table 3-1, all the major sewers in the sewer system have the ability to convey present estimated peak wet-weather flows and also have some reserve capacity for varing amounts of additional flow. The only exception is the old 18-inch interceptor in Water Street between State Pier and Union Street, which is presently at capacity with little additional capacity for any sewer expansion.

Existing Infiltration/Inflow. In the Infiltration/Inflow (I/I) Analysis (Appendix J), it was concluded that there was possibly "excessive" I/I in the collection system, and a sewer system evaluation survey (SSES) was recommended to confirm this. It was estimated that 0.58 million gallons per day (mgd) of peak I/I could cost-effectively be removed. This SSES survey was begun during the summer of 1983 and will address in detail the cost-effectiveness of I/I reduction.

In 1980, the EPA published the results of a 2-year study that evaluated the national I/I program. The study shows that actual I/I reductions generally have been substantially less than originally predicted, especially reductions in infiltration. At the present time, the EPA is reevaluating its I/I policy. It is anticipated that future policy might be based on the following assumptions:

1. Infiltration reduction is not cost-effective, especially in old systems like Plymouth's.

2. Inflow reductions in the range of 20 to 40 percent (the range which was considered in the I/I Analysis of Plymouth's system) may be cost-effective.

3. Occasional sewage overflows might be tolerated if the impact on receiving waters were minimal.

Septage Facilities. The Night Soil Disposal Facility was constructed in 1972 and is located approximately 1 1/2 miles from the WWTP on Long Pond Road. Septage haulers unload septage (waste removed from septic tanks) collected from within Plymouth at this facility. After the septage enters the sewer system, it becomes mixed with and diluted by wastewater entering the sewer en route to the WWTP. Grit present in the septage is retained in the facility and is regularly removed and landfilled.

Wastewater Treatment Plant

The Town's WWTP, constructed during the late 1960's and put into operation in 1969, utilizes the activated sludge process to provide the incoming wastewater with secondary levels of treatment. It is located on a 3-acre site on Water Street just west of Town Pier. A schematic flow diagram of the treatment process is shown in Figure 3-2. Wastewater enters the plant from a 30-inch influent sewer, passes through a preliminary treatment system to the plant's influent pump station, and is pumped to the secondary treatment system. The flow then passes by gravity through the secondary system into the outfall, which extends 1840 feet into Plymouth Harbor.

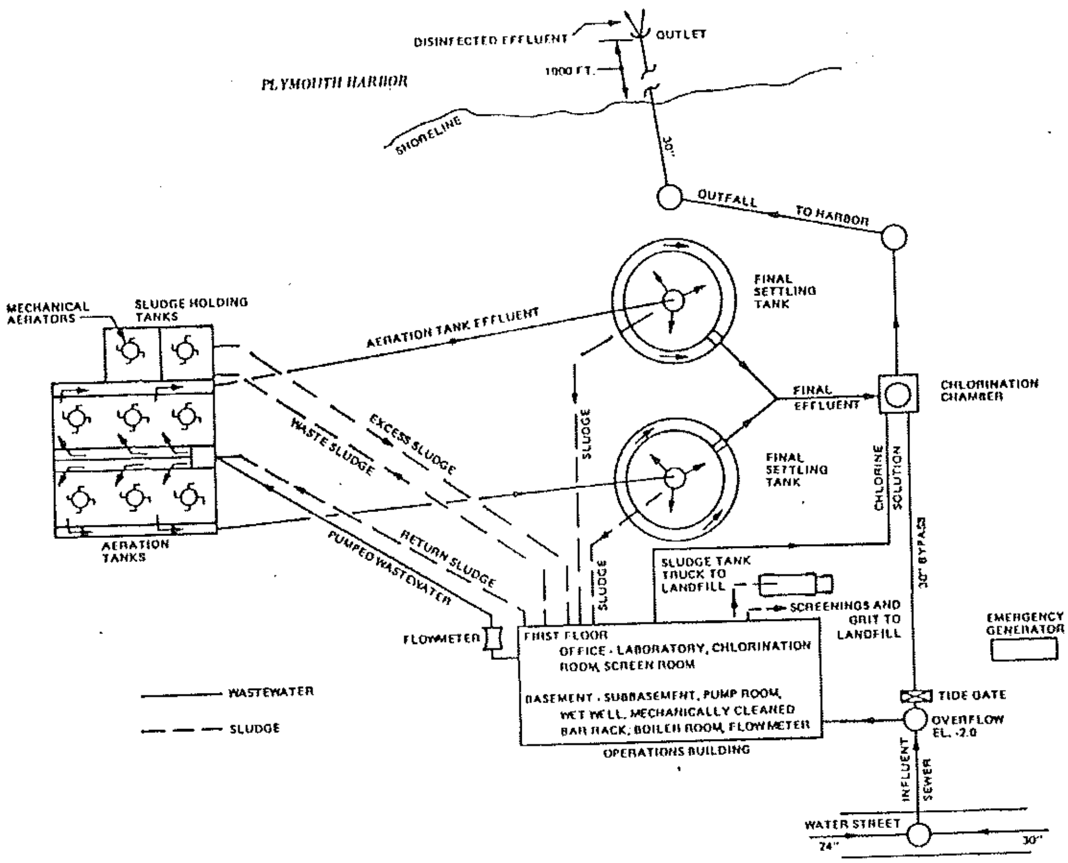

FIG. 3-2 SIMPLIFIED FLOW DIAGRAM OF PLYMOUTH'S EXISTING WASTEWATER TREATMENT

3-9

Preliminary treatment is provided by a bar rack, which removes objects such as boards and rags, and a grit removal system. Screenings are mechanically removed from the bar rack, deposited in barrels, and subsequently landfilled. The wastewater passes through the bar rack and flows into the wet well, from which four influent pumps pump wastewater through a flow meter into the secondary treatment system. Solids which settle in the wet well are pumped to a cyclone degritter and classifier, which dewaters the solids, retains the grit for disposal, and returns the organic matter back to the wet well. The grit removed from the classifier is trucked to a landfill and buried. The rate of wastewater flow is measured in the flow tube and recorded in the operations building.

The secondary treatment process units include two aeration tanks (each containing three mechanical aerators), two final settling tanks, and chlorination facilities. The ocean outfall pipe is the only facility available to provide chlorine contact time. The basic design data for the major treatment units is tabulated in Appendix A of this report.

The aeration and settling units utilize the extended-aeration variation of the activated sludge process. By this process, the substances which would exert an oxygen demand on the receiving waters are metabolized by microorganisms which are maintained under controlled conditions in the aeration tanks. These organisms convert organic matter to carbon dioxide, which is released in the form of a harmless gas, and water. They also absorb and adsorb other pollutants in the wastewater. Oxygen is furnished to the microorganisms by aerators which are operated intermittently to conserve energy while maintaining adequate dissolved oxygen levels. The "mixed liquor" which contains these microorganisms is conveyed from the aeration tanks to the settling tanks, where the microorganisms and other solid matter settle out. Most of the settled solids, or "activated sludge", are returned to the aeration tanks to maintain the population of microorganisms. The excess sludge is wasted to the sludge holding tanks where it is digested aerobically and stored for eventual delivery by truck to sludge drying beds at the landfill in Manomet. Scum is also collected from the surfaces of the settling tanks and conveyed by truck to the sludge drying beds at the landfill.

Flows, Wasteloads, Performance and Staffing of WWTP

Wastewater flows, wasteloads and plant performance data for 1981 and 1982 are presented in Table 3-2, which summarizes treatment plant records.

Wastewater Flow. The WWTP was designed to treat an average daily wastewater flow of 1.75 mgd and a peak flow rate of 5.2 mgd. Average annual daily flows recorded at the plant for 1981 and 1982 were about 2.6 and 2.7 mgd, respectively. The maximum daily flow was 4.6 mgd (recorded on June 6, 1982) and the peak flow rate was 5.2 mgd (which is the peak pumping capacity).

Wasteloads. From Table 3.2, the following average wasteloads of BOD5 and SS received at the plant may be calculated for the year 1982:

TABLE 3-2. TREATMENT PLANT PERFORMANCE DATA

Month	Average flow, mgd: For month	Average flow, mgd: Sampling days	No. of days sampled	BOD$_5$ Influent mg/l	BOD$_5$ Effluent mg/l	BOD$_5$ % red	SS Influent mg/l	SS Effluent mg/l	SS % red
1981									
Jan	2.34	2.32	9	233	6	97	132	17	87
Feb	2.55	2.68	8	235	13	95	173	21	88
Mar	2.35	2.32	8	136	11	92	144	26	82
Apr	2.54	2.56	9	111	4	96	217	15	93
May	2.52	2.52	9	174	4	98	178	12	93
Jun	2.64	2.67	8	132	5	96	198	18	91
Jul	2.64	2.63	9	152	5	97	171	16	91
Aug	2.73	2.74	8	101	3	97	187	16	91
Sep	2.67	2.71	9	148	5	97	200	17	91
Oct	2.62	2.64	9	156	6	96	262	28	89
Nov	2.52	2.53	8	147	10	93	247	30	88
Dec	2.69	2.69*	9	168	6	96	214	23	89
AVG.	2.57	2.58	103	158	7	96%	194	20	90
1982									
Jan	2.61	2.57	8	146	10	93	187	39	79
Feb	2.74	2.76	8	192	12	94	203	26	92
Mar	2.67	2.70	9	190	7	96	215	23	89
Apr	2.70	2.72	9	168	10	94	272	54	80
May	2.65	2.69	8	183	17	91	277	61	78
Jun	2.90	2.98	9	209	12	94	230	29	87
Jul	2.57	2.64	9	132	6	95	236	34	86
Aug	2.76	2.81	8	223	5	98	230	26	89
Sept	2.64	2.69	8	198	5	97	247	21	91
Oct	2.62	2.74	9	139	5	96	272	32	88
Nov	2.70	2.81	8	173	3	98	206	30	84
Dec	2.70	2.70	8	124	3	97	163	32	80
AVG.	2.69	2.73	101	173	8	95%	228	34	85
AVG. 1981-1982:	2.63	2.66	204	166	8	95%	211	27	88

*Estimated. Sample dates were not included in WWTP Monthly Report for this month.

BOD_5 - 3900 lb/day (173 mg/l @ 2.69 mgd)

SS - 5200 lb/day (228 mg/l @ 2.69 mgd)

Since approximately 13,000 gallons per day of septage is included in the raw wastewater, a portion of these loadings is derived from the septage. Assuming average septage concentrations of 5000 mg/l for BOD_5 and 13,300 mg/l for SS, approximately 550 lb/d of BOD_5 and 1470 lb/d of the SS may be attributed to septage. Based on an assumed sewered population of 11,250, assumed commercial and industrial flow of 0.31 mgd, and assumed commercial and industrial wastewater BOD_5 and SS concentrations of 200 mg/L and 250 mg/L, respectively, it may be estimated that current wasteload contributions are distributed as follows:

	BOD_5	SS
domestic, lb/d	2850	3030
septage, lb/d	550	1470
industrial and commercial lb/d	500	700
Total, lb/d	3900	5200

Samples of the raw wastewater were collected on November 28 and 29, 1983 and analyzed for toxic parameters (i.e., priority pollutants). The sampling methodology and results are presented in Appendix O. None of the parameters measured were found to be present in concentrations that might inhibit activated sludge processes. However, the concentrations of chromium (0.95 mg/l) and copper (0.41 mg/l) are high enough to warrant a review of the industries and commercial establishments in the area to identify and monitor potential dischargers of heavy metals.

Plant Performance. Table 3-2 presents performance data for the existing plant for about 200 sampling days during the years 1981 and 1982. From the data, it can be seen that, although the design average daily flow is being exceeded, the plant removes almost 96 percent of the influent BOD5, easily complying with its NPDES monthly average permit requirement of 20 mg/l. However, average effluent concentrations for suspended solids exceeded the NPDES monthly average permit value (30 mg/l) during several months of 1982, as solids are increasingly being washed out of the final settling tanks during peak flow periods. This problem is discussed further in Chapter 4.

In extended aeration plants such as Plymouth's, nitrification (conversion of ammonia nitrogen to nitrate nitrogen) can be provided by operating the aerators in such a way that the dissolved oxygen level in the aeration tanks is maintained at 2 mg/L or higher. This promotes the growth of nifrifying bacteria, which oxidize the ammonia in the wastewater to nitrate. Such oxidation reduces the oxygen demand on the receiving waters. Data collected over the years at the Plymouth WWTP indicates that nitrification normally does occur.

Staffing. In the WWTP "Operation and Maintenance" manual prepared for Plymouth in April 1971 by Metcalf & Eddy, a staff of

5 plant personnel was recommended. The present staff at the WWTP consists of the following:

1 Superintendent and Chief Operator

1 Operator

2 Assistant Operators

2 Maintenance

The WWTP is manned during the week one shift per day. The staff also includes 3 sewer maintenance personnel, who are responsible for the operation and maintenance of the sewer system and pump stations, and report to the Superintendent and Chief Operator.

Sludge Dewatering and Disposal

Sludge and scum from the WWTP are hauled in a tank truck to the Town's 11 sand drying beds, located at the Manomet Landfill. Averages of about 120,000 gallons per month of sludge (with a solids content of 1-1/2 to 2 percent) and 10,000 to 80,000 gallons per month of scum (the higher figure occurring during the summer months) are presently hauled.

Samples of the undewatered sludge and scum have been collected and analyzed for toxic parameters and nutrients. EP toxicity tests were also run. The sampling methodology and results are presented in Appendix D. The concentrations of metals in the sludge and scum leachates are below levels that would classify these substances as "hazardous." However, relatively high levels of copper, chromium and toluene were found in the sludge and scum. While the levels found are below those expected to cause inhibition of the activated sludge process, they are high enough to cause the sludge to be classified as Type III (restricted use) under the DEQE criteria for land application of sludge.

The measured nitrogen and phosphorus contents of the sludge, on a dry weight basis, were 6 percent and 0.9 percent, respectively, while those of the scum were 12 percent and 1.3 percent.

On-Lot Wastewater Disposal Systems

General. The remaining developed non-sewered areas in Plymouth use cesspools or septic tank systems for on-lot wastewater treatment and disposal. All new construction must be provided with a septic tank and soil absorption (leaching area) system in accordance with rules and regulations of the Plymouth Board of Health and Title 5 of the Environmental Code of the Commonwealth of Massachusetts (310 CMR 15.00). Examination of soils, measurements of groundwater elevations and percolation tests are used to determine the suitability of any property to be served by an on-lot disposal system. This data is reviewed by the Board of Health in order to determine whether approval of a system and issuance of a permit will be granted.

Types of On-Lot Systems. A brief description of the various types of on-lot systems found in Plymouth follows.

The pit privy (more commonly known as an outhouse), is typically a small, shallow pit or trench which normally receives only human waste and paper. A small shed is generally constructed over the pit to provide privacy and for aesthetic reasons. The solids are retained in the pit while the liquid fraction seeps into the soil.

The cesspool is typically a 5 to 6 foot diameter hole several feet below ground surface and lined with stone. The facility receives wastewater directly from the house sewer. The larger solids settle to the bottom of the cesspool or are otherwise trapped inside while the liquid fraction seeps out through openings in the sides and bottom.

The septic tank and leaching area is the most modern type of on-lot system and the only type which will be approved for new construction. It is more reliable than the two aforementioned systems. In Plymouth, the most common type of leaching area used is the leaching pit. However, a number of other leaching systems are available and in common use today. The septic tank provides for the removal of scum, grease and settleable solids from the wastewater by gravity separation. Microorganisms in the tank digest much of the settled solids material. The clarified effluent flows to the leaching pit, where holes in the sides of the pit allow effluent to flow through the surrounding media of crushed stone, and then continue on through the soil medium.

The proper functioning of these wastewater disposal systems relies upon the ability of adjacent soils to receive the wastewater, treat it, and allow it to be transmitted via the natural soil system. Soil bacteria existing in soil are capable of assimilating the organic materials contained in wastewater and utilizing the oxygen of the atmosphere to oxidize these organic materials to carbon dioxide and water. These soil bacteria are essential to the proper functioning of the leaching area, because they prevent the accumulation of wastes which would eventually clog the leaching area and prevent further seepage of waste-water. However, even under the most efficient conditions (including proper maintenance) there is always a small amount of insoluble organic and inorganic material from the metabolism of the wastes, and this is discharged to the leaching area. This residue accumulates in the pores of the soils and eventually clogs it. Thus, the percolation potential of the soil under the leaching area will eventually be exhausted. In the case of a cesspool, which combines the functions of the septic tank and leaching area in a single structure, more frequent problems may occur due to greater clogging potential.

Ideally, an on-lot system can provide satisfactory service for 10, 15, or more years, depending on soils and other factors. However, if a leaching area becomes clogged, wastewater is no longer able to pass through the pores of the earth and tends to pond and rise to the surface, and as a result the leaching area must be replaced or installed in another location. Replacement of a leaching area is feasible only where ample space is available, and where subsurface conditions meet all criteria required for new installations. When setting minimum lot sizes in residentially-zoned areas to be served by on-lot wastewater disposal systems, consideration should be given to providing ample space for one or more leaching area replacements. For this reason lots should be large enough to allow for expansion or total replacement of permanent on-lot systems. In some areas of Plymouth (as discussed in Chapter 4) lot sizes are very small and problems have resulted.

Because septic tank and cesspool systems depend heavily upon the functions of bacteria for their successful operation, products have been developed purporting to stimulate the action of these bacteria. There are many additives on the market which supposedly enhance the operation of the cesspool or septic tank system; however, none of these remedies has had any demonstrated efficiency under controlled investigations. Therefore, it is the nearly unanimous opinion of those who understand subsurface disposal systems that no chemicals or other additives should be employed in the operation of septic tanks or cesspools. The rules and regulations of the Town's Board of Health prohibit the use of additives.

Sludge and scum eventually accumulate and form relatively compact layers in the septic tank. Although sludge and scum undergo biological decomposition, the rate of decomposition is slower than the incoming rate of solids. Because of this, the accumulated solids (collectively referred to as septage or night soil) must be removed periodically. Private haulers pump septage out of septic tanks and cesspools at the request of the owners and deliver it to the Night Soil Disposal Facility on Long Pond Road.

CHAPTER 4

PROBLEM IDENTIFICATION

General

The purpose of this chapter is to document the nature and extent of existing wastewater related problems found in the existing North Plymouth wastewater collection, treatment and disposal system and in the many individual on-lot systems found throughout the unsewered areas of town.

Problems in Existing Sewered Area

There are several deficiencies in the Town's wastewater collection system and treatment plant (WWTP). The WWTP is hydraulically overloaded, largely due to leakage of groundwater and seawater (infiltration) into the wastewater collection system. The WWTP was designed to treat 1.75 million gallons per day (mgd) of wastewater. However, during 1982, the WWTP treated an average flow of 2.7 mgd, resulting in the violation of the plant's discharge permit (see Appendix N) and the Massachusetts Ocean Sanctuaries Act (which also limits the average discharge to the design value of 1.75 mgd). It is estimated that 56 percent of the wastewater flow during 1982 was infiltration. Much of the collection system was constructed in the early 1900's when wastewater was discharged directly into Plymouth Harbor and little attention was paid to keeping groundwater out of the system.

Many problems are associated with the Town's major interceptor sewers. As shown in the capacity analysis in Chapter 3, portions of the Harbor Interceptor servicing Subareas 7,8,9,10, and 11 are presently at capacity. This 18" interceptor is made of vitrified clay pipe with joints every 2 feet. Because of its deterioration (cracked pipe and leaky joints) due to age and its limited capacity, the Harbor Interceptor between the State Pier and Sandwich Street is in need of replacement. The 15" interceptor in Water Street between Union Street and Sandwich Street is also in need of replacement. The Cordage Interceptor, although it has reserve capacity, is also in need of replacement due to its in very poor condition (separated joints, dips, excessive infiltration, and salt water intrusion). Large volumes of sand must frequently be removed from both main interceptors and pump stations due to open joints. The Town has made numerous attempts to locate and remove infiltration from the Cordage Interceptor with little success through the spring of 1983. Repairs to a section of the interceptor using an insitu liner in the fall of 1983 have apparently reduced flows, but it is too soon to quantify this reduction accurately. Because of the high rate of infiltration into this interceptor, the Town has had to throttle the influent sluice gate to the Knapp Terrace Pump Station at all times. During a heavy rainstorm combined with a high tide, the high water alarm is typically activated at this pump station.

There are several known connections of roof leaders and catch basins to the collection system. These connections are being identified by smoke testing as part of the Sewer System Evaluation Survey being conducted as part of this facilities plan. Further discussion of problems related to the collection system may be found in Appendix J.

The existing collection system lacks pump out facilities for collecting wastes from boats located in the harbor.

Largely because of inflow, the peak rate of wastewater flow reaching the plant reportedly exceeds the design peak capacity of 5.2 mgd approximately six times per year. When such an episode occurs during low tide, the tide gate in the plant bypass channel opens and discharges raw wastewater directly to the chlorine manhole downstream from the normal point of chlorine addition. When an episode occurs at high tide, the excess wastewater backs up into the interceptor sewers; in the past, wastewater has occasionally surcharged and overflowed from a manhole on the Harbor Interceptor off Water Street near Town Pier. The frequency, duration, and rate of bypassing are not recorded.

A second limitation of the WWTP is its inability to dewater sludge. The decanted sludge removed from the sludge holding tanks has a solids content of only 1.5 to 2 percent and must be transported in its dilute state via tank truck to drying beds located at the Manomet Landfill. Over 100,000 gallons of sludge per month are trucked to the drying beds. In addition, large volumes (about 2600 gallons per day) of floating sludge or scum are removed from the surfaces of the clarifiers during the summer months, and this too must be trucked to the drying beds. Sufficient drying bed area is not available for the quantities of sludge and scum produced, and operation of the beds is restricted by wet weather and the requirement that the beds be raked by hand. Sludge frequently is permitted to accumulate in the aeration tanks and clarifiers at the WWTP, and mixed liquor suspended solids concentrations ranged from 6000 mg/l to 11,000 mg/l during 1982. This has resulted in solids loading rates in excess of 70 lb/sq. ft/day on the clarifiers and suboptimal removal of suspended solids, especially during the early afternoon hours when the wastewater flow rate is greater than average. The WWTP thus discharged an effluent containing average suspended solids concentrations in excess of secondary treatment levels (30 mg/L) for six of the twelve months of 1982.

The above problems have caused the Massachusetts Department of Environmental Quality Engineering (DEQE) to limit additional connections to the existing sewer system. In addition, shellfishing areas in Plymouth Harbor have been closed, partially due to the outfall discharge and the fact that bypasses exist at the wastewater treatment plant and at the Holmes Point Pump Station.

There are also other significant deficiencies at the plant. The sludge holding tanks become completely frozen during the winter months; when the holding tanks are not operational, dilute sludge must be wasted directly from the clarifiers to the tank truck. The emergency electrical generator must be started manually in the event of a power failure. In addition, there is a shortage of space for maintenance equipment and parts storage.

Complaints of odors at the Night Soil Disposal Facility on Long Pond Road are occassionally received during the summer months. Odor complaints have also been received from an industry located near the Night Soil Disposal Facility (the sewer into which this facility discharges is vented to the roof of the industry's building). In addition, the lack of screening facilities at the Night Soil Disposal Facility is believed to have contributed to a rapid rate of rag accumulation at the Holmes Point Pump Station which has periodically caused flooding at this station.

Only one potential water quality problem attributable to an unsewered discharge was identified in the existing service area. Samples of water taken from a pipe to the shore on the Kingston side of the Plymouth/Kingston boundary by Whitman & Howard in 1981 had very high bacteria counts. It has been alleged that the contamination of this water is attributable to wastewater from the China Dragon Restaurant.

Problems have also been associated with storm drains within the existing service area. Samples collected by the Department of Water Pollution Control (DWPC) in Westboro from storm drains along the Plymouth Harbor waterfront during a period of dry-weather flow during the summer of 1983 indicated the potential existence of sanitary tie-ins in several cases. Levels of fecal coliform bacteria in excess of 1000/100 ml were reported for drains discharging measurable flow at Boundary Lane, the parking lot utilized by Ocean Spray, the breakwater, and the Mayflower Restaurant. It has been suggested by C.E. Maguire, Inc., that the existence of the shellfish closure zone along the shoreline in Plymouth Harbor is related more to bacterial contamination emanating from storm drains than to the treatment and disposal of wastewater.

Problems in Existing Unsewered Area

The unsewered area of the town uses individual on-lot systems for the disposal of wastewater. While the various types of on-lot systems were described in Chapter 3, this section will describe the different types of problems encountered, the criteria used to determine whether there is a problem or not, and a description of the degree of severity of such problems in the various subareas of Plymouth.

Types of Problems. There are two types of failures which are associated with an on-lot system. A failure of the on-lot system to dispose of the wastes and a failure of the system to properly treat the wastes prior to entering the groundwater. Along with these failures, are many different types of problems as a direct result of the failure. Some problems, such as overflowing septic tanks, are readily detected, while others, such as a contaminated body of water, are not. Examples of typical on-lot system failures and their associated problems are given in Table 4-1.

TABLE 4-1. PROBLEMS RESULTING FROM COMMON WASTEWATER DISPOSAL SYSTEM FAILURES

Cause of Failure	Associated Problem
Disposal Failures	
Blocked pipe	Inability to use bathroom and kitchen facilities
Broken baffle in septic tank	Clogged leaching area, overflowing tank and/or odors
Tilted distribution box	A portion of the leaching area clogged, overflowing tank and/or odors
Undersized leaching area	Inability to fully use water facilities in house and overflowing of tank
Treatment Failures	
Coarse sands	Limited treatment is available and groundwater contamination may occur.
Less than 4' to groundwater from bottom of leaching area	Partial treatment by the soil in the leaching area
Leaching area in groundwater	Very limited treatment by the soil and contamination of groundwater probable.

A disposal failure occurs when the on-lot system is unable to dispose of the wastewater for one of many reasons. One of the more common reasons for this type of failure is an improperly designed on-lot system. Most of the systems constructed prior to 1960 were cesspools, which have since been designated as illegal by the State Environmental Code (Title 5) and by the Plymouth Board of Health due to their inability to properly dispose of and treat the waste. Even many of the on-lot systems of the 1960's and 1970's were designed using criteria which have since been upgraded to reflect the more stringent requirements now believed needed to properly dispose of the waste. A minimum leaching area is required to dispose of the wastewater with the area dependent on the type of soil and the expected flow. If this minimum leaching area is not provided, the leaching field will eventually clog and the on-lot system will fail, resulting in wastewater effluent seeping out of the ground, foul odors, and/or backing up of the toilets and sinks.

A treatment failure occurs when the wastewater passes through the soil underlying the leaching area so quickly that some contaminants may pass into the groundwater. Such problems typically occur when soils are coarse or the distance between the bottom of the leaching area and the groundwater is less than four feet. This type of failure degrades the quality of the underlying groundwater system, and may jeopardize the public's health.

Criteria to Determine Problem Areas. Numerous criteria or factors have been considered in assessing the severity of problems in the individual subareas of Plymouth. Some of these factors are as follows:

1. The soil suitability for on-lot disposal systems
2. Design regulations
3. Density of housing
4. Water quality

5. Board of Health records

6. Questionnaire survey

These criteria have been investigated for each study subarea in Plymouth, and are discussed individually in the paragraphs that follow.

<u>Soil Suitability for On-Lot Disposal Systems</u>. The U.S. Department of Agriculture's Soil Conservation Service (SCS) has mapped and classified Plymouth's soils as presented as Figure 4-1. There are two dominating soil associations in Plymouth, the Carver-Gloucester association and the Carver-Peat association. There are also some smaller pockets of varying soil types existing throughout Plymouth, but they are not as prevalent as the aforementioned associations.

The Carver-Peat association, occupying the western portion of Plymouth, consists of a large, nearly level, sandy outwash plain that is pitted with kettle holes. The Carver series of soils, which consist of dry coarse sands that formed in deep deposits of sand, on nearly level plains and along the steep sides of kettle holes and stream channels, occupies about 70 percent of this association. The Peat series occupies about 10 percent, and secondary soils occupy the remaining 20 percent.

The Carver-Gloucester association, which consists of deep, dry, loamy sands, occupies most of the remaining portion of Plymouth. Its chief features are a series of wooded and boulder-strewn moraine hills with slopes that are moderate to steep and complex. The Carver soils occupy about 50 percent of the

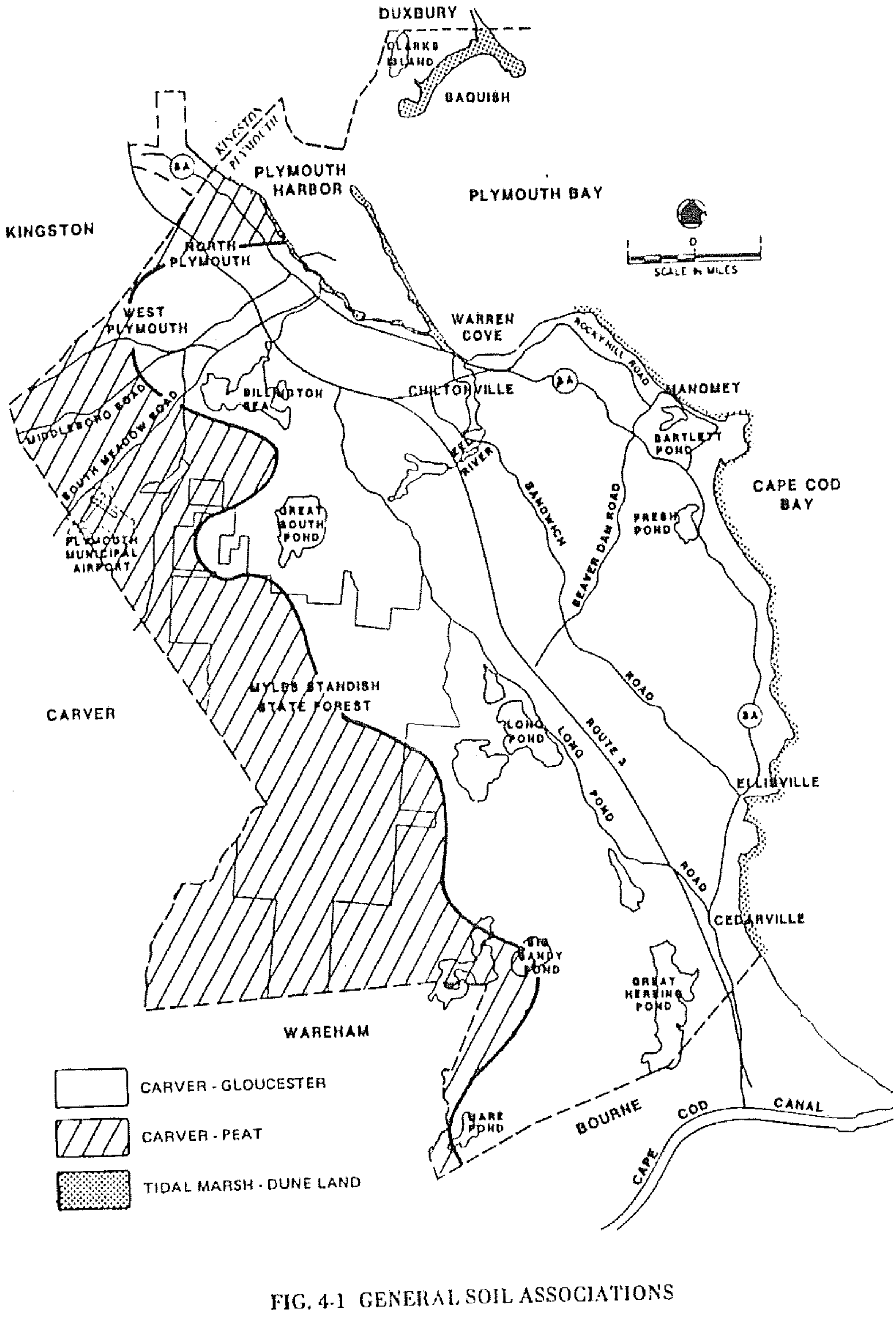

FIG. 4-1 GENERAL SOIL ASSOCIATIONS

association and the Gloucester soils, which have many boulders, about 40 percent. Secondary soils occupy the remaining portion.

The Carver-Gloucester soils are generally very good for the disposal of wastes with their only limitations being steep slopes and shallow depth to groundwater in some areas. Plymouth's Board of Health records on percolation tests indicate that the vast majority of the Town's soils have percolation rates of less than 2 minutes per inch. This means that the applied wastewater will take less than 2 minutes to drain one inch in depth through the ground. However, these soils may not be capable of properly treating all the wastes from the leaching field, especially nitrates. Therefore, careful planning of developments is needed where a groundwater aquifer exists or where water supply wells are located nearby.

Also shown on Figure 4-1 is Tidal Marsh-Dune Land, which consists of regularly flooded organic and mineral deposits and unstable sands along the seashore.

Design Regulations. Prior to 1962 there were few regulations governing on-lot disposal systems. In 1962, the Commonwealth published the first rules and regulations, entitled "Minimum Requirements for Disposal of Sanitary Sewage in Unsewered Areas"; in 1975 the requirements were revised and named, "Title 5 of the State Sanitary Code". And, in January of 1982, the Town of Plymouth published "Supplements to Title 5", which are more stringent regulations than those currently found in Title 5 and are more applicable for the conditions found in Plymouth.

Title 5 contains design criteria for the sizing of on-lot wastewater disposal systems. These are minimum design requirements that all cities and towns must use as guidelines for the design of on-lot systems. The guidelines contain information such as the minimum flow expected per bedroom per house (110 gallons per day), the size of a leaching area as a function of the percolation rate (e.g., for a percolation rate of 2 min. per inch or less use 0.4 sq. ft. of sidewall area per gallon of wastewater and 1.0 sq. ft. of bottom area per gallon of wastewater), minimum distances to property lines from septic tanks and leaching areas, etc.

Major changes incorporated in the "Supplements" were as follows:

1. The minumum allowance for design daily flows was increased from 110 gallons per day per bedroom to 400 gallons per day.
2. The minimum design percolation rate allowable in Plymouth was increased from 30 min per inch to 20 min per inch.
3. Most of the minimum required distances between the on-lot systems and private wells, property lines, etc. were increased.

It is the general policy of the Board of Health to eliminate sub-standard on-lot systems by attrition whenever possible. Therefore, in general, if there is reconstruction of more than 50 percent of the physical value of the home, the on-lot system must comply with both Title 5 and the Town's "Supplements" as nearly as possible. The primary concern of the Board of Health is to insure that any future on-lot systems or future repairs be made in accordance with requirements that are in the best interest of the public safety and health.

Density of Housing. Acceptable present day on-lot disposal facilities usually consist of a septic tank and leaching pit or field. Under favorable conditions an on-lot system can provide satisfactory service for 15 or more years, after which it is usually necessary to relocate the field to another area on the lot. Obviously, this is feasible only where ample space is available. Consequently, consideration must be given to providing adequate space within the lot for one or more leaching field replacements when setting minimum lot size limitations for on-lot systems in residentially-zoned areas. This criterion is also useful as a planning aid in determining public sewerage needs for areas already developed.

Minimum lot sizes based on the Plymouth Board of Health Rules and Regulations, assuming a one-family detached dwelling with three bedrooms and a garbage grinder, are shown in Table 4-2. Those lot sizes include an adequate area for a minimum of one leaching area replacement. Also, two different types of systems are considered: the septic tank and leaching pit, and the septic tank and leaching trench (these are the most common types of systems presently being installed in Plymouth).

TABLE 4-2 MINIMUM LOT SIZES
RECOMMENDED FOR ON-LOT DISPOSAL SYSTEMS(1)

Leaching(2) pit	Leaching(3) trench
For lots with public water supply	
8,000 $ft.^2$	13,000 $ft.^2$
For lots with on-lot well water	
22,000 $ft.^2$	29,000 $ft.^2$

1. Assumptions:
   - (1) Percolation rate = 3 min./inch or faster
   - (2) 400 gallons per day minimum for an individual lot.
   - (3) One reserve area for future use is included.
   - (4) A 10 percent factor of safety is included in all areas to account for unusable land.
   - (5) With on-lot wells, a minimum of 145 ft. has been added to all lot dimensions for a backyard location of the well.
   - (6) All leaching areas have been increased by 25 percent for use of a garbage grinder.
2. Depth to seasonal high groundwater from ground surface is 13 ft.
3. Depth to seasonal high groundwater from ground surface is 6 ft.

Water Quality. When considering the effectiveness of an on-lot system, the quality of the underlying groundwater and nearby surface waters is an important factor. In Chapter 2 the various types of classifications and standards for water quality were discussed.

Water quality studies have been undertaken in the past for the various ponds in Plymouth. For example, Lyons-Skwarto Associates undertook a water quality survey of 45 ponds in 1979-1980. Geoscience published a report in March of 1981 entitled, "Groundwater Resources and Lakes Management Study, and IEP published a shoreline water quality survey in July, 1980. Also, the Town's wells are monitored to assure that they comply with the State's standards for drinking water quality. The recreational ponds and bathing beaches are also monitored for water quality to determine if there is any presence of contaminants in the water. These numerous sources of information were reviewed to determine whether evidence existed that a group of on-lot systems was performing inadequately.

Along with the present conditions of the waters, the future conditions must also be considered. Hence, any group of on-lot systems that might potentially degrade a body of water must be considered a problem as well as any group of systems currently degrading the waters.

Over the past few years, the public has become increasingly aware of the importance of water supply. Many towns are restricting use and looking for alternative sources of water because of contaminated wells or reservoirs with insufficient capacity. Plymouth does not have this problem because it overlies one the the largest groundwater aquifers in New England. Other towns and agencies have identified the need for water and are currently investigating the possibility of tapping into Plymouth's water resources. Therefore, this could become a very important and valuable asset for Plymouth in the future.

The Town, recognizing the need to protect this valuable resource, designated an aquifer protection district as part of its zoning by-law. This section of the by-law was passed in April 1981, and has been in force ever since. The protected areas are shown in Figure 4-2.

One of the provisions of the by-law states that no new unsewered development having a density greater than one dwelling unit per 40,000 sq. ft. shall be allowed within the district. Unless sewers are provided, wastewater quantities discharged within the district are limited to 330 gallons per day per 40,000 square feet and are required to meet certain minimum standards for quality. Among the other provisions of the by-law applying to the aquifer protection district is a requirement that if more than 25 percent of any lot is rendered impervious, all runoff from the developed surfaces shall be captured and treated to remove potential contaminants such as oil and grease. In addition, certain uses or activities are prohibited including the disposal of solid wastes or <u>any</u> waste other than normal domestic wastewater, the storage or transmission of petroleum products, and the open storage or use of road salt or deicing chemicals.

<u>Board of Health Records</u>. Prior to the issuing of a building permit in a unsewered area, a permit for construction of an on-lot disposal system must be issued by the Board of Health. Likewise, if an on-lot system fails, the homeowner must apply for a permit before reconstruction of the system can begin. These permit records are kept on file in the Health Department at Town Hall and date back to 1962. These records can be useful because they reveal the structure of the lot's soil along with its

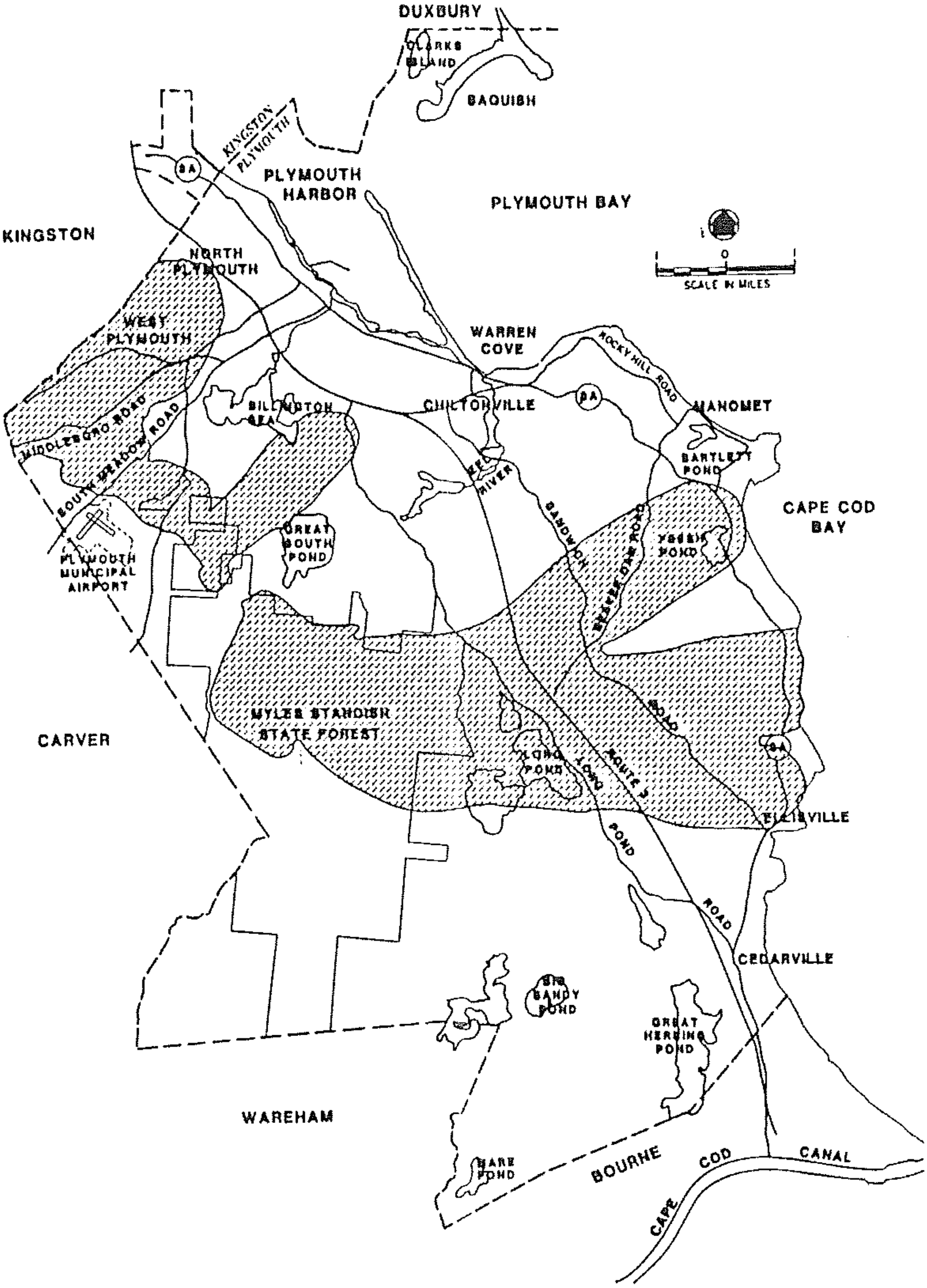

FIG. 4-2 TOWN OF PLYMOUTH AQUIFER PROTECTION DISTRICT

drainage characteristics and depth to groundwater. All of this is required information for the permit.

Another source of records from the Board of Health is the complaint file. Any problems believed to be causing a violation are reported to the health officer and must be answered. Complaints found in this file consist of odors or overflowing cesspools, septic tanks, or leaching areas. These problems are usually reported by neighbors or other affected parties.

During the course of this study, the entire complaint file and most of the permit records were reviewed to determine where problems had occurred in the past and whether there was any correlation between the problems encountered and items such as age of the system, water table, soil type, density of housing construction, etc. In addition the Director of Health was consulted to obtain his first hand knowledge of on-lot system problems around Plymouth.

Questionnaire Survey. The largest and most extensive source of information concerning the on-lot systems in Plymouth was the questionnaire survey which the Town undertook during January of 1982, entitled "Survey of Existing On-site Wastewater (Sewage) Disposal Systems." Different questionnaires were sent to residences, restaurants and industries. Copies of the questionnaires can be found in Appendix G.

The response to the questionnaire survey was excellent. There were over 4,000 returns from an estimated 8,000 unsewered residences to which questionnaires were mailed, yielding a return rate of approximately 50 percent.

The purpose of the questionnaire was to determine the types of problems that exist, how severe these problems were considered to be by the homeowner, and the locations of these problems. Information requested included: the age of the system, the owner's evaluation of the system's performance, whether the owner ever limits his water use, whether there are any odor problems, and the number of repairs made to the system.

Considerable judgment must be used in the interpretation of any questionnaire. For example, it is likely that some of the questions were misinterpreted by some homeowners. In addition, a homeowner may not admit the existence of a problem if he fears that his property may be condemned if he reveals the problem. The information provided by the questionnaire must be interpreted and summarized with this in mind.

The results of those residential questionnaires returned were grouped into eleven geographical study subareas as shown in Fig. 4-3. The survey results are summarized in Table 4-3.

Study Subareas

The following sections discuss the results of the evaluations of on-lot disposal problems for each of the study subareas of Plymouth.

Billington Sea. This subarea is located in the northwest portion of Plymouth, just south of the business section of town. Most of the homes surrounding the Sea are approximately 30

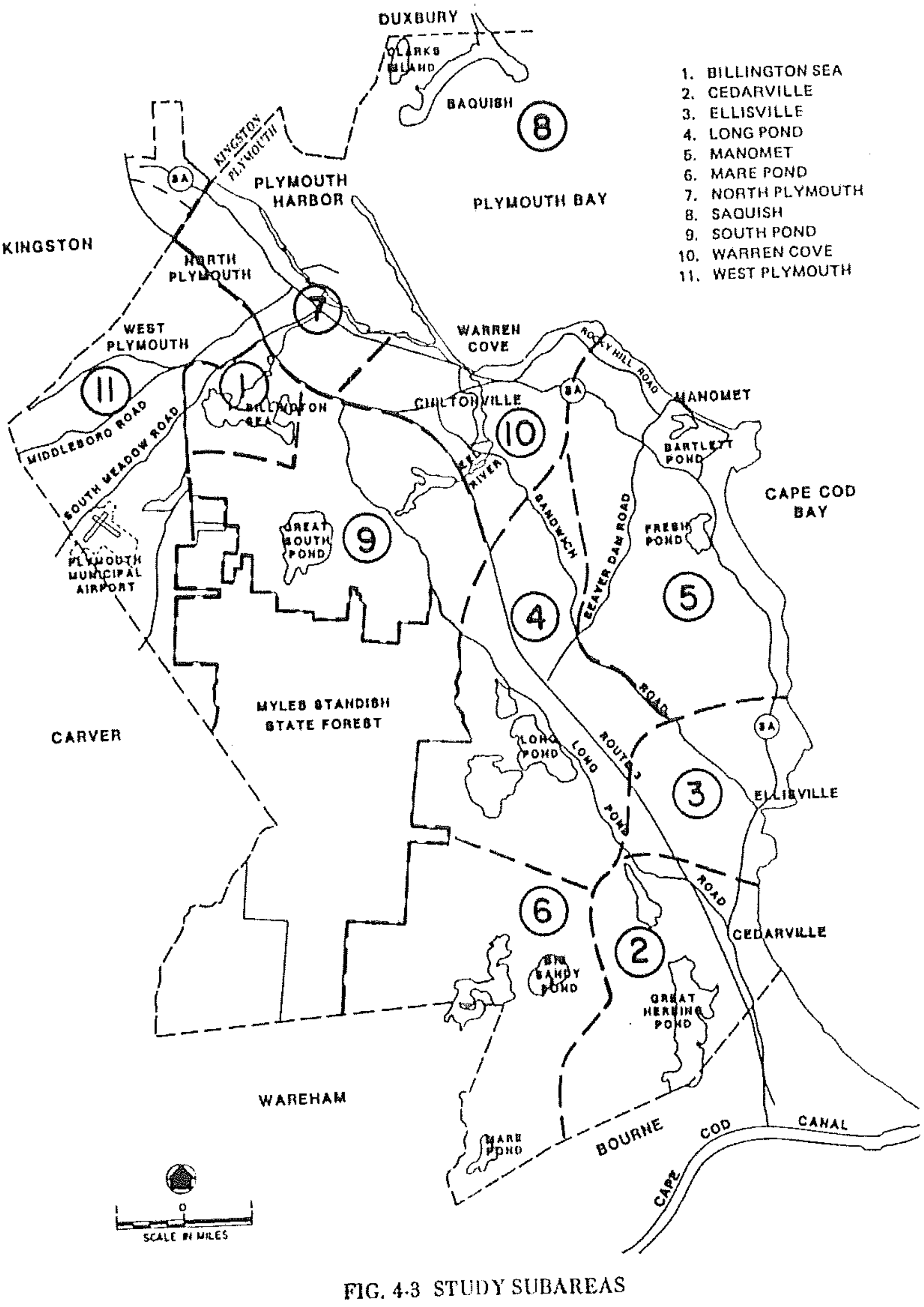

FIG. 4-3 STUDY SUBAREAS

TABLE 4-3. SUMMARY OF PLYMOUTH PROPERTY OWNERS' RESPONSES TO QUESTIONNAIRE

Subarea	No. of replies	% of all replies	Owner's evaluation of system			% Fair and poor	Type of problem		No. of systems repaired	% of replies reporting repairs	Homes requesting sewers	% of replies requesting sewers
			Good	Fair	Poor		Limited water use	Sewage odor				
Billington Sea	79	2.1	69	8	-	10	8	-	9	11	6	7
Cedarville	447	12.1	377	53	5	13	63	13	33	7	90	20
Ellisville	70	1.9	61	8	1	13	13	1	6	8	10	14
Long Pond	263	7.1	236	20	4	9	32	12	22	8	35	13
Manomet	1,291	35.1	1,033	186	42	18	370	81	252	20	302	23
Mare Pond	191	5.2	163	21	3	13	31	6	11	6	33	17
No. Plymouth	56	1.9	45	6	-	11	9	1	3	5	13	23
Saquish	26	.7	21	2	1	12	4	-	1	3	5	19
South Pond	128	3.4	95	9	-	7	9	2	15	12	12	9
Warren Cove	234	6.3	188	31	14	19	62	14	52	22	56	24
West Plymouth	889	24.2	778	89	16	12	136	29	46	5	151	17
TOTALS	3,674	100 %	3,066	433	86	14	737	159	450	12	713	20

years old. There are a few seasonal residences on the periphery of the sea, but overall this is a year-round community.

Many of the house lots on the western side of Billington Sea are 300-400 ft. long and approximately 70-100 ft. wide. Because the soils found in this area are well drained, these lots are large enough to sustain an on-lot system.

The results of the questionnaires did not indicate any serious problems with the on-lot systems. There were only nine repairs reported by the 79 respondents from this area. Most of these were associated with homes over 30 years old, indicating no serious problems (since the expected life of an on-lot system is around 15 to 20 years).

Billington Sea has been classified as eutrophic, that is, enriched with nutrients to the point that the growth of algae has made the water less desirable for such uses as drinking water, swimming, boating, and fishing. During the course of the study some concern has been expressed by some citizens that on-lot disposal systems in the Billington Sea watershed may be the cause of the eutrophication of that waterbody. A review of previously conducted water quality studies and additional sampling conducted as part of this study by the EIS contractor, C.E. Maguire, Inc. and its subcontractor, New England Research, Inc. (NER), have led to the conclusion that on-lot systems have not had any serious impact on Billington Sea. In its report, "Water Quality Studies For Plymouth, Massachusetts, Part III Final Report", NER states that, "Billington Sea has been eutrophic for at least the past decade," and "relatively high concentrations of nutrients enter into Billington Sea from some surface and groundwater sources;" but "the preponderance of chemical, physical and bacteriological data appear to indicate that the relatively high nutrient contaminants cannot be directly correlated with domestic effluent". NER concluded that "it is unlikely that any localized effort to alter current domestic waste disposal practices along the shoreline of Billington Sea will have any measurable effect on the lake's trophic status," that "there is no bacteriological evidence that Billington Sea or Little Pond receives effluent from surrounding septic systems," and that "reported eye and ear irritations among swimmers in Little Pond may be due to opportunistic pathogens introduced into Little Pond by swimmers."

Because no serious problems could be attributed to on-lot wastewater systems in this subarea, it is not considered to be a problem area.

Cedarville. This subarea is located in the southeastern corner of Plymouth with Cape Cod Bay abutting it on the east and the Town of Bourne to the south. Residences to the east of Route 3A average 15 yrs in age while those homes to the west are almost double that in average age. The lots sizes vary substantially in this area, ranging From 3000 square feet in a seasonally-occupied development bordering the northwestern portion of Great Herring Pond to 1 acre or more in much of the remaining area.

Three of the ponds in this study area are classified as eutrophic. They are: Hedges, Island, and Herring Ponds. In their review of previous water quality studies, C.E. Maguire concluded that there is no apparent correlation between the number of homes surrounding these ponds and the degree of eutrophication.

During 1979, residents surrounding the Cedarville landfill began complaining of the poor quality of their well water. This condition is believed to be a result of leachate from the closed landfill entering the groundwater system and not from malfunctioning on-lot disposal systems. In response to this problem, the Town has extended the Town water distribution system to those homes requiring immediate attention.

Relatively few problems were reported in the questionnaire for an area this size. Only seven percent of the residents reported past problems, and almost all of those which had required repairs had systems older than 20 years. Hence, Cedarville is not considered a problem area.

Ellisville. Ellisville abuts Cedarville on its southern border and Cape Cod Bay on the east. This subarea is sparsely developed, consisting of homes with large lots and areas with large vacant lots.

Six homes reported repairs to their systems, and the Board of Health records revealed only a few problems. Because these homes are over 20 years old, the Ellisville area is not considered a problem area.

Long Pond. The Long Pond subarea, located in central Plymouth, is basically a rural area with some residential development bordering Long Pond.

The Lyons-Skwarto report reported two ponds in this area, Halfway Pond and Little Long Pond, as being ultra-eutrophic. Since there is very little development surrounding either of these ponds, there is apparently no correlation between on-lot system failures and the eutrophic states of the ponds.

There were very few repairs reported in this study area. The few residents that did report repairs were from relatively new homes with lot sizes of approximately one acre, more than enough for what is needed to properly dispose of the wastes for a single family house in this area. The failed systems were determined to be under-designed relative to the new code established by the Board of Health. In most cases the leaching area was expanded and no subsequent problems have been reported.

No serious wastewater-related problems are known to exist in this area.

Manomet. Located on the northeastern corner of Plymouth and extending five miles south from Rocky Point, this is the second most populated of the unsewered study subareas. Most of the development exists along the coast, with the bulk of the remainder occurring along Route 3A.

Manomet was the source of over 35 percent of the returned questionnaires and was the subarea having the highest number of respondents requesting sewers. Over half the reported problems and over half the reported system repairs were in Manomet.

The Manomet subarea has been divided into six sections for study purposes, as shown in Fig. 4-4. Section 1, the northern-most section of Manomet, has been further subdivided into five subsections that are designated "1A" through "1E", as shown in Fig. 4-5.

Section 1 is the most densely developed area of Manomet and contains the subarea's two prime recreational beaches, White Horse Beach and Priscilla Beach. Approximately 60 percent of the residents of this section occupy their homes year round. The remaining 40 percent are seasonal residents, with most of the seasonal people living on the bluffs of White Horse Beach in Subsection 1A.

