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TECHNOLOGY AND POLICY ISSUES INVOLVED IN THE BOSTON HARBOR CLEANUP

by
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RALPH M. PARSONS LABORATORY
HYDROLOGY AND WATER RESOURCE SYSTEMS

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Introduction

Introduction

Throughout history, disposing of human wastes has been a problem. In the late 1800's, when communities began to flourish, it was generally believed that human waste was the source of many diseases. The principal method of disposal at this time was to discharge the waste into local waters – rivers, lakes, or the ocean. In the Boston area, the receiving water was the Boston Harbor.

In the late-1800's, Boston's waste was combined with water, transported to Deer Island and Moon Island, chlorinated, and discharged into the harbor. In the early-1900's, Boston's disposal method was considered innovative because wastewater from Moon Island was discharged only on the outgoing tide. By the mid-1900's, Boston realized that its method of wastewater disposal was beginning to adversely effect the water quality and aesthetics of the harbor.

Primary treatment of the wastewater did not begin until the late-1960's when the Deer Island and Nut Island facilities were constructed. Due to Boston's rapid population growth, these primary treatment plants were obsolete the moment they began operation. Both plants discharge chlorinated effluent and chlorinated digested sludge into the President Roads area of the Boston Harbor. The sludge is still discharged only on the outgoing tide.

Boston and some of its neighboring communities have a combined sewer system. This system transports both wastewater and stormwater to the treatment facilities. During periods of excessive flows, the combined sewers are designed to overflow into nearby waters – such as Boston Harbor and the freshwater rivers that feed the harbor. These discharges occur frequently (more than one hundred times per year) in the Boston area which results in an estimated overflow of 40 billion liters per year of wastewater and stormwater into the harbor. These overflows cause beach and shellfish bed closures throughout the area.

The Clean Water Act (CWA), created in 1972 and amended in 1977, required all municipalities to provide uniform (secondary) treatment to the wastewater before releasing it into the receiving water. At this time, the federal government was providing funding for 85% of the construction costs for the secondary treatment facilities. In the amended act, a

provision, under section 301(h), allowed municipalities the right to waive the secondary treatment requirements if they were discharging into marine waters through a long outfall. The Metropolitan District Commission (MDC), who at that time was responsible for providing treatment to the greater Boston area wastewater, believed that the benefits of secondary treatment were not justified. This was based on a number of studies that determined the priority of 52 projects related to the Boston Harbor cleanup. Secondary treatment ranked 43rd. Based on these findings, MDC applied for the secondary treatment waiver. It was not until nearly eight years after the enactment of the amended CWA that MDC was tentatively denied its waiver. During this time, nothing was done to improve the wastewater treatment facilities because no funding was made available during the waiver application process. Political tensions surrounding the Boston Harbor cleanup issues began to emerge and a new agency, Massachusetts Water Resources Authority (MWRA), was created. MWRA is responsible for providing treatment to the wastewater.

When MWRA was established, there was strong political and legal pressure to cleanup the Boston Harbor. Due to this pressure, MWRA gave up its right to appeal the tentative denial of the secondary treatment waiver and began plans for the construction of a secondary treatment facility. The construction schedule consisted of consolidating the treatment process at a single location on Deer Island. The treatment process would provide primary treatment by 1995 and secondary treatment by 1999 of all MWRA wastewater. This schedule also included the construction of an outfall to transport the effluent 14 km off the shore of Deer Island, and to stop ocean disposal of the sludge. The estimated cost of this project is $6.1 billion. At this time, the funding by the federal government for the construction of secondary treatment facilities is no longer available; therefore, the burden of paying for these facilities is on the taxpayers.

Ever since the enactment of the CWA, scientists have been questioning the need for disposing secondary treated effluent into deep coastal waters. This question has been the focal point of many debates on Boston Harbor. Although, an even more important question for the taxpayers and MWRA is – whether the $6.1 billion project will cleanup the harbor?

This document summarizes the enormous amount of literature available on the technical and political issues related to the Boston Harbor cleanup. Chapter 1 and 2 discuss the physical aspects of the Boston Harbor and the existing treatment facilities. Chapter 3 addresses the technical and political issues that led to the decision to construct secondary

treatment facilities. The components of the proposed secondary treatment facilities are discussed in detail in Chapter 4. The management of sludge, which is the residual byproduct of secondary treatment, is discussed in Chapter 5. Controlling the combined sewer overflows – a major source of pollution to the Boston Harbor – are discussed in Chapter 6. Chapter 7 discusses the modeling and monitoring efforts done to date and the efforts recommended for the future. Finally, Chapter 8 discusses alternate ways of approaching the cleanup.

Chapter 1

Physical Aspects of Boston Harbor

Physical Aspects of Boston Harbor

Introduction

The physical setting, water characteristics, and meteorological characteristics of Boston Harbor will be discussed. These parameters are important in modeling the effects of effluent discharges on the ocean environment.

Physical Setting

Boston Harbor is on the western edge of Massachusetts Bay. The bay is nearly longitudinal and can be found in the vicinity of 42°N, 70°W. The bay is approximately 100 km long and 40 km wide and is located in the Gulf of Maine. Boundaries of the bay extend north to Cape Ann, south to Cape Cod, and east to the eastern coast of Massachusetts (Fig. 1-1).

The average depth of the bay is 35 m. The chief topographic feature is a submarine ridge rising within 20 m of the sea surface along the open ocean boundary. This ridge is named Stellwagen Bank (Fig. 1-1). The Stellwagen Bank increases tidal velocities as the waves cross this ridge and head towards shore.

The harbor (Fig. 1-2) topography is characterized by two shipping channels (President Roads and Nantasket Roads) and several small islands scattered throughout the harbor. The harbor is relatively shallow (1-10 m) with depths reaching 20 m in the channels. Flow enters and leaves the harbor primarily through the two channels (Fig. 1-3 - 1-6), which results in a complex circulation pattern.

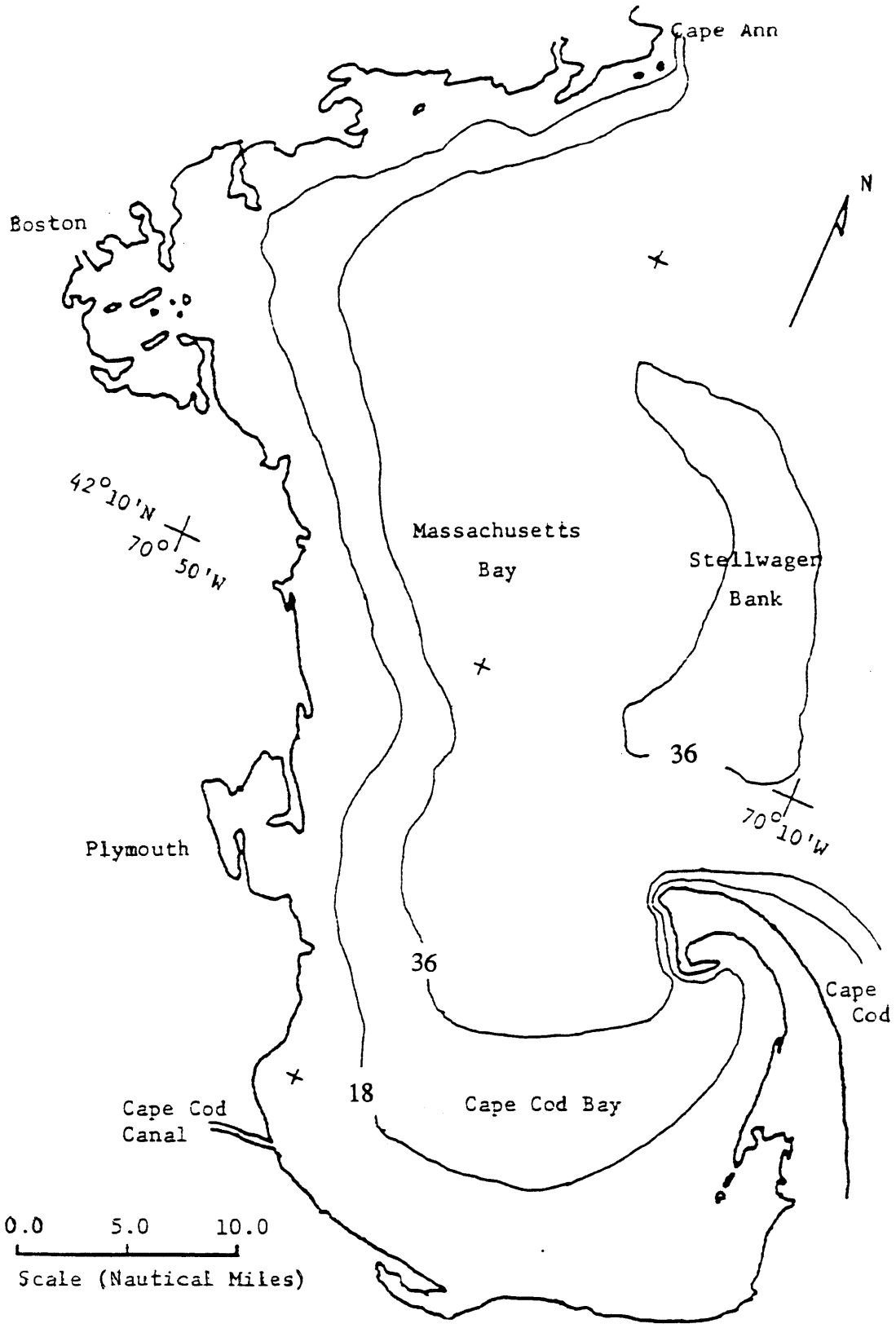

Figure 1-1: Massachusetts Bay, Depth Contours in Meters

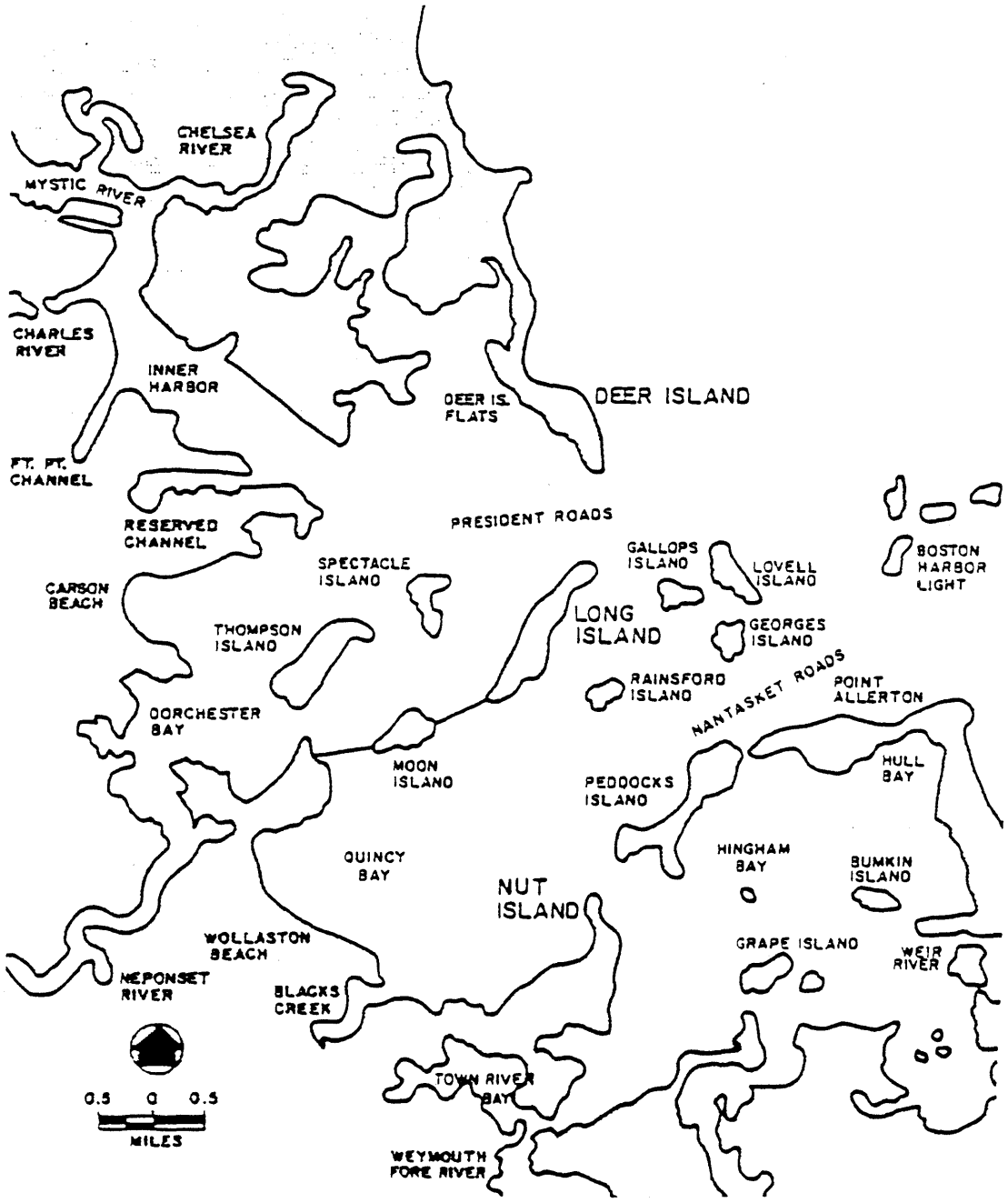

Figure 1-2: Map of Boston Harbor

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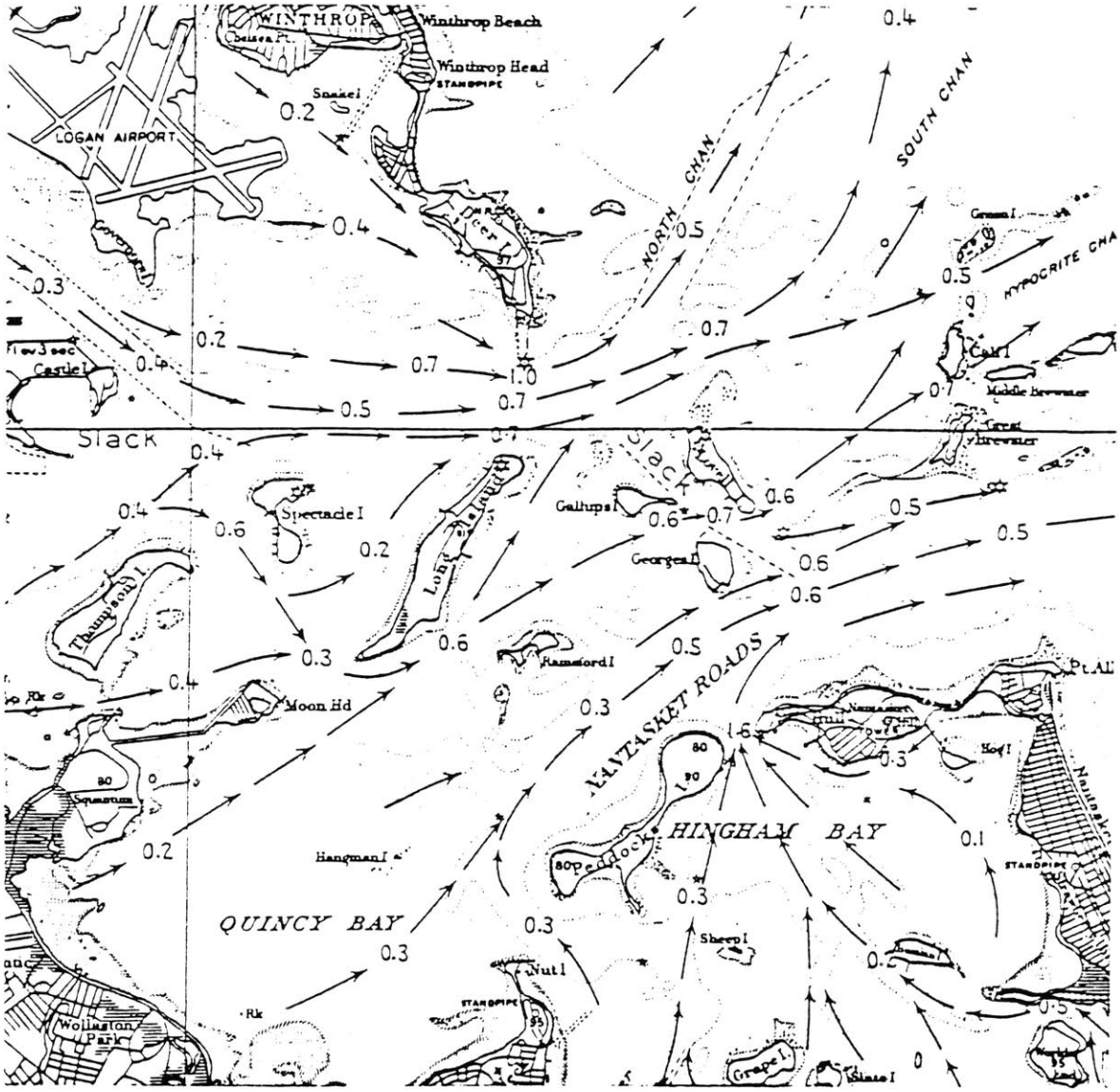

Figure 1-3: Current Velocities in Boston Harbor 1 Hour After High Tide (Kossik et al., 1986)

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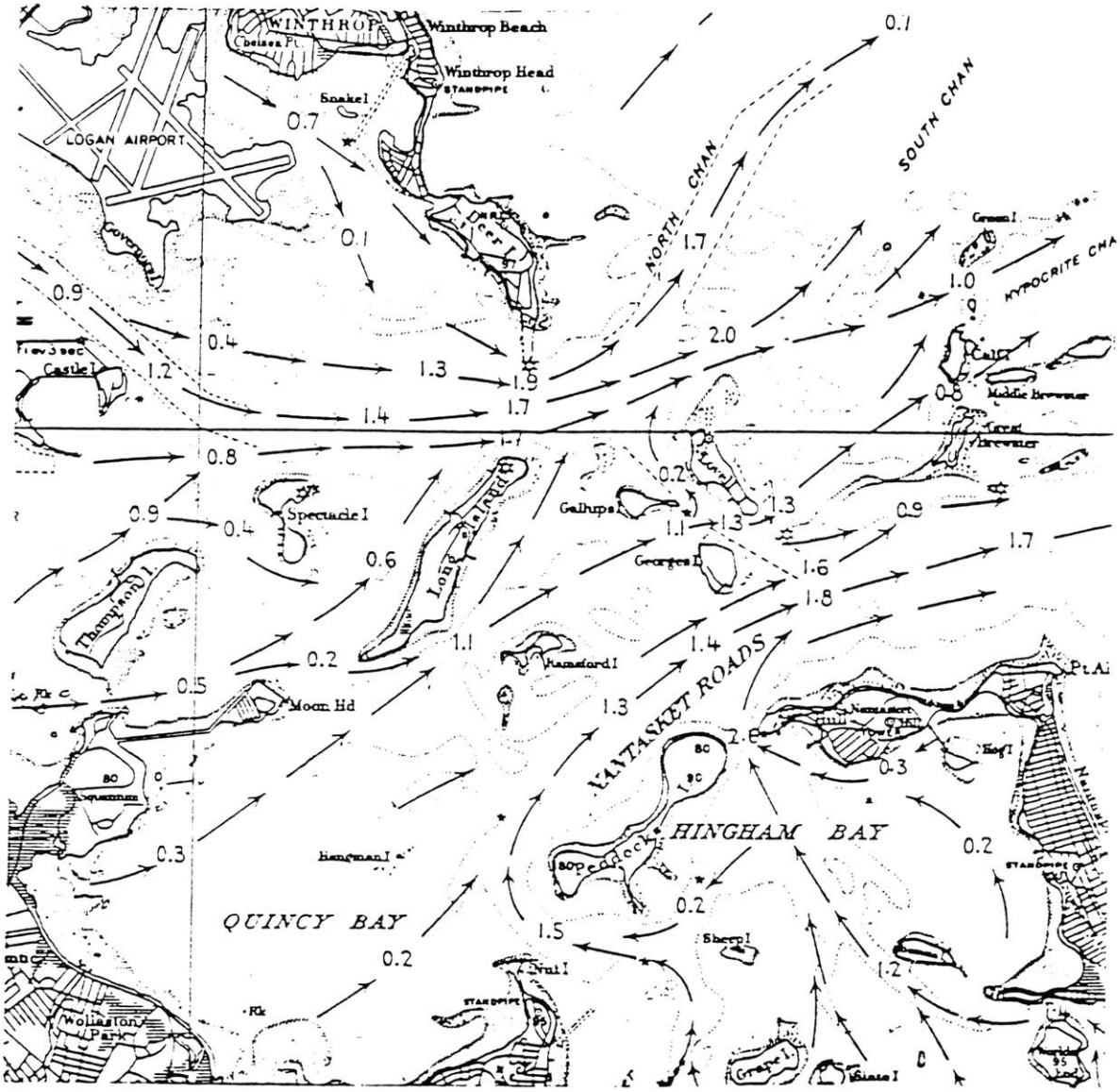

Figure 1-4: Current Velocities in Boston Harbor 4 Hour After High Tide (Kossik et al., 1986)

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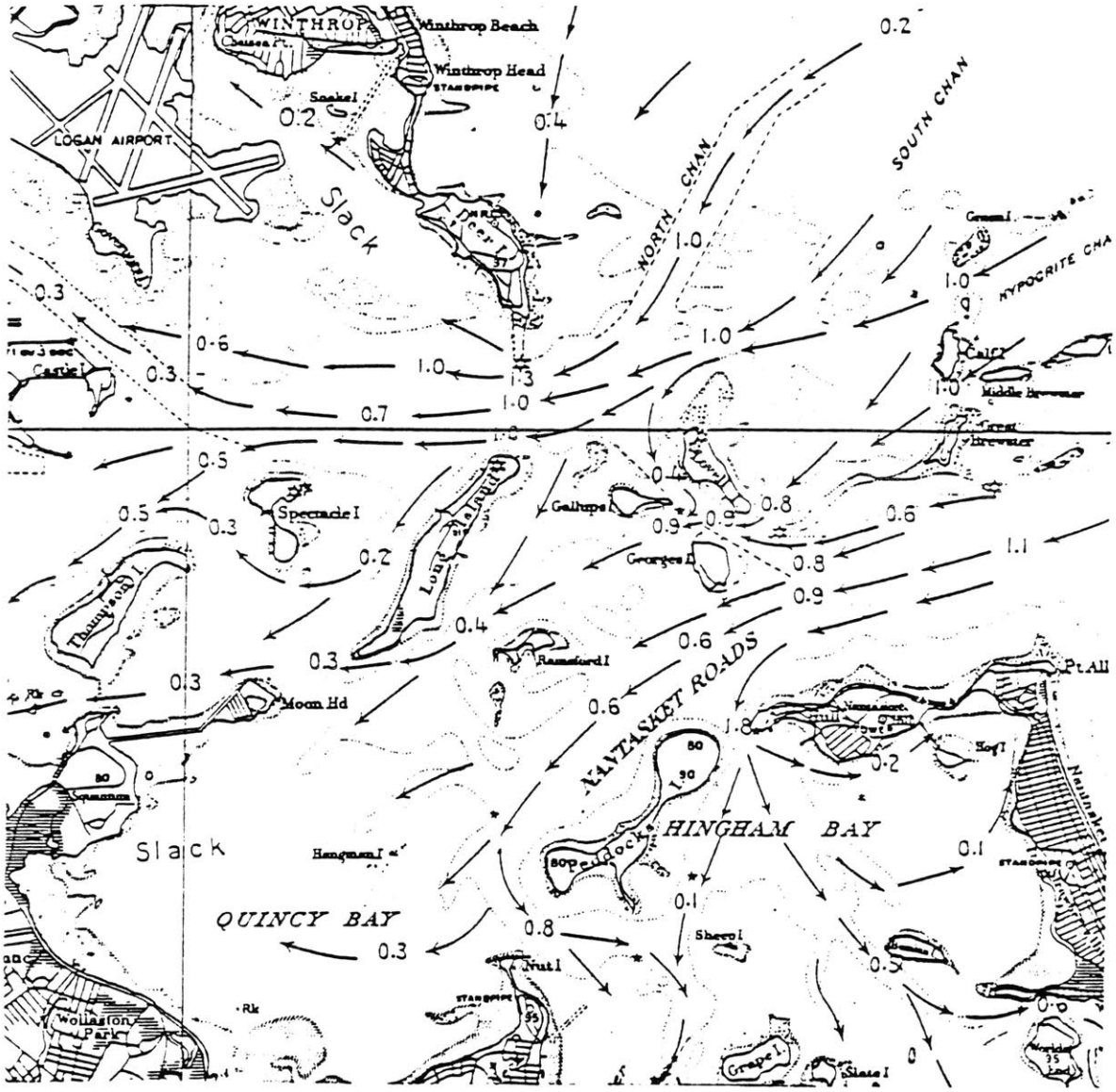

Figure 1-5: Current Velocities in Boston Harbor 1.5 Hour After Low Tide (Kossik et al., 1986)

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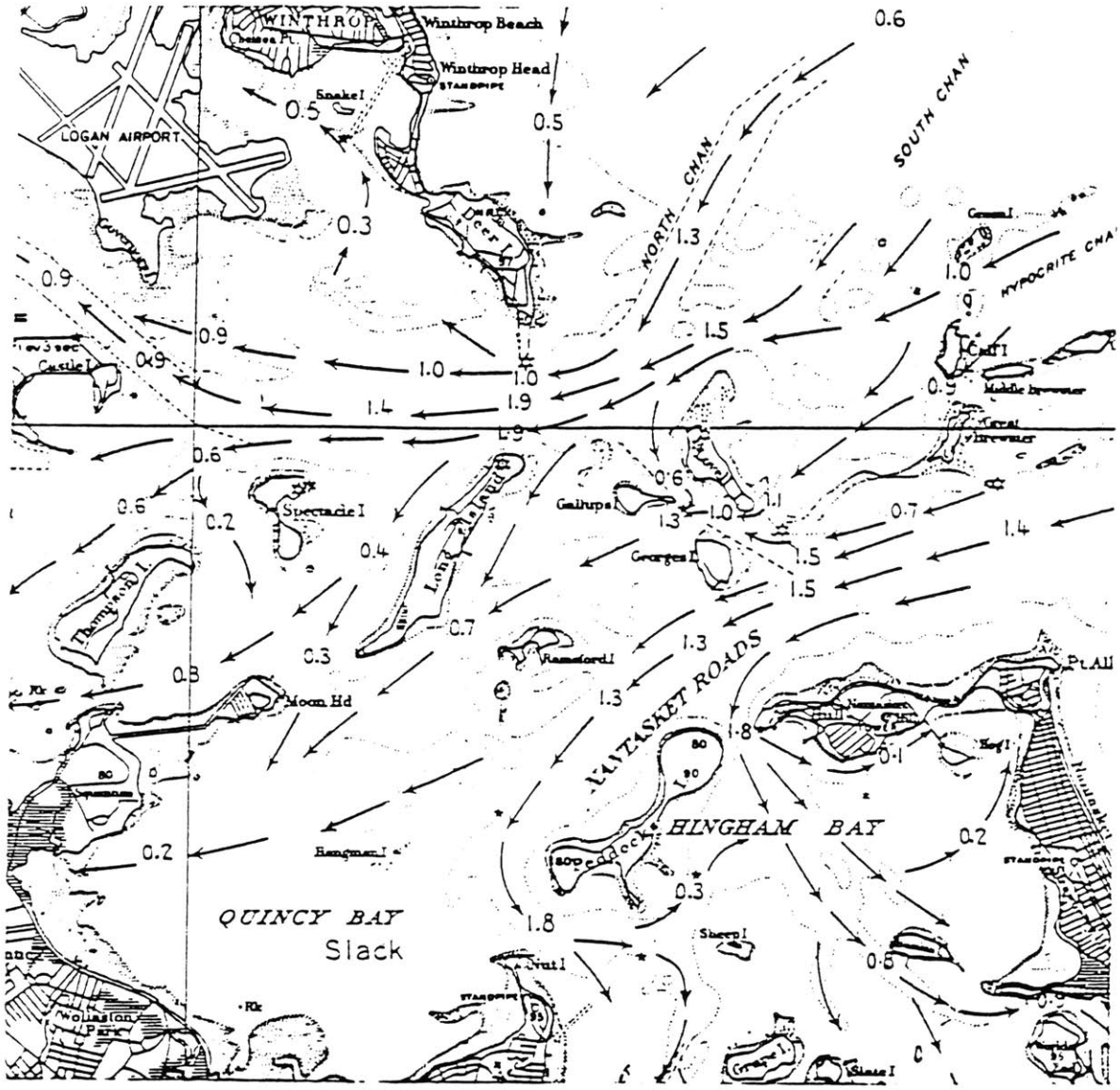

Figure 1-6: Current Velocities in Boston Harbor 4.5 Hour After Low Tide (Kossik et al., 1986)

Water Characteristics

Light, temperature, salinity and dissolved oxygen are the primary variables that influence biological and chemical interactions in sea water. Light is needed for photosynthesis. The depth to which light can pass through a column of water is often determined by how turbid the water is. Since Boston Harbor is generally quite shallow, wind causes scouring and resuspension of the sediment which increases turbidity and recycles material into the water column (Resource Analysis, Inc., 1976).

Temperature affects dissolved oxygen concentration, density, and algal growth. Temperature differences cause density differences within the water column, which causes the Boston Harbor to stratify during the summer months. This stratification is characterized by a warmer, more or less homogeneous, circulating, and turbulent surface layer. This upper surface layer is produced by wind action and absorption of solar radiation. This layer lays on top of a colder, relatively undisturbed region, in which the temperature decreases slightly with depth. These two layers are separated by a thermoclinal region in which the temperature decreases rapidly with depth. The temperature gradient of the thermocline inhibits the exchange of heat, nutrients, and oxygen between the upper and lower regions of the water column.

The harbor begins to stratify in May and is stably stratified by late July. In September, cooling begins and the thermocline deepens until November when the fall overturn begins. In December, the overturn continues and the entire water column is isothermal. The water column remains isothermal, with the coldest temperature occurring in February, until April when spring warming begins. A profile of the average annual temperature in Massachusetts Bay is shown in Figure 1-7.

Greatest variations in salinity occur near the surface and shoreline (where there is freshwater inflow). Salinity affects density, dissolved oxygen, and the types of aquatic life present in the sea water. In Boston Harbor, salinity varies in both the vertical and horizontal directions. The surface isohalines of Bumpus (1974) generally trend north and south (Fig. 1-8) during any month, with salinity increasing offshore. Maximum salinities occur in March, minimum salinities occur in May.

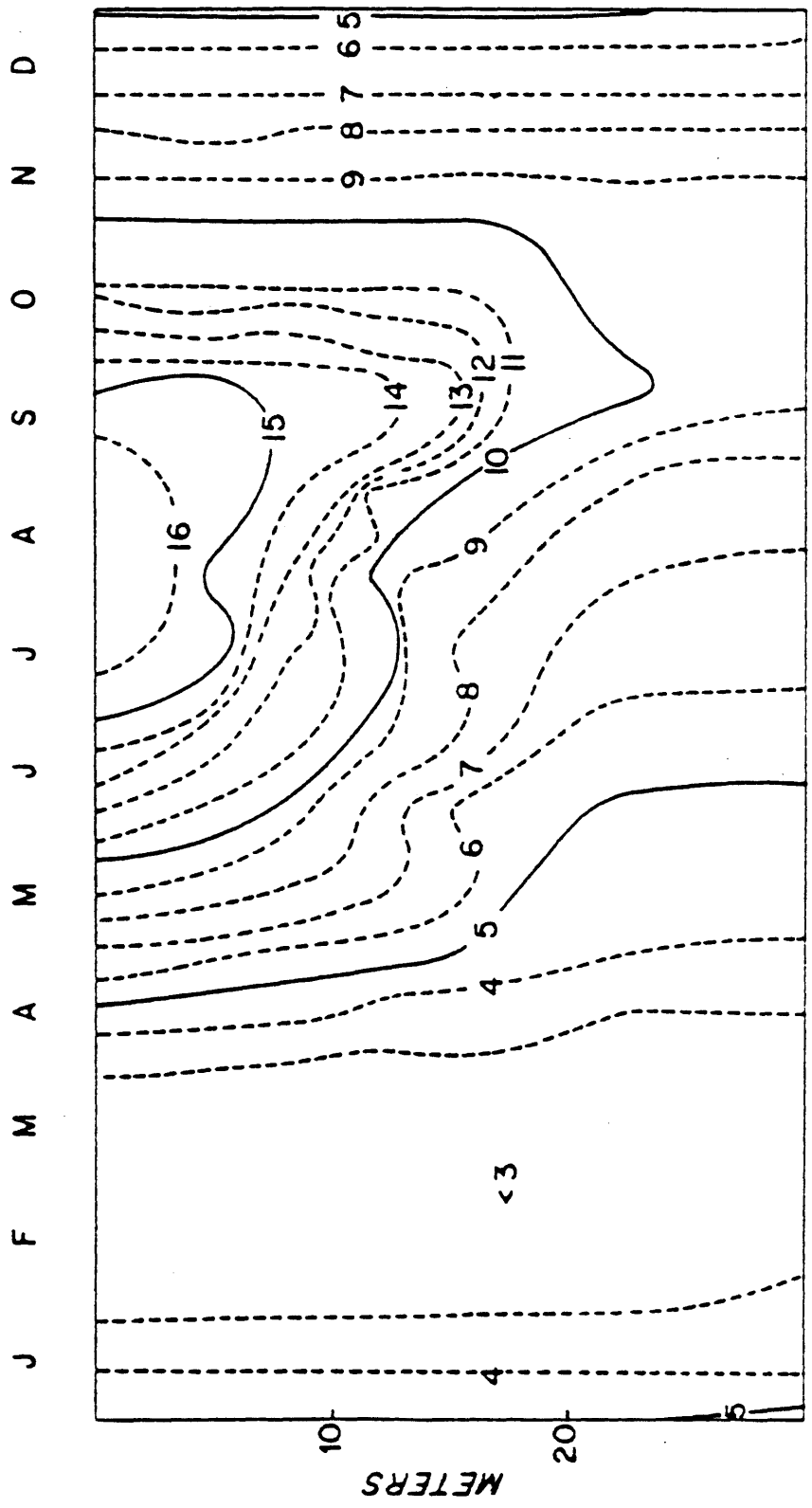

Figure 1-7: Profile of Mean Annual Temperature Cycle (°C) at Boston Lightlight, 1956-1970 (Bumpus, 1974)

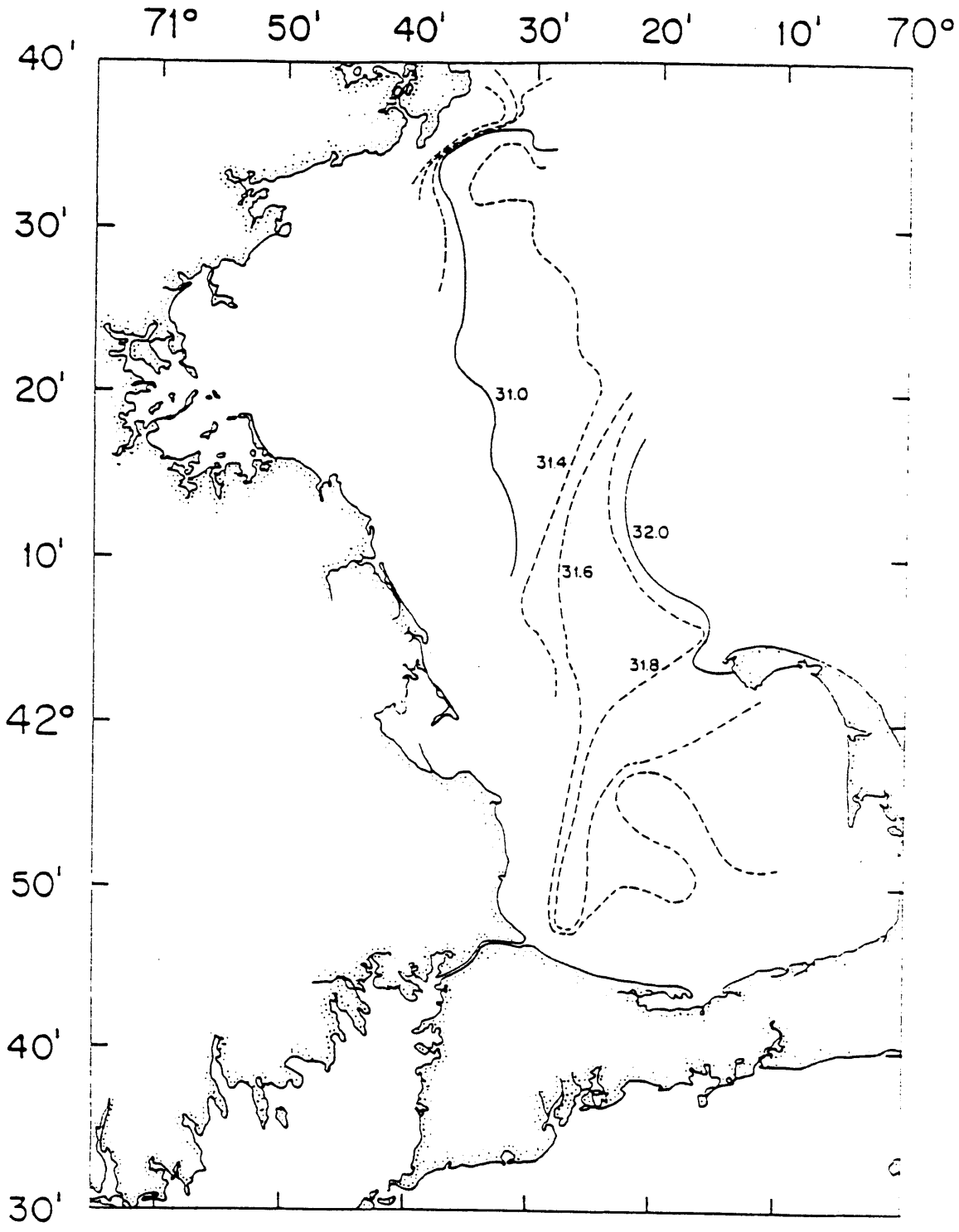

Figure 1-8: Surface Salinity (ppt) in Boston Harbor (Bumpus, 1974)

Freshwater flow into a harbor is the means by which salinity is lowered and pollutants are introduced into the oceanic environment. In Boston Harbor, the fresh water flow is delivered by the Merrimac River (above Cape Ann), the Charles, Mystic, and Neponset Rivers, outfalls at Deer and Nut Islands, and combined sewer overflows (CSO). The average input of all rivers into the harbor is approximately 1.7×10^6 m^3/day. The average flow from the outfalls and CSOs is approximately 1.5×10^3 m^3/day.

The harbor experiences semi-diurnal tidal oscillations with a mean tidal range of 3 m. Tidal flow dominates the water exchange, with nearly half of the volume of the harbor leaving on the outgoing tide. The volume exchange during each tidal cycle is approximately 1.1×10^8 m^3/day (Kossik et al., 1986). The volume of freshwater flow is small in comparison to tidal action and does not have a significant effect on flushing in the harbor except perhaps during spring runoff periods (Resource Analysis, Inc., 1976). Kossik et al., (1986) showed that the flushing time in the harbor is one-to-two weeks.

Meteorological Characteristics

The Atlantic Ocean has a significant role in moderating the effects of the winter cold and summer heat. The wind is predominantly westerly with an average wind speed of 20 km/hr, although, it is highly variable (Fig 1-9). Hurricanes and coastal storms bring significant amounts of rain and snow.

Wind-driven currents and tidal activity play a significant role in the mixing and transportation of nutrients and toxins within the harbor. Boston Harbor has a complex system of currents dominated by a southward flow across the mouth of the bay due to the Gulf of Maine. The flood tides are westerly and the edd tides are easterly (Fig. 1-10).

Boston Harbor is fairly windy and Parker and Pearce (1975) showed that wind-driven waves can be as large as one meter. Net drift is southerly, although drift tends to be seasonally variable and unpredictable in Boston Harbor and Massachusetts Bay (Kossik et al., 1986). Briggs and Madson (1974) used a drogue experiment to show that during periods of pronounced stratification, the upper stratified layer tended to move opposite to the direction predicted during ebb tide.

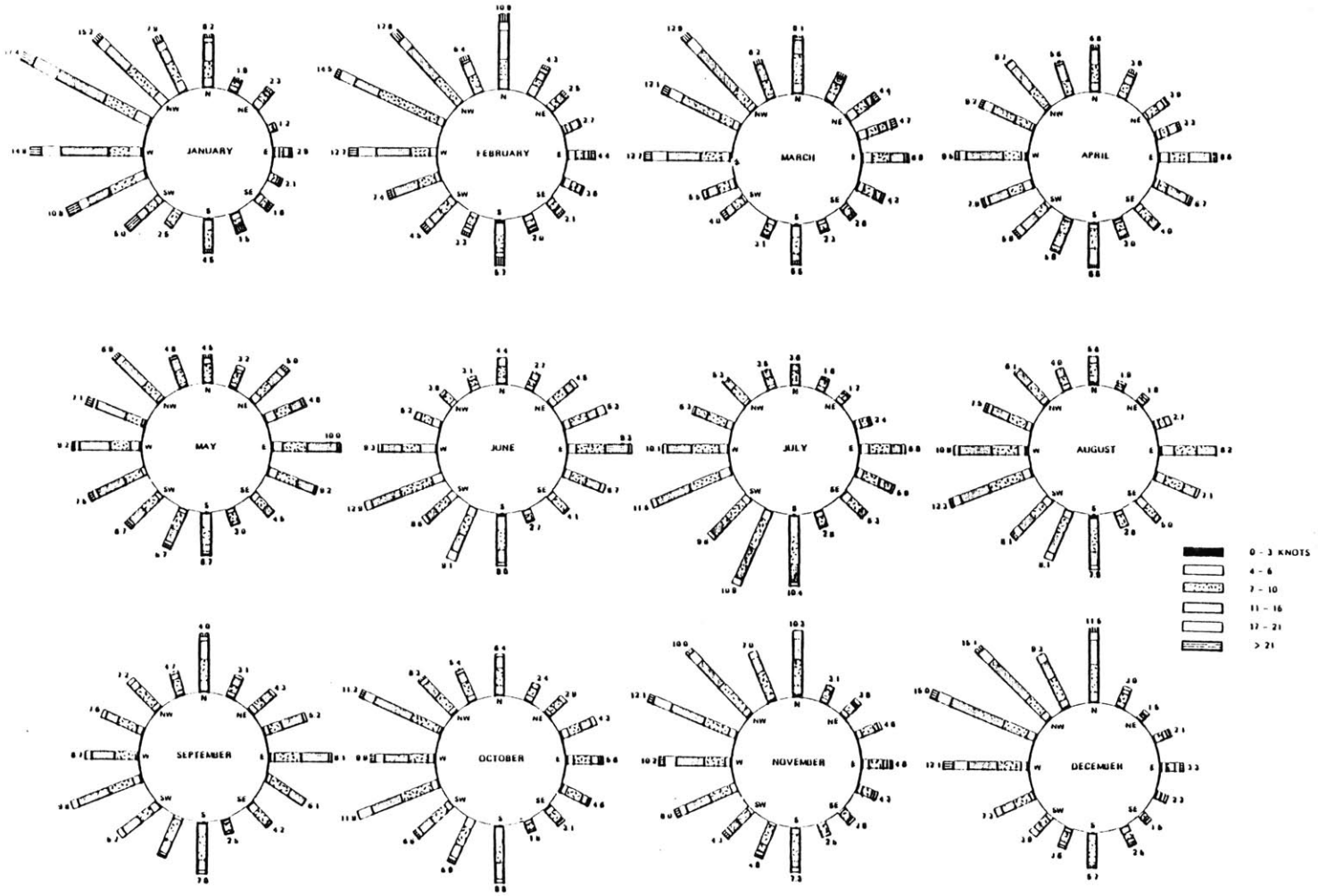

Figure 1-9: Wind Roses from Historical Data (Kossik et al., 1986)

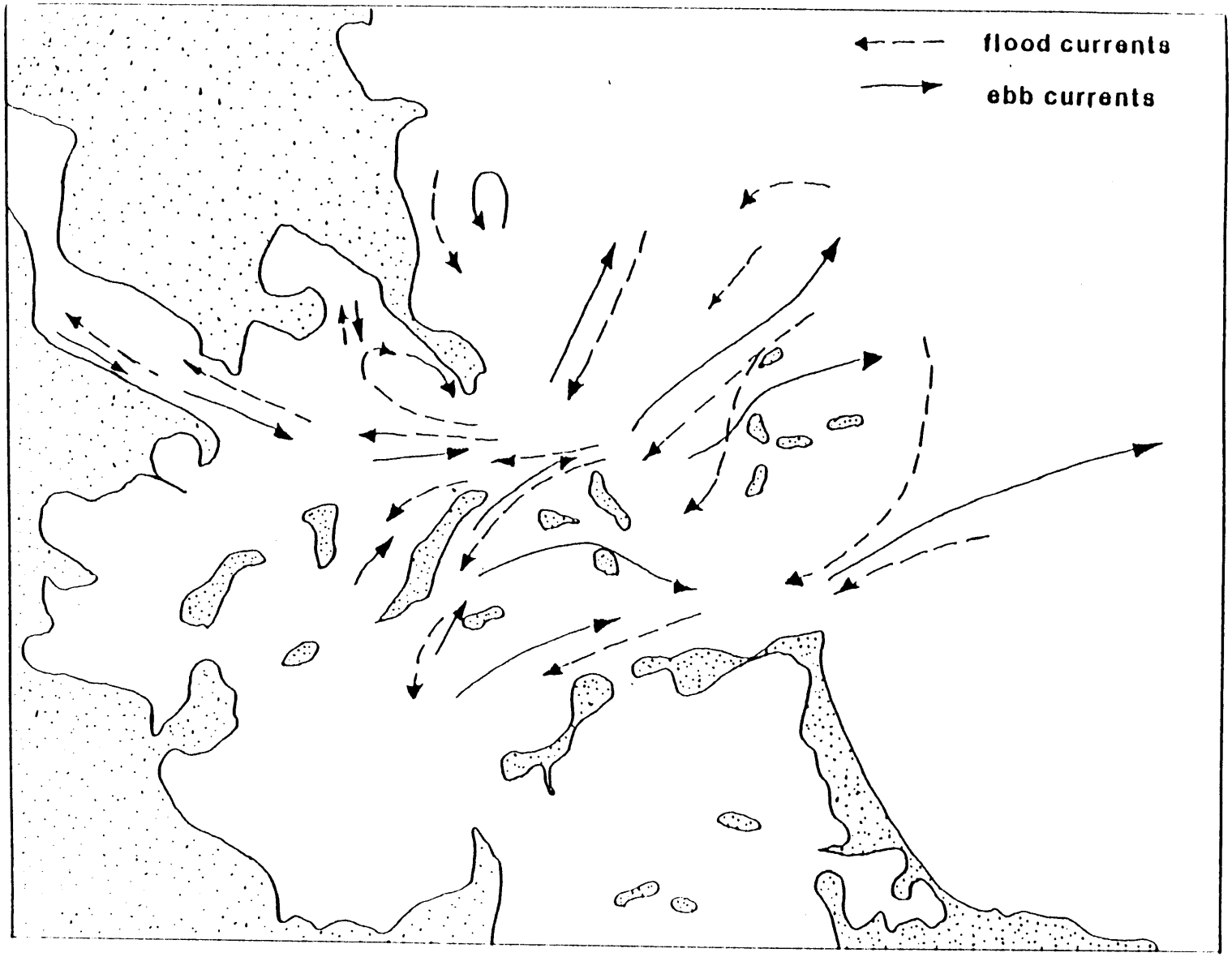

Figure 1-10: Generalized Circulation Patterns in Boston Harbor (Kossik et al., 1986)

On the average, precipitation occurs one out of every three days throughout the year. The average yearly precipitation is approximately 110 cm with the first measurable snowfall occurring in November and the last in March (MWRA, 1988).

Temperature in Boston ranges from over 32°C in summer to below -18°C in the winter. The mean annual temperature is 10°C. The coldest months are usually in January and February with the warmest months in July and August. The average relative humidity is 67% (MWRA, 1988).

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Chapter 2

Boston Harbor Pollution History

Boston Harbor Pollution History

Introduction

Boston Harbor has been the site of waste disposal since before the Revolutionary War. The disposal of domestic, commercial, and industrial waste dates back to the year 1847 when the City of Boston took possession of Deer Island for "sanitary purposes" (MWRA, Exec. Summary,1988).

By 1900, raw wastewater was being discharged at two locations in the harbor: Deer and Moon Island. At Deer Island, a total discharge of 190 million liters per day (mld) was released during all tidal periods. At Moon Island, 380 mld of raw wastewater was stored and discharged on the outgoing tide (Commonwealth of Massachusetts, 1900).

In 1939, a study was done which recommended that primary treatment plants be constructed on Deer, Moon, and Nut Islands (Commonwealth of Massachusetts, 1983). Today, only two plants have been built, one on Nut Island and the other on Deer Island. The first treatment plant was completed in 1952 on Nut Island. It was designed for an average flow of 420 mld and a peak flow of 1140 mld. The second primary treatment plant was completed in 1968 on Deer Island. It was designed for an average flow of 1300 mld and a peak flow of 3210 mld.

Currently, the MWRA's wastewater is split into two service areas, the North and South system. The South Systems wastewater is treated at the Nut Island Primary Treatment Plant and the North System's wastewater is treated at the Deer Island Primary Treatment Plant.

The existing sewerage system (Fig. 2-1) consists of approximately 362 km of separate and combined interceptor and trunk sewers servicing nearly 8000 km of local sewers. A total of 43 communities, consisting of approximately 1.9 million people, produce an average of 1720 mld of domestic, industrial, and commercial waste (MWRA, V-III, 1988)

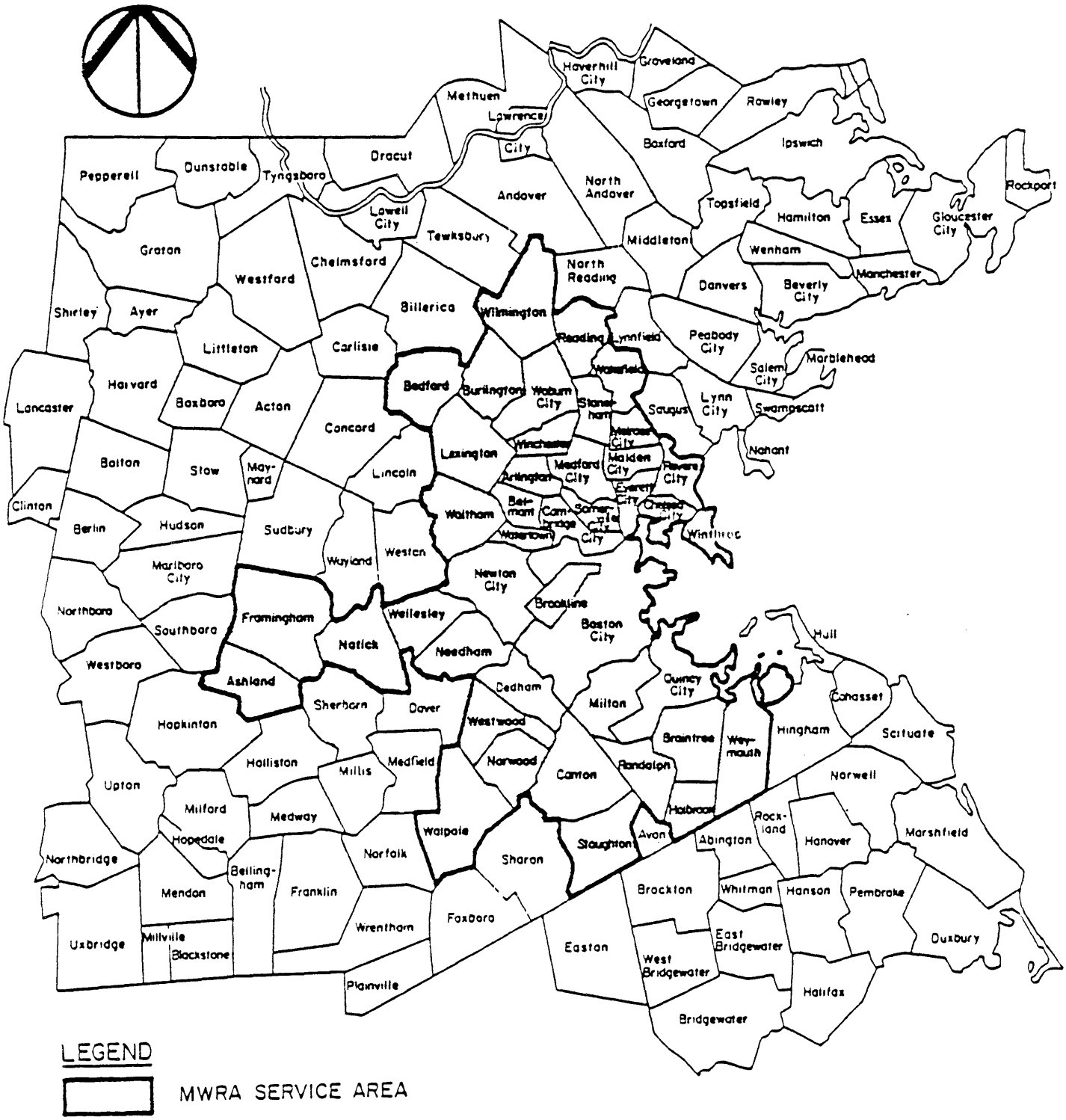

Figure 2-1: MWRA Service Area (MWRA, V-III, 1988)

Of the 43 communities that are serviced by the Nut Island and Deer Island treatment plants, four communities (Boston, Cambridge, Chelsea, and Somerville) have combined sewers. These combined sewers handle domestic waste, commercial waste, industrial waste, and storm water runoff. During wet periods, a larger volume of flow goes through the system than the two treatment plants can handle. This additional flow is diverted into the harbor through combined sewer overflows (CSO). At present, there are over 100 CSOs, of which 87 are discharging into the harbor (Fig. 2-2).

New primary and secondary treatment facilities, which will serve both the North and South systems, will be constructed on Deer Island. The existing treatment facilities on Nut Island will be decommissioned following startup of the new primary facilities on Deer Island and preliminary treatment, consisting of screening and grit removal, will be provided at a new headworks on Nut Island.

North System

The Deer Island Primary Treatment Plant provides treatment for wastewater from MWRA's North System, which encompasses an area of approximately 440 km^2. The total population of the area is approximately 1.3 million of which 96% are connected to the sewerage system. The system services the following 26 communities:

Arlington	Lexington	Stoneham
Bedford	Malden	Wakefield
Belmont	Medford	Waltham
Boston (portion)	Melrose	Watertown
Brookline (portion)	Milton (portion)	Wilmington
Burlington	Newton (portion)	Winchester
Cambridge	Reading	Winthrop
Chelsea	Revere	Woburn
Everett	Somerville	

Wastewater from the North System is treated at the Deer Island Treatment Plant. Figure 2-3 shows a schematic of the headworks and tunnel system entering the plant. Wastewater enters the plant from the Winthrop Terminal and the Main Pumping Station.

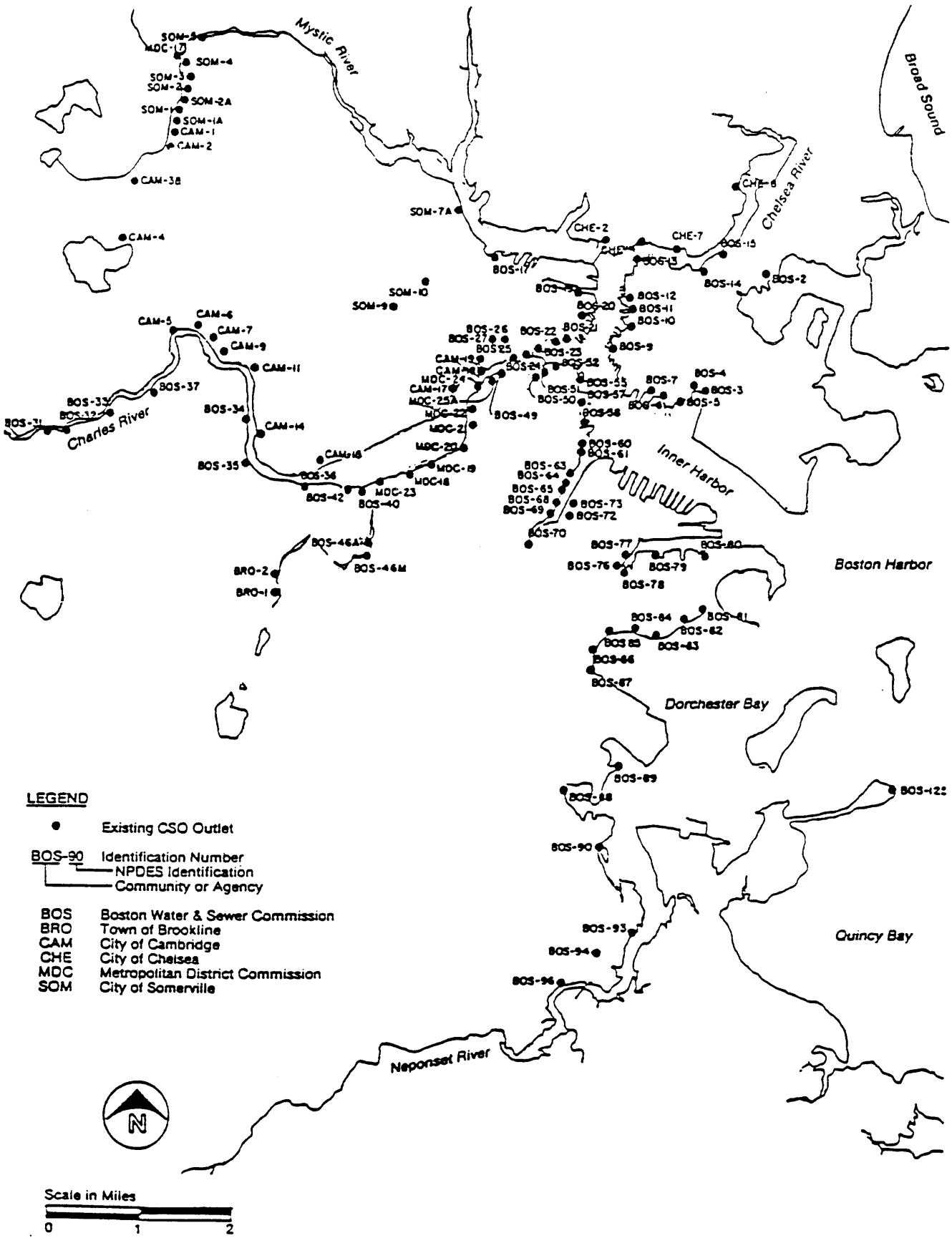

Figure 2-2: Locations of existing Combined Sewer Overflows (CH2M HILL, 1988)

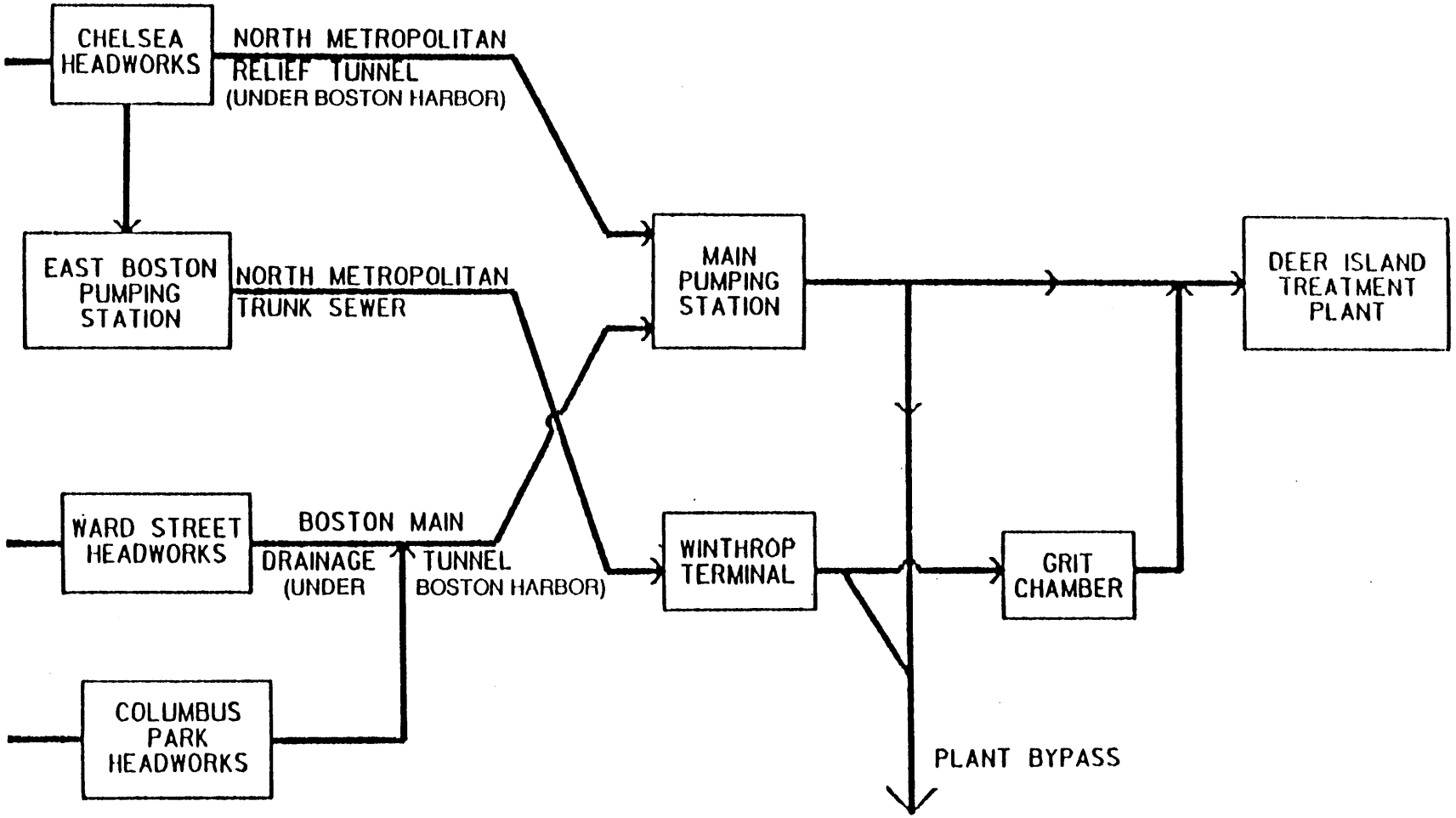

Figure 2-3: Schematic of the North System Headworks and Tunnels (MWRA, V-III, 1988)

Wastewater entering the Winthrop Terminal is pumped from the East Boston Pumping Station through the North Metropolitan Trunk Sewer which has a maximum flow rate of 470 mld. Flow in excess of 470 mld is devirted and discharged into the harbor through CSOs. Flow that reaches the Winthrop Terminal is screened. It then goes to the grit chamber, which can treat 280 mld. Flows in excess of 280 mld are discharged into the harbor before reaching the grit chamber.

Wastewater enters the Main Pumping Station from three headworks (Chelsea Creek, Ward Street, and Columbus Park) through two deep rock tunnels under the Boston harbor (North Metropolitan Relief Tunnel and Boston Main Drainage Tunnel). All three headworks were built in 1968 with screening and grit removal done at each location. The Chelsea Creek headworks can handle 1320 mld of flow, which is conveyed to the Main Pumping Station through the North Metropolitan Relief Tunnel. The Ward Street headworks can handle 970 mld and the Columbus Park headworks can handle 690 mld. Flow from both headworks is conveyed through the Boston Main Drainage Tunnel. All wastewater in excess of the maximum flow at each headworks is diverted and discharged into the harbor through CSO's (Fig. 2-2).

The wastewater from the North System receives primary treatment at the Deer Island facility. The following are the major components of this facility:

One influent pumping station, with nine pumps
Two pre-aeration channels
Eight primary sedimentation tanks
Four sludge thickening tanks, with three pumps each
Four anaerobic digesters
Four gravity thickening tanks
Five chlorinators
Five outfall dischargers

Figure 2-4 shows a schematic of the Deer Island Primary Treatment Plant. The wastewater flow enters the plant through the Main Pumping Station and the Winthrop Terminal at a maximum rate of 3460 mld. The maximum flow that can be treated at the plant is 3210 mld. Hence, all the flow in excess of 3210 mld is diverted and discharged through one of the effluent outfalls.

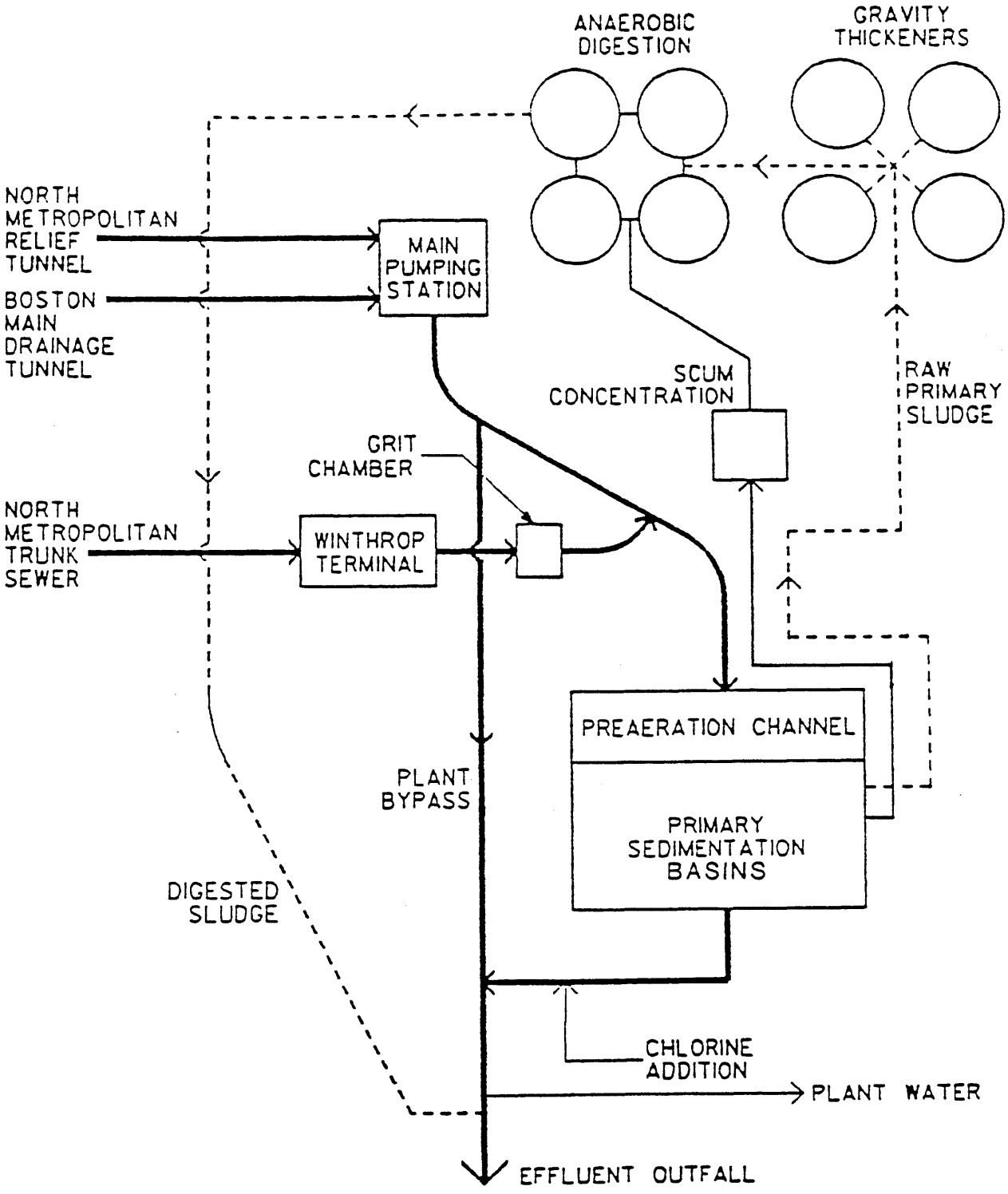

Figure 2-4: Schematic of the Existing Deer Island Treatment Plant (MWRA, V-III, 1988)

Wastewater entering the plant passes through two preaeration channels into eight primary sedimentation basins. Prior to December 1988, scum was removed from the sedimentation basins and released into the harbor after going through the anaerobic digesters and chlorination. As of December 1988, the scum is mixed with cement kiln dust (Watkins, 1988), a by-product of the cement making process, and allowed to harden. This process has two major advantages: it binds into a solid waste which otherwise is an unmanageable substance, and it significantly reduces bacteria and odor. The bound scum can either be disposed of on land or used as a cover for landfills. A different process that involves separating the liquid from the solid portion will be used at Nut Island by February 1989. Both processes will be evaluated for use at the new treatment plant proposed for 1995.

The effluent from the sedimentation basins is chlorinated, to provide disinfection, and is discharged throught two diffuser-equiped outfalls into President Roads approximately 610 m from Deer Island (Fig. 2-5). Two relief outfalls extend about 15 and 152 m respectively offshore. An emergency outfall also exists just offshore. Construction of these outfalls dates back to the 1890's, with the newest outfall being constructed in 1964. In 1978, the plant was capable of 32% removal of biochemical oxygen demand (BOD) and 58% removal of total suspended solids (TSS). The sludge from the sedimentation basins is pumped to four gravity thickeners, passed through the anaerobic digesters, is chlorinated, and then discharged into President Roads on the outgoing tide.

South System

The Nut Island Primary Treatment Plant provides treatment for wastewater from MWRA's South System, which encompasses an area of approximately 621 km^2. The total population of the area is approximately 750,000 of which 85% are connected to the sewerage system. The system services the following 21 communities:

Ashland	Hingham	Quincy
Boston (portion)	Holbrook	Randolph
Braintree	Milton (portion)	Stoughton
Brookline (portion)	Natick	Walpole
Canton	Needham	Wellesley
Dedham	Newton (portion)	Westwood
Framingham	Norwood	Weymouth

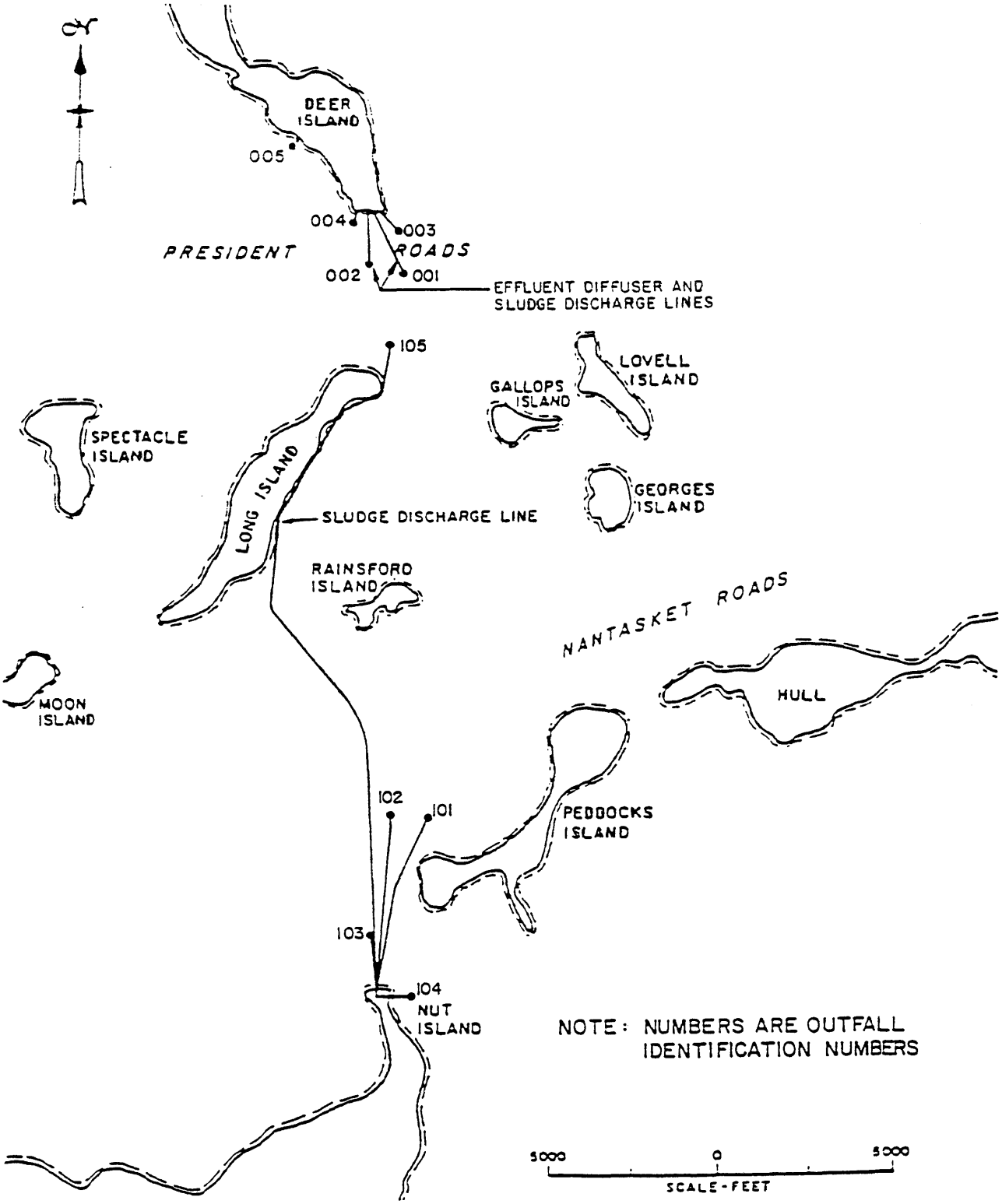

Figure 2-5: Existing Effluent and Sludge Discharge Locations (MWRA, V-III, 1988)

Wastewater from the South System is treated at the Nut Island Treatment Plant. The system consists of a series of sewers converging into the High Level Sewer (Fig 2-6). This sewer can handle a maximum flow of 1320 mld. Wastewater is lifted at five pumping stations (Quincy, Squantum, Hingham, Braintree–Weymouth, and Hough's neck) along the way. No screening or grit removal is done until the wastewater reaches the Nut Island treatment plant.