The on-lot wastewater disposal systems presently used by many of the residents in Subsection 1A, also known as the East White Horse Beach area, are standard block cesspools or are cesspools constructed from 55-gallon metal drums. These systems are inadequate for the proper treatment of wastewater and can be very dangerous when subjected to any type of weight (especially at the beginning of the summer season when they are empty and there is no hydrostatic pressure available to support the cylinder wall) and are not in conformance with present state and local codes which ban the construction of all cesspools. There are approximately 170 cottages plus 15 year-round residences on

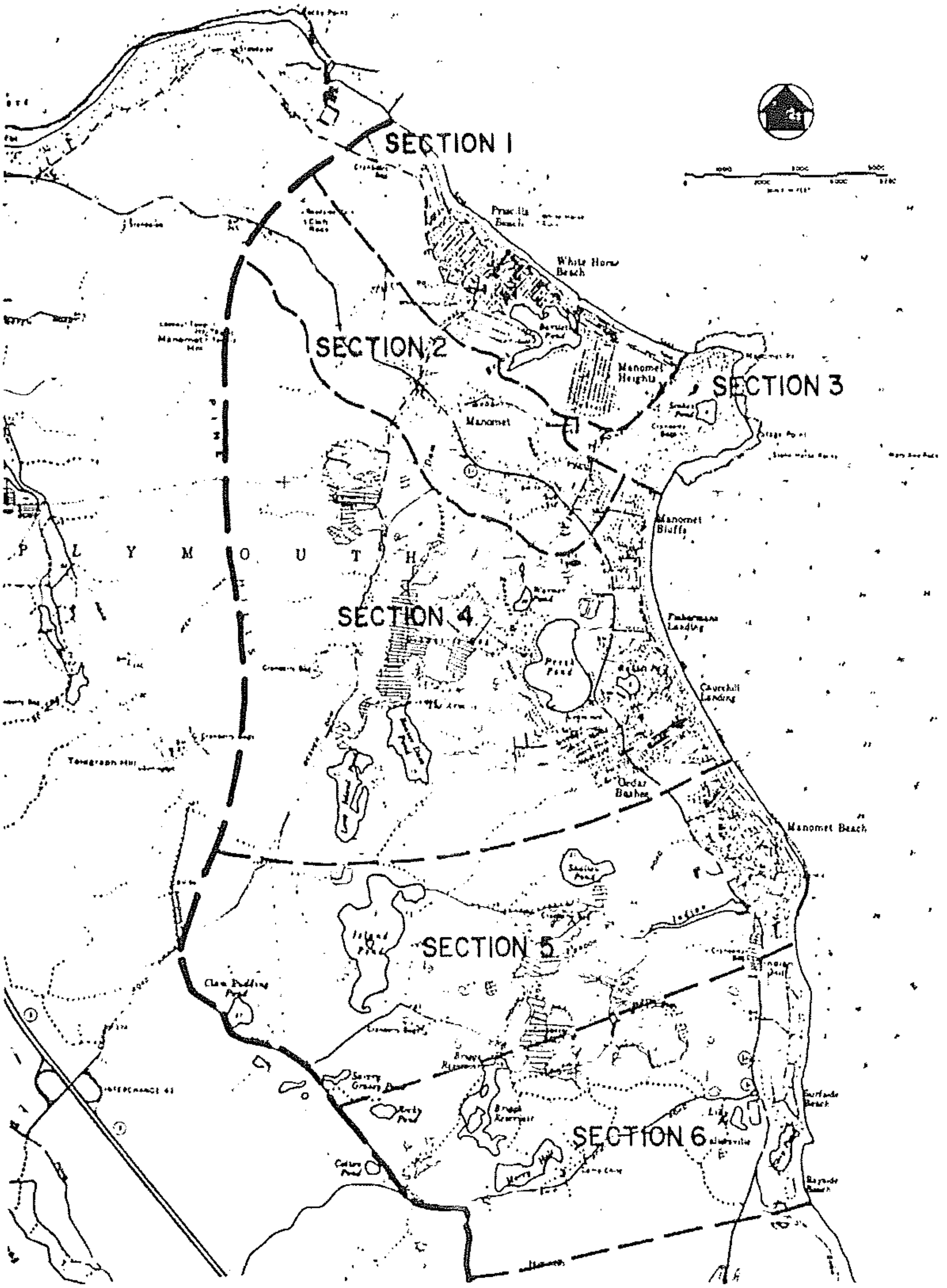

FIG. 4-4 MANOMET STUDY SUBAREA

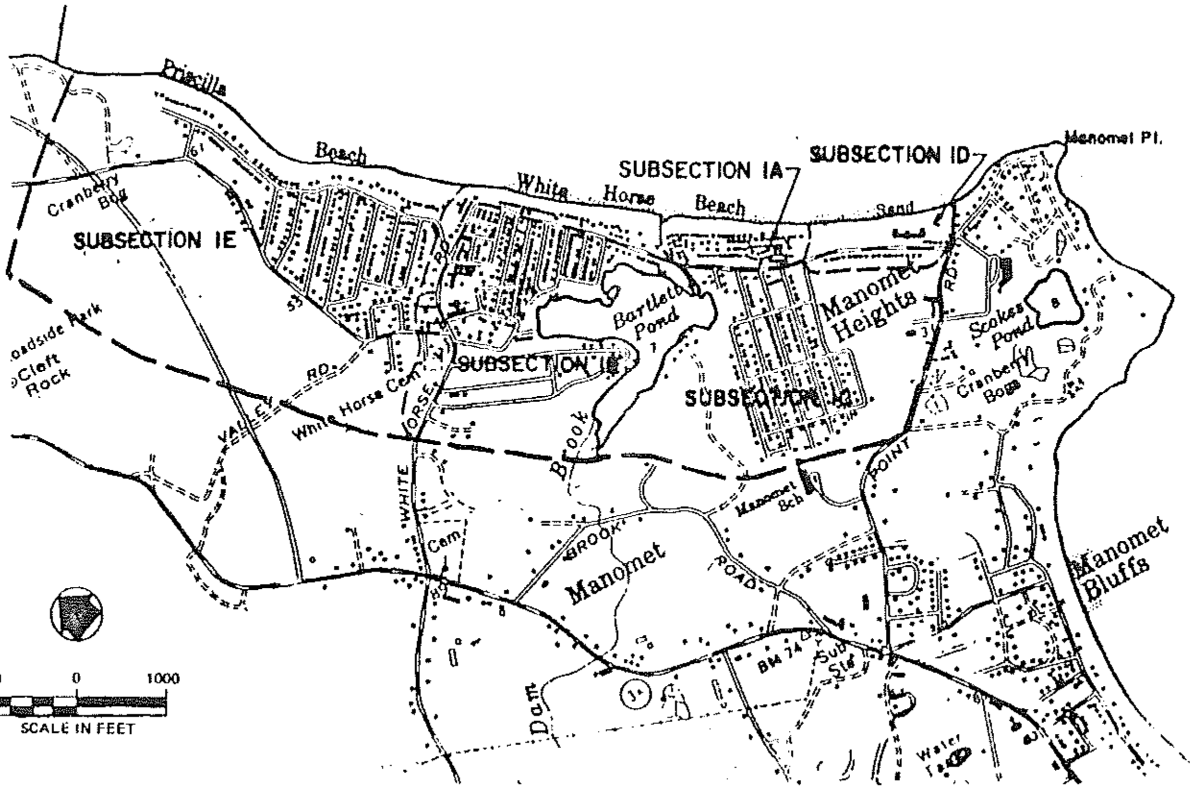

FIG. 4-5 SECTION 1 - MANOMET

about 13 acres of land in Subsection 1A. This yields an average lot size of 2,600 sq. ft., with some lots as small as 1,300 sq. ft. Lots of this size do not have the land area required to construct an on-lot wastewater disposal system in conformance with the State Environmental Code (Title 5) or the supplements to the code as mandated by the Plymouth Board of Health (See Table 4-2). Most of these homes are presently violating one or more of the minimum distance requirements for on-lot systems (i.e., distances between the leaching area and property line, house, water lines, etc.). In addition, as shown on Fig. 4-6, there are eleven homes located on the eastern side of the Bartlett Pond outlet that do not have the minimum required set back distance of 75 ft. between their systems and the waterway.

Because of the small lot sizes, the only approvable type of on-lot system that most of the cottage owners could physically build on their lots would be a septic tank and leaching pit However, for this type of system, the minimum required depth to high groundwater is 13 feet. As the highest ground surface elevation in Subsection 1A has a maximum depth to groundwater of approximately 10 ft*, the pit-type system is not applicable in this area. Hence, most lots in this subsection cannot comply with Title 5 or the Board of Health regulations and are likely sources of groundwater contamination. Owners of these lots will not be able to obtain a permit for reconstruction of their on-lot systems when failure occurs.

*This assumes the high groundwater elevation is at Elevation 8.0 ft.

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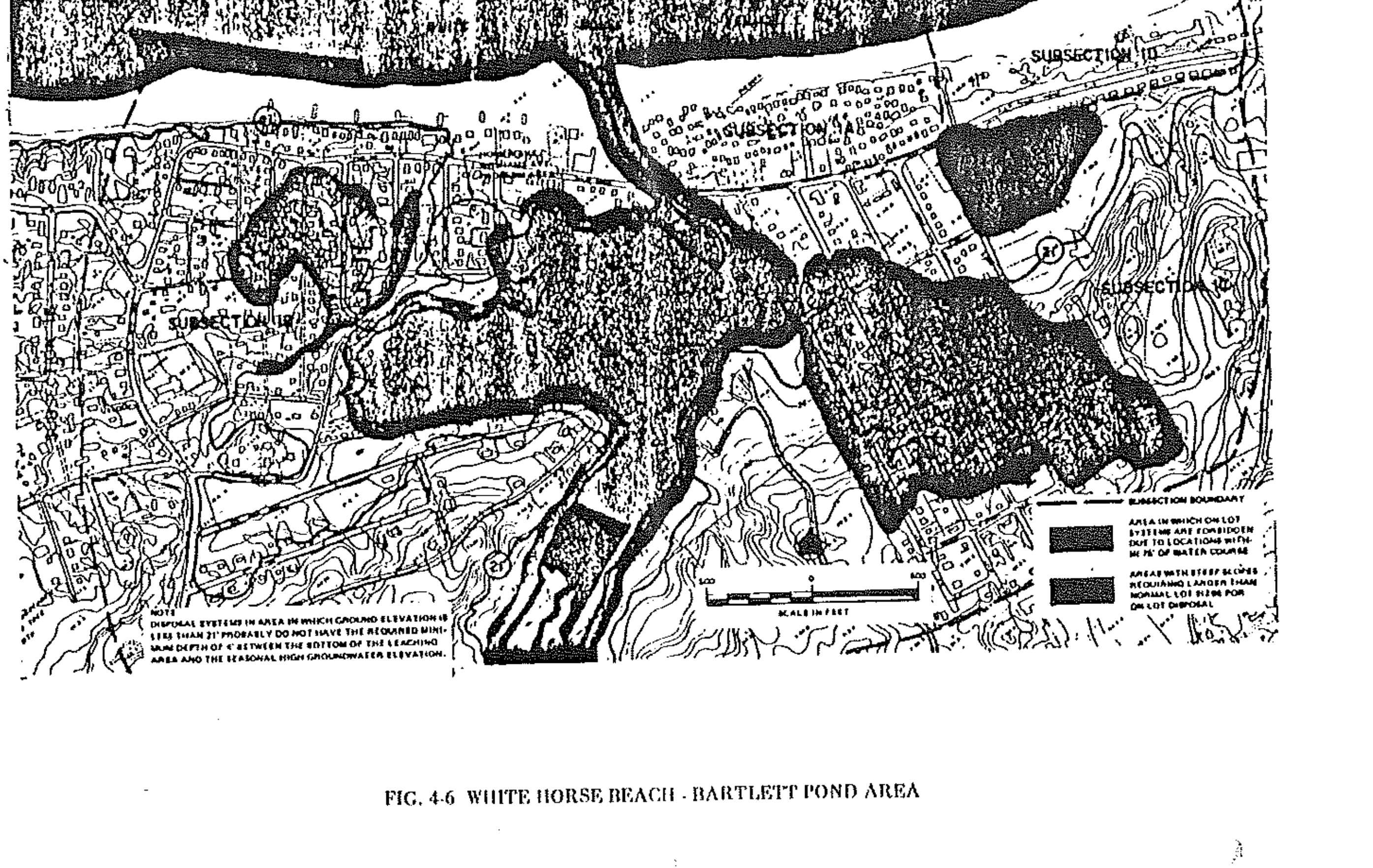

FIG. 4-6 WHITE HORSE BEACH - BARTLETT POND AREA

Because the lots in Subsection 1A are not large enough for any type of on-lot disposal system, this area will always pose a potential health hazard until an alternative means of wastewater disposal is implemented.

Subsection 1B extends westward from the outlet of Bartlett Pond to White Horse Rd. and is bounded by White Horse Beach on the north. Many of the homes in this area, especially those along Homer Ave. and Williams Ave. (highlighted in Fig. 4-6), are lowlying and thus have poor drainage. The homes in this area were observed to have groundwater and or leachate from their on-lot wastewater disposal systems running through their yards and into the street for most of 1982. Because an inadequate depth of unsaturated soil is available to effect the proper amount of treatment of the wastewater, a potential health hazard is present.

The Plymouth Board of Health requires a minimum of 75 ft. from the edge of a body of water to the limits of a leaching area. In Figure 4-6, a set back line 75 ft. from the edge of the water has been drawn around the periphery of Bartlett Pond. Any on-lot system within this set back line is in violation of the regulations. There are about 40 lots in Subsection 1B located within 75 ft. of the pond.

In Table 4-2, the minimum lot sizes for a pit-type system and a trench-type system and their minimum respective depths to seasonally high groundwater are given. Assuming the groundwater is at elevation 8.0 ± ft. (based on the average water surface elevation of Bartlett Pond) a front or backyard elevation of 21 ft., as shown on Figure 4-6, is needed to construct a pit type system with a minimum lot size of 8,000 sq. ft. and a front or backyard elevation of 14.0 ft. is needed to support a trench-type system with a minimum lot size of 13,000 sq. ft. Approximately 40 homes exist with yards less than Elevation 14.0 ft. and have an average lot size of less than 5,000 sq. ft. Therefore, those homes will always be out of conformance with Title 5 and the Board of Health regulations as long as on-lot disposal systems are used. Approximately 25 additional homes are located within the 21 ft. contour interval, with an average lot size of 5,000 sq. ft. These on-lot systems are also out of conformance with Title 5 and the Board of Health regulations.

Sampling conducted by the Massachusetts Department of Environmental Quality Engineering (DEQE) during July and August 1982 found relatively high total and fecal coliform levels at various locations in Bartlett Pond (coliforms are organisms which indicates the presence of pathogenic or disease-causing organisms). One count in excess of 1600 fecal coliforms per 100 ml was recorded, and three out of six samples indicated levels in excess of 200 per 100 ml. The Massachusetts Water Quality Standards require that the average fecal coliform level not exceed 200 per 100 ml and that fewer than 10 percent of the samples exceed 400 per 100 ml for a Class B body of waters such as Bartlett Pond. The DEQE sampling thus indicates that a potential public health problem does exist in Bartlett Pond.

During September 1982, the DEQE collected two water samples from a section of Whitehorse Beach. Both contained total coliform bacteria counts in excess of Class SA standards, and one contained 920 total coliforms per 100 ml, a level which would result in failure to meet Class SB standards and closure of the beach if it occurred in greater than 20 percent of the samples taken.

Additional sampling conducted on August 23, 1983 by C.E. Maguire, Inc. found relatively low fecal coliform levels at three locations in Bartlett Pond, the highest one being 75 per 100 ml. These samples thus did not indicate the presence of any immediate public health threat.

The lot sizes for the remaining portion of Subsection 1B also average 5,000 sq. ft. Although most of these lots may have adequate depth to groundwater, their on-lot systems are in violation of one or more of the minimum set-back requirements except in cases where adjacent vacant lots are owned by the individual homeowners.

In summary, there are many lots on the periphery of Bartlett Pond whose on-lot systems do not have the minimum required depth to groundwater. Contaminants from these systems are likely to be passing into the groundwater and eventually into Bartlett Pond, adding to its already degraded state (the Lyons-Skwarto report classified Bartlett Pond as ultra-eutrophic). The major portions of the remaining area within Subsection 1B cannot meet one or more of the minimum setback requirements. Therefore this area is classified as having severe problems with wastewater disposal and is in need of a wastewater management plan.

Subsection 1C is known as the "Hilltop" section of Manomet. As its name implies, this neighborhood is located on a hill just east of Bartlett Pond and south of White Horse Beach. Although most of the residences in this area are occupied year-round, some seasonal homes are scattered throughout the area.

Approximately thirty percent of the residents of this subsection reported that repairs had been made to their systems. While this is not unusually serious, considering the average age of the homes in this neighborhood is around 25 years, it does not necessarily imply there are no problems. Referring to Fig. 4-6, the average elevation between Taylor Avenue and Spruce St. is approximately 13.0 ft. with the estimated high groundwater elevation at 9.0 ft. This leaves a depth of only 4.0 ft. to groundwater from the ground surface. Since a minimum soil depth of 6.0 ft. to groundwater is required for any type of system, all homes in this area are out of conformance with the code, and most of these systems are probably contaminating the groundwater.

Also shown in Figure 4-6 is the 75 ft. minimum required distance between the edge of water and a wastewater disposal system. Any on-lot system within this strip is not in compliance with the regulations of the Board of Health and may also be contaminating Bartlett Pond.

Most of the Hilltop area contains slopes of 5-12 percent. Given the assumptions listed in Table 4-2, and the further assumption that a trench type system requires an additional 1000-3000 sq. ft. in area if slopes are greater than 5 percent, any lot with a pit-type system within this area and containing less than 8000 sq. ft. or any lot with a trench-type system containing less than 14,000 sq. ft. of area is not in compliance with the present Board of Health regulations.

Most of the repairs that have been made to the systems in Subsection 1C have consisted of system enlargements. It was determined from reviewing health department records that most of the old on-lot systems were undersized, and this was largely due to inadequate design guidelines at time of construction. The present design requirements restrict the construction of on-lot systems on lots as small as those found in this neighborhood. As system failures occur, many of the homeowners are finding it impossible to reconstruct systems on their lots without being granted one or more variances. Subsection 1C thus has many existing wastewater disposal problems and is considered a problem area.

Subsection 1D extends from Manomet Point Rd. to Hilltop Ave. as shown in Figure 4-5. It consists of approximatley 22 homes and a hotel.

There are eight homes located adjacent to an unnamed marshy pond south of Taylor Ave. that are all within 75 ft. of the shoreline, making them all in noncompliance with the regulations of the Board of Health. Also the average depth to groundwater along Taylor Ave. is approximately 3 ft. to 4 ft., indicating that most or all of the on-lot systems are in the groundwater. Therefore, there is very little treatment of the wastewater by the soil before it passes into the groundwater. A sample collected by C.E. Maguire, Inc. from a small pond of standing water in the unnamed marshy pond on August 23, 1983 contained 1100 fecal coliform bacteria per 100 ml., a value which exceeds the Class B standard and may be indicative of contamination by either on-lot systems or waterfowl. Because of the likelihood of on-lot contamination of groundwater, Subsection 1D is also considered a problem area.

Subsection 1E encompasses the remainder of Section 1 extending westward from White Horse Rd. to the Pilgrim Nuclear Power Station at Rocky Point. The lot sizes in this area vary from 5,000 to 7,000 sq. ft. The soils in this area are similar to those found in most of Section 1 (i.e., a coarse to medium sand) with the exception of a small pocket of a less permeable soil existing between Emerson and Cochituate Roads. The depth to groundwater in this neighborhood is sufficient for any type of on-lot system.

The average age of the homes in this subsection is approximately 25 years. Therefore, the residents here have the same types of problems concerning renovation of their systems as the residents of the Hilltop area. Although the lot sizes are generally slightly larger, they are nevertheless too small to satisfy the codes. Those residents between Emerson Rd. and Cochituate Rd. will have the greatest difficulty in replacing their systems because of the larger leaching area required due to the more impermeable soil layer existing between these streets.

The sampling conducted by the Massachusetts DEQE during July and August of 1982 found relatively high total coliform bacteria levels in a section of Priscilla Beach. The total coliform counts of four samples taken during this period ranged from 79 to over 1,600 MPN per 100 ml. The Class SA standard for this location, as published in the "Massachusetts Water Quality Standards", states total coliform bacteria shall not exceed a median value of 70 MPN per 100 ml and not more than 10 percent of the samples shall exceed 230 MPN per 100 ml in any monthly sampling period. The levels measured are not conclusive evidence that some of the systems are not properly treating the wastes, but do indicate that this may be the case. Only an extensive survey over a long period of time would be conclusive.

In summary, Section 1 of Manomet from Rocky Point to Manomet Point is densely populated. The on-lot wastewater disposal systems in general for the entire section are undersized, and replacement, where needed, will be permitted only in portions of the section where variances to the state and local codes may be granted by state and local health officials. The entire section is considered to be a problem area in need of a wastewater management program. Because of potential health problems associated with on-lot systems located in areas with high groundwater and areas in close proximity to surface waters, variances are unlikely to be granted to lot owners in Subsections 1A and 1D and portions of Subsections 1B and 1C. These subsections are thus considered to have serious wastewater problems.

Section 2 of Manomet extends from White Horse Rd. along Route 3A to the intersection of Manomet Point Rd. This area is mostly a commercial area consisting of hardware and grocery stores, a couple of small shopping plazas and a larger, recently built shopping plaza, gas stations, and other businesses. Lot sizes vary from approximately 5,000 sq. ft. up to one or more acres.

Very few existing problems were reported in this area, and only a few permits have been issued for repairs to these systems. This is probably a result of the light use that an on-lot system receives from commercial establishments such as the ones found in the area. Also, most of the lots are large enough to replace an on-lot system one or more times if needed. Therefore, this section does not appear to have any serious on-lot system problems.

Section 3 is known as Manomet Point. It is located to the east of Manomet Point Rd. and extends outward to the shore line, as shown in Fig. 4-4. Most of the lots to the southwest of Scokes Pond are an acre or more in area, while the lots to the north of Scokes Pond are very small averaging 5,000 sq. ft. in area. However, the latter lots were not developed to maximum density, and the actual area available per house averages approximately 10,000 sq. ft. Vacant lots in this area may be available for further replacement of failed leaching areas.

Twenty-two percent of the residents of this section who returned questionnaires reported repairs to their systems. This is not a high percentage of failures in light of the 43-year average age of the homes. Therefore this is not a problem area.

Section 4 is the area extending from Manomet Bluffs on the north and centered along Route 3A to the south, terminating in an area known as Cedar Bushes. Lot sizes range from 5,000 sq. ft. in the older neighborhoods to 20,000 sq. ft. in the more recently developed areas. One very large recreational body of water, Fresh Pond, and the Wanno's Pond well are located in this section.

Fewer than twenty percent of the respondents from this section reported problems with their on-lot systems, and these incidents were distributed relatively uniformly throughout this section. Because there is no evidence of a public health threat in this area, and because it is likely that variances would be granted to residents in this area seeking to replace their failed on-lot systems, there doesn't appear to be a serious on-lot disposal problem in this section. However, two potential problems exist: maintaining the water quality of Fresh Pond, and protection of the Wanno's Pond well. Fresh Pond was classified as mesotrophic by Lyons-Skwarto Associates and ranked sixth in quality out of forty-one ponds studied in Plymouth. Fresh Pond is quite large, consisting of over 300 million gallons in a 62-acre pond. Although there are approximately 40 homes on the shore of Fresh Pond, the on-lot systems probably have a negligible input of nutrients when compared to the effects of the 45± acres of cranberry bogs nearby (primarily due to the large amount of fertilizers used in the bogs).

The Wanno's Pond well supplies approximately 10 percent of the total water used by the Town. Associated with the well is a recharge area which is part of the aquifer protection district shown in Fig. 4-2. As mentioned previously, no future unsewered development will be allowed on lots smaller than 40,000 square feet in a recharge area for a well. But, because of existing development on lot sizes of less than 40,000 square feet, any future development must be carefully controlled along with periodic sampling of the well.

Section 5 extends from Cedar Bushes to the Indian Hill area, as shown in Fig. 4-4. Almost all of the development has occurred east of Route 3A, with only a few homes and the Indian Brook School existing to the west of Route 3A. The average age of the homes is approximately 24 years, and the soils are of a coarse to medium-grained sand with a strip of hardpan lying along the bluffs south from Indian Brook to the Surfside Beach area.

Only 14 percent of the residents of this section reported having made repairs to their systems, and 28 percent reported limiting their water use as a result of on-lot disposal problems. These percentages are not high considering the average

age of the homes. Also, the residents reporting limited water use could probably expand their systems to increase the amount of water disposed, remembering that the system design of twenty years ago provided leaching areas that are inadequate by today's standards.

Indian Pond is the only significant body of water in Section 5. The Lyons-Skwarto report ranked this pond 36th in quality out of the 41 ponds studied, classifying it as ultra-eutrophic. However, its shallow depth (average is 4 feet) and the fact that there are over 80 acres of cranberry bogs in its drainage basin are more likely causes of its poor water quality than wastewater from the few homes that are adjacent to it.

In summary, there do not appear to be any wastewater-related problems of a serious nature in Section 5.

Section 6 is the final area in Manomet to be considered. It extends from Indian Hill to Bayside Beach. The average age of the homes is twenty years, and the lot sizes vary from 5000 sq. ft. to one acre or more.

Only 13 percent of the residents of this section reported problems with their on-lot systems, and this is not high in light of the average age of the homes being twenty years.

Two of the ponds in this area, Morey Hole and Ship Pond, were studied in the Lyons-Skwarto report, and both were classified as eutrophic. However, no conclusions were drawn as to the cause of their degraded state.

The Ship Pond well is located off of Ship Pond Rd. just west of Route 3A. This well supplies approximately 6 percent of the Town's total water supply. There is some development located within the cone of depression for this well. However, this development has yet to prove detrimental to the water in the well. As with the other supply wells in Plymouth, this well should be regularly monitored as development within the recharge area increases.

This section of Manomet does not have any current on-lot disposal problems that are serious enough to designate it as a problem area.

<u>Mare Pond Area</u> Located in the southernmost section of Plymouth, the Mare Pond subarea abuts Bourne on the south and Wareham on the west. The neighborhood to the south and west of Mare Pond, called Buttermilk Shores, is basically a year-round community with lot sizes averaging 7000 - 8000 sq. ft. in area, while the remainder of this area consists of development surrounding Ezekiel, Wall and Big Sandy Ponds with lot sizes ranging from 5,000 - 20,000 sq. ft. and averaging around 8,000 sq. ft. in size. Buttermilk Shores is serviced by the Buzzard's Bay Water Co. The homes surrounding the remaining ponds have private wells.

Four ponds in this area were investigated in the Lyons-Skwarto report. Ezekiel, Little Sandy, and Wall Pond were classified as eutrophic, while Big Sandy was classified as mesotrophic. All but Little Sandy have substantial development around them. However, it was not determined whether or not the on-lot systems are contributing excessive nutrients to the ponds or if the morphology of the pond lends itself to eutrophication.

Very few wastewater-related problems were reported in this area. Most of the homes that did report problems were over 20 years of age, or in the upper part of the range of expected life for an on-lot system. Thus, this area is not considered to have serious wastewater problems.

Saquish. Although Saquish is politically a part of Plymouth, the area is segregated from "mainland" Plymouth by the harbor. This community is geographically more a part of the Town of Duxbury, being located on the southeastern tip of Duxbury. This area is a summer community consisting of cottages on lots that average 5,000 sq. ft. in area. The average age of the cottages is 30 years.

The number of reported problems (four) was very low. This is attributed to the fact that the homes are principally used on weekends, allowing the on-lot disposal systems to rest during the week. Four residents also reported their wells as having poor water quality. However, most of the lots in Saquish are very small, and it is doubtful that any of the homes have the minimum required distance between their wells and leaching fields.

The biggest problem for this area is its lack of water services. A community water service would be of a great benefit to these cottages. However, that topic is beyond the scope of this study.

Because this area is a summer community and has a very low number of reported problems, this area is not considered to have a serious wastewater problem.

South Pond. This subarea is located in central Plymouth, abutting Route 3 on the north, and extending to the southern border of Plymouth, encompassing Myles Standish State Park. This section of Plymouth is very sparsely populated.

No major problems were reported, and those homes that did report past problems averaged 35 yrs in age. The two ponds classified by Lyons-Skwarto as eutrophic, Hoyts and Russell Mill, each have only a few homes surrounding them. Hence, their eutrophic state is probably not attributable to the on-lot systems in the area. The South Pond subarea is not considered to be a problem area.

Warren Cove. This area , located in the north central part of Plymouth, is one of the more established areas of Plymouth, with the average age of the homes being around 43 years. The lot sizes vary substantially, with some of the older homes occupying more than one acre and an average for the entire area of approximately 15,000 sq. ft.

The soils in this area differ slightly from those in the majority of Plymouth. The area to the northeast along Rocky Hill Rd. consists of stoney, coarse, sandy loam with slopes from 3 to 25 percent. In the lower lying areas, just east of Chiltonville and surrounding the Plymouth Country Club, there is a wide variety of soil types. The majority of these soils are some variation of a sandy loam with some silty loams with slopes ranging from 0-15 percent.

Although 26 percent of the questionnaire respondents reported past problems and 19 percent evaluated their systems as being fair or poor, this is not considered to be serious, because the average age of the homes is 43 years. It could be expected that every home would experience a problem at least once in 20 years. Because the lots in this subarea are large, there is generally sufficient space available to replace a failed leaching area.

Bert's Restaurant, located on the waterfront in the lowest part of the area, does have recurring disposal problems. The restaurant's on-lot problems appear to be due to a lack of leaching area. This lot is small, and it is doubtful if the proper type of treatment is available for the amount of wastes that are discharged into the system.

A relatively high coliform bacteria count (1600 fecal coliform per 100 ml) was measured in a sample of water taken from the Eel River by the DEQE in June, 1980. While no correlation has been made between the high count and the homes bordering the river, it is possible that these bacteria are wastewater related, and further study (beyond the scope of this study) will be necessary to trace the bacteria to their source.

Overall, there are not any widespread wastewater-related problems of a serious nature in the Warren Cove subarea.

West Plymouth. This subarea is located in the westernmost part of Plymouth abutting North Plymouth along Route 3. The Town of Kingston borders it on the north and Carver on the southwest. This is a relatively new area in Plymouth, with most of the development occurring within the last 15 years. The majority of the new homes are located between Plympton Rd. (Route 80) and South Meadow Rd. This subarea includes commercially-zoned land on both sides of Route 44 adjacent to Route 3. There also exists approximately 230 acres of undeveloped residentially-zoned land along the north side of Routes 44 and 80 extending to the town border, a triangular shaped piece of land consisting of 75 acres that is commercially zoned and has yet to be developed, and approximately 400 acres of industrially-zoned land that has yet to be utilized. Thus, this subarea has a large growth potential of which only a small portion has been tapped.

There were relatively few problems reported from this subarea in the questionnaires. With approximately 40 percent of the homes responding, only 5 percent (50 homes) reported repairs to their systems. Because most of these homes were built prior to the adoption of more stringent design regulations by the Board of Health, most homes requiring repairs probably have had underdesigned systems. In most areas of West Plymouth, the lots are larger than 20,000 square feet. Hence, sufficient area generally exists to expand those leaching areas which were originally undersized.

Most of West Plymouth lies within the Town of Plymouth Aquifer Protection District (refer to Fig. 4-2). Much of this area overlies the recharge area of the North Plymouth Well, as shown in Figure 4-7. This is an important well for Plymouth because it currently provides approximately 35% of the public water supply for the Town of Plymouth. Samples of water collected at the North Plymouth well have not yet shown elevated concentrations of nitrates or bacteria, parameters which would be indicative of contamination by on-lot systems. Wastewater from the industrial park in the vicinity of the North Plymouth well is discharged to sewers and conveyed out of the recharge area to the wastewater treatment plant. The Town's Aquifer Protection Bylaw (included in Appendix K) presently prevents residential development on lots smaller than 40,000 sq. ft. within this recharge area and also prevents the on-lot disposal of industrial and commercial wastewater "containing contaminants other than normal domestic waste."

West Plymouth is an area for future consideration. Although there is no present sign of well-water contamination by the housing or the industrial and commercial development in the recharge zone, only very limited and controlled future growth can take place to prevent the possibility of well contamination unless sewers are extended into the area. Such limited and controlled growth is <u>not</u> consistent with the Town's economic development plan which was accepted at the Annual Town Meeting for 1980 and called the "Village Centers Plan".

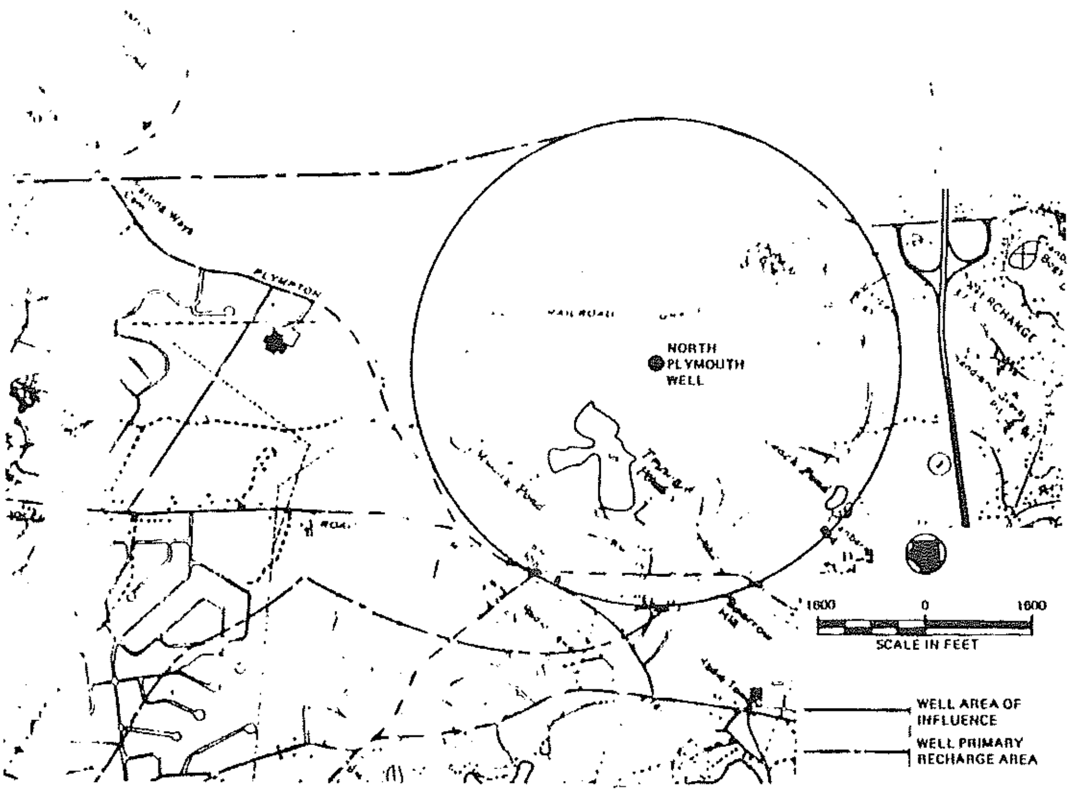

FIG. 4-7 RECHARGE AREA OF NORTH PLYMOUTH WELL

Summary

Several serious problems exist in the existing sewered area of Plymouth. The wastewater treatment plant is hydraulically overloaded, largely due to the infiltration of groundwater into the Town's sewer system. As a result, the plant is in violation of both its permit value for flow and the Massachusetts Ocean Sanctuaries Act. Because of the overload, episodes of bypassing of inadequately treated wastewater occur several times per year. The WWTP is not equipped to dewater sludge, resulting in the storage of sludge solids in the treatment units at the plant. During periods when flow through the plant is high, these solids are often flushed into the plant effluent, causing violation of the plant's permitted value for suspended solids. Other deficiencies also exist in the collection system and at the Night Soil Disposal Facility, as discussed in the text.

Only one of the Town's ten unsewered subareas is currently considered to have problems serious enough to require immediate attention. This is the Manomet area, and more specifically, the area surrounding Bartlett Pond. The problems are dense housing and high groundwater, both of which are likely affecting the quality of the water in Bartlett Pond and along Priscilla and White Horse Beaches. The West Plymouth subarea is a potential problem area if development proceeds without sewers.

None of the remaining subareas have the problems nor the potential for serious problems that Manomet and West Plymouth have. Most of these subareas are sparsely populated or consist of lots that are large enough to maintain properly functioning on-lot systems.

CHAPTER 5

WASTEWATER MANAGEMENT METHODS

General

Since wastewater disposal problems have, in the past, existed throughout developed areas in Plymouth, it can reasonably be expected that problems will occur in areas of future development. This wastewater management plan includes an evaluation of the existing needs of the Town as well as the projected needs in both existing and future developments. Construction of sewers in areas experiencing on-lot system problems may be the most feasible alternative in some cases, but in other areas, particularly sparsely-developed ones, alternatives to sewers must be given serious consideration.

Priorities for providing solutions can be established on the basis of need, economic feasibility and public demand for improvements. Where density of development is high, soils are unsuitable, additional land is limited because of small lot sizes, and severe problems exist, installation of sewers is the most desirable long-term solution to eliminate detrimental effects of wastewater overflows to lawns, contamination of surface waters and groundwaters, and the objectionable aesthetic impacts to sight and smell. Other areas of lesser development which do not experience frequent or severe problems can be provided with interim measures to deal with relatively isolated problems. A review of structural and non-structural alternatives has been made to determine their feasibility and applicability, including measures to upgrade or extend the life of on-lot systems. Extended (permanent) use of on-lot disposal systems is probable only in those areas where zoning and land use provide space for continued replacement of leaching areas.

No Action Alternative

A "no-action" course on the part of the Town of Plymouth is a non-structural alternative which would result in continued worsening of existing problems and would forego the opportunity to develop the most economical solution to existing and potential problems. "No-action" is considered unacceptable in terms of maintaining and improving the quality of life in Plymouth with respect to protection of public health, degradation of the natural environment and the aesthetic quality of public and private property and provides no alternatives for homeowners who develop problems in the future.

At the other end of the spectrum, construction of sewers for the entire Town is unwarranted on the basis of existing needs and extreme costs. A plan must be developed which solves, in the most economical (cost-effective) manner, existing problems and precludes or minimizes future problems.

Evaluation Criteria

The cost impact of sewer systems usually generates considerable public and official attention. In recent years, the eligibility and cost-effectiveness requirements for federally-funded wastewater collection systems have been defined by the EPA (similar requirements exist for state funding). The major EPA requirements are summarized below:

1. Construction of new sewers is eligible for federal funding if areas to be served contained substantial human habitation on October 18, 1972.
2. It must be demonstrated that new sewers are necessary and are the most cost-effective solution.
3. Problems created by existing on-lot systems must be documented. A community survey can be made if necessary to collect this information.
4. Unfavorable characteristics for on-lot systems (soils, groundwater, rock, etc.) must be documented.
5. Where population densities are less than 10 persons per acre, alternatives to sewers must be evaluated including:
    a. improved operation and maintenance of on-lot systems
    b. upgrading existing on-lot systems with new septic tanks, leaching areas or mounds (filled fields)
    c. use of holding tanks
    d. neighborhood or cluster systems serving several houses
    e. water conservation and reuse

Evaluations under these criteria must consider the suitability of any alternative measures to achieve with reasonable certainty and reliability the goals of protection of public health and maintenance of water quality. Various alternatives for improving existing wastewater management are presented below.

Central Wastewater Collection

Central collection is a structural alternative, which provides the most positive means of removing wastewater from densely developed areas. Several options available under this alternative are explained below. The types of collection systems available are gravity (conventional), vacuum and pressure sewers.

Gravity Systems. This alternative has been universally employed for collection of wastewater. The system is the simplest conceptually, in that natural topography is used to allow the wastewater to flow by gravity through a network of pipes to a desired point. There is little maintenance with these systems excepting a yearly inspection and occasional cleaning and flushing. The systems can be limited by natural topography, and pumping is required in some gravity systems as an alternative to unreasonably deep sewer construction.

Vacuum Sewers. Vacuum sewers employ a central vacuum source which is maintained at a negative pressure. A gravity-vacuum interface valve separates atmospheric pressure in the home service or toilets from the vacuum in the collection main. When the interface valve opens, wastewater enters the main, followed by a volume of atmospheric air. After a preset interval, the valve closes. The liquid, called a slug, is propelled into the main by the differential pressure of vacuum in the main and the higher atmospheric pressure behind the slug. After a distance, the slug is broken down by frictional and gravitational forces, allowing the higher pressure air behind the slug to slip past the liquid. With no differential pressure across it, the liquid then flows to the lowest local elevation and vacuum is restored to the interface valve for the subsequent operation. When the next upstream interface valve operates, identical actions occur, with that slug breaking down and air rushing across the second slug. That air then impacts the first slug and forces it further down the system. After a number of operations, the first slug arrives at the central vacuum station. When the wastewater accumulates in the collection tank at the central vacuum station, a sewage pump is actuated to pump the wastewater to another system.

Vacuum sewer technology is new, and there are very few of these systems currently in use. One of the major drawbacks of using new techniques is that improvements are constantly being made on the equipment, and some parts may be quickly outdated with replacements being difficult to order.

Vacuum systems are limited in the lift available, and are therefore best suited to flat terrain. These systems may be adversely affected by low initial use-to-design ratios because of inefficient operation and high cost per unit of volume of wastewater transported. Generally, all system malfunctions result in wastewater accumulation at the home.

Pressure Sewers. There are two major types of pressure sewer systems: the septic tank effluent pump (STEP) system and the grinder pump system. The major differences between these alternative systems are in the on-lot equipment and layout. Neither system requires any modification of household plumbing. In both designs, wastewater is collected in the building sewer and conveyed by gravity to the pressurization facility. The on-lot piping arrangement includes at least one check valve and one gate valve to permit isolation of each pressurization system from the main pressure sewer. Both systems have the advantage of relatively low capital cost for pipeline construction, as pressure sewers are smaller and shallower than gravity sewers. Because of their shallow depth, pressure sewers may also be constructed more easily in densely developed areas than gravity sewers.

In the STEP systems, wastewater receives intermediate treatment in a septic tank, and the effluent flows to a holding tank which houses the pressurization device, control sensors, and valves required for the system. Small centrifugal pumps pump the effluent from the tank to the pressurized system. The primary disadvantage with this system is that the septic tank must still be pumped out periodically, just as with a conventional on-lot disposal system.

In the grinder pump system, wastewater from the building sewer flows by gravity to a grinder pump. The pump can be located either inside or outside the building, although the basement location is preferable for easier access and maintenance. All solids are mascerated by the grinder pump, and the effluent is discharged into the pressurized pipe conveyance system. This system has been used in several locations throughout the United States and Europe and is considered very reliable. Many problems associated with earlier installations have been eliminated. Thus, the grinder pump system is considered a viable alternative under appropriate conditions, such as when there is inadequate space in which to construct a conventional gravity wastewater collection system or to provide the septic tanks needed for a STEP System.

Cluster (Neighborhood) Systems

Small wastewater collection and disposal facilities could be considered for developed unsewered areas which have been designated either existing or potential on-lot disposal problem areas. Neighborhood systems have promising applications where development is in clusters and in areas remote from existing treatment facilities. Several disposal options are compatible with these systems, including subsurface disposal, land application, and package treatment units with disposal to surface waters. Each alternative requires a wastewater collection system to convey the wastewater to a treatment and disposal site and has advantages and disadvantages which must be evaluated in terms of viability and cost-effectiveness.

Subsurface Disposal. Community subsurface disposal systems (large leaching areas) which serve more than one household are generally limited to special applications where the flow is low (less than 15,000 gallons per day) and land area is abundant. The design of a community system is more sophisticated than an individual system and may require pretreatment, and a wastewater collection system would be required to convey the septic tank effluent from the households to the leaching field site. The Massachusetts DEQE rules and regulations must be strictly adhered to.

Land Application. On a very small scale, this option generally has relatively high capital costs due to requirements concerning pretreatment, storage, distribution, buffer zones and fencing. Subsurface conditions can restrict the location of these facilities and create additional costs. Generally, other alternatives will be more cost-effective, environmentally sound and acceptable from the public's point of view. This alternative is most attractive for larger cluster systems which have no low cost alternative available for discharge to surface waters.

Package Treatment Plants. Commercially available wastewater treatment plants, or "package plants" are sold as prefabricated units or in early assembled components. They are available with capacities up to 1 mgd, but are not commonly used for flows of greater than 200,000 gpd. These units have higher manpower requirements associated with their use than other community systems. They cannot be installed and expected to run by themselves. Daily attention is required, and anything less will result in an inefficient operation. The most common package plants utilize the activated sludge process, which produces a secondary effluent requiring disposal.

Discharge of treated effluent to inland waterways may cause degradation of small streams which previously received no wastewater effluent and may jeopardize the public water supply.

On-Lot Disposal

Requirements for on-lot disposal systems are established under Title 5 of the State Environmental Code (current edition dated January 1, 1978) and supplemented by regulations of the Plymouth Board of Health effective January 1982, as discussed in Chapter 4. Current rules prohibit the installation of cesspools, and a septic tank leaching area system is required for on-lot disposal.

The following sections describe various methods for addressing on-lot disposal problems, including structural and non-structural alternatives.

Improved Operation and Maintenance Practices. Periodic removal of sludge and scum from septic tanks and cesspools by pumping is necessary to prevent leaching area clogging. On-lot systems have been known to give service for 20 years or more without pumping. However, most systems would be expected to fail in a much shorter period of time under such circumstances. Some authorities recommend that a septic tank or cesspool be pumped every three to five years. A more positive approach in determining when to pump (and one which recognizes the variability of sludge and scum accumulation rates) is to inspect the tank periodically, and measure the amount of sludge and scum accumulation. The U.S. Public Health Service recommends such inspection at least once a year.

Annual inspection and pumping of septic tanks will not guarantee the permanent functioning of an on-lot system, but it will ensure that the efficiency and life of a septic system will be maximized. This practice can also defer the need for extensions of the sewer system. The cost for pumping out a septic tank can vary from $35 to $75, but typically is about $60 in the Plymouth area.

In Plymouth, virtually all on-lot systems are privately owned and operated, the vast majority by individual homeowners. Thus, there is no institutional system that presently assumes their operational responsibilities. This can be attributed largely to the fact that properly installed on-lot systems require relatively little operational attention (i.e., periodic inspection and cleaning). It is widely acknowledged, however, that homeowners are typically derelict in providing even this minimal degree of attention, their usual operational undertakings being limited to responding when malfunctioning occurs by having the tank pumped out. The primary cause of such neglect is most likely one of simply not knowing that septic tanks require periodic maintenance, nor being aware of the consequences of neglect. Based on the questionnaire returns, it is evident that a very significant number of homeowners in Plymouth never have their septic tanks and cesspools cleaned.

Local jurisdictions are not always fully aware of operational problems with on-lot systems in their areas due to: 1) the lack of a regular inspection program and 2) the reluctance of homeowners to report problems (or answer questionnaires) that might violate health codes and require action on their part. The lack of such awareness tends to make on-lot systems appear more successful than they are; it is therefore quite likely that there are more individual disposal system failures in some areas than have been reported.

The primary concern is one of improving the maintenance of existing and new on-lot systems. Alternatives for doing so are as follows:

1. Educational Program - This could be initiated by Plymouth's Board of Health. Its purpose would be to inform individual on-lot system owners, both present and future, as to the need for proper maintenance, how to go about providing it, and the consequences of neglect. One way of providing the information is to distribute leaflets directly to individual homeowners via the mail or otherwise.

2. Imposing Maintenance Requirements - This is really a regulatory function, but it is presented here for convenience. Under this alternative, the Town could impose the requirement that all on-lot systems be inspected and pumped on a periodic basis (say, every two or three years). It could be made retroactive in addition to being required of all future septic tank installations. Such a regulation could be enforced by

requiring each owner to forward a certificate of inspection to the Town every two or three years. This certificate would be issued by Town-permitted septage hauling firms. Failure to forward the necessary certificate would subject the homeowner to fines set at a rate sufficiently above the cost of inspection and cleaning so that it would be more attractive to the homeowner to have the on-lot system inspected than to be faced with the fine. This enforcement approach, although theoretically effective, is somewhat impractical.

Instead of such a comprehensive enforcement program directed at homeowners, the Town could require septage hauling firms to maintain accurate records that are transmitted to the Town each year. This record keeping requirement could be imposed by Town as a condition to each firm's permit. The Town would review those records to identify on-lot systems which have not been inspected or cleaned for a set period of time and then issue notices requiring inspections. Again, the administrative burden associated with this approach makes it somewhat impractical.

Lastly, and perhaps more practical from an administrative standpoint, is to require each on-lot owner, both present and future, to enter into a service contract with a Town-permitted setage hauling firm under which the tank is to be inspected and cleaned in accordance with Town requirements. This service contract would be in effect for the life of the system. Present on-lot system owners would be given individual notice of this requirement and an adequate period of time to comply. As regards future on-lot system installations, such a contract could be a condition of Town approval of the system prior to its use. In order to monitor this mechanism, the Town could require annual reports to be submitted by the septage hauling firms.

3. Operation by the Town - Under this alternative the Town would assume responsibility for the proper maintenance of on-lot systems within their jurisdiction (it could also be responsible for installation). As its major responsibilities with regard to septic tanks, the Town would conduct periodic inspections, perform the necessary cleaning and septage disposal, respond to problems, and perhaps monitor groundwater quality in designated areas. Advantages of this arrangement include: 1) improved maintenance; 2) possibility of Federal grant assistance (EPA has expressed a concern about the tendency to install public sewers in areas where septic tanks, or other on-lot disposal systems would be adequate, and has stated that grants will me made more available for on-lot disposal systems); and 3) institution of a means to investigate and implement cluster systems (i.e., septic tanks serving more than one home). The agency empowered to inspect individual disposal systems should have the legal right of reasonable access to private property for purposes of inspection and cleaning. As regards financing, there are several options: 1) flat monthly charge from all septic tank users; 2) sanitary property tax levy; 3) monthly charge plus billings for actual cleanings made;

4) special assessments; and 5) rate equivalent to that charge to users of the sewer system.

By conducting maintenance programs on a municipal scale, adequate operating records could be maintained and useful correlations established with site conditions and other factors. To provide the information necessary to better assess site suitability, detailed surveys of individual disposal systems could be conducted to obtain more factual information concerning system performance. Performance data might be expressed in the form of "survival curves"* which plot the percentage of individual systems which have failed versus system lifetime and other relevant parameters such as generalized soils data. The extent to which generalized soils data correlate with actual performance of individual systems could be better established. If adequate correlation is found, soils surveys should be completed in those areas for which detailed soils data are not available. Furthermore, more intensive inspection programs could become feasible, including, for example, bacteriological testing of adjacent streams and ponds. One of the greatest factors leading to misconceptions about the adequacy of septic tanks is the lack of accurate records of performance on the local level. Where comprehensive surveys have been conducted, the results often refute the assumption of satisfactory service.

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*Cotteral, J. A., and Norris, D. P., 1969. "Septic Tank Systems," Journal of the Sanitary Engineering Division, ASCE, Vol. 95, No. SA4 (August 1969).

An effective surveillance and maintenance program would include the periodic inspection of each on-lot system to ensure that septic tanks were pumped and defective systems repaired or replaced as necessary. Each on-lot system should be inspected every year. More frequent inspection may be warranted in critical areas such as areas adjacent to lakes and in cases where failure is anticipated but not yet apparent. Each inspection should determine the general condition of the on-lot system with particular reference to any structural defects,the need for sludge pumping or grease removal, and signs of current or recent failure of the drain field. Where evidence of possible failure appears, additional inspections scheduled for periods of adverse conditions should be arranged. The inspection should include a reconnaissance of the area reserved for replacement of the leaching field, to check for incompatible use. A program of thorough and regular inspection of on-lot systems conducted by trained inspectors will reveal system malfunctions as they occur. When defects are uncovered, prompt and effective remedial action should be taken.

Problems affecting the septic tank or the distribution system are normally straightforward and simply corrected at relatively low cost. Failure of a leaching area, however, requires the design and construction of a replacement system and is as complicated as the original system design. The reservation of a leaching area replacement equivalent to the size of the original leaching area will ensure that a replacement system can be installed. The adequacy of the site for the replacement system should again be thoroughly investigated. The agency will then have the advantage of specific knowledge of the construction, maintenance, performance, and life of the original system on that site, and should be able to predict the performance of the replacement system. The possibility of variations in local conditions, however, makes adequate subsurface investigations a prerequisite to replacement system approval.

When a leaching area is installed to replace one which has failed because of the normal loss of infiltrative capacity resulting from cumulative clogging, an excellent opportunity is presented to take advantage of the beneficial effects of leaching area testing. A diversion box should be installed which will permit the alternating use of either the initial or the replacement drain field. Several years of rest for the original drain field should effect sufficient recovery to allow subsequent periodic alternation. By this procedure, total system life can be extended beyond the sum of the lives of the two drain fields.

Although the notion that a Town authority is to inspect and operate individual disposal systems may seem to be an overwhelming task, the assumption of responsibility could be staged to permit the Town to develop an orderly program. Initially, the Town might assume authority for all new installations and for existing installations in certain failureprone areas. Remaining areas would then be included as problems developed.

Upgrading Existing On-Lot Systems. Replacement of existing failed leaching areas, by installation of new leaching areas of approved design, is often an alternative to sewers. There are a few areas within the Town where high groundwater and/or steeply sloping land exists. The replacement of a failed leaching field in such areas will not alleviate the problem, and an alternative means of disposal should be considered. Replacement of the leaching field will serve to "buy time" for the homeowner, but the homeowner may well be faced with replacement of the system again within a short period. Continued replacement of leaching fields is further limited in some areas by the availability of land for replacement of the leaching area.

Replacement of existing leaching fields with "mound systems" (filled fields) is another alternative. Under this alternative, the leaching area is raised above the natural soil by building a leaching area on a pervious fill material, usually sand. This technique may be used for on-lot disposal where shallow permeable soils exist over impermeable soils or over water-saturated soils. The distribution pipes are placed in graded gravel which overlies the sand mound. The pipes are covered with additional gravel, fill material, and topsoil. Effluent from the septic tank is normally pumped to the filled field, where it seeps through the fill material.

In order to ensure adequate drainage of effluent through the fill and into the underlying soil, mound systems are not used in areas where the groundwater level is less than 2 feet from the ground surface, or where bedrock is at or near the surface. In all cases the completed system shall maintain at least 4 feet of permeable material below the leaching area without the presence of impervious material or groundwater.

Mound systems, where used with conventional septic tanks, add no significant operating and maintenance costs (other than minor pumping costs) over conventional septic tank leaching field systems. However, they do result in significantly larger capital investments. Mounds require sufficient borders for sloping to existing grade. The minimum area required to construct a mound system is about 7,000 square feet. Due to the significant areas required, mounds are most applicable to larger lots (at least 20,000 sq. ft. or more).

Holding Tanks. Holding tanks simply provide storage of wastewater, usually in an underground structure similar to a septic tank, and are periodically pumped out. These tanks may be constructed in areas which are unsuitable for any type of soil absorption system, either because of impermeable soils, water saturated soils, or a limitation on the amount of land available. A tank with a holding capacity of one to four weeks' wastewater production is normally installed. Included with the tank is an alarm to notify the homeowner that pumping is required. The tank is then pumped out by a septage hauler, and the contents discharged to a wastewater treatment facility.

In addition to the high cost of frequent pumping, the use of holding tanks on a large scale does not result in any reduction in the size or cost of the wastewater treatment facilities, as the septage hauler must convey the entire wastewater load from a household to the treatment facility. Existing treatment facilities do not have capacity to receive the volume of septic wastewater which would be generated from the use of holding tanks. The use of large numbers of holding tanks would also increase truck traffic throughout the Town.

The use of holding tanks is not considered a practical or long-term solution to wastewater disposal needs, but may represent a possible interim solution to problems encountered by a homeowner where alternative wastewater facilities are likely to be provided in a reasonably short period of time.

Waterless Toilets. The variety of waterless toilets presently sold ranges from incinerating toilets, which use natural gas or electricity to combust wastes, to composting toilets which use biological oxidation to stabilize and reduce the volume of waste material. Because the incinerating units are energy-intensive and have been associated with mechanical and odor problems in the past, their household use is not herein recommended. Composting toilets have been used for some time in Europe, and are now being marketed in this country. Being an aerobic process, the composting toilet requires a flue pipe and an adequate draft to draw air through the compost heap. The manufacturers claim this ventilation maintains the desired bacterial population and prevents odors and insects from entering the house either through the toilet stool or through the chute provided for garbage.

Title 5 regulations require that a properly functioning septic system must be installed to treat waste streams from sinks, showers and tubs, washing machines and other sources; referred to as "gray water". The leaching area may be reduced by up to 40 percent of normal requirements to account for the reduced water usage in a house equipped with a composting toilet. However, sufficient leaching area must be reserved to expand the field to the normal size. The use of such toilets and reduced leaching area is subject to written approval by the State. Also subject to approval is the manner and location for disposal of the end products of composting toilets. Title 5 regulations require that the end product be covered in an approved disposal area by a minimum of 2 feet of compacted earth, which would prohibit the use of this material as a soil conditioner and fertilizer. In addition, the buried material must be at least 4 feet above the high groundwater level. Thus, burial of a 1-foot layer of humus, with 4 feet to groundwater and 2 feet to ground level, would only be allowed where surface depth to groundwater is 7 feet or more.

Installation of a composting toilet is most economically done in new house construction. The large size of the unit (about 10 feet long, 4 feet wide, and 8 feet tall) and the radical plumbing changes makes retrofitting very difficult. Self-contained units are available for existing homes but their limited capacity (about two months) requires frequent toilet cleanout. In addition, proper disposal of the end product during wintertime's frozen ground conditions would be difficult.