The wastewater from the South System receives both preliminary (screening and grit removal) and primary treatment at the Nut Island facility. The following are the major components of this facility:

Two canterary-type bar screens
Six grit chambers
One pumping stations with four pumps
Five pre-aeration basins
Six primary sedimentation basins
Four anaerobic digesters
Five outfall dischargers

Figure 2-7 shows a schematic of the Nut Island Primary Treatment Plant. The wastewater flow enters the plant through the High Level Sewer at a maximum rate of 1360 mld. The maximum flow that can be treated at the plant is 1060 mld. Hence, all the flow in excess of 1060 mld is diverted and discharged through one of the four effluent outfalls.

As the raw wastewater enters the plant it is screened, using two canterary-type bar screens, and then it passes through six grit chambers, which use the chain-and-bucket method. The screenings and grit are hauled offsite and disposed of. The effluent from the grit chambers is pumped to five pre-aeration basins and then to six primary sedimentation basins.

The effluent is chlorinated and discharged through two primary outfalls (Fig. 2-5), which were originally built in 1904 and 1914 and then reconditioned in 1986. A third outfall is used for overflow and a fourth, built in 1946 is used only in emergencies.

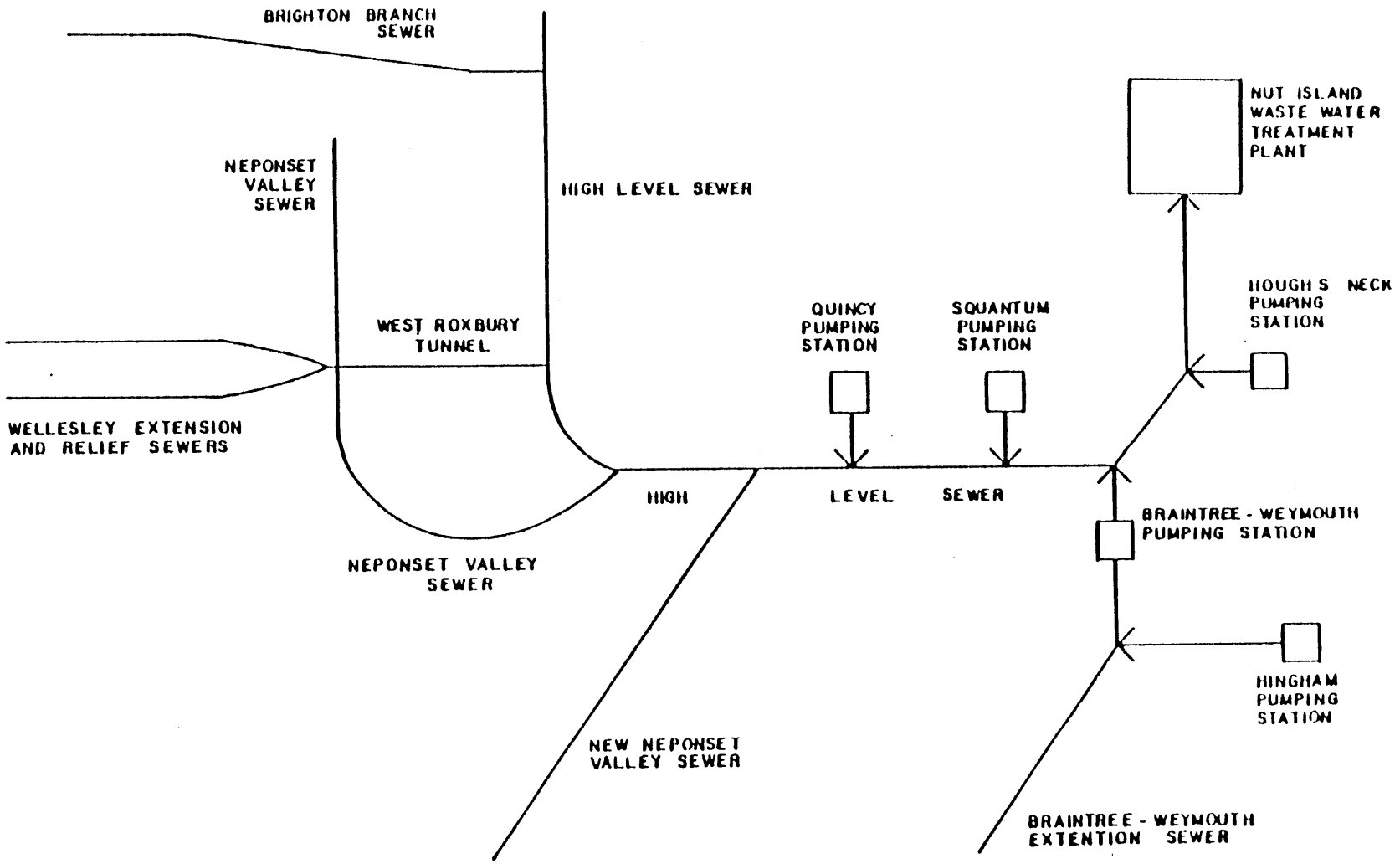

Figure 2-6: Shematic of the South Collection System (MWRA, V-III, 1988)

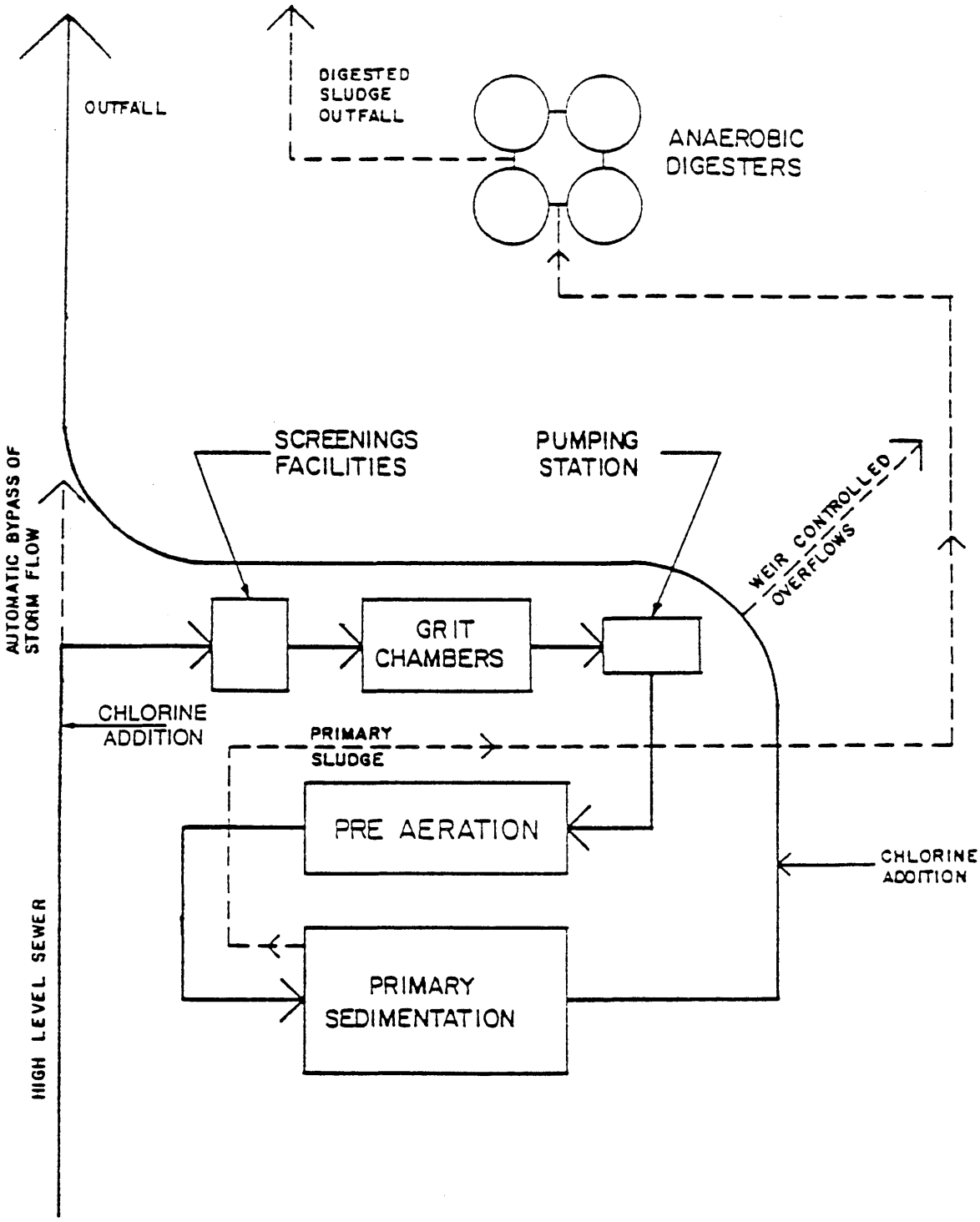

Figure 2-7: Schematic of the Existing Nut Island Treatment Plant (MWRA, V-III, 1988)

In 1978, the plant was capable of 25% removal of BOD and 53% removal of TSS. The sludge and grease produced is piped to four anaerobic digesters, disinfected, and then discharged through a fifth outfall, built in 1952, into the President Roads.

Flows and Loadings

There are four sources of flow that contributes to the wastewater entering MWRA's treatment plants on Deer and Nut Islands. These four sources are:

- domestic wastes from residential activity
- non-domestic wastes from commerial, industrial and other business related activities
- infiltration and inflows entering due to the age, condition, and location of the sewer pipes to the groundwater level
- stormwaters resulting from street drainage intentionally allowed to enter the sewer ssystem.

Loading of pollutants results from conventional and non-conventional sources. Conventional pollutants are biochemical oxygen demand (BOD) and total suspended solids (TSS). Non-conventional pollutants are metals, acid-based neutrals (ABN's), pesticides and PCB's, and volatile organic compounds (VOC's).

As seen in Fig. 2-8 wastewater from domestic and nondomestic sources ranges from 800 mld to 1100 mld. Domestic wastewater contributes an average of 470 mld and non-domestic wastewater contributes an average of 340 mld to the overall flow entering the system. The maximum estimated infiltration and inflow for both the North and South system ranges from 570 mld to 1280 mld during low groundwater conditions and from 1630 mld to 2610 mld during high groundwater conditions. Low groundwater conditions are expected June through January (eight months) and high groundwater conditions are expected February through May (four months). The maximum annual stormwater event adds an additional 2080 mld to the flow from both systems.

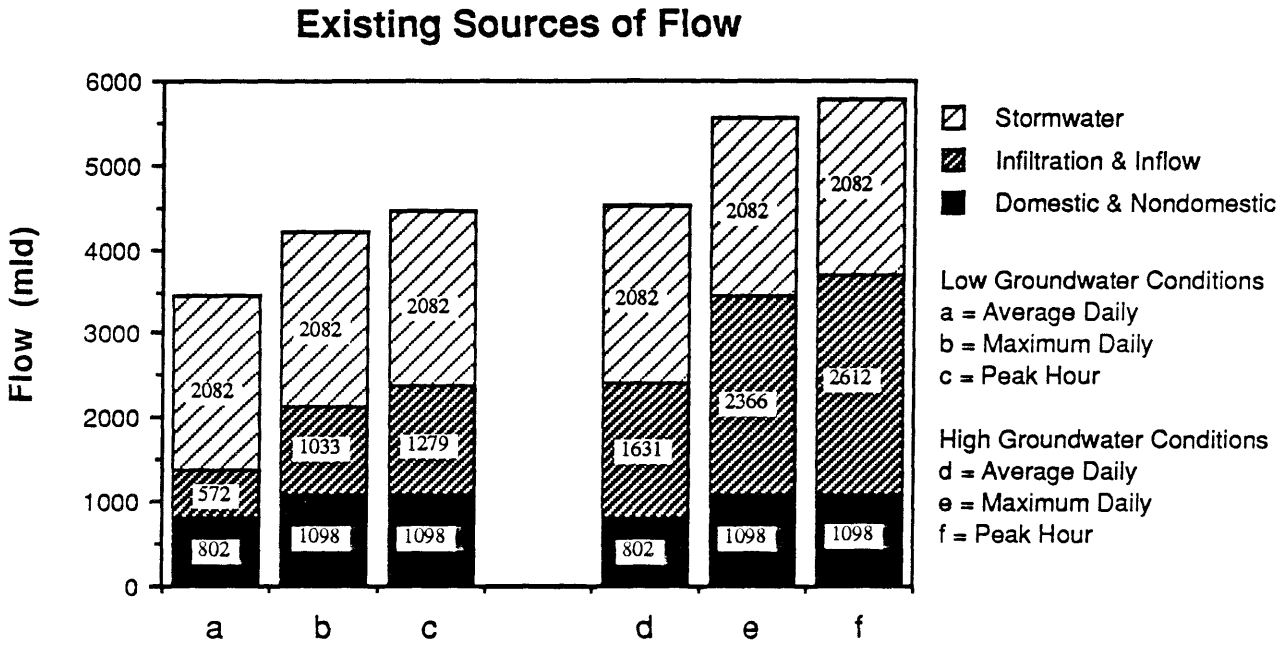

Figure 2-8: Existing Sources of Flow for the North and South Systems.

The influent and effluent loading and concentration of conventional pollutants are shown in Table 2-1 with BOD removals ranging from 25 to 32% and TSS removals ranging from 53 to 58%. Non-conventional concentrations where determined using an average loading during non-storm events from the Deer Island plant plus the average loading during non-storm events from the Nut Island plant. The existing metals loading can be found in Table 2-2, the existing ABN's loading can be found in Table 2-3, there were no detectable loading for PCB's and pesticides, and the existing VOC loading can be found in Table 2-4.

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Table 2-1

Existing Influent and Effluent Loadings and Concentrations of Conventional Pollutants

	Influent		Effluent	
Constituents	Loading	Concentration	Loading	Concentration
BOD_5	226,800	180	158,700	126
TSS	201,800	155	88,800	68

Note: Loading is in kg/day. Concentration is in mg/liter

Table 2-2

Existing Metal Loadings
(kg/day)

Constituents	Average Loading	Standard Deviation
Antimony	4.9	1.0
Arsenic	2.7	0.9
Boron	571	1110
Cadmium	3.2	1.2
Chromium	34.4	13.4
Copper	156	46.7
Cyanide, Total	24.3	6.3
Lead	22.6	9.9
Mercury	1.9	2.4
Molybdenum	7.8	4.7
Nickel	29.9	13.4
Selenium	16.0	15.1
Silver	7.0	1.7
Zinc	86.2	382

Note: Average load represents the non-storm mean load at Deer Island plus the non-storm load at Nut Island. Loadings are based on influent values only (MWRA, V-III, 1988).

Table 2-3

Existing ABN Loadings
(kg/day)

Constituents	Average Loading	Standard Deviation
Phenol	24.5	12.3
Benzyl Alcohol	31.4	11.2
1,2-Dichlorobenzene	29.5	10.6
2-Methylphenol	32.2	8.3
4-Methylphenol	27.8	10.5
Benzoic Acid	122	63.0
Naphthalene	20.5	12.9
2-Methylnaphthalene	22.5	12.1
2,4,5-Trichlorophenol	158	43.4
Dimethyl Phthalate	31.6	8.8
Diethyl Phthalate	25.9	11.1
N-Nitrosodiphenylamine (1)	31.4	8.9
Di-n-butyl Phthalate	26.3	11.7
Butylbenzyl Phthalate	24.5	10.9
Bis (2-ethylhexyl) Phthalate	30.7	9.1
Di-n-octyl Phthalate	25.9	9.7

Note: Average load represents the non-storm mean load at Deer Island plus the non-storm load at Nut Island. Loadings are based on influent values only (MWRA, V-III, 1988).

Table 2-4

Existing Volatile Loadings
(kg/day)

Constituents	Average Loading	Standard Deviation
Bromomethane	24.6	8.8
Methylene Chloride	47.6	34.1
Acetone	153	122
Carbon Disulfide	12.5	3.4
trans-1,2-Dichloroethane	11.6	3.5
Chloroform	8.0	3.7
2-Butanone	37.4	19.9
1,1,1-Trichloroethane	19.0	8.1
Trichloroethane	16.4	8.7
Benezene	5.7	1.0
4-Methyl-2-Pentanone	29.3	10.7
Tetrachloreethane	21.5	12.6
1,1,2,2-Tetrachloroethane	13.3	2.9
Toluene	27.6	17.3
Chlorobenzene	12.7	3.4
Ethylbenzene	13.1	5.9
Styrene	13.7	3.3
Total Xylene; M, O, P	39.0	26.9

Note: Average load represents the non-storm mean load at Deer Island plus the non-storm load at Nut Island. Loadings are based on influent values only (MWRA, V-III, 1988).

Chapter 3

Conflicts Between Primary and Secondary Treatment (Technology versus Policy)

Conflicts Between Primary and Secondary Treatment (Technology versus Policy)

Clean Water Act

Prior to 1972, the Federal Water Pollution Control Act (FWPCA) prescribed a regulatory system consisting mainly of state-developed ambient water quality standards applicable to interstate or navigable waters. The standards depended on the uses that the state wanted to facilitate. Enforcement was possible only when discharges reduced the quality of the receiving water below the specified level. This system failed due to lack of enforcement. There were no regulations on ocean disposal at this time.

In 1972, Congress amended the FWPCA and established a system of standards, permits, and enforcements aimed at having fishable and swimmable water by 1983 and totally eliminating pollutant discharges into navigable water by 1985. Dischargers were required to have uniform (secondary) treatment for all wastes regardless of the receiving water by 1977 and best practicable waste treatment over the life of the works by 1983. Also in this amended FWPCA, a permit system was created. Under the permit system, the National Pollution Discharge Elimination System (NPDES), discharge permits could be granted by the Environmental Protection Agency (EPA) or by states with EPA approved programs. Any discharge not in compliance with the permit was unlawful.

Under the 1972 amendment, compliance would be enforced by the state, although, the federal government could also enforce the act. Under section 505 (Water Pollution Control Federation, 1982), citizens could sue for failure to perform nondiscretionary regulatory duties. For example, in 1982, the City of Quincy sued for unlawful discharges into Quincy Bay.

Also at this time, the federal government made funding available (up to 85% of the cost) for the construction of secondary treatment facilities.

In 1977, the FWPCA was again amended and became known as the Clean Water Act (CWA). A provision, section 301(h), authorized the EPA to waive the secondary treatment requirement. Waivers could be granted under these conditions: discharges were into marine water with a long outfall, the treated sewage would not interfere with the

attainment or maintenance of the receiving water quality, and the protection of public water supply, aquatic life, and recreational activities was assured.

The CWA has been a controversial act since its creation. It set technology standards rather than receiving water standards as the Clean Air Act (CAA) did before it. The setting of technology standards restricts the application of new technology and precludes site specific investigations. These two arguments have been the focus of controversy since the enactment of the CWA and are again surfacing in the issues associated with the Boston Harbor cleanup (Boston Globe, 1989; Harleman, 1989b - see Appendix A2).

On June 15, 1979, EPA published regulations for the 301(h) waiver applications (Federal Register, 1979). This was two years after the passage of the CWA and the EPA allowed 90 days to prepare and file the application in accordance with its very detailed regulations. Eighty municipalities filed for the 301(h) waiver.

Waiver

In the Fall of 1979, the Metropolitan District Commission (MDC) filed a 301(h) waiver application. MDC felt that they were entitled to the waiver based on the information obtained during previous studies on Boston Harbor. This information is summarized below (The Commonwealth of Massachusetts, 1979):

- In spite of Boston Harbor's long history of receiving untreated sewage, conditions at the existing effluent outfalls were found to be relatively unpolluted.

- There had been no oxygen violations found in the harbor except in the Inner harbor where the primary cause of these violations were from Combined Sewer Overflows (CSO) of raw sewage.

- Violations of the NPDES permit were bacterial in nature and found to occur during or following storms, and when chlorination at the treatment plants was not operating properly.

- Industrial pretreatment requirements were being initiated and metals discharges were expected to be reduced.

- Alternative methods of sludge disposal were being developed.

- Studies indicated that there would be minimal environmental improvement with secondary treatment compared to the increased cost over primary treatment. Money should be used for more significant pollution problems such as CSOs.

During the time that the waiver application was being reviewed by EPA (by their consultants Tetra Tech, Inc.), no federal funding for Boston Harbor was made available.

In July 1981, EPA requested additional information on the sensitivity of the far-field model, analysis of priority pollutants in Deer and Nut Islands effluent, additional sediment and benthic sampling and analysis, and an assessment of sediment deposition and resuspension (The Commonwealth of Massachusetts, 1982). No request at that time was made for further information related to dissolved oxygen (DO). Although, DO violations were later sited as the main reason for denial of the waiver application.

On December 17, 1982, the City of Quincy, filed suit against MDC charging them with violation of the laws prohibiting discharges into coastal and tidal waters, and violations of the common law of nuisance.

On July 8, 1983, Judge Paul Garrity of the court of Massachusetts appointed Charles M. Haar, Harvard Law School Brandeis Professor of Law, as Special Master. The Special Master was charged with resolving disputed issues concerning facts, hearing evidence, and compiling of facts associated with the Boston Harbor "cleanup". The following is a summary of the recommendations (The Commonwealth of Massachusetts, 1983) by Haar.

- Planned Infiltration and Inflow (I/I) reduction. This would be accomplished by removing two liters of untreated sewage from the system for every new liter of untreated sewage entering the system.

- Increase capacity and upgrades to the present primary treatment facilities at Deer and Nut Islands in order to eliminate raw sewage discharges.

- Identify and control the pollutants discharged through CSO.

- Establish an authority responsible for the cleanup of Boston Harbor. This authority would have the ability to issue bonds to financially support the cleanup.

On June 30, 1983, EPA announced a tentative decision to deny MDC's request for a waiver from secondary treatment. This denial was based on potential DO violations, excessive deposition of suspended solids, potential adverse impacts on marine life, and violation of PCB water quality criteria. After reviewing EPA's decision, MDC met with the EPA National Waiver Task Force of the Office of Marine Discharge. It became apparent that the issues raised by EPA could be resolved by MDC and that they should resubmit the 301(h) waiver application. The additional documents submitted for reapplication (The Commonwealth of Massachusetts, 1984) indicated:

- Additional characterization of the discharge from MDCs treatment plants.
- Additional DO data collection and analysis.
- Additional toxic pollutant data collection and analysis, particularly related to PCB's.
- Additional biological data collection and analysis including phytoplankton data.
- Evaluation of alternative outfall diffuser locations.
- Further assessment of sediment deposition and resuspension.

In the process of preparing the resubmitted waiver application, MDC was able to collect an extensive amount of data in Massachusetts Bay on physical oceanography, physical sediment characteristics, water quality, and marine biology. In particular, this investigation showed that the median water column average DO concentration was 8.4 milligrams per liter (mg/l) with the average DO concentration in the bottom 10 meters of 7.6 mg/l during 1978 - 1979. During 1984, the median water column average DO concentration in the proposed region was 8.1 mg/l with the average DO concentration in the bottom 10 meters of 7.9 mg/l. No DO profiles dropped below the state standard of 6.0 mg/l (Metcalf & Eddy, Inc. 1985).

In the summer of 1984, MDC submitted the revised waiver application.

On March 29, 1985, EPA issued the second tentative denial of the waiver from secondary treatment. EPA presented its decision on the basis of the following seven findings (EPA, 1985):

- "The proposed discharge is expected to violate the Commonwealth of Massachusetts' water quality standard for dissolved oxygen during summer resuspension events, but is not expected to violate the Commonwealth's standard for suspended solids."

- "The proposed discharge is expected to interfere with the protection and propagation of a balanced, indigenous population of marine life and may not allow for recreational activities. The proposed discharge will not adversely impact public water supplies."

- "The Applicant has established a system for monitoring the impact of its discharge. ... This program contains deficiencies ..."

- "The proposed discharge may impact other point and non-point sources to the north and west."

- The Applicant has developed a program to enforce all applicable pretreatment requirements. ... A recent EPA audit shows that the program has not been adequately administered and enforced."

- "The Applicant has proposed a schedule of activities intended to limit the entrance of toxic pollutants from nonindustrial sources into the treatment works. ... This schedule contains deficiencies ..."

- "There may be new or substantially increased discharges of pollutants in the effluent from the proposed discharge above that specified in the permit."

Three of the seven findings EPA used as basis for denial are procedural in nature. These include the establishment of a monitoring program, development of enforcement procedures for an industrial pretreatment program, and the development of non-industrial

source control program to eliminate toxic pollutants from entering MDC's sewers. Procedural matters should not be a basis upon which a denial decision is based.

Massachusetts Division of Water Pollution Control conducted a study to determine if the proposed MDC discharge would effect areas to the north and west such as Lynn, Swampscott, and Gloucester and found that these communities would not suffer any adverse affects. Similarly, EPAs contractors concluded that under worst conditions, the potential combined impact is below the level used in their waiver review process to indicate potential impacts (Tetra Tech, Inc., 1984). Therefore, this should not be a basis for denial.

EPA's claim that MDC's flow will substantially increase over the amount stated in the permit is unlikely. The flow estimates used in the 301(h) waiver application are the same values as the ones used by MDC and EPA in completing its Environmental Impact Statement (EIS) on Boston Harbor (Metcalf & Eddy, Inc., 1985). Therefore, this should not be a basis for denial.

The remaining two findings were related to DO criteria and oxygen impacts on the biological community. Of these two findings, only one is quantitative in nature and deals with DO values falling below the Commonwealth's water quality standard.

In evaluating DO impacts for the waiver process, four categories were assessed:

- Initial DO demand
- Far-field DO demand
- Benthic DO demand
- DO demand caused by an abrupt sediment resuspension event.

When reviewing MDC's waiver application, Tetra Tech, Inc., under contract from EPA to review all 301(h) waiver applications, concluded that there would not be any water quality violations based on the four DO categories mentioned above (Tetra Tech, Inc., 1984). This result was obtained even though Tetra Tech, Inc. claimed that there was almost a two order of magnitude difference in the settling accumulation rate calculated by MDC in their 1984 waiver application to EPA. This difference results almost entirely from the selected values for settling velocity distribution. Tetra Tech selected 0.1 cm/sec MDC selected 0.01 cm/sec. Sediments that would settle at a rate of 0.1 cm/sec are removed in the

primary sedimentation tanks prior to discharging the effluent (Morrissey and Harleman, 1989 - see Appendix A3).

At this point, EPA requested Tetra Tech to reassess the DO impacts based on a worst case hypothetical DO profile. The hypothetical profile (Fig. 3-1) was determined from subsets of all DO data representing the lowest DO concentrations at each depth (Tetra Tech, Inc., 1985). Based on this worst-case profile and an abrupt resuspension event after a critical 90 day accumulation period of sedimentation, Tetra Tech concluded that this would result in a DO violation (fall below 6.0 mg/l) of the Commonwealth of Massachusetts water quality standard. Therefore, the discrepancy in the settling velocity distribution and the choice of a hypothetical DO profile were the ultimate cause of the waiver application denial (Harleman, 1989a - see Appendix A1).

Creation of a New Authority

In 1984, political mayhem broke out in Boston because of controversy over the Boston Harbor cleanup issues. This resulted in a bill creating the Massachusetts Water Resource Authority (MWRA). This bill was signed into law by Governor Michael Dukakis in mid-December 1984. The bill gave MWRA the power to issue bonds in order to raise money for the Boston Harbor cleanup.

In January 1985, Michael Deland, head of EPA's Northeast Regional Office, sued MWRA in federal court as the responsible party for the cleanup of Boston Harbor, forcing MWRA to work under a federal court order since the day they were created.

In the final analysis, MWRA decided not to pursue the appeal for the 301(h) waiver denial and proceeded with plans to build a secondary treatment plant with a 14.5 km outfall, and sludge disposal facilities at an estimated cost of $6.1 billion. The right to appeal the tentative denial decision, for the 301(h) waiver, was forfeited 30 days after MWRA agreed to a new NPDES permit (Folley, 1989). This permit limited the effluent discharged from MWRA's proposed treatment facilities.

Currently, MWRA has a scheduled court order for completing the various stages of the secondary treatment facilities (see Chapter 4). Also, the 85% funding of this construction by the federal government is no longer available.

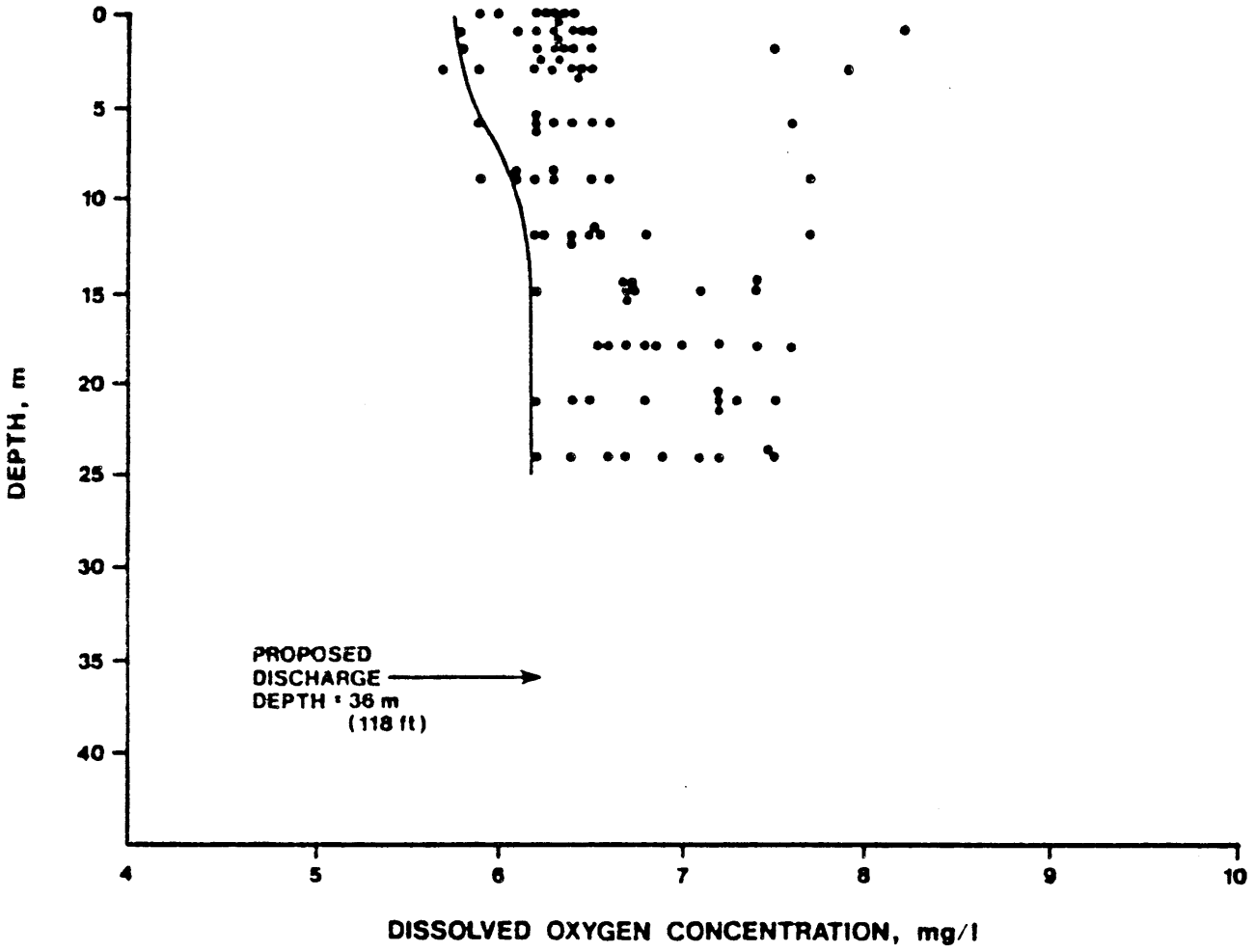

Figure 3-1: Selected dissolved oxygen concentrations measured near the proposed discharge site (Tetra Tech, Inc., 1985)

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Chapter 4

Proposed Treatment Facilities

Proposed Treatment Facilities

Introduction

Introduction

Currently, wastewater is treated by two primary treatment facilities on Deer and Nut Islands. In 1972, EPA mandated full secondary treatment of wastewater. In 1979, EPA allowed for exceptions to full secondary treatment under certain conditions. The Boston Metropolitan area applied for a waiver from secondary treatment (see Chapter 3) and was denied. As a result, MWRA and EPA agreed to a NPDES permit which set limitations on the conventional pollutants discharged from the proposed treatment plant into the ocean. This permit requires MWRA to meet the following discharge limits:

	TSS	BOD
Monthly Average Basis	30 mg/l	30 mg/l
Weekly Average Basis	45 mg/l	45 mg/l
Daily Average Basis	50 mg/l	50 mg/l
Minimum Monthly Removals	85%	85%

MWRA has proposed to meet these discharge limitations by consolidating both the Deer and Nut Island treatment plants into one facility on Deer Island. This facility will be able to handle a flow of 4810 mld and consists of the following major components:

- Preliminary Treatment
- Primary Treatment
- Secondary Treatment
- Disinfection
- Ocean Outfall
- Sludge Disposal

Once the new Deer Island treatment facilities are operational, the existing Nut Island primary treatment plant will be decommissioned and replaced with a new headworks. This headworks will provide screening and grit removal for the South system wastewater flow before it enters the new Deer Island treatment facilities.

The newly proposed treatment plant (Fig. 4-1) is expected to be completely operational by 1999. A break down of the components of the treatment plant (Fig. 4-2) is given below:

- North System Remote Headworks
- North Main Pumping Station
- Winthrop Terminal
- North System Grit Removal Facilities
- Nut Island Headworks
- Inter-Island Conveyance System
- South Pump Station
- Primary Treatment Facilities
- Secondary Treatment Facilities
- Air Emissions Facilities
- Disinfection
- Ocean Outfall

These components will be discussed in detail in the remainder of this chapter.

Construction Schedule

Two broad construction schedules were outlined.