The use of composting toilet requires suitable soil for on-lot disposal of gray water, an approved site for burial of end-products, and a basement with sufficient space to accommodate the large composting unit. Their use in precluded on lots with unacceptable soils for on-lot disposal or in houses with no basement. Installation of composting toilets in existing houses often involves extensive and costly remodeling, and installations are restricted to the first floor level.

Improved Standards for On-Lot System Construction. Minimum standards are required by Title 5 and Plymouth's "Supplements to Title 5," as discussed in Chapter 4. In Plymouth, the Board of Health is responsible for overseeing the design and installation of all on-lot disposal systems. A review has been made of the applicable standards and the procedures of enforcement, and we have concluded that the requirements of the standards are being applied. Strict adherence to rules and regulations for use of on-lot systems is a minimal requirement to insure adequate functioning of these systems.

Title 5 allows the use of garbage grinders. Garbage grinders contribute large amounts of additional solids to the watewater system. The concentration of suspended solids may be 25 percent to 100 percent higher in wastewater from a residence equipped with a garbage grinder than in wastewater from a residence which is not so equipped. The additional load generated by garbage grinders can contribute to accelerated deterioration of the leaching area, especially when frequent pumping of septic tanks is not practiced. Although the Supplements to Title 5 require a 25 percent increase in the sizing of a septic tank and leaching area to allow for the increased solids loading, it is recommended that garbage grinders should not be permitted with septic tank and leaching area systems.

In pervious soils such as sand, where high percolation rates are found, Title 5 permits a hydraulic loading of between 1 and 2.5 gallons per square foot per day, based on the assumption that the soil is the governing medium. The Plymouth "Supplement to Title 5" permits a maximum hydraulic loading of 1 to 1.3 gallons per square foot per day. The basis of these permissable rates, however, has been brought into question by some studies in recent years which have shown that the bacterial layer which accumulates on the surface of the leaching area can be the governing medium. The authors of these studies have suggested that loading rates should not exceed 0.35 gallons per day per square foot of leaching area. These studies, then, suggest that there should be some reduction in permissible wastewater loading rates.

Another apparent reason for system failures is the presence of fines in the soils surrounding the leaching area. Title 5 requires a 2-inch layer of washed stone placed over leaching trenches. This layer, however, does not prevent fines from the overlying soils from seeping into the trenches. In some installations, roofing paper has been laid over the stone to prevent the entry of fines. While this precaution may be a satisfactory preventative measure initially, the paper will eventually deteriorate at which time the fines will begin to seep into the trench. This condition could be prevented if a synthetic filter fabric were used in lieu of the roofing paper. Recent studies* conclude that a spunbonded polyfin fabric can be utilized. The fabric is readily available, durable, easy to apply and permits the leaching field to breathe while preventing the entry of fines.

Water Saving Devices. Water conserving fixtures in the home can have a measured impact on water usage and wastewater quantities. Studies have shown that the conventional toilet (about 5.5 gallons per flush) can account for as much as 40 percent of the average home's daily water use. Conventional showers can represent an additional 25 to 35 percent of the average water use. In order to effect any significant reduction in water consumption, these two fixtures must be modified.

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*Harkin, John M., "Status Report on the Porox (Hydrogen Peroxide) Process," in Individual Onsite Wastewater Systems, Proceedings of the Sixth National Conference, ed. by N.I. McClelland, Ann Arbor:Ann Arbor Science Publishers, Inc. (1979).

Water-saving toilets are available which can reduce overall water use an estimated 15 percent. Other devices such as faucet aerators; water-saving showerheads, washing machines and dishwashers; and the prohibition of garbage disposals would reduce water use still further.

Another method which has been proposed for reducing water use is the recycling of so-called "gray water" through the toilet. Gray water is the wastewater from sinks, baths, and washing machines. The recycling of gray water, even through homes not equipped with other water-saving devices, could reduce use of water by 20 percent. Such a system would require a collection tank for gray water, a filter to remove solids, a pressure tank to provide a supply of water under pressure to the toilet, and a pump to recirculate the water. The regular drains from the tub or shower, lavatory, and laundry must be disconnected and the sewer connection sealed. The drains from these fixtures must be repiped to the collection tank. The use of this type of system in conjunction with water-saving showerheads, washing machines, and dishwashers would reduce water use still further.

It must be emphasized that the use of water-saving appliances or systems does not eliminate the need for an on-lot disposal system. The reduction in flow from a home using these devices would allow for construction of a smaller leaching field, although sufficient land must be available to accommodate a full-sized field. In existing homes presently experiencing difficulties with the on-lot disposal system, a reduction of the flow to the leaching fields could have some beneficial effect. In general, the degree to which the on-lot system would be benefitted cannot be assessed. Variables such as the age of the system, soil types, total water usage and other factors would influence the increased efficiency or extension of the useful life of existing systems whcih could be achieved.

It would be impractical to require water-saving devices to be retrofitted in existing homes. A policy of encouraging or requiring homeowners to install such fixtures at replacement time would be feasible and is recommended. The Town should also consider making the use of water-saving fixtures mandatory for all new construction. Benefits to be derived from this policy would be reduced area requirements for leaching fields, potential increased life of on-lot systems and lessening of demand for water services.

The use of "gray water" systems poses potential health hazards which have not been fully resolved. Significant quantities of organic material and microorganisms are contained in gray water. Exposure to these untreated wastes could transmit pathogens. Such contact is very possible, especially by children who place their hands or toys in the toilet. The State codes contain no express provisions for reuse of gray water, and such proposals would be subject to review and approval by local and State health authorities. Gray water reuse is not recommended.

Zoning and Land Use Controls. The effects of increased lot sizes on septic system longevity are functions of probability, rather than measurable performance criteria. A larger lot provides greater area for location of replacement leaching fields. If a mound system is to be constructed, the area requirements can be substantial, especially where groundwater or impervious materials are near the surface. Percolation tests and observation pits may be used to locate the most favorable disposal area within the expanse of the larger lots. An increase to larger lot sizes of about 1 acre can have a beneficial impact in those areas not yet developed. The future pressures for public sewerage would be lessened, and potential growth would be reduced in areas of rezoning. Plymouth's Aquifer Protection By-Law, discussed in Chapter 4, limits development to 1-acre lots in unsewered areas overlying the recharge areas to water supply wells.

The Town of Plymouth can and has exercised zoning and land use as tools for controlling growth and to accelerate or retard the pressures for sewerage. Existing developments which have on-lot disposal problems have not been affected by zoning changes. Larger lot sizes in areas of future development would maximize the potential for successful on-lot disposal, but cannot guarantee that extension of sewers would never be necessary. Potential construction of multiple dwelling units and development of public or private recreational lands could have major impacts on public sewerage facilities and projected needs if present controls are relinquished.

Summary

Several alternative methods for wastewater management have been presented. Structural alternatives include conventional, pressure, and vacuum sewers, as well as neighborhood cluster systems. Other nonsewering structural alternatives which require significant capital and operating expenditures include:

1. Upgrading existing on-lot disposal systems.
2. Holding tanks.
3. Waterless toilets.

Nonstructural alternatives for improving and extending the use of on-lot disposal systems include:

1. Improved operation and maintenance (O&M) practices.
2. Improved standards for on-lot system construction.
3. Water-saving devices.
4. Zoning and land use controls.

A review of the benefits related to improvements of on-lot disposal systems indicates that some non-sewering alternatives have application in Plymouth. A summary of these factors and a qualitative evaluation of associated impacts for each of these measures are presented in Table 5-1. Each of the alternatives has potential for solving wastewater disposal problems and may be suited for certain situations in Plymouth.

TABLE 5-1. EVALUATION OF NONSEWERING ALTERNATIVES

Item	Applicability	Advantages	Disadvantages
1. Replacement of conventional leaching field.	Suitable area must be available for replacement field.	Least cost of rehabilitation techniques.	Lack of space or poor soils may prohibit use.
2. Replacement of leaching field with a filled field system.	Large lots with suitable subsurface conditions.	With sufficient land the on-lot system could be extended indefinitely.	Very high cost, aesthetically obtrusive.
3. Holding tanks.	Limited to use where on-lot systems cannot function and sewers are not available.	Can be utilized as last resort.	High cost and lack of treatment facilities for pumped wastes.
4. Waterless toilets.	New homes.	Reduces waste load to septic system.	Very high cost, limited to first floor installation, difficult to retrofit, disposal of end product.
5. Improved operation and maintenance practices.	Desirable for all existing and future on-lot systems to extend useful life.	Low cost, most effective technique available to homeowner.	Difficult to enforce.
6. Improved standards for on-lot systems.	Upgrade future systems, expand State criteria as required.	Maximizes life of septic system.	May increase initial costs.

TABLE 5-1 (Continuation). EVALUATION OF NONSEWERING ALTERNATIVES

Item	Applicability	Advantages	Disadvantages
7. Water-saving devices.	New and existing homes.	Low cost, reduces wastewater volume, water use, and energy consumption.	Insignificant
8. Zoning and land use controls.	Undeveloped land.	Minimizes potential demand for municipal facilities.	Significant cost increase for private development. Political issue with intense advocates on both sides.

Conclusions

In reviewing the alternatives described in this chapter the following are given further consideration for potential application in Plymouth.

1. Pressure sewers.
2. Upgrading existing on-lot disposal systems.
3. Improved operation and maintenance practices for existing on-lot systems.
4. Holding tanks for interim use.
5. Improved standards for on-lot system construction.
6. Water-saving devices.
7. Zoning and land use controls.

CHAPTER 6

WASTEWATER MANAGEMENT PLANNING

General

In Chapter 4, the eleven study subareas within the Town of Plymouth, their general characteristics, and their associated wastewater-related problems were identified. Chapter 5 defined the various alternatives available for the solution of wastewater disposal and treatment problems.

Chapter 6 integrates Chapter 5 and the portion of Chapter 4 dealing with the ten unsewered subareas and the existing collection system by discussing alternative solutions to resolve the problems in each of the subareas identified as having problems. Alternatives considered include conventional sewers, pressure sewers, upgrading existing on-lot disposal systems, improved operation and maintenance practices for existing on-lot systems, holding tanks for interim use, improved standards for on-lot systems, water-saving devices, and zoning and land use controls.

Analysis of Subareas

As previously summarized at the end of Chapter 4, there are only two unsewered subareas in Plymouth that require a detailed analysis for the determination of a solution to their wastewater-related problems. These subareas are Manomet and West Plymouth. Section 1 of Manomet (Figures 4-4 and 4-5) is considered to be the only existing problem area within the

Manomet study subarea. The industrially and commercially zoned areas overlying the recharge areas of the North Plymouth well, shown in Figure 4-3, are considered to be the only potential problem in the West Plymouth study subarea.

Manomet. Section 1 has the most severe wastewater related problems. The area is densely developed with an average population density varying from 10 persons per acre during the off season to 20 persons per acre during the summer. It is also the only section of Manomet conditionally recommended for sewering in the Section 208 Areawide Water Quality Management Plan prepared by SRPEDD in 1978 "if the 201 study determines that on-site sewer systems cannot function properly," as sewering other sections of Manomet "would promote excessive development in that area."

As discussed in Chapter 4, the great majority of the on-lot wastewater disposal systems in Section 1 do not comply with the current State and Town codes because of the small lot sizes in this area. Because development in this section is presently almost at saturation, the alternative of implementing more stringent zoning controls is not applicable here. Upgrading the existing on-lot systems to meet the codes by replacing or expanding the leaching areas or providing mound systems is impossible because of the lack of available space.

The use of water-saving devices and the adoption of improved operation and maintenance practices for existing on-lot disposal systems can be implemented immediately and with minimal cost to the homeowner. While these alternatives will reduce the probability of a failure, they are only partial remedies and do not eliminate the contamination of groundwater by systems that have insufficient depth to groundwater. For the portion of Section 1 labeled Area A in Figure 6-1, an area which includes Subsections 1A and 1D and portions of Subsections 1B and 1C, the only alternatives available which can eliminate the potential public health problem associated with wastewater disposal are the provision of sewers and a wastewater treatment facility or the provision of holding tanks with hauling of wastewater to the Night Soil Disposal Facility in North Plymouth. In this area, which encompasses 600 lots, it is unlikely that most homeowners whose on-lot systems fail in the future would be permitted to replace their systems, because: 1) it is unlikely that the permitting authorities would grant variances to the code requirements that systems not be constructed on lots having high groundwater or on lots in close proximity (75 feet) to surface waters; and 2) it is unlikely that the permitting authorities would grant variances to permit reduced on-lot system sizing in cases where the required system will not physically fit within the lot boundary.

However, in the remaining areas of Section 1, which fail to meet code requirements for reasons related solely to failure to meet minimum setback requirements, it is likely that home-owners <u>will</u> continue to be granted variances to replace failed systems, as these systems would represent a lesser threat to public health.

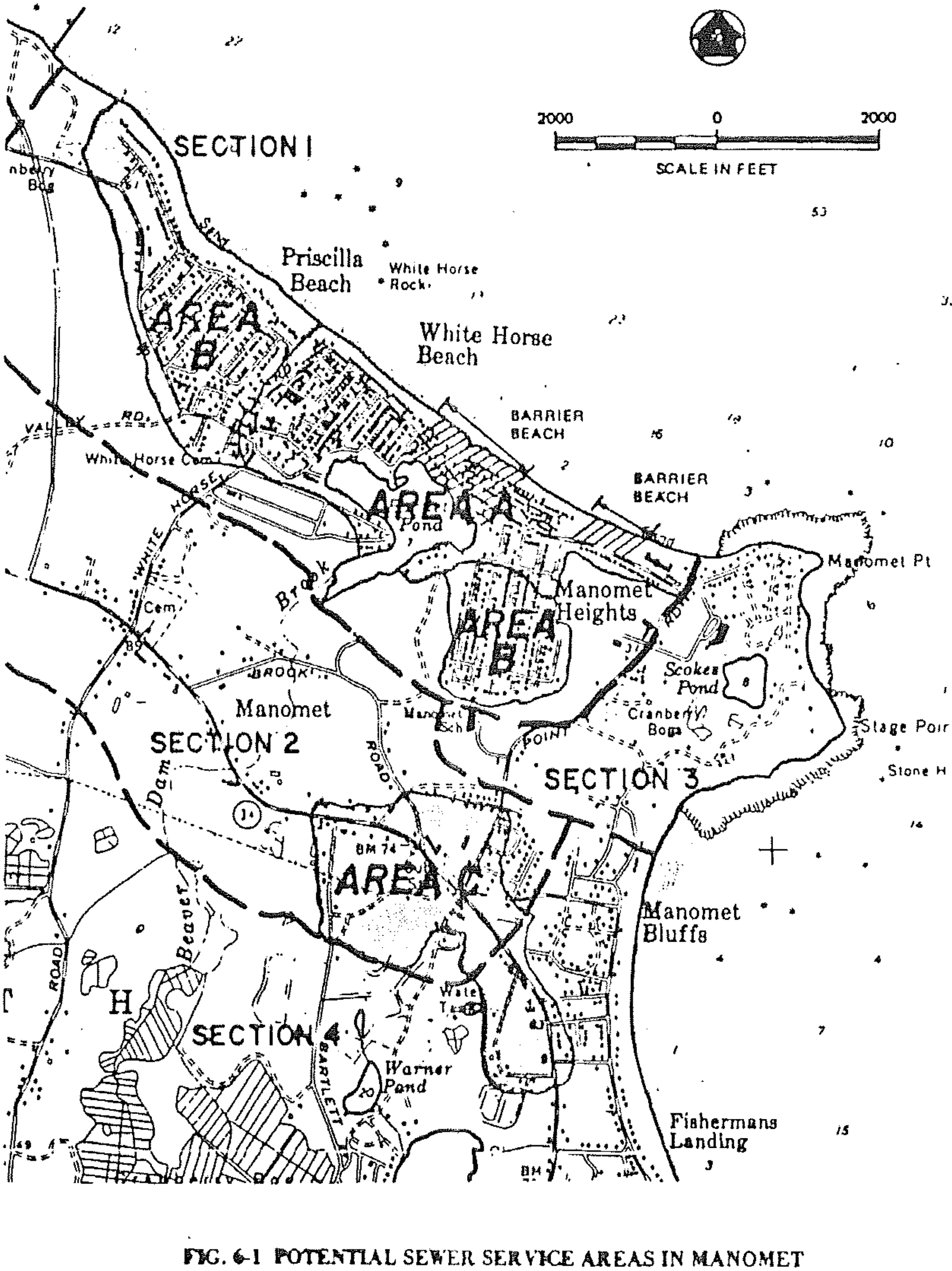

FIG. 6-1 POTENTIAL SEWER SERVICE AREAS IN MANOMET

Eight alternative schemes have been considered to reduce the public health threat associated with on-lot wastewater disposal in Section 1.

Under Scheme 1, shown in Figure 6-2, sewers would be provided for approximately 1,200 lots in Areas A and B of Section 1. Gravity sewers would be used wherever the topography permits. Wastewater would be conveyed to a new wastewater treatment facility, treated and discharged to the ocean or to a land disposal facility located nearby (as discussed in Chapters 8, 9 and 10).

Under Scheme 2, gravity sewers would be provided for only Area A. This area would include all lots for which it is unlikely that variances to code requirements would be granted in the future, as well as a few adjacent properties which could conveniently be included at little additional cost. The remaining lots in Section 1 would continue to use on-lot disposal and would replace their systems with new systems as they fail.

The Massachusetts Executive Office of Environmental Affairs has officially designated two areas in Section 1 as barrier beaches (see Figure 6-2). These two areas contain approximately 95 lots. The Massachusetts Office of Coastal Zone Management (CZM) has issued guidelines for implementing Executive Order No. 181, which established a framework for the management of barrier beaches in the State (see Appendix K). The intent of the executive order and the guidelines is to prevent the use of federal and state funds for projects which would encourage growth

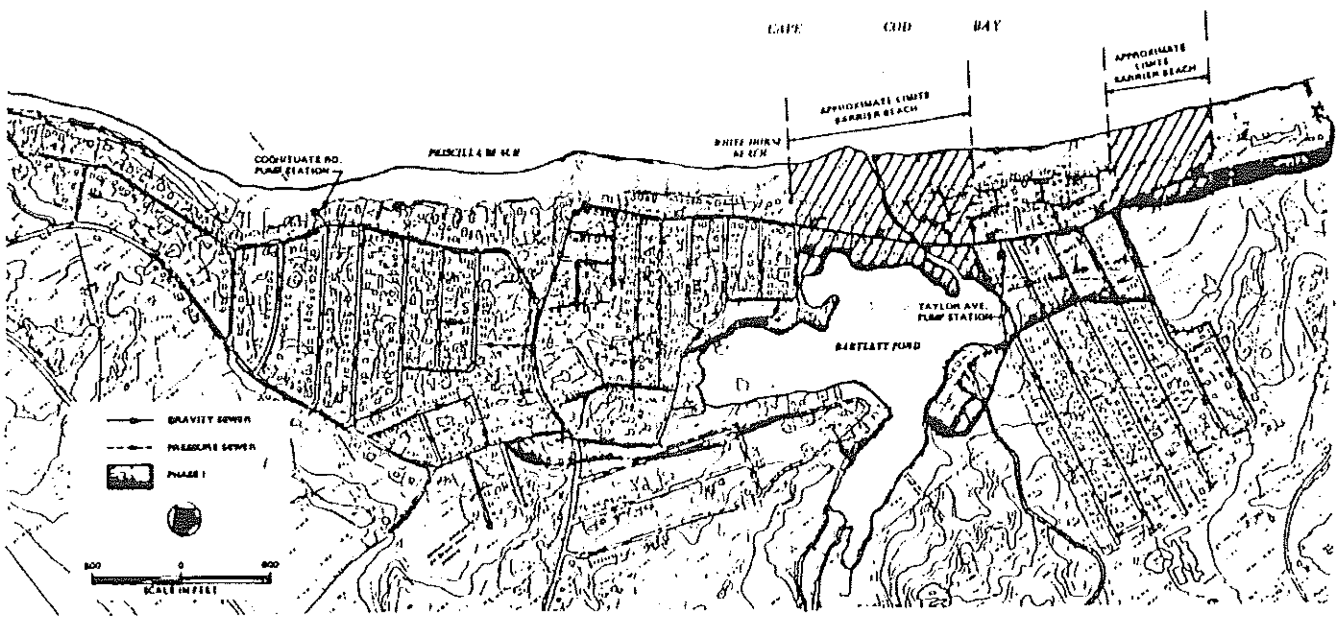

FIG. 6-2 MANOMET - SCHEME I

and development on barrier beaches, which are dynamic landforms that are considered unsafe for the contruction of permanent structures. Under the guidelines, new sewer lines may not be constructed on barrier beaches unless no other feasible alternative exists. CZM has indicated that in cases where sewers are the only feasible alternative, the construction of gravity sewers will not be funded, or even permitted, on barrier beaches. In spite of the fact that it is highly questionable whether the provision of gravity sewers on the Manomet barrier beaches, which are already highly developed, would have any impact on further development, only pressure sewers having capacity for serving only existing development may be approved. Schemes 1 and 2 must therefore be rejected.

Schemes 3 through 6 are included to address the CZM concerns. Schemes 3 and 4 are the same as Schemes 1 and 2, respectively, except that low-pressure sewers with grinder pumps would be provided to serve the 95 lots located on the two barrier beaches. It should be noted that 10" and 12" gravity sewers serving areas beyond the barrier beaches would pass through the barrier beaches along Taylor Avenue even if all houses on the barrier beaches themselves are served by pressure sewers. In order to avoid the presence of these gravity sewers along Taylor Avenue, the following additional costs would need to be incurred:

1. the costs of constructing and operating an additional pumping station in the vicinity of Williams Avenue and Taylor Avenue;
2. the costs of a pressure sewer and grinder pumps to serve homes beyond the eastern end of the barrier beach in Subsection 1D; and
3. the cost of an additional force main having a length of at least 4,000 feet to deliver wastewater from Subsections 1A, 1C and 1D around the western side of Bartlett Pond.

It is therefore deemed infeasible to avoid a gravity intercepting sewer along Taylor Avenue under these schemes.

Scheme 5 is the same as Scheme 2 except that the on-lot systems on the barrier beaches would immediately be replaced with holding tanks. Scheme 6 is the same as Scheme 5 except that the holding tanks would be installed gradually over a ten-year period as the on-lot systems "failed".

Two schemes were considered wherein no sewers would be constructed in Manomet. Under Scheme 7, holding tanks would immediately replace all on-lot systems in Section 1 which have inadequate depth to groundwater or inadequate distances to a surface watercourse (approximately 330 lots*). This alternative would have similar environmental benefits to Schemes 2,4, and 5. Scheme 8 would be similar to Scheme 7 except that the holding tanks would be installed gradually over a ten-year period as the

* Including all 185 lots in Subsection 1A, 75 lots in Subsection 1B, 50 lots in Subsection 1C, and 20 lots in Subsection 1D, as discussed in Chapter 4.

on-lot systems "failed". Cluster systems using subsurface disposal areas were not included as a scheme because of the need to address problems over a widespread area rather than for small groups of homes.

The estimated capital, annual O&M, and present worth costs of the alternative schemes for addressing wastewater problems in Section 1 of Manomet are compared in Table 6-1. Present worth costs are presented to facilitate comparison of equivalent long-term costs of the various schemes (a discussion of the concept of present worth and the assumptions used in the cost analyses contained in this facilities plan is included in Chapter 9). However, as the benefits of each of the schemes are not the same, it cannot be said that the lowest cost scheme is necessarily the most cost-effective.

It can be seen that the capital costs of the schemes which incorporate sewers are far higher than those which do not. The capital cost for providing sewers for both Areas A and B is estimated to be over $7.5 million, or about $3100 per lot served.

Schemes 3 and 4 are more costly than Schemes 1 and 2, respectively, because the low-pressure sewers with grinder pumps to be provided on the barrier beaches are considerably more costly than gravity sewers.

Schemes 7 and 8 have several disadvantages when compared with Schemes 1-4. First, they do not provide a permanent

TABLE 6-1. COMPARISON OF COSTS OF ALTERNATIVE WASTEWATER SCHEMES FOR [illegible]

	Scheme 1: Provide sewers for Areas A and B (1200 lots)	Scheme 2: Provide sewers for Area A (600 lots); continue use of on-lot disposal in Area B	Scheme 3: Same as Scheme 1 except for provision of low pressure sewers for 95 lots on barrier beaches	Scheme 4: Same as Scheme 2 except for provision of low pressure sewers for 95 lots on barrier beaches	Scheme 5: Replace 95 on-lot systems on barrier beaches with holding tanks immediately; provide sewers for remainder of Area A only	Scheme 6: Replace 95 on-lot systems on barrier beaches with holding tanks as systems fail[1]; provide sewers for remainder of Area A only	Scheme 7: Provide no sewers; provide holding tanks for 350 lots with high groundwater immediately	Scheme 8: Provide no sewers; provide holding tanks for 350 lots with high groundwater as systems fail[1]
Capital Costs								
Collection system	$4,214,000[2]	$1,898,000[2]	$4,161,000	$1,845,000	$1,681,000	$1,681,000	$ 0	$ 0
House connections @ $1000 per lot	1,200,000	600,000	1,200,000	600,000	505,000	505,000	0	0
Treatment and disposal[3]	2,184,000	1,525,000	2,184,000	1,525,000	1,525,000	1,525,000	0	0
Holding tanks @ $3700 per lot[4]	0	0	0	0	351,000	0	1,221,000	0
Grinder pumps[5]	0	0	215,000	215,000	0	0	0	0
TOTAL	$7,598,000	$4,023,000	$7,760,000	$4,185,000	$4,062,000	$3,711,000	$1,221,000	$ 0
Annual O&M Costs								
Collection and delivery system	$ 9,300	$ 4,000	$ 11,600	$ 6,300	$ 4,000	$ 4,000	$ 0	$ 0
Treatment and disposal	60,700	52,500	60,700	52,500	51,500	51,500	0	0
Installation of new holding tanks[6]	0	0	0	0	0	35,200	0	122,100
Holding tank pumpouts[7]	0	0	0	0	78,700	48,800 (avg)	392,700	240,700 (avg)
Replacement of on-lot systems[8]	0	204,000	0	204,000	204,000	204,000	295,800	295,800
On-lot system pumpouts[9]	0	18,000	0	18,000	18,000	19,100 (avg)	26,100	29,900 (avg)
TOTAL	$ 70,000	$ 278,500	$ 72,300	$ 280,800	$ 356,200	$ 362,600	$ 714,600	$ 688,500
Present Worth Costs								
Capital costs	$7,598,000	$4,023,000	$7,760,000	$4,185,000	$4,062,000	$3,711,000	$1,221,000	$ 0
Land acquisition	113,000	64,000	115,000	64,000	64,000	64,000	0	0
Interest during construction	598,000	317,000	611,000	330,000	320,000	297,000	96,000	0
P.W. of annual cost	694,000	2,760,000	717,000	2,783,000	3,380,000	3,483,000	7,082,000	6,437,000
Less P.W. of salvage value								
Collection system	-556,000	-250,000	-549,000	-243,000	-221,000	-221,000	0	0
House connections	-158,000	- 79,000	-158,000	- 79,000	- 67,000	- 67,000	0	0
Treatment and disposal[3]	-178,000	-128,000	-178,000	-128,000	-178,000	-128,000	0	0
Holding tanks	0	0	0	0	0	0	0	0
Land acquisition	- 39,000	- 26,000	- 39,000	- 26,000	- 26,000	- 26,000	0	0
Total present worth cost	$8,072,000	$6,681,000	$8,279,000	$6,886,000	$7,334,000	$7,104,000	$8,399,000	$6,437,000
Annualized cost	$ 811,000	$ 674,000	$ 835,000	$ 695,000	$ 760,000	$ 777,000	$ 847,000	$ 650,000

1. It is assumed that the average life of an on-lot system is 10 years.
2. Costs for collection system assuming all sewered areas, including those on barrier beaches, utilize gravity sewers.
3. Including cost of pump stations and piping required to deliver wastewater from the collection system to the treatment plant (see Chapters 9 and 10).
4. Cost based on provision of 3,000-gallon, buried, concrete holding tanks.
5. Cost based on 2 homes per grinder pump.
6. This cost is assumed to occur annually only for the first 10 years of the study period.
7. It is assumed that the annual cost of pumpouts is $1820 for a year-round resident and $550 for a seasonal resident. Average annual cost over 20-year planning period is given.
8. Assuming average replacement cost of $3400 per system.
9. Assuming average pumpout cost and frequency of $60 and two years, respectively.

6-10

solution for any of the homes in Manomet, since: 1) nearly 900 homes in Section 1 would still be dependent on on-lot disposal systems which fail to comply with State and local codes, and 2) holding tanks such as those provided to serve the remaining homes are generally not considered to be an acceptable permanent solution. Secondly, no State and Federal funding would be available to provide holding tanks; thus, the local shares of these alternatives would greatly exceed those of Schemes 3 and 4. Thirdly, the distribution of costs would be far from uniform, as the annual pumping cost to individual homeowners using holding tanks on a year-round basis would approach $2,000 per year while the cost to homeowners permitted to retain their on-lot disposal systems would be only on the order of $30 per year. Finally, Scheme 8 would not address the potential health threat immediately, but only gradually over a 10-year period.

Schemes 5 and 6 are similar to Schemes 7 and 8 in that they do not provide a permanent solution for a large number of homes (homes in Area B will still be out of compliance with existing codes). Scheme 6 can easily be rejected because it is more costly on a present worth basis than Scheme 4 and also delays the elimination of the potential public health threat for ten years.

Schemes 4 and 5 are not so easily rejected, but have two significant disadvantages when compared with Scheme 3: 1) they fail to provide any permanent solution for Area B, and 2) because the recurring costs of on-lot system replacements must be borne by the homeowners, the local present worth costs for Schemes 4 and 5 are actually estimated to be somewhat higher than that of Scheme 3.

Because it offers a permanent solution to the potential public health problems in the area, eliminates noncompliance with Title 5, and addresses the objections of CZM to the construction of gravity serves on barrier beaches, it is recommended that Scheme 3 be selected as the preferred alternative for Section 1 of Manomet. The sewers associated with the 600 lots in Area A (which has the highest need) would be constructed during Phase I, and the remaining 600 lots would be sewered under Phase II of construction, as shown in Figure 6-2.

The remaining sections of Manomet were found to have fewer and less severe wastewater related problems than Section 1. On-lot systems in these sections which fail in the future can generally be replaced without threatening public health. Improved operation and maintenance along with a general public education program should be provided in these sections to keep problems to a minimum.

Some neighborhoods in the Fresh Pond area (Section 3) have lots as small as 5,000 sq. ft., and some of the homes in these neighborhoods rely on private wells for drinking water. Referring to Table 4-2, the minimum required lot sizes range from 22,000 sq. ft. to 29,000 sq. ft. for homes that are not served by the public water supply. Water supply lines do exist throughout the Fresh Pond area. Therefore, any homes that might experience well water contamination will be able to connect to the Town water supply.

Many homes in Section 6 on lots of 5000 sq. ft. still have private wells. These homes are out of compliance with current Board of Health regulations (refer to Table 4-2) and should be connected to Town water as soon as possible.

Portions of Sections 2 and 4 centered along Route 3A between Bartlett Road and Samoset Ave. were designated as a village center under the Plymouth Comprehensive Plan, which was adopted in 1980. This area, shown as Area C in Figure 6-1, is expected to experience significant commercial development in the future, and it overlies the recharge area of the existing Wanno's Pond well, which is located just north of Fresh Pond and supplies town drinking water to the Manomet area. If sewers are provided in Section 1, provision should be made to expand the sewer system to serve this area in the future, as was recommended in the Comprehensive Plan.

West Plymouth. As discussed in Chapter 4, future growth is this subarea is presently limited, as the subarea overlies the recharge area of the North Plymouth well. The zoning controls of the Aquifer Protection Bylaw protect the groundwater which supplies this well.

If industrial and commercial growth consistent with the Plymouth Comprehensive Plan (or "Village Centers Plan") adopted at the 1980 town meeting is to occur, the only alternatives applicable to this area will be the provision of sewers or holding tanks. Neither the use of water-saving devices or improved operation and maintenance of on-lot systems would protect the aquifer from contamination from wastes of a non-domestic nature, which will require disposal outside the area. Since holding tanks are only a temporary solution, sewers represent the only permanent solution consistent with the desired development of the area. Both the Plymouth Comprehensive Plan and the Areawide Water Quality Management (208) Plan called for sewers to be constructed at least as far as the intersection of Route 44 and Plympton Road (Route 80). On April 5, 1983, the Town's Citizens Advisory Committee concurred that the capacity to serve this area should be provided in the wastewater facilities presently being planned.

Figure 6-3 shows the proposed area of expansion of the existing sewer system into West Plymouth. Expansion of the sewer system would encompass all industrially-zoned land north of Route 44 and east of Plympton Road and would also serve residentially- zoned land along Route 44 and Plympton Road and commercially- zoned land in the proposed village center to the west of the intersection of Route 44 and Plympton Road.

Several alternative piping configurations have been considered to convey wastewater from this area to the existing treatment plant, as shown in Figures 6-4 through 6-9. Common to and included with these delivery system alternatives is the reduction of excessive I/I, as discussed in Appendix J, and the replacement of two segments of existing interceptors with new,

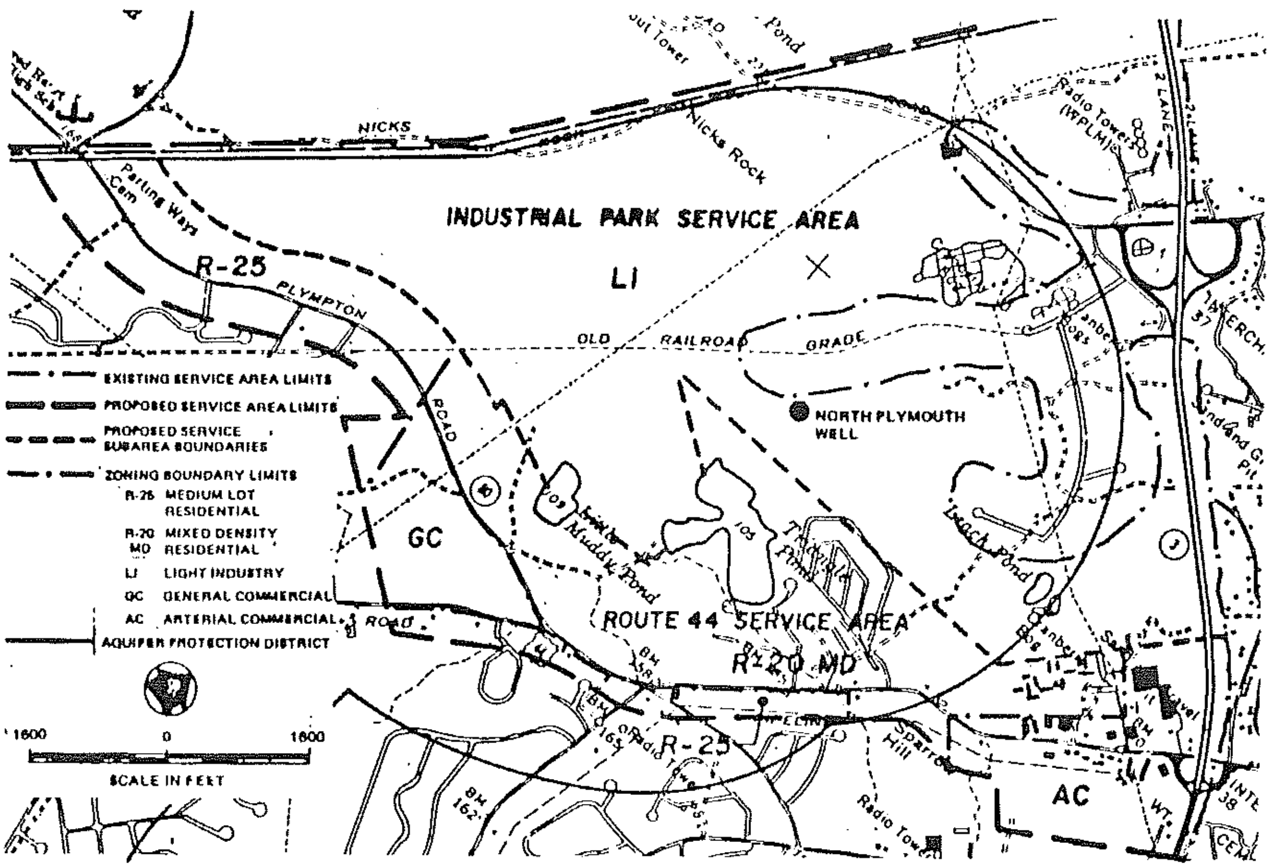

FIG. 6-3 PROPOSED WEST PLYMOUTH SERVICE AREAS

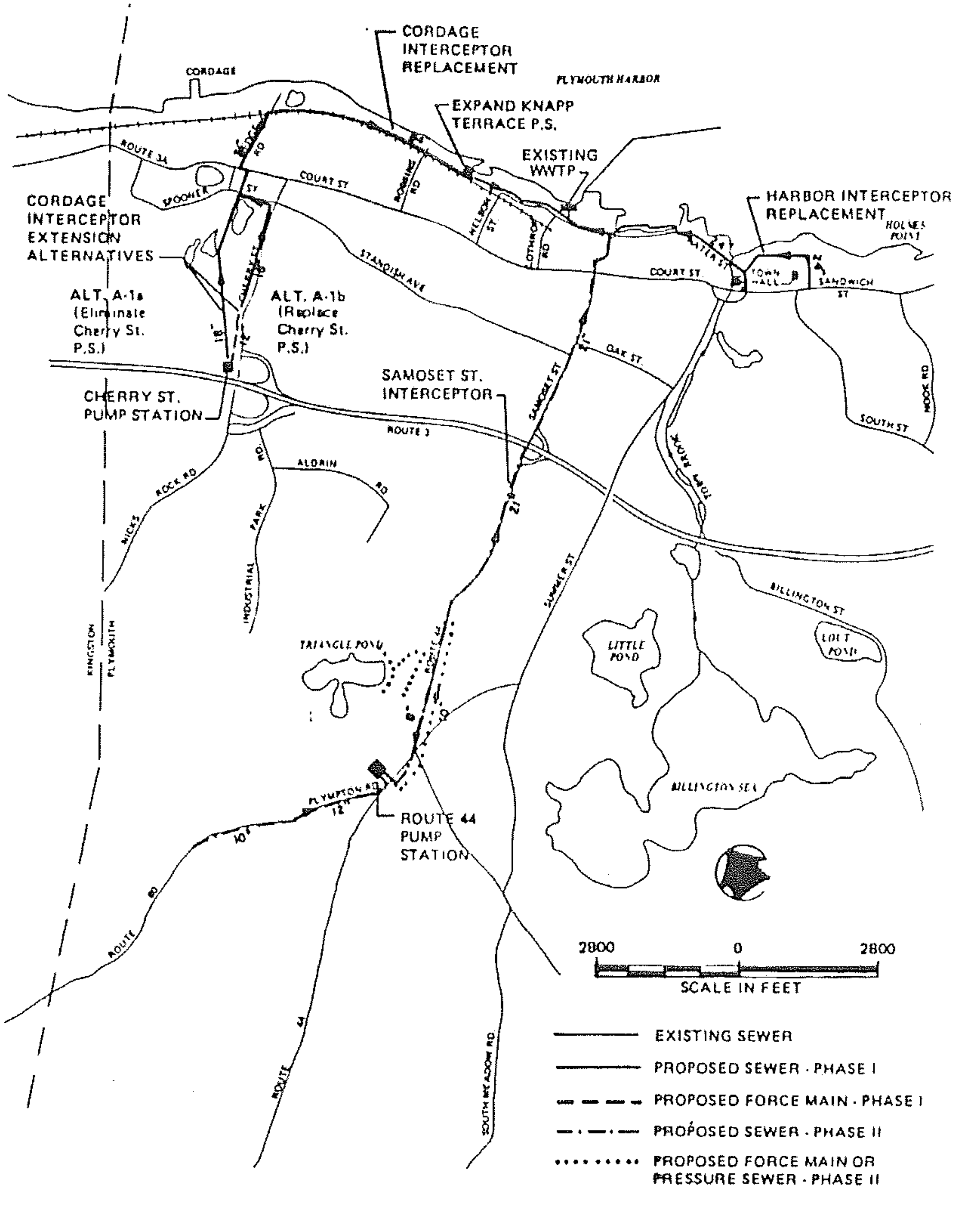

FIG. 6-4 DELIVERY SYSTEM ALTERNATIVE A-1

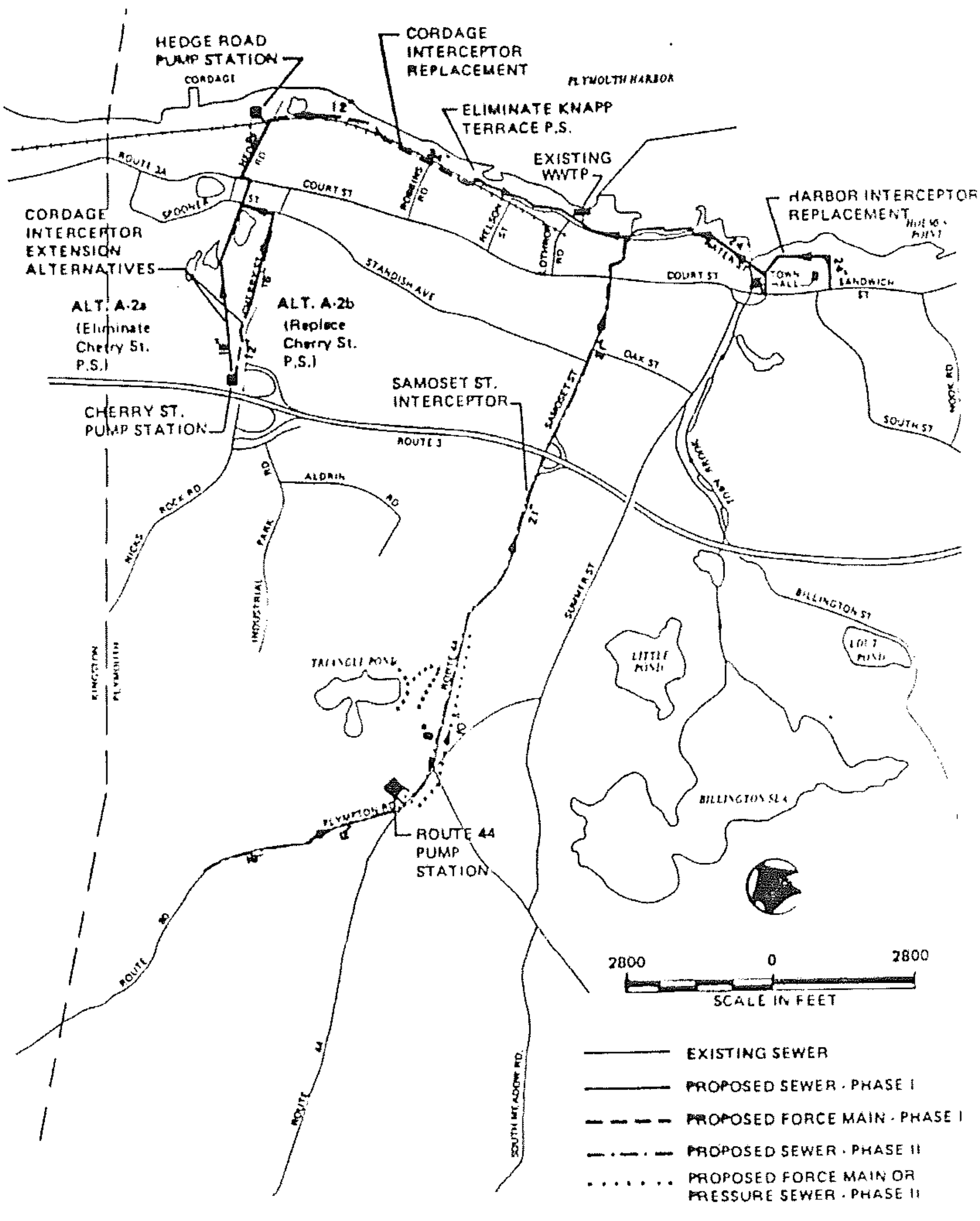

FIG. 6-5 DELIVERY SYSTEM ALTERNATIVE A-2

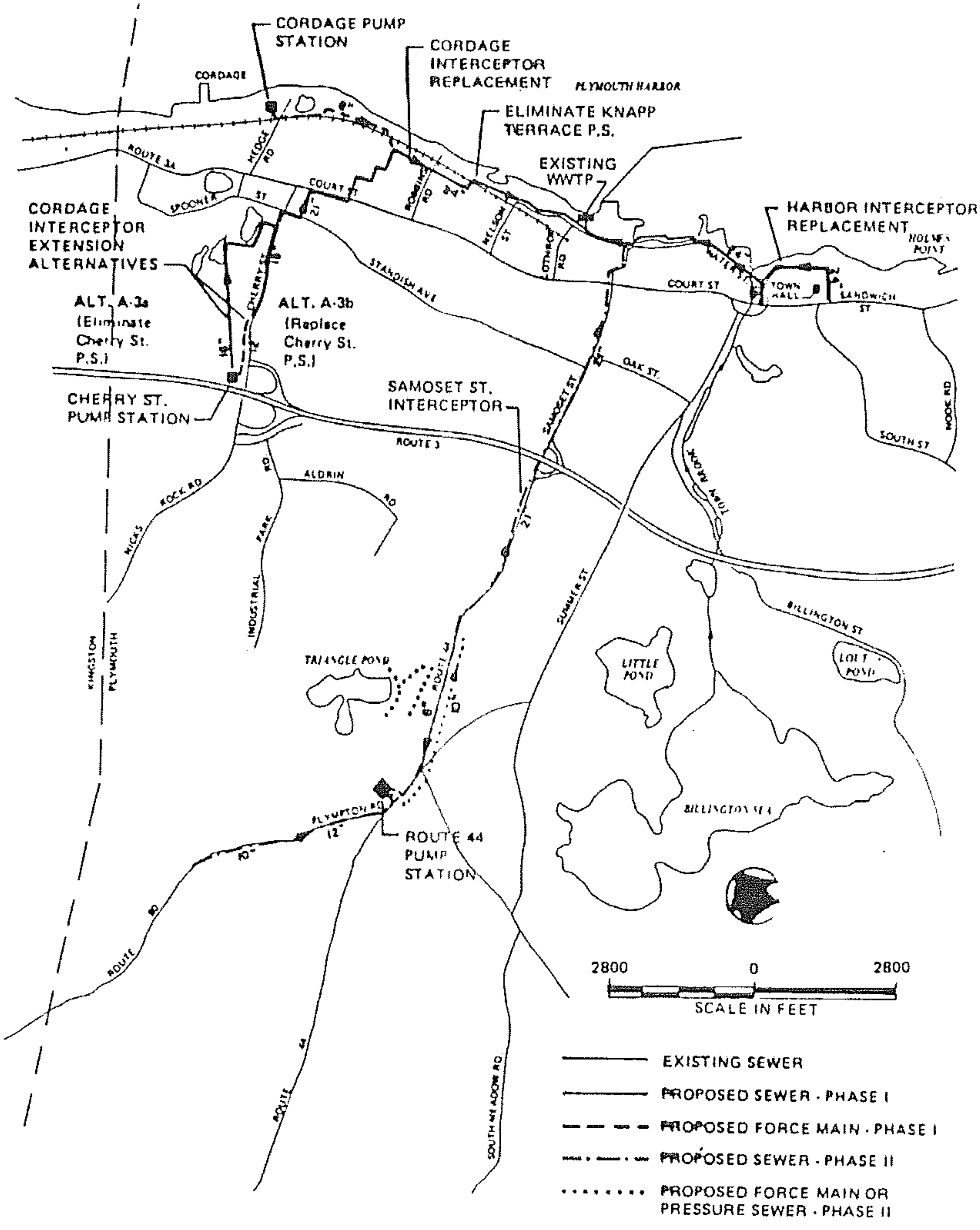

FIG. 6-6 DELIVERY SYSTEM ALTERNATIVE A-3

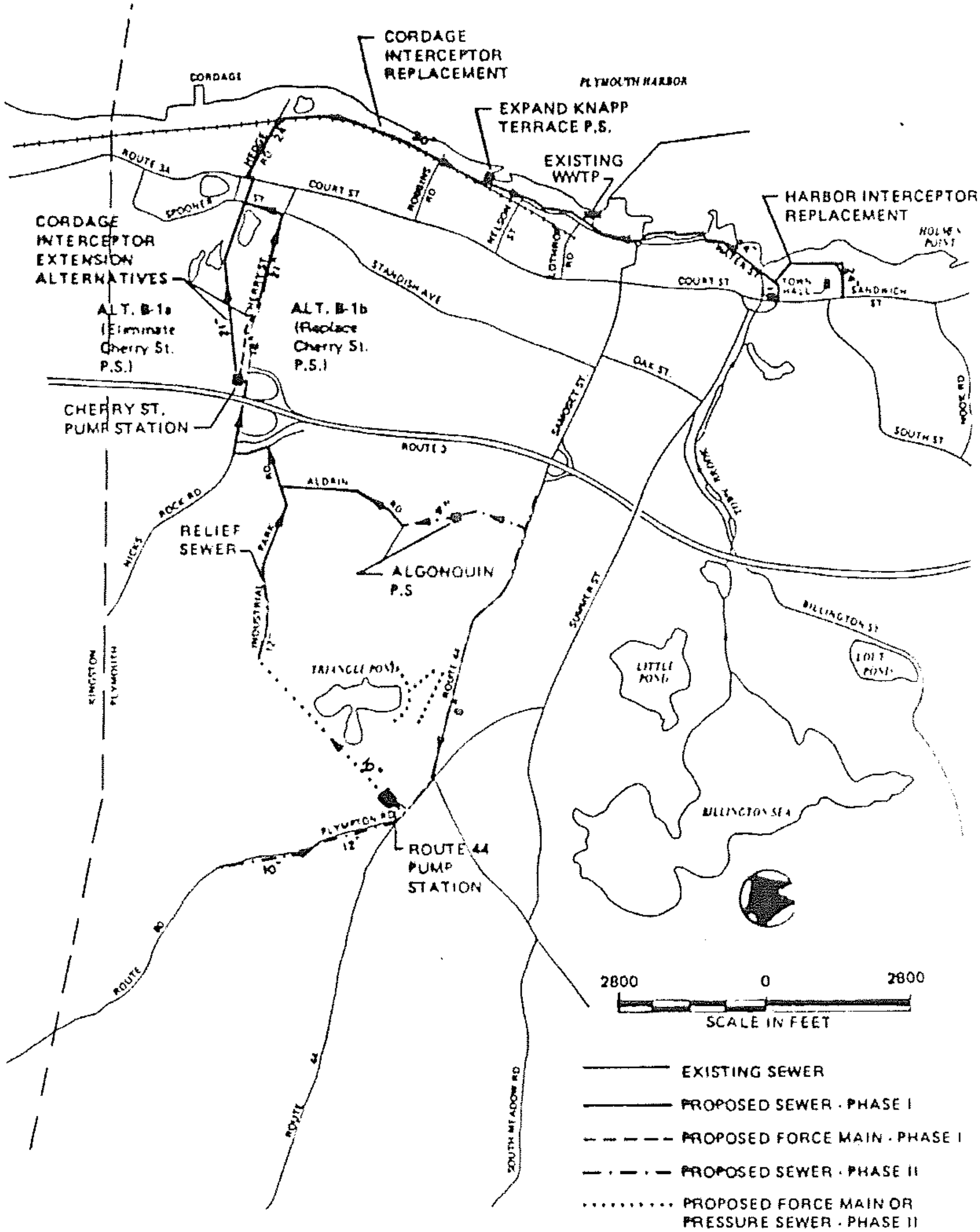

FIG. 6-7 DELIVERY SYSTEM ALTERNATIVE B-1

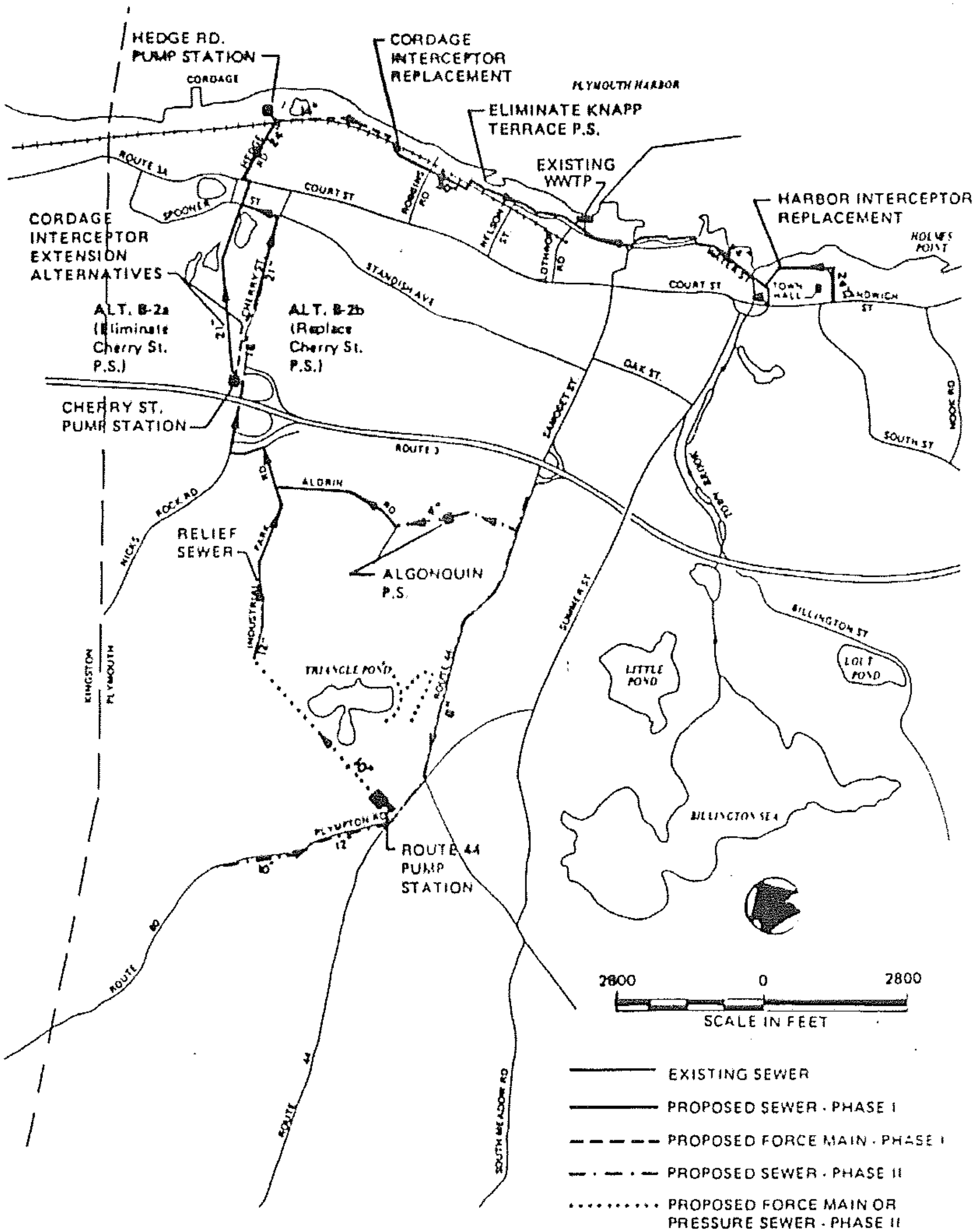

FIG. 6-8 DELIVERY SYSTEM ALTERNATIVE B-2

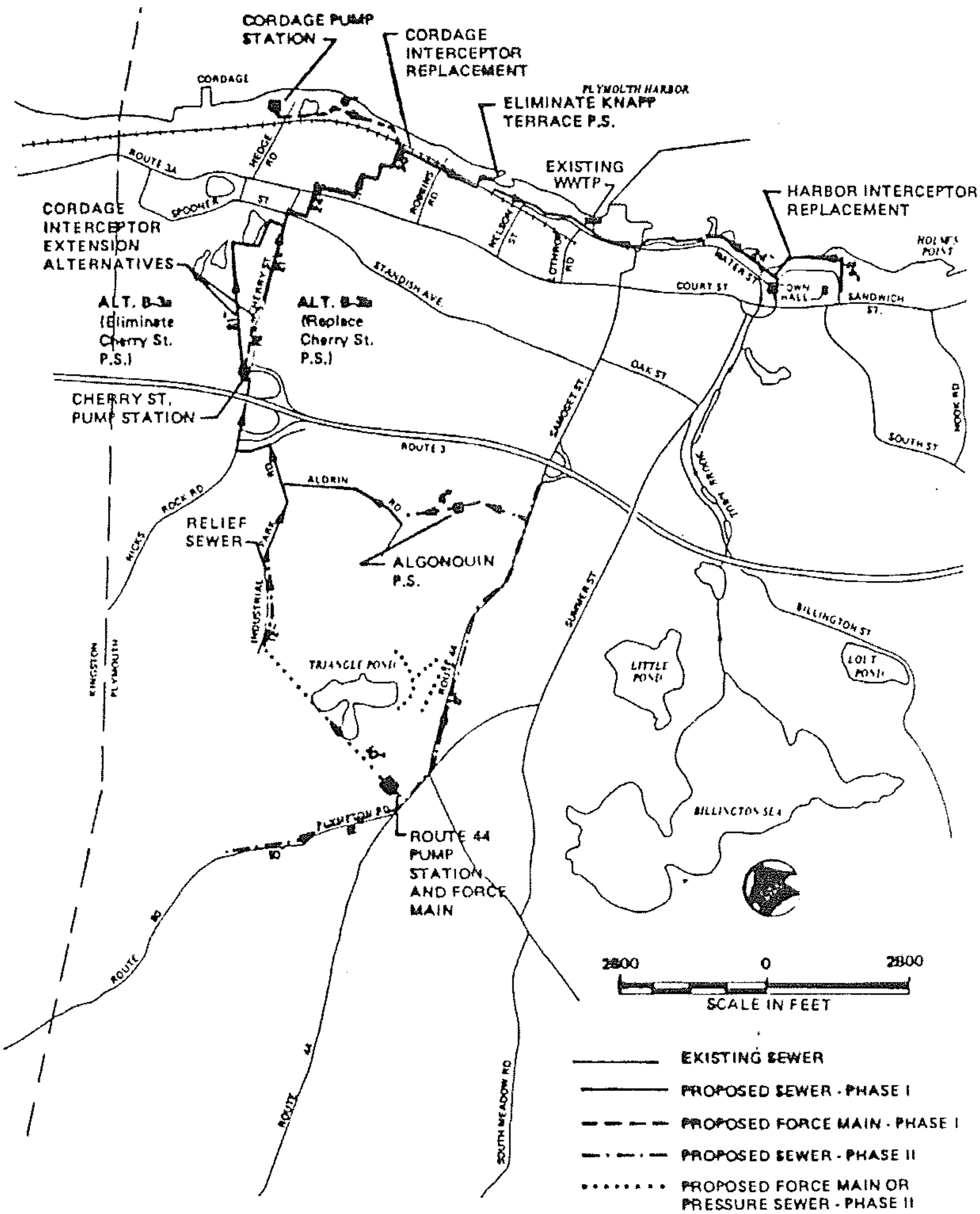

FIG. 6-9 DELIVERY SYSTEM ALTERNATIVE B-3

larger sewer lines: the portion of the Harbor Interceptor between State Pier and Sandwich Street and the 15" interceptor in Water Street from Union Street to Sandwich Street. Estimated costs for the various alternatives, including alternatives in which the service area is not expanded or only partially expanded, are presented in Tables 6-2 and 6-3. Under all alternatives, the existing Cherry Street Pump Station would either be eliminated or replaced, as the capacity of this station is inadequate to handle the projected increases in flow attributable to the expanded service area. Table 6-2 presents costs for alternatives in which the existing Cherry Street Pump Station is eliminated and replaced with a gravity sewer which would be constructed alongside of the streambed which runs between the existing pump station and Sawmill Pond. Table 6-3 presents costs for alternatives in which the existing Cherry Street Pump Station is replaced with a larger pump station or rehabilitated.