- Construction Schedule – Base Plan
  - 100% Primary by 1995
  - 100% Secondary by 1999

- Construction Schedule – Alternate Plan
  - 100% Primary by 1996
  - 25% Secondary by 1996
  - 100% Secondary by 1999

In February 1989, MWRA published a detailed schedule (MWRA, 1989) for the construction related to the proposed secondary treatment facilities (Table 4-1). This schedule also outlines contract bidding information. On May 31, 1989, it was started that a decision was made to accelerate the "cleanup" schedule so that construction of 25% of the secondary treatment facility will be started in 1993 instead of 1995 (Tate, 1989).

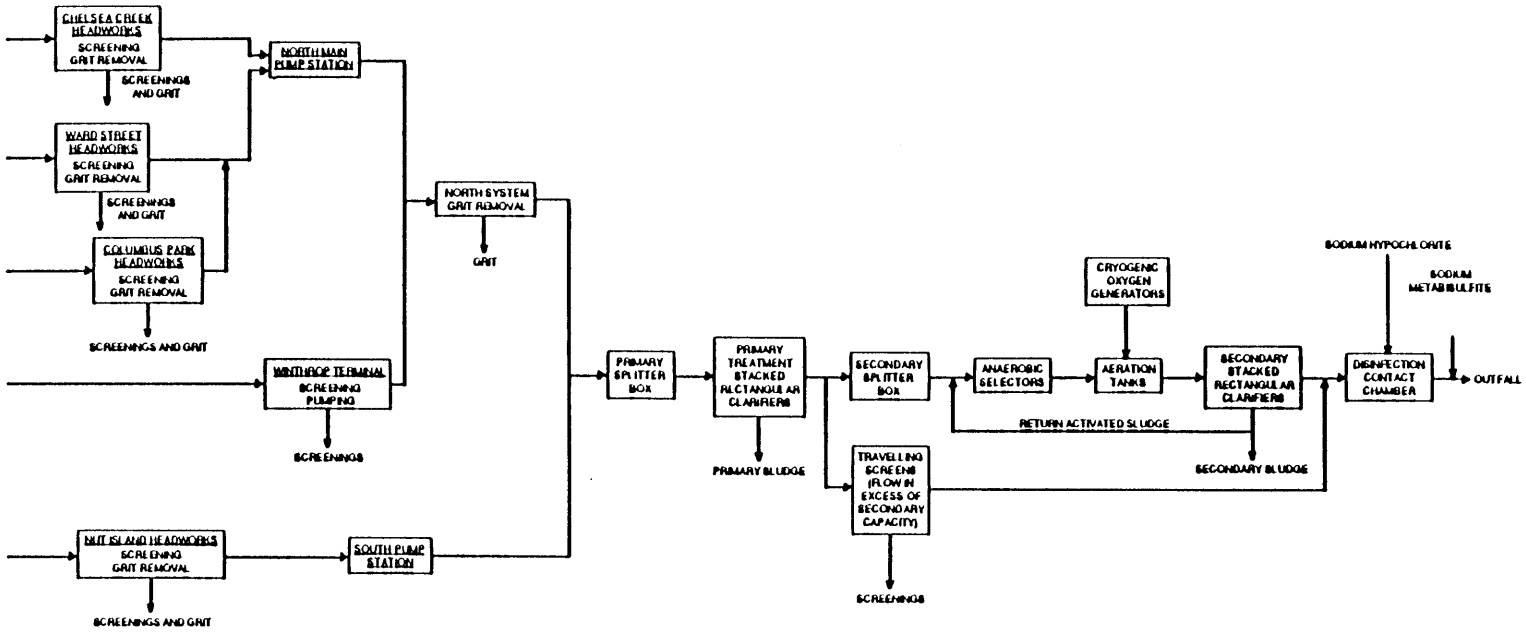


Figure 4-1: Schematic of the Proposed Treatment Facilities (MWRA V-III, 1988)

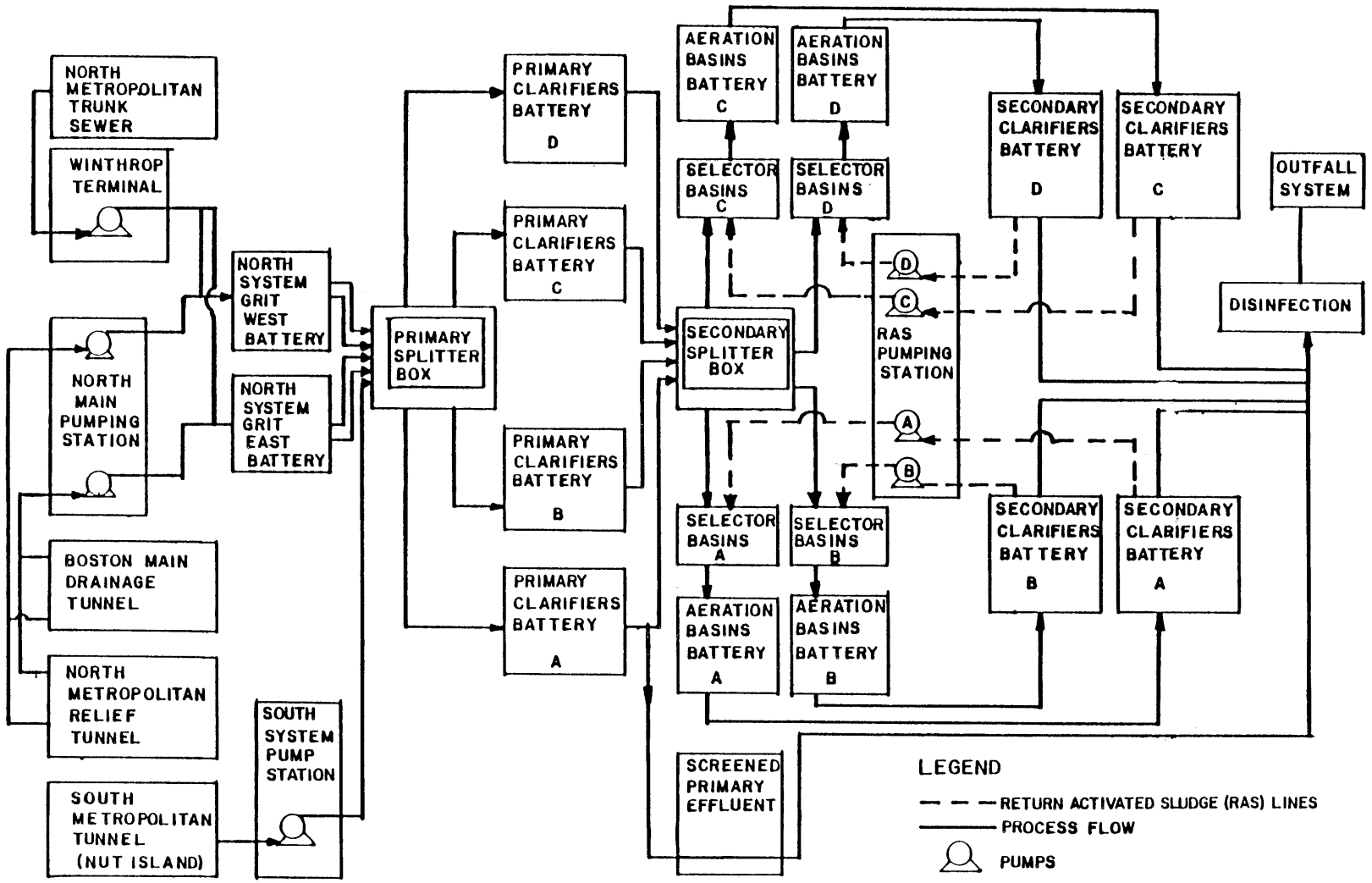


Figure 4-2: Flow Schematic of the Wastewater Treatment Process (MWRA V-III, 1988)

Table 4-1: Construction Schedule for Proposed Treatment Facilities (MWRA, 1988)

LDE & P/CM CONT.PKG.#	FACILITY	Dsgn. Pkgs.	Designer	DESIGN SCHEDULE		CONSTRUCTION SCHEDULE			
				Start	Finish	Advertise	Award	Start	Finish
GROUP 000	GENERAL SITE FACILITIES								
1	MARINE FACILITIES AT DEER ISLAND	N/A	MGI		COMPLETED		JMC	Aug-88	Nov-89
2	ON-SHORE MARINE TRANSPORTATION FACILITY - QUINCY FRSA	N/A	PBQD		COMPLETED		JMC	Apr-89	May-90
3	FRSA UPLAND FACILITIES	N/A	PBQD	Oct-88	Jun-89	Jun-89	Sep-89	Sep-89	Sep-89
4	ON-SHORE MARINE TRANSPORTATION FACILITY - MYSTIC RIVER	20	FRH	Jul-89	Mar-90	Mar-90	Jun-90	Jun-90	Dec-90
5	SQUANTUM POINT PIER FACILITY	20	"	May-89	Nov-89	Sep-89	Mar-90	Mar-90	Mar-91
6	BEVERLY STREET PIER FACILITY	20	"	May-89	Oct-89	Nov-89	Mar-90	Mar-90	Jul-90
7	REVERE BUS TERMINAL	20	"	May-89	Oct-89	Oct-89	Feb-90	Mar-90	Sep-90
20	ASBESTOS REMOVAL	01	BA	Oct-88	COMPLETED	Mar-89	DT	Jun-89	Oct-89
21	DEMOLITION - BUNKERS, ETC.	01	"	Oct-88	COMPLETED	May-89	JMC	Jul-89	Jul-90
22	EARTHWORK, ROAD, LANDFILL	01	"	Oct-88	Aug-89	Sep-89	Nov-89	Jan-90	Mar-91
23	OUTFALL PIPE PROTECTION, PUMP STATION, ETC.	01	"	Oct-88	Aug-89	Aug-89	Oct-89	Oct-89	Jun-90
24	ADMINISTRATION BUILDING & LABORATORY	07	M & E	Jan-90	Jul-90	Sep-90	Jan-91	Feb-91	Nov-92
25	MAINTENANCE BUILDING	08		Sep-89	Aug-90	Sep-90	Mar-91	May-91	May-93
40	PRISON DEMOLITION	12	M & E	Mar-90	Feb-91	Apr-91	Dec-91	Jan-92	Jun-92
41	LATE DRUMLIN EXCAVATION	12	"	Mar-90	Feb-91	Oct-91	Mar-92	May-92	Mar-93
42	INITIAL PAVING & LANDSCAPING	12	"	Mar-90	Feb-91	Oct-94	Mar-95	Apr-95	Sep-95
43	LATE SITE PREP & DEMOLISH OLD PLANT	14/15		Jan-92	Dec-92	Apr-95	Oct-95	Nov-95	Jul-96
44	FINAL PAVING & LANDSCAPING	14/15		Jan-92	Dec-92	Oct-97	May-98	Jun-98	Dec-99
45	DRY STORAGE BUILDING	18		Jul-91	Jan-93	Apr-95	Jan-96	Apr-96	Jan-98
46	VEHICLE MAINTENANCE BUILDING	18		Jul-91	Jan-93	Apr-95	Jan-96	Apr-96	Jan-98
GROUP 100	PRIMARY TREATMENT FACILITIES								
101	NORTH SYSTEM FORCE MAIN	09	M-P	Jun-89	Feb-90	May-90	Nov-90	Dec-90	Jun-92
102	NORTH MAIN PUMP STATION MODIFICATIONS	09	"	Jun-89	Jul-90	Sep-91	May-92	Jun-92	Jun-95
103	NORTH SYSTEM HEADWORKS FACILITY	10	M & E	May-89	Apr-90	May-90	Dec-90	Apr-91	Dec-93
104	SOUTH SYSTEM PUMPING STATION	10	"	May-89	Sep-90	Jul-91	Feb-92	Apr-92	Nov-94
105	PRIMARY CLARIFIERS BATTERIES A&B	11	M & E	May-89	Jun-90	Jul-90	Feb-91	Apr-91	Jun-94
130	PRIMARY CLARIFIERS BATTERIES C&D	12	M & E	Mar-90	Feb-91	Apr-92	Dec-92	Mar-93	Jun-95
150	MARINE FACILITIES AT NUT ISLAND	N/A	MGI		COMPLETED		SCC	Jan-89	Jan-90
151	INTER-ISLAND TUNNEL & SHAFTS	05	SC	Feb-89	Dec-89	Jan-90	Dec-90	Apr-91	Sep-94
152	NUT ISLAND HEADWORKS, ODOR CONTROL, AND DEMOLITION	04		Mar-90	May-91	Jun-91	Dec-91	Apr-92	Sep-94

Table 4-1: Construction Schedule for Proposed Treatment Facilities (Continued)

GROUP 200	**SECONDARY TREATMENT FACILITIES**								
201	PILOT PLANT	19	CDM	Jun-89	Dec-89	Jan-90	Mar-90	Apr-90	Dec-90
202	CRYOGENIC GENERATOR UNIT 1 OF 3	14/15		Oct-90	Mar-92	Apr-92	Dec-92	Jan-93	Jun-95
203	SECONDARY CLARIFIER BATTERY A	14/15		Oct-90	Mar-92	Apr-93	Dec-93	Apr-94	Aug-96
204	DISINFECTION FACILITIES	16	M & E	Oct-90	May-91	Oct-91	Mar-92	Apr-92	Mar-94
205-b	WATER STORAGE TANK - 2 MG CAPACITY	21		Aug-90	Jul-91	Aug-91	Mar-92	Apr-92	Sep-93
205-a	DIESEL TANKS	23		Oct-89	Oct-90	Oct-90	May-91	May-91	May-92
240	SECONDARY CLARIFIER BATTERY B CRYOGENIC GENERATOR - UNIT 2 OF 3	14/15 14/15		Oct-90	Mar-92	Apr-95	Nov-95	Dec-95	Oct-97
260	CRYOGENIC GENERATOR - UNIT 3 OF 3	14/15		Jan-92	Dec-92	Apr-96	Dec-96	Mar-97	Jun-99
261	SECONDARY CLARIFIER BATTERY C & D ANAEROBIC SELECTOR BASIN C & D AERATION BASINS C & D SECONDARY ODOR CONTROL - BATTERY C& D EQUIPMENT	14/15 14/15 14/15 14/15		Jan-92	Dec-92	Apr-96	Dec-96	Mar-97	Jun-99
281	EFFLUENT OUTFALL SHAFT EFFLUENT OUTFALL TUNNEL & DIFFS.	06a 06	M & E PBQD	Mar-89 Feb-89	Dec-89 Dec-89	Jan-90	Oct-90	Oct-90	Jul-95
GROUP 300	**RESIDUALS PHASE I**								
301	RESIDUALS 1-A for PRIMARIES A & B	13	B & V	Jun-89	Sep-90	Apr-91	Sep-91	Sep-91	Apr-94
302	RESIDUALS 1-B for PRIMARIES C & D	13	"	Jun-89	Sep-90	Nov-91	Apr-92	May-92	Apr-95
303	RESIDUALS 1-C for SECONDARY A	17	"	Jun-89	Sep-90	Jul-92	Dec-92	Apr-93	May-96
330	RESIDUALS 2 for SECONDARIES B, C, & D	17		Dec-90	Oct-92	Oct-94	May-95	Jun-95	Jun-99
GROUP 400	**UTILITIES (PERMANENT)**								
401	INCOMING WATER, GAS, & CONDUITS	N/A	B & V	Dec-88	Oct-89	Oct-89	Feb-90	Feb-90	Jun-91
402	YARD PIPING PHASE I - WEST & SOUTH	24		Sep-89	May-90	Jun-90	Dec-90	Apr-91	Dec-93
420	CROSS-HARBOR CABLE DEER ISLAND INTERIM SUBSTATION - BECO SIDE	N/A N/A	BE/SW "	Sep-88	COMPLETED Jul-89	Oct-88	CL Aug-89	Sep-89 Aug-89	Jul-90 May-90
423	DEER ISLAND INTERIM SWITCHGEAR - 13.8 kV SIDE	22	LA	Jul-89	Jan-90	Jan-90	Apr-90	Apr-90	Oct-90
424	POWER DISTRIBUTION - PHASE I (W & S)	03		Aug-89	Feb-90	Feb-90	Sep-90	Sep-90	Mar-94
425	POWER DISTRIBUTION - PHASE II (E & N)	03		Aug-89	Sep-90	Apr-93	Dec-93	Apr-94	Dec-96
426	PERMANENT SWITCHGEAR - BECO 115 kV SIDE	N/A	BE/SW	Jun-89	Mar-90	Jul-90	Feb-91	Apr-91	Jun-94
427	PERM. SWITCHGEAR & TRANSFER YARD - 13.8 kV SIDE	03		Aug-89	Feb-90	Jul-90	Jan-91	Apr-91	Jun-94
428	GAS TURBINE FACILITY	23		Dec-89	Dec-90	Jan-91	Apr-91	Apr-91	Jun-94
450	INSTRUMENTATION & CONTROL - PHASE I	25		Sep-89	Aug-90	Apr-92	Dec-92	Apr-93	Jun-96
451	INSTRUMENTATION & CONTROL - PHASE II	25		Sep-89	Aug-90	Apr-96	Dec-96	Apr-97	Jun-99
452	PLANT COMMUNICATION SYSTEM	25		Sep-89	Aug-90	Apr-92	Dec-92	Apr-93	Jun-96
470	YARD PIPING - RESIDUALS	13	B & V	Jun-89	Sep-90	Oct-90	Mar-91	Apr-91	Sep-93
471	POWER DISTRIBUTION - RESIDUALS	13	"	Jun-89	Sep-90	Oct-90	Mar-91	Apr-91	Sep-93
472	INSTRUMENTATION & CONTROL - RESIDUALS	13	"	Jun-89	Sep-90	Oct-90	Mar-91	Apr-91	Sep-93

Table 4-1: Construction Schedule for Proposed Treatment Facilities (Continued)

GROUP 900	CONSTRUCTION SERVICES								
901	CONSTRUCTION SUPPORT BUILDING	22	LA	Jul-89	Dec-89	Jan-90	Oct-90	Nov-90	Jul-91
902	CONSTRUCTION WATER SYSTEM	22	"	Jul-89	Jan-90	Feb-90	Aug-90	Sep-90	Apr-91
903	CONSTRUCTION POWER DISTRIBUTION SYSTEM	22	"	Jul-89	Jan-90	Feb-90	Aug-90	Sep-90	Apr-91
904	CONSTRUCTION ROADS, PARK, FENCE, DRAINS	22	"	Jul-89	Jan-90	Apr-90	Sep-90	Oct-90	May-91
905	CENTRAL MIX CONCRETE PLANT	N/A	KE	Mar-89	Aug-89	Sep-90	Jan-90	Jun-90	Dec-90
906	CONSTRUCT TOILET & UTILITY BUILDINGS	22	LA	Jul-89	Dec-89	Dec-89	Apr-90	May-90	Sep-90
907	WATER TRANSPORTATION SYSTEM - ROLL-ON/ROLL-OFF	N/A	KE	Aug-88	COMPLETED	Jun-89	Oct-89	Jun-90	Dec-99
918	WATER TRANSPORTATION SYSTEM - PERSONNEL	N/A	KE	Aug-88	COMPLETED	Jun-89	Oct-89	Jun-90	Dec-99
908	BUSING SERVICE	N/A	KE	Nov-88	Jun-89	Sep-89	Dec-89	Apr-90	Dec-99
909	SECURITY	N/A	KE	Mar-89	COMPLETED	May-89	Jul-89	Aug-89	Aug-90
910	ROAD MAINTENANCE & SNOW REMOVAL	N/A	KE	Sep-89	Dec-89	Jan-90	May-90	May-90	Dec-99
911	TESTING LABORATORY SERVICES	N/A	KE	Jun-89	Sep-89	Oct-89	Dec-89	Jan-90	
913	RODENT & PEST CONTROL	N/A	KE	Dec-88	COMPLETED	Mar-89	A-1	Jul-89	May-91
914	SURVEYING	N/A	KE	Jun-89	Sep-89	Oct-89	Dec-89	Jan-90	
917	FUEL DEPOT	22	LA	Jul-89	Jan-90	Jan-90	Jun-90	Jun-90	Jun-91

N/A - Not Applicable
PBQD - Parsons Brinckerhoff Quade & Douglas, Inc.
DT - Dec-Tam Corp.
SCC - Sciaba Construction Corp.
B & V - Black & Veatch, Inc.
CL - Cable de Lyon
A-1 - A-1 Exterminators
MGI - Maguire Group Inc.
FRH - Frederick R. Harris, Inc.
M & E - Metcalf & Eddy, Inc.
SC - Sverdrup Corp.
BE - Boston Edison Co.
LA - Lin Associates, Inc.
JMC - J. M. Cashman, Inc.
BA - Bryant Associates, Inc.
M-P - Malcolm-Pirnie, Inc.
CDM - Camp, Dresser & McKee, Inc.
SW - Stone & Webster Engineering Corp.
KE - Kaiser Engineers, Inc.

**** THIS SCHEDULE, WHICH IS CURRENT THROUGH THE FIRST QUARTER OF 1989, IS FOR INFORMATION PURPOSES ONLY AND IS SUBJECT TO CHANGE.***

Additional Projects

Additional projects that are related to the Boston Harbor "cleanup" and that will be done concurrently with the new secondary treatment facility are as follows:

- Site Preparation
- Power & Electrical Facilities
- Short-term Residual Management
- Long-term Residual Management
- Water Transport Facilities
- Combined Sewer Overflows (only through the planning stage)
- Harbor Research and Monitoring

References

Massachusetts Water Resources Authority (MWRA). February 1988. Harbor Prospects, Volume II, Issue 1. pp 4-5.

Tate, N. May 31, 1989. "Plans for harbor wastewater plant moved up", published in The Boston Globe.

All other material for this section was extracted from the following documents:

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Executive Summary, Final Report.

Massachusetts Water Resources Authority (MWRA). March 1988. Secondary Treatment Facilities Plan, Volume III: Treatment Plant, Final Report.

North System Remote Headworks

Introduction

The MWRA's wastewater sewage is split into two service areas, the North and South systems. Flow from the North system passes through the Main pumping station and the Winthrop terminal before going to the Deer Island treatment plant. The flow from the South system will travel through the new inter-island conveyance system to the new Deer Island treatment plant.

The North system wastewater enters the Main Pumping Station (Fig. 4-3) from three remote headworks (Chelsea Creek, Ward Street, and Columbus Park) through two existing deep rock tunnels under the Boston harbor (North Metropolitan Relief Tunnel and Boston Main Drainage Tunnel). All three headworks were built in 1968 with screening and grit removal done at each location. The Chelsea Creek headworks can handle 1320 million liters per day (mlg) of flow, which is conveyed to the Main Pumping Station through the North Metropolitan Relief Tunnel. The Ward Street headworks can handle 970 mld and the Columbus Park headworks can handle 690 mld. Flow from both headworks is conveyed through the Boston Main Drainage Tunnel. All sewage in excess of the maximum flow at each headworks is diverted and discharged into the harbor through CSO's. The fate of CSOs is under consideration with the the construction of deep rock tunnels, to store the overflow before being conveyed to the new treatment plant, being the primary emphasis.

Even though each of the headworks has different capabilities, they are similar in physical layout and unit processing.

Recommended Plan

It is recommended that the remote headworks be upgraded with a fast-track plan. The fast-track plan includes replacing the existing screens, replacing most of the existing equipment with similar equipment, and adding a new ventilation and odor control system. The existing screens will be replaced with three mechanically cleaned climber-type screens with 2.2 cm spacings.

In order to acheive effective grit removal, the maximum velocity in the grit chamber must be 30 cm/sec or less. Currently, this is not possible at the remote headworks. To acheive this velocity, longer grit chambers are required; however, these were determined to

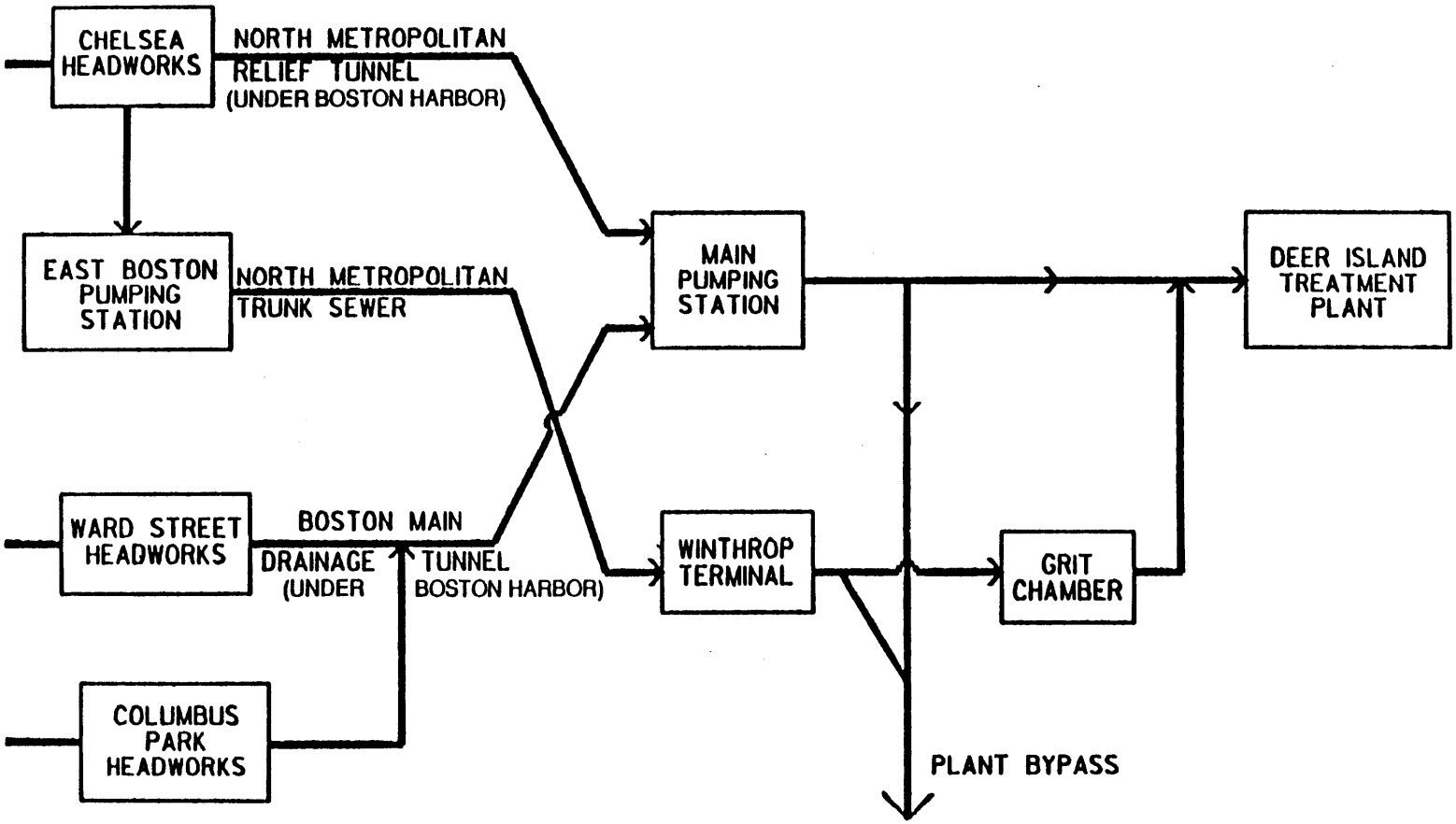


Figure 4-3: North System Headworks and Tunnel Schematic (MWRA V-III, 1988)

be impractical since additional grit removal will be done at the new Deer Island grit removal facility. It has also been determined that the current rate of removal is adaquate to protect the tunnels and the pumps at the Main Pumping Station from grit deposition. Currently, the three headworks produce a total of 11.5 to 45.9 m^3/day of screenings and grit.

References

All material for this section was extracted from the following documents:

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Executive Summary, Final Report.

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Volume III: Treatment Plant, Final Report.

North System Main Pumping Station

Introduction

The existing North System Main Pumping Station receives wastewater from two deep-rock tunnels, located 91 m below sea level. The deep rock tunnels (Boston Main Drainage tunnel and North Metropolitan Relief tunnel) transport North System wastewater and have a maximum hydraulic capacity of 1660 million liters per day (mld) and 1320 mld respectively.

Recommended Plan

The recommended plan for the Main Pumping Station (Fig. 4.4) is to modify the existing station to meet the needs of the new Deer Island treatment plant. The major changes to the existing pumping station will be: replacing or modifying all ten pump units including the electric motors, and variable speed drives, and adding new knife gate valves on the suction side of five pumps and on the discharge side of all ten pumps (Fig. 4-5). The estimated cost (in 1986 dollars) for modifying the pumping station is $43.4 million with an annual operating and maintenance cost of $3.3 million.

Of the ten pumps, nine will be needed during periods of peak flow leaving one as standby. The existing pumping station has room for only ten pumps; therefore, it has been recommended that two additional pump units be stored onsite in case of a breakdown. The pumps will be rated at 3,000 hp and must be able to handle 340 mld at a total dynamic head of 41.1 m.

In addition to the pumping station, two force mains will be constructed to convey wastewater from the pumping station to the new North System Grit facility. The two force mains will be 3.5 m in diameter and slope constantly downhill to the grit facility (Fig. 4-6).

Construction Schedule

The Main Pumping Station and the two force mains are expected to be completed by 1993. It is anticipated that construction should last 2.5 yrs. Five pump units will be replaced as part of the fast track improvements.

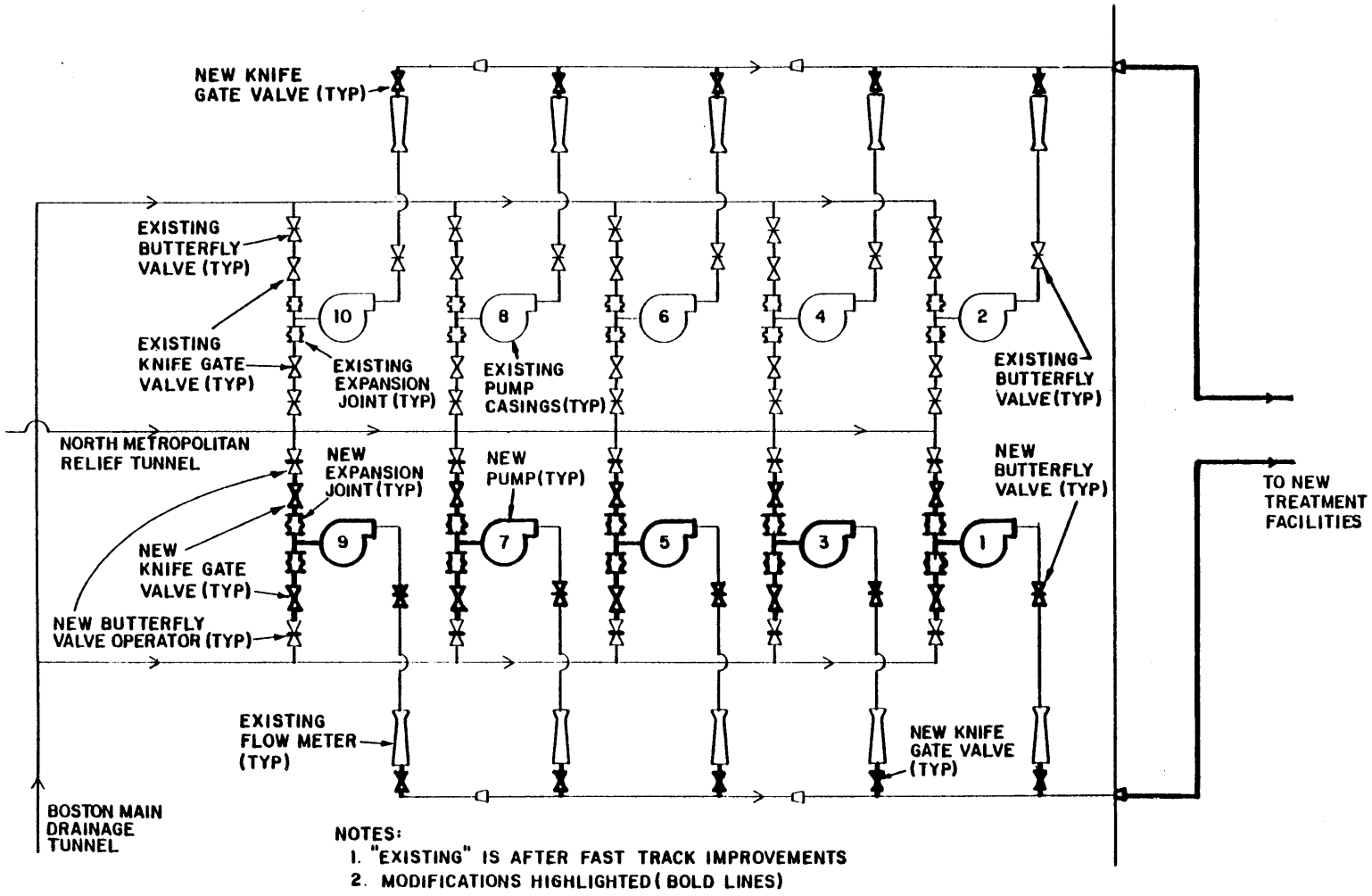


Figure 4-4: Flow Diagram for the Modified Main Pumping Station (MWRA V-III, 1988)

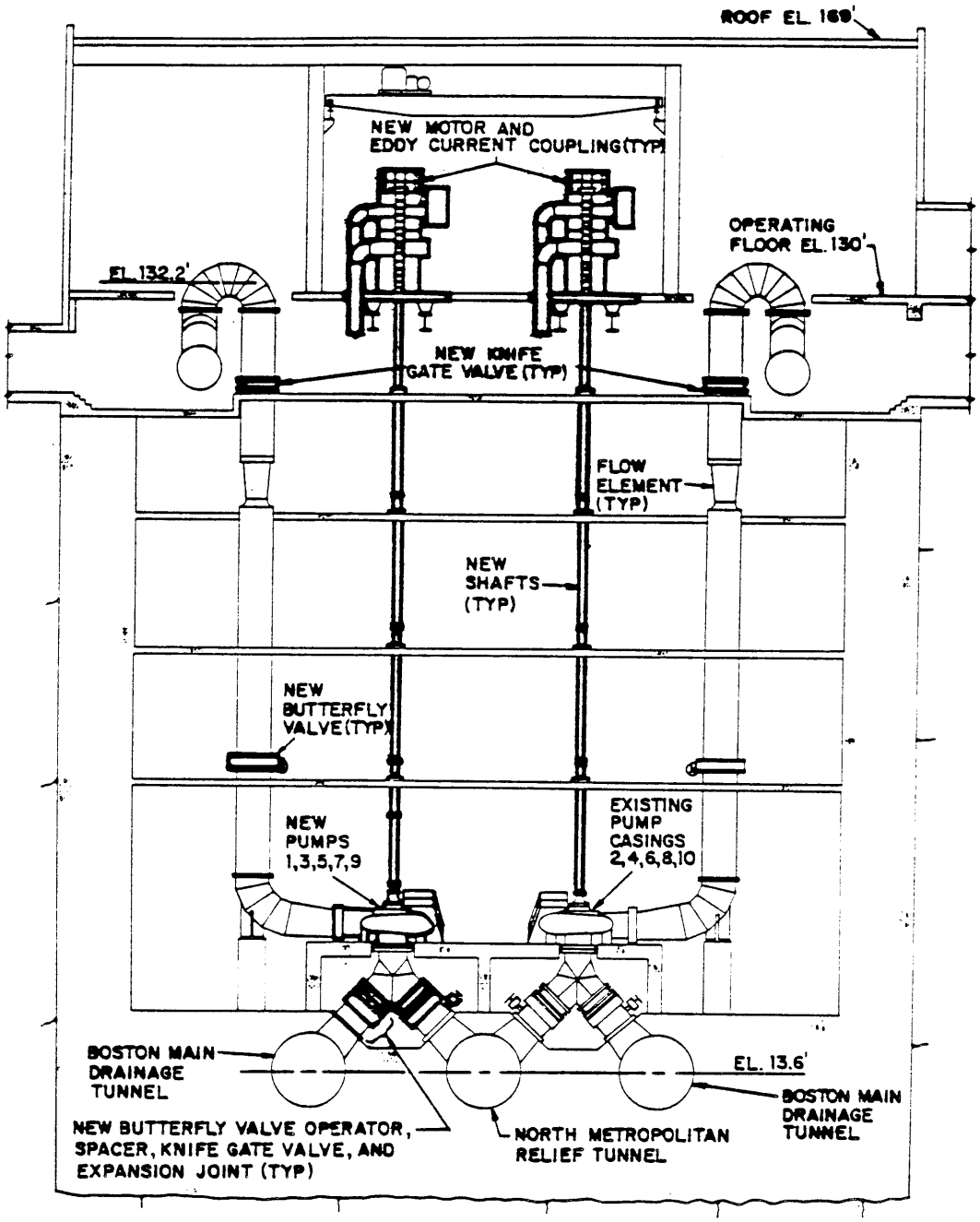


Figure 4-5: Modifications to the Pumps at the Modified Main Pumping Station (MWRA V-III, 1988)

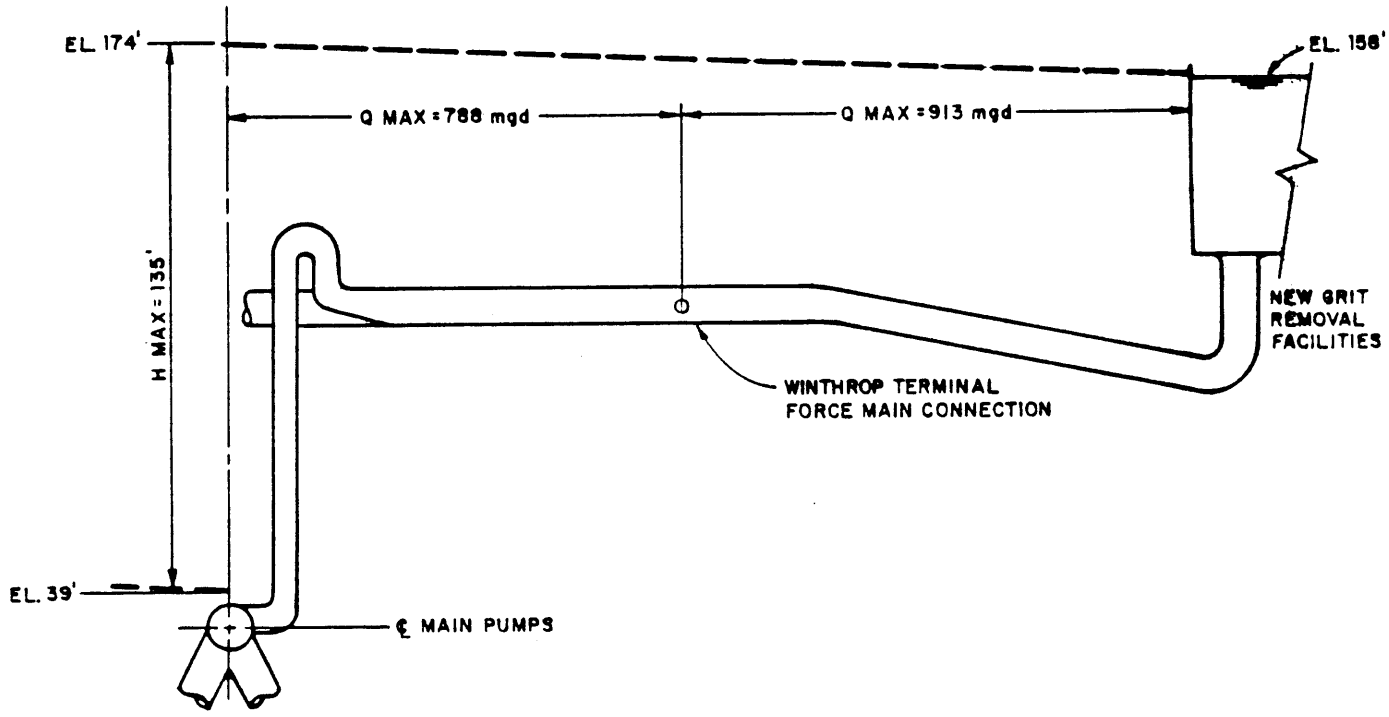


Figure 4-6: Hydraulic Profile from the Main Pumping Station to the New Treatment Facility (MWRA V-III, 1988)

References

All material for this section was extracted from the following documents:

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Executive Summary, Final Report.

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Volume III: Treatment Plant, Final Report.

Winthrop Terminal

Introduction

The MWRA's wastewater is split into two service areas, the North and South systems. Flow from the North system passes through the Main Pumping Station and the Winthrop Terminal before going to the Deer Island treatment plant. The flow from the South system will travel through the new inter-island conveyance system to the new Deer Island treatment plant.

The North system wastewater entering the Winthrop Terminal is screened and then passed through a grit chamber. The grit chamber can treate a maximum of 230 million liters per day (mld). Therefore, any wastewater in excess of 230 mld bypasses the plant and is discharged into the harbor. Wastewater feeds into the Winthrop Terminal through the North Metropolitan Trunk Sewer (Fig. 4-3) which has a design flow rate of 80 mld to 470 mld and averages 91 mld.

Recommended Plan

The recommended plan is to upgrade the Winthrop terminal. This upgrade includes a fast-track and a long term plan. The fast-track plan includes replacing the existing screens, upgrading the pumps, and eliminating the bypass of wastewater into the harbor. The existing screens will be replaced with three mechanically cleaned climber-type screens with 2.2 cm spacings. The six pumps at the terminal will be upgraded to maintain a flow of 470 mld. This will be accomplished with four 87 mld pumps with a total dynamic head of 11.6 m, and two 170 mld pumps with a total dynamic head of 10.1 m. All pumps will have variable speed motors to accommodate the variable flow. The Winthrop terminal facility will be able to treate the maximum design flow through the North Metropolitan Trunk Sewer, which is 470 mld. Flows in excess of the maximum design flow will be diverted through CSOs before entering the North Metropolitan Trunk Sewer.

The long term plan will retain the new screens, replace all pumps, and abandon the existing grit facility. Flow from the Winthrop terminal will pass through the new Deer Island grit removal facility eliminating the need for the existing grit removal facility at Winthrop terminal. The existing pumps will be replaced in order to increase the discharge head since the elevation of the new grit removal facility will increase by 9.1 m (Fig. 4-7).

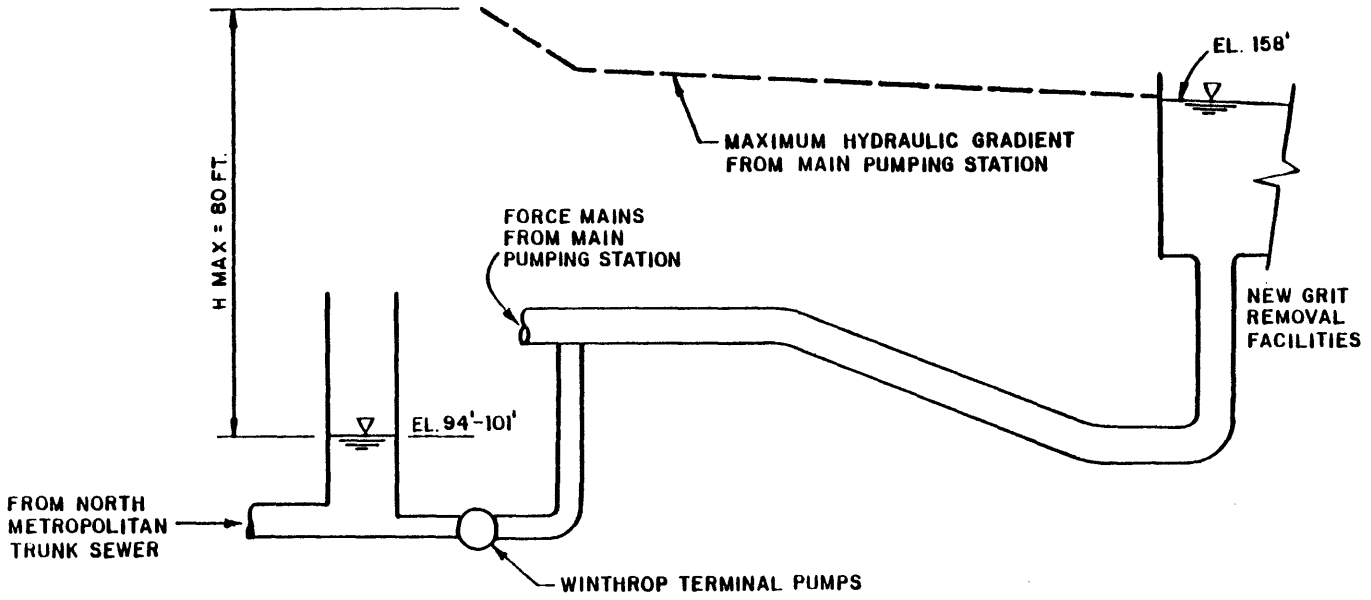


Figure 4-7: Hydraulic Profile from Winthrop Terminal to the New Treatment Facility (MWRA V-III, 1988)

The six new pumps can handle a flow of 120 mld with a total dynamic head of 24.4 m. The pumps will be equipped with 600 hp electric motors with variable speed motors and eddy current coupling drives. In addition to the new pumps, most of the equipment at the terminal will be replaced with new equipment (Fig. 4-8).

The Winthrop terminal (Fig. 4-9) is approximately 13.7 m wide by 29.3 m long. The present worth cost of the upgrade is approximately $13.1 million (1986 dollars) and includes a 25% allowance for engineering and contingency costs. The estimated operations and maintainence costs are $0.5 million per year. The modifications to Winthrop terminal are expected to be completed by mid 1993.

References

All material for this section was extracted from the following documents:

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Executive Summary, Final Report.

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Volume III: Treatment Plant, Final Report.

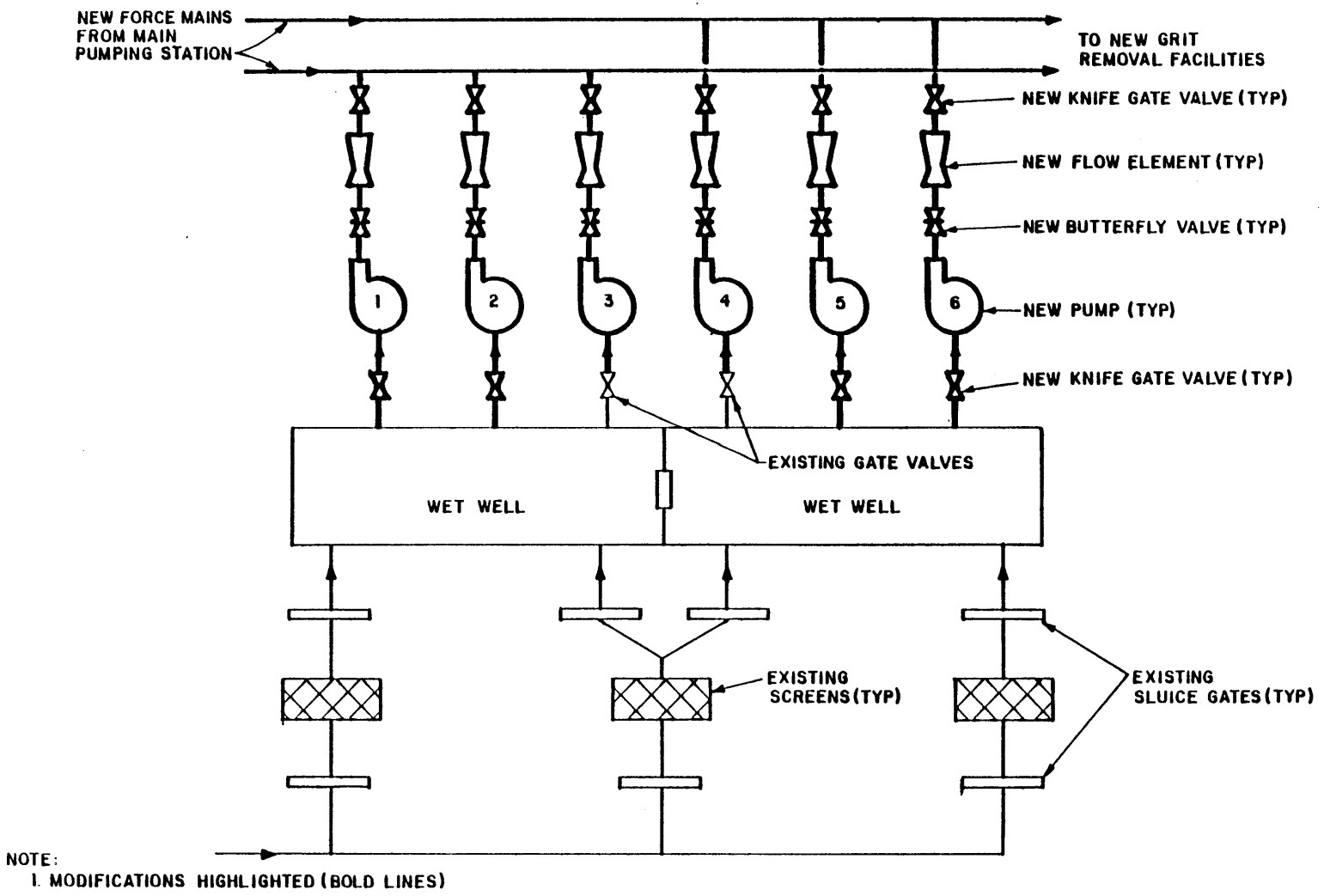


Figure 4-8: Modified Winthrop Terminal Facilities Flow Diagram (MWRA V-III, 1988)

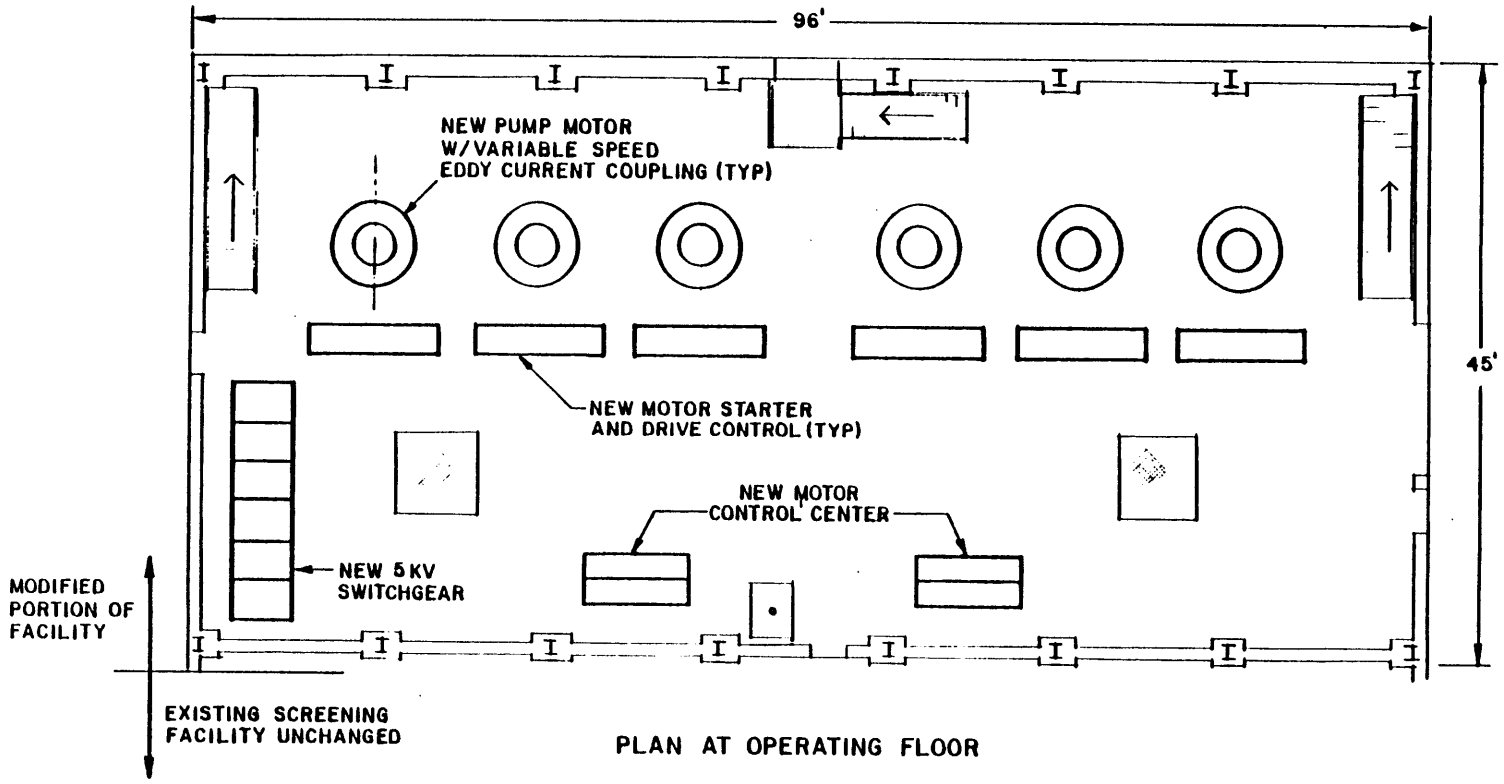


Figure 4-9: Modified Winthrop Terminal Facilities (MWRA V-III, 1988)

Deer Island Grit Removal Facility

Introduction

The flow will enter the new Deer Island grit removal facility from two sources: the Winthrop Terminal Pumping Station and two force mains from the North System Main Pumping Station. Currently, flow from the two force mains, which are 3.5 m in diameter, receive grit removal adequate enough to prevent deposition in the deep rock tunnels conveying wastewater from the North system and to protect the raw wastewater pumps at the North System Main Pumping Station. Grit consists of sand size particles which can cause damage to mechanical equipment and accumulate in low flow areas. Grit is not removed from the flow through Winthrop Terminal Pumping Station. It has been determined that additional grit removal facilities are needed to protect new mechanical equipment and minimize deposition within the treatment plant.

Recommended Plan

The new grit removal facility consists of centrifugal grit chambers, grit concentrating and washing equipment, grit truck loading bays, and air emission control systems. There will be two batteries of eight centrifugal grit chambers. These chambers will be 7.3 m in diameter and have a maximum capacity of 265 mld. The peak flow into the facility is 3460 mld. It will take seven chambers of each battery to handle the peak flow.

Grit from the chambers is pumped to the grit concentrators and washers by 32 pumps (two for each chamber, one as a standby). These constant-speed 5 hp motors have a capacity of 570 to 760 lpm and a 9.1 m total dynamic head (tdh). The grit separation is done with 10 grit cyclone separators (only six are needed). For each cyclone separator, there is one inclined classifier that washes the grit.

The separated and washed grit is conveyed to one of the two loading bays where it is trucked by roll-on/roll-off barges to residual handling facilities. It is estimated that a volume of 23 to 73 m^3 per day of grit will have to be disposed of.

The grit concentration and washing equipment and grit loading bays will be housed in one grit handling building occupying an area of 280 m^2. Construction costs (1986

dollars) for the new grit removal facilities are estimated at $6.9 million with an annual operating and maintenance cost of $630,000.

References

All material for this section was extracted from the following documents:

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Executive Summary, Final Report.

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Volume III: Treatment Plant, Final Report.

Nut Island Headworks

Introduction

Wastewater from MWRA's South System is currently treated at the Nut Island Wastewater Treatment Plant. This treatment plant provides grit removal, preaeration, primary sedimentation, and disinfection. When the new Deer Island Treatment Plant is operational, the existing facility on Nut Island will be decommissioned and replaced with a new headworks. The headworks will screen and remove grit from the wastewater before it is transported to the new treatment plant on Deer Island through the proposed inter-island conveyance system. Figure 4-10 shows a schematic of the flow.

Recommended Plan

The new Nut Island headworks (Fig. 4-11) will provide preliminary screening of the flow from the High Level Sewer, which is the collection sewer for all the South System wastewater flow. Flow will range from 420 mld to 1360 mld. This wastewater flows through a 60 m long hydraulic transition section that distributes the flow evenly to the four screens. The screens will be enclosed in a screening building, which will cover an area of 1,800 m^2. Following screening, wastewater is collected in a common channel and transported to the headworks building for grit removal. The headworks building, which will be equipped with air emission control equipment, will cover an area of 440 m^2. The estimated cost for the total project is approximately $4.1 million.

Screening, which is the first stage of preliminary treatment, removes rags, trash, and other large size solids. There will be four wastewater screens - two screens can handle the maximum flow of 1360 mld leaving two for standby. These screens will have 1.9 cm bar spacings and will be equipped with climber cleaning mechanisms. Each screen will be placed at an 80 degree angle in one of the four channels which are 3.5 m wide by 3.5 m high by 12 m long. Flow will be diverted to any channel in the hydraulic transition section.

The climber raking cleaning mechanism will be automatically activated after a specified amount of time or by pressure sensors on the screens. The screened material is discharged to a cross-conveyor, then transported to one of three residual transport conveyors, where it is brought to truck loading bays and properly disposed of. The average annual volume of screening will be about 8.4 m^3/day with as much as 25 m^3/day during peak flows.

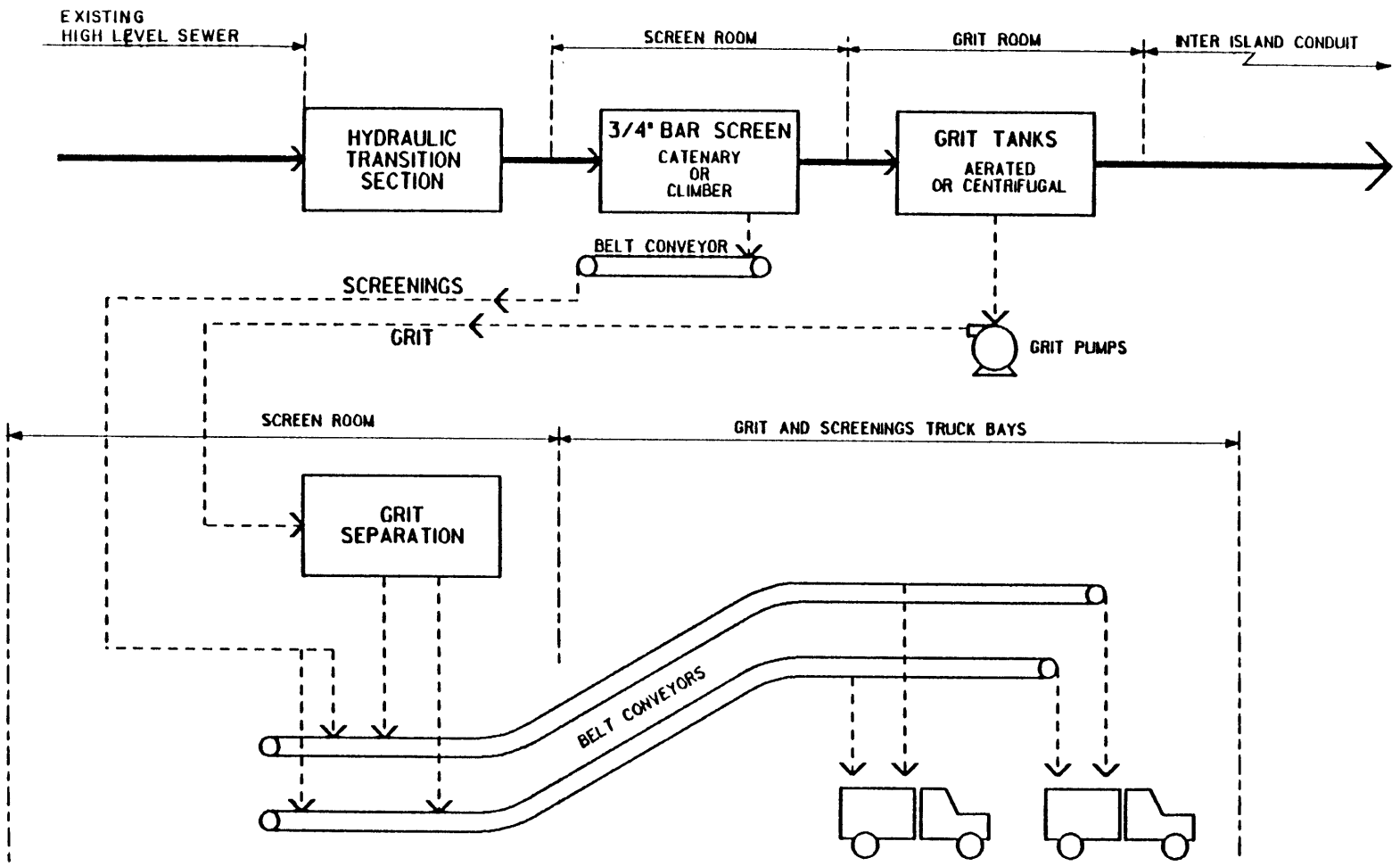


Figure 4-10: Flow Schematic of the Nut Island Headworks Facility (MWRA V-III, 1988)

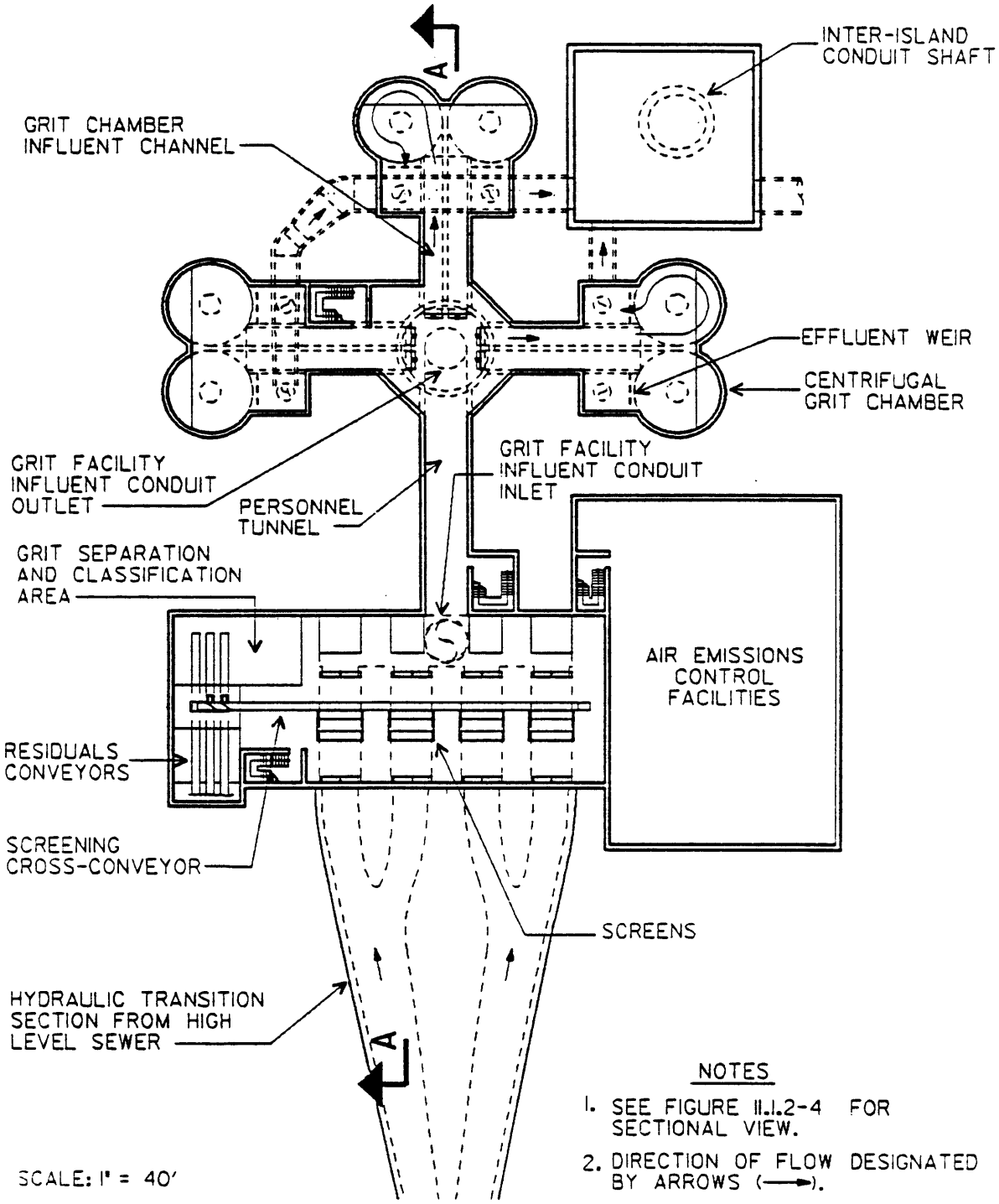


Figure 4-11: Proposed Nut Island Headworks Facility (MWRA V-III, 1988)

Following screening, wastewater is collected in a common channel and transported to centrifugal grit removal chambers. There will be six grit chambers with a diameter of 7.3 m and a maximum capacity of 270 mld. During peak flows of 1360 mld only five chambers are needed. Removal of grit is accomplished by centrifugal action and gravity. Particles that are coffee ground size and larger are removed with this process.

Grit is pumped from the grit collection hopper to one of the six grit separators and cyclone-classifiers. There are a total of 12 vortex pumps, two for each grit chamber, which can handle 570 lpm at a total dynamic head of 9 m and run at 5 hp. Grit from the chambers is concentrated and dewatered in the grit separators and then washed in the cyclone-classifiers. The dewatered, washed grit is then transported to the truck loading bays by conveyors and properly disposed of. The average annual volume of grit removed will be about 8.4 m^3/day with as much as 25 m^3/day during peak flows.

The hydraulic profile for Nut Island (Fig. 4-12) is influenced by the high level sewer and the inter-island conveyance system. Expected flow levels will range from 34.8 to 34.1 m at the high level sewer to 32.6 to 31.1 m at the inter-island conduit shaft. The level at the conduit shaft is controlled by the South system pumping station.

Construction Schedule

The construction sequence for the new Nut Island Headworks will be as follows:

- Remove the sludge and digester gas from the four anaerobic digesters, demolish the digester, and remove the associated piping. This is the location of the new headworks.

- Construct the new headworks screening building, grit chambers, and the hydraulic transition section.

- Connect the headworks to the inter-island conduit and begin operation of the headworks facility.

- Divert flow from the High Level Sewer to the hydraulic transition section.

- Decommission and demolish the existing wastewater treatment plant.

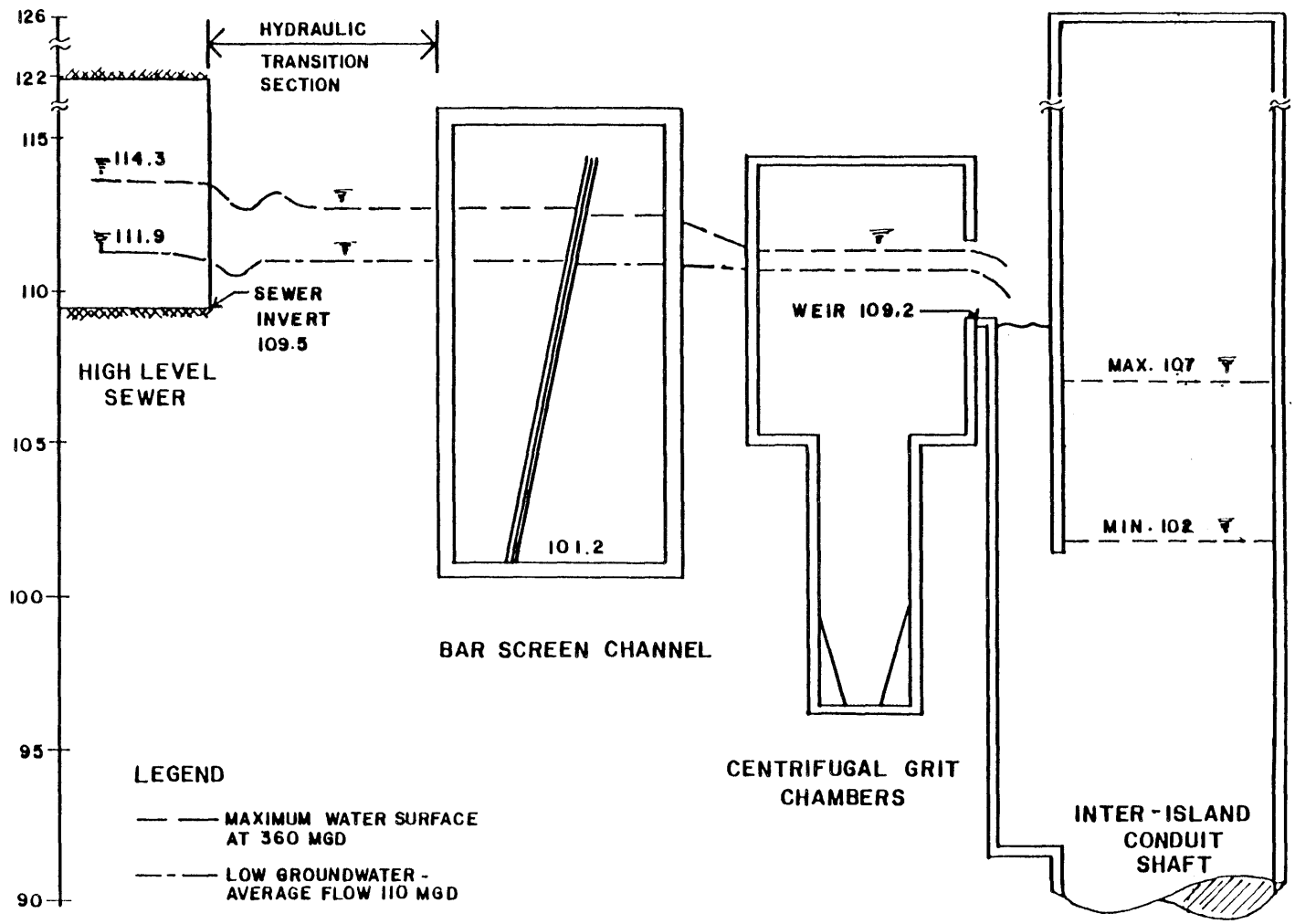


Figure 4-12: Hydraulic Profile at Proposed Nut Island Headworks Facility (MWRA V-III, 1988)

The new Nut Island Headworks is expected to be completed by December 1994. It is anticipated that construction should last 2 yrs. The present Nut Island Wastewater Treatment Plant must be decommissioned within 6 months after startup of the headworks.

Potential environmental impacts related to construction included the generation of fugitive dust, increased potential for erosion of cleared surfaces, construction generated noise, and the movement of construction personnel, equipment, and material on-site. These impacts are determined to be minimal.

References

All material for this section was extracted from the following documents:

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Executive Summary, Final Report.

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Volume III: Treatment Plant, Final Report.

Inter-Island Conveyance System

Introduction

The MWRA's wastewater sewage is split into two service areas, the North and South systems. The new inter-island conveyance system will transport flow from the South system to the new Deer Island wastewater treatment plant. The flow from the North system goes directly to Deer Island.

Currently, the South system wastewater receives primary treatment at the Nut Island Plant and the effluent is discharged into the harbor through two outfalls. It is proposed that the Nut Island treatment plant be decommissioned and replaced by a headworks that will screen and remove grit from the wastewater before it reaches the new Deer Island treatment plant. The flow from Nut Island to Deer Island will travel through a new inter-island conveyance system.

The inter-island conveyance system will transport 1360 mld of wastewater from the new Nut Island headworks to the new treatment plant at Deer Island, which is a distance of approximately 8 km. A South System Pumping Station will be built on Deer Island to pump the wastewater from the Nut Island headworks through the conveyance system to the primary splitter boxes and primary clarifiers located on Deer Island. The recommended plan is based on technical adequacy, cost considerations, environmental impacts, and institutional considerations.

The MWRA considered three alternatives for the inter-island conveyance system and chose a deep rock tunnel over a marine pipeline or sunken tube.

The marine pipeline would have been placed across the harbor in a trench covered with backfill and gravel for protection. The pipe would have been 3.4 m in diameter, made of concrete, and placed in 4.9 m sections of pipe. The sunken tube would have been placed in much the same manner as the marine pipeline in 60 m sections. The tube would have been constructed of a thin steel tube fitted with interior and exterior concrete.

The marine pipeline and sunken tube options were not chosen because:

- The harbor is generally quite shallow and problems might arise from dragging anchors and scouring.

- There is a large amount of traffic from ships and it would be difficult to reroute traffic during construction.