The lowest cost alternative delivery system including the servicing of the Rte. 44 Service Area is Alternative B-3a. Under this alternative, which eliminates the existing Cordage Interceptor, Knapp Terrace Pump Station and Cherry Street Pump Station, new interceptors would be constructed along Route 44, and new pump stations would be constructed at the intersection of Routes 44 and 80 and in the Arterial Commercial Area. New force mains would be provided to convey wastewater from Route 44 to Industrial Park Road. From there, wastewater would flow by

TABLE 6-7. COMPARISON OF COSTS OF ALTERNATIVE COLLECTION AND DELIVERY(1) SYSTEM IMPROVEMENTS FOR NORTH AND WEST PLYMOUTH ASSUMING ELIMINATION OF CHERRY STREET PUMPING STATION

	Alternatives Including Severing of Route 44 Service Area						No Severing of Route 44 Service Area – Replacement and Rehabilitation in Existing Service Area and Expansion of Industrial Park Service Area Only
	Service through New Samoset Street Interceptor			Service through Cordage Interceptor			
	Delivery System Alt. A-1a	Delivery System Alt. A-2a	Delivery System Alt. A-3a	Delivery System Alt. B-1a	Delivery System Alt. B-2a	Delivery System Alt. B-3a	
Capital Costs							
Route 44 and Industrial Park Area Pump stations, interceptors and force mains	$3,036,000	$3,036,000	$3,036,000	$2,487,000	$2,487,000	$2,487,000	$ 0
Lateral sewers	283,000	283,000	283,000	283,000	283,000	283,000	0
Subtotal	$3,319,000	$3,319,000	$3,319,000	$2,770,000	$2,770,000	$2,770,000	$ 0
Replacement of Cordage Interceptor	$2,565,000	$2,245,000	$2,275,000	$1,261,000	$2,554,000	$2,412,000	$2,275,000
Replacement of portion of Harbor Interceptor	$ 704,000	$ 704,000	$ 704,000	$ 704,000	$ 704,000	$ 704,000	$ 704,000
Reduction of excessive I/I	467,000	467,000	467,000	467,000	467,000	467,000	467,000
Total capital cost	$7,055,000	$6,735,000	$6,765,000	$7,184,000	$6,495,000	$6,353,000	$3,446,000
Annual O&M Costs(2)							
Route 44 and Industrial Park	$ 9,000	$ 9,000	$ 9,000	$ 11,700	$ 11,700	$ 11,700	$ 0
Cordage Interceptor	-2,400	-2,400	-3,800	-3,000	0	-3,800	-3,800
Total annual O&M	$ 6,600	$ 6,600	$ 5,200	$ 8,700	$ 11,700	$ 7,900	$ -3,800
Present Worth Costs							
Capital cost	$7,055,000	$6,735,000	$6,765,000	$7,184,000	$6,495,000	$6,353,000	$3,446,000
Land acquisition	11,000	30,000	30,000	13,000	32,000	32,000	30,000
Interest during construction	556,000	530,000	532,000	565,000	511,000	500,000	271,000
P.W. of annual O&M cost	65,000	65,000	52,000	86,000	116,000	78,000	-18,000
Less P.W. of salvage value							
Structures & piping	-529,000	-494,000	-516,000	-512,000	-439,000	-455,000	-257,000
Land	-4,000	-12,000	-12,000	-5,000	-13,000	-13,000	-12,000
Total present worth	$7,154,000	$6,854,000	$6,851,000	$7,331,000	$6,702,000	$6,495,000	$3,440,000
Annualized cost	$ 722,000	$ 692,000	$ 691,000	$ 740,000	$ 676,000	$ 655,000	$ 347,000
RANKING	5	4	1	6	2	1	

1) The term "delivery system", as used in this table, includes pump stations, interceptors and force mains required to convey wastewater to the existing WPCF site.
2) Costs account for credits from eliminated facilities and are additive to present O&M costs.

TABLE 6-1. COMPARISON OF COSTS OF ALTERNATIVE COLLECTION AND DELIVERY(1) SYSTEM IMPROVEMENTS FOR NORTH AND WEST PLYMOUTH ASSUMING REPLACEMENT OR REHABILITATION OF CHERRY STREET PUMPING STATION

	Alternatives Including Sewering of Route 44 Service Area						No Sewering of Route 44 Service Area - Replacement and Rehabilitation in Existing Service Area and Expansion of Industrial Service Area Only
	Service through New Samoset Street Interceptor			Service through Cordage Interceptor			
	Delivery System Alt. A-1b	Delivery System Alt. A-2b	Delivery System Alt. A-3b	Delivery System Alt. B-1b	Delivery System Alt. B-2b	Delivery System Alt. B-3b	
Capital Costs							
Route 44 and Industrial Park Area Pump stations, interceptors and force mains	$3,036,000	$3,036,000	$3,036,000	$2,487,000	$2,487,000	$2,487,000	$ 0
Lateral sewers	283,000	283,000	283,000	283,000	283,000	283,000	0
Subtotal	$3,319,000	$3,319,000	$3,319,000	$2,770,000	$2,770,000	$2,770,000	$ 0
Replacement of Cordage Interceptor	$3,151,000	$2,831,000	$2,656,000	$3,927,000	$3,238,000	$2,874,000	$2,656,000
Replacement of portion of Harbor Interceptor	$ 704,000	$ 704,000	$ 704,000	$ 704,000	$ 704,000	$ 704,000	$ 704,000
Reduction of excessive I/I	467,000	467,000	467,000	467,000	467,000	467,000	467,000
Total capital cost	$7,641,000	$7,321,000	$7,146,000	$7,868,000	$7,179,000	$6,815,000	$3,827,000
Annual O&M Costs(2)							
Route 44 and Industrial Park	$ 9,000	$ 9,000	$ 9,000	$ 11,700	$ 11,700	$ 11,700	$ 0
Cordage Interceptor	3,700	3,700	2,300	7,000	10,000	6,200	2,300
Total annual O&M	$ 12,700	$ 12,700	$ 11,300	$ 18,700	$ 21,700	$ 17,900	$ 2,300
Present Worth Costs							
Capital cost	$7,641,000	$7,321,000	$7,146,000	$7,868,000	$7,179,000	$6,815,000	$3,827,000
Land acquisition	11,000	30,000	30,000	13,000	32,000	32,000	30,000
Interest during construction	602,000	577,000	562,000	620,000	565,000	537,000	301,000
P.W. of annual O&M cost	126,000	126,000	112,000	185,000	215,000	177,000	23,000
Less P.W. of salvage value							
Structures & piping	-554,000	-519,000	-521,000	-522,000	-449,000	-458,000	-267,000
Land	-4,000	-12,000	-12,000	-5,000	-11,000	-13,000	-12,000
Total present worth	$7,822,000	$7,523,000	$7,317,000	$8,159,000	$7,529,000	$7,090,000	$3,907,000
Annualized cost	$ 789,000	$ 759,000	$ 738,000	$ 823,000	$ 760,000	$ 715,000	$ 394,000
RANKING	5	3	2	6	4	1	

1. The term "delivery system", as used in this table, includes pump stations, interceptors and force mains required to convey wastewater to the existing WPCP site.
2. Costs account for credits from eliminated facilities and are additive to present O&M costs.

gravity to the wastewater treatment plant. The existing Cherry Street package pump station, which would not have sufficient capacity to convey the increased flow, would be abandoned, and a new gravity interceptor ranging in size from 21" to 30" would be constructed between the Cherry Street and Knapp Terrace pump stations. A small package pump station would be constructed in the vicinity of Cordage Park and Hedge Road to serve Cordage Park and the area north of Hedges Pond. A new force main would connect to the new 30" interceptor constructed in the railroad right of way, replacing the old Cordage Interceptor. Although a 12-inch relief sewer would need to be constructed by the Town along the Industrial Park Road, all lateral sewers within the industrial park and within the mobile home park east of Triangle Pond would be provided by the users at their own expense. An evaluation of alternatives to collect wastewater from the residential development south of Triangle Pond showed that pressure sewers would be less costly than gravity sewers for this small area.

Alternative B-3a has the disadvantage of requiring the construction of a sewer in a sensitive area owned by the Conservation Commission and also in a floodplain. Because this is undesirable, it is recommended that Alternative B-3b (the lowest cost alternative incorporating the replacement of the Cherry Street Pump Station with a larger pump station) be adopted instead. Table 6-4 summarizes the pumping station design requirements for this alternative.

TABLE 6-4. PUMPING STATION DESIGN REQUIREMENTS FOR NORTH AND WEST PLYMOUTH DELIVERY SYSTEM ALTERNATIVE B-3b

	Design Pumping Rate, mgd		Total Head at Peak flow, feet
	avg.	peak	
Cordage Pump Station	0.5	1.2	100
Cherry St. Pumping Station	1.5	3.5	61
Route 44 Pump Station	0.29	2.0	91
Algonquin Pump Station	0.11	0.31	100

The final columns of Tables 6-2 and 6-3 provide estimates of the costs of collection system improvements which would be recommended in the event the Town decided to expand the industrial park service area but not to sewer the Route 44 service area. In addition, capital costs were estimated for collection system improvements which would be recommended in the event that no expansion of the system were provided for, as follows:

Replacement of Cordage Interceptor	$1,258,000
Replacement of a portion of the Harbor Interceptor	704,000
Reduction of excessive I/I	467,000
TOTAL	$2,429,000

Under this alternative, small increases in wastewater flow would occur as vacant land within the existing service area is developed, and additional costs would be incurred by the Town to rehabilitate the existing Cherry Street and Knapp Terrace Pump Stations during the 1990's as equipment in these facilities passes its design life and larger pumps are needed to handle increasing flow. Insignificant additional annual operating costs would be associated with this alternative.

To reduce the problem of rapid rag accumulation at the Holmes Point Pump Station and eliminate the odor problems at the Night Soil Disposal Facility, it is recommended that the latter facility be modified or eliminated. If a new wastewater treatment plant were to be built, the facility could be eliminated and replaced with a new facility at the new plant site. If the existing plant is retained, the facility should be modified by the construction of a receiving structure which would totally enclose the septage trucks during unloading. This structure would be provided with a scrubber for odor control and improved grit and screenings removal equipment. The facility would remain unmanned. It is estimated that such a facility would have a capital cost of $255,000 and an annual operating cost of about $1600.

To reduce the problem of disposal of wastewater from boats in the harbor and in accordance with the 208 Plan, it is recommended that the Town encourage the private or municipal development of a marine pumpout facility linked to the Town sewer system.

All construction within the existing service area (i.e., the replacement of the Cordage and Harbor Interceptors, replacement or rehabilitation of the Cherry Street Pump Station, modification of the Night Soil Disposal Facility, and reduction of excessive I/I) would be completed during Phase I, and the construction along Route 44 and in the industrial park could proceed later under Phase II.

Billington Sea. Concern regarding the eutrophication of Billington Sea has been expressed by many of the Town's residents and officials. Although no correlation between on-lot systems and the eutrophication problem has been found, the Town may decide to take measures to ensure that no wastewater contamination occurs from on-lot systems immediately adjacent to the sea by providing sewers to serve those homes or by rearranging these systems to improve their treatment capabilities. If sewers were provided to serve the approximately 550 homes expected to be located near the upgradient (western) side of Billington Sea by the year 2007, the capacity to treat an additional 170,000 gpd average flow would have to be provided at the WWTP or in a neighborhood disposal system. The boundaries of the service area in question and its location with respect to the existing and proposed West Plymouth service areas are shown in Figure 6-10. The cost of providing sewers to serve these homes (exclusive of treatment and disposal) is estimated to be approximately $1.4 million. Alternatively, existing on-lot systems which are determined by further study (beyond the scope of this facilities plans) to be providing inadequate treatment due to insufficient depth to groundwater could be reconstructed to improve treatment. Either alternative may be eligible for Federal and State funding,

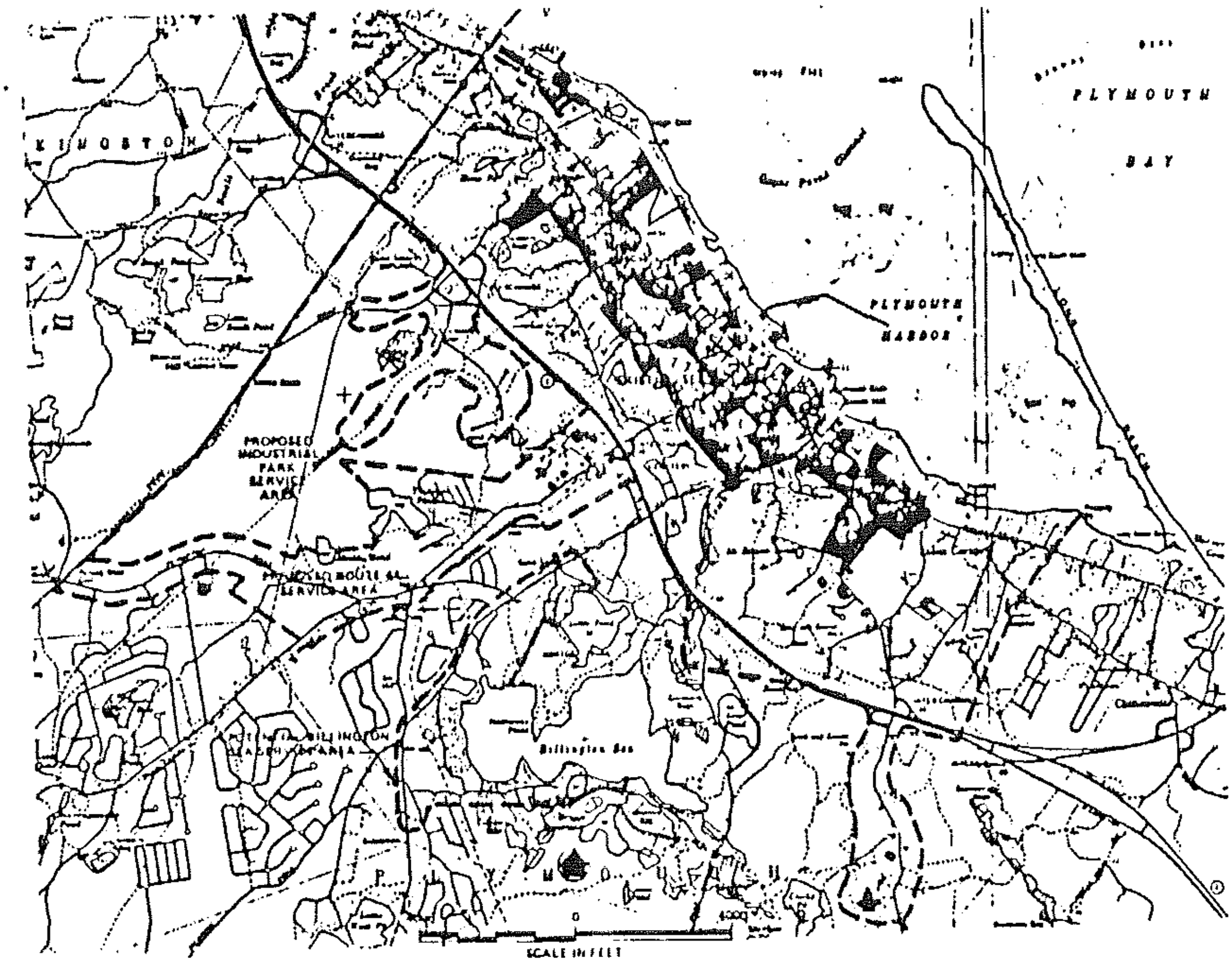

FIG. 6-10 POTENTIAL BILLINGTON SEA SERVICE AREA

but neither would be likely to be assigned a high ranking on the State's construction grants priority list due to the lack of evidence that wastewater-related problems exist in the area.

Other Areas. The homes located on Saquish Neck and Gurnet Point are not presently provided with Town water. Because of the remote location of the homes (access is by way of Duxbury), it is doubtful whether the Town of Plymouth will ever be able to supply them with public water. Two actions should be considered immediately for this neighborhood. The first item to be considered is an educational program to inform residents on how to maintain their disposal systems at maximum efficiency. Second, the installation of water-saving devices should be encouraged in all homes in this area that do not already have them.

Holding tanks or waterless toilets may have to be considered for undersized lots in this area, since Duxbury does not have a nearby sewer system and a regional sewer alternative is not available.

Although the 208 Plan recommended that sewers be provided out Route 3A to Warren's Cove and Chiltonville, the problems in the Warren's Cove Subarea were not found to be severe, and the lot sizes are generally large. Thus, it will be more cost effective to replace failed leaching areas than to provide sewers in this subarea. However, on-lot systems located along the Eel River may be contaminating the Eel River, as was noted in Chapter 4. If this is confirmed in the future, alternatives such as mound systems or a neighborhood system should be considered for this portion of the subarea. If Federal funding is still available when confirmation occurs, this work may be eligible for such funds.

In general, the remainder of the subareas did not have any serious wastewater-related problems that cannot be solved on site and that affect more than a few homes in one area. Educational programs on improved operation and maintenance of septic tanks and leaching areas are recommended for all homeowners with on-lot systems. In addition, the Town should adopt a program of water conservation, as recommended in the 208 Plan, "by educating the public to the need for such measures and adopting a bylaw mandating the installation of low flow devices on all new construction. A program of water conservation will not only aid in the operation of the wastewater treatment plant but will also help septic systems function better in other parts of Town."

Regional Considerations

Both the 208 Plan and the earlier South Shore Basin Plan prepared under Section 303(e) of PL 92-500 recommended that the Plymouth WWTP be expanded to accept flows from the Town of Kingston. During the recent preparation of the Kingston Facilities Plan Report, one of the alternatives considered for the disposal of wastewater from the Rocky Nook area of Kingston (shown in Figure 1-1) was the delivery to this wastewater of the Plymouth WWTP. Whitman and Howard, Inc, the engineers for the Town of Kingston, estimated that the average and peak wastewater flows from this area in the year 2005 would be 0.21 mgd and 0.72 mgd, respectively.

Regional alternatives such as this may involve various arrangements for construction, operation and management. They have several potential advantages, including savings in personnel, materials and supplies, higher operator skill levels, better performance of treatment, and fewer discharge points (which can reduce direct environmental impacts).

In a letter to the Kingston Citizens' Advisory Committee in December 1982, the Plymouth Board of Selectmen stated that "when discussing regionalizm, ... due to its geographic size, growing population, diverse land use, and environmental qualities, Plymouth should be considered a region of its own." They said they would not approve of sharing a facility with Kingston or any other town unless it was already designed to serve all of Plymouth for the future, and encouraged the regionalization of Kingston with other communities such as Duxbury and Marshfield.

Although the treatment and disposal of wastewater from Kingston was given consideration during the early stages of development of this facilities plan, this consideration was dropped with the concurrence of DEQE as a result of the position taken by the selectmen and a concurrent decision by the Town of Kingston that it preferred to construct its own treatment and disposal system.

Contingency Planning

While the overall wastewater management plan must be comprehensive in scope and explicit in purpose, it must retain the flexibility to handle long-term uncertainties. Although immediate needs can be reasonably defined, future needs are more speculative and are dependent on estimates of population growth, rate of development and expected failure rates of on-lot systems. Unanticipated growth and widespread failure of on-lot systems in additional areas represent the major long-term uncertainties in the proposed plan.

The wastewater management plan will provide for phased construction within the recommended service areas based on need, as already outlined. In the event that the needs of the community change over the 20-year planning period, the wastewater management plan must adapt to the new conditions. Phasing of construction will allow the resolution of immediate problems while spreading the tax burden over a longer period.

Conclusions

The conclusions developed in the foregoing analysis are:

1. Sewers should be provided in Section 1 of Manomet by a phased construction program which eliminates on-lot systems which pose the greatest public health threat during the first phase.
2. In the North and West Plymouth area, sewer construction should also follow a two-phased program. The Cordage and Harbor Interceptors and the Cherry Street

Pump Station would be replaced during the first phase of construction. Also during the first phase, excessive I/I would be reduced and efforts would be made to locate additional sources of wastewater contamination of storm drains which presently pollute the harbor, as discussed in Chapter 4. Construction of sewers to serve Route 44 would proceed under Phase II.

3. On-lot disposal systems should be retained in all other areas, where they are viable in the long term.
4. In order to increase the viability of existing and future on-lot systems, it is recommended that the Town take the following actions:
    a. Mandate the use of water-saving devices in all new houses throughout Plymouth and encourage their installation in existing houses in Saquish and Gurnet Point.
    b. Encourage or require improved O&M practices for septic tank and cesspool pumping townwide.
    c. Enact a special lakeshore protection bylaw to regulate lot size, summer house conversion, on-lot disposal systems, etc. so as to restrict development on the upgradient side of all lakes, as recommended in the 208 Plan.
    d. Encourage the private or municipal development of a marine pumpout facility linked to the Town sewer system.

CHAPTER 7

DESIGN CRITERIA FOR WASTEWATER FACILITIES

General

The design of wastewater facilities requires knowledge of projected wastewater flows, which reflect the land use and density of development within potential service areas and normally include domestic, commercial and industrial components. Allowances for infiltration and special flows, such as schools and other institutions, are also included in computing the design flow for wastewater facilities. Septage quantities must be developed from areas which are not sewered. The increase in wastewater quantities from the initial to the design year must also be estimated because it can have a significant impact on the system performance.

Period of Design

Because sewers and wastewater treatment facilities cannot be expanded with equal ease and economy, different design periods and correspondingly different quantities of wastewater are generally used in estimating capacity requirements.

Sewers are relatively permanent structures and are expensive to construct. The useful life of many pipe materials exceeds 50 years. Once constructed, additional capacity can only be added by replacing pipes with larger ones or by adding relief sewers. Significantly higher flows can be accommodated by a small increase in pipe diameter. Therefore, the cost of additional capacity would be higher if provided at a later date than if provided initially. For this reason, it is considered prudent to design sewers for the flows expected at least 40 to 50 years in the future or for near full development of the area delineated for sewer service.

By comparison, conventional pump stations and treatment facilities are more easily expanded and have mechanical equipment which has a shorter service life, usually 20 years or less. These facilities may be designed for smaller capacities than the incoming sewers with provisions made for either increase in capacity or complete replacement. In general, such facilities should be designed initially to handle flows expected 20 years beyond the date of installation.

Since it will take at least 3 or 4 years to design and construct any proposed wastewater facilities, 1987 has been designated as the initial year for major system expansion. The design year for pumping and treatment facilities has been established as 2007 (20 years), while the design year for sewers and appurtenances is 2037 (50 years).

Land Use

In order to estimate future wastewater flows, present and projected land uses have been considered. The Town's present zoning map and regulations have been used as a guide. Non-buildable land is excluded from consideration in estimating wastewater flows. These areas are sometimes referred to as "open space" land and are comprised mainly of designated flood plains or inland wetlands, State forests, conservation land, cemeteries, highway rights-of-way, ponds, rivers and/or steep sloped areas. It is assumed that developed land would remain in its present use.

Various sections of a community produce wastewater flow of lesser or greater magnitude per unit area due to different land uses. Land use criteria have been used for three categories, residential, commercial, and light industry, in order to develop wastewater flows from each classification.

For purposes of establishing future domestic wastewater flows, average residential densities per acre of future development have been used for each zoning classification. These densities are based on the following criteria:

1. Minimum lot size is assumed within each classification.
2. A percentage of available land is deducted for streets.
3. An average of 3 persons per housing unit is assumed.

Zoning classifications	Minimum lot size, square feet	Estimated persons per acre
R-25	25,000	7
R-20 SL	20,000	8
R-20 MD	-	15
R-20 MF	-	36

Population

Populations for Plymouth, as reported by the United States Census, are plotted on Figure 7-1, together with population projections for the design year 2007. While the Town has

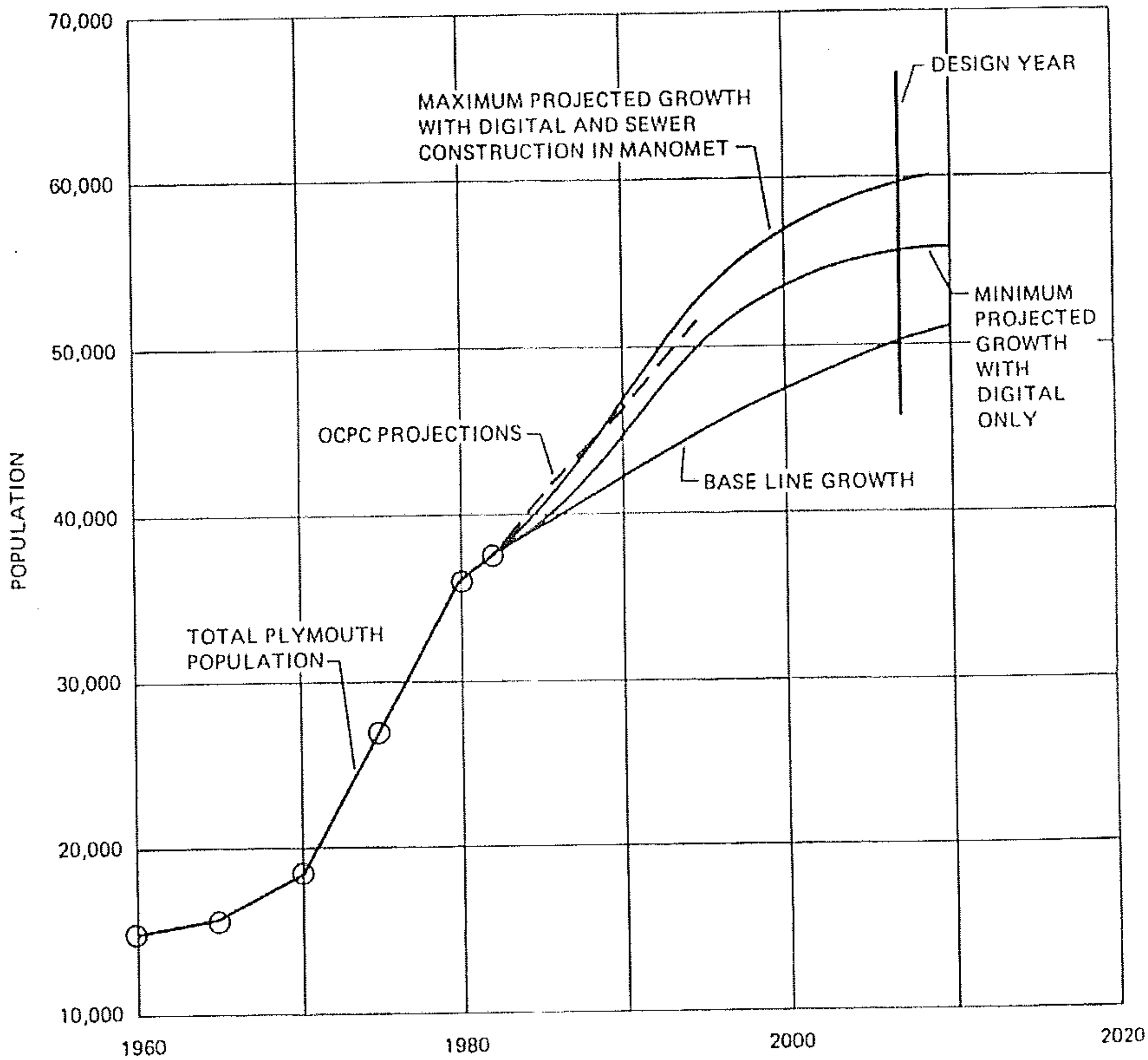

FIG. 7-1 PROJECTED POPULATION GROWTH - TOWN OF PLYMOUTH

experienced a rapid growth rate over the past 15 years, we have projected a slight decline in the rate of baseline growth for the future based on discussions with Plymouth's Planning Director and the Old Colony Planning Council. The reduction in the baseline growth rate reflects an expected gradual decline in the average number of building permits over the second half of the planning period due to an expected decrease in the supply of buildable lots and a dampening of growth-inducing forces anticipated in advance of the decommissioning of Boston Edison's Pilgrim I Nuclear Power Plant.

Incremental values of population growth were then added to the baseline growth rate to account for the direct and indirect impact of the development of the large Digital Equipment Corporation facility being planned for a 3,600-acre site in Central Plymouth between the Miles Standish State Forest and Manomet, and also for the likely conversions of many seasonal residences to year-round residences in the event that a portion of Manomet is sewered.

The following are our projections for future population in the Town of Plymouth:

	Manomet not sewered	Manomet partially sewered
1982	37,300	37,300
1987 (initial year)	41,200	41,600
2007 (design year)	57,000	58,500

Wastewater Design Flow Parameters and Components

Wastewater design flows consist of residential, commercial, industrial, special, and extraneous flow components. The extraneous component is essentially an allowance made for the quantity of groundwater infiltration and inflow that inadvertently enters the wastewater collection system.

Residential Wastewater Flow. The quantity of residential wastewater is directly related to the population served and to per capita water use. Therefore, estimates of population to be served and of future per capita water use are utilized for estimating this portion of the flow.

Most of the Town's water supply is generated from 7 wells, with the remaining portion drawn from the surface water impoundments of Little South Pond and Great South Pond. The Water Department records water pumpage (water production) and meters approximately 80 percent of its residential, commercial and industrial customers. Residential wastewater flow is closely related to residential water use.

The procedure for estimating annual average residential per capita water use (from water production data) is demonstrated in Table 7-1. A significant portion of the residential water use is used for lawn sprinkling, gardening, and car washing, and therefore is not included as wastewater flow. This volume for Plymouth can be estimated at about 20 to 25 percent of the

TABLE 7-1. ESTIMATE OF RESIDENTIAL PER CAPITA WASTEWATER FLOW (1981)

Average daily water production, in mgd		4.3
Less system losses through leakage plus unaccounted for water, etc. estimated at 30 percent, in mgd		1.3
Average daily water use, in mgd		3.0
Less major commercial, industrial and special use, in mgd		0.5
Net average daily residential[1] water use in mgd		2.5
Total population	37,300	
Population served by water system	28,230	
Residential per capita water use, in gcd[2]	89	
Estimated percentage of residential water use returned to sewers	75 %	
Residential per capita wastewater flow, in gcd	67	

1. Includes incidental amounts of commercial, industrial and special uses.
2. Gallons per capita per day

total. Accordingly, the estimated per capita residential wastewater flows are about 75 percent of estimated water use values, as shown in Table 7-1.

For design purpeses, we have assumed a small increase in per capita water use and therefore have adopted a per capita wastewater flow of 70 gcd. Although water-saving devices dicussed in Chapters 5 and 6 could significantly affect overall Town water use, their installation in existing homes would tend to be limited to those areas where on-lot systems would be retained and provide a motivation for their use. Thus, water-saving devices are not expected to significantly impact on the overall average per capita water use for those areas receiving sewers.

Commercial Wastewater Flow. It is estimated, based on Town water use records, that commercial and business establishments in North Plymouth produce an average of about 90,000 gallons of wastewater per day. These users are served by Town sewers and represent most of the Town's commercial activities except for those establishments in West Plymouth, Manomet, and Cedarville which are without public wastewater facilities.

Wastewater flow from future commercial areas is commonly estimated by assuming a daily gallon allowance per acre of commercial zoned or developed land. For this study, we have adopted a design allowance of 800 gad (gallons per acre per day) for average commercial flows. This allowance is based upon water consumption data and our experience in other similar communities.

Industrial Wastewater Flow. It is estimated, based on Town water use records, that industrial firms in North Plymouth and in the West Plymouth Industrial Park produce an average of about 220,000 gallons of wastewater per day. Industrial wastewater can be comprised of varying degrees of sanitary, industrial process and cooling water flows. In Plymouth, industries are of the "light" or "dry" category. This type of industry discharges very little, if any, process wastewater.

Unless present zoning is changed, future industries located within the Town will be of the "light" or "dry" category. For this reason, an average flow allowance of 1,000 gad for future industrial discharges has been made in this study. In the event a "wet" industry should locate in the area, special consideration would have to be given to providing sewerage and treatment plant capacity at the appropriate time.

For the purposes of this study, it has been assumed that all wastewater associated with the proposed Digital Equipment Corporation facility in Central Plymouth would be treated and disposed of within the boundaries of the Digital site and thus have no impact on the Town collection and treatment facilities.

Special Allowance Flow. Where appropriate, special allowances have been made for wastewater flows not classified in the previously-mentioned categories. For public schools, an allowance of 20 gcd has been used to estimate average flow.

For the Jordan Hospital and the Plymouth County Jail, we have used average wastewater flow allowances of 20,000 gpd and 10,000 gpd based on metered water use.

In addition, an allowance has been made to account for wastewater to be received from marine toilets. Recent laws requires each harbor vessel containing toilet facilities to be equipped with a Type II treatment and discharge unit or holding tanks, which would discharge to the sewer system by means of a boat pump-out facility to be constructed in the future. It has been assumed that 10,000 gpd would be generated by marine vessels in Plymouth Harbor.

Peak Flows. In addition to average wastewater flow, estimates of peak flow rates are required to properly design wastewater collection and treatment facilities. Variable peaking factors based on the cumulative average wastewater flow component (sum of domestic, commercial, industrial and special flow contributions) are applied to this component. This variation is attributable to the phenomenon that individual peak discharges from a small area have a higher probability of occurring at about the same time, whereas in a larger area nearby peak discharges pass a specific point sooner than discharges from more remote parts of the area. Another influence tending to produce smaller peaking factors for large areas is the hydraulic dampening effect on the peaks because of storage and the longer travel time in the collection system. Peaking factors used in this study vary between 2.5 and 3.4 in accordance with the curve shown in Figure 7-2.

Infiltration. Infiltration is groundwater which may enter the sewer system at pipe joints, building connections, or manholes. Therefore, the amount of infiltration depends upon the extent of the sewer system, the density of the dwellings, and the number of pipe joints, connections, and manholes. In addition, the quantity of infiltration also varies with the elevation of the groundwater relative to the sewer location, the closeness of watercourses, the nature of soil, and the other topographical

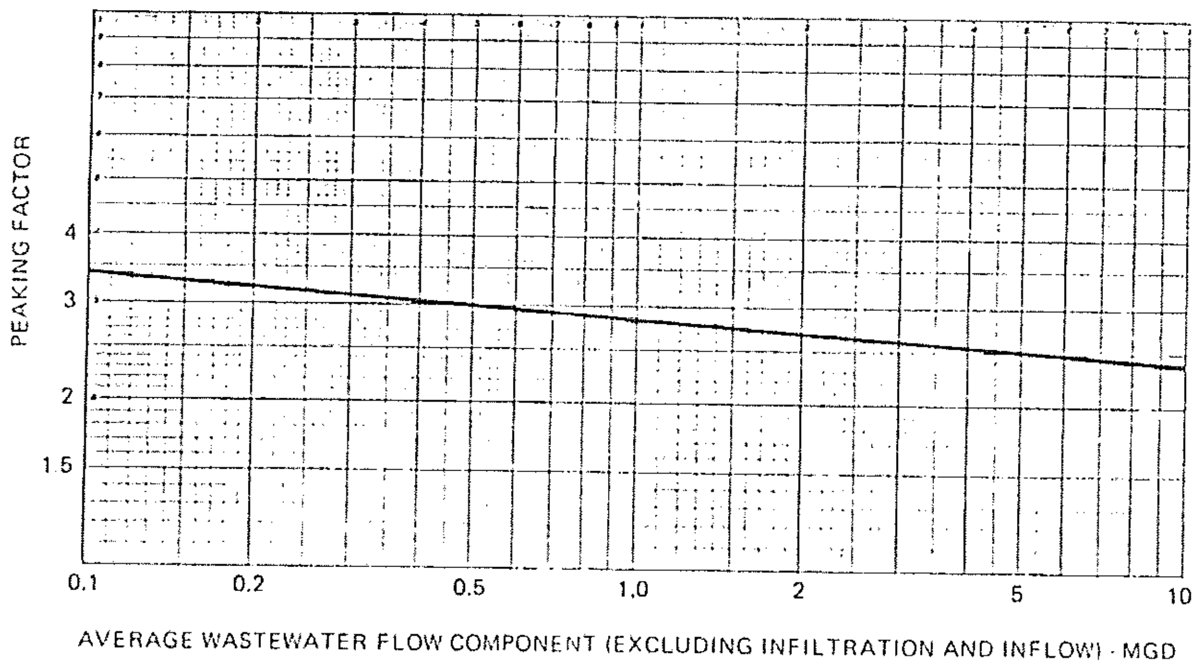

FIG. 7-2 PEAKING FACTOR FOR WASTEWATER FLOWS

features. Also, other factors being the same, higher unit rates of infiltration are usually found in older sewer systems. As the majority of Plymouth's sewers were installed before 1920, the Plymouth system experiences high rates of infiltration year round. On an annual average basis, existing infiltration represents nearly 56 percent of the total wastewater recorded at the treatment plant, and about 63 percent of the total flow is infiltration during periods of high groundwater (see Appendix J, I/I Analysis for further information).

Advances in pipe jointing systems and special connecting devices at manholes have resulted in significant reductions in rates of infiltration in new sewer systems. Based on the use of these improved systems in areas sewered in the future, we have adopted an average infiltration design flow allowance of 900 gallons per day per inch-mile of sewer (gpd/in-mile). This rate is equivalent to an infiltration rate of 300 gad from residential areas with lot sizes averaging between 20,000 and 25,000 square feet. The ratio of peak to average infiltration rates in new systems generally varies between 1.3 and 2.0. Accordingly, we have used a peaking factor of 1.7 for new sewer construction and a corresponding peak infiltration design flow allowance of about 1500 gpd/in-mile (500 gad).

Inflow. Inflow, for the most part, is stormwater runoff which enters the sewer system during storms and is characterized by an almost immediate increase in wastewater flow.* Possible sources are roof leaders, yard and areaway drains, manhole covers, cross connections from storm drains and catch basins, and combined sewers. New wastewater collection sewers are not designed to receive and convey inflow.

The Town's sewer system has an inflow problem. The treatment plant is capable at pumping a peak rate of 5.2 mgd, but during severe storms, the treatment plant receives more wastewater flow than it can handle, and partially-treated wastewater can be bypassed to Plymouth Harbor. The peak rate of inflow in the system is essentially unmeasurable. Based on plant records, the maximum 24-hour inflow during 1982 was estimated to be 1.6 mgd. This inflow occurred on a day when no bypassing was reported. Because only flow pumped through the plant is metered, it was not possible to estimate the peak hourly inflow rate. For the purpose of this report, the peak existing inflow rate has been assumed to be 1.6 mgd. This number may be refined upward during the course of the SSES work which is presently underway.

Because of problems resulting from inflow, it is probable that some degree of inflow reduction may be achieved in the future, possibly through the on-going EPA I/I and SSES Program. Based upon the I/I analysis included in Appendix J, it is assumed that the peak rate of inflow will be reduced to 1.4 mgd.

Septage. Records maintained on the night soil disposal facility indicate that the facility receives about 4 million gallons of septage annually, or about 11,000 gpd. Actual septage quantities are probably somewhat higher, as the septage haulers

* Another category of inflow produces a steady flow and includes cellar and foundation drains, cooling water discharges, and drains from springs and swampy areas.

are on the honor system to report their discharges. During non-winter months when septic tank cleanouts are more frequent, the average rate is about 16,000 gpd.

It is estimated that septage quantities will average 18,000 gallons per day and 24,000 gallons per day for the years 1987 and 2007, respectively.

Design of Sewers

General. To avoid surcharging sewers, which in turn could result in overflows from manholes or backups into basements, it is sound engineering practice to design pipelines to convey estimated peak rates of flow. Therefore, new sewers are designed on the basis of the peak hourly flow rate of all components including domestic, commercial and industrial wastewaters, plus unavoidable groundwater infiltration and any other miscellaneous sources such as schools or other institutional buildings.

Hydraulic Criteria. Capacities of new sewers are determined using a Manning's "n" (friction factor) value of 0.013. To provide for adequate cleaning and to minimize potential blockages, the minimum lateral sewer size is normally 8 inches in diameter. Building or service connections should be a minimum of 6 inches in diameter. To prevent deposition of solids, minimum velocities of 2 fps (feet per second) are desired in sewers flowing full. Velocities in excess of 10 to 15 fps can cause erosion or scouring of pipes and structures, and should be avoided. Under gravity flow conditions, velocities are a function of the slope of the pipeline. Recommended minimum slopes are presented in the following tabulation.

Sewer diameter, in.	Minimum slope, ft/ft
8	0.0040
10	0.0030
12	0.0022
15	0.0015
18	0.0012
21	0.0010
24 and larger	0.0008

When excessive deposition of material occurs in a section of the system, the opportunity for blockage increases. When this occurs, some method of scraping, cleaning or flushing is required. For preventive maintenance purposes, all sanitary sewers should be inspected and cleaned as necessary. This should be done at least once a year or when deposits are found to have accumulated.

<u>Depth of Sewers</u>. In general, it has been assumed for this study that new sewers constructed in Plymouth will be laid with the top of the pipe at least 7 feet below the street surface to insure that residences on the street will receive basement service without the need for individual wastewater pumps. Sewers located in rights-of-way or low areas could be constructed with only 4 feet of earth cover or enough to prevent damage from vehicular traffic and frost action.

Design of Pump Stations, Force Mains, and Pressure Sewers

General. It is usually not possible to connect all sewer service areas of a municipality by gravity flow without excessive construction costs. Pump stations, therefore, are constructed at strategic locations to pump wastewater into an adjoining service area for conveyance to the treatment facility. Pump stations are also used in smaller areas where gravity flow to an intercepting sewer is uneconomical. In this situation, small package pump stations or ejector stations are constructed to serve a limited number of establishments.

Ejector stations consist of metal pots (at least two) that are alternately allowed to fill and discharge. Discharge is accomplished by applying air pressure to the pot for a sufficient duration to empty the contents of the pot. Since small ejector stations have far less moving equipment than pumping stations, they are less costly to maintain and operate. Ejector stations are not normally used in those installations where capacity requirements exceed 200 gpm (gallons per minute) because pump stations are more economically provided for larger capacities.

In a basic pump facility, wastewater is collected at a low elevation and pumped through a pipe (force main) to a higher elevation. The type of facility and the sophistication of its equipment are dependent on the quantity of wastewater to be pumped and the degree of reliability required. Centrifugal pumps are generally used for municipal wastewater pumping.

Hydraulic Criteria. Wastewater pumps or ejectors are designed to handle the peak hourly flow generated in the design year of the facility. The pumps can be designed to operate either continuously or intermittently, depending on the expected flow range. The continuously operating pump is usually able to operate at various speeds to match the incoming flow rate. Pumps which operate intermittently do so at a relatively constant pumping rate. Associated piping and equipment are also sized for the design flow, although consideration is usually given to handling a future peak flow. As an example, pumps would be sized to handle the flow expected in 20 years, but in the sizing of suction and outlet piping of the station as well as the force main, flows expected in 40 to 50 years, or at saturation development, might be considered. However, it may not be possible to follow this procedure for long force mains in unsewered areas as flows may be too small for flushing. In such cases, two parallel force mains, smaller in size, may be necessary. The pump station structure is sized to contain any equipment which can be foreseen to handle flows from the future population of the service area. This could include the wet well in a pump station which would have to provide storage capacity for both design and future flows.

Force Mains and Pressure Sewers. Force mains and pressure sewers are sized to maintain velocities of approximately 3 fps. Friction losses are computed using an average Hazen-Williams "C"

TABLE 7-2. CONVENTIONAL TREATMENT DESIGN PARAMETERS AND COMPONENTS

Component	Primary design variables
Influent pumping and force main	Peak hourly flow
Aerated grit chamber	Peak hourly flow
Primary settling tanks	Peak hourly flow
Biological treatment units	BOD, average flow
Secondary settling tanks	Peak hourly flow
Flow equalization tanks	Average and peak hourly flow
Chlorination tank	Peak hourly flow
Effluent pumping and outfall	Peak hourly flow
Sludge processing and dewatering	SS, BOD

factor of 120. To prevent freezing and damage from surface loads, minimum cover of 5 feet is provided.

Design of Wastewater Treatment Facilities

Estimates of average, maximum, and minimum flows are required for the design of wastewater treatment facilities. In addition, the designer must have a characterization of wastewater components such as suspended solids (SS) and biochemical oxygen demand (BOD). Design of units within the treatment plant differ in the variables of primary significance as shown in Table 7-2.

The wastewater treatment facilities should be sized for the flow expected in the design year 2007. Provisions should also be made to allow the plant to be efficiently operated in the initial years. Design flow estimates for evaluating future needs at the existing North Plymouth and proposed Manomet wastewater treatment facilities are presented in Tables 7-3 and 7-4, respectively. Design loadings are presented in Appendix A.

Design of Outfall and Diffuser

Ocean disposal of the treated wastewater would be affected by a submerged pipe, called an outfall, in the harbor. Such an outfall consists of a pipe to transport the wastewater away from shore and a multiport diffuser section to dilute the wastewater with the receiving harbor water.

Outfalls are normally designed to provide larger diameter pipe and lower wastewater velocities than sewers because effluent wastewater does not contain settleable solids. The larger diameter minimizes friction losses which might otherwise require pumping to overcome.

Criteria for sizing the diffuser are as follows:

Manifold velocity at peak flow	$\leq$ 7 feet per second and $\geq$ 2 feet per second
Port discharge velocity at peak flow	15 feet per second
Minimum port diameter	5 inches

Design of Land Treatment and Disposal

The basic objective of a land treatment and disposal system is for the controlled application of wastewater onto land to achieve treatment and disposal of wastewater through natural physical, chemical and biological processes within the plant soil-water systems. Secondary objectives, such as economic

TABLE 7-3. DESIGN FLOWS FOR NORTH PLYMOUTH WASTEWATER TREATMENT FACILITIES

Description	Present (1982)	Initial (1987)	Design (2007)
Total town population	37,300	41,600	58,500
Per capita domestic wastewater flow	67 (1)	70	70
<u>Existing Service Area</u>			
Sewered population	11,240	11,950	13,740
Average annual flow, mgd			
Domestic	0.75	0.84	0.97
Commercial	0.09	0.19	0.26
Industrial	0.22	0.32	0.47
Infiltration	1.64	1.32	1.32
Special	0.0	0.10	0.15
Total	2.7	2.8	3.2
<u>West Plymouth Expansion (including industrial park and Rte 44 service areas)</u>			
Sewered population	-	1,380	2,750
Average annual flow mgd			
Domestic (Rte 44 S.A.)	-	0.10	0.19
Commercial (Rte 44 S.A)	-	0.09	0.14
Industrial (Industrial Park S.A.)	-	0.00	0.28
Infiltration/Inflow (Rt. 44 S.A.)	-	0.04	0.22
Total		0.2	0.8
<u>Combined Service Area</u>			
Sewered population	11,240	13,330	16,490
Average annual flow,mgd	2.7	3.0	4.0
Average monthly flow (wet weather), mgd	2.9	3.2	4.2
Peak hourly flow (dry weather), mgd	4.8	6.0	8.1
Peak hourly flow (wet weather), mgd	6.4 (2)	7.4 (3)	9.5 (3)

1. Includes incidental commercial and special flow.
2. Includes 1.6 mgd of existing peak inflow.
3. Includes 0.2± mgd of existing peak inflow removed.

TABLE 7-4 DESIGN FLOWS FOR MANOMET WASTEWATER TREATMENT FACILITIES

Schemes 1 and 3 (1200 lots)	Initial (1987) Summer	Initial (1987) Winter	Design (2007) Summer	Design (2007) Winter
Sewered population	3,600	2,000	3,600	2,000
Per capita domestic flow, gcd(1)	65	65	65	65
Domestic flow, mgd	0.23	0.13	0.23	0.13
Infiltration	0.10	0.16	0.12	0.20
Total average flow, mgd	0.33	0.29	0.35(1)	0.33
Peak hourly flow, mgd	0.83	0.59	0.85	0.63
Schemes 2, 4, 5, and 6 (600 lots)				
Sewered population	1,830	840	1,830	840
Per capita domestic flow, gcd(2)	65	65	65	65
Domestic flow, mgd	0.12	0.06	0.12	0.06
Infiltration	0.04	0.07	0.06	0.09
Total average flow, mgd	0.16	0.13	0.18	0.15
Peak hourly flow, mgd	0.43	0.25	0.45	0.27

1. Future flow of 0.45 mgd should be provided for if expansion of sewer system to include Manomet Village is planned (see Table A-2 in Appendix A).
2. Includes incidental commercial and special flow.

return from use of the wastewater and its nutrients for the cultivation of crops, could be realized. Three major land treatment processes are slow rate irrigation, rapid infiltration and overland flow. These processes are discussed individually in Appendix D.

The fundamental design parameter used in this type of system is hydraulic loading, which is usually limited by the infiltration capacity of the soil or the nitrogen loading.

Some of the major design parameters and components are presented in Table 7-5.

TABLE 7-5. LAND TREATMENT AND DISPOSAL DESIGN PARAMETERS AND COMPONENTS

Description	Design Parameter
Type of system	Subsurface conditions (permeability and groundwater depth) and available land area
Application rate	1 in/week to 200 ft/yr depending on system selected and conditions
Wetted field area	Average hydraulic loading
Buffer area	200 feet minimum around perimeter
Storage area	Length of growing season and climate
Pretreatment	Screening with primary or secondary treatment

Design of On-lot Disposal Systems

Properly functioning on-lot disposal systems receive wastewater and provide treatment and disposal through the natural soil system. The design of an on-lot disposal system is based on a flow rate and the percolation rate of the receiving soil. As discussed in Chapter 4, design requirements are established by Title 5 of the State Environmental Code and supplemented by the rules and regulations of the Plymouth Board of Health. Design parameters in Plymouth are as follows:

Design Flow	110 gallons per day per bedroom (400 gallons per day minimum)
Septic tank volume	1000 gallons minimum
Leaching pit area	300 square feet minimum
Leaching trench area	400 square feet minimum

Leaching field area	600 square feet minimum
Soil percolation rate	20 minutes per inch minimum
Groundwater	4 feet below bottom of leaching area minimum

CHAPTER 8

ALTERNATIVES FOR EFFLUENT DISPOSAL

General

Ultimate disposal of treated wastewater effluents may be accomplished by dilution in receiving waters, by discharge onto the land, or by evaporation into the atmosphere. The latter alternative is only practical in desert areas.

The Town of Plymouth contains numerous ponds and small streams, including Town Brook, Eel River, Beaver Dam Brook, Indian Brook, and the Herring River. All of these streams come under the antidegradation clause of the Massachusetts Water Quality Standards (314 CMR4.04(2) as presented in Chapter 2 of this facilities plan). Because they presently receive no municipal discharge, they are protected from future discharges.

The Town of Plymouth thus has only two realistic wastewater disposal options - disposal to the ocean or disposal to the land. The alternatives considered and general conclusions are discussed below. Costs of these alternatives are dependent on alternative treatment plant locations and are thus presented together with treatment costs in Chapter 10.

Ocean Disposal

The coastal waters along Plymouth's shoreline have been assigned Class SA standards. Such waters are of the highest quality and are useful for contact recreation, fishing and shellfishing (see Chapter 2). Because of deficiencies at the wastewater treatment plant and the presence of high levels of fecal coliform bacteria in discharges from storm drains in Plymouth Harbor, the harbor presently falls short of SA quality (see Chapter 4 and also Appendix B). Samples collected by DEQE along the Manomet beaches have indicated that these areas also fall short of Class SA quality, while generally complying with Class SB standards (see Chapter 4). Further out into Plymouth Bay, the impacts of wastewater and stormwater discharges become negligible, and SA standards are presently being met.

In addition to the protection afforded by the Massachusetts Water Quality Standards, marine waters off the Plymouth coastline are protected by the Massachusetts Ocean Sanctuaries Act and Federal effluent quality standards, as discussed in Chapter 2. The Commissioner of the Department of Environmental Management has interpreted the Ocean Sanctuaries Act to mean that the only permissable rate of flow from any expanded Plymouth Wastewater Treatment Plant that would not violate the Ocean Sanctuaries Act would be a discharge that would not exceed the nominal design capacity of 1.75 mgd (see correspondence in Appendix K). In spite of this fact, several alternatives incorporating ocean discharges which would not comply with Ocean Sanctuaries Act were studied, in accordance with federal regulations which require the consideration of all feasible alternatives. The methodology and results of these studies are discussed in Appendix B of this report.

North Plymouth Wastewater. Wastewater collected in the existing North Plymouth service area is treated at the plant on the waterfront and discharged into Plymouth Harbor via an 1840-ft long, 30-inch diameter reinforced concrete outfall which was constructed in 1968 (see Figure 8-1). The outfall, which has no diffuser, discharges at a depth of 6.5 ft (measured from the elevation of mean low water to the centerline of the outfall).

To determine the effect on Plymouth Harbor of the projected increased flow of secondary-level effluent from the existing outfall as modified only by the addition of a diffuser to improve initial dilution, numerical modeling was conducted, as discussed in Appendix B. The modeling, which incorporated several conservative assumptions, indicated that Class SA standards would not be met at the boundary of the zone of initial dilution of this outfall alternative. However, numerical modeling of the same discharge at the other outfall locations (see Figure 8-1) indicated that compliance with Class SA standards would be anticipated at all other outfall locations under consideration (see Tables B-19 and B-20 of Appendix B). The other locations were all shown to be substantially better than the existing one, primarily because their discharge depths were each at least double that of the existing discharge, resulting in much higher initial dilutions.