- Deepening the shipping channels would not have been possible.

Recommended Plan

The recommended plan for the inter-island conveyance system is a deep rock tunnel with vertical access tunnels at both Deer and Nut Islands. A conceptual view of the tunnel location is shown in Figure 4-13. The actual location of the tunnel will be determined by further geotechnical investigations. The estimated cost of the project will be approximately $83 million.

Deep Rock Tunnel

The deep rock tunnel (Fig. 4-14) will begin with a vertical access shaft at the headworks on Nut Island. This shaft will be approximately 5 m in diameter and 60 to 90 m below sea level. Starting at the access shaft, a 3.4 m diameter tunnel 7600 m long will be drilled. The tunnel will have a concrete lining of a minimum thickness of 30 cm.

The tunnel will connect to another vertical access shaft on Deer Island where a pump station will be constructed to lift wastewater to the treatment plant. The pump station will be equipped with six 340 mld pumps. These pumps will be required to lift the wastewater a minimum of 25 m.

The predominate rock that the tunnel will be drilled through is Cambridge Argillite, a medium-hard rock. The tunnel is anticipated to be drilled using a tunnel boring machine, although, some portions of the tunnel may require conventional drill and blast techniques if extremely hard rock is encountered. The excavation rate for the tunnel boring machine is estimated to be 20 m/day.

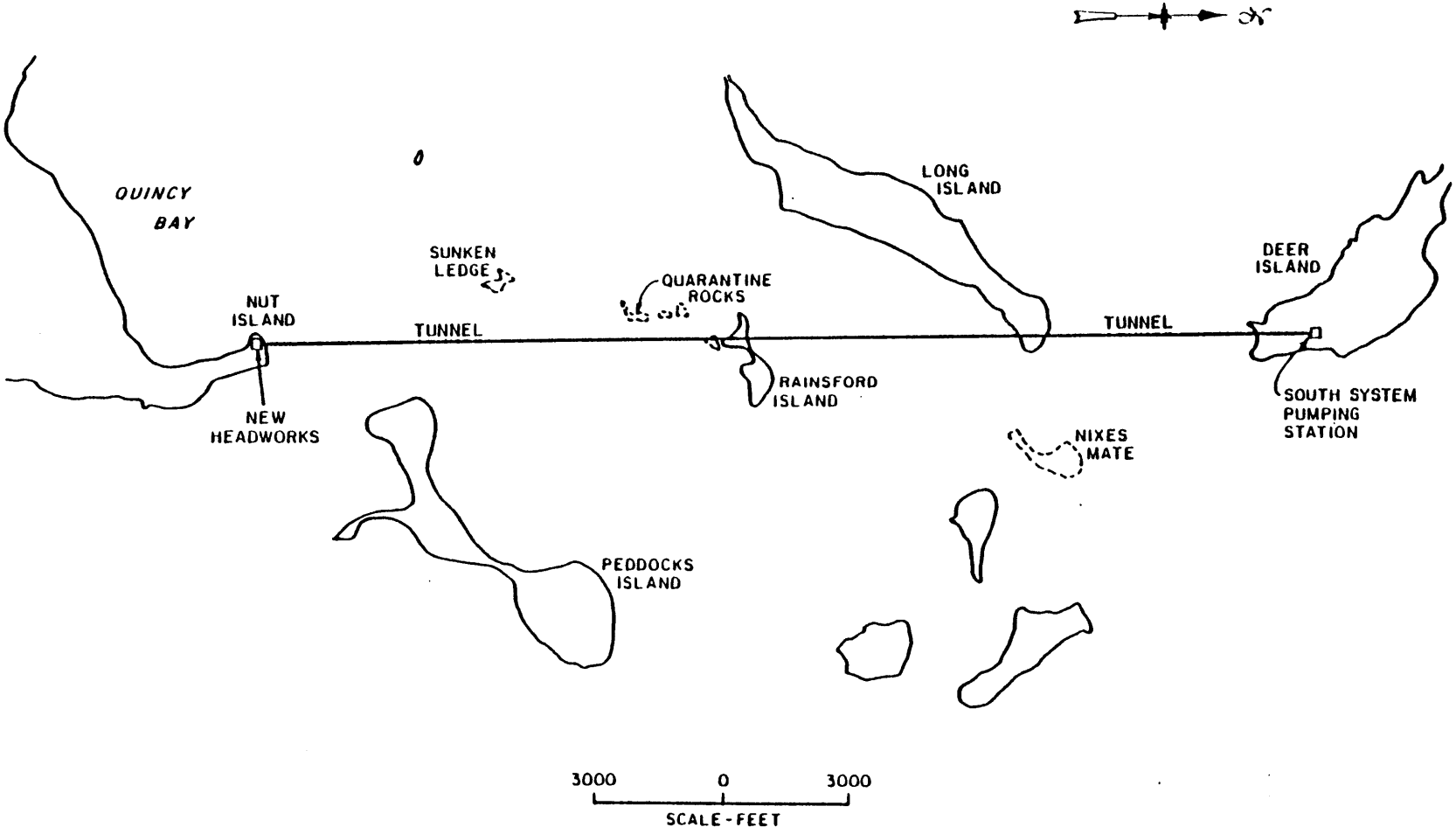


Figure 4-13: Conceptual View of the Inter-Island Conveyance System (MWRA V-IV, 1988)

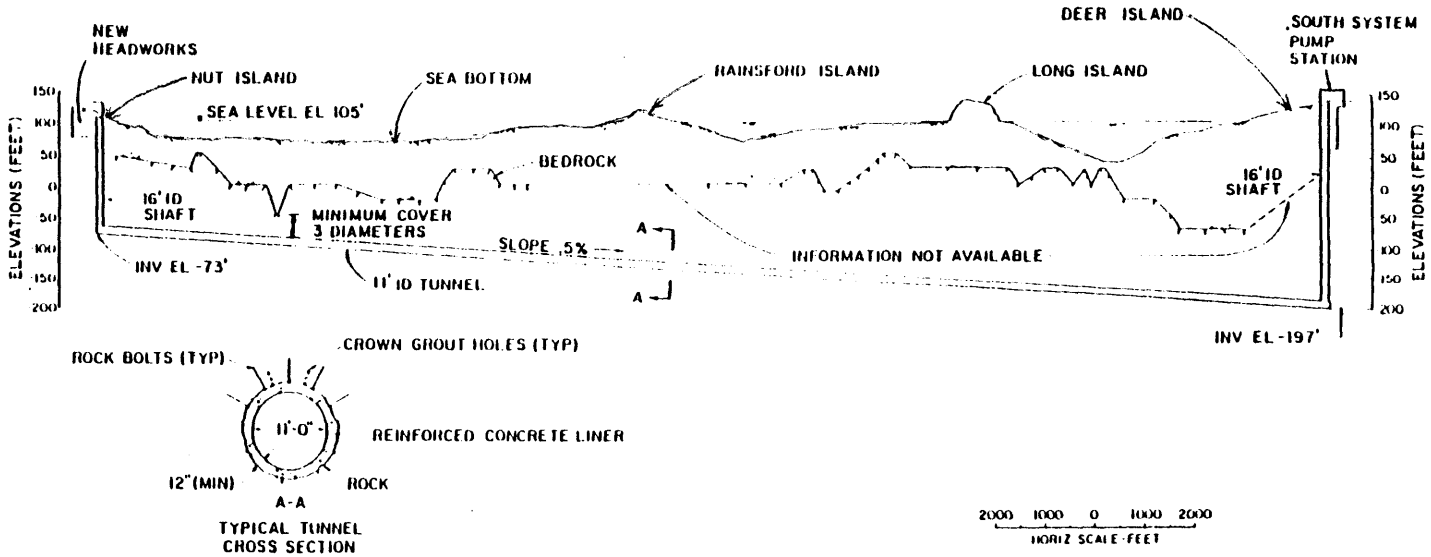


Figure 4-14: Sectional View of the Deep -Rock Tunnel (MWRA V-IV, 1988)

The tunnel will be lined with either precast concrete sections or cast-in-place concrete. As the tunnel is excavated, the lining will be placed. A concrete liner will be used to minimize friction head losses, which decreases the size requirements of the tunnel.

The primary design criteria for the tunnel are; velocity, static lift (head), and frictional head loss. If the velocity in the tunnel is too high, there is an increased chance of erosion of the liner, separation of flow (resulting in cavitation damage to the lining), excess hydraulic head loss (resulting in increased pumping costs), hydraulic surging (water hammer) during startup and shutdown of pumps, and flow control stability problems. Based on the above criteria, the maximum velocity was set at 1.8 m/sec.

If the velocity in the tunnel is too low, a larger more expensive tunnel would be necessary to decrease the chances of excessive deposition of organic and inorganic material. Based on this and a maximum tunnel diameter of 3.4 m, the minimum velocity was set at 0.3 m/sec.

To determine the size of the pumping equipment and the depth at which to place the pumps below grade, the maximum expected head loss must be determined. The tunnel for the conveyance system will always run full; therefore, the pumping station must overcome friction head loss and static lift (head). Static lift is the vertical difference between the free level at the Nut Island headworks and the free surface level at the South system pumping station (Fig. 4-15).

Friction head loss is the head required to overcome the resistance to flow. The Darcy-Weisbach equation was used to determine the friction head loss. This equation can be written as follows:

$$H_f = f(L/D)(V^2/2g)$$

where:
H_f = friction head loss, m
f = friction factor
L = length of tunnel, m
D = diameter of tunnel, m
V = velocity of flow, m/sec
g = acceleration due to gravity, m/sec^2

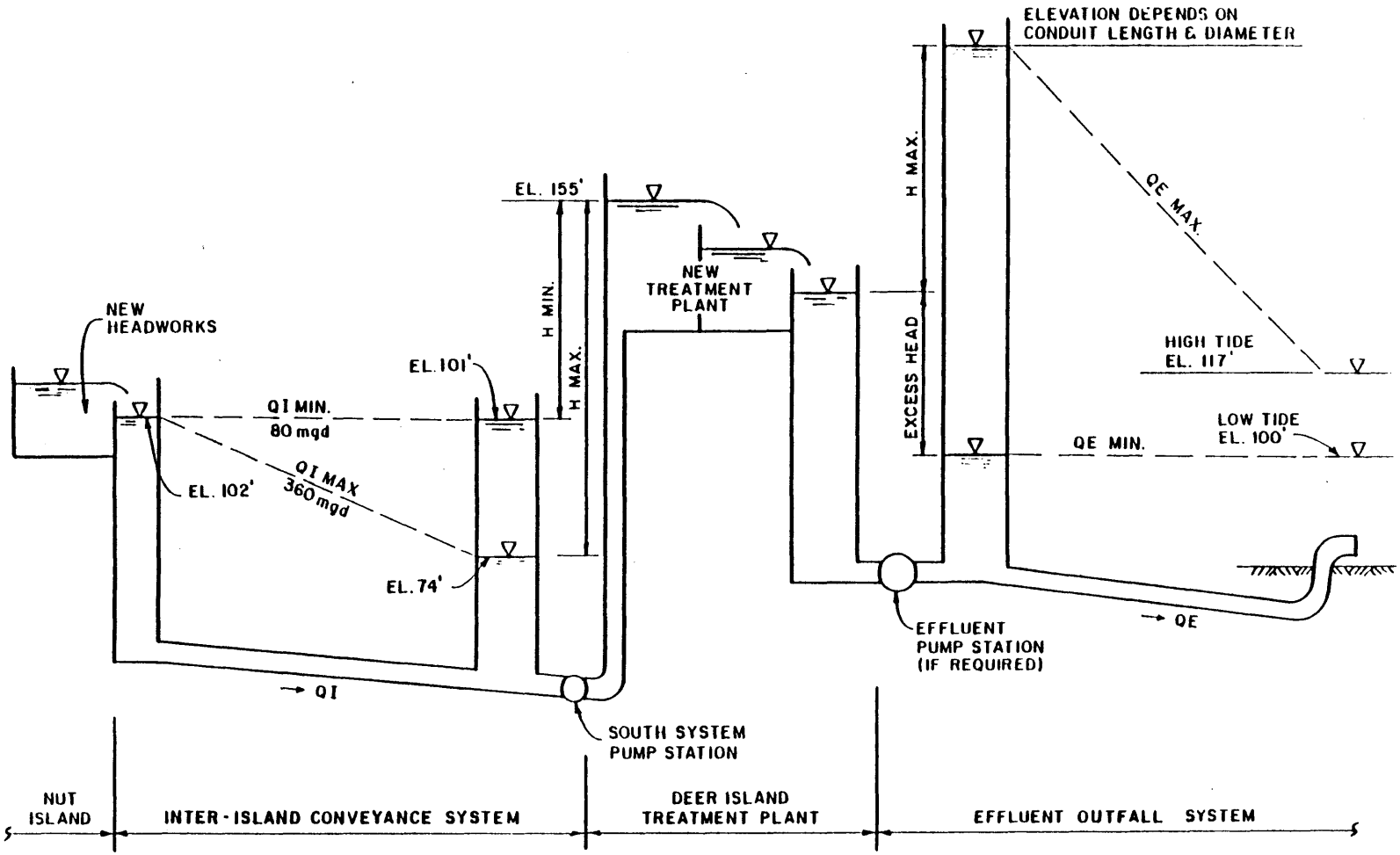


Figure 4-15: Hydraulic Profile from Proposed Nut Island Headworks to the New Effluent Outfall Tunnel (MWRA V-IV, 1988)

The unknown variable in this equation is the friction factor, which may be determined using values found in literature and experiments run on other tunnels. In the literature, the friction factor for concrete pipes with the diameter and flow rate given ranged from 0.012 to 0.020. Experiments were preformed on the two existing deep rock tunnels (North Metropolitan Relief Tunnel and the Boston Main Drainage Tunnel) and values ranged from 0.018 to 0.022. A friction factor of 0.020 was chosen.

Construction Schedule

The construction sequence for the inter-island conveyance system will be as follows:

- Construct the vertical access shaft at Nut and Deer Islands.
- Excavate the deep rock tunnel while removing the spoils through the access tunnels.
- Line the tunnel with concrete as the tunnel is being constructed.

The inter-island conveyance system is expected to be completed by 1994. It is anticipated that construction should last 3.5 yrs. A comprehensive schedule for the tunnel construction is given in Figure 4-16.

Potential environmental impacts related to construction include habitat removal, slight increase in noise level, increase in traffic, and disposal of 153,000 m^3 of excavated material. These impacts are determined to be minimal.

References

All material for this section was extracted from the following documents:

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Executive Summary, Final Report.

Massachusetts Water Resources Authority (MWRA). March 1988. Secondary Treatment Facilities Plan and EIR, Volume IV: Inter-Island Conveyance System, Final Report.

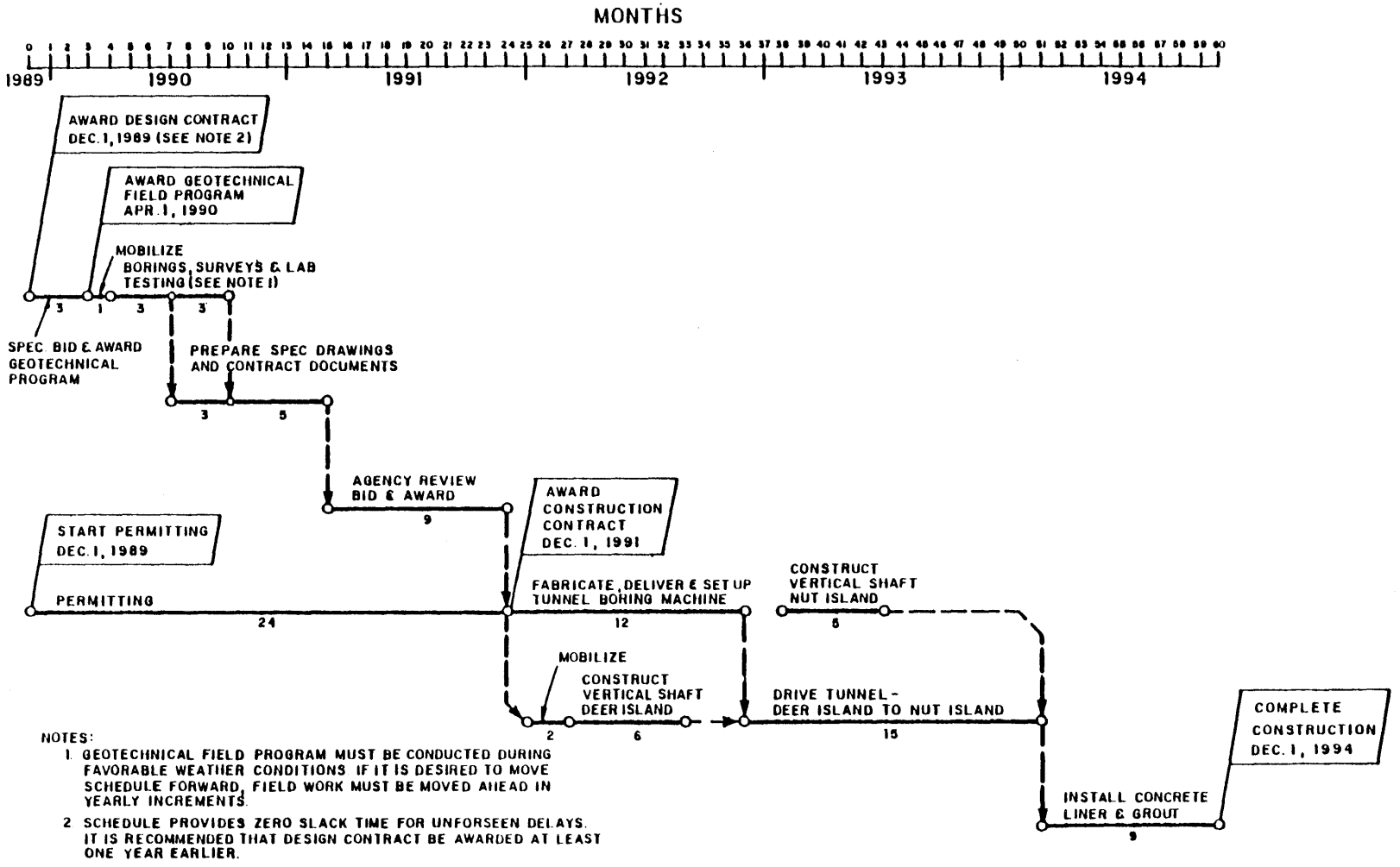


Figure 4-16: Comprehensive Construction Schedule for the Inter-Island Conveyance System (MWRA V-IV, 1988)

South System Pumping Station

Introduction

The MWRA's wastewater sewage is split into two service areas, the North and South systems. The new inter-island conveyance system will transport flow from the South system to the new Deer Island wastewater treatment plant. The flow from the North system goes directly to Deer Island.

Currently, the South system wastewater is treated at the Nut Island Treatment Plant and the effluent is discharged into the harbor through two outfalls. It is proposed that the Nut Island treatment plant be decommissioned, due to its inability to provide adequate treatment, and replaced by a headworks that will screen and remove grit from the wastewater before it reaches the new Deer Island Treatment Plant. The flow from Nut Island to Deer Island will travel through a new inter-island conveyance system. As shown in the flow diagram (Fig. 4-17), the South system pumping station will receive flow from the conveyance system and deliver it to the primary splitter box on Deer Island.

The inter-island conveyance system will transport 1360 mld of wastewater from the new Nut Island headworks to the new treatment plant at Deer Island, which is a distance of approximately 8 km. A South System Pumping Station will be built on Deer Island to pump the wastewater from the Nut Island headworks through the conveyance system to the primary splitter boxes and primary clarifiers located on Deer Island. The recommended plan is based on technical adequacy, cost considerations, environmental impacts, and institutional considerations.

Due to the large variability of flow to the South System, 300 to 1360 mld, variable speed drives are needed, two drive alternatives were considered: variable frequency and eddy current coupling. Both choices were comparable in all aspects except price; therefore, the lower priced alternative, eddy current coupling, was chosen.

Recommended Plan

The south system pumping station (Fig. 4-18) will be approximately 24 m wide by 40 m long by 15 m high, and is expected to generate 23,000 m^3 of spoils. The construction is estimated to take 30 months at a cost of $38 million.

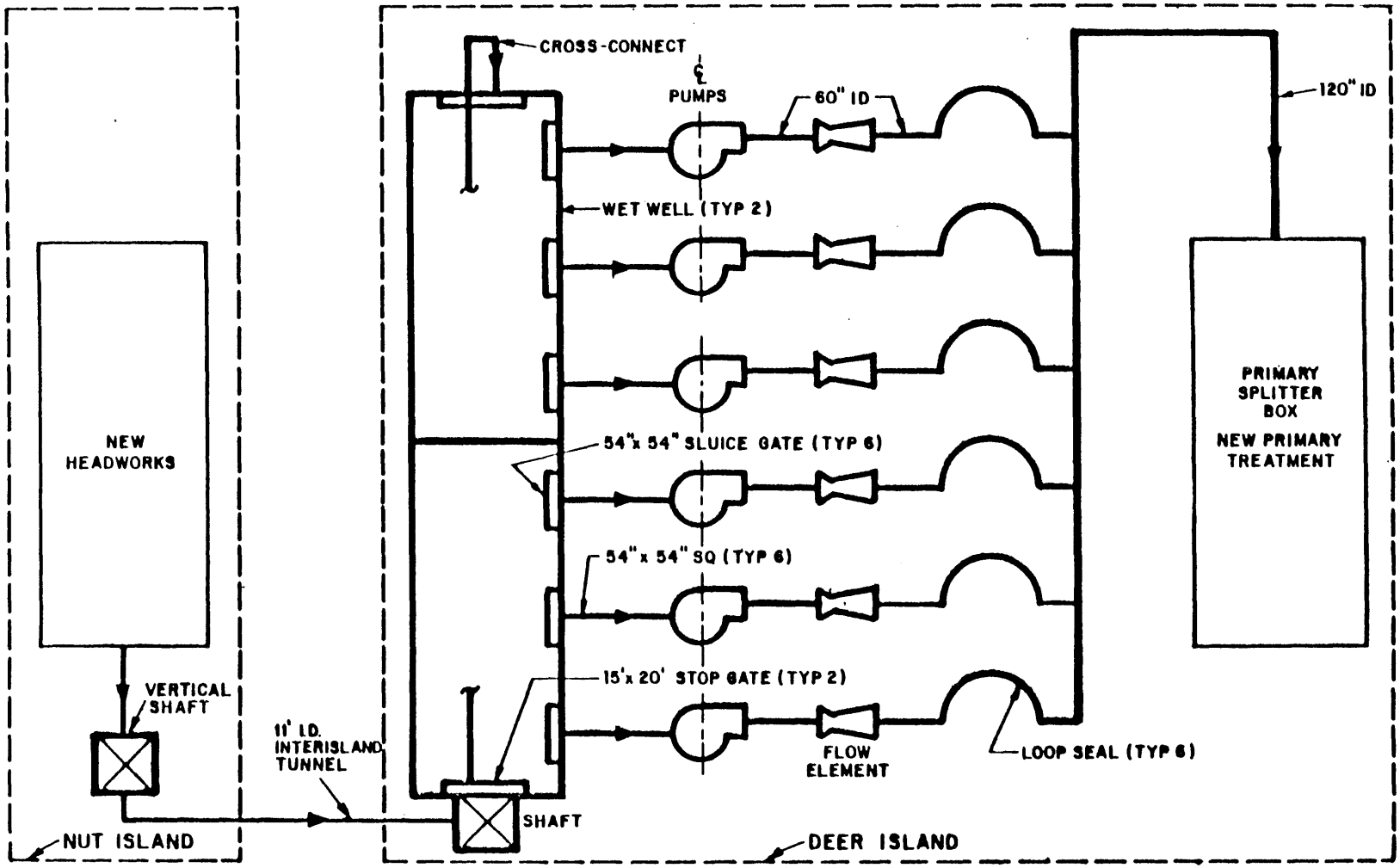


Figure 4-17: Schematic of the Flow Through the South System Pumping Station (MWRA V-III, 1988)

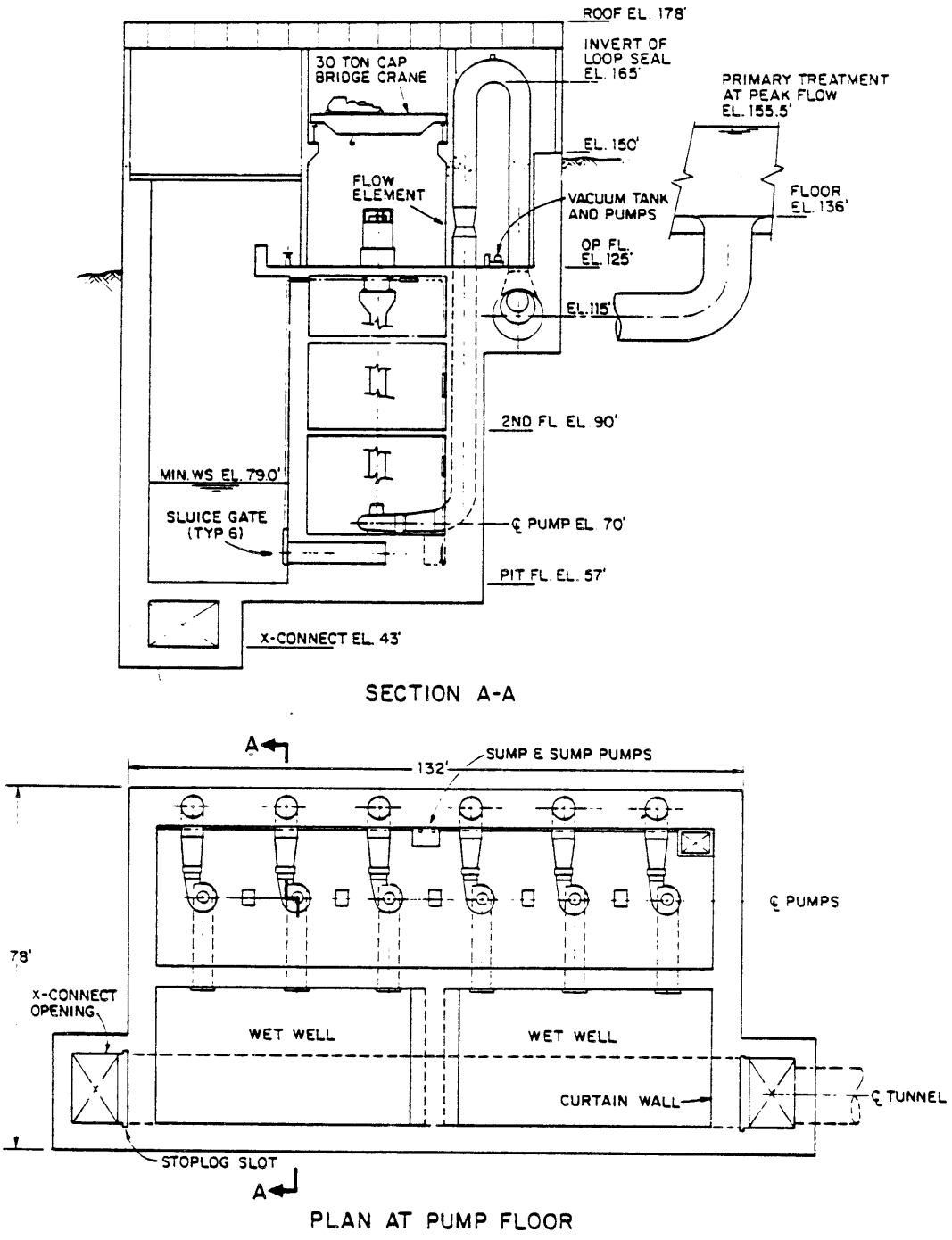


Figure 4-18: South System Pumping Station (MWRA V-III, 1988)

The flow from the south system will range from 300 to 1360 mld; therefore, only variable speed pumps were considered. Six 340 mld pumps are needed to ensure that a flow of 1360 mld can be maintained even during maintenance periods and breakdowns. The pumps must be able to lift the wastewater a maximum distance of 26 m. All pumps will be driven by an electric motor through a variable speed eddy-current coupling drive. Each motor will be rated at 2,000 hp.

Construction Schedule

The South system pumping station is expected to be completed by 1994. It is anticipated that construction should last 2.5 yrs. A comprehensive schedule for the pumping station construction is given in Figure 4-19.

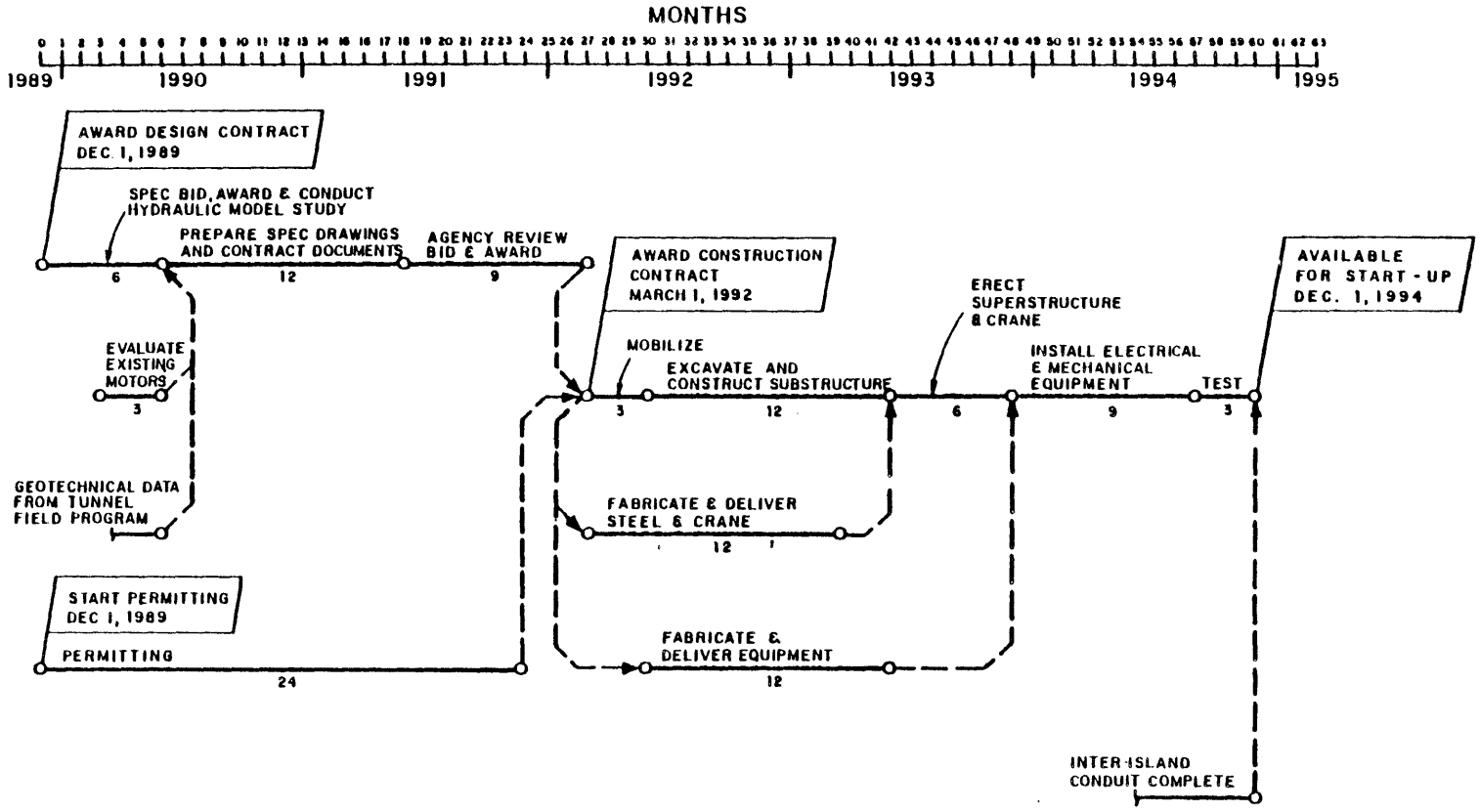
References

All material for this section was extracted from the following documents:

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Executive Summary, Final Report.

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Volume III: Treatment Plant, Final Report.

Massachusetts Water Resources Authority (MWRA). March 1988. Secondary Treatment Facilities Plan and EIR, Volume IV: Inter-Island Conveyance System, Final Report.



NOTE:

1. ALLOW 3 MONTHS FOR START-UP WHICH MAY BEGIN WHEN NEW PRIMARY TREATMENT PLANT AND NEW EFFLUENT OUTFALL ARE READY TO RECEIVE WASTEWATER.

Figure 4-19: Comprehesive Construction Schedule for the South System Pumping Station (MWRA V-III, 1988)

Primary Treatment Facilities

Introduction

Primary treatment is a mechanical process that reduces the biochemical oxygen demand (BOD) and total suspended solids (TSS) by removing solids in sedimentation tanks. Wastewater that enters the sedimentation tanks (Fig. 4-20) remains in the tanks for approximately two to three hours. As this wastewater moves through the tanks, solid particles settle out by gravity. This settling process removes about 60% of the TSS and 30% to 40% of the BOD_5; although, BOD_5 reduction is not the goal of primary treatment.

The new Deer Island Wastewater Treatment Plant will have a maximum capacity of 4810 mld. All of the wastewater that reaches the plant will receive primary treatment. When the secondary treatment portion of the new plant is operational (1999), all wastewater will still receive primary treatment, but only 4090 mld will receive secondary treatment.

Recommended Plan

At the South system pumping station, flow is collected and mixed in the primary splitting boxes. The splitting boxes are used to mix, regulate and split the flow into the primary clarifiers. The flow from the South system pumping station enters the primary sedimentation tanks (clarifiers) along with the flow from the new grit removal facility.

Due to space limitations, it has been recommended that stacked rectangular clarifiers be used at the new treatment plant for primary clarification. Stacked clarifiers have not been used in the United States; although, they have been used in Japan for over a decade, but not for the amount of flow anticipated at the new treatment plant. The stacked rectangular clarifiers at Japanese plants have produced effluent as good or better than conventional rectangular clarifiers. The areas of concern that will be addressed before installing stacked retangular clarifiers are:

- Risk of resuspension when the sludge from the upper tanks passes through the lower tank to the common hopper.

- Transport and collection of sludge may be hindered by lack of a free surface on the lower tank.

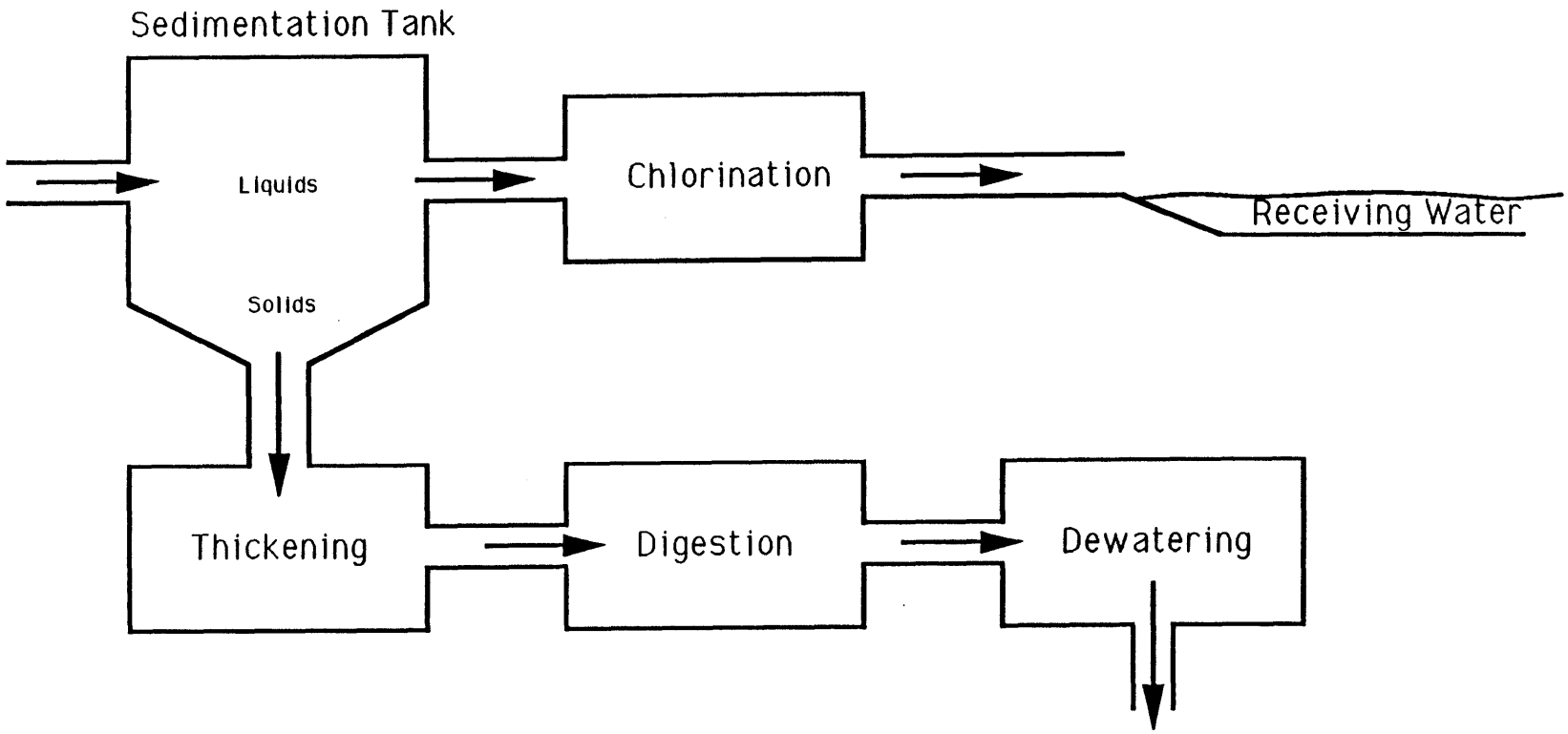


Figure 4-20: Primary Treatment Process

- Risk of flocculant entrainment into the upflow leaving through the effluent weirs.