Several of the alternative discharge locations cannot be given serious consideration for the following reasons:

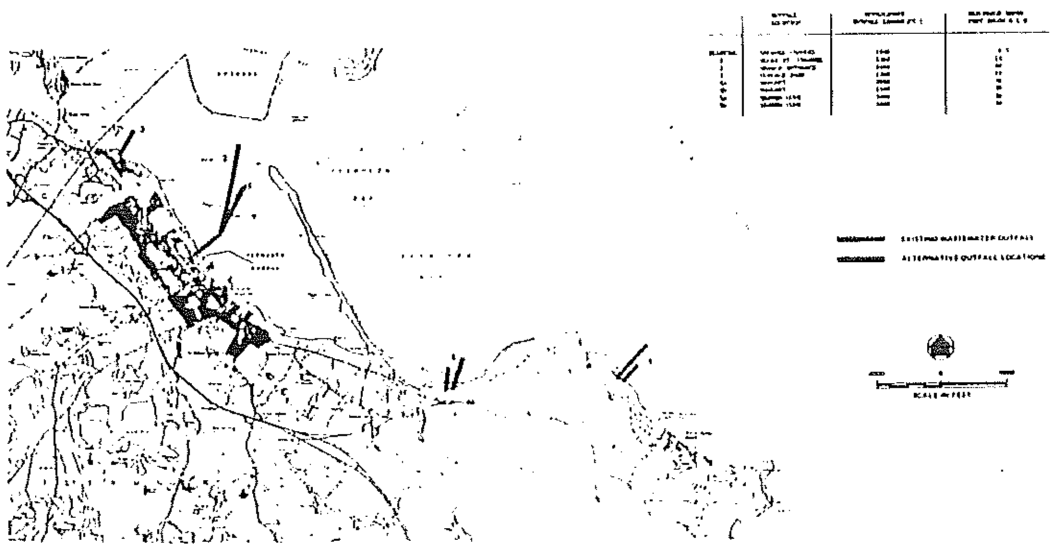

FIG. 8-1 ALTERNATIVE OUTFALL LOCATIONS

1. Location 3 (Cordage Park) would require the discharge to be located in a channel which has been dredged in the past. The presence of the outfall in the channel would limit future dredging which may be required to serve maritime dependent industries in the Cordage "port area." This location would also require the closure of a small shellfish area which is presently open.
2. Location 4 (Manomet) would require a costly 6 1/4-mile long pipeline to convey wastewater from North Plymouth. This alternative is thus not cost-effective (see Chapter 10). In addition, there was vocal opposition to <u>any</u> outfall off Manomet during the public participation process because of concern over the effects of such an outfall on the nearby beaches.
3. Location 5 (Warren Cove) would be perceived as a potential threat to Plymouth Beach, as particle path simulations described in Appendix B indicated the possibility of on-shore motion toward the beach. There is intense public concern over bathing beaches in Plymouth.

The remaining alternative discharge locations (1 and 2), shown on an aerial photograph of Plymouth Harbor in Figure 8-2, have lesser impacts, as discussed in Appendix B. Of these, Location 1 (Goose Point Channel) has the lowest cost and is therefore the recommended ocean discharge alternative for North Plymouth.

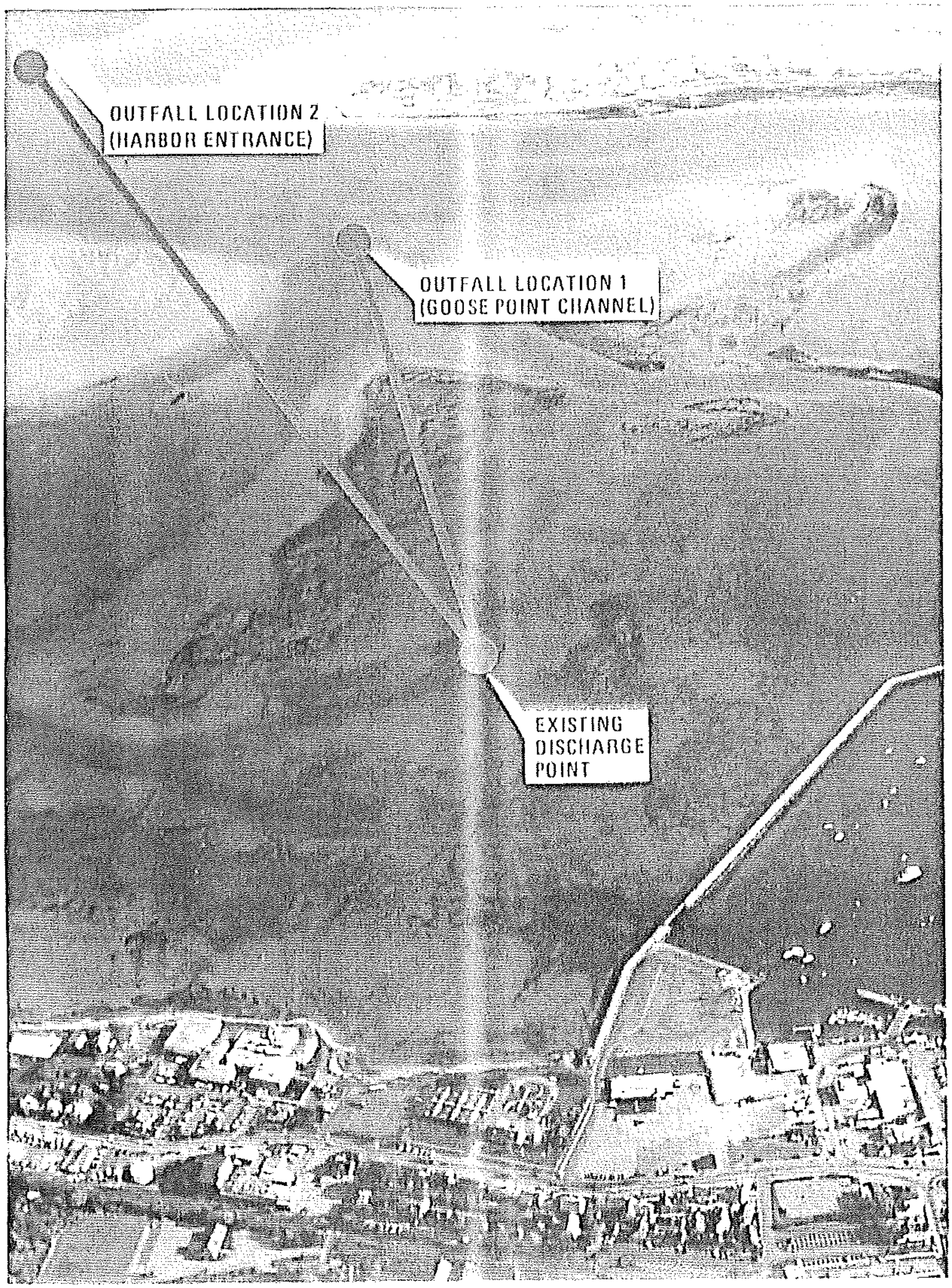

FIG. 8-2 ACCEPTABLE OUTFALL EXTENSIONS FOR NORTH PLYMOUTH

Manomet Wastewater. In the 1976 "Report to Manomet Sewage Disposal Study Committee on Proposed Manomet Sewerage System," it was proposed that a 1.25-mgd treatment facility be constructed in Manomet and discharge its treated effluent to a 1650-foot long 16" diameter outfall to be constructed off White Horse Beach. This proposal was not acted upon, and the proposed wastewater collection alternatives of the present study would result in the discharge of considerably less flow (0.35 mgd under Schemes 1 and 3 and 0.18 mgd under Schemes 2, 4, 5 and 6).

Two ocean outfall alternatives were considered for the current study, both at Location 4 south of the Pilgram Nuclear Power Station. This location was selected because of its proximity to the Edison Access Road treatment plant site, because the length of outfall necessary to reach deep water is minimized, because construction would not impact the beaches, and because prevailing currents there are away from the beaches. The first alternative would discharge at a depth of 20 feet (1100 feet long) and the second at a depth of 30 feet (2800 feet long). These locations are just beyond the northern extremity of a heavily-used bathing beach, and a clear showing of the unacceptability of ocean discharge anywhere in the Manomet area was evident during the public meetings. From a technical standpoint, however, numerical modeling showed that neither alternatives would cause degradation of water quality off Manomet, even in the event that the combined wastewater flow of North Plymouth and Manomet were discharged these (see Appendix B).

The issue became moot when costs of these alternatives were compared with those of lands disposal alternatives, as ocean disposal was not found to be cost-effective for Manomet (see Chapter 10).

Land Disposal

Land application is the oldest method used for treatment and disposal of wastes. Man has used land disposal of wastewater for over 2000 years, beginning in ancient Greek and Roman times. In more recent history, the use of wastewater effluents for the various forms of land application has been termed "sewage farming" and references to sewage farming in Europe as far back as the 1550's have been encountered. In the United States, sewage farms have been used since 1872.

Land treatment and disposal have been encouraged by Federal law since the adoption of the 1972 Clean Water Act, and must be considered in all facilities planning studies.

The alternative types of land disposal initially considered to be most applicable in Plymouth included slow-rate irrigation, rapid infiltration, and injection wells, each of which is discussed in Appendix D. It was determined that the rapid infiltration method of land disposal was most appropriate in Plymouth, and 33 potential sites were identified within the Town boundaries, as shown in Figure 8-2. The screening process used to identify those sites meriting field investigations is discussed in Appendix D. In general, sites located very much to the west of Route 3 were given low rankings because of the large pumping distance required to convey wastewater to these sites and because of a greater potential for degradation of the Plymouth aquifer should these sites be used (a large portion of the northern half of Plymouth that is west of Route 3 lies within recharge areas of existing and proposed public wells, as indicated by the aquifer protection district boundaries shown in Figure 8-3).

Five sites (A,E,F,G, and U) were initially selected for field investigations, and additional field testing was later conducted on two more sites (DD and LL). Detailed geotechnical evaluations including test pits, borings, and infiltration testing, were conducted on the two sites having the greatest potential to handle North Plymouth's wastewater (A and DD) and the two sites having the greatest potential to handle Manomet wastewater (U and LL).

North Plymouth. The projected design flow (maximum month) for the North Plymouth and West Plymouth service areas is 4.2 mgd. The Massachusetts DEQE has will not permit any rapid infiltration system to be loaded at a rate higher than 5 gallons per day per square foot, which is equivalent to an annual rate of 244 feet per year. Metcalf & Eddy recommends that an upper limit of 200 feet per year be used, assuming ideal soil conditions. A reduced maximum rate is applicable if the waste to be applied consists of primary rather than secondary effluent, as organic loadings should be kept under 100 pounds per acre per day. For North Plymouth wastes, assuming ideal soil conditions (i.e., highly permeable soils), Table 8-1 presents the minimum disposal area

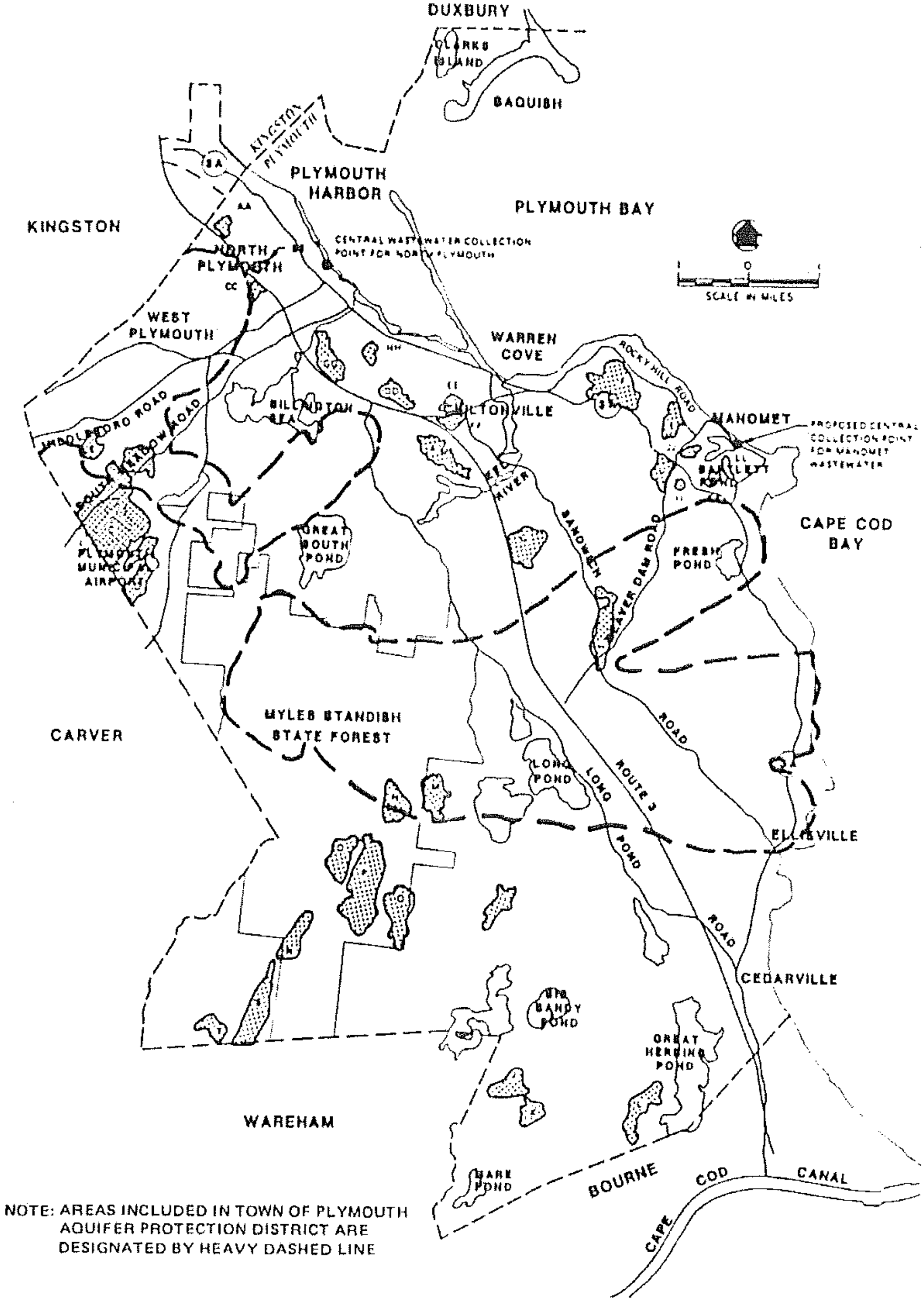

FIG. 8-3 POTENTIAL SITES FOR RAPID INFILTRATION

TABLE 8-1. MINIMUM AREA REQUIREMENTS FOR RAPID INFILTRATION OF NORTH PLYMOUTH WASTEWATER

Flow, mgd		
Annual average	2.25	4.0
Maximum month	2.45	4.2
Basin Area, acres		
Primary effluent	-	51
Secondary effluent	13	23.5
Basin Plus Buffer Area(1), acres		
Primary effluent	-	96(2)
Secondary effluent	31	46

(1) Includes a 200-ft buffer zone around entire site.
(2) Includes 5 acres for a treatment plant.

requirements for the alternatives under consideration. The 2.25 mgd flow was considered in conjunction with an alternative under which 1.75 mgd would continue to be discharged to Plymouth Harbor with the remainder going to the land.

Based on the site ranking process used in this study, Site A on Russell Mill Road was identified as the highest ranking site for North Plymouth.

Disposal layouts for both primary and secondary effluent was evaluated for Site A, as discussed in Appendix D. Under the secondary effluent alternative, treatment would be accomplished either at a new secondary treatment plant site or the existing WWTP site, and effluent would be transmitted to the disposal site. Under the primary effluent alternative, treatment would occur at a new plant located on Site A and the existing plant would be abandoned. Delivery system routes and cost estimates for these alternatives are presented in Chapter 10.

Although the EIS contractor found no serious adverse environmental impacts associated with the use of Site A for wastewater disposal, intense public opposition developed to the use of this site during the course of the study. The primary concerns expressed by the public were potential effects on the aquifer and the water quality of the Eel River system.

A second site on the County Farm (Site DD) was then evaluated, as discussed in Appendix D. Only secondary effluent was considered for disposal at this site due to its smaller size. Costs for this alternative were also estimated and found to be similar to those associated with Site A (see Chapter 10).

On January 3, 1984, the Plymouth County Commissioners voted to deny the use of land at the County Farm for wastewater disposal because the County "uses those parcels in question for agricultural purposes," plans "to expand the existing House of Corrections physical plant" at the site, "has some concerns over the proximity of the proposed disposal facilities to the densely populated House of Correction," and "has some concerns with being a good neighbor" to the Jordan Hospital and Plymouth-Carver Regional High School. This vote is documented in Appendix K.

Because the two most promising sites had been rejected and the fact that other potential sites were likely to be associated with more adverse environmental effects and higher costs than the ocean disposal alternative, the Plymouth Citizen's Advisory Committee voted on January 12, 1984 to recommend ocean disposal and drop further consideration of land disposal of North Plymouth wastewater.

Manomet. The projected design flows (maximum month) for the two Manomet sewering alternatives are 0.35 mgd (Schemes 1 and 3) and 0.18 mgd (Schemes 2,4,5, and 6). For Manomet wastes, assuming ideal soil conditions, Table 8-2 presents the minimum disposal area requirements for the alternatives under consideration.

TABLE 8-2. MINIMUM AREA REQUIREMENTS FOR RAPID INFILTRATION OF MANOMET WASTEWATER

Flow, mgd	0.18	0.35	0.45
Basin area, acres	1.0	2.0	2.5
Site area including preliminary treatment and buffer zone, acres	13	17	19

(1) Note that additional area is required at the site if future expansion of the sewer system to include the Manomet Village Center is to be accomodated.

Based on the site ranking process used in this study and preliminary field rating Site U at the corner of Beaver Dam Road and Route 3A was identified as the site to be given a detailed geotechnical evaluation. However, initial soil boring results and resistance of the property owner forced the abandonment of further consideration of this site.

After conducting additional preliminary field studies on three sites, further detailed evaluations were made of the site on the Edison Access Road (Site E), as discussed in Appendix D. With this site alternative, wastewater from either collection system alternative would be pumped to the disposal site. Delivery system routes and cost estimates for these alternatives are presented in Chapter 10.

No serious adverse environmental impacts were identified as being associated with the use of Site E for wastewater disposal, and little if any public opposition to this site arose during the public meetings. Boston Edison, the owner of the property, did not object to "serious consideration" of this site, but did state that "there may be circumstances beyond Edison's control that may preclude the use of" the site (see their letter of November 15, 1982 in Appendix K).

Under new regulations effective October 15, 1983, any facility which discharges over 15,000 gallons per day to the groundwater must apply to the DWPC for a permit (314 CMR 5.09). All groundwaters in the State are to be classified Class I (potable water supply source), Class II (saline waters) or Class III (nonpotable waters). Discharges of wastewater in Class I or II groundwaters must comply with stringent standards, including secondary treatment with partial nitrogen removal (314 CMR 5.10.3-4). If the Edison site is to be used, the Town should apply to the DWPC to have the groundwater downgradient from this site classified Class III using the procedures required in 314 CMR 6.04 and 6.05.

CHAPTER 9

WASTEWATER TREATMENT PROCESS ALTERNATIVES AND COSTS

General

The degree of wastewater treatment which will be provided in Plymouth and Manomet will be dependent upon the means selected for final effluent disposal.

If disposal of wastewater is to be to the ocean, secondary treatment will be required at any of the discharge locations considered. This requirement is a condition on the present NPDES permit for the North Plymouth Plant, and the numerical modeling conducted during this study confirmed the fact that a higher degree of treatment is not necessary (see Appendix B).

In the case of land disposal, either primary or secondary effluent may be applied to the land with similar effects on groundwater quality, as treatment to advanced levels will occur as the wastewater percolates through the unsaturated soil. Primary effluent may actually be preferable in cases where removal of nitrates is required to protect groundwater supplies, as it provides a carbon source for denitrification. The major adverse affects of applying a primary rather than secondary effluent to a rapid infiltration system are a larger area requirement (due to limitations on surface organic loading), a higher frequency of required disking of the surfaces of the infiltration basins, and the potential for increased odor generation due to the higher surface organic loadings. Advanced treatment is sometimes practiced prior to land disposal if the discharge is to a drinking water supply or a high degree of nutrient removal is necessary. The EIS contractor has not identified the need for high levels of nutrient removal prior to application onto any of the sites considered in detail during this study.

Wastewater Treatment Process Considerations

In evaluating treatment alternatives for wastewater, many factors must be considered. Among the most important of these considerations are: the ability to achieve the required effluent quality over a range of operating and climatic conditions, the capital (construction) cost and the O&M costs associated with the alternative, the reliability and ease of operation, the availability and suitability of land in the area, and finally, the environmental impact of the treatment facilities on the surrounding area.

However, no single item should dictate the selection of a process alternative. Every factor must be weighed to determine the overall favorability of each potential system. For example, a process which presents the lowest overall cost, but is considered unreliable, or difficult to operate, might be rejected in favor of an alternative which, though more costly, would more consistently meet Federal and State effluent standards. The selected alternative then should combine the characteristics of high reliability, relative ease of operation, and acceptable environmental impact, together with the lowest possible cost.

The wastewater treatment process alternatives considered to have application for wastewater treatment in North Plymouth and Manomet are

1. Primary settling alone (prior to land disposal only).
2. Extended aeration activated sludge.
3. Conventional activated sludge with or without primary treatment.
4. Rotating biological contactors.
5. Aerated facultative lagoons

Other methods of wastewater treatment which were initially screened include physical-chemical treatment and trickling filters. Both of these methods have been discarded. Physical-chemical processes for secondary treatment have proven to be very costly, both in construction costs and in annual operating costs. In addition, the complexity of operation and high degree of operator attention are factors contributing to the undesirability of physical-chemical treatment. Trickling filters, on the other hand, are quite simple to operate and require a minimum of operator attention. However, they generally result in large head losses through the plant requiring either intermediate pumping or very deep excavation. Both measures would be very costly. Perhaps the most important drawback to trickling filters is their difficulty in functioning efficiently during cold weather. The NPDES permit used for Plymouth requires year-round secondary treatment, making trickling filters a questionable choice for wintertime use without additional treatment.

The following paragraphs describe the treatment process alternatives considered to be most applicable in Plymouth:

Primary Settling. Primary settling is used to remove settleable and floating material from wastewater under quiescent conditions. Solids settling in the tanks are scraped to a hopper, and scum is skimmed from the surface by mechanical means. Generally, removals of BOD_5 and suspended solids are about 30 percent and 60 percent, respectively. Soluble pollutants are unaffected by primary treatment.

Primary settling by itself is generally not considered to provide an acceptable degree of treatment and would not be approved by Federal and State regulatory agencies for Plymouth except possibly in the case where effluent disposal would be to a land treatment system.

Extended Aeration Activated Sludge. This is the secondary treatment process currently used at the North Plymouth WWTP. It is most commonly used in plants having capacities of 2 mgd or less and would thus be more applicable to Manomet than North Plymouth in the future. The process is described in Chapter 3. A hydraulic detention time of 16 to 24 hours at design average flow is commonly provided. Generally, primary settling tanks are not used in conjunction with extended aeration systems to simplify sludge treatment and disposal.

Extended aeration yields an effluent low in BOD while producing a minimum of sludge, due to breakdown of microbial cells during endogenous respiration. The reduced sludge quantities minimize the size of sludge dewatering facilities as well as the costs of ultimate disposal. Also, the cost of conditioning chemicals for sludge dewatering are minimized. However, extended aeration requires a large aeration tank volume and secondary settling tank surface area. In addition, a relatively high aeration capacity is required, resulting in relatively high capital costs and electrical power costs.

Waste sludge from the extended aeration system is commonly pumped to an aerobic digester prior to dewatering, in order to further reduce and stabilize sludge solids and to thicken the sludge preparatory to dewatering.

Conventional Activated Sludge. This secondary treatment process is similar to the extended aeration process except the hydraulic detention time is much shorter (typically about 6 hours) and sludge production is higher due to the lower degree of endogeneous respiration associated with the reduced aeration time. The sludge can be concentrated by using flotation thickeners, in which air bubbles are used to carry sludge solids to the surface of the unit where they can be removed, thereby minimizing the required sludge dewatering capacity. Conventional activated sludge is generally more cost-effective than extended aeration for plants larger than 2 mgd.

Primary settling is normally provided ahead of conventional activated sludge systems to reduce the organic loading to the system.

Rotating Biological Contactors. Another secondary treatment alternative which has been considered for use at Plymouth uses rotating biological contactors (also called RBC's or biodiscs). The biodisc process is a "fixed film" process as opposed to a "suspended growth" process such as activated sludge.

The process utilizes large diameter corrugated plastic disks, mounted on a horizontal shaft, which rotate in a tank contoured to accommodate the disks. Submerged to about 40 percent of their surface area, the discs rotate slowly through the wastewater, alternately contacting the biomass on the disk with the wastewater and the atmosphere. The wastewater passes over the disks in a thin film and absorbs oxygen from the air. The microorganisms attached to the disk remove organics and dissolved oxygen from the thin film. The disk then rotates through the wastewater picking up more organics, completing the process. The adsorbed BOD is oxidized and synthesized by the bacteria, resulting in the oxidation products of carbon dioxide and water, and the production of new cells.

The shearing forces created by the constant rotation cause excess biomass growing on the disc to be sloughed off and carried downstream to settling tanks for removal. The solids generated by the biodisc process are not recycled, but are wasted to sludge processing via the primary settling tanks.

Unlike activated sludge systems, biodisc units are always enclosed by either prefabricated fiberglass covers or by a building.

The biodisc system requires less process control than does activated sludge, reducing laboratory analyses and manhours. The operating horsepower (used for the rotating drive) of the biodisc units is usually significantly less than that of activated sludge aeration systems. This results in lower electrical costs for biodisc systems.

The biodisc process produces sludge in similar quantities as the conventional activated sludge. The sludge produced is amenable to subsequent treatment and disposal.

Aerated Facultative Lagoons. Aerated lagoons are a means of biological treatment similar to activated sludge except that settling tanks are not normally provided and sludge is not normally recycled. The lagoons themselves have very long detention periods (10 to 20 days or more), require relatively large amounts of land, and thus are applicable only to relatively small plants. The term facultative means that two zones exist in the ponds -- an aerobic surface zone and an anaerobic bottom layer. Facultative lagoons do not have to be completely mixed and thus require less power than a true aerated lagoon. Aerated facultative lagoons can be designed to provide 70 to 85 percent BOD removal.

A lagoon is typically an earthern basin provided with an impermeable liner. Aeration is provided by diffused air or by mechanical aerators. If adequate depth is provided, sludge may be allowed to decompose anaerobically on the bottom, eliminating the need for sludge processing equipment or routine sludge disposal. Odors may occur for short periods during the spring when the lagoon experiences its thermal overturn.

The 208 Plan stated that lagooning appears to be very feasible of Manomet's wastewater prior to land disposal, and such facilities have generally been acceptable to the regulatory agencies.

Sludge Processing Considerations

The processing and disposal of sludge represents a major potential impact on the environment, as well as a significant cost factor of wastewater treatment. The presence of heavy metals and pathogenic organisms in sludges present the most cause for concern in the disposal of sludge. Other factors to be considered in selecting a method of sludge disposal are the high nitrogen content of sludge which can present problems to nearby surface and groundwater sources when sludge is applied to land, and odor problems resulting from poorly stabilized sludge or poor disposal methods. Many variables must be weighed in selecting a sludge processing alternative in order to minimize adverse environmental effects and to keep sludge processing costs as low as possible. Two factors which could limit available sludge management options in Plymouth are:

1. The size of the proposed treatment facility - sludge processing alternatives can be dictated by the magnitude of the sludge operation. For example, in plants on the scale of the proposed Plymouth facility, sludge incineration has not been found to be

economically feasible, unless land for sludge disposal is unavailable.

2. Present and proposed land use in the area - in an area where farming is a major land use, the use of sludge or compost as a soil conditioner or fertilizer may be indicated. However, in a nonfarming area such as Plymouth, there may not be sufficient demand for fertilizer or soil conditioners on a continuing and long-term basis.

Sludge Stabilization. Sludges removed from wastewater treatment processes can undergo anaerobic biological decomposition, causing odors and other problems. Prior to final disposal, the sludge must be treated to stabilize and inactivate the biomass.

The sludge stabilization alternatives considered in this study include lime stabilization, aerobic digestion, anaerobic digestion, and composting. At the request of the Citizen's Advisory Committee, consideration was also given to the Union Carbide Dual Digestion Process.

1. Lime Stabilization. Disposal of raw sludge to land is objectionable because of the highly putrescible nature of the sludge solids, the large number of pathogenic organisms, and the possibility of heavy metals in the sludge. However, it has been found that raising the pH of the sludge to 11 or 12 by the use of lime results in almost complete kill or inactivation of the microorganisms in the sludge. In addition to destroying pathogens in the sludge, the high pH also inactivates the

organisms which decompose the organic sludge solids and cause objectionable odor problems. The destruction of disease-causing organisms and the elimination of objectionable odors allows the sludge to be disposed of on land or in landfills. The high pH also serves to tie up any heavy metals in the sludge.

Lime stabilization may be practiced on either raw or dewatered sludge. In the former case, it is pumped to a chemical conditioning tank where the sludge is dosed with large quantities of hydrated lime. The raw sludge is mixed with approximately 20 to 30 percent lime (by weight) and with ferric chloride (a sludge conditioner added to improve dewatering) immediately prior to dewatering. Since lime is only slightly soluble in water, the addition of 30 percent lime to the sludge solids increases the total solids to be disposed of by almost 30 percent.

Sludge may also be stabilized with lime after it is dewatered. One method is the Roediger Process. Here, dewatered sludge cake is conveyed to equipment which mixes it with quick lime, creating a crumbly, stable substance that is easy to handle and dispose of. Stabilization occurs both because of the high pH and the pasteurizing effect of the heat caused by the exothermic reaction of quicklime with the sludge. Hydrated lime may also be used in conjunction with a pug mill to achieve post dewatering lime stabilization. Somewhat higher dosages of lime may be required with post dewatering lime stabilization than if lime is dosed before sludge dewatering.

2. Aerobic Digestion. Aerobic digestion is the stabilization technique currently used at the Plymouth WWTP. It is similar to the activated sludge process. The organic material in the waste activated sludge or mixture of primary and waste activated sludge is converted, under aerobic conditions, to carbon dioxide, water and nitrates. The process is carried out in an open, aerated tank having a detention period of 15-20 days. To date, aerobic digestion has been used primarily in small plants, particularly those using extended aeration or contact stabilization.

3. Anaerobic Digestion. In the anaerobic digestion process, the organic material in mixtures of primary settled and biological sludges is biologically converted under anaerobic conditions to methane (CH_4) and carbon dioxide (CO_2). The process is carried out in an gastight reactor. Sludges are introduced continuously or intermittently and retained in the reactor for varying periods of time. The stabilized sludge, which is withdrawn continuously or intermittently from the process, is nonputrescible, and its pathogen content is greatly reduced.

Two types of digesters are now in use: standard-rate and high-rate. In the standard-rate digestion process, the contents of the digester are usually unheated and unmixed. Detention times for this process vary from 30 to 60 days. In a high-rate digestion process, the contents of the digester are heated and completely mixed. The required detention time is typically about

15 days. A modification of these two basic processes is known as the two-stage process. The primary function of the second stage is to separate the digested solids from the supernatant liquor; however, additional digestion and gas production may occur.

4. Composting. Composting of raw dewatered sludge with wood chips has been evaluated for possible use in Plymouth. Composting can be defined as the thermophilic decomposition of organic wastes by aerobic organisms to produce a stable humus-like material. The humus, essentially free of pathogens and odor, can be used as a soil conditioner to increase the organic content of the soil, to increase the soil water retention, and to increase soil aeration.

Other advantages of composting include conservation of landfill space normally reserved for burial of sewage sludge and possible cost savings on peat moss and topsoil which might normally be applied to Town land. Composted sludge can also be applied to agricultural land and to lawns and gardens. As little farming is done in Plymouth, it is anticipated that the primary use of the compost material would be by homeowners. However, as these uses represent seasonal needs, it is evident that the composting operation must also be seasonal, or capacity must be provided to store the material during those months when there is no demand. The substantial capital investment required to initiate a composting operation would normally dictate that composting be carried on year-round with storage capacity provided for the low demand period.

Raw dewatered sludge from the treatment facility would be hauled to the compost site five days per week. At the site which would consist of a paved pad about two acres in area, the sludge would be mixed with wood chips and placed on the forced aeration compost piles.* Small air blowers would be used to draw air through the compost in order to maintain aerobic conditions throughout the piles. The forced air compost piles would be maintained for about three weeks. During that time, thermophilic conditions (temperatures > 55 deg C) would develop in the pile, which act to disinfect the sludge, as well as to facilitate the stabilization of the organic material in the sludge. Gases drawn from the pile by the blower during the composting reaction would be deodorized by passing them through piles of screened compost which tend to absorb most odors.

At the end of the three-week aeration period, the composted material would be placed in curing piles where it would remain for about a month. Curing allows additional composting to take place, and assures good pathogen kill through attrition and through continued thermophilic temperatures. After curing, the composted material would be spread out for drying and then be screened to remove and recycle the bulking material.

During wetter periods of the year, it would not be possible to dry the composted sludge sufficiently to permit

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* Wood chips are added as a bulking material to create air voids and to reduce the moisture content of the compost pile. This facilitates air passage through the pile and allow attainment of the high temperatures required in the composting process.

screening. Therefore, an area at the compost site would be provided to stockpile composted, unscreened sludge for 6 months. Runoff from the site would be collected in a storage pond for later conveyance to the wastewater treatment plant.

Public acceptance of the compost material is crucial to the success of the composting operation. If sufficient demand in the compost does not exist, excess material would require burial in the landfill thus negating the major advantage of the process. A well-coordinated effort by town government, citizens' groups, and the local media would be necessary to ensure an efficient composting operation. Sale of composted material could offset some of the costs of the operation, with the price depending on the level of public acceptance and resulting demand for the compost. Since public acceptance of such an operation is unknown, the assumption has been made in the cost analyses used in this facilities plan that the compost would be distributed at no charge.

5. Lotepro Dual Digestion Process. This proprietary process consists of a short detention time autothermal aerobic digester utilizing high-purity oxygen followed by an anaerobic digestion step having a detention time of at least eight days. Like other digestion processes, it produces a stabilized sludge. The manufacturer claims that the process yields greater net energy production than conventional anaerobic digestion at lower capital costs.

This process is still in the developmental stage, and has only been tested at one facility (in Hagerstown, Maryland). It is best suited to facilities using pure oxygen activated sludge systems. The process requires a thickened sludge feed to achieve autothermal aerobic digestion. This is a serious disadvantage at a small WWTP site such as Plymouth's, as it would result in a significant cost increase and a larger sludge handling building to house the thickeners. New tankage would also be required for both the aerobic and anaerobic digesters, and a pressurized liquid oxygen storage tank surrounded by a large clear safety zone would also be required. In addition, potentially explosive liquid oxygen would have to be trucked into the center of Plymouth. In view of these considerations, we do not feel that this process is appropriate for Plymouth.

Sludge Dewatering. The ultimate disposal of sludge is greatly facilitated by disposing of the driest sludge possible. Runoff and leachate problems are increased with liquid sludge, and adverse effects on groundwater and surface water sources can result. It is also less costly to transport a relatively dry sludge to a disposal site than to transport the same amount of solids inflated by the weight of more water. Means considered for dewatering sludge are sludge drying beds (Manomet only), vacuum filters, and belt-type filter presses.

1. Sludge Drying Beds. Sludge drying beds are a commonly used method for dewatering digested sludges. Their use is generally restricted to small plants due to significant land

requirements associated with required drying periods of at least 10 to 15 days. They are generally not suitable for use in proximity to residential areas. Dried cake solids of about 40 percent are typical.

Conventional drying beds have typically used a slow draining sand media, but an improved technique using stainless steel wedgewire panels as media has been shown to produce a shovelable sludge cake (about 15 percent solids) within a drying period of one or two days. This is possible because the wedgewire media prevents the migration of fine particles which cause media blinding. This is accomplished by controlling the pressure against the media by restricting the drainage rate.

2. Vacuum Filters. Vacuum filtration is the most widely used method of dewatering sludge by mechanical means. The process utilizes a rotating drum through which a vacuum is created. The drum is enclosed by a porous filter medium, usually cloth or coil springs, which retains sludge solids while the water in the sludge is pulled through the medium. This water, called filtrate, is returned to the treatment plant.

The drum is rotated through a vat of sludge. The sludge is conditioned with chemicals, usually ferric chloride and lime, to allow the solids to coagulate and release bound water. Conditioned sludge dewaters more readily and yields higher solids concentrations in the filter cake. As the drum emerges from the vat, the surface is covered with wet sludge. Air is drawn through the wet sludge by use of a vacuum pump, causing a negative pressure beneath the cake and extracting water from the sludge. The sludge cake is then removed before it reenters the vat by a knife edge, roller, or by gravity discharge. The cake is discharged directly onto a conveyor belt at about 18-22 percent solids. Filter cake is usually trucked to the point of final disposal.

3. <u>Continuous Belt-Type Filter Presses</u>. The belt-type filter press consists of two or more endless filter belts which are set in parallel at their interface. Belts may be woven of either metal or synthetic fibers to the desired mesh size or drainage characteristics. Compression rollers are set both above and below the interface of the filter belts. The vertical spacing between the compression rollers decreases from the beginning to the end of the compression cycle as the conditioned sludge is continuously fed between the two filter belts. Sludge passes through a gravity drainage zone followed by a compression zone and a shear zone. Filtrate is collected in a sump and returned to the wastewater flow upstream of the aeration tanks. The dewatered sludge is scraped off the belts by fixed blades where it is then collected on a conveyor system and discharged through a hopper onto a truck for conveyance to the disposal site.

Polymers are used for sludge conditioning. Lime stabilization may not be practiced ahead of a belt filter press, because polymers do not flocculate well at a pH of 12 and because lime scaling adversely affects the belt media and bearings. The sludge cake typically contains between 16 and 22 percent solids depending on the characteristics of the feed sludge.

Sludge Utilization and Disposal. Following the stabilization, chemical conditioning and dewatering processes, the sludge is considered suitable for land application. The methods of land application may be broadly classified as utilization and disposal. Utilization of sewage sludge is a recognition of sludge as a useful commodity. Sludge contains nitrogen, phosphorus, and organic material, and under proper conditions can serve as a soil conditioner and low grade fertilizer. Sludge may be used to increase the organic content of a sandy soil; it may be used to fertilize certain crops, or it may be used to reclaim unusable land such as strip-mined land.

Certain characteristics of sludge require that the impacts of its use on land be carefully considered. Heavy metals such as zinc, copper, lead, mercury, chromium, nickel, and cadmium can be found in varying amounts in most sludges, particularly in industrialized areas. In high enough concentrations, heavy metals may be toxic to plant life, and to animal life feeding on affected plants. Given the scarcity of process industry in Plymouth and the highly domestic nature of Plymouth's wastewater, the heavy metal content of the sludge should be relatively low.* However, even a low metal content in the sludge can be

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* Analyses of sludge and scum from the existing Plymouth WWTP, included in Appendix O, indicated levels of chromium and copper which would restrict the use of this sludge as a soil conditioner. It is expected that these levels will be reduced to acceptable levels in the future when sludge wasting rates are increased.

accumulated and concentrated in the soil system, and over a period of time build up to toxic levels.

The relatively high nitrogen content of sludge, generally 1.5 to 8 percent as total nitrogen, must also be considered, both for cropland use and for landfill disposal. In fact, nitrogen is most often the limiting element for soil application, not the heavy metals. Although almost all of the nitrogen in sludge is in the organic or ammonia forms, as water leaches out of the sludge, the ammonia carried with it is oxidized by nitrifying bacteria in the soil. The resulting nitrates, unless denitrified by soil bacteria or taken up by plants, can present a problem to nearby surface water and particularly groundwater sources.

There are a number of sludge disposal methods commonly employed throughout the country today. The majority of communities dispose of sludge to the land; however, some communities do discharge sludge to the ocean. The policy of the EPA is to prohibit additional ocean disposal of sludge; hence, ocean disposal is not considered further.

Disposal of sludge on land may be accomplished at various stages of sludge treatment. The principal methods used include:

a. spreading on land as a soil conditioner or fertilizer

b. landfill or burial

c. incineration followed by one of the preceding.

1. Using Sludge as a Soil Conditioner or Fertilizer. Spreading municipal wastewater sludge on land is both an acceptable and beneficial solution to the sludge disposal

problem, provided the sludge is given adequate pretreatment. There are a multitude of factors which affect the feasibility of its use on land. These include government regulations, economics, proximity of the land disposal areas to the treatment plant, ownership of the disposal areas, continued acceptance of sludge by landowners, proximity of the land disposal sites to densely populated areas and to surface or underground water supplies, method and frequency of applying sludge, presence of heavy metals in the sludge, assimilative capacity of the soil, fertilizer value, soil conditioner value, geography, topography, climate and geology.

Essential to long-term use of sludge as a soil conditioner is the continuing availability of land requiring soil improvement. In Plymouth, a non-agricultural area having cold winters, the need for soil conditioner has not been established and at best would be seasonal, requiring burial of sludge during periods of low demand. The distribution of sludge stabilized, say, by composting, at the landfill during high demand periods would have the advantage of increasing the life of the landfill.

2. Landfill Disposal of Sludge. Burial of sludge in a sanitary landfill is the most commonly used method of sludge disposal. It is generally the most economical method where sludge utilization is not feasible. Sludge is usually disposed of in a landfill by mixing with municipal refuse, compacting the mixture, and by covering the mixture daily with a suitable cover material. However, sludge-only landfills may also be used.

Sludge and scum from the existing Plymouth WWTP are presently trucked in the liquid form to drying beds located adjacent to the Manomet Landfill. After drying, the solids are mixed with refuse and buried in the landfill. As of January 1983, the 20-acre Manomet Landfill (which was opened in 1955) had an expected life of 4.9 years as a combined refuse/sludge landfill, according to the "Report on the Proposed Expansion and Operation of the Existing Landfill for the Town of Plymouth" by E.J. Flynn Engineers. The Town has negotiated a long-term contract with SEMASS, operators of a proposed 1500 ton/day resource recovery plant in Rochester, Massachusetts to take Plymouth's solid wastes. Refuse will be burned to produce electricity. If the SEMASS project goes forward, it could begin accepting Plymouth refuse as early as 1987, about the same time that the Manomet Landfill reaches its capacity as presently designed. SEMASS is presently unwilling to accept wastewater sludge at its facility because it would then be a co-disposal facility and subject to different regulations than a solid waste disposal facility.

The Manomet Landfill is the only active landfill in town and is the only landfill site at which continued landfilling is considered to be politically feasible in the future. Several possibilities exist for continuing use of the Manomet Landfill as a sludge-only landfill in the future, as follows:

1. The landfill could be expanded into the existing drying bed area, which will be abandoned. The area made available, however, would be less than two acres (see Figure 9-1).
2. The height of the existing landfill could be increased within limitations imposed by slope and terracing requirements.
3. The landfill could be expanded onto adjoining properties if the Town could acquire the land for this purpose. An area of about 5 acres should be adequate for the 20-year planning period.

The latter alternative is considered by the Town to be infeasible since the adjoining properties are owned by the Town's Conservation Commission and Digital Equipment Company (who recently purchased the land for its new industrial development in Plymouth).

It is estimated that 7-9 years of capacity is available in the existing drying bed area and another 7-9 years could be made available by increasing the design height of the existing landfill by 15 feet.

According to the "Policy on the Design and Operation of Sludge Landfills" as published by the Massachusetts DEQE on March 31, 1983, a liner and leachate collection system would be required on any virgin landfill area, such as the drying bed area, but no liner would be required for the portion of the landfill previously used for refuse and sludge disposal.

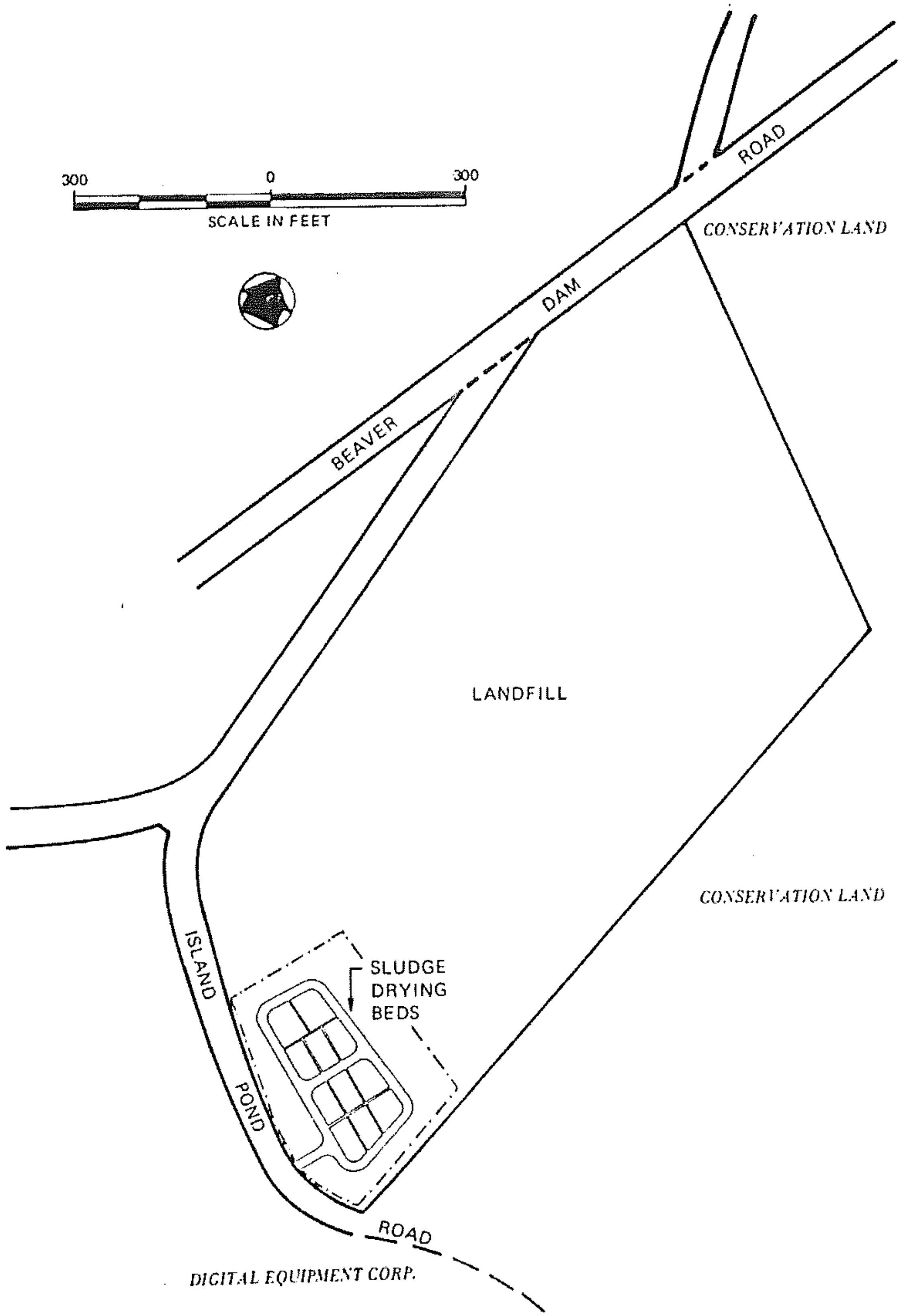


FIG. 9-1 MANOMET LANDFILL AND DRYING BED AREA

Incineration. Incineration of wastewater sludge may be viable where suitable land disposal sites are remote, and resulting transportation costs are excessive. Sludge is oxidized by use of multi-hearth incinerators or fluidized bed reactors, which are designed specifically to handle sludge. Incineration has proven to be economically unattractive for facilities on the scale of those proposed for Plymouth. Therefore, sludge incineration at the Plymouth treatment plant has been ruled out.

Expansion of Existing North Plymouth Plant

Because the existing Plymouth WWTP is only 15 years old and is centrally located within the collection system, serious consideration had to be given to continued use of the existing plant site in spite of the limited area available for expansion. The existing 3-acre property has only sufficient space to add two settling tanks and additional disinfection and effluent pumping equipment. The 1.2-acre lot adjacent to the plant site on the north side was purchased by the Town shortly after the WWTP was constructed for the purpose of providing space for future expansion. This property is still owned by the Town and is presently leased to Ocean Spray, Inc. for use as a parking lot for its visitor center. Figure 9-2 presents an aerial photograph of the site showing both the WWTP and the adjacent lot proposed fur use in expanding the plant.

During the early phases of the study, consideration was given to filling in the portion of the harbor immediately adjacent to the WWTP and expanding the WWTP onto the newly-created land. However, upon being advised by the DEQE that such

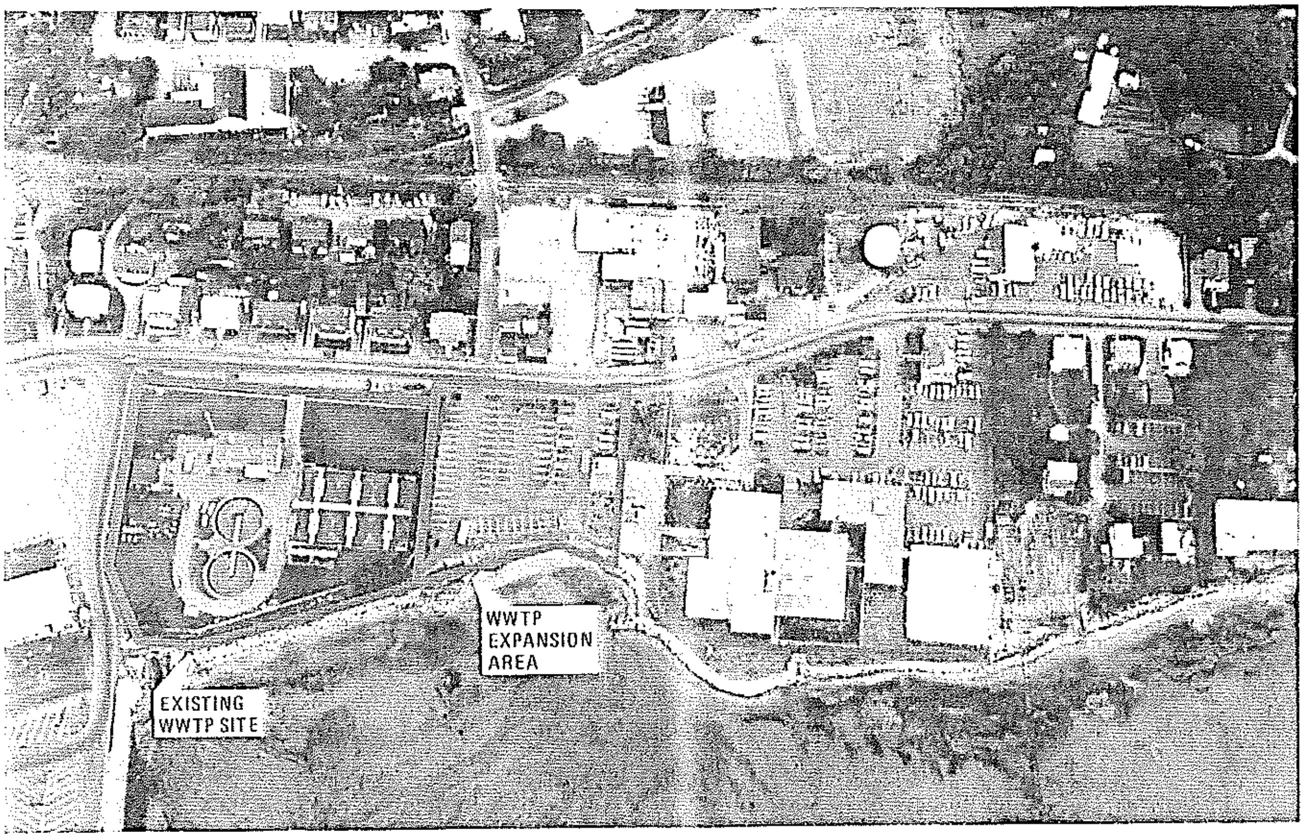


FIG. 9-2 EXISTING PLYMOUTH WWTP AND ADJACENT WATERFRONT PROPERTY

action is forbidden by the 1978 Wetlands Act unless "no feasible alternatives" exist, consideration of this alternative was dropped. On November 23, 1982, the Plymouth Board of Selectmen directed the CAC to assume that the Town property presently leased to Ocean Spray would be available for use by the WWTP in the event that this space were required for expansion of the plant.

Preliminary evaluation during this study revealed that the existing plant could be expanded up to a capacity of 5.0 mgd on the existing site if the adjacent lot were used. The proposed design year average flow of wastewater from North Plymouth is only 4.0 mgd, as shown in Table 7-3.

If the existing WWTP is expanded, the design will have to account for the artesian groundwater conditions on the site. Site dewatering costs during construction will be above average.

Several alternatives for wastewater treatment and sludge processing at the existing site were developed, as discussed in the following sections.

General Modifications. The following plant modifications are common to all the treatment alternatives considered:

1. Modifications would be made to the plant headworks, including replacement of the mechanically-cleaned bar rack, raw wastewater pumps and flow meter. Some structural and piping modification are also necessary to accommodate the increased flows.

2. The existing grit removal system would be replaced with enclosed aerated grit chambers. The enclosure would be furnished with an activated carbon odor control system. A vac-all truck will be provided for removing grit from the grit chamber.
3. The aeration tanks would be modified to increase flexibility in operating modes. The tanks would be compartmentalized to permit operation in the complete-mix, plug flow, step aeration and contact stabilization modes. New aerators would be installed in the tanks to replace the existing units.
4. Two new 55-ft. diameter final settling tanks and new sludge pumping equipment would be added to accommodate the increased flows. All four final settling tanks would be protected from the 100-year frequency flooding. The design elevation of the tops of the outer tank walls will be based on predictions of wave runup to be included in a Federal Emergency Management Administration study which is presently underway, and will be somewhere between Elevation 12.5 and Elevation 14.
5. Chlorination capacity would be increased to accommodate increased flows.
6. Chlorine contact tanks would be constructed to provide a minimum of 30 minutes of contact time (assuming discharge is to the ocean).

7. An effluent pumping station would be provided to permit the discharge of peak flows at extreme high tide (effluent pumping would not be required under normal flow and tide conditions).

8. A new 300-Kw emergency power generator would be provided to replace the existing 100-kw unit. The generator would be equipped with automatic start instrumentation.

9. A new sludge processing building would be provided to house dewatering and chemical feed equipment, a new laboratory, a maintenance shop, an office, and storage.

10. The existing aerobic digesters would be modified for use as sludge holding tanks with provisions for returning decanted supernatant to the aeration tanks. The existing surface aerators would be replaced with submerged turbine aerators to permit operation during the winter months.

11. The plant would be equipped with a drainage system to enable any of the tanks to be dewatered by gravity back to the influent pumping station.

Wastewater Treatment Alternatives. Three alternatives for wastewater treatment and sludge thickening were evaluated for use at the existing site. All three would make use of the existing aeration and settling basin tankage as follows:

Alternative P-1. Conventional Activated Sludge With Primary Settling.

Under this alternative, depicted schematically in Figure 9-3, wastewater would receive preliminary treatment (consisting of screening and a new grit removal system), would pass through new primary settling tanks where settleable organic solids would be removed, and then would enter the existing aeration tanks. The process is similar to the existing extended aeration process described in Chapter 3 with the following exceptions:

1. The presence of primary settling tanks reduces the concentration of BOD_5 and suspended solids in the wastewater entering the aeration tanks; reductions of 30 percent and 60 percent, respectively may be expected.
2. The detention time in the aeration tanks is much shorter than with extended aeration (7 hours versus 17 hours). This results in a lesser degree of biodegradation of the solids formed in the process (or, stated in another way, the production of more sludge per pound of organic material entering the process).

Under this alternative, waste activated sludge would be returned to the head of the plant for thickening and storage in the primary settling tanks. At design capacity, approximately 9000 lb/day of sludge solids would require dewatering under this alternative.