- Need for high entrance head loss to ensure equal distribution of flow among 128 tanks may cause sludge resuspension and velocity redistribution problems.

Effective operation of the clarifiers requires that the flow be uniform, with respect to solids and fluid. Therefore, each battery of stack clarifiers will receive its influent through 2.6 m wide aeration channels that have two 100 hp blowers diffusing coarse bubbles of air throughout the influent. The purpose of the aeration channels is to uniformly mix the fluid and solids before entering the clarifiers. It is proposed that four batteries with 24 stacked clarifiers (a total of 192 tanks) will be operable at the new treatment plant. Each clarifier will have a 55.2 m long by 6.2 m wide upper tank and a 58.2 m long by 6.2 m wide lower tank. Each tank will have a minimum sidewall depth of 3.7 m. Only 21 stacked clarifiers from each battery will need to be operating in order to treat the peak flow of 4810 mld at an overflow rate of 8.1×10^4 lpd/m^2. The stacked clarifier (Fig. 4-21,22,23) consists of two tanks, one one top of the other, with a common free surface. Each tank will be fed independently and non-metallic chain and flight collectors will convey the suspended solids to a common hopper. The chain and flight collectors travel counter to the current with the solids being directed downward by sludge pumps.

There will be 20 sludge pumps, one pump for every two stacked clarifiers. Each pump is rated at a flow of 454 lpm. Eight pumps will be used as standby. The average daily sludge production will be approximately 152 dry tonnes per day (dtpd). The upper tanks of the clarifiers will be 55.2 m long by 6.25 m wide with a sidewater depth of 3.7 m. The bottom tank will be 58.2 m long, 6.25 m wide, and have a sidewater depth of 3.7 m.

Once secondary treatment begins, 4090 mld will be distributed evenly over three batteries. One of the batteries will provide primary treatment for the flow above 4090 mld and not in excess of 4810 mld. The flow over 4090 mld will receive primary treatment, additional screening, disinfection, and will be discharged along with the secondary effluent. Sizing the clarifiers was based both on overflow rates and particle size removals. For a maximum flow of 4810 mld, the overflow rate is 8.1×10^4 lpd/m^2 and particles with a fall velocity greater than 0.09 cm/sec will be removed. Once secondary treatment begins, 4090 mld will flow through three of the four batteries of stacked clarifiers. These three batteries will have an overflow rate of 9.4×10^4 lpd/m^2 and particles with a fall velocity

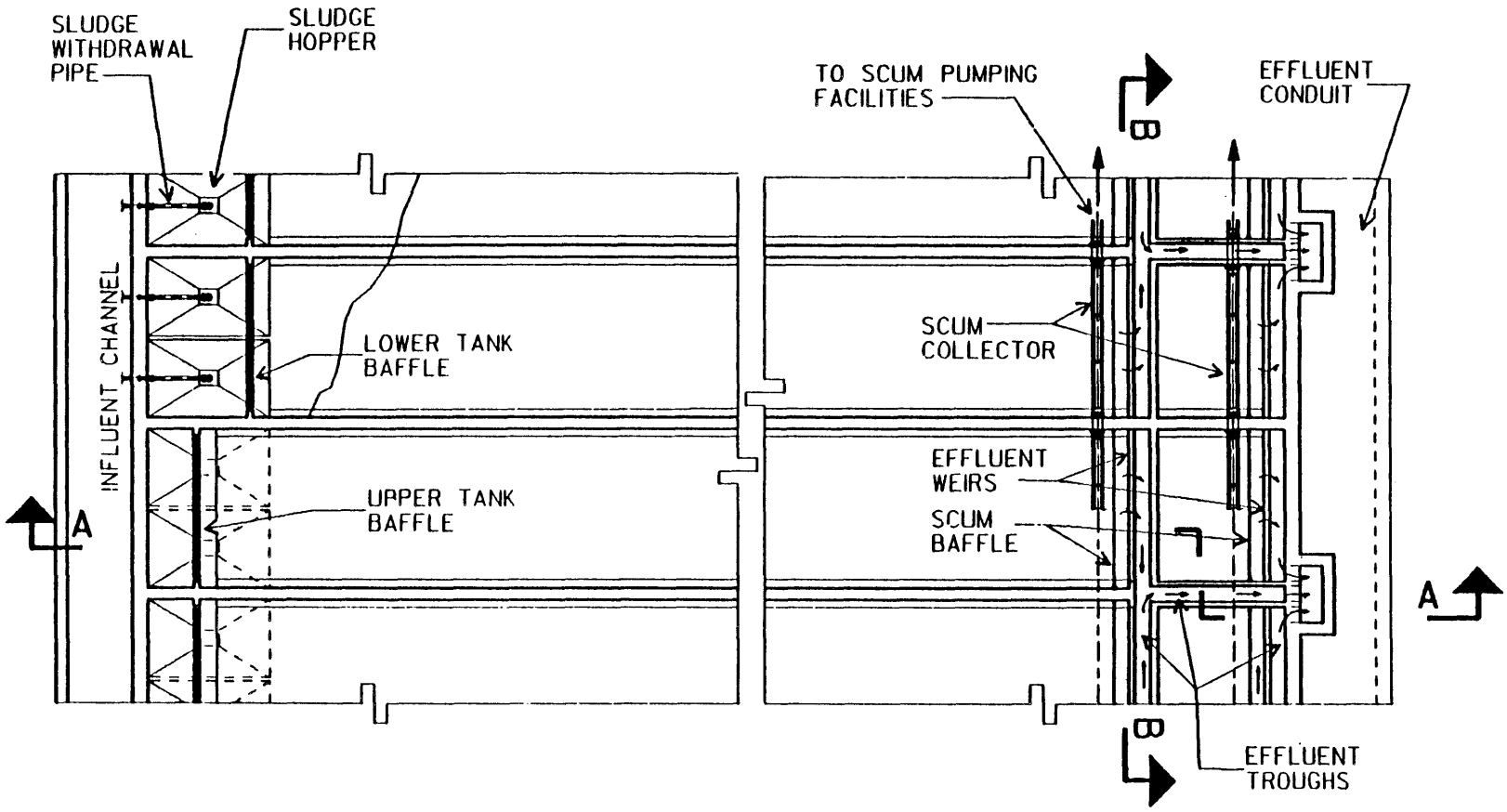
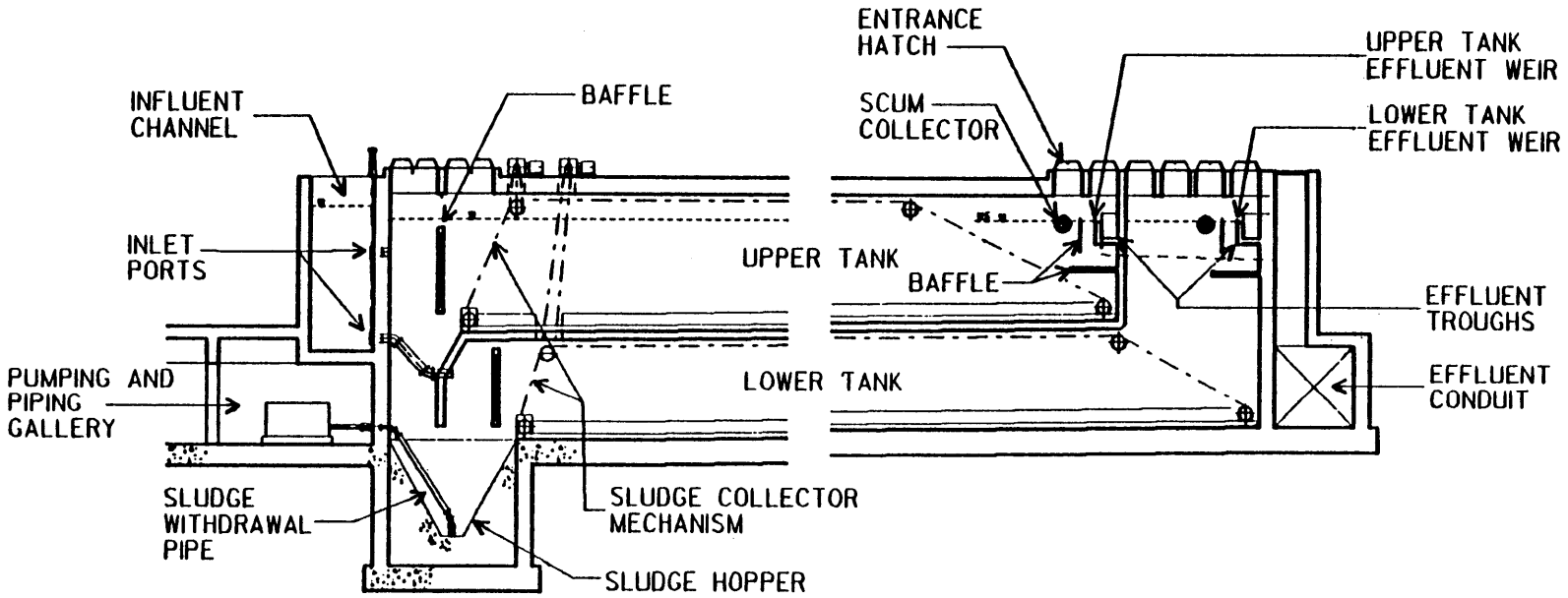


Figure 4-21: Stacked Primary Clarifier - Plan View (MWRA V-III, 1988)



SECTION A-A

NOTE

1. SEE FIGURE II.I.3-17 FOR THE PLAN VIEW OF THE PRIMARY CLARIFIERS

SCALE: 1" = 20'

Figure 4-22: Stacked Primary Clarifier - Longitudinal View (MWRA V-III, 1988)

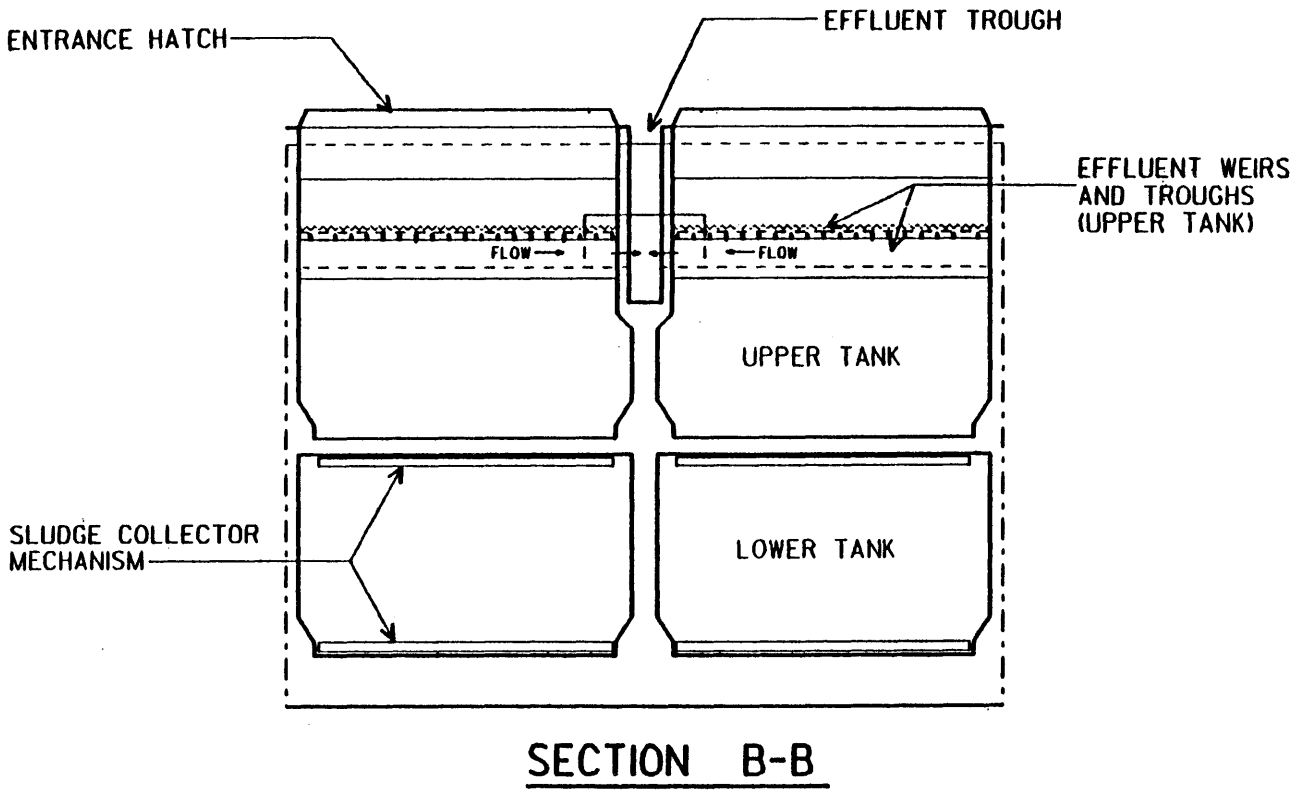


Figure 4-23: Stacked Primary Clarifier - Cross Section (MWRA V-III, 1988)

greater than 0.11 cm/sec will be removed. Flow in excess of 4090 mld and up to 4810 mld, will flow through the one remaining battery of clarifiers. The overflow rate for this battery will be 4.9×10^4 lpd/m^2 and particles with a fall velocity greater than 0.06 cm/sec will be removed.

Floatable material and scum are also collected in the primary clarifiers. Rotating scum troughs are used along the top of the tanks. Chain and flight sludge collectors are used on the lower level. It is anticipated that 64 to 190 dtpd of scum must be handled.

Flows and Loadings

There are four sources of flow that contribute to the wastewater entering MWRA's treatment plants on Deer and Nut Islands. These four sources are:

- domestic wastes from residential activity
- non-domestic wastes from commercial, industrial, and other business related activities
- infiltration and inflows entering due to the age, condition, and location of the sewer pipes relative to the groundwater level
- stormwaters from street drainage intentionally allowed to enter the sewer system in a portion of the system that has combined sewers

Loading of pollutants results from conventional and non-conventional sources. Conventional pollutants are biochemical oxygen demand (BOD) and total suspended solids (TSS). Non-conventional pollutants are metals, acid-based neutrals (ABN's), pesticides and PCB's, and volatile organic compounds (VOC's).

As seen in Fig. 4-24, wastewater from domestic and non-domestic sources ranges from 905 mld to 1240 mld. Domestic wastewater contributes an average of 503 mld and non-domestic wastewater contributes an average of 390 mld to the overall flow entering the system. The maximum estimated infiltration and inflow for both the North and South system ranges from 571 mld to 1033 mld during low groundwater conditions and from 1631 mld to 2366 mld during high groundwater conditions. Low groundwater conditions are expected June through January (eight months) and high groundwater conditions are

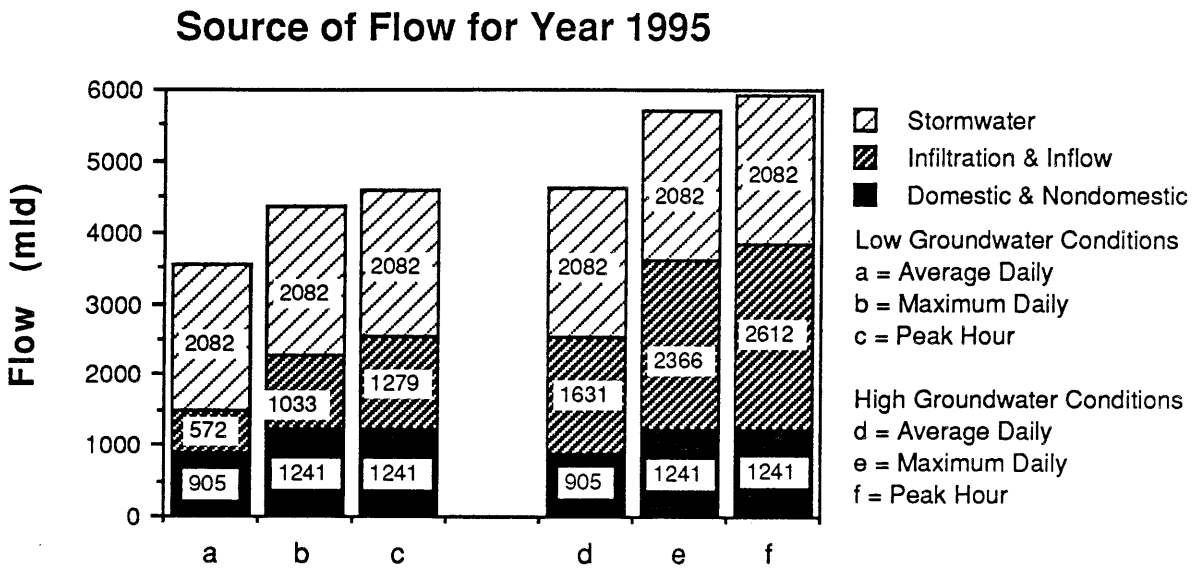


Figure 4-24: Source of Flow for Year 1995

expected February through May (four months). The maximum annual stormwater event adds an additional 2082 mld to the flow from both systems.

The influent and effluent loading and concentration of conventional pollutants are shown in Table 4-2 with BOD_5 removals ranging from 31% to 36% and TSS removals ranging from 58% to 60%. Non-conventional concentrations were determined using an average loading during non-storm events from the Deer Island plant plus the average loading during non-storm events from the Nut Island plant. The projected metals loading can be found in Table 4-3, the projected ABN's loading can be found in Table 4-4. There was no detectable loading for PCB's and pesticides. The projected VOC loading can be found in Table 4-5.

Construction Schedule

The primary treatment facility is expected to be completed by 1995. It is anticipated that construction should last 5 yrs. A comprehensive schedule for the primary treatment facility construction is given in Table 4-1. The construction is estimated to cost $243 million (in 1986 dollars) and operation and maintenance costs will be approximately $4.3 million per year.

References

All material for this section was extracted from the following documents:

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Executive Summary, Final Report.

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Volume III: Treatment Plant, Final Report.

Table 4-2

Projected Influent and Primary Effluent Loadings and Concentrations of Conventional Pollutants for the Year 1999

Constituents	Influent Loading	Effluent (low flow) Loading	Effluent (low flow) Concentration	Effluent (high flow) Loading	Effluent (high flow) Concentration
BOD_5	269,300	172,300	121	185,900	75
TSS	239,200	95,700	67	100,700	41

Note: Loading is in kg/day. Concentration is in mg/liter

Table 4-3

Average Metal Concentrations
Primary Treatment
Year 2020

Constituents	Influent Loading	% Removal	Effluent Concentration
Antimony	7.3	30	2.8
Arsenic	3.4	25	1.4
Boron	712	2	384
Cadmium	3.8	15	1.8
Chromium	40	40	13
Copper	181	35	65
Cyanide, Total	51	10	25
Lead	31	46	9.4
Mercury	2.3	22	1.0
Molybdenum	9.7	10	4.8
Nickel	36	15	17
Selenium	24	10	12
Silver	8.2	30	3.1
Zinc	393	40	130

Note: - Effluent concentration based on an average flow of 480 mgd.
- Influent loadings are in kg/day. Effluent concentrations are in µg/l.

Table 4-4

Average ABN Concentrations
Primary Treatment
Year 2020

Constituents	Influent Loading	% Removal	Effluent Concentration
Phenol	29	20	13
Benzyl Alcohol	39	NA (1)	22
1,2-Dichlorobenzene	34	NA	19
2-Methylphenol	40	NA	22
4-Methylphenol	35	NA	19
Benzoic Acid	152	NA	84
Naphthalene	24	0	13
2-Methylnaphthalene	28	NA	15
2,4,5-Trichlorophenol	182	NA	100
Dimethyl Phthalate	36	24	15
Diethyl Phthalate	31	0	17
N-Nitrosodiphenylamine (1)	39	NA	21
Di-n-butyl Phthalate	31	0	17
Butylbenzyl Phthalate	29	0	16
Bis (2-ethylhexyl) Phthalate	36	0	20
Di-n-octyl Phthalate	30	0	16

Note: - Effluent concentration based on an average flow of 480 mgd.
- Influent loadings are in kg/day. Effluent concentrations are in µg/l.

(1) Where removal efficiences are not availble, no removal was assumed.

Table 4-5

Average Volatile Organic Concentrations
Primary Treatment
Year 2020

Constituents	Influent Loading	% Removal	Effluent Concentration
Bromomethane	28	NA (1)	16
Methylene Chloride	55	0	30
Acetone	190	NA	105
Carbon Disulfide	16	NA	8.6
trans-1,2-Dichloroethane	14	36	4.8
Chloroform	10	NA	5.6
2-Butanone	47	NA	26
1,1,1-Trichloroethane	23	40	7.4
Trichloroethane	20	20	8.7
Benezene	7.5	0	4.1
4-Methyl-2-Pentanone	37	NA	20
Tetrachloreethane	28	0	15
1,1,2,2-Tetrachloroethane	16	NA	8.6
Toluene	32	0	18
Chlorobenzene	15	NA	8.4
Ethylbenzene	15	0	8.3
Styrene	17	0	9.4
Total Xylene; M, O, P	49	NA	27

Note: - Effluent concentration based on an average flow of 480 mgd.
- Influent loadings are in kg/day. Effluent concentrations are in µg/l.

(1) Where removal efficiencies are not available, no removal was assumed.

Secondary Treatment Facilities

Introduction

Secondary treatment is a biological process that reduces the biochemical oxygen demand (BOD) and total suspended solids (TSS). Microorganisms are used in secondary treatment to convert dissolved solids in the wastewater to suspended solids, which settle out in sedimentation tanks. The type of secondary treatment that will be used at the new Deer Island treatment plant is called activated sludge and is the most common method of secondary treatment.

The activated sludge treatment method consists of three processes: aeration, clarification, and the return activated sludge (RAS) (Fig. 4-25). In the aeration process, microorganisms convert the dissolved organic matter in the primary effluent into water, carbon dioxide, and new microorganisms. This is accomplished in an aeration tank by adding air (oxygen) and causing agitation. When the aeration process is complete the clarification process begins. The mixture of microorganisms and effluent (termed mixed liquor suspended solids (MLSS)) flows into the secondary clarifiers, which are similar to the ones used in primary treatment. The solids that settle are termed sludge. This sludge contains a high concentration of active microorganisms and other solids that have settled, hence termed activated sludge. The RAS process returns a portion of the activated sludge to the aeration tank. The RAS, is used to seed the aeration tank with microorganisms in order to increase the efficiency of the purification process.

The biological process described above removes about 90% of TSS and 90% of BOD. The removal of BOD is the primary goal of secondary treatment.

Three alternatives for the activated sludge process were considered by MWRA: air activated sludge, oxygen activated sludge, and a coupled system. All options were similar in terms of noise, constructability, effluent quality, residual quality and quantity, capital costs, operating costs, and life cycle costs. The coupled system required slightly more area, had greater power consumption, and was operationally more complex. Since the coupled system had no clear advantage over the other options, it was eliminated. The oxygen activated sludge process was eventually chosen because it is better at handling variable flows and variable organic loadings, and it has less processing equipment than the air activated sludge process.

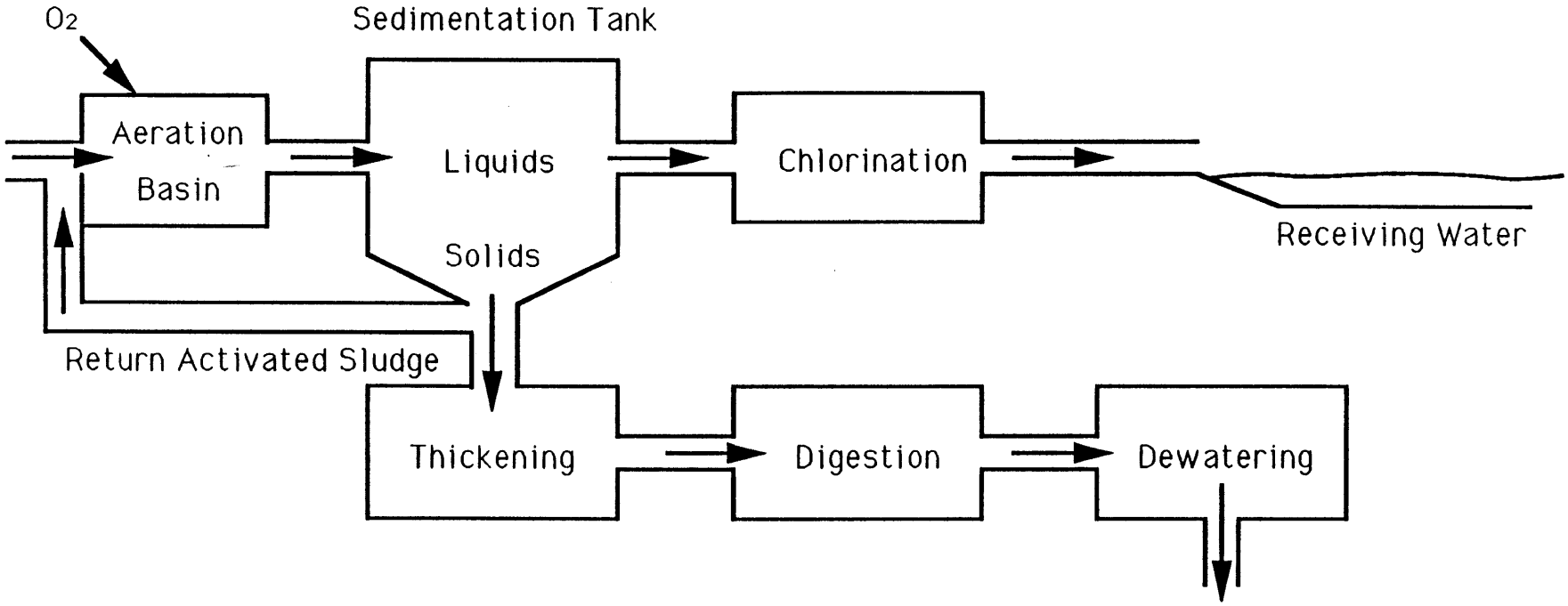


Figure 4-25: Secondary Treatment Process

The new Deer Island Wastewater Treatment Plant will have a maximum capacity of 4810 mld. All of the wastewater that reaches the plant will receive primary treatment. When the secondary treatment portion of the new plant is operational (1999), all wastewater will still receive primary treatment but only 4090 mld will receive secondary treatment.

Recommended Plan

Primary effluent enters the secondary treatment plant through the secondary splitter box. The purpose of the splitter box is to mix and equalize the flow into the anaerobic selector basins (the RAS is introduced just prior to the selector basins). These basins add stability and reliability to the purification process and produce a better-setting sludge by inhibiting the growth of filamentous organisms, which cause sludge bulking. At low concentrations of nutrients, filamentatious bacteria grow faster than floc-forming bacteria. This is because filamentatious bateria have a lower soluble organic matter uptake rate and storage ability than floc-formers. Relying on this principle, if a feed-starve cycle can be achieved, the filamentatious bateria will die off faster in the starve part of the cycle. This is the reason for the anaerobic selector basins.

Inhibiting sludge bulking decreases the secondary clarifier size and potentially reduces the overall oxygen requirement in the aeration basin. The new treatment plant will have a total of four batteries of anaerobic basins, each 39.6 m by 54.9 m with a 5.5 m sidewater depth and 0.9 m of freeboard. Each battery will be divided into four connected parts 19.5 m wide by 27.4 m long (Fig. 4-26). There will be two 20 hp mechanical mixers per compartment (a total of 32 mixers) providing mixing to maintain solids in suspensions. Sizing of the anaerobic selector basins was determined using a detention time of 20 minutes based on 3370 mld and a RAS flow of 8000 mg/l TSS.

The effluent from the anaerobic selector basins enters an aerated influent channel before entering the aeration basins. There are four batteries of aeration basins. The aerated channels are 4.3 m wide and 39.0 m long and are aerated by a coarse-bubble diffused air system in order to maintain the solids in suspension. The diffuser is run by a 20 hp motor and another motor is available as a standby. Within each aeration basin, there are three treatment trains with four stages each (Fig. 4-26). Each stage is 21.3 m wide by 21.3 m long and has a 7.2 m sidewater with a 1.5 m freeboard. There are four aeration motors (one with 150 hp and three with 100 hp) used to keep the solids in suspension. Sizing of

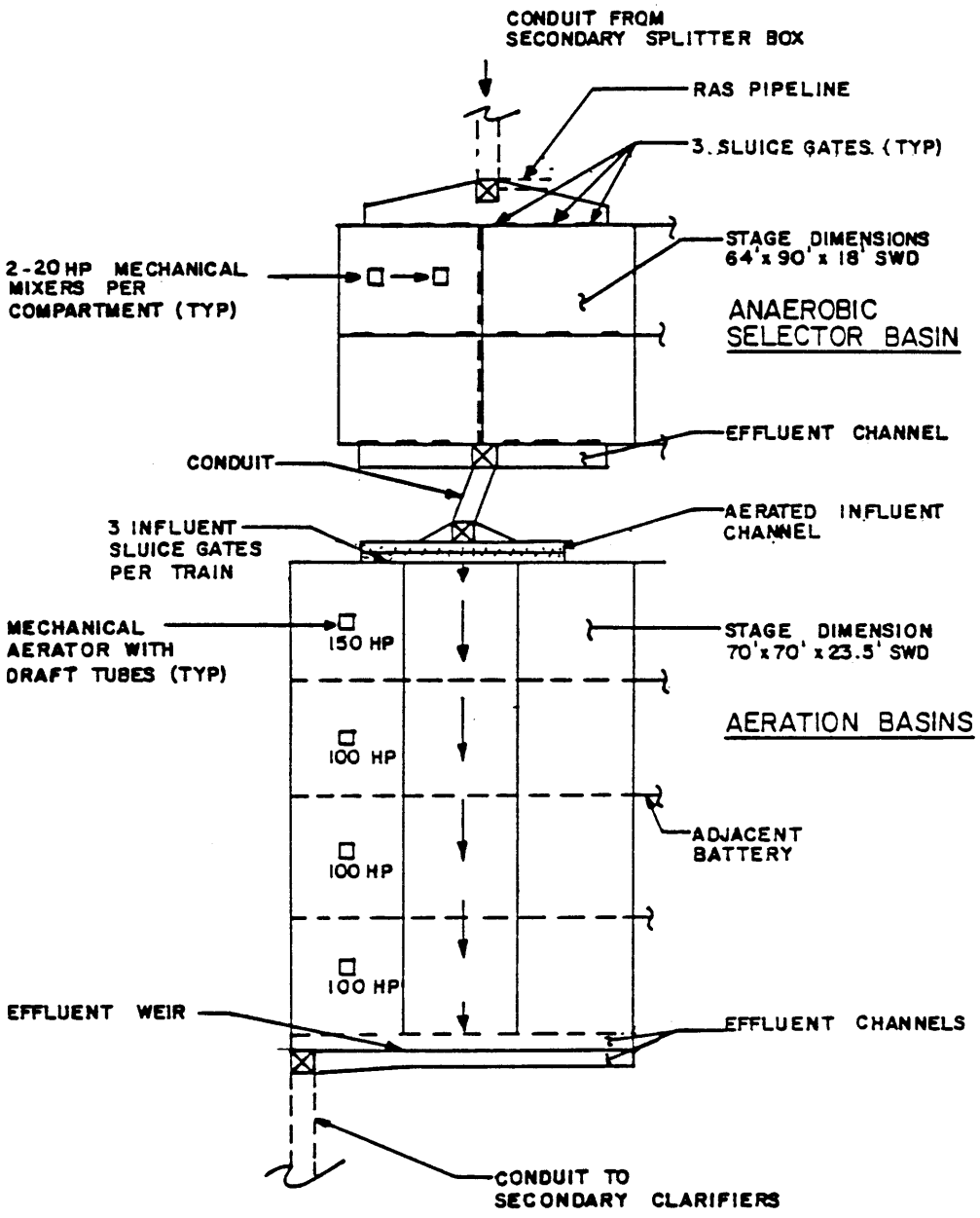


Figure 4-26: Anaerobic Selector and Aeration Basins - Plan View (MWRA V-III, 1988)

the basins was based on maintaining a MLSS of 2000 mg/l at a Solids Retention Time (SRT) of 2.3 days. The total volume of wastewater under aeration is 0.16×10^9 liters.

Pure oxygen (95%) will be introduced above the wastewater contained within the covered aeration basins. This oxygen will be produced on site in cryogenic oxygen generators, which separate oxygen from the air. Atmospheric air is essentially 79% nitrogen and 21% oxygen. The separation process compresses and cools the fed air until liquification occurs. Impurities are removed by condensation. Oxygen is then separated from the air by fractional distillation, which is based on the differences in boiling points of nitrogen and oxygen to separate out the oxygen. A capacity of 910 tonnes of liquid oxygen will be stored for peak demands. Two 270 tonnes per day generators will be available - only one is required for the estimated peak daily demand and one will be used as standby. The building required to house the generators and storage facility will occupy approximately 910 m^2.

Due to space limitations, it has been recommended that stacked rectangular clarifiers be used at the new treatment plant for secondary clarification. Stacked clarifiers have not been used in the United States; although, they have been used in Japan for over a decade but not for the amount of flow anticipated at the new treatment plant. The stacked rectangular clarifiers at Japanese plants have produced effluent as good or better than conventional rectangular clarifiers. The areas of concern that will be addressed before installing stacked retangular clarifiers are:

- Risk of resuspension when the sludge from the upper tanks passes through the lower tank to the common hopper.

- Transport and the collection of sludge may be hindered by lack of a free surface on the lower tank.

- There is a possibility of flocculant entrainment into the upflow leaving through the effluent weirs.

- The need for high entrance head loss to ensure equal distribution of flow among 128 tanks may cause sludge resuspension and velocity redistribution problems.

Effective operation of the clarifiers requires that the flow be uniform, with respect to solids and fluid. Therefore, each battery of stack clarifiers will receive its influent through 128 m long by 2.6 m wide aeration channels that will have two 150 hp blowers diffusing coarse bubbles of air throughout the influent (one blower is for standby). It is proposed that four batteries with 36 stacked clarifiers (a total of 288 tanks) will be operable at the new treatment plant. Of these, only 32 stacks of each battery will need to be operating in order to treat the peak flow of 4090 mld. The stacked clarifier (Fig. 4-27) consists of two tanks, one on top of the other, with a common free surface. Each tank will be fed independently and non-metallic chain and flight collectors will convey the suspended solids to a common hopper. The chain and flight collectors travels counter to the current with the solids being directed downward by gravity.