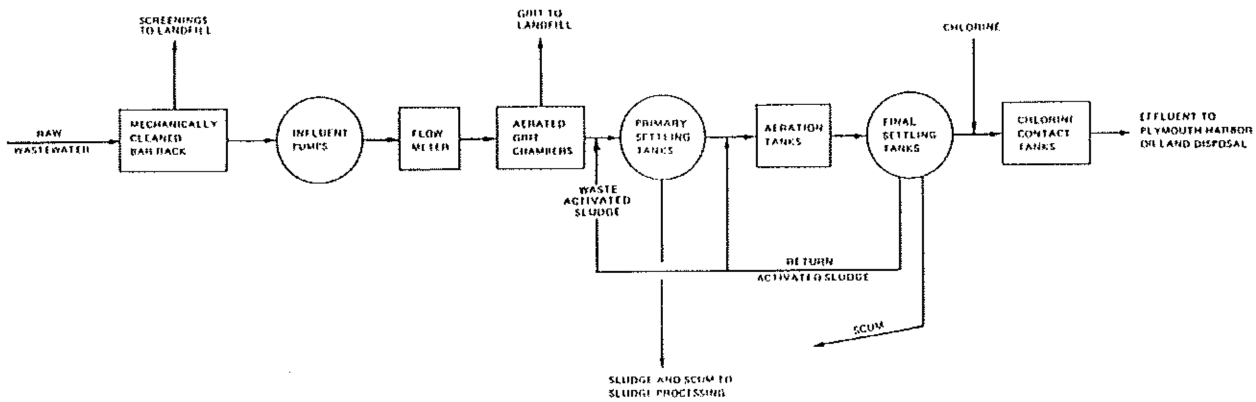


FIG. 9-3 TREATMENT ALTERNATIVE P-1
ACTIVATED SLUDGE WITH PRIMARY SETTLING

Alternative P-2. Conventional Activated Sludge Without Primary Settling

Under this alternative, depicted schematically in Figure 9-4, wastewater would receive preliminary treatment and then pass directly into the aeration tanks. The organic loadings on the aeration system would be higher than with the previous alternative (approximately 7000 lb/day versus 5000 lb/day BOD5).

While this is not an alternative that would normally be considered for a plant of 4-mgd capacity, it appears particularly applicable in this case because of the space limitations at the existing site, the relatively weak nature of the wastewater which is attributable to the high infiltration flow, the more than adequate aeration tankage available due to the fact that the original plant design used a lowly-loaded extended aeration system, and concern about odors from primary settling on a congested site.

Under this alternative, at design capacity, approximately 5500 pounds per day of unthickened waste activated sludge would require dewatering and disposal.

Alternative P-3. Conventional Activated Sludge Alone with Flotation Thickening

This alternative, depicted schematically in Figure 9-5, is similar to the previous one with one exception. Waste activated sludge would be pumped to flotation thickeners located in the new sludge processing building. Here, the

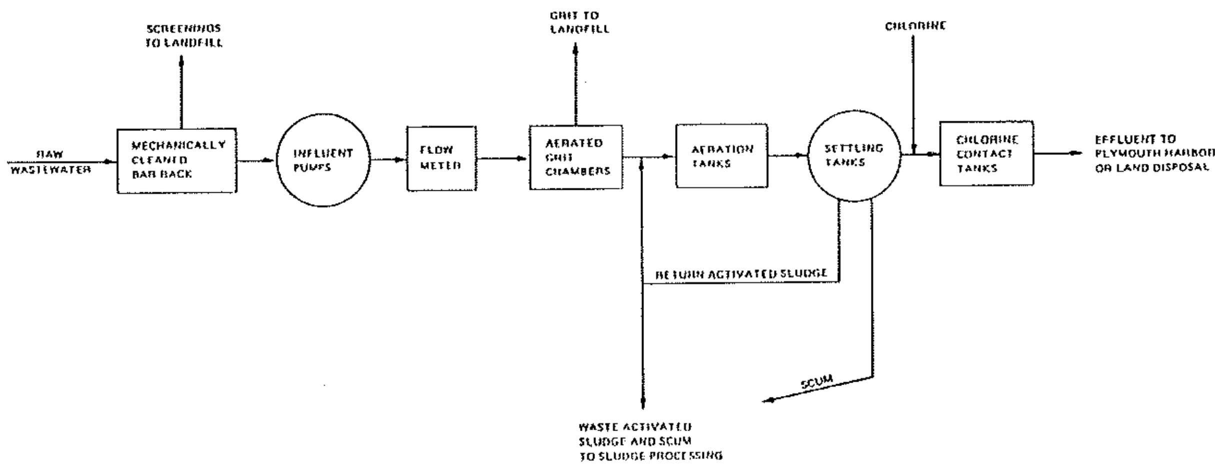


FIG. 9-4 TREATMENT ALTERNATIVE P-2
ACTIVATED SLUDGE WITHOUT PRIMARY SETTLING

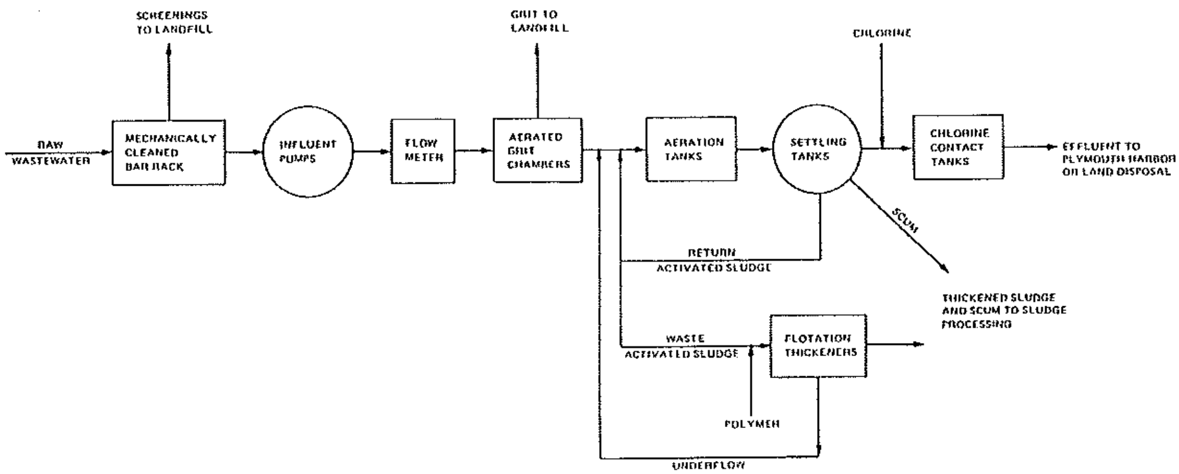


FIG. 9-5 TREATMENT ALTERNATIVE P-3
ACTIVATED SLUDGE ALONE WITH FLOTATION THICKENING

9-33

solids would be concentrated from 1.5% to 4% (measured on a dry-weight basis), reducing the capacity requirement for sludge dewatering equipment. About 5500 lb/day of sludge solids would require dewatering. However, a larger sludge processing building would be required under this alternative to house the thickeners.

Sludge Stabilization and Dewatering Alternatives. For each of the three wastewater treatment alternatives, three sludge stabilization and dewatering alternatives were evaluated. The alternatives considered were limited in number due to site constraints (e.g., anaerobic digestion was ruled out due to space limitations of the site). The existing method of stabilization, aerobic digestion, was not considered due to the increases in the capacity of the plant and in estimated sludge production, which would make its use uneconomical. The three alternatives considered are as follows:

a. Lime Stabilization, Vacuum Filtration and Landfilling

Under this alternative, depicted schematically in Figure 9-6, sludge and scum would be stored in the existing sludge holding tanks, mixed with lime and ferric chloride in conditioning tanks at a pH of 12 to provide lime stabilization, and dewatered on vacuum filters. The dewatered sludge would then be hauled to the Manomet Landfill for disposal.

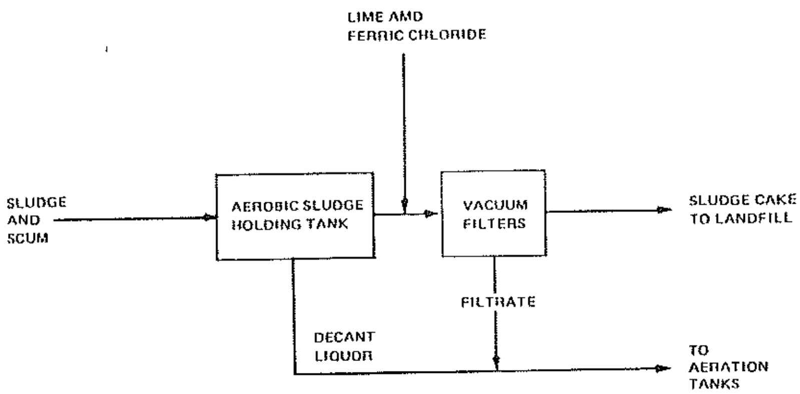


FIG. 9-6 SLUDGE STABILAZATION ALTERNATIVE P-a
LIME STABILIZATION WITH VACUUM FILTERS

b. Belt Filter Presses, Lime Stabilization and Landfill

Under this alternative, depicted schematically in Figure 9-7, sludge and scum would be conditioned with polymer in conditioning tanks and dewatered on belt filter presses. The dewatered sludge would then be stabilized with lime using the Roediger Process and hauled to the Manomet Landfill for disposal.

c. Belt Filter Presses Followed by Offsite Composting.

It was recommended in the 208 Plan that composting be considered for the stabilization and disposal of Plymouth's sludge. Under this alternative, depicted in Figure 9-8, sludge and scum would be stored in the existing sludge holding tanks, conditioned with polymer in conditioning tanks, and dewatered on belt filter presses. The dewatered sludge would be hauled to the composting facility (assumed to be located at the Manomet landfill). For cost estimating purposes it has been assumed that all compost produced would be disposed of at no cost to the Town through pickup by users.

Construction of a New Plant to Serve North Plymouth

Serious consideration has been given to abandoning the existing WWTP site in North Plymouth and relocating the plant on a new site. This alternative would have the advantage of making the existing site available for more appropriate uses. However,

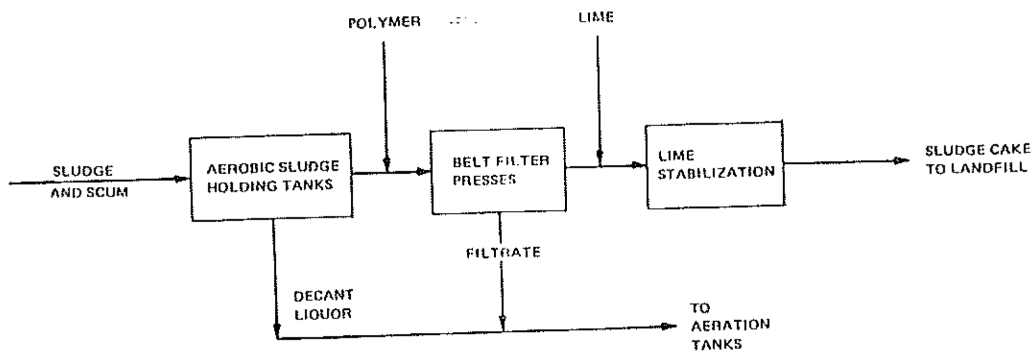


FIG. 9-7 SLUDGE STABILIZATION ALTERNATIVE P-b
BELT FILTER PRESSES WITH LIME STABILIZATION

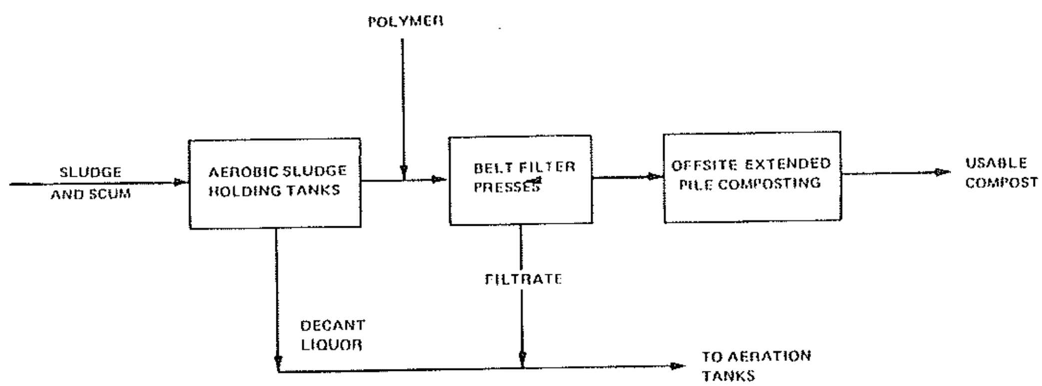


FIG. 9-8 SLUDGE STABILIZATION ALTERNATIVE P-c
BELT FILTER PRESSES WITH OFFSITE COMPOSTING

9-38

since the Town's sewers presently converge at the existing site, it would be necessary to pump the wastewater to any new site not located along the waterfront.

Several potential wastewater treatment plant sites have been considered in Chapter 10. The analysis in this chapter is limited to addressing treatment alternatives and their associated costs.

Wastewater Treatment Alternatives Considered. Three alternatives for wastewater treatment were evaluated for use at a new site. Two of these alternatives would provide secondary treatment and the third primary treatment. The latter alternative would only be considered in conjunction with land disposal of the effluent. For all alternatives, a new septage receiving facility consisting of covered, aerated holding tanks, pumping, screening, grit removal and odor control equipment would be located at the plant site.

Alternative N-1. Activated Sludge

Under this alternative, the flow pattern would be similar to that shown in Figure 9-2 except that influent pumping would not be necessary since the raw wastewater would be entering the plant under pressure from a pump station located at the existing plant site.

Alternative N-2. Rotating Biological Contactors

This alternative, depicted schematically in Figure 9-9, is similar to Alternative N-1 except that RBC's are used to

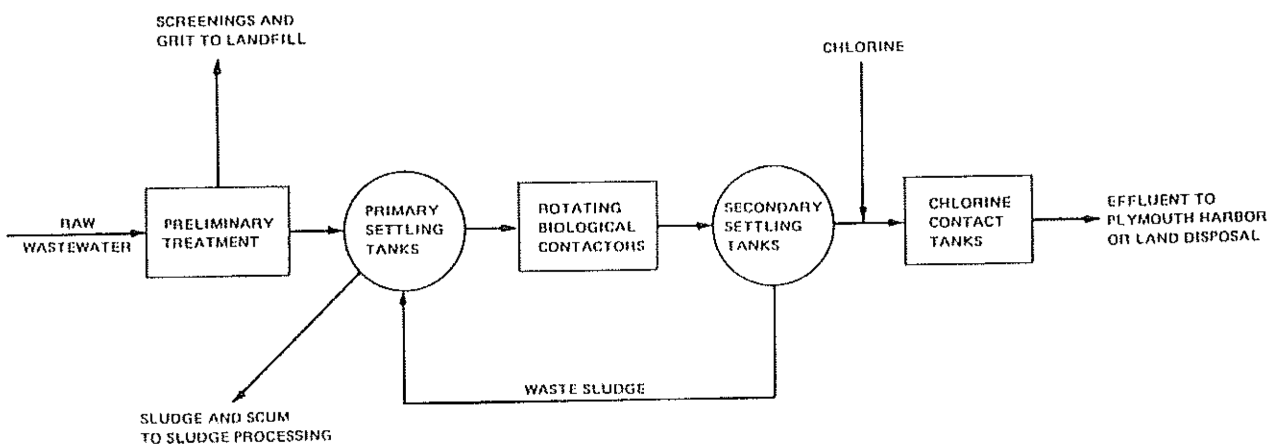


FIG. 9-9 TREATMENT ALTERNATIVE N-2
ROTATING BIOLOGICAL CONTACTORS

provide biological treatment rather than an activated sludge system. Sixteen covered RBC units would be required.

Alternative N-3. Primary Treatment

Under this alternative, depicted schematically in Figure 9-10, screenings and grit would be removed from the wastewater during preliminary treatment, settleable solids would be removed in the primary settling tanks, and the effluent would be chlorinated prior to land disposal. Biological treatment would not be provided, resulting in lower capital and annual O&M costs than those associated with Alternatives N-1 and N-2.

Sludge Stabilization Alternatives. For each of the two secondary wastewater treatment alternatives (N-1 and N-2) two sludge stabilization alternatives were evaluated. These consisted of belt filter press dewatering with lime stabilization and landfill disposal (the alternative determined to have the lowest cost in the evaluation of modifications to the existing plant site) and anaerobic digestion with belt filter press dewatering and landfill disposal. These alternatives are shown schematically in Figure 9-11. In the case of the anaerobic digestion alternative, two-stage digestion would be provided, requiring two 50-foot diameter digesters equipped with mixing and heating capabilities. In both cases, the dewatered sludge cake would be hauled to the Manomet Landfill for disposal.

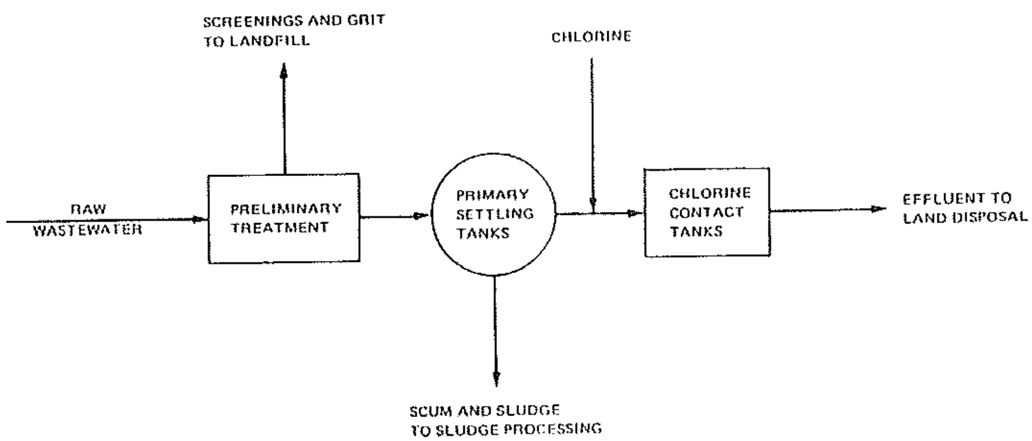
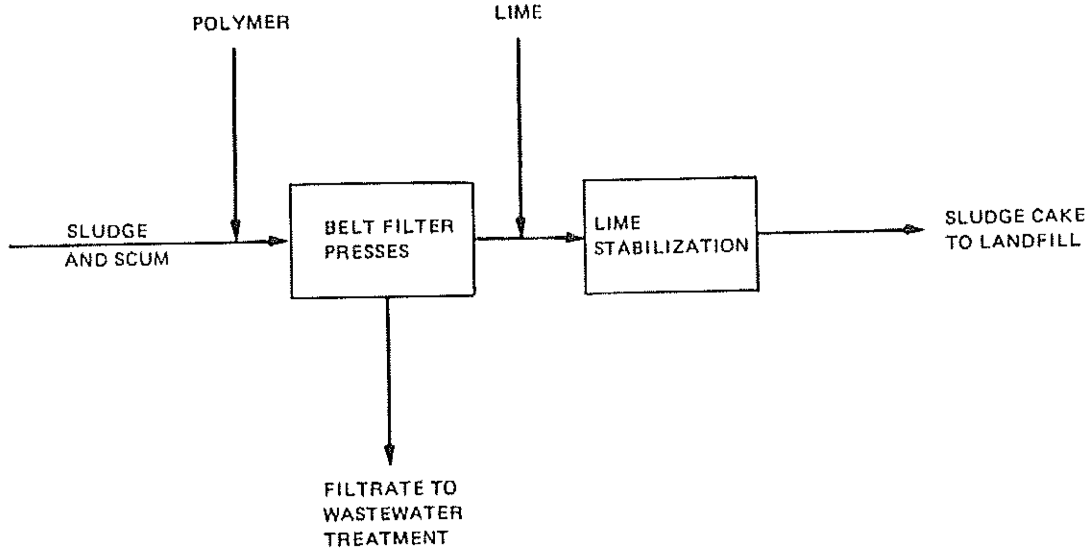
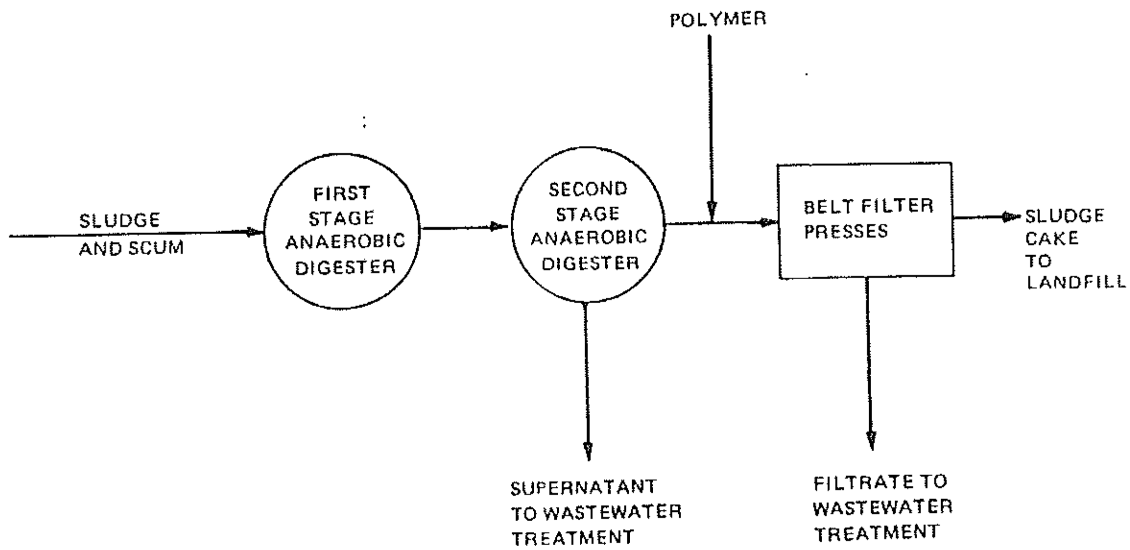


FIG. 9-10 TREATMENT ALTERNATIVE N-3
PRIMARY TREATMENT



a. BELT FILTER PRESS DEWATERING/LIME STABILIZATION



b. ANAEROBIC DIGESTION WITH BELT FILTER PRESS DEWATERING

FIG. 9-11 SLUDGE PROCESSING ALTERNATIVES
FOR NEW NORTH PLYMOUTH PLANT

Construction of a New Plant to Serve only Manomet

Two basic schemes of servicing Section 1 of Manomet have been proposed in Chapter 6. The design flows associated with these alternatives are 0.35 mgd and 0.18 mgd, as developed in Chapter 7.

The 1976 report prepared by Metcalf & Eddy regarding sewerage alternatives for Manomet concluded that conveying wastewater to a joint treatment facility serving both North Plymouth and Manomet would not be cost-effective due to the 6-mile distance between the centers of the two service areas. The subject of a joint plant was reexamined in a preliminary fashion during this study and again determined not to be cost-effective. A discussion of this subject is presented in Chapter 10.

Alternative wastewater treatment facility sites considered for Manomet are discussed in Chapters 8 and 10. The analysis in this chapter is limited to addressing treatment alternatives and their associated costs.

Wastewater Treatment Alternatives. Three alternatives for wastewater treatment were evaluated for Schemes 1 and 3 (0.35 mgd) and two for Schemes 2, 4, 5 and 6 (0.18 mgd).

Alternative M-1 (0.35 mgd). Extended Aeration

Under this alternative, the wastewater processes and flow pattern would be similar to those of the existing WWTP in North Plymouth (refer to Figure 3-2) except that influent pumping would not be necessary as wastewater would reach the plant via a force main.

Alternative M-2 (0.35 mgd). Rotating Biological Contactors.

This alternative is identical to the one considered for North Plymouth and shown schematically in Figure 9-8.

Alternatives M-3 (0.35 mgd) and M-5 (0.18 mgd). Aerated Facultative Lagoons.

Under these alternatives, represented schematically in Figure 9-12, wastewater would be treated in lined aerated basins using diffused air. The basins would be constructed to provide series operation with a total detention period of over 20 days. An effluent containing about 50 mg/L of BOD5 would be produced.

Alternative M-4 (0.18 mgd). Package Plant

Under this alternative, a commercially-available, prefabricated treatment plant utilizing the extended aeration process would be purchased and installed. Such units are normally economically feasible only for plants having design flows under 0.2 mgd. The tankage is usually constructed of steel. Advantages of package plants include small land requirements, low-cost installation, and low sludge production. Aerobic sludge digestion would be included as part of the package plant.

Sludge Stabilization and Dewatering Alternatives. For each of the wastewater treatment alternatives except the aerated

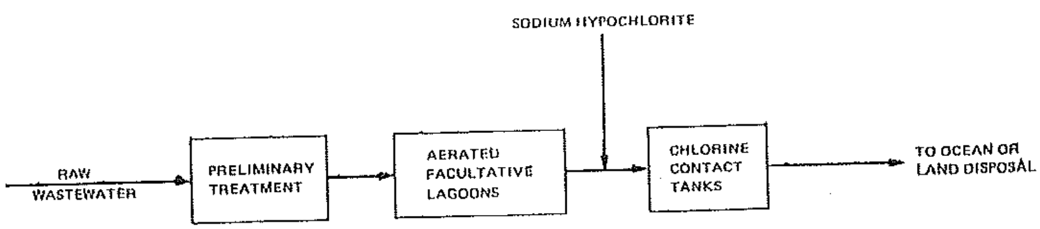


FIG. 9-12 TREATMENT ALTERNATIVES M-3 AND M-5
AERATED FACULTATIVE LAGOONS

facultative lagoon alternatives, two sludge stabilization and dewatering alternatives have been considered. These consisted of aerobic digestion with sludge drying beds and trucking of liquid unstabilized sludge to the North Plymouth WWTP. Under the latter alternative, the sludge would be dewatered and stabilized with the North Plymouth sludge.

Design Criteria

Design criteria used to size tankage and equipment in evaluating process alternatives are presented in Table 9-1.

Present Worth Cost Comparisons of Alternatives

The present worth comparison is a numerical method used to evaluate alternatives with differing capital and O&M costs. The cash flow for an alternative which is high in capital cost and low in annual O&M costs is different from the cash flow for a low capital, high O&M cost alternative. The present worth analysis treats O&M costs as equivalent capital costs. These costs may then be added together and the totals compared. The alternative with the lowest present worth cost, representing the optimum combination of capital and O&M costs, is then identified. As previously stated, the lowest cost alternative is selected when it also is shown to provide high reliability, is operationally manageable, and does not impact unfavorably on the environment. When these criteria are met, the least cost alternative can also be considered to be "cost effective."

Evaluation of Costs. The cost evaluations in this chapter only present comparisons of costs for alternative wastewater

TABLE 9-1. DESIGN CRITERIA FOR PROCESS ALTERNATIVES

Process Item	Design Value
Aerated grit chambers	
Detention period at peak flow	≥ 3 min.
Air supply range	1.5-4.5 cfm/l.f.
Primary settling tanks (with WAS recycle)	
Overflow rate at peak flow	≤ 1500 gpd/sf
Aeration tanks--extended aeration	
Organic loading, average	≤ 12 lb BOD_5/1000 cu ft
F:MLVSS	0.1 lb BOD5/lb MLVSS
Aeration tanks--conventional activated sludge	
Organic loadings, average	≤ 50 lb BOD_5/1000 cu ft
F:MLVSS	0.25 to 0.35 lb BOD_5/lb MLVSS
RBCs	
Organic loading	≤ 2 lb soluble BOD/1000 sf
Hydraulic loading	≤ 2.5 gpd/sf
Secondary settling tanks--extended aeration	
Overflow rate at peak flow	≤ 800 gpd/sf
Secondary settling tanks--conventional activated sludge	
Overflow rate at peak flow	≤ 1200 gpd/sf
Solids loading rate at peak flow	≤ 45 lb/sf/day
Secondary settling tanks, RBCs	
Overflow rate at peak flow	≤ 1500 gpd/sf
Aerated facultative lagoons	
Removal rate constant	0.14 days-1
Detention time	21 days
Chlorine contact tank	
Ocean disposal (peak flow)	30 min
Land disposal (peak flow)	15 min
Flotation thickeners	
Loading rate	≤ 1.5 lb/sf/hr
Operation time	35 hours/week

TABLE 9-1 (Continued). DESIGN CRITERIA FOR PROCESS ALTERNATIVES

Process Item	Design Value
Belt filter presses	
Loading rate--primary and WAS	$\leq$ 600 lb/hr/meter
Loading rate--WAS only	$\leq$ 400 lb/hr/meter
Loading rate--extended aeration	$\leq$ 300 lb/hr/meter
Operation time	30-35 hours week
Vacuum filters	
Loading rate--primary and WAS	3.5 lb/hr/sf
Loading rate--WAS only	2.5 lb/hr/sf
Operation time	30-35 hours/week
Drying beds (wedgewire)	
Average dry solids loading	1-1.5 lb/sf/day
Aerobic digesters	
Solids retention time, average	$\geq$ 15 days
Anaerobic digesters	
Volatile solids/loading	0.10 lb/cu ft/day
Solids retention time	$\geq$ 15 days
Composting	
Ratio of sludge to woodchips by volume	1:3.3
Woodchip recovery rate	60 percent
Sludge-only landfilling	
Bulking ratio by volume	1:1

treatment processes. Total system cost evaluations including costs of WWTP sites, delivery systems, effluent pumping and disposal are presented in Chapter 10. The evaluation of costs for purposes of alternative comparison has been done in accordance with Appendix A of the Construction Grants Regulations, 40 CFR 35. The following factors have been used:

1. Interest Rate. The interest rate used for the analysis is 7-7/8 percent, the rate established in the "Standards for Planning Water and Related Land Resources" of the Water Resources Council for fiscal year 1983.
2. Salvage Value. The salvage value of the treatment works is calculated using the service life criteria established in the Construction Grants Regulations, that is, structures - 50 years, process equipment - 20 years, auxiliary equipment - 10 years. Straightline depreciation is used, assuming zero salvage value at the end of the service life. Replacement of auxiliary equipment is included in the materials and supplies under O&M costs.
3. Interest During Construction. The interest during construction is the interest cost accrued on the funds paid to the contractor over the period of construction. Interest during construction represents interest which the Town must pay on short-term investments over the construction period, because it

requires cash for payment to the contractor while awaiting grant monies. While there is significant variation in the pattern of payments to a contractor, the average interest period is assumed to be one-half of the construction period.

For purposes of alternative comparison, a construction period of two years is assumed, along with an interest rate of 7-7/8 percent.

4. Cost Index. All costs have been calculated using an ENR index value of 4000, which was the average value for February 1983.

5. Operation and Maintenance Costs. Annual O&M costs for design year (2007) flows and loadings have been differentiated into the following components: energy, chemicals, materials and supplies, and labor. The unit prices used in the calculation of the O&M costs are shown in Table 9-2 and are 1983 prices.

6. Land. Each piece of land considered for use in this study was given individual consideration as to its value. The Town Assessor was consulted to obtain reasonably accurate evaluations for each site. The following summarizes the land values used:

   Waterfront near Plymouth WPCP - $300,000/acre

   Waterfront in Cordage Park and Manomet - $65,000-$80,000/acre

   Other land - $1,000-$44,000/acre.

TABLE 9-2. OPERATION AND MAINTENANCE COSTS - UNIT PRICES

Item	Unit Price
Labor	$11.70/hour
Power	$0.065/kwh
Lime (as CaO)	$100/ton
Ferric Chloride	$0.20/lb
Polymer	$2.00/lb
Chlorine	$300/ton
Sodium hypochlorite	$.40/lb Cl_2
Wood chips	$6/cu. yd.

* Including fringe benefits and taxes.

For salvage value calculation, land has been assumed to appreciate at a rate of 3 percent per year.

7. Engineering and Contingencies. A factor of 30 percent has been added to the subtotal of capital costs for each alternative to account for engineering costs during design and construction plus construction contingencies.

Costs to Expand Existing North Plymouth WWTP. Costs have been developed for each of the wastewater treatment alternatives in combination with each of the three sludge stabilization and dewatering alternatives considered. Estimated capital, annual O&M, and present worth costs are presented in Tables 9-3, 9-4, and 9-5 respectively.

TABLE 9-3. COMPARISON OF ESTIMATED CAPITAL COSTS FOR EXPANDING EXISTING NORTH PLYMOUTH WWTP TO 4.0 MGD CAPACITY

(All costs in thousands of dollars.)

Structure	Alternative F-1 Primary and activated sludge: F-1a lime stab., vacuum filters, landfill disposal	F-1b belt filter press, lime stab., landfill disposal	F-1c belt filter press, offsite composting	Alternative F-2 Activated sludge alone: F-2a lime stab., vacuum filters, landfill disposal	F-2b belt filter press, lime stab., landfill disposal	F-2c belt filter press, offsite composting	Alternative F-3 Activated sludge alone with flotation thickening: F-3a lime stab., vacuum filters, landfill disposal	F-3b belt filter press, lime stab., land fill disposal	F-3c belt filter press, offsite composting	Alternative F-4: Same as F-2b except sludge handling build. is located at Cordage Pk.
Headworks modifications	$ 203	$ 203	$ 203	$ 203	$ 203	$ 203	$ 203	$ 203	$ 203	$ 203
Aerated grit chambers	225	225	225	225	225	225	225	225	225	225
Primary settling tanks primary sludge pumping	941	941	941	0	0	0	0	0	0	0
Aeration tank modifications	295	295	295	349	349	349	349	349	349	349
Final settling tanks ary sludge pumping modifications	1,166	1,166	1,166	1,166	1,166	1,166	1,166	1,166	1,166	1,166
Chlorination equipment	65	65	65	65	65	65	65	65	65	65
Chlorine contact tanks (1)	451	451	451	451	451	451	451	451	451	451
Flotation thickeners	0	0	0	0	0	0	300	300	300	0
Sludge holding tank modifications	93	93	93	93	93	93	93	93	93	93
Pumping to sludge dewatering	94	94	94	94	94	94	94	94	94	108
Vacuum filters	668	0	0	668	0	0	668	0	0	0
Belt filter presses	0	540	540	0	460	460	0	378	378	460
Lime stabilization	50	110	0	50	110	0	50	110	0	110
Sludge handling building	1,433	1,261	1,335	1,413	1,261	1,335	2,095	1,939	1,805	1,261
Landfill expansion	150	150	0	150	150	0	150	150	0	150
Composting facility	0	0	671	0	0	671	0	0	671	0
Vehicles	255	255	255	255	255	255	255	255	255	255
Standby generator	113	113	113	113	113	113	113	113	113	113
Subtotal	$ 5,987	$ 5,962	$ 6,243	$ 5,095	$ 4,995	$ 5,276	$ 6,073	$ 5,891	$ 6,172	$ 5,009
Sitework and piping	598	596	624	510	500	528	607	589	617	489
Electrical and instrumentation	726	721	755	613	606	638	735	713	767	606
General and special conditions	438	437	452	323	346	387	445	412	452	335
Subtotal	$ 7,742	$ 7,716	$ 8,074	$ 6,595	$ 6,565	$ 6,824	$ 7,860	$ 7,625	$ 7,985	$ 6,703
Engineering and contingencies	2,321	2,315	2,424	1,979	1,939	2,049	2,358	2,287	2,396	2,011
Total WWTP Capital Cost	$10,063	$10,031	$10,501	$ 8,574	$ 8,504	$ 8,876	$10,218	$ 9,912	$10,384	$ 8,714

1. Assuming detention time of 30 minutes at peak flow.

TABLE 9-4. COMPARISON OF ESTIMATED O&M COSTS FOR EXPANDING EXISTING NORTH PLYMOUTH WWTP TO 4.0 MGD CAPACITY

Item	Alternative P-1 Primary and activated sludge			Alternative P-2 Activated sludge alone			Alternative P-3 Activated sludge with flotation thickening			Alternat. P-4
	P-1a lime stab., vacuum filters, landfill disposal	P-1b belt filter press, lime stab., landfill disposal	P-1c belt filter press, offsite composting	P-2a lime stab., vacuum filters, landfill disposal	P-2b belt filter press, lime stab., landfill disposal	P-2c belt filter press, offsite composting	P-3a lime stab., vacuum filters, landfill disposal	P-3b belt filter press, lime stab., landfill disposal	P-3c belt filter press, offsite composting	Same as P-2b except sludge handling building is located at Cordage Park
Energy										
Power	$ 84,600	$ 80,500	$ 86,900	$ 99,400	$ 95,900	$ 99,500	$105,700	$102,300	$106,000	$ 96,200
Natural Gas	20,000	20,000	18,000	20,000	20,000	18,000	25,000	25,000	23,000	20,000
Fuel-sludge disposal*	3,400	3,100	3,000	3,000	2,500	2,500	2,300	2,200	2,200	2,900
Chemicals										
Chlorine	9,200	9,200	9,200	9,200	9,200	9,200	9,200	9,200	9,200	9,200
Lime	33,000	50,500	0	19,500	27,500	0	18,600	27,500	0	27,500
Ferric chloride	33,200	0	0	19,600	0	0	18,800	0	0	0
Polymer	0	33,600	33,600	0	20,400	20,400	0	37,200	37,200	20,400
Wood Chips	0	0	115,200	0	0	87,400	0	0	69,600	0
Materials and supplies										
Sludge disposal*	63,300	54,000	33,500	51,300	41,500	22,000	37,100	32,800	21,100	41,500
Other	53,900	56,200	53,900	50,400	52,700	50,400	50,400	52,200	50,400	52,700
Labor										
Sludge disposal*	35,700	32,800	80,500	31,800	27,800	55,300	25,900	24,900	53,800	27,800
Other	172,000	172,700	162,400	156,900	156,400	138,200	172,800	172,000	163,400	156,400
Annual WWTP O&M Cost	$510,000	$513,200	$596,200	$461,000	$453,900	$502,800	$465,800	$485,300	$535,900	$454,600

*Offsite costs associated with sludge hauling, landfilling and composting.

TABLE 9-5. COMPARISON OF ESTIMATED PRESENT WORTH COSTS FOR EXPANDING EXISTING NORTH PLYMOUTH WWTP TO 4.0 MGD CAPACITY
(All costs in thousands of dollars)

Item	Alternative P-1 Primary and activated sludge			Alternative P-2 Activated sludge alone			Alternative P-3 Activated sludge alone with flotation thickening			Alternat. P-4
	P-1a lime stab., vacuum filters, landfill disposal	P-1b belt filter press, lime stab., landfill disposal	P-1c belt filter press, offsite composting	P-2a lime stab., vacuum filters, landfill disposal	P-2b belt filter press, lime stab., landfill disposal	P-2c belt filter press, offsite composting	P-3a lime stab., vacuum filters, landfill disposal	P-3b belt filter press, lime stab., landfill disposal	P-3c belt filter press, offsite composting	Same as P-2b except sludge handling building is located at Cordage Park
WWTP capital cost	$10,065	$10,031	$10,503	$ 8,574	$ 8,404	$ 8,878	$10,218	$ 9,912	$10,384	$ 8,714
Land acquisition										
Sludge building	0	0	0	0	0	0	0	0	0	33
Sludge disposal site*	0	0	0	0	0	0	0	0	0	0
Interest during construction	793	790	827	675	662	699	805	781	818	686
P.W. of annual WWTP O&M cost	5,062	5,086	5,908	4.569	4,498	4.983	4,616	4,809	5,311	4,505
Less P.W. of salvage value										
WWTP	-488	-474	-512	-414	-396	-434	-505	-479	-521	-396
Sludge building land	0	0	0	0	0	0	0	0	0	-13
Sludge disposal site land	0	0	0	0	0	0	0	0	0	0
Total present worth	$15,432	$15,433	$16,726	$13,404	$13,168	$14,126	$15,134	$15,023	$14,992	$13,529
Annualized cost	$ 1,557	$ 1,557	$ 1,688	$ 1,352	$ 1,329	$ 1,425	$ 1,527	$ 1,516	$ 1,674	$ 1,365

*Assuming composting and/or sludge disposal at the Manomet Landfill.

From the tables, it can be seen that Alternative P-2b, activated sludge alone with belt filter press dewatering, lime stabilization and landfill disposal of sludge shows the lowest capital, O&M, and present worth costs of the above nine alternatives. Costs for a tenth alternative (P-4), identical to Alternative P-2b but with the sludge processing bulding located remotely at Cordage Park, were also estimated and presented in Tables 9-3, 4, and 5 and to assess the impact of retaining the major portion of the lot adjacent to the WWTP as a parking lot for use by Ocean Spray Cranberries. The costs of Alternative P-4 are higher than P-2b due to the additional piping and pumping required for sludge conveyance. This alternative is not recommended because certain of the facilities to be included in the sludge processing building (i.e., the new laboratory and office) are best located at the WWTP site.

Costs to Construct a New Plant to Serve North Plymouth. Costs have been developed for wastewater treatment alternatives N-1 and N-2 in combination with each of the sludge stabilization alternatives. Costs have also been developed for wastewater Alternative N-3 in combination with the sludge stabilization alternative determined to have the lowest cost in earlier evaluations. Estimated capital, annual O&M and present worth costs are presented in Table 9-6, 9-7, and 9-8, respectively.

TABLE 9-6. COMPARISON OF ESTIMATED CAPITAL COSTS FOR NEW 4.0 MGD SWTF FOR NORTH PLYMOUTH

(All costs in thousands of dollars)

Structure	Alternative N-1 Activated sludge: N-1a belt filter press, lime stabilization and landfill disposal	Alternative N-1 Activated sludge: N-1b anaerobic digestion with belt filter press and landfill disposal	Alternative N-2 RBC's: N-2a belt filter press, lime stabilization and landfill disposal	Alternative N-2 RBC's: N-2b anaerobic digestion with belt filter press and landfill disposal	Alternative N-3 Primary treatment (land disposal): belt filter press, lime stabilization and landfill disposal
Septage receiving facility	$ 175	$ 175	$ 175	$ 175	$ 175
Preliminary treatment	600	600	600	603	600
Primary settling tanks and primary sludge pumping	638	638	638	638	578
Aeration tanks with mechanical aerators	992	992	992	992	0
RBC's	0	0	1,400	1,400	0
Secondary settling tanks with sludge pumping	638	638	638	635	0
Chlorination equipment	98	98	98	98	98
Chlorine contact tanks	354 (1)	354 (1)	354 (1)	354 (1)	582 (2)
Anaerobic digestion	0	1,253	0	1,253	0
Belt filter presses and feed sludge pumping	560	560	560	560	460
Lime stabilization	110	0	110	0	110
Operations/sludge handling building	1,230	1,600	1,230	1,600	1,230
Landfill expansion	150	150	150	150	150
Vehicles	255	255	255	255	255
Subtotal	$ 6,195	$ 7,407	$ 6,857	$ 7,855	$6,163
Sitework and piping	619	751	685	786	616
Electrical and instrumentation	774	890	824	953	501
General and special conditions	468	561	501	576	303
Subtotal	$ 8,277	$ 9,587	$ 8,857	$10,168	$5,363
Engineering and contingencies	2,483	2,876	2,657	3,050	1,608
Total SWTF capital cost	$10,759	$12,463	$11,572	$13,218	$6,969

1. Assuming detention time of 30 minutes at peak flow.
2. Assuming detention time of 15 minutes at peak flow.

TABLE 9-7. COMPARISON OF ESTIMATED ANNUAL O&M COSTS FOR NEW 4.0 MGD WWTP FOR NORTH PLYMOUTH

Item	Alternative N-1 Activated sludge		Alternative N-2 RBC's		Alternative N-3 Primary treatment (land disposal)
	N-1a belt filter press, lime stabilization and landfill disposal	N-1b anaerobic digestion with belt filter press and land-fill disposal	N-2a belt fil-ter press, lime stab., and landfill disposal	N-2b anaerobic diges-tion with belt filter press and landfill disposal	Belt filter press, lime stabilization, and landfill disposal
Energy					
Power	$ 51,200	$ 51,500	$ 37,200	$ 37,500	$ 29,140
Fuel oil-heating	20,000	18,000	20,000	18,000	20,000
Fuel-sludge disposal*	3,700	2,600	3,700	2,600	1,900
Chemicals					
Chlorine	9,200	9,200	9,200	9,200	0
Lime	45,500	0	45,500	0	29,200
Ferric chloride	0	0	0	0	0
Polymer	24,000	24,000	24,000	24,000	12,000
Materials and supplies					
Sludge disposal*	64,700	40,700	64,700	40,700	42,700
Other	46,300	47,200	39,600	40,500	26,700
Labor					
Sludge disposal*	37,100	28,100	37,100	28,100	26,900
Other	158,800	164,500	154,400	160,100	123,400
Annual WWTP O&M cost	$460,500	$385,800	$405,400	$360,700	$311,900

*Offsite costs associated with sludge hauling to Manomet Landfill assuming plant location at Town Brook site for Alternatives N-1 and N-2 and R.I. Site A for Alternative N-3.

TABLE 9-8. COMPARISON OF ESTIMATED PRESENT WORTH COSTS FOR NEW 4.0-MGD WWTP FOR NORTH PLYMOUTH

(All costs in thousands of dollars)

	Alternative N-1 Activated sludge		Alternative N-2 RBC's		Alternative N-3 Primary treatment (land disposal)
	N-1a belt filter press, lime stabilization and landfill disposal	N-1b anaerobic digestion with belt filter press and landfill disposal	N-2a belt filter press, lime stab., and landfill disposal	N-2b anaerobic digestion with belt filter press and landfill disposal	Belt filter press, lime stabilization and landfill disposal
WWTP capital cost	$10,759	$12,463	$11,517	$13,218	$ 6,969
Land acquisition					
WWTP (1)	44	44	44	44	15
Sludge disposal (2)	0	0	0	0	0
Interest during construction	847	981	906	1,040	549
P.W. of annual WWTP O&M costs	4,564	3,823	4,018	3,575	3,091
Less P.W. of salvage value					
WWTP	-453	-551	-539	-636	-346
WWTP land (1)	-17	-17	-17	-17	-6
Sludge disposal land	0	0	0	0	0
Total present worth	$15,744	$16,743	$15,924	$17,224	$10,272
Annualized cost of total present worth	$ 1,589	$ 1,689	$ 1,607	$ 1,738	$ 1,036

1. Assuming WWTP location at Town Brook site for Alternatives N-1 and N-2 and at R.I. Site A for Alternative N-3.
2. Assuming composting and/or sludge disposal at the Manomet Landfill.

It can be seen from the tables that Alternative N-3 (primary treatment) has the lowest costs, as would be expected. However, the costs for this treatment alternative cannot be directly compared with those of secondary treatment alternatives as the disposal costs associated with primary versus secondary treatment must also be taken into account. This is done in Chapter 10.

Among the secondary treatment alternatives, Alternative N-1a (activated sludge with belt filter press dewatering of sludge) has estimated costs slightly lower than those of Alternative N-2a (RBCs).

Costs to Construct a New Plant to Serve only Manomet. Estimated costs for the wastewater treatment and sludge stabilization alternatives considered for a new Manomet treatment facility are presented in Tables 9-9, 9-10, and 9-11. The present worth costs of Alternatives M-3 and M-5 (aerated lagoons) are estimated to be less than those of the other alternatives considered. Although these alternatives will not produce effluent of quite the same quality as the other alternatives, no significant disadvantages of these alternatives with respect to the other alternatives are foreseen, assuming land disposal is selected for Manomet.

Alternative Disinfection Methods

Chlorination has been presented as the method of disinfection in all of the preceding cost analyses. This has been done because this is the method presently used in Plymouth,

TABLE 9-9. COMPARISON OF ESTIMATED CAPITAL COSTS FOR MARSHFIELD WASTEWATER TREATMENT

	Schemes 1 and 3 -- Average Flow = 0.35 mgd					Schemes 2, 4, 5 and 6 Average Flow = 0.18 mgd		
	Alternative M-1 Extended aeration		Alternative M-2 RBC's		Alternative M-3	Alternative M-4 Package plant		Alternative M-5
Structure	M-1a (with drying beds)	M-1b (with sludge hauled to N. Plymouth)	M-2a (with drying beds)	M-2b (with sludge hauled to N. Plymouth)	Aerated facultative lagoons	M-4a (with drying beds)	M-4a (with sludge hauled to N. Plymouth	Aerated facultative lagoons
Preliminary treatment	$ 78,000	$ 78,000	$ 78,000	$ 78,000	$ 50,000	$ 50,000	$ 50,000	$ 30,000
Primary clarifiers and primary sludge pumping	-	-	265,000	265,000	-	-	-	-
Aeration tanks with mechanical aerators	518,000	518,000	-	-	-	-	-	-
RBC's	-	-	385,000	385,000	-	-	-	-
Secondary clarifiers with sludge pumping	295,000	295,000	200,000	200,000	-	-	-	-
Chlorination equipment	7,000	7,000	7,000	7,000	7,000	-	-	7,000
Chlorine contact tank*	38,000	38,000	38,000	38,000	38,000	-	-	23,000
Aerobic digesters with blowers	168,000	-	317,000	-	-	-	-	-
Sludge holding tanks	-	84,000	-	158,000	-	-	-	-
Sludge drying beds	86,000	-	140,000	-	-	50,000	-	-
Operations building	150,000	150,000	150,000	150,000	100,000	75,000	75,000	75,000
Trucks	50,000	10,000	50,000	10,000	10,000	10,000	10,000	10,000
Package treatment plant***	-	-	-		-	192,000	192,000	-
Aerated facultative lagoons with diffused air	-	-	-	-	315,000	-	-	195,000
Subtotal	$1,390,000	$1,180,000	$1,630,000	$1,291,000**	$520,000	$377,000	$327,000	$340,000
Sitework and Piping	139,000	118,000	163,000	129,000	52,000	59,000	59,000	34,000
Electrical & Instrumentation	168,000	143,000	197,000	156,000	63,000	22,000	22,000	41,000
General & Special conditions	102,000	86,000	119,000	95,000	38,000	27,000	23,000	25,000
Subtotal	$1,799,000	$1,527,000	$2,109,000	$1,671,000	$ 673,000	$ 485,000	$ 431,000	$ 440,000
Engineering & Contingencies	$ 540,000	458,000	633,000	501,000	202,000	145,000	129,000	132,000
Total WWTP Capital Cost	$2,339,000	$1,985,000	$2,742,000	$2,172,000	$ 875,000	$ 630,000	$ 560,000	$ 572,000

*Assuming 15 min. d.t. at peak flow.
**Ignoring cost of additional sludge dewatering equipment at No. Plymouth Plant
***Including chlorination equipment and contact tankage.

TABLE 9-10. COMPARISON OF ESTIMATED ANNUAL O&M COSTS FOR MANOMET WASTEWATER TREATMENT

Item	Average Flow = 0.35 mgd					Average Flow = 0.18 mgd		
	Alternative H-1 Extended aeration		Alternative H-2 RBC's		Alternative H-3	Alternative H-4 Package plant		Alternative H-5
	H-1a (with drying beds)	H-1b (with sludge hauled to N. Plymouth)	H-2a (with drying beds)	H-2b (with sludge hauled to N. Plymouth)	Aerated facultative lagoons	H-4a (with drying beds)	H-4b (with sludge hauled to N. Plymouth)	Aerated facultative lagoons
Energy								
Power	$29,500	$29,500	$ 9,100	$ 9,200	$17,600	$12,700	$12,800	$ 9,100
Fuel oil-heating	2,000	2,000	2,000	2,000	2,000	1,000	1,000	1,000
Fuel - sludge hauling*	100	1,000	200	2,000	0	70	700	0
Chemicals**	1,800	5,100	1,800	10,000	1,800	900	2,500	900
Materials & supplies								
Wastewater treatment	15,000	15,000	14,500	15,200	4,900	9,200	9,200	3,900
Sludge management*	50	500	100	1,000	0	30	300	0
Labor								
Wastewater treatment	25,200	22,900	24,700	21,100	22,120	16,900	15,300	19,800
Sludge management*	1,200	10,300	2,300	14,500	0	800	6,900	0
Annual WWTP O&M cost	$74,850	$86,300	$54,700	$75,000	$48,400	$41,600	$48,700	$29,700

*Offsite costs assuming sludge hauled to North Plymouth would be lime stabilized and landfilled in the Manomet landfill, while sludge dried on drying beds in Manomet would be hauled directly to the Manomet landfill.

**Cost includes chemicals used to dewater sludge at North Plymouth WWTP for Alternatives H-1b, H-2b and H-4b.

TABLE 9-11. COMPARISON OF ESTIMATED PRESENT WORTH COSTS FOR MANOMET WASTEWATER TREATMENT

Item	Average Flow = 0.35 mgd: Alternative M-1 Extended aeration: M-1a (with drying beds)	M-1b (with sludge hauled to N. Plymouth)	Alternative M-2 RBC's: M-2a (with drying beds)	M-2b (with sludge hauled to N. Plymouth)	Alternative M-3: Aerated facultative lagoons	Average Flow = 0.18 mgd: Alternative M-4 Package plant: M-4a (with drying beds)	M-4a (with sludge hauled to N. Plymouth)	Alternative M-5: Aerated facultative lagoons
WWTP capital cost	$2,339,000	$1,985,000	$2,742,000	$2,172,000	$ 875,000	$ 630,000	$ 560,000	$ 572,000
WWTP land acquisition*	7,000	7,000	7,000	7,000	28,000	7,000	7,000	21,000
Interest during construction	184,000	156,000	216,000	171,000	69,000	50,000	44,000	45,000
P.W. of annual WWTP O&M cost	742,000	855,000	542,000	743,000	480,000	412,300	483,000	294,000
Less P.W. of WWTP salvage value	-123,500	-106,700	-150,000	-120,600	-45,600	-28,400	-22,900	-29,300
Less P.W. of WWTP land salvage value	-2,800	-2,800	-2,800	-2,800	-4,800	-2,800	-2,800	-8,400
Total present worth	$3,146,000	$2,894,000	$3,354,000	$2,970,000	$1,401,600	$1,068,100	$1,068,000	$894,000
Annualized cost	$ 317,000	$ 292,000	$ 338,000	$ 300,000	$ 141,000	$ 108,000	$ 108,000	$ 90,000

*Assuming 2-acre WWTP site for all alternatives except M-3 and M-5; assuming 8-acre WWTP site for M-3 and 6-acre WWTP site for M-5.

and numerical modeling of the effects of chlorinated effluent on the receiving waters at the various alternative discharge locations has not predicted any serious adverse effects, as discussed in Appendix B.

At the request of the DEQE we have examined in preliminary fashion the costs of an alternative method of disinfection at the expanded Plymouth WWTP. The methods which have been suggested for Plymouth include ultraviolet radiation (UV), gamma radiation, and ozonation.

UV and gamma radiation cannot be considered seriously because of the experimental nature of these processes and high cost. Gamma radiation is not used as an effluent disinfectant anywhere in North America. UV has been used successfully elsewhere as an effluent disinfectant. However, the potential of photoreactivation of bacteria has important implications where shellfish resources are at issue, as they are in Plymouth Harbor. Thus, research would be required to address this problem. Also, the costs of installing a UV disinfection system have been estimated recently for a similar facility in Scituate, Massachusetts and were high relative to a system including both chlorination and dechlorination.

An ozone disinfection system could be given serious consideration as a substitute for chlorination at the Plymouth WWTP. The contact tank required for ozone disinfection would only be half the size of the tank required for chlorination (15 minutes versus 30 minutes at peak flow). However, a large amount of new building space would be required to house the ozone generators, ozone destruct unit, air compressors, air dryer, cooling water pumps, and electrical equipment required. The Scituate study, which included a detailed cost evaluation of ozone disinfection, indicated that a 32' x 64' building was required for that plant (which has only a quarter the design capacity of the expanded Plymouth WWTP). The cost of that building and ozone equipment was estimated to be $680,000, and a higher cost would be expected in the case of Plymouth. On the other hand, in Plymouth, the existing chlorinator room and scale room could be utilized if chlorination is continued in the future. Thus, capital costs would be higher for ozone disinfection as follows:

	Ozone Disinfection	Chlorine Disinfection
Contact tank	$ 442,000	$759,000
Building and equipment	680,000+	109,000
	$1,102,000+	$858,000

Furthermore, operating costs for ozone are also higher than for chlorine. Assuming that a 5 mg/L dosage of chlorine and a 5 mg/L dosage of ozone would be required, with a power requirement of 11 kwh per pound of ozone generated (using an air ozone generator), the power cost of ozone generation would be $43,000 per year versus $9200 per year for purchasing chlorine. Labor costs would also be expected to be higher for ozone disinfection than chlorine disinfection due to the more complex nature of the process.

In view of the fact that both capital and annual O&M costs for ozone disinfection would be higher than for chlorine disinfection, and considering the fact that no serious adverse effects are expected if chlorine disinfection is continued, chlorination is clearly the most cost-effective means of disinfection for Plymouth.

Summary

Alternative treatment processes have been reviewed, and the lowest cost wastewater treatment alternatives have been identified in this chapter. However, to judge the true cost-effectiveness of these alternatives, simultaneous consideration must also be given to treatment plant site locations and disposal alternatives. This subject is addressed in Chapter 10.

CHAPTER 10

WASTEWATER TREATMENT FACILITY SITE ALTERNATIVES AND COSTS

General

The process used to identify alternative land disposal sites was discussed in Chapter 8 and Appendix D, alternative outfall locations were discussed in Chapter 8 and Appendix B, and treatment process alternatives were evaluated in Chapter 9. This chapter presents an evaluation of treatment and disposal at alternative sites in North Plymouth and Manomet.

Preliminary Screening of Sites

Because of the limited expansion area available at the existing WWTP (shown in Figure 9-1), alternatives incorporating a relocated WWTP were considered. From past experience, it was determined that about 11 acres would be desirable for a 4-mgd secondary treatment plant, 5 acres for a 4-mgd primary treatment plant, about 3 acres for a 0.5-mgd aerated facultative lagoon facility to serve Manomet, and about 15 acres for a 4.5-mgd secondary treatment plant to serve both North Plymouth and Manomet. Early in the study, a USGS topographic map of Plymouth was reviewed to identify parcels of open land located in the proximity of the wastewater collection systems being considered for both North Plymouth and Manomet, and near the proposed ultimate discharge points as well to minimize the amount of piping and pumping required. In North Plymouth, few sites of sufficient area remain undeveloped. Sites identified are shown in Figure 10-1.

10-2

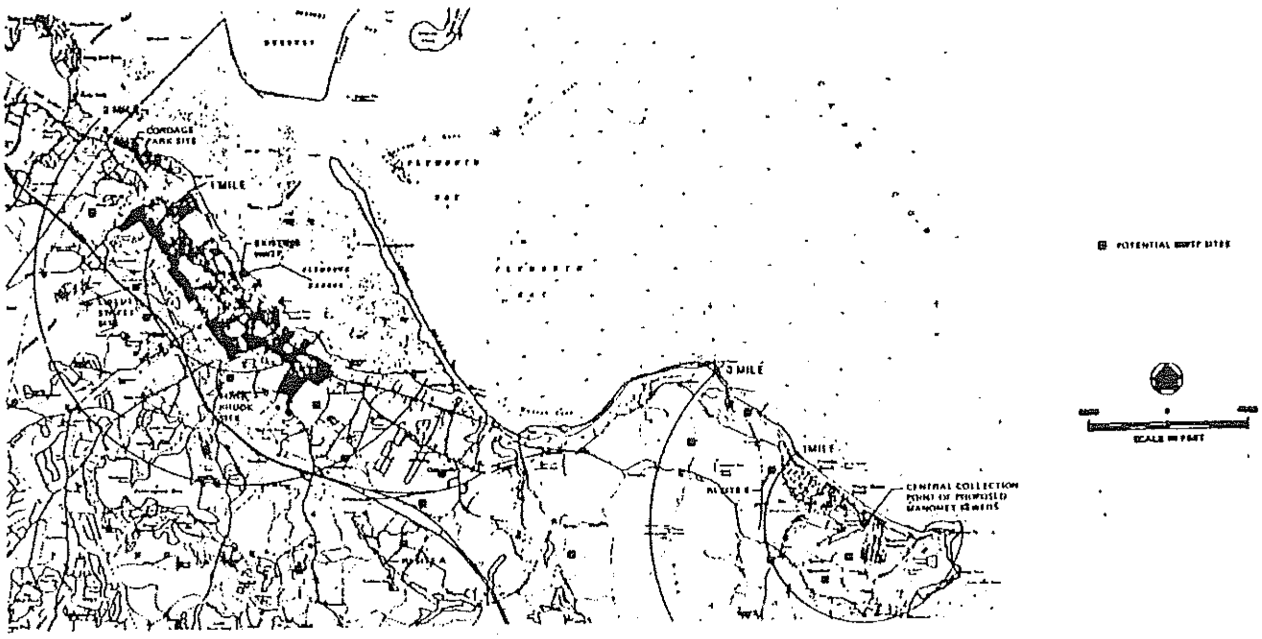


FIG. 10-1 POTENTIAL WWTP SITES

After it was determined that the most cost-effective and environmentally-acceptable outfall alternative would probably involve the extension of the existing outfall (see Chapter 8), three alternative treatment plant sites located within a reasonable distance from the outfall were selected for cost evaluations. These sites have been designated Town Brook (Stafford Street), Liberty Street and Cordage Park in Figure 10-1. A fourth site, located within the boundaries of Rapid Infiltration Site A, was also selected for evaluation as a primary treatment plant site to be used in conjunction with land disposal.

In the case of Manomet, where the land requirement for treatment is small and there is no existing disposal location, the selection of treatment facility location alternatives for further study was simplified. For the land disposal alternatives, it was assumed that the treatment lagoons would be located at the disposal sites (to eliminate the need for pumping treated effluent to the disposal basins). In the case of ocean disposal alternative, a site convenient to both the service area and proposed outfall location and high enough to discharge to the ocean by gravity was desired. In the latter case, a site adjacent to Boston Edison's Pilgrim Nuclear Power Station was attractive, but had to be eliminated from consideration when Boston Edison informed us that the site was in the station's exclusion zone (see correspondence in Appendix K). Therefore, a site within the station's buffer zone along the Edison Access Road was selected for further study.

Existing North Plymouth WWTP Site

Because the analysis in Chapter 9 concluded that estimated costs for expanding the existing WWTP to 4 mgd are less than those for constructing a new 4 mgd secondary WWTP, several disposal alternatives were considered in conjunction with the continued use of the existing WWTP site, as follows:

1. Ocean disposal
   a. using the existing outfall as modified only by the addition of a diffuser at the end
   b. extending the existing outfall to Location 1 (Goose Point Channel)
   c. extending the existing outfall to Location 2 (Harbor Entrance)
2. Land disposal
   a. at Rapid Infiltration Site A (Russell Mill Road)
   b. at Rapid Infiltration Site DD (County Farm)
3. Ocean disposal of 1.75 mgd using the existing outfall without extension and land disposal of 2.25 mgd at Rapid Infiltration Site A.

Additional discharge alternatives were considered but eliminated before undergoing detailed evaluation, as discussed in Chapter 8.

Under any alternative which includes a discharge to shellfish waters (all ocean discharge alternatives in Plymouth), the DEQE will require that and effluent holding tank be constructed. The major purpose of the tank would be to allow sufficient time to warn shellfishermen of any temporary closures resulting from a major plant failure. It is expected that this tank would be used no more than a few times a year during emergencies, when it would provide storage capacity for seven hours' wastewater flow (under average conditions), or roughly the equivalent of half a tidal cycle. For the proposed plant expansion, then, a volume of about 1.2 million gallons would be required. This tank would thus occupy a large space on the site and represent a significant cost. The tank would receive effluent from either the final settling tanks on the chlorine contact tanks, would not be equipped with mixing or sludge collection equipment, and would be designed to drain by gravity to the wet well at the head of the plant.

Under any of the three alternatives which include a discharge of all the treated flow through the existing 30-inch outfall, whether or not it is extended, an effluent pumping station would be required to discharge the peak flow rate during the highest tide level in Plymouth Harbor. Gravity discharge will generally occur except during periods when all the influent pumps are operating and/or the tide is high.

Under the two land disposal alternatives, effluent from the expanded WWTP would be pumped via a 30-inch force main to the land disposal site (see Figures 10-2 and 10-3). Table 10-1 summarizes the pumping distances and heads associated with conveying tne effluent to each alternative site. A high head

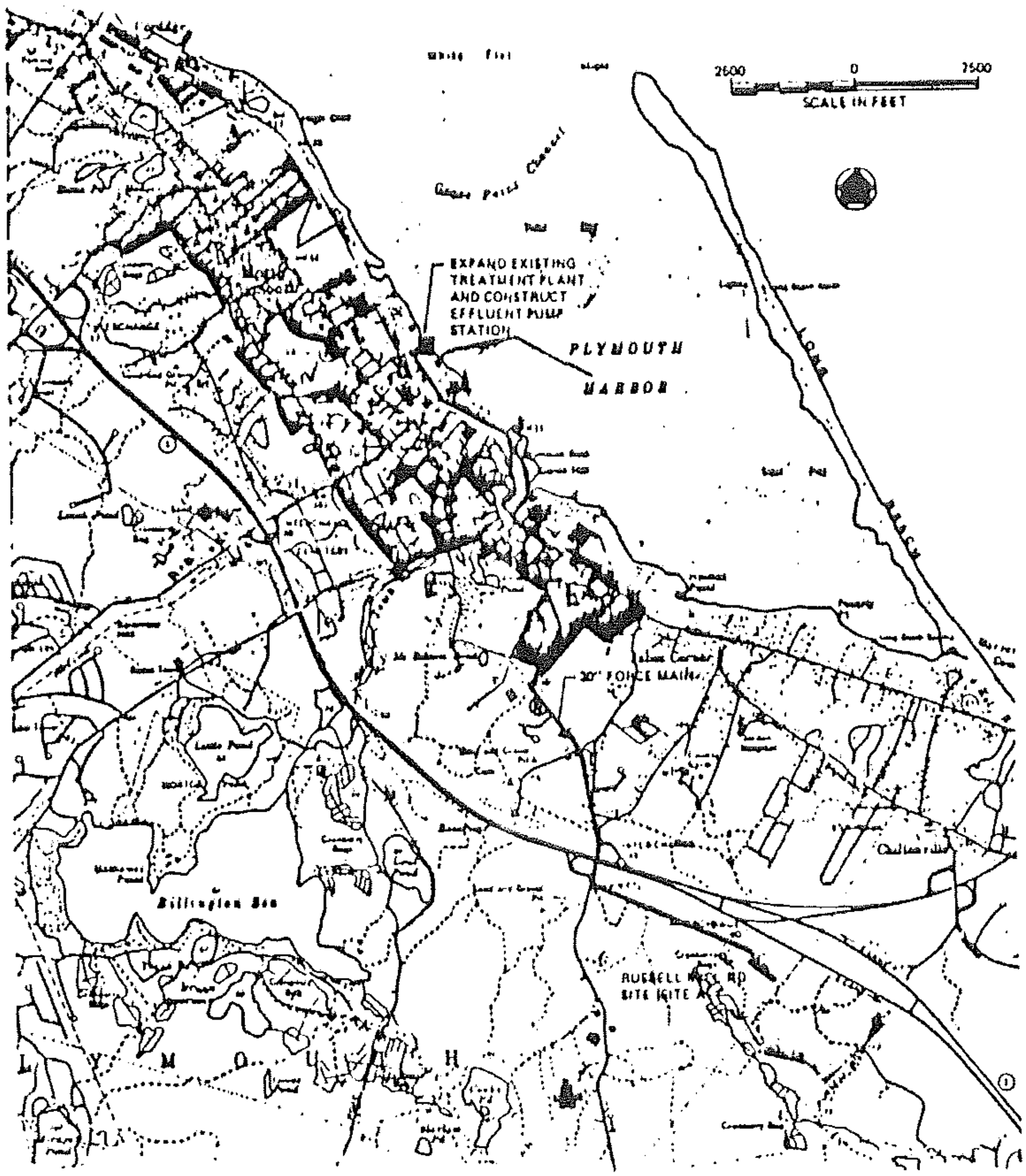


FIG. 10-2 TRANSMISSION ROUTE FOR CONVEYING SECONDARY EFFLUENT TO RUSSELL MILL ROAD SITE

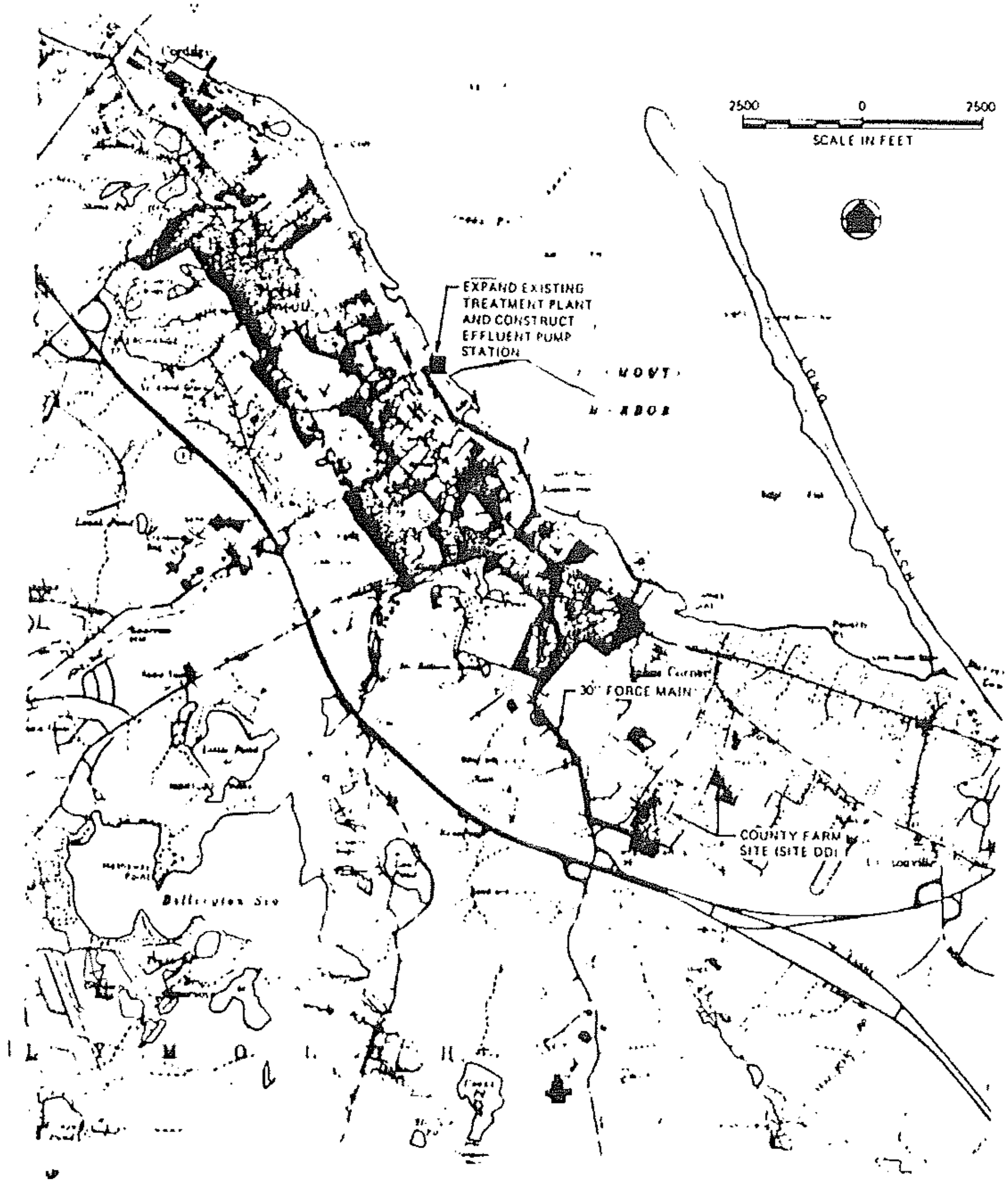


FIG. 10-3 TRANSMISSION ROUTE FOR CONVEYING SECONDARY EFFLUENT TO COUNTY FARM SITE

TABLE 10-1. PUMPING DISTANCES AND HEADS ASSOCIATED WITH LAND DISPOSAL OF SECONDARY EFFLUENT FROM EXPANDED WWTP

	Site A Russell Mill Road	Site DD (County Farm)
Transmission distance from existing WWTP, ft.	19,000	14,000
Total pumping head at peak flow, (9.5 mgd), ft.	185	170

(series) effluent pumping station would have to be constructed at the WWTP for either alternative.

Under these two alternatives no flow equalization tank would be required, and no chlorine contact tank would be needed since the required 15 minutes of detention time at peak flow would be provided in the force main conveying the effluent to the disposal site. It is also conceivable that the disinfection requirement might be waived by the DEQE if land disposal is selected (determinations are made on a case-by-case basis).

An alternative was considered utilizing the WWTP's permitted 1.75-mgd discharge allocation and pumping the remaining flow to Rapid Infiltration Site A. Under this alternative, requirements would exist for a smaller effluent holding tank, a chlorine contact tank, a high-head effluent pump station and force main, and rapid infiltration basins. This alternative also has the disadvantage of impacting upon both marine water and groundwater resources.

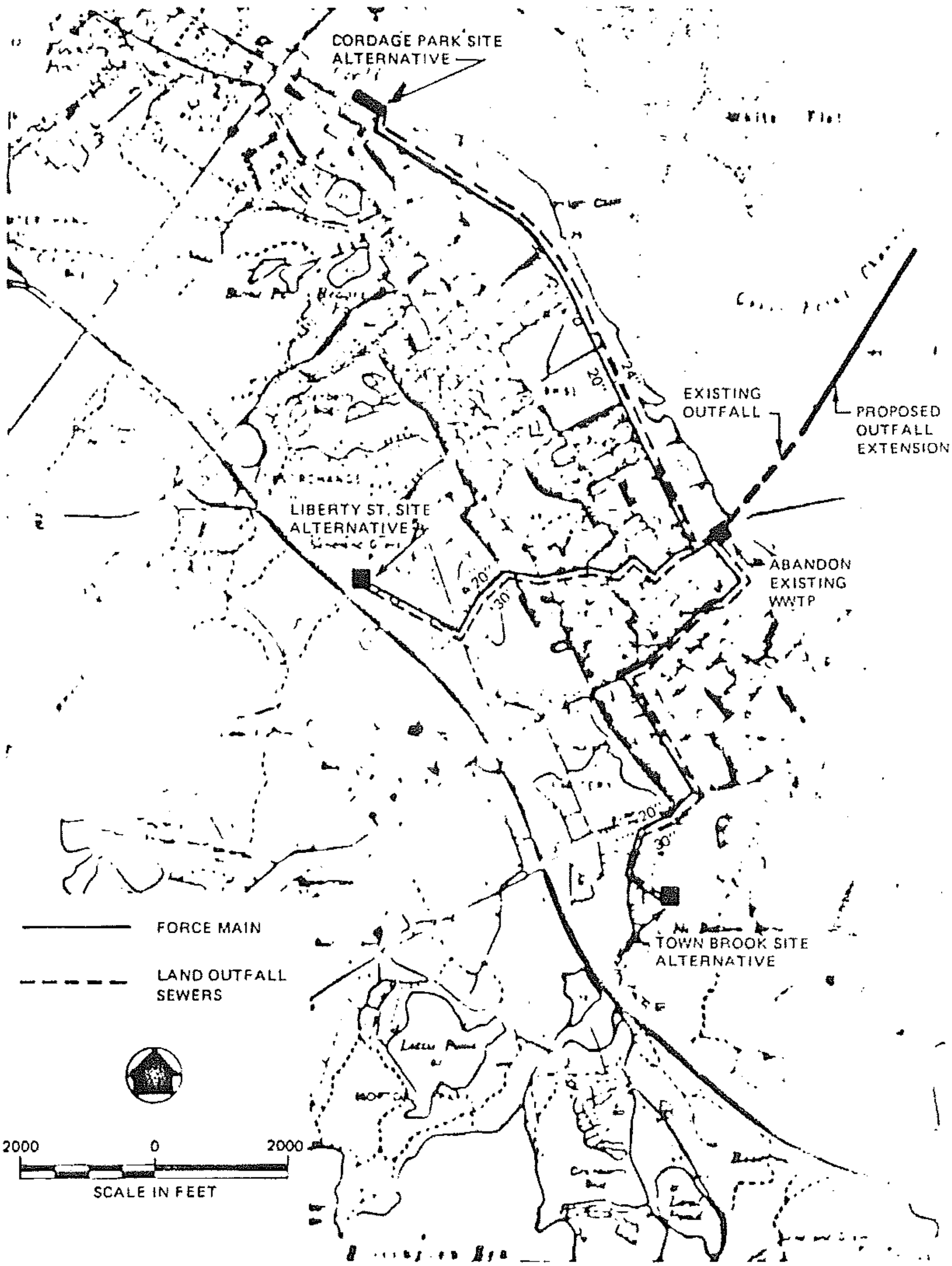


FIG. 10-4 TRANSMISSION ROUTES FOR ALTERNATIVE NORTH PLYMOUTH WWTP SITES

New Sites to Serve North Plymouth

The three alternative sites for new secondary treatment plants serving North Plymouth are shown in Figure 10-4 along with piping configurations necessary to convey the raw wastewater to each site from the existing WWTP site and return it to the outfall. Table 10-2 summarizes the pumping distances and heads associated with conveying wastewater to and from each site.

TABLE 10-2. PUMPING DISTANCES AND HEADS ASSOCIATED WITH ALTERNATIVE SECONDARY WWTP SITES TO SERVE NORTH PLYMOUTH

	Town Brook Site	Liberty St. Site	Cordage Park Site
Transmission distance from existing WWTP, ft.	7,500	6,500	9,000
Total raw wastewater pumping head at peak flow (9.5 mgd), ft.	250	225	100
Total effluent pumping head at peak flow (9.5 mgd), ft.	0	40	100

Each of the three sites has its own advantages and disadvantages, as follows:

1. Both the Cordage Park Site and the Liberty Street Site would require effluent pump stations to return treated effluent to the harbor through the existing outfall. The Town Brook site is located high enough to permit gravity flow to the harbor.

2. Both the Town Brook and Liberty Street sites would require the construction of a high head (series) pumping station at the existing WWTP site to transmit raw wastewater to the sites without intermediate pumping. A conventional pumping station would be adequate to deliver wastewater to the Cordage Park site.

3. Use of either the Town Brook or Liberty Street sites would permit the construction of a lower cost effluent holding tank than either the Cordage Park site or the existing WWTP site due to less adverse site conditions.

4. Adoption of the Cordage Park WWTP site would result in the elimination of the need for a small pump station at Hedge Road (see Figure 6-9) and reduce the delivery system costs given in Tables 6-2 and 6-3.

All tnree sites offer the following advantages over the existing WWTP site:

1. They could take advantage of reduced chlorine contact tank volume due to the detention time available in the land outfall piping.

2. Their use would permit the demolition of the existing WWTP and free most of the existing WWTP site for other, more appropriate, uses.

3. Their selection would assure the future expandability of the WWTP. The present WWTP site cannot be easily

expanded to handle the projected ultimate average flow rate of 7.9 mgd (which could occur in the future if the proposed sewer service area is fully developed in accordance with existing zoning).

The feasibility of constructing a new primary treatment plant at Rapid Infiltration Site A was also investigated. Under this alternative, raw wastewater would be pumped to Site A via a 19,000-foot long, 24-inch diameter force main, as shown in Figure 10-5. The total pumping head would be 250 feet at peak flow, necessitating series pumping. Under this alternative, a larger area of rapid infiltration basins would be required than for the disposal of secondary effluent, as discussed in Appendix D.

Cost Analysis for North Plymouth Sites

A present worth cost analysis has been prepared to evaluate comparative costs of each treatment and disposal alternative, in accordance with the assumptions given in Chapter 9. Estimated capital, annual O&M, and present worth costs are presented in Tables 10-3, 10-4, and 10-5, respectively. It should be noted that these tables do not include costs associated with collection system modifications.

From the tables, it can be seen that the alternative involving the expansion of the existing WWTP site with no extension of the outfall has the lowest present worth cost. However, as discussed in Chapter 8, this alternative has been eliminated for environmental reasons. The next lowest cost alternative is the expansion of the existing WWTP with an extension of the outfall to Location 1 (Goose Point Channel).

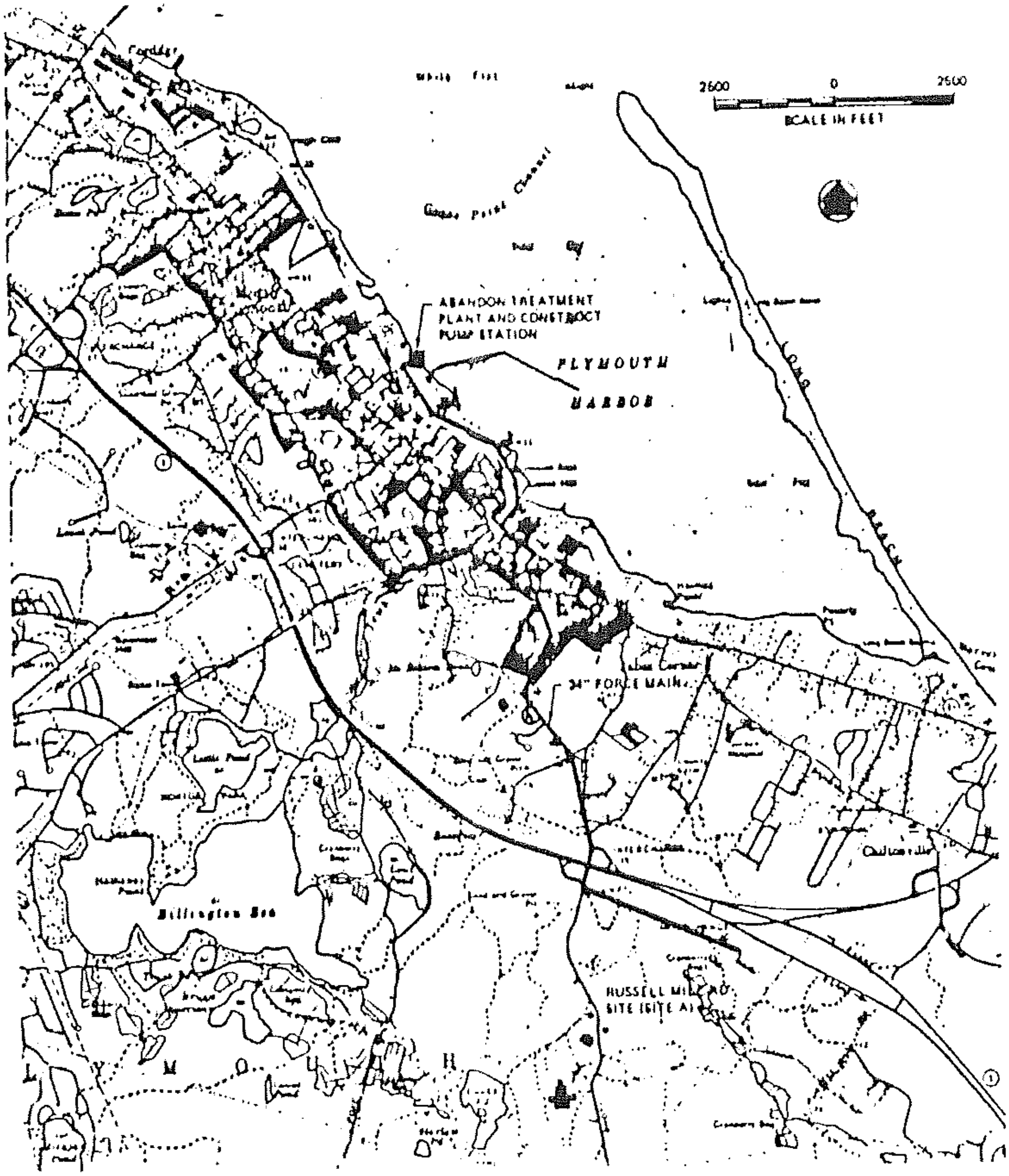


FIG. 10-5 LAND DISPOSAL OF PRIMARY EFFLUENT

TABLE 10-1. COMPARISON OF WASTEWATER TREATMENT AND DISPOSAL FACILITY CAPITAL COSTS[1]
FOR ALTERNATIVE TREATMENT SITES TO SERVE NORTH PLYMOUTH
ASSUMING 4-mgd WWTP
(All costs in thousands of dollars.)

	Secondary treatment at existing WWTP site, with lime stabilization and landfilling of sludge (Alt. P-2b)						Secondary treatment at new WWTP site with lime stabilization and landfilling of sludge (Alt. N-1a). Assuming Outfall Location 1.			Primary treatment and land disposal at R.I. Site A with lime stabilization & landfilling of sludge
	Ocean disposal of 4.0 mgd			Ocean disposal of 1.75 mgd at existing outfall location, land disposal of 2.25 mgd at R.I. Site A	Land disposal of 4.0 mgd at R.I. Site A	Land disposal of 4.0 mgd at R.I. Site DD				
	Existing outfall location	Outfall Location 1	Outfall Location 2				Town Brook WWTP site	Liberty St. WWTP site	Cordage Pk WWTP site	
Delivery System[2]										
Pump station(s)	-	-	-	-	-	-	$ 1,792	$ 1,709	$ 1,040	$ 1,870
Piping	-	-	-	-	-	-	735	566	577	1,820
Effluent holding tank[3]	$ 1,650	$ 1,650	$ 1,650	$ 900	$ 0	$ 0	1,350	1,350	1,650	0
WWTP and sludge disposal facilities	8,404	8,404	8,404	8,404	8,404	8,404	10,759	10,759	10,759	6,969
Chlorination eqt. and contact tanks	-	-	-	-	-259[4,5]	-259[4,5]	-205[6]	-169[6]	-595[6]	-[7]
Ocean disposal										
Effluent pump station	390	390	390	-	-	-	-	830	1,067	-
Land outfall	-	-	-	-	-	-	1,248	1,014	643	-
Ocean outfall and diffuser	100	2,800	5,600	100	0	0	2,800	2,800	2,800	-
Land disposal										
Pump station	-	-	-	1,037	1,589	1,552	-	-	-	0
Delivery piping	-	-	-	1,870	2,191	1,723	-	-	-	0
Rapid infiltration system	-	-	-	1,116	1,573	1,925	-	-	-	1,060
Credit for savings in collection system	-	-	-	-	-	-	-	-	-1,198	-
Total capital cost for treatment and disposal	$10,544	$11,244	$16,044	$11,497	$12,998[5]	$12,845[5]	$18,479	$18,859	$16,693	$13,219[7]

1. Costs exclude collection system and include an allowance for engineering and contingencies; ENR-4000.
2. Costs for delivery of wastewater from existing WPCP site to new WPCP site.
3. Cost assumes effluent holding tank having 2-hr. detention time at average flow rate.
4. Cost reduction due to lack of any need for contact tanks with land treatment.
5. The additional sum of $109,000 may be deducted here if disinfection is determined not to be required prior to land disposal.
6. Credit for reduction in size of chlorine contact tank.
7. The additional sum of $480,000 may be deducted here if disinfection is determined not to be required prior to land disposal at R.I. Site A.

10-14

TABLE 10-4 COMPARISON OF WASTEWATER TREATMENT AND DISPOSAL FACILITY ANNUAL O&M COSTS FOR ALTERNATIVE TREATMENT SITES TO SERVE NORTH PLYMOUTH ASSUMING 5-mgd WWTP

Item	Secondary treatment at existing WWTP site, with lime stabilization and landfilling of sludge (Alt. P-2b): Ocean disposal of 4.0 mgd: Existing outfall location	Outfall location 1	Outfall location 2	Ocean disposal of 1.75 mgd at existing outfall location, land disposal of 2.25 mgd at R.I. Site A	Land disposal of 4.0 mgd at R.I. Site A	Land disposal of 4.0 mgd at R.I. Site 00	Secondary treatment at new WPCP site with lime stabilization and landfilling of sludge (Alt. N-1a). Assuming Outfall location 1.: Town Brook WWTP Site	Liberty St. WWTP site	Cordage Pk. WWTP site	Primary treatment and land disposal at R.I. Site A with lime stabilization and landfilling of sludge
Energy										
Delivery system	-	-	-	-	.	-	$ 88,500	$ 85,000	$ 16,100(1)	$ 91,500
Effluent holding tank	$ 0	$ 0	$ 0	$ 0	.	-	0	0	0	-
WWTP and sludge landfilling	118,400	118,400	118,400	118,400	$118,400	$118,400	74,900	74,900	74,900	51,000
Offsite sludge disposal(2)	0	0	0	0	0	0	0	300	600	0
Ocean disposal	7,000	3,000	4,000	0	-	-	0	11,500	20,000	-
Land disposal(4)	-	-	-	68,000	84,000	80,300	-	-	-	0
Chemicals for WWTP	57,100	57,100	57,100	57,100	57,100(3)	57,100(3)	78,700	78,700	78,700	41,200(3)
Materials and supplies										
Delivery system	-	-	-	-	-	-	5,000	5,000	2,900(1)	5,000
Effluent holding tank	0	0	0	0	.	-	0	0	0	-
WWTP and sludge landfilling	94,200	94,200	94,200	94,200	94,200	94,200	111,000	111,000	111,000	69,400
Offsite sludge disposal(2)	0	0	0	0	0	0	0	600	900	0
Ocean disposal	500	500	500	0	-	-	0	5,000	5,000	-
Land disposal(4)	-	-	-	9,600	10,500	10,500	-	-	-	19,400
Labor										
Delivery system	-	-	-	-	.	-	7,500	7,500	5,000(1)	7,500
Effluent holding tank	0	0	0	0		-	0	0	0	-
WWTP and sludge landfilling	184,200	184,200	184,200	184,200	184,200	184,200	195,900	195,900	195,900	150,100
Offsite sludge disposal(2)	0	0	0	0	0	0	0	1,000	1,000	0
Ocean disposal	6,000	6,000	6,000	6,000	-	-	0	5,000	5,000	-
Land disposal(4)	-	-	-	18,600	22,700	22,700	-	-	-	37,500
Total Net Annual O&M Cost	$462,400	$463,400	$468,400	$536,100	$571,100(3)	$562,400(3)	$561,500	$581,400	$536,800	$472,800(3)

1. Including credit for savings in collection system costs.
2. Incremental cost for sludge hauling.
3. Deduct $9,200 if chlorination is not required for land disposal.
4. Including delivery of wastewater to disposal site.

TABLE 10-5 COMPARISON OF WASTEWATER TREATMENT AND DISPOSAL FACILITY PRESENT WORTH COSTS FOR ALTERNATIVE TREATMENT SITES TO SERVE NORTH PLYMOUTH ASSUMING 4-mgd WTP

(All costs in thousands of dollars.)

	Secondary treatment at existing WWTP site, with lime stabilization and landfilling of sludge (Alt. P-2b)						Secondary treatment at new WPCP site with lime stabilization and landfilling of sludge (Alt. N-1a). Assuming Outfall Location 1.			Primary treatment and land disposal at R.I. Site A with lime stabilization and landfilling of sludge
	Ocean disposal of 4.0 mgd			Ocean disposal of 1.75 mgd at existing outfall location, land disposal of 2.25 mgd at R.I. Site A	Land disposal of 4.0 mgd at R.I. Site A	Land disposal of 4.0 mgd at R.I. Site DD				
Item	Existing outfall location	Outfall location 1	Outfall location 2				Town Brook WPCP site	Liberty St. WPCP site	Cordage Pk. WPCP site	
Total capital cost	$10,544	$13,244	$16,044	$13,397	$17,998(1)	$12,845(1)	$18,479	$18,859	$16,963	$13,719(2)
Land acquisition										
New WWTP site	-	-	-	-	-	-	44	17	300	15
R.I. site	-	-	-	145	171	180	-	-	-	233
Sludge disposal site(3)	0	0	0	0	0	0	0	0	0	0
Interest during construction	830	1,043	1,263	1,055	1,024	1,012	1,455	1,485	1,336	1,080
P.W. of Net Annual O&M Cost	4,582	4,592	4,642	5,313	5,660(4)	5,623	5,565	5,762	5,122	4,686(4)
Less P.W. of salvage value										
Structures and piping	-573	-852	-1,136	-794	-728	-663	-1,109	-1,096	-986	-801
New WWTP land	-	-	-	-	-	-	-17	-7	-119	-6
R.I. site land	-	-	-	-58	-68	-71	-	-	-	-92
Less sale value of existing WWTP site land(5)	-	-	-	-	-	-	-480	-480	-480	-480
Total present worth	$15,383	$18,027	$20,813	$19,058	$19,057	$18,926	$23,937	$24,540	$22,136	$16,274(2,4)
Annualized cost	$ 1,552	$ 1,819	$ 2,100	$ 1,923	$ 1,923	$ 1,910	$ 2,415	$ 2,476	$ 2,234	$ 1,844
RANK	1	2	7	6	5	4	9	10	8	3

1. The sum of $109,000 may be deducted from the capital cost if disinfection is determined not to be required prior to land disposal at R.I. site.
2. The sum of $480,000 may be deducted from the capital cost if disinfection is determined not to be required prior to land disposal at R.I. site.
3. Assuming capacity is available at existing Hanover landfill.
4. The sum of $91,000 may be deducted from this total if disinfection is determined not to be required prior to land disposal at R.I. site.
5. Assuming two acres of the existing site can be sold at $300,000 per acre, less $120,000 for demolition of existing aeration tanks and settling tanks.

This alternative has been shown to be environmentally acceptable. The alternatives utilizing new secondary WWTPs are all significantly more costly than those utilizing the existing plant.

The disposal of 4 mgd of secondary effluent at either of the two land disposal sites considered would have costs similar to each other and only slightly higher than those associated with the Goose Point channel alternative. If it were politically feasible for the Town to utilize land disposal at these sites, these alternatives would be deemed "cost effective" by Federal and State regulatory agencies since land disposal is considered "alternative technology" and the present worth costs of these alternatives are within 115 percent of the least costly alternative. The land disposal alternative would thus be eligible for a higher funding percentage by Federal and State grants, and the Town's snare of capital costs would be reduced. However, the higher annual O&M costs associated with land disposal would be paid by the Town.

The present worth costs of the alternative employing a new primary treatment plant at Site A are not substantially higher than the lowest cost environmentally-sound alternatives. However, this alternative is not considered to be politically feasible in light of the substantial opposition to the application of secondary effluent at Site A (discussed in Chapter 8).

The detailed present worth cost analyses have been conducted on the basis of an average annual flow of 4 mgd (on the assumption that sewers would be extended to the intersection of Route 44 and Route 80, as discussed in Chapter 6). If the "Route 44 Service Area" (Figure 6-3) were eliminated from the system, the design flow would be reduced to approximately 3.5 mgd and the capital costs given in Table 10-3 for the expansion of the existing WWTP with ocean disposal at Outfall Location 1 (Goose Point Channel) would be reduced by the following amounts:

Flow equalization	$150,000
WWTP expansion	80,000
	$230,000

It has been proposed during this study that consideration be given to an ocean discharge alternative under which treated effluent would only be discharged during the outgoing tide, thereby providing further reduction of the effects of the discharge on the harbor. To accomplish this, additional effluent holding tank volume sufficient to store the maximum accumulated flow during 1/2 tidal cycle, or approximately 2.4 million gallons, would be required (assuming a 4 mgd plant having a peak flow of 9.5 mgd). The capital cost of such a tank would add more than $3 million to the cost of the project. As the tank would be in use every day, a flushing system would be needed to facilitate the removal of solids from the tank. In addition, the effluent pumping station would have to be sized for a capacity of 19 mgd instead of 9.5 mgd, and the capital cost of the outfall extension would probably increase by approximately $300,000 as a 42-inch diameter extension would probably be required, instead of a 30-inch diameter extension. These additional costs cannot be justified, since the increased benefits to be gained by discharging only on the outgoing tide would not be substantial.

The proposed layout of the expanded WWTP, assuming discharge to the harbor, is shown in Figure 10-6 (refer to Figures 9-3 and 9-6 for process flow diagrams and Table A-1 in Appendix A for design data). Two new settling tanks, new chlorine contact tanks, new sludge pumping equipment, a new effluent pumping station, a new emergency generator, and much of the effluent holding tank would be constructed within the present site boundaries. The remainder of the effluent holding tank, the aerated grit chambers, and the sludge processing building would be constructed on the adjacent lot. An area would be reserved on this lot for the installation of primary settling tanks should they become necessary at some future date.

New Site to Serve Manomet

Detailed evaluation was conducted of both land and ocean disposal alternatives for each of the two sewering schemes considered for Manomet. As discussed in Chapter 8 and Appendix D, the most favorable land disposal alternative would utilize Site E on the Edison Access Road. Wastewater would be pumped from the sewer service area to this site for both treatment and disposal. As discussed earlier in this chapter, Site E was also selected for locating a treatment facility in conjunction with the ocean discharge alternative for Manomet.

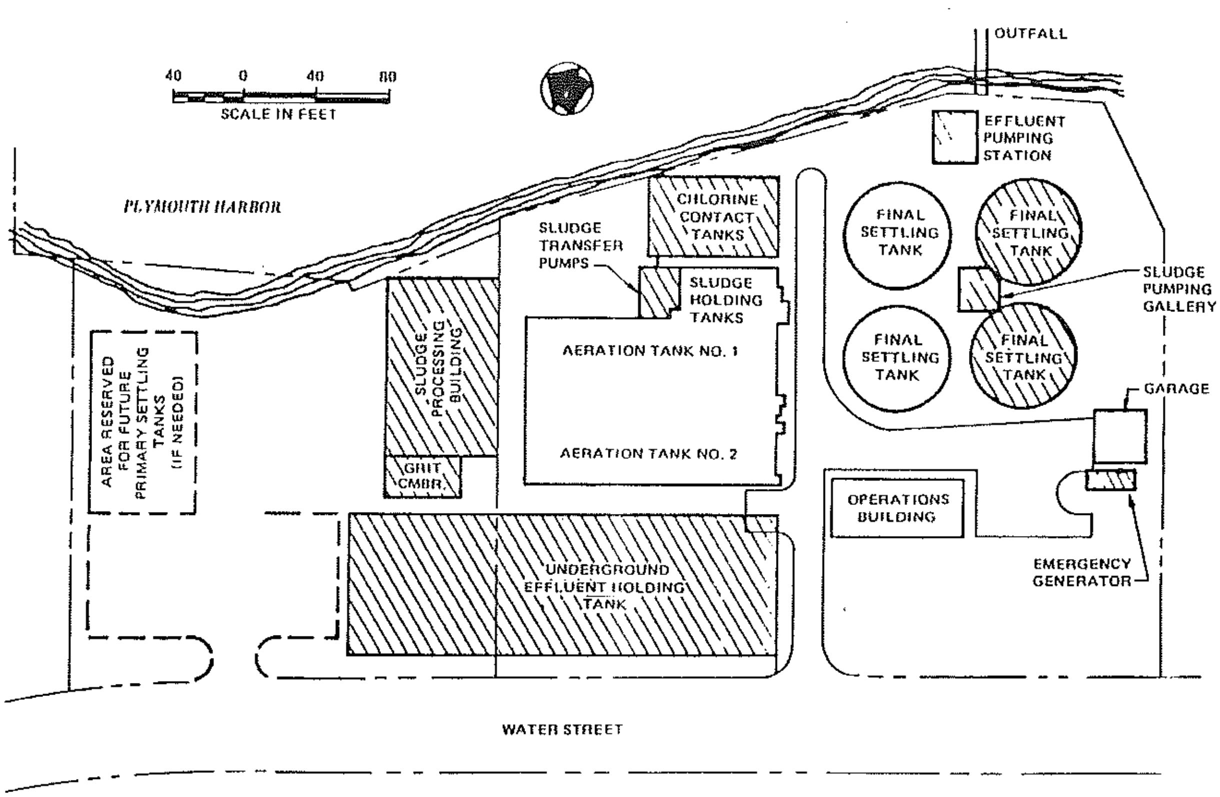


FIG. 10-6 PROPOSED LAYOUT OF EXPANDED WWTP FOR NORTH PLYMOUTH

EXISTING FACILITIES

PROPOSED FACILITIES

10-28

Alternative delivery systems investigated for Manomet are shown in Figures 10-7, 10-8 and 10-9. The first two figures depict Alternatives E-1 and E-2 for collection Schemes 1 and 3 (0.35 mgd) while the third figure depicts Alternative E-3 for collection Schemes 2, 4, 5, and 6 (0.18 mgd). Pumping station design data are given in Table 10-6.

TABLE 10-6. PUMPING STATION DESIGN DATA FOR MANOMET DELIVERY SYSTEM ALTERNATIVES

	Alt. E-1	Alt. E-2	Alt. E-3
Taylor Ave. Pumping Station			
Summer average pumping rate, mgd	0.28	0.28	0.18
Peak pumping rate, mgd.	0.7	0.7	0.45
Total head at peak flow	150	150	157
Cochituate Road Pumping Station			
Summer average pumping rate, mgd.	0.35	0.065	-
Peak pumping rate, mgd.	0.85	0.15	-
Total head at peak flow	98	67	-

Under each alternative, a force main from the Taylor Ave. Pump Station would pass through the barrier beach along Taylor Avenue. Due to the increased length of piping required to route this force main around Bartlett Pond and the need to construct a 10" and 12" gravity sewer line through the barrier beach along Taylor Avenue in any event (as discussed in Chapter 6), it is deemed most feasible to construct the force main along Taylor Avenue in the same trench as the gravity sewer line.

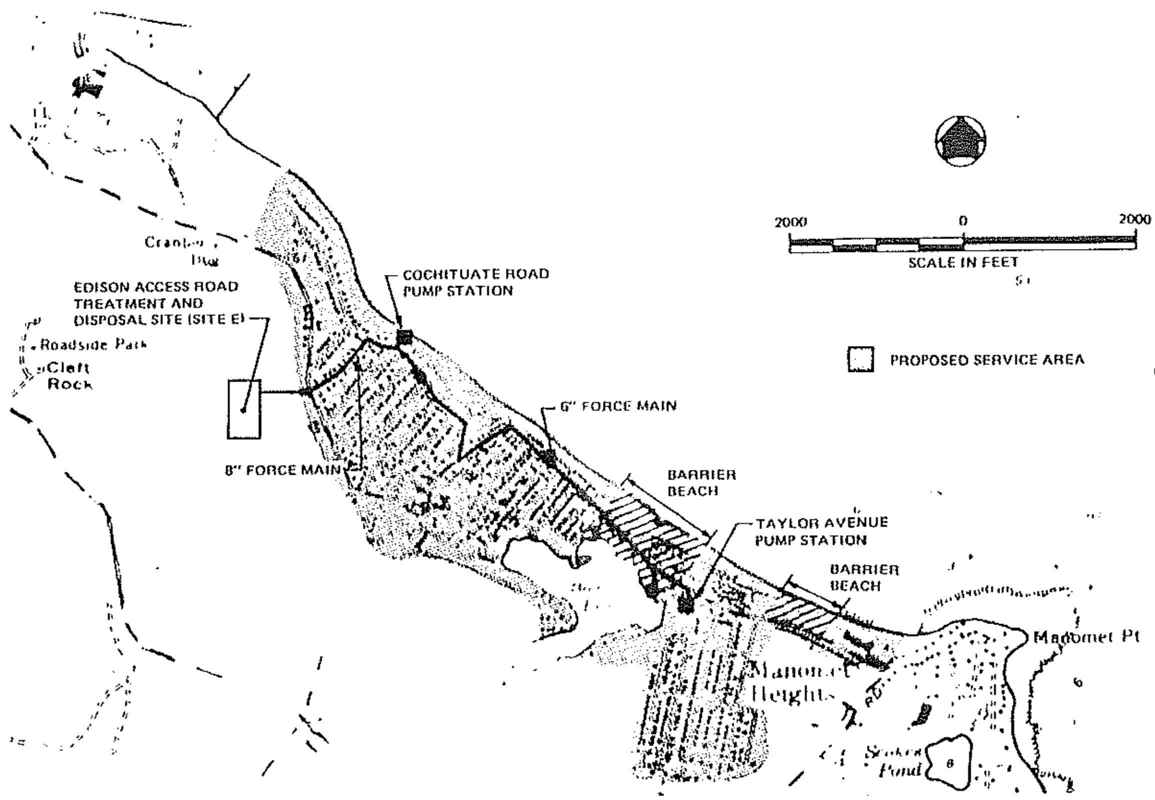


FIG. 10-7 MANOMET DELIVERY ALTERNATIVE E-1

Fig. 10-7
100% Black overlay

10-22

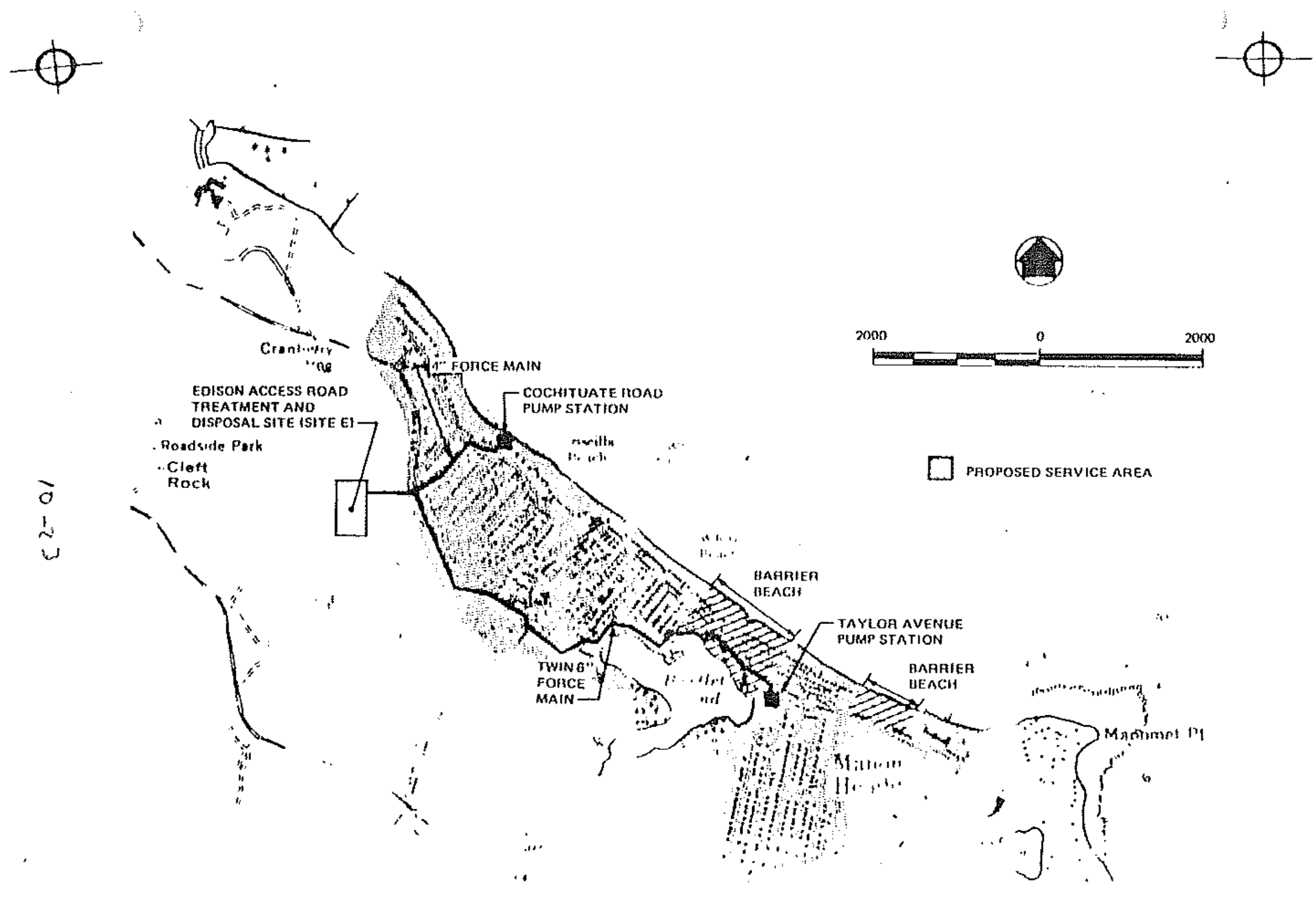


FIG. 10-8 MANOMET DELIVERY ALTERNATIVE E-2

Fig 10-8

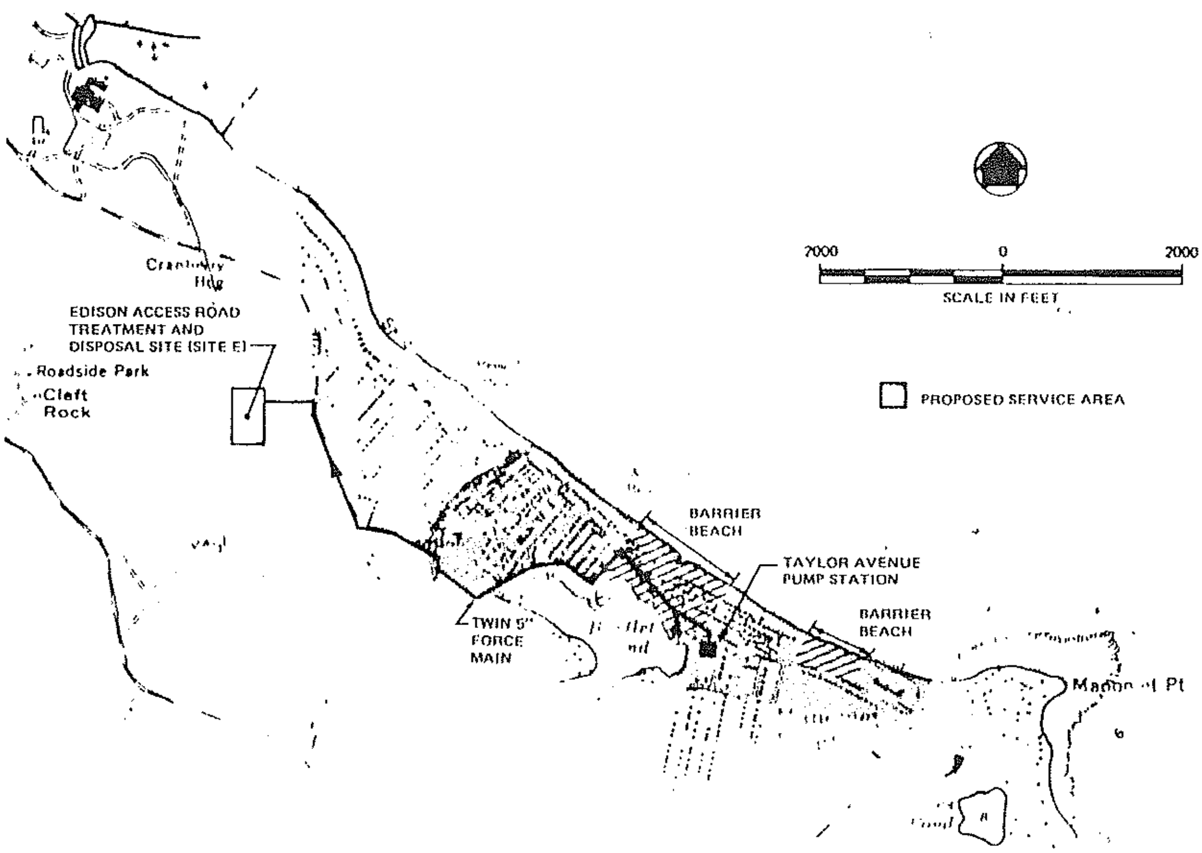


FIG. 10-9 MANOMET DELIVERY ALTERNATIVE E-3

Fig. 10-9

10-24

Cost Analyses of Manomet Disposal Alternatives

A present worth cost analysis has been prepared to evaluate comparative costs of each treatment and disposal alternative, in accordance with the assumptions given in Chapter 9. Estimated capital, annual O&M and present worth costs are presented in Tables 10-7, 10-8, and 10-9, respectively. It should be noted that the tables do not include costs associated with collection system modifications.

From Table 10-9 it is evident that the land disposal alternatives for Manomet have costs considerably less than the ocean disposal alternatives.

The proposed layouts of the Manomet WWTP assuming design average flow rates of 0.35 mgd and 0.18 mgd are shown in Figures D-20 and D-21, respectively, of Appendix D (refer to Figure 9-11 for the process flow diagram and Table A-2 in Appendix A for design data).

Joint Treatment and Disposal Alternative

The costs of a joint treatment and disposal alternative under which wastewater from North Plymouth and Manomet would be treated at a common WWTP located in Manomet and discharged to the ocean were estimated during this study and compared to the sum of the costs associated with the expansion of the existing North Plymouth WWTP with discharge to Goose Point Channel and the land disposal of Manomet wastewater at Site E. The costs are is presented in Tables 10-10 through 10-12. Assumptions used in this analysis are as follows:

TABLE 10-7. COMPARISON OF CAPITAL COSTS FOR WASTEWATER TREATMENT AND DISPOSAL FACILITY ALTERNATIVES TO SERVE MANOMET

Item	Average Flow = 0.35 mgd Aerated Facultative Lagoons (Alt. M-3)			Average Flow = 0.18 mgd Aerated Facultative Lagoons (Alt. M-5)	
	Land disposal at R.I. Site E (Delivery System Alt. E-1)	Ocean disposal at Outfall Location 4: Deliv. Syst. Alt. E-1	Ocean disposal at Outfall Location 4: Deliv. Syst. Alt. E-2	Land disposal at R.I. Site E (Delivery System Alt. E-3)	Ocean disposal at Outfall Location 4 (Del. Syst. Alt. E-3)
Delivery System*					
Pump stations	$ 520,000	$ 520,000	$ 357,000	$ 260,000	$ 260,000
Piping	214,000	262,000	462,000	338,000	386,000
Effluent holding	0	42,000	42,000	0	35,000
WWTP and sludge disposal Facilities	875,000	875,000	875,000	572,000	572,000
Chlorine contact tank**	0	26,000	26,000	0	20,000
Ocean disposal					
Land outfall	0	189,000	189,000	0	189,000
Ocean outfall and diffuser	0	803,000	803,000	0	803,000
Rapid infiltration system	575,000	0	0	355,000	0
Total capital cost for treatment and disposal	$2,184,000	$2,717,000	$2,754,000	$1,525,000	$2,265,000

* Includes pump stations, force main, and other piping necessary to convey wastewater from the collection system to the WWTP.
** Incremental cost to provide 30-minute d.t. versus 15-minute d.t. at peak flow.

TABLE 10-8. COMPARISON OF ANNUAL O&M COSTS FOR WASTEWATER TREATMENT AND DISPOSAL ALTERNATIVES TO SERVE MANOMET

Item	Average Flow = 0.35 mgd Aerated Facultative Lagoons (Alt. M-3): Land disposal at R.I. Site E (Delivery System Alt. E-1)	Average Flow = 0.35 mgd Aerated Facultative Lagoons (Alt. M-3): Ocean disposal at Outfall Location 4, Deliv. Syst. Alt. E-1	Average Flow = 0.35 mgd Aerated Facultative Lagoons (Alt. M-3): Ocean disposal at Outfall Location 4, Deliv. Syst. Alt. E-2	Average Flow = 0.18 mgd Aerated Facultative Lagoons (Alt. M-5): Land disposal at R.I. Site E (Delivery System Alt. E-3)	Average Flow = 0.18 mgd Aerated Facultative Lagoons (Alt. M-5): Ocean disposal at Outfall Location 4 (Del. Syst. Alt. E-3)
Energy					
delivery system	$ 2,400	$ 2,400	$ 1,700	$ 500	$ 500
WWTP	19,600	19,600	19,600	10,100	10,100
Chemicals	1,800	1,800	1,800	900	900
Materials and supplies					
delivery system	800	800	700	400	400
WWTP	4,900	4,900	4,900	3,900	3,900
land disposal	3,400	0	0	3,300	0
Labor					
delivery system	6,100	6,100	4,900	3,100	3,100
WWTP	22,100	22,100	22,100	19,800	19,800
land disposal	8,900	0	0	8,400	0
Total annual O&M cost	$70,000	$57,700	$55,700	$50,400	$38,700

TABLE 10-9. COMPARISON OF PRESENT WORTH COSTS FOR WASTEWATER TREATMENT AND DISPOSAL ALTERNATIVES TO SERVE HANOHET

Item	Average Flow = 0.35 mgd Aerated Facultative Lagoons (Alt. H-3)			Average Flow = 0.18 mgd Aerated Facultative Lagoons (Alt. H-5)	
	Land disposal at R.I. Site E (Delivery System Alt. E-1)	Ocean disposal at Outfall Location 4 Deliv. Syst. Alt. E-1	Ocean disposal at Outfall Location 4 Deliv. Syst. Alt. E-2	Land disposal at R.I. Site E (Delivery System Alt. E-3)	Ocean disposal at Outfall Location 4 (Del. Syst. Alt. E-3)
Total capital cost	$2,184,000	$2,717,000	$2,754,000	$1,525,000	$2,265,000
Land acquisition					
pump station sites	40,000	40,000	40,000	21,000	21,000
WWTP site	28,000	28,000	28,000	21,000	7,000
R.I. site	45,000	0	0	23,000	0
Interest during construction	172,000	214,000	217,000	120,000	178,000
P.W. of annual O&M cost	694,000	572,000	552,000	500,000	384,000
Less P.W. of salvage value					
structures and piping	-178,000	-248,000	-265,000	-128,000	-232,000
pump station sites	-16,000	-16,000	-16,000	-8,000	-8,000
WWTP site	-5,000	-5,000	-5,000	-8,000	-8,000
R.I. site	-18,000	0	0	-10,000	0
Total present worth	$2,946,000	$3,302,000	$3,305,000	$2,056,000	$2,607,000
Annualized cost	$297,000	$333,000	$334,000	$207,000	$263,000
RANK	1	2	3	1	2

TABLE 10-10. COMPARISON OF CAPITAL COSTS FOR SEPARATE VERSUS JOINT WASTEWATER TREATMENT AND DISPOSAL FACILITIES TO SERVE NORTH PLYMOUTH AND MANOMET
(assumption: average annual design flows of 4.0 mgd for North Plymouth and 0.35 mgd for Manomet)

Structure	Separate Facilities North Plymouth Alt. P-2b, with Outfall Location 1 and Manomet Alt. M-3 with Land Disposal at Site E	New Joint Facility on Edison Access Road in Manomet with Ocean Discharge at Outfall Location 4
Delivery system		
Pump stations - No. Plymouth	-	$3,357,000
Pump stations - Manomet	$ 520,000	520,000
Piping - No. Plymouth	-	3,380,000
Piping - Manomet	214,000	262,000
Effluent holding		
No. Plymouth	1,650,000	-
Joint WWTP	-	67,000
WWTP and sludge disposal facilities		
No. Plymouth	8,404,000	-
Manomet	875,000	-
Joint WWTP	-	11,251,000
Ocean disposal		
Land outfall - joint WWTP	-	452,000
Ocean outfall - North Plymouth	2,800,000	-
Ocean outfall - joint WWTP	-	2,450,000
Effluent pump sta. - No. Ply.	390,000	-
Land disposal		
Manomet	575,000	-
Total capital cost	$15,428,000	$21,739,000

TABLE 10-11. COMPARISON OF ANNUAL O&M COSTS FOR SEPARATE VERSUS JOINT WASTEWATER TREATMENT AND DISPOSAL FACILITIES TO SERVE NORTH PLYMOUTH AND MANOMET (assumption: average annual design flows of 4.0 mgd for North Plymouth and 0.35 mgd for Manomet)

Item	Separate Facilities North Plymouth Alt. P-2b, with Outfall Location 1 and Manomet Alt. M-3 with Land Disposal at Site E	New Joint Facility on Edison Access Road in Manomet with Ocean Discharge at at Outfall Location 4
Energy		
Delivery system - North Plymouth	$ -	$ 142,000
Delivery system - Manomet	2,400	2,400
WWT - North Plymouth	118,400	-
WWTP - Manomet	19,600	-
WWTP - joint	-	82,400
Ocean disposal - North Plymouth	3,000	-
Chemicals		
No. Plymouth	57,100	-
Manomet	1,800	-
Joint	-	86,600
Materials and Supplies		
Delivery system - North Plymouth	-	10,000
Delivery system - Manomet	800	800
WWTP - North Plymouth	94,200	-
WWTP - Manomet	4,900	-
WWTP - joint	-	122,100
Ocean disposal - North Plymouth	500	-
Land disposal - Manomet	3,400	-
Labor		
Delivery system - North Plymouth	-	15,000
Delivery system - Manomet	6,100	6,100
WWTP - North Plymouth	184,200	-
WWTP - Manomet	22,100	-
WWTP - joint	-	215,000
Ocean disposal - North Plymouth	6,000	-
Land disposal - Manomet	8,900	-
Total annual O&M cost	$533,400	$682,400

TABLE 10-12. COMPARISON OF PRESENT WORTH COSTS FOR SEPARATE VERSUS JOINT WASTEWATER TREATMENT AND DISPOSAL FACILITIES TO SERVE NORTH PLYMOUTH AND MANOMET
(assumption: average annual design flows of 4.0 mgd for North Plymouth and 0.35 mgd for Manomet)

Item	Separate Facilities No. Ply. Alt. P-2b with Outfall Location 1 and Manomet Alt. M-3 with Land Disposal at Site E	New Joint Facility on Edison Access Rd. in Manomet with Ocean Discharge at Outfall Location 4
Total capital cost	$15,428,000	$21,739,000
Land acquisition		
Pump station sites - No. Plymouth	-	16,000
Pump station sites - Manomet	40,000	40,000
WWTP site - Manomet	28,000	-
WWTP site - joint	-	52,000
RI site - Manomet	45,000	-
Interest during construction	1,215,000	1,712,000
P.W. of annual O&M Cost	5,286,000	6,763,000
Less P.W. of salvage value		
Structures and piping - No. Ply.	-852,000	-
Structures and piping - Manomet	-178,000	-
Structures and piping - joint	-	-1,183,000
Pump station sites - No. Plymouth	-	-6,000
Pump station sites - Manomet	-16,000	-16,000
WWTP site - Manomet	-5,000	-
WWTP site - joint	-	-10,000
RI site - Manomet	-18,000	-
Total present worth	$20,973,000	$29,107,000
Annualized cost	$ 2,116,000	$ 2,937,000

1. The existing WWTP on the waterfront would be abandoned.
2. The new WWTP would have a design average flow capacity of 4.5 mgd.
3. The new WWTP would be located at Site E on the Edison Access Road.
4. The outfall would be at Outfall Location 4 and would discharge at a depth of 30 feet.

Raw wastewater for North Plymouth and Manomet would be transmitted along the routes shown in Figure 10-10. Two pump stations (one at the existing WWTP site and one in the vicinity of Bert's Restaurant) and 33,000 feet of force main would be required to deliver the North Plymouth wastewater to the new WWTP.

From Table 10-12, it is evident that the costs of this combined flow alternative are much higher than those of two separate facilities. Consideration was also given to pumping treated wastewater to Manomet to the Russell Mill Road site for joint land disposal with North Plymouth wastewater. This alternative was dropped when preliminary calculations showed that the cost of local disposal of Manomet wastewater would be substantially less than that of delivering it to the North Plymouth site.

Proposed Ocean Spray Site Expansion

Ocean Spray Cranberries, Inc., which is headquartered adjacent to the WWTP, would like to expand its ofices onto the

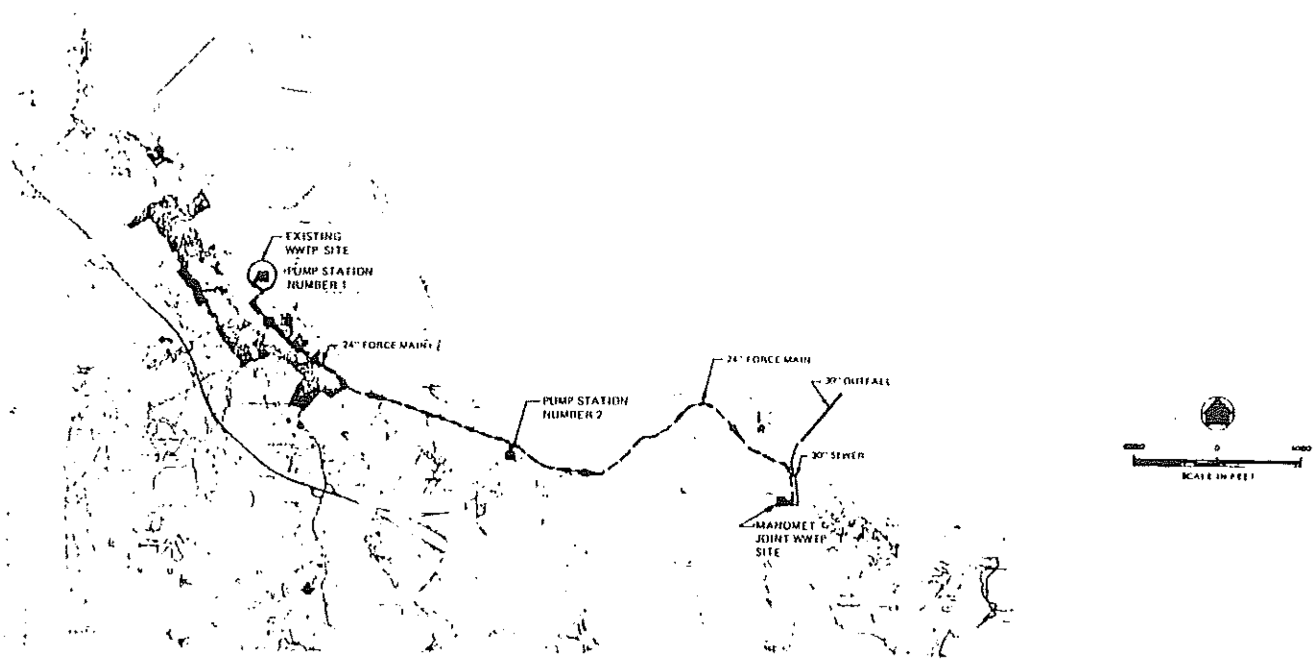


FIG. 10-10 JOINT TREATMENT AND
DISPOSAL ALTERNATIVE

FIG 10-10 OVERLAY
100% SOLID Black

Town-owned lot being considered for use in the expansion of the WWTP. Ocean Spray has proposed to fill in 12.5 acres of tidelands between the breakwater and its property, of which 4.5 acres would be a gift to the Town for its use in expanding the WWTP, in exchange for the 1.2 acre lot presently used by Ocean Spray as a parking lot. Ocean Spray's proposal would also create a shellfish hatchery, a boat docking area, an aquaculture facility to harvest seaweed, and a new visitors center for Cranberry World.

Although the Town conservation commission, selectmen and CAC have thus far favored the plan, the approval of State regulatory agencies remains in doubt. Ocean Spray plans to file the necessary applications early this summer.

Should the 4.5 acres of filled land become available to the Town, the layout of some new WWTP facilities would need to be revised. The additional area offered by this site may permit the planning of a plant with further expansion capabilities.

CHAPTER 11

RECOMMENDED PLAN

General

This chapter presents a description of the proposed facilities and identifies issues that must be resolved prior to the design phase of the project. The wastewater needs of citizens not scheduled to receive public sewerage under this recommended plan are also considered.

Areas to be Served by Sewers

North Plymouth Phase I. Several problems to be corrected during the initial phase of this project include:

1. elimination of inadequate treatment by increasing the design capacity of the WWTP to 4 mgd to handle present flows with some reserve for service area expansion into limited portions of West Plymouth, elimination of plant bypassing, and upgrading to include improved disinfection,improved grit removal, and mechanical sludge dewatering, and increased emergency power capacity;
2. further reduction of the risk of WWTP contamination of shellfish beds in Plymouth Harbor by constructing an effluent "flow holding" tank;
3. elimination of the existing discharge at a mean low water depth of 6.5 feet by extending the outfall to Goose Point Channel, where the mean low water depth of 14 feet will provide additional dilutions;

4. elimination of excessive I/I in the existing collection system;
5. replacement of the deteriorated Cordage and Harbor interceptors and the Knapp Terrace Pump Station with new interceptors and a new Cordage Pump Station;
6. elimination of a developing capacity bottleneck at the Cherry St. Pump Station by replacing the station;
7. improvements to the existing Night Soil Receiving Facility to eliminate odor and rag problems; and
8. abandonment of the existing sludge drying beds and expansion of the Manomet Landfill into the area now occupied by the beds.

No extensions to the sewer system would be constructed by the Town under Phase I. New sewer tie-ins would be permitted at the expense of the homeowner or industrial user within the existing service area. Further development within the industrial park overlying the recharge area of the North Plymouth Well would be permitted on the condition that the developer connect the new facilities to the sewer system. Capacity for up to 0.42 mgd of additional wastewater flow from the industrial park would be provided at the WWTP, including 0.14 mgd from areas served by the existing sewer.

North Plymouth - Phase II. Expansion of the service area to serve the "Route 44 Service Area" depicted in Figure 6-3 will be deferred until Phase II. This area, which includes commercially and residentially-zoned land, also overlies the recharge area of the North Plymouth Well and is theref presently subject to development limitations imposed by the Town's Aquifer Protection Bylaw. A wastewater flow of 0.55 mgd would be expected from this area by the year 2007. It is proposed that the Route 44 and Algonquin Pump Stations and all new piping to the west of Route 3 shown in Figure 6-9 be constructed during Phase II. The costs of providing these facilities must be borne completely by the Town because no problem presently exists in this proposed service area.

North Plymouth - Issues to be Resolved. Before the Town can construct an outfall extension and obtain a permit to discharge 4 mgd of treated wastewater to Plymouth Harbor, it must obtain a variance to the Massachusetts Ocean Sanctuaries Act, which presently restricts Plymouth's discharge to 1.75 mgd. Two approaches to obtaining this variance are possible. Under the first approach, a bill would be filed in the legislature amending the act to permit the DWPC to consider granting variances on a case-by-case basis. The second approach is to file a bill in the legislature to obtain a special variance for Plymouth. A bill taking the first approach has been filed by Plymouth's state representative. If the bill is passed, Plymouth should immediately apply for a variance. If it doesn't pass, Plymouth should take the second approach and request a variance by special legislation. If both approaches turn out to be dead ends, the Town will be forced reconsider land disposal of at least part of its effluent. In any event, it is recommended that the Town proceed immediately with the design of those facilities which are common to both land and ocean disposal alternatives in order to eliminate present problems as soon as possible by improving treatment.

A second isssue remaining to be resolved is the need to better define the peak inflow rate at the WWTP. Attempts are being made to quantify the peak flow rate during bypassing events to permit optimum design of plant modifications and eliminate future bypassing. It is hoped that flow monitoring being conducted this spring as part of the ongoing SSES work in Plymouth will provide a better picture of peak inflow rates. The Town should also attempt to resolve the problems of bacterial contamination from miscellaneous pipes and storm drains identified in Chapter 4.

A third issue remaining to be resolved is the proposed Ocean Spray site expansion, which would make 4.5 acres of filled tidelands available to the Town for expansion of the WWTP, as discussed in Chapter 10. If this proposal is approved by State regulatory agencies, the site plan will be revised as necessary during the design phase.

A meeting should be set up with the owner of the railraod right of way proposed to be used for the relocated Cordage Interceptor early during the design phase to obtain early approval and avoid construction delays.

Manomet - Phase I. Severe problems have been identified in Area A of Section 1 of Manomet (shown in Figure 6-1). The problems include evidence of at least occasional bacterial contamination of Bartlett Pond and White Horse Beach and widespread noncompliance with Title 5 of the State Sanitary Code. In view of the high density of development in this area (lot sizes of 1,300 - 5,000 square feet), the limited availability of vacant land for expansion of on-lot disposal systems, and the high water table, the continued use of on-lot systems in this area is not recommended. Improved operation and maintenance on the existing on-lot systems would be helpful but would likely have a small positive impact. New on-lot systems cannot be installed in Area A because variances to the codes will not be granted by local public health authorities and the DEQE. Provision of sewers for this area was shown to have several advantages over the other alternatives considered, and it is thus recommended that sewers be installed in this area immediately.

Wastewater would be collected as shown in the Phase I area outlined in Figure 6-2 except that pressure sewers would be provided instead of gravity laterals within the barrier beaches. Pump stations would be constructed on Taylor Avenue and Cochituate Road to deliver the wastewater to the treatment and disposal facility to be located on the Edison Access Road. The force mains shown in Figure 10-7 would all be constructed during Phase I. The treatment and disposal facility would be constructed to have a capacity of 0.35 mgd to account for expansion of the collection system during the planning period.

Manomet - Phase II. Problems with the continued use of on-lot systems in the remainder of Section 1 of Manomet (Area B in Figure 6-1) have also been identified.

Lots in this area are also too small to comply with Title 5. Improved operation and maintenance practices and water conservation may extend the life of on-lot systems in this area, but these measures will not provide a long-term solution to wastewater disposal problems. While variances to the code in this area are routinely granted, the presence of a sewer system and treatment and disposal facility serving Area A would make it practical to provide sewers for Area B as well. State funding of sewer construction in this area under the Chapter 557 program is possible as long as a two-year period separates construction (see Appendix K). Therefore, sewers should be provided for Area B during Phase II.

Mamomet - Issues to be Resolved. Before sewers may be installed within the barrier beach areas of Manomet, it is likely that CZM will require further water quality sampling along Bartlett Pond, Beaver Dam Brook, and White Horse Beach. In addition, they may require the Town to conduct a detailed survey of septic systems on the beaches to answer the questions included in their February 3, 1983 memorandum to DWPC (included in Appendix K). These actions have been proposed by CZM to demonstrate the existence of a water quality problem in this area.

A second issue to be resolved in Manomet is the availability of the proposed treatment and disposal site on the Edison Access Road. This land is located within the buffer zone of the Pilgrim Nuclear Power Station and is currently being managed as forestry land, as discussed in Boston Edison's letter of November 15, 1982 (included in Appendix K). The Town should meet with Boston Edison and the Department of Environmental Management forestry people to determine what steps are required to reserve this land for use as a treatment and disposal site. Additional borings and infiltration testing will also be necessary during design of the disposal facilities.

<u>Recommended Areas for On-Lot Disposal Systems</u>

The key to evaluating an area being considered for possible sewering lies in the determination of the viability of on-lot disposal systems over the design period of twenty years. These probabilistic determinations must be made on past experience with similar soils, similar density of development, systems maintenance practices and existing on-lot problems in the area.

As widespread problems with on-lot systems do not exist in other areas of Plymouth, and as lots in those areas having reported problems generally are of adequate size to accommodate replacement systems when failure occurs, it is recommended that the use of on-lot systems be continued outside the proposed sewer service areas.

The following measures are recommended to prolong the effectiveness and useful life of on-lot systems in the unsewered areas of Plymouth:

1. Encourage or require the use of water-saving devices in all new houses throughout Plymouth and encourage their installation in existing houses in Saquish and Gurnet Point. Installation of water-saving devices can improve the operation of on-lot systems and prolong their use, chiefly through reduction in the quantify of wastewater and through maintenance of aerobic conditions in the leaching area, which reduces the potential for clogging.
2. Encourage or require improved O&M practices for septic tank and cesspool pumping townwide, as discussed in Chapter 5. Timely pumping eliminates high buildup of solids which can carry over into and clog leaching areas during hydraulic surges.
3. Prohibit the installation of garbage grinders. Although the code requires additional capacity where grinders are installed, their use is not recommended for on-lot systems. The resulting increase in grease and solids can significantly reduce the life of a system.

Enactment of these measures cannot quarantee that on-lot systems in the nonsewered areas will remain viable over the full planning period. The success of such measures in prolonging the life of an on-lot system can only be predicted in probabilistic terms. In other words, if the actions are taken, the chances of prolonging on-lot system use and postponing the extension of sewers is greatly enhanced.

An issue remaining to be resolved is the question of the potential existence of a wastewater-related problem in the Eel River, as noted in Chapter 4. Further study will be necessary to resolve this question.

North Plymouth Treatment and Disposal Facilities

General. As indicated in Chapter 10, the recommended wastewater treatment and disposal plan is to increase the capacity of the existing WWTP to 4 mgd and discharge disinfected secondary effluent to Goose Point Channel in Plymouth Harbor. Sludge and scum generated from the treatment process will be stabilized with lime and dewatered and buried in the Manomet Landfill.

Design Criteria. Because of changing technologies, changes in water quality standards and anticipated life of mechanical equipment, it is generally practical to design wastewater treatment facilities for a design life of 20 years. Because a 3 or 4 year period is required for design and construction, the year 2007 has been selected as the design year. Flows and loadings for the design year were developed in Chapter 7 and are summarized in Appendix A.

Wastewater Treatment Process Description. A schematic process flow diagram for the recommended treatment system is shown in Figure 11-1. A plant layout is shown in Figure 10-6. The recommended treatment process includes:

1. Bar screening, influent pumping and flow metering
2. Grit removl
3. Biological treatment using activated sludge
4. Solids settling
5. Disinfection with chlorine
6. Effluent pumping
7. Emergency flow holding
8. Ocean discharge

A brief description of the recommended treatment processes follows:

1. Bar Screening, Influent Pumping and Flow Metering. Wastewater entering a treatment facility contains rags and other large solids which can clog downstream equipment and adversly affect treatment. The existing mechanically-cleaned bar rack in the Operations Building will be replaced with a larger unit sized for the increased flow, and minor structural modifications will be made to the existing inlet channel and overflow weir structure. The existing raw wastewater pumps will be replaced with four new 3.2-mgd variable-speed pumps, and alterations will be made to the existing piping and valves. The flow tube will be replaced with a unit of 10-mgd capacity.

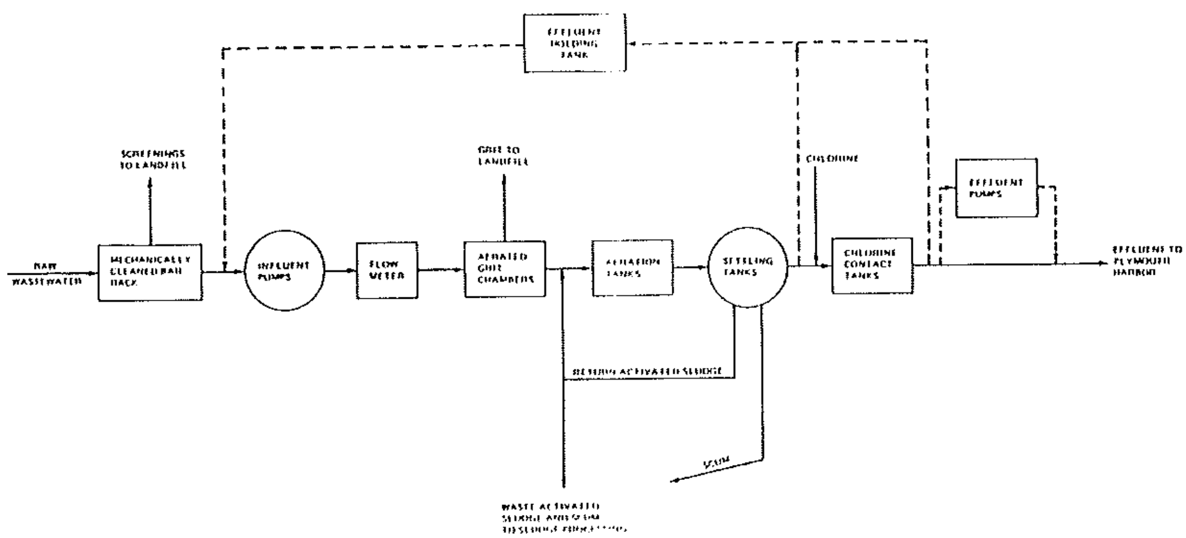


FIG. 11-1 RECOMMENDED PROCESS FLOW DIAGRAM FOR NORTH PLYMOUTH WWTP

2. Grit Removal. Raw wastewater also contains grit such as sand and other heavy solids which can cause premature wear on mechanical equipment and settle in process tankage, reducing usable capacity. The existing cyclone grit removal system will be replaced with two aerated grit chambers having improved capacity. These chambers will be covered, and exhaust gasses vented to an odor control system using activated carbon. Grit will be periodically removed from the chambers using a new vac-all truck, which can also be used by the Town for cleaning catch basins.

3. Biological Treatment Using Activated Sludge. Following grit removal, the wastewater will pass to the aeration tanks for secondary treatment. The purpose of secondary treatment is to remove organic materials and nonsettleable solids by bio-oxidation and by flocculation and settling.

   Oxygen introduced to the wastewater by mechanical aerators in the aeration tanks permits the growth of micro-organisms which break down and sythesize the oxidizable organics into new cells, carbon dioxide and water. The cell material or activated sludge is then settled in the settling tanks. A portion of the sludge is wasted and must be disposed of. The rest is returned to the aeration tanks to maintain the process.

The two aeration tanks, which were originally designed to be operated in the extended aeration variation of the activated sludge process (see Chapter 3) will be modified to operate as a conventional activated sludge system (see Chapter 9). The organic loading rate will reach 43 lb BOD_5 per 1000 cu. ft. per day under average conditions in the design year (2007). The aerators will be replaced with larger units and the tanks compartmentalized to permit flexibility for operation in the complete mix, plug flow, step aeration and contact stabilization modes.

4. Solids Settling. Two new 55-foot diameter settling tanks will be added to supplement the two existing tanks of the same size. The overflow rate at peak flow will be 1,000 gallons per square foot per day versus the value of 1,200 gallons per square foot per day normally used for design. It is felt that making all four tanks the same size will assure hydraulic balance and provide an extra means of safety to assure consistent solids reduction below the permit limitation of 30 mg/L suspended solids. The outer walls of these tanks will be raised, if necessary, to protect the tanks from the 100-year frequency flood elevation being established currently by the Federal Emergency Management Administration. New waste

and return activated sludge pumps will replace the existing pumps.

5. Disinfection With Chlorine. Prior to discharge, secondary effluent will be disinfected, as is presently done, with chlorine to reduce the presence of harmful bacteria to acceptable levels. To optimize efficiency, it is proposed that flash mixing be provided at the point of chlorine addition. Two new chlorinators will be provided in the existing chlorinator room, and two contact tanks having a detention to the period of 30 minutes at the peak flow rate will be constructed. Features to assure reliable duplicate disinfection will include chlorine cylinder scales, a standby injection units pump, an emergency power supply, automatic changeover from one cylinder to the next, a chlorine residual analyzer/recorder to ensure reasonable effluent while protecting against excessive toxic chlorine, and alarms on each chlorinator to warn of loss of vacuum or low chlorine pressure. Chlorine leak detectors will also be provided to safeguard plant workers.

6. Effluent Pumping. Because the outfall will be extended 3200 feet and the peak flow rate will be nearly doubled, friction losses in the outfall pipe will increase and pumping will be required to permit the discharge of peak flows during high tide

conditions. Three low-head axial-flow pumps are proposed for intermittent use when pumping is required.

7. Emergency Flow Holding Tank. A 1.2-million gallon underground holding tank will be provided to prevent the discharge of inadequately treated wastewater during times of plant upset or, at a minimum, delay such a discharge until shellfishermen have been warned. It will be possible to direct effluent from the settling tanks or chlorine contact tanks to the holding tank during such occurrences, which are expected to be rare. The tank will be unmixed, have a sloped bottom to facilitate washdown, and drain by gravity to the wet well at the head of the plant.

8. Ocean Discharge. The existing 30-inch diameter outfall pipe will be extended approximately 3200 feet to Goose Point Channel. Here, treated effluent will be discharged through a multi-port diffuser (see Appendix B) at a depth of 14 feet at mean low tide.

Sludge Process Description. A schematic process flow diagram for the recommended sludge handling system is shown in Figure 9-7. The recommended processes include:

1. Aerobic sludge holding
2. Continuous belt filter press dewatering
3. Lime stabilization
4. Sludge landfilling

A brief description of the recommended process follows:

1. Aerobic Sludge Holding. Sludge wasted from the settling tanks will be pumped to the existing aerobic digesters, which will be modified for use as sludge holding tanks. The existing surface aerators, which cannot operate during the winter months due to freezing problems, will be replaced with submerged turbine aerators. Blowers to provide air to the tanks would be housed in the solids processing building or the new sludge transfer pump gallery adjacent to the holding tanks.

2. Continuous Belt Filter Press Dewatering. Sludge will be pumped from the holding tanks to the new sludge processing building by new progressing cavity pumps housed in the new sludge transfer pump gallery adjacent to the holding tanks. The decanted sludge is expected to have a solids content of about 2 1/2 percent. The sludge will be conditioned with polymer and fed to two new 1.5-meter belt filter presses. The presses are expected to dewater the sludge to a solids content of about 18 percent. Filtrate will be collected in a sump and returned to the aeration tanks. The sludge cake will be scraped off the belts and delivered to the lime stabilization system. Odor control would be provided by scrubbers.

3. Lime Stabilization. Lime will be blended with the sludge cake in a Roediger System housed in the solids processing building. After blending, the stabilized cake will be discharged to a truck for conveyance to the Manomet Landfill.

4. Sludge Landfilling. Under the recommended plan, the Manomet Landfill will be expanded into the sludge drying bed area. The volume available in this area may be maximized by excavating sand below the beds before installing the liner and leachate collection system. Leachate from the 2-acre area will be collected and trucked to the WWTP for disposal. Additional sludge-only landfill capacity will be made available by increasing the design height of the existing portion of the landfill by 15 feet, as discussed in Chapter 9. Because the available space is expected to be inadequate to accommodate sludge throughout the entire 20-year planning period, consideration will be given during design to the provision on the site of a paved compost pad to provide flexibility for sludge disposal during warm weather periods and extend the life of the landfill.

Night Soil Disposal Facility. The existing facility on Long Pond Road will be modified by constructing a structure to completely enclose the septage trucks during unloading. A scrubber will be installed for odor control, and improved grit and screenings removal equipment will be provided. The facility will remain unmanned.

Manomet Treatment and Disposal Facilities

General. As indicated in Chapter 10, the recommended wastewater treatment and disposal plan is to construct a 0.35 mgd treatment and land disposal facility on the Edison Access Road.

Design Criteria. This facility, like the North Plymouth facility, would be designed for year 2007 flows. Flows and loadings for the design year were developed in Chapter 7 and are summarized in Appendix A.

Wastewater Treatment Process Description. A schematic process flow d' m flor the recommended treatment system is shown in Figure 9-12. A site layout is shown in Figure D-20. The recommended treatment processes include:

1. Preliminary treatment
2. Aerated facultative lagoons
3. Disinfection with sodium hypochlorite
4. Disposal by rapid infiltration

A brief description of the recommended processes follows:

1. Preliminary Treatment. All raw wastewater would be screened to remove rags and other large solids prior to further treatment.
2. Aerated Facultative Lagoons. Three 12-foot deep lagoons are proposed, one having an area of 1.4 acres and the remaining two having areas of 0.8 acres each. The lagoons would be provided with impermeable

liners and diffused air aeration systems, and would be designed to permit parallel or series operation. Provision would be made to add a fourth lagoon in the future to accomodate flows from Area C.

3. Disinfection with Sodium Hypochlorite. All effluent from the lagoons would be disinfected with liquid sodium hypochlorite (about 8 gallons per day would be required at design flows) and passed through a contact tank having a detention period of 15 minutes at peak flow.

4. Disposal by Rapid Infiltration. After treatment and disinfectin, the effluent will flow by gravity to three rapid infiltration basins for disposal. These basins will have an area of 0.87 acres each and will be rested regularly to permit air drying and prevent clogging. Only one basin will be in use at any time. Because of the sandy nature of the soil at this site, it is expected that no ponding will occur in the basins. The effluent will percolate through the soil and flow the path of groundwater follow to the ocean. Space will be reserved for a fourth 0.87-acre basin to be constructed in the future when the Village Center service area (Area C) is sewered. Monitoring wells would be provided to determine that the

discharge does not result in violation of the Massachusetts Groundwater Quality Standards.

Cost of the Recommended Plan

The estimated capital costs for the recommended plan are presented in Table 11-1. It should be noted that these costs are based on an ENR index of 4000 (February 1983), and actual costs as likely to be 20 percent higher (due to inflation) by the time the bids are taken. Estimated annual O&M costs (including energy, chemicals, materials, supplies, and labor) are presented in Table 11-2.

TABLE 11-1. ESTIMATED CAPITAL COSTS* OF RECOMMENDED PLAN

North Plymouth	
Phase I	
Delivery system	
Cordage Interceptor	$ 2,874,000
Harbor Interceptor	704,000
I/I reduction	467,000
Subtotal	$ 4,045,000
Night soil disposal facility	255,000
WWTP and sludge disposal modifications	8,404,000
Effluent holding tank	1,650,000
Effluent pump station	390,000
Outfall extension	2,800,000
Total Phase I	$17,544,000
Phase II (Route 44 Service Area)	
Delivery system	$ 2,770,000
Total Phases I and II	$20,314,000
Manomet	
Phase I (Area A)	
Collection system	$ 2,060,000
Delivery system	734,000
Treatment and disposal system	1,450,000
Land acquisition	113,000
Total Phase I	$ 4,357,000
Phase II (Area B)	
Collection system	$ 2,316,000
Total Phases I and II	$ 6,673,000
Total Plan Capital Cost	$26,987,000

* Excluding house connection costs, which would be paid by the homeowner. Costs given include an allowance for engineering and contingencies and are based on an ENR index of 4000 (February 1983).

TABLE 11-2. ESTIMATED ANNUAL O&M COSTS TO TOWN

	Initial Year (1987)	Design Year (2007)
North Plymouth		
Collection and delivery system	$ 69,000*	$ 97,000
Night soil disposal facility	1,600	1,600
WWTP, ocean disposal and sludge landfilling	389,700	463,400
Subtotal	$ 460,300**	$ 562,000**
Manomet		
Delivery system	$ 8,100	$ 9,000
Treatment and disposal	46,100	61,000
Subtotal	$ 54,200	$ 70,000
Total	$ 514,600	$ 621,600

* Based on present annual cost of $69,000 (fiscal year 1983).
** this number may be compared with $349,000 budgeted for the sewer division in fiscal year 1983.
Note: Costs are based on 1983 prices.

CHAPTER 12

FINANCING AND IMPLEMENTATION

General

The purpose of this chapter is to present the possible methods for financing costs for constructing, operating and maintaining the wastewater system.

Sources of Funds

In Massachusetts, funds for financing the construction of wastewater facilities are commonly raised from a combination of four principal sources:

1. Federal and/or State grants.
2. General obligation bonds paid from taxes on real estate.
3. Betterment assessments upon properties which abut new collection sewers.
4. Sewer service charges to sewer users.

The first three items - grants, real estate taxes and betterment assessments - are commonly used to finance bond payments for initial construction costs. The fourth item, the sewer service charge, is used to recover the operation and maintenance costs.

Federal and State Grants. Presently there are three major construction grants programs, one Federal and two State, under which grants may be obtained for construction of eligible facilities, such as wastewater treatment plants, interceptor sewers, pump stations, and sewers.

Under Public Laws 92-500 and 95-217, the Federal Water Pollution Control Act Amendments of 1972 and 1977, eligible facilities can receive grants of 75 percent of the construction costs. However, subsequent amendments to the Act will decrease the federal participation from 75 percent to 55 percent for any project receiving a construction grant after September 30, 1984 (as will be the case for Plymouth). Funded facilities usually consist of treatment facilities, interceptors, major pump stations and associated force mains, provided that they are part of a pollution abatement system need to correct existing wastewater-related problems. Federal grants of 85 percent (75 percent after September 30, 1984) may be obtained for "innovative and alternative (I/A) treatment technologies.

The State of Massachusetts currently provides 15 percent of the construction costs for eligible wastewater facilities. It is our understanding that the State is considering increasing its involvement to 35 percent for non-I/A construction to compensate for the upcoming reduction in Federal funding, but as of yet, no decision has been reached. Therefore, for the purpose of estimating financial requirements of the recommended plan, the State grant has been assumed to be 15 percent of the eligible cost.

Under Chapter 557 of the Acts of 1979 a separate State funding program was added to the Massachusetts Clean Water Act (Chapter 21 of MGL) to help municipalities defray the cost of wastewater collection system projects not likely to receive grant assistance under the EPA Construction Grants Program. This program awards grants up to a maximum grant amount of $2 million in the amount of 50 percent of eligible construction costs (up to a maxium of $2 million) for such projects on the basis of either economic benefit or pollution abatement.

Bonds and Betterments. The financing of construction of a wastewater collection and treatment system by real estate taxes alone unfairly spreads the burden of cost over an entire community, regardless of the amount of individual use or benefit received. Similarly, financing the construction of an entire system through betterment assessments levied against only those properties connected to the system could also be unfair since the cost of capacity provided for future users would be paid for entirely by the initial users.

Because of these considerations, it has been found practical and equitable to devise a combination of methods for financing such systems. The most desirable combination would equitably distribute the cost of the system among all property owners in proportion to both the amount of use and extent of benefit derived by each owner.

A typical program for financing the cost for expansion of a wastewater system is presented in Table 12-1.

TABLE 12-1. SOURCES OF REVENUE FOR FINANCING WASTEWATER SYSTEM EXPANSION

Description	Paid By
1. Treatment plant expansion and upgrading, intercepting sewers, pump stations, and force mains	General taxes and Federal and State grants
2. Collection sewers	Property owner and State grants (Chapter 557)
3. Building sewers (house connections)	Property owner
4. Maintenance and operation	Sewer service charge

Financing Major Facilities

Since the major components of the two proposed Phase I construction projects (i.e. the treatment and disposal facilities, the major interceptors and pump stations) will benefit the Town as a whole, it is equitable for the entire Town to share in these costs. Therefore, we recommend that the Town's share for these portions of the proposed North Plymouth and Manomet systems be financed through general obligation bond issues to be paid from revenues derived from real estate taxes.

As of July 1983, the Town of Plymouth owed $200,000 in bonds issued in 1968 to finance the existing wastewater treatment plant. These bonds were issued at a 4 percent interest rate and will be paid off in 1988.

Financing Collection Sewers

New collection sewers which abut properties and provide sewer service for these properties are generally financed by a betterment assessment levied against the property served. There are several methods that the Town can use in making assessments.

1. The Frontage Assessment. Under this assessment method, a town makes an assessment at a uniform rate per front foot or property served. The disadvantage commonly associated with this method is that the property owner having a greater frontage than neighboring properties would pay a higher assessment though his benefit might be no greater. Further assessment difficulties arise with odd-shaped lots and corner lots.

2. Local Unit Cost Assessment. Under another method, a town may divide the actual cost of the collection sewer among the actual number of properties served by the particular collection sewer. This method is not a popular or particularly fair method because the cost to an individual property owner varies widely with the depth of the sewer, the material in which the sewer is constructed, and the length of the frontages of the properties served. Thus, a property owner in a rocky area would pay more for a street sewer than would an owner in a sandy area even though the benefits to both owners were the same.

3. Average Cost Assessment. An assessment method which corrects some of the inequities of the first two is the average unit cost method. Under this method, each property owner would pay the average cost of the lateral sewers in a project, thus apportioning construction difficulties equally to all users. This method more fairly allocates cost to a user on the basis of the benefit received.

The actual determination of an appropriate assessment method and rate is beyond the scope of this report and will require further study by town officials. For planning purposes, it has been assumed that the local share of collection sewers in Manomet would be financed entirely by betterment assessments. In the event that betterments are used to finance only a portion of these costs, the tax rate would increase while the cost to each sewer user would decrease.

Financing Building Sewers

Building sewers (connections from the individual properties to the Town sewer) are generally paid for by the property owner. However, the construction of the connection from the collection sewer in the street to the property is typically included included in street sewer contract. The cost to the property owner for that portion of the connection would be in addition to the betterment assessment if the frontage assessment method is used.

Financing Maintenance and Operation

For projects which receive Federal and State funding under the EPA construction grants program, Federal regulations require that the cost for operation and maintenance of these facilities be allocated to each user or user class. Currently, Plymouth recovers a portion of these costs by user charges which are billed once each year in May. The charge is 70 cent per 100 cubic feet of water used during on the six-month period from November to May. Additional costs are recorded in the form of septage fees.

If the user charges were sufficient to offset the actual annual O&M cost of all wastewater facilities, they would meet the Federal requirements. However, the expenditure and revenue figures for fiscal year 1983, shown in Table 12-2, indicate that expenditures exceeded revenue from user charges amd septage fees, with remaining revenues made up from general taxation.

Implementation of a user charge system, while increasing the direct cost to the sewer user, reduces the tax rate for both sewer user and non-user.

TABLE 12-2. PRESENT IMPACT OF SEWER DIVISION
O&M COSTS ON GENERAL TAX RATE
(F.Y. 1983)

Total Sewer Division budget	$349,295
Less Revenues from user charges	175,599
Less revenues from septage fees	20,293
Total paid by general taxation	$153,412
Total assessed valuation (000)	$1,021,302
Portion of tax rate used to pay O&M costs $/$1,000	$0.15
Percentage of tax rate* used to pay O&M costs	0.69%

* Tax rate for fiscal year 1983 was $21.70 per $1,000 assessed valuation.

A user charge system should meet the following requirement, as stated in 40 CFR 35.935-13(b)(1), as published in the February 11, 1974 Federal Register: "The user charge system must result in the distribution of the cost of operation and maintenance of treatment works in the grantee's precise area to each user (or user class) in proportion to such user's contribution to the total wastewater loading of the treatment works. Factors such as strength, volume, and delivery flow rate characteristics shall be considered and included as the basis for the user's contribution to ensure a proportional distribution of operation and maintenance costs to each user (or user class)."

In Plymouth it is recommended that the cost of operation and maintenance of the facilities be apportioned to the users based on flow, as the present charges are, since essentially all sewer users would typically discharge wastewater of normal domestic strength. This method of apportionment would meet Federal regulations and provide the most equitable arrangement which the Town could implement. If a specific industry or other establishment discharges wastewater exhibiting unusual characteristics, an additional charge can be made based upon BOD5 and SS loadings in excess of those found in typical domestic strength wastewater. In this case, the user's share would be based on each users reported total flow and a surcharge for the excess BOD5 and SS loading. This information would be obtained from a yearly waste survey. The cost of the operation and maintenance of the wastewater treatment facility would be allocated among the wastewater parameters and allocated accordingly to each user.

A user charge rate would be based on the metered water consumption of all individual residences, businesses, commercial and institutional establishments and industries. Adjustments would be necessary in cases where significant quantities of water do not reach the sewer or where wastewaters are of above normal strength. In any event, all residences, buinesses, commercial and institutional establishments and industries would have to be metered including those with private water supplies such as wells.

Financing the Initial Program

The total capital costs of installation of the recommended plan is $27 million (see Table 11-1). Because the level of need varies in different locations, construction of the proposed wastewater facilities in North Plymouth and Manomet would each be completed in two phases. Those facilities included under Phase II construction are less urgently needed than those included in Phase I. The total capital cost costs of installing the Phase I programs for North Plymouth and Manomet are $17.5 million and $4.4 million respectively.

A summary of the Phase I costs indicating the expected shares to be paid by the Town and State and Federal grants are shown in Table 12-3. It has been assumed for the purposes of this table that all expected development in the Industrial Park Service Area will be documented by the Town in advance, as required to make the projected flows from this area eligibile for State and Federal grants (see letter of January 18, 1983 from DEQE to the Board of Selectmen in Appendix K). If all of the projected flow from this service area is not documented, the Town share would increase.

As discussed earlier, the financing of the Town share would be accomplished by a combination of betterments and general taxes. Table 12-4 summarizes the recommended Phase I funding sources. These cost distributions are preliminary and are intended only to indicate in an approximate manner the economic impact of the initial construction program. When cost apportionment methods are finalized and actual construction costs are known, a reevaluation of the cost distributions presented here should be made.

TABLE 12-3. EXPECTED PHASE I CONSTRUCTION GRANT SUMMARY
(all costs in thousands of dollars)

	Federal Share	State Share	Town Share	Total
North Plymouth				
Collection and Delivery System				
Cordage Interceptor	$1,461*	$398*	$1,015*	$2,874
Harbor Interceptor	387	106	211	704
I/I Reduction	257	70	140	467
Night Soil Disposal Facility	140	38	77	255
Treatment and Disposal				
WWTP and Sludge Disposal	4,578	1,261	2,565	8,404
Effluent Holding Tank	908	248	494	1,650
Effluent Pump Station	215	59	116	390
Outfall	1,540	420	840	2,800
Total North Plymouth	$9,486	$2,600	$5,458	$17,544
Manomet				
Collection System	$0	$1,030	$1,030	$2,060
Delivery System	404	110	220	734
Treatment and Disposal	1,088**	217**	145**	1,450
Land Acquisition	85	17	11	113
Total for Manomet	$1,577	$1,374	$1,406	$4,357
Total Phase I	$11,063	$3,974	$6,864	$21,901

* Based on the assumption that the Town can document all projected industrial park flow.
** Assuming I/A funding is obtained.
Note: All costs are based on ENR = 4000.

TABLE 12-4. SUMMARY OF RECOMMENDED PHASE I FUNDING SOURCES
(all costs in thousands of dollars)

Source	North Plymouth	Manomet	Total
Federal and State Grants	$12,086	$2,951	$15,037
Betterment Assessments	0	1,030	1,030
General Taxes	5,458	376	5,834
Total	$17,544	$4,357	$21,901

In developing the cash flows and tax impacts of the Phase I construction programs, a number of assumptions were made, consistent with the applicable General Laws of the Commonwealth of Massachusetts. A summary of these assumptions is shown in Table 12-5. In projecting assessment revenues, it has been assumed that all Manomet residents whose property values are improved by the installation of sewers will begin paying assessments the first year after the sewers are constructed. Remaining costs not covered by betterment assessment or by Federal and State grants would be paid from general taxes.

TABLE 12-5. BASIS FOR FINANCIAL ANALYSIS OF TOWN'S SHARE OF PHASE I PROGRAM

	North Plymouth Construction	Manomet Construction
Financing Costs		
Total Estimated Town Share	$5,458,000	$1,406,000
Total Interest Charges 7.0% for 20 years Serial Bonds	4,011,628	1,033,410
Total Cost	$9,469,628	$2,439,410
Annual Principal	$272,900	$70,300
Revenues		
Average Assessment (estimated)	-	$1,750 per lot x 600 lots
Terms of Assessment Payment	-	5% for 20 years

The cash flow projections, together with yearly incremental impact on the general tax rate for the two Phase I construction programs, are as shown in Tables 12-6 and 12-7. The total property valuation projections were projected by the Executive Secretary through the year 1990 (year 3) and have been assumed to continue growing at 8.5 percent per year thereafter.

A summary of estimated costs to property owners is presented in Table 12-8.

TABLE 12-6. PROJECTED CASH FLOWS AND TAX ADJUSTMENTS PHASE I PROGRAM FOR NORTH PLYMOUTH

Year	Debt Retire-ment	Assess-ment Revenue	To Be Raised by Taxation	Total Town Property Valuation (000)	Projected Tax Increment /$1,000
1	$654,960	0	$654,960	$1,722,067	$0.38
2	635,857	0	635,857	1,870,633	0.34
3	616,754	0	616,754	2,031,083	0.30
4	597,651	0	597,651	2,203,725	0.27
5	578,548	0	578,548	2,391,042	0.24
6	559,445	0	559,445	2,594,280	0.22
7	540,342	0	540,342	2,814,794	0.19
8	521,239	0	521,239	3,054,051	0.17
9	502,136	0	502,136	3,313,646	0.15
10	483,033	0	483,033	3,595,306	0.13
11	463,930	0	463,930	3,900,907	0.12
12	444,827	0	444,827	4,232,484	0.11
13	425,724	0	425,724	4,592,245	0.09
14	406,621	0	406,621	4,982,586	0.08
15	387,519	0	387,519	5,406,106	0.07
16	368,412	0	368,412	5,865,625	0.06
17	349,312	0	349,312	6,364,203	0.05
18	330,209	0	330,209	6,905,160	0.05
19	311,106	0	311,106	7,492,099	0.04
20	292,003	0	292,003	8,128,927	0.04

TABLE 12-7. PROJECTED CASH FLOWS AND TAX ADJUSTMENTS
PHASE I PROGRAM FOR MANOMET

Year	Debt Retire-ment	Assess-ment Revenue	To Be Raised by Taxation	Total Town Property Valuation (000)	Projected Tax Increment $/1,000
1	$168,720	$84,252	$84,468	$1,722,067	$0.05
2	163,799	84,252	79,547	1,870,633	0.04
3	158,878	84,252	74,626	2,031,083	0.04
4	153,957	84,252	69,705	2,203,725	0.03
5	149,036	84,252	64,784	2,391,042	0.03
6	144,115	84,252	59,863	2,594,280	0.02
7	139,194	84,252	54,942	2,814,794	0.02
8	134,273	84,252	50,021	3,054,051	0.02
9	129,352	84,252	45,100	3,313,646	0.01
10	124,431	84,252	40,179	3,595,306	0.01
11	119,510	84,252	35,258	3,900,907	0.01
12	114,589	84,252	30,337	4,232,484	0.01
13	109,668	82,252	25,416	4,592,245	0.01
14	104,747	84,252	20,495	4,982,586	0
15	99,826	84,252	15,574	5,406,106	0
16	94,905	84,252	10,653	5,865,625	0
17	89,984	84,252	5,732	6,364,203	0
18	85,063	84,252	811	6,905,160	0
19	80,142	84,252	0	7,492,099	0
20	75,221	84,252	0	8,128,927	0

TABLE 12-8. ESTIMATED COST PER HOUSEHOLD WITH PHASE I CONSTRUCTION

	New Sewer User-Manomet	New Sewer User-North Plymouth	Existing Sewer User	On-Lot System User
Estimated Annual Cost With No Sewers in Manomet				
User charge(1)	-	$ 78.00	$ 78.00	$ 0
Taxes (diminishes over 20 years to $3.20)(2)	-	30.40	30.40	30.40
Total Annual Cost per household	-	$108.40	$108.40	$30.40
Estimated Annual Cost Including Phase I Sewers In Manomet				
Betterment assessment cost(3)	$140.42	$ 0	$ 0	$ 0
User charge(4)	79.00	79.00	79.00	0
Taxes (diminishes over 20 years to $3.20)(5)	34.40	34.40	34.40	34.40
Total annual cost per household	$253.82	$113.40	$113.40	$34.40
Estimated One-Time Cost				
Building sewer (private residence)	$1,000	$1,000	0	0

1. Average user charge townwide based on estimated 4,341 households served in 1987.
2. Based on initial tax increase of $0.38 per $1,000 (Table 12-6) and an average property assessed at $80,000
3. Based on a betterment of $1,750 over a 20-year period at 5 percent interest.
4. Average sewer user charge townwide based on estimated 4,881 households served in 1987.
5. Based on initial tax increase of $0.43 per $1,000 (Tables 12-6 and 12-7) and an average property assessed at $80,000.

Implementation

Responsibilities of the Town. The Town currently provides wastewater collection and treatment for a portion of its residents. Policy and financial decisions are made by the Department of Public Works, and the operation and maintenance of the system is the responsibility of the Director.

Construction and operation of the new facilities is proposed to be undertaken by the Town with financial assistance for construction provided by the Federal and State governments through EPA and State construction grants programs. The project will be administered through the Department of Public Works. Recommendations for implementation of the project and meeting the financial requirements are presented hereinafter.

Implementation Steps. As a first step, the Town should review and approve the Facilities Plan and submit this document to the EPA and DWPC for approval. Concurrently, the Town should prepare and submit to the EPA and DWPC an application for a State planning advance to cover 90 percent of the eligible design (Step 2) costs of the selected Phase I project(s). Town Meeting action is required to appropriate the necessary funds for the Step 2 project costs before the Town can receive the Step 2 planning advance. Upon receipt of the planning advance for the Step 2 work, the Town would proceed with the design of the selected facilities.

During the Step 2 phase, Town Meeting approval would be required to appropriate the necessary funds for the construction

(Step 3) of the selected project(s). The Town must appropriate its share of the Federal and State funded work plus the amount needed to construct the ineligible portions of the project(s). The Town is required to complete this action prior to receiving a grant for the construction of the Federally-funded project.

Plans and specifications for the project(s) would be developed and submitted to the State and EPA for approval, along with a request for State and Federal funding of Step 3 of the project(s). Upon receiving a Step 3 grant, advertisement for bids for construction of the project(s) will be made. Bids will be received and reviewed by the Town, its engineering consultant, the State and EPA, with award to the lowest qualified bidder. Construction of the modifications to the existing wastewater treatment plant and other Phase I construction would take approximately two years.

Implementation Schedule. According to an Administrative Order issued to the Town of Plymouth by the DEQE in January 1984, the Town must comply with the following schedule for planning, design and construction of the wastewater treatment facilities for North Plymouth:

Action	Date
1. Submit a complete Step 1 application for design of the recommended facilities.	June 1, 1984

2.	Complete the design of an expanded wastewater treatment plant in accordance with the approved facilities plan and submit a complete Step 3 application for construction of the recommended facilities.	July 1, 1985
3.	Begin construction of the wastewater treatment plant.	February 1, 1986
4.	Complete construction of the wastewater treatment plant.	February 1, 1988
5.	Obtain operation limits for new treatment plant (secondary limits).	June 1, 1988
6.	Take all steps necessary to insure that the discharge and the expanded facility comply with the Ocean Sanctuaries Act.	-