Each battery has five 1.14×10^5 liters per minute (lpm) RAS pumps with 300 hp variable-speed motors (two as standby) to convey the RAS to the anaerobic selector basins. Each battery also has five 1.9×10^4 lpm Waste Activated Sludge (WAS) pumps with 7.5 hp motors (two as standby) to convey the WAS to the residual processing unit. The upper tanks of the clarifiers will be 48.8 m long by 6.3 m wide with a sidewater depth of 4.0 m. The bottom tank will be 54.9 m long, 6.3 m wide, and have a sidewater depth of 4.0 m.

Floatable material and scum is also collected in the secondary clarifiers. Rotating scum troughs are used along the top of the tanks. Chain and flight sludge collectors are used on the lower level. It is anticipated that 64 to 190 dry tonnes per day (dtpd) of scum must be handled. The scum will be pumped to the residual treatment process by $16 - 2.7 \times 10^3$ lpm scum pumps with 40 hp motors, 8 pumps as standby.

Once secondary treatment begins, flow in excess of 4090 mld will receive primary treatment, additional screening, and disinfection, and will then be discharged along with the secondary effluent. All flow up to 4090 mld will receive both primary and secondary treatment. Sizing for the clarifiers was based on a peak overflow rate of 4.9×10^4 lpd/m^2 for a flow of 4090 mld and an overflow rate of 3.1×10^4 lpd/m^2 for an average high groundwater flow of 2540 mld.

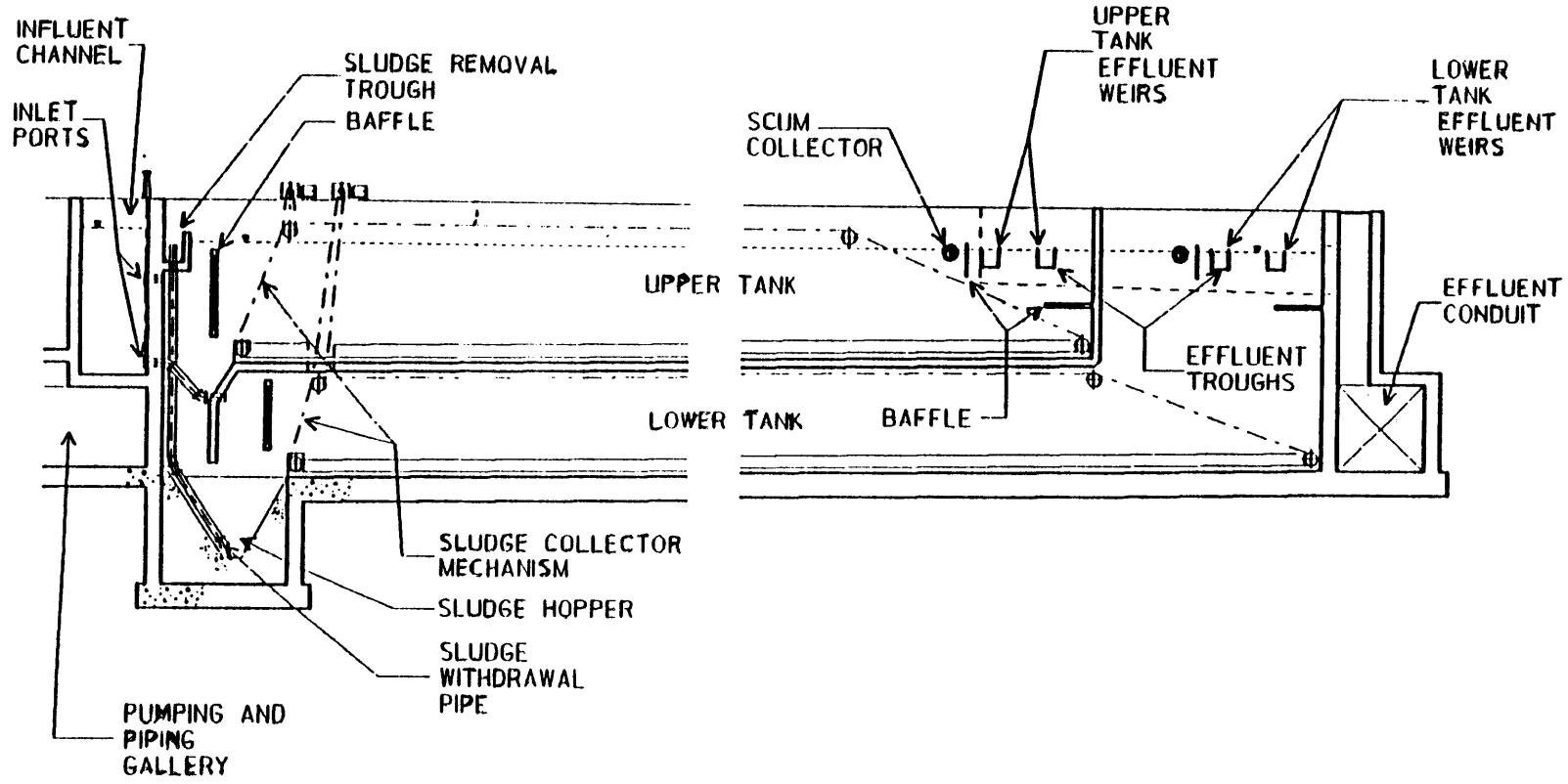


Figure 4-27: Stacked Secondary Clarifier - Longitudinal View (MWRA V-III, 1988)

Flows and Loadings

There are four sources of flow that contribute to the wastewater entering MWRA's treatment plants on Deer and Nut Islands. These four sources are:

- domestic wastes from residential activity
- non-domestic wastes from commercial, industrial and other business related activities
- infiltration and inflows entering due to the age, condition, and location of the sewer pipes to the groundwater level
- stormwaters from street drainage intentionally allowed to enter the sewer system.

Loading of pollutants results from conventional and non-conventional sources. Conventional pollutants are biochemical oxygen demand (BOD) and total suspended solids (TSS). Non-conventional pollutants are metals, acid-based neutrals (ABN's), pesticides and PCBs, and volatile organic compounds (VOC's).

As seen in Figure 4-28, wastewater from domestic and non-domestic sources ranges from 905 mld to 1241 mld. Domestic wastewater contributes an average of 515 mld and non-domestic wastewater contributes an average of 390 mld to the overall flow entering the system. The maximum estimated infiltration and inflow for both the North and South system ranges from 572 mld to 1033 mld during low groundwater conditions and from 1631 mld to 2366 mld during high groundwater conditions. Low groundwater conditions are expected June through January (eight months) and high groundwater conditions are expected February through May (four months). The maximum annual stormwater event adds an additional 2082 mld to the flow from both systems.

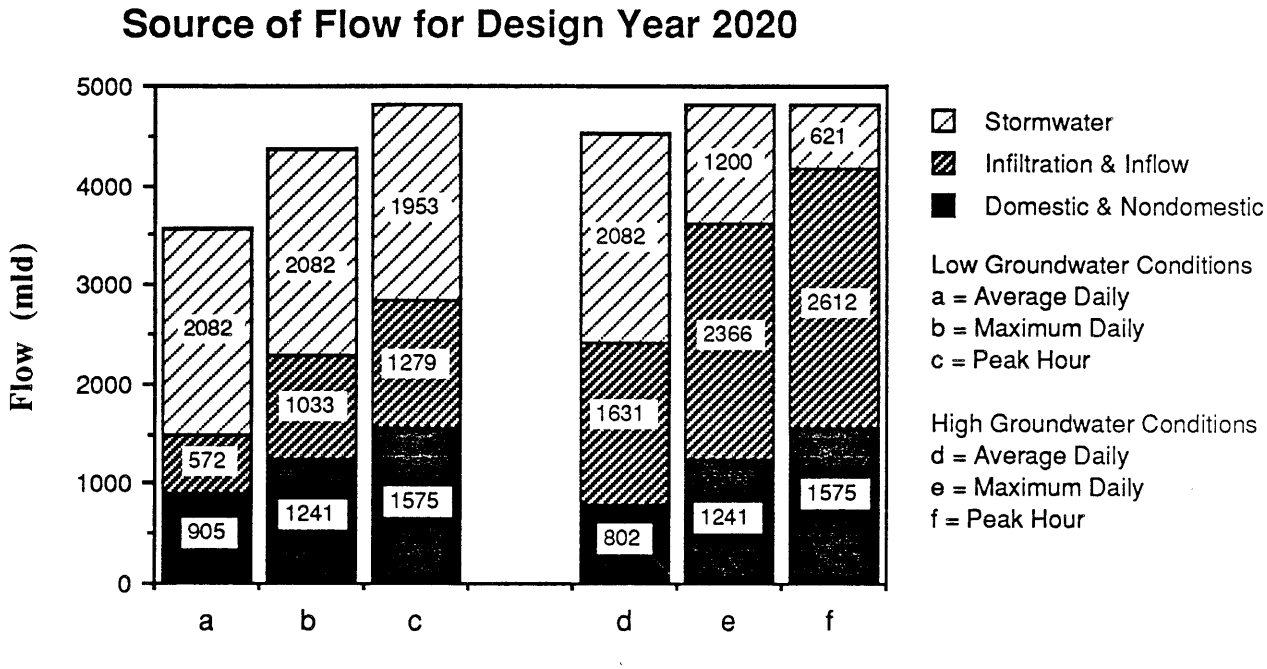


Figure 4-28: Source of Flow for Design Year 2020

The influent and effluent loading and concentration of conventional pollutants are shown in Table 4-6 with BOD_5 removals ranging from 79 to 91% and TSS removals ranging from 77 to 90%. Non-conventional concentrations were determined using an average loading during non-storm events from the Deer Island plant plus the average loading during non-storm events from the Nut Island plant. The projected metals loading can be found in Table 4-7. The projected ABN's loading can be found in Table 4-8. There was no detectable loading for PCBs and pesticides. The projected VOC loading can be found in Table 4-9.

Construction Schedule

The secondary treatment facility is expected to be completed by 1999. It is anticipated that construction should last 5 yrs. A comprehensive schedule for the secondary treatment facility construction is given in Table 4-1. The construction is estimated to cost $567 million (in 1986 dollars) and operation and maintenance costs will be approximately $7.35 million per year.

References

All material for this section was extracted from the following documents:

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Executive Summary, Final Report.

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Volume III: Treatment Plant, Final Report.

Table 4-6

Projected Influent and Secondary Effluent Loadings and Concentrations of Conventional Pollutants for the Year 2020

Constituents	Influent Loading	Effluent (low flow) Loading	Effluent (low flow) Concentration	Effluent (high flow) Loading	Effluent (high flow) Concentration
BOD_5	258,500	23,400	15	54,000	20
TSS	233,600	23,400	15	54,000	20

Note: Loading is in kg/day. Concentration is in mg/liter

Table 4-7

Average Metal Concentrations
Secondary Treatment
Year 2020

Constituents	Influent Loading	% Removal	Effluent Concentration
Antimony	7.3	60	1.6
Arsenic	3.4	50	1.0
Boron	712	5	372
Cadmium	3.8	50	1.1
Chromium	40	76	5.3
Copper	181	82	18
Cyanide, Total	51	60	11
Lead	31	57	7.3
Mercury	2.3	75	0.3
Molybdenum	9.7	50	2.7
Nickel	36	32	13.4
Selenium	24	50	6.6
Silver	8.2	90	0.5
Zinc	393	76	52

Note: - Secondary treatment removals include removals in primary treatment.
- Effluent concentration based on an average flow of 480 mgd.
- Influent loadings are in kg/day. Effluent concentrations are in µg/l.

Table 4-8

Average ABN Concentrations
Secondary Treatment
Year 2020

Constituents	Influent Loading	% Removal		Effluent Concentration
Phenol	29	95		0.8
Benzyl Alcohol	39	90	(1)	2.1
1,2-Dichlorobenzene	34	90		1.9
2-Methylphenol	40	90	(1)	2.2
4-Methylphenol	35	90	(1)	1.9
Benzoic Acid	152	90	(1)	8.4
Naphthalene	24	95		0.6
2-Methylnaphthalene	28	90	(1)	1.5
2,4,5-Trichlorophenol	182	90	(1)	10
Dimethyl Phthalate	36	95		1.0
Diethyl Phthalate	31	90		1.7
N-Nitrosodiphenylamine (1)	39	90	(1)	6.7
Di-n-butyl Phthalate	31	90		1.7
Butylbenzyl Phthalate	29	95		0.8
Bis (2-ethylhexyl) Phthalate	36	90		1.9
Di-n-octyl Phthalate	30	90		1.6

Note: - Secondary treament removal includes removal in primary treatment.
- Effluent concentration based on an average flow of 480 mgd.
- Influent loadings are in kg/day. Effluent concentrations are in µg/l.

(1) Where removal efficiences are not availble, 90% removal was assumed based on the chemical similarities with constituents of known removal..

Table 4-9

Average Volatile Organic Concentrations
Secondary Treatment
Year 2020

Constituents	Influent Loading	% Removal		Effluent Concentration
Bromomethane	28	95		0.8
Methylene Chloride	55	95		1.5
Acetone	190	95		5.2
Carbon Disulfide	16	95		0.4
trans-1,2-Dichloroethane	14	90		0.7
Chloroform	10	90		0.5
2-Butanone	47	90	(1)	2.6
1,1,1-Trichloroethane	23	95		0.6
Trichloroethane	20	95		0.5
Benezene	7.5	95		0.2
4-Methyl-2-Pentanone	37	90	(1)	2.0
Tetrachloreethane	28	90		1.5
1,1,2,2-Tetrachloroethane	16	90		0.8
Toluene	32	90		1.8
Chlorobenzene	15	90		0.8
Ethylbenzene	15	95		0.4
Styrene	17	90		0.9
Total Xylene; M, O, P	49	95		1.3

Note: - Secondary treament removal includes removal in primary treatment.
- Effluent concentration based on an average flow of 480 mgd.
- Influent loadings are in kg/day. Effluent concentrations are in µg/l.

(1) Where removal efficiences are not availble, 90% removal was assumed based on the chemical similarities with constituents of known removal..

Air Emission Control Facilities

Introduction

Odor in wastewater is usually caused by the volatilization of organic compounds and the biodegradation of organic matter. Volatile organic compounds (VOCs) include all hydrocarbons (except methane and ethane) with a vapor pressure equal to or greater than 0.1 mm of mercury. Elimination of these odors is a major concern in the design and operation of treatment facilities. Odors are important to control more because of psychological stress they produce, which is greater than the harm they impose on the human body. The most characteristic wastewater odor is hydrogen sulfide.

Sources of odor/VOCs are the facilities where the wastewater comes in contact with the air. These interfaces exist at tank surfaces, channels, weirs, and aeration tanks. The principal sources of odor for the new treatment facilities are as follows:

- North System Remote Headworks
- Nut Island Headworks
- North System Main Pumping Station Tunnel Vent Shafts
- Winthrop Terminal Pumping Station
- South System Pumping Station Wet Well
- Deer Island Grit Removal Facility
- Grit Handling Building
- Primary Wastewater Treatment
  - splitter box
  - influent channels
  - primary clarifiers
- Primary Screening Building
- Secondary Wastewater Treatment
  - splitter box
  - anaerobic selectors
  - aeration basins
  - secondary clarifier influent channels
  - return activated sludge wet well vent

In Massachusetts, odors/VOCs are regulated by the following agencies:

- Local Boards of Health
- State Department of Health
- Division of Air Pollution Control
- Division of Water Pollution Control

These agencies set limits for odorous compounds and VOCs in general terms rather than numerical limits.

Recommended Plan

The proposed treatment facilities at Deer Island will be completely covered from the centrifugal grit chambers to the aeration tank effluent weirs (Fig. 4-2). Covers will be used to prevent the emission of uncontrolled air pollution. The emissions captured by the covers will be collected and delivered to an air emission control system before release to the atmosphere.

Annual release rates from existing facilities and for future facilities (with emissions control) are shown in Table 4-10. Future treatment values for secondary treatment at Deer Island include oxygen and air activated sludge processes. Nut Island currently hosts a primary treatment plant. In the future, this plant will be demolished and replaced by a headworks facility.

Odors can be controlled by removing or controlling pollutants in the liquid and air streams. The methods available to control odors/VOCs in the liquid stream include: chemical oxidation, raising the oxidation/reduction potential, and pH control. The methods available to control odors/VOCs in the air stream include: ozonation, wet scrubbing, carbon adsorption, incineration, and condensation. Below is a short description of each method.

- Chemical oxidation is the addition of chemicals in order to inhibit the growth of bacteria, which produces odorous chemicals.

- Oxidation/reduction is the addition of oxygen/nitrate to inhibit the production of sulfides.

Table 4-10: Comparison of Annual Controlled Constituent Emission Estimates for Existing and Future Treatment Systems on Deer and Nut Islands (tons/yr) (MWRA V-III, 1988)

	Deer Island			Nut Island	
	Existing	Future		Existing	Future
Constituent		oxygen	air		
Benzene	0.64	0.20	0.20	0.09	0.01
Chloroform	0.91	0.39	0.41	0.13	0.02
Ethylbenzene	1.38	0.53	0.50	0.25	0.03
Methylene chloride	5.16	1.58	1.64	0.80	0.13
Tetrachloroethene	2.26	0.70	0.73	0.37	0.06
Toluene	3.10	0.92	0.90	0.30	0.05
Trans 1,2-dichloroethene	1.37	0.41	0.44	0.23	0.03
1,1,1-Trichloroethane	2.15	0.92	1.11	0.27	0.04
Trichloroethene	1.68	0.80	0.77	0.26	0.04
Trichlorofluoromethane	1.97	0.55	0.61	0.30	0.05
Styrene	1.47	0.64	0.71	0.23	0.04
Acetone	0.43	0.13	0.09	0.10	0.01
2-Butanone	0.00	0.02	0.02	0.08	0.00
Total xylenes	4.65	1.29	1.30	0.46	0.08
1,1,2,2-Tetrachloroethane	0.14	0.13	0.18	0.08	0.00
Methyl mercaptan	2.84	0.59	0.63	0.60	0.09
Bromomethane	2.80	1.30	1.50	0.48	0.08
2-Propanone, 1-fluoro	0.02	0.07	0.03	0.03	0.00
Carbon disulfide	1.25	0.24	0.25	0.22	0.04
Butanone, 3-methoxy, 3-methyl	0.02	0.06	0.04	0.01	0.00
Ethyl ether	0.09	0.10	0.12	0.02	0.00
Phenol	0.00	0.02	0.00	0.04	0.00
Naphthalene	1.29	0.64	0.64	0.58	0.10
Chlorobenzene	1.32	0.45	0.45	0.22	0.04
o-Cresol	0.00	0.00	0.00	0.00	0.00
p-Cresol	0.00	0.00	0.00	0.05	0.00
1,2-Dichlorobenzene	2.36	1.31	1.41	0.60	0.10
Benzenamine	0.00	0.07	0.00	0.00	0.00
Hexone (MIBK)	0.08	0.11	0.14	0.02	0.00
Benzyl alcohol	0.08	0.04	0.02	0.04	0.00
Pentane, 3-meth, 2,2,4-trimethyl	0.81	0.43	0.46	0.16	0.03
Dimethyl disulfide	0.00	0.02	0.00	0.00	0.00
Dimethyl sulfide	0.00	0.00	0.00	0.00	0.00
TOTAL	40.40	14.66	15.31	7.01	1.06

- Lime or caustic soda is added to raise the pH in order to inhibit the release of hydrogen sulfide.

- Ozonation brings the odorous air in contact with ozone in a long baffled chamber to promote mixing and oxidation.

- Wet scrubbing brings the odorous air in contact with the scrubbing liquid, allowing the odorous air to be absorbed into the liquid.

- Carbon adsorption systems allow odorous air to enter the vessel and flow through carbon beds which adsorb the odorous-causing compounds and VOCs.

- Incineration destroys VOCs by introducing the VOCs to heat.

- Condensation cools the odorous air to below the dew point of water. The VOCs, and the liquid are condensed and collected.

MWRA decided that the odor control systems will include packed tower scrubbers that are five to nine ft in diameter, exhaust fans, and 12 ft diameter dual bed carbon adsorbers.

Packed tower scrubbers (Fig. 4-29) are filled with a plastic media in order to create turbulence and increase the interfacial area between the liquid and air. Air is injected into the tower below the plastic packing media. Water is then sprayed over the packing so that the odorous chemicals and VOCs absorb into the liquid. The treated air is then released through the top of the scrubber. The liquid is filtered and recycled to spray the packing again. A caustic soda is also added in order to raise the pH and inhibit the release of hydrogen sulfide. This method is highly effective in removing odorous compounds, but it is not as effective in removing VOCs.

Air treated in the packed tower scrubbers is then blown into a dual-bed carbon adsorber. The dual-bed carbon adsorber (Fig. 4-30) is highly effective in removing VOCs. The air passes through the two activated carbon beds which adsorb the VOCs and odorous causing compounds. The treated air is then discharged through stacks into the atmosphere. The carbon will be desorbed approximately once a week with nitrogen or air. The life of

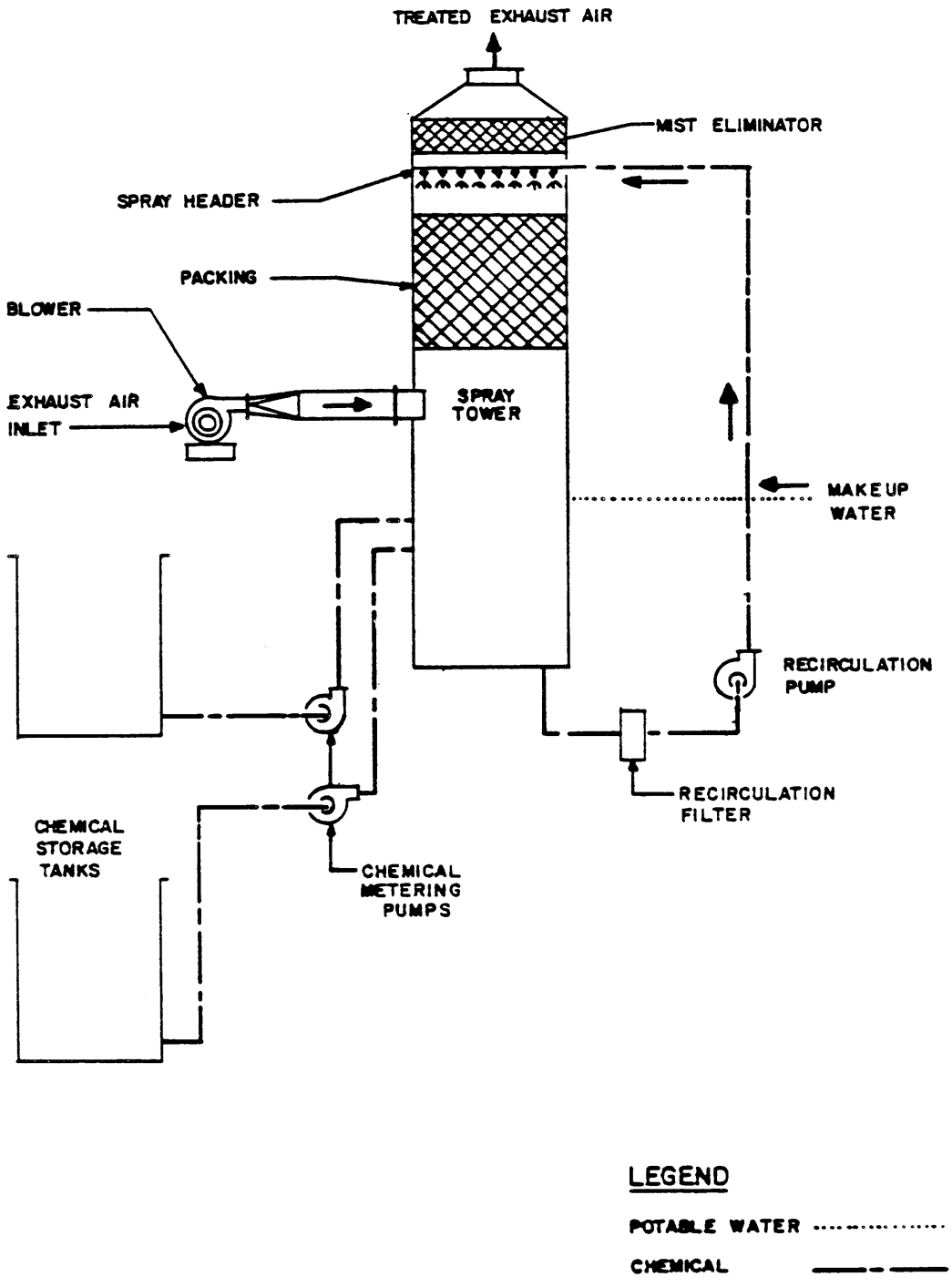


Figure 4-29: Packed-Tower Scrubber (MWRA V-III, 1988)

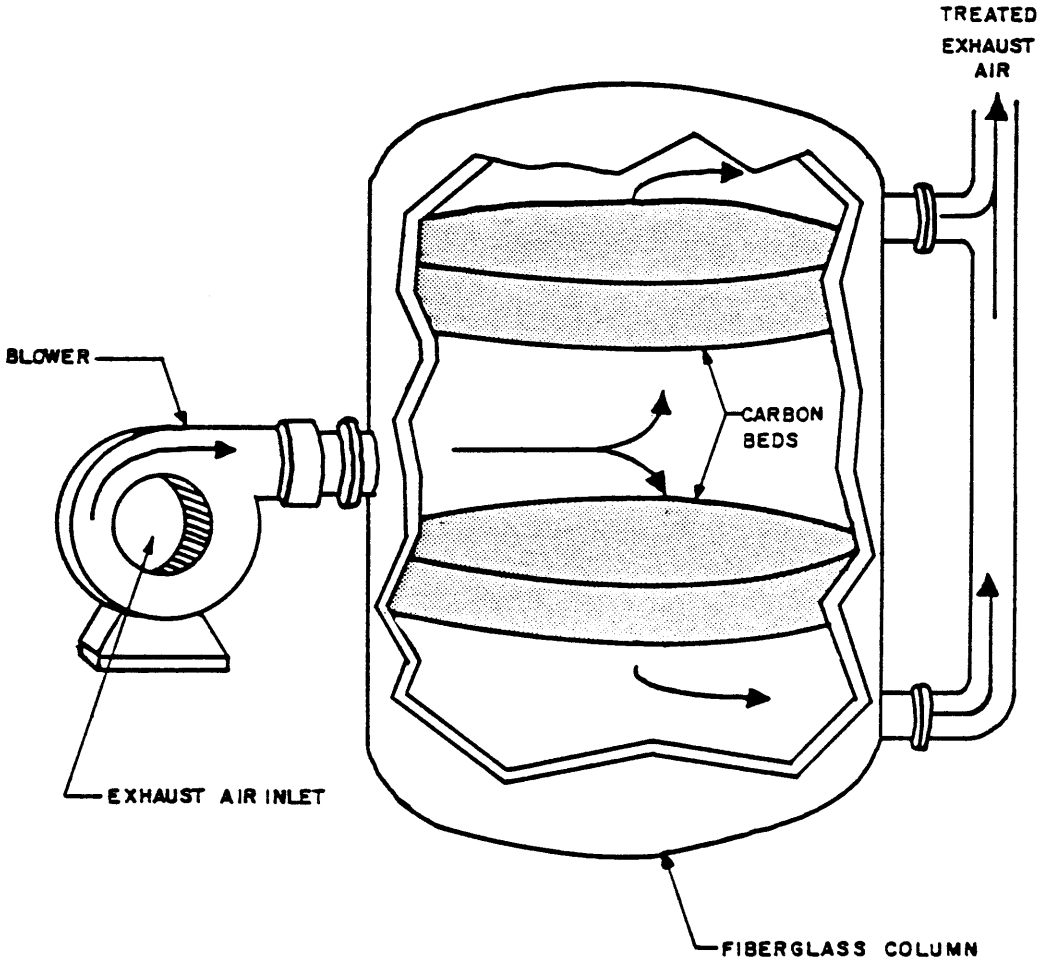


Figure 4-30: Section of Dual-Bed Activated Carbon Column (MWRA V-III, 1988)

the activated carbon beds is approximately one year. At such time, the carbon beds will be taken out and regenerated or replaced.

Wet scrubbing followed by carbon adsorption is estimated to remove 95-99% of the hydrogen sulfide and 70-90% of the VOCs from the air stream. A description of the air emission control facilities can be found in Table 4-11.

References

All material for this section was extracted from the following documents:

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Executive Summary, Final Report.

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Volume III: Treatment Plant, Final Report.

Table 4-11: Description of Air Emission Control Facilities (MWRA V-III, 1988)

Location	System Capacity (scfm)	(No.) Diameter (ft) of Packed Tower Scrubbers	(No.) Diameter (ft) of Dual Bed Carbon Adsorbers	(No.) Size of Exhaust Fans (scfm)	Building Size (l x w x h)
Winthrop Terminal	15,300	(2) 6	(2) 12	(2) 15,300	50 ft x 80 ft x 28 ft
East Air Emission Control Complex:	--	--	--	--	85 ft x 150 ft x 30 ft
Grit Chambers	19,000	(2) 6	(2) 12	(2) 19,000	--
Grit Handling Bld.	21,000	(2) 7	(2) 12	(2) 21,000	--
Primary Clarifiers	39,800	(2) 9	(2) 12	(2) 39,800	--
Clarifier Maint.	22,100	(1) 7	(1) 12	(1) 22,100	
West Air Emission Control Complex:	--	--	--	--	80 ft x 125 ft x 30 ft
Grit Chambers	19,000	(2) 6	(2) 12	(2) 19,000	--
Primary Clarifiers	32,600	(2) 8	(2) 12	(2) 32,600	--
Clarifier Maint.	22,100	(1) 7	(1) 12	(1) 22,100	--
Screening Building	21,000	(2) 7	(2) 12	(2) 21,000	60 ft x 75 ft x 28 ft
Secondary Treatment (East)	21,000	(2) 7	(2) 12	(2) 21,000	50 ft x 80 ft x 28 ft
Secondary Treatment (West)	18,000	(2) 5	(2) 12	(2) 18,000	50 ft x 80 ft x 28 ft
Nut Island Headworks	55,000	(3) 7	(4) 12	(4) 27,500	65 ft x 100 ft x 20 ft

Disinfection

Introduction

Disinfection is the most important part of the wastewater treatment process from a public health stand point. Disinfection kills bacteria and microorganisms thereby reducing the number of indicator bacteria present in the effluent from the treatment plant to an acceptable level before releasing it into the environment. The disinfection methods may vary, but they must meet effluent standards. State water quality standards and regulations dictate the acceptable amount of bacteria in effluent. Massachusetts requires that disinfection by chlorination have a minimum detention time of 15 minutes at peak flow, and if discharged into shellfish beds, a minimum detention time of 30 minutes.

Discharge permits require that the most probable number (MPN) of fecal coliforms should not exceed a count of 200 per 100 ml at the shoreline. The effluent must also pass the Whole Effluent Toxicity Test (WET test) which states that residuals from disinfection must not be lethal to fish or other marine organisms (this is the reason for dechlorination).

The primary effluent will require more disinfection than the secondary effluent, and since the new treatment plant will operate with primary treatment for five years, the design criteria was based on the primary effluent. Sodium hypochlorite (NaOCl) will be used because it is the only disinfection method readily available that reduces fecal coliform counts in primary effluent to an acceptable level. Sodium hypochlorite is a sodium salt of hypochlorous acid. It is applied as a solution, is sensitive to sunlight, and will decay with time. The WET test indicated that dechlorination will also be required. Sodium bisulfate ($NaHSO_3$) will be used to dechlorinate after disinfection. Sodium bisulfate can effectively reduce the free and combined chlorine residuals which form during chlorination.

There are other uses for the disinfectant at the treatment plant. These uses include non-potable water, bulking control, and odor control. Non-potable water will be drawn from the secondary effluent for use in cleaning basins, chemical mixing, etc. Disinfectants in the non-potable water will prevent slime growth and health hazards. Disinfectants are added to the Return Activated Sludge (RAS) to control bulking of the sludge by filamentatious bacteria. At low concentrations of nutrients, filamentatious bacteria grow faster than floc-forming bacteria. This is because filamentatious bacteria have a lower soluble organic matter uptake rate and storage ability than floc-formers. Using this

principle, if low concentrations of a disinfectant is introduced into the wastewater stream, the first type of bacteria to die off is the filamentatious bacteria. The amount of disinfectant needed to control bulking is highly variable depending on the time of year (more in the summer months). It is also expected that disinfectants will be needed for odor control within the facilities.

Recommended Plan

The recommended plan for disinfection of the effluent from the treatment plant is to chlorinate with sodium hypochlorite and then dechlorinate with sodium bisulfate. Both of these chemicals will be barged to Deer Island and stored within the treatment plant.

Since dechlorination is required, disinfection by sodium hypochlorite will take place in four contact basins – not in the tunnel outfall. These contact basins are designed for a peak flow of 4630 mld and a minimum detention time of 15 minutes. The contact basins need to be 7.3 m deep, 6.1 m wide, and 1100 m long. Space requirements dictate the total length of the basins, hence, the contact basins will be setup in a serpentine fashion consisting of three passes with each portion of the basin being 7.3 m deep, 18.3 m wide, and 90.0 m long.

The sodium hypochlorite, with a concentration of 1% to 15% chlorine, will be distributed by diffusers at the head-end of the contact channel. Thirteen metering pumps will supply the sodium hypochlorite to the diffusers. Regulations require that these pumps deliver 25 mg/l of chlorine for average high groundwater conditions (3600 mld). To meet this requirement, only 11 pumps are needed - two as standbys. The amount of chlorine released through each pump is controlled by chlorine residue analysis. The analysis is performed at the tail-end of the channels. Dechlorination by sodium bisulfate is done in the contact basin effluent channel.

Storage facilities for the sodium hypochlorite and sodium bisulfate were designed based on the disinfection requirements for eight days of primary effluent at a maximum rate of high groundwater flow of 3600 mld. When secondary treatment begins, the amount of sodium hypochlorite needed to disinfect the effluent will drop causing a redundance in some of the storage facilities. There needs to be storage for 2.8×10^6 liters of both chemicals. This requires a total of six tanks with a diameter of 12.2 m, a height of 9.1 m, and a capacity of 0.95×10^6 liters. The storage tanks will be made of concrete or steel and will be lined with epoxy or polyethelene to prevent corrosion. The tanks will be

surrounded by a containment ditch to control spillage, valve failures, and tank leakage. The construction is estimated to cost $22 million (1986 dollars).

References

All material for this section was extracted from the following documents:

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Executive Summary, Final Report.

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Volume III: Treatment Plant, Final Report.

New Ocean Outfall

Introduction

A new outfall is being designed to transport effluent from the proposed Deer Island Wastewater Treatment Plant to a location in the ocean. Possible locations for the outfall terminus are shown in Figure 4-31. The recommended site is approximately 15 km from Deer Island. The effluent water will enter the ocean designated as class SA under the standards established by the Division of Water Pollution Control (DWPC) of the DEQE. The standards designate different uses ranging from SA to SC with SA begin the highest use class. The criteria applicable to the class SA waters are shown in Table 4-12. These criteria need to be met and maintained for permitting an effluent outfall.

Whether the effluent meets these standards depends on the physical characteristics at the terminus location because these characteristics influence the transport and fate of the effluent. Physical characteristics include dilution, dispersion, and sedimentation. Wastewater effluent undergoes three phases of transport and fate: near-field mixing, intermediate-scale transport and impacts, and far-field mixing.

Near-field mixing, or initial dilution, occurs immediately after discharge and has a time scale of a few minutes. The major driving forces are momentum and buoyancy. In the near-field mixing zone, large quantities of ambient waters are entrained with the effluent causing initial dilution. Most state and federal water quality criteria are required to be met at the end of this process. This process depends on the multi-port diffuser (location, size, and orientation), the effluent properties (velocity and density), and the receiving water characteristics (depth, stratification, velocity, and currents).

The model used to evaluate the near-field mixing was EPA's ULINE. ULINE uses parameters such as diffuser characteristics, effluent flow rate and density, ambient currents, and stratification to calculate the near-field initial dilution and the maximum height of rise of the effluent plume. The height of rise of the plume is associated with the density profile of the water column. If there is strong stratification (large density differences) then the effluent plume is restricted from mixing with the upper layers of the water column; consequently, reducing the amount of initial dilution and trapping the plume beneath the surface.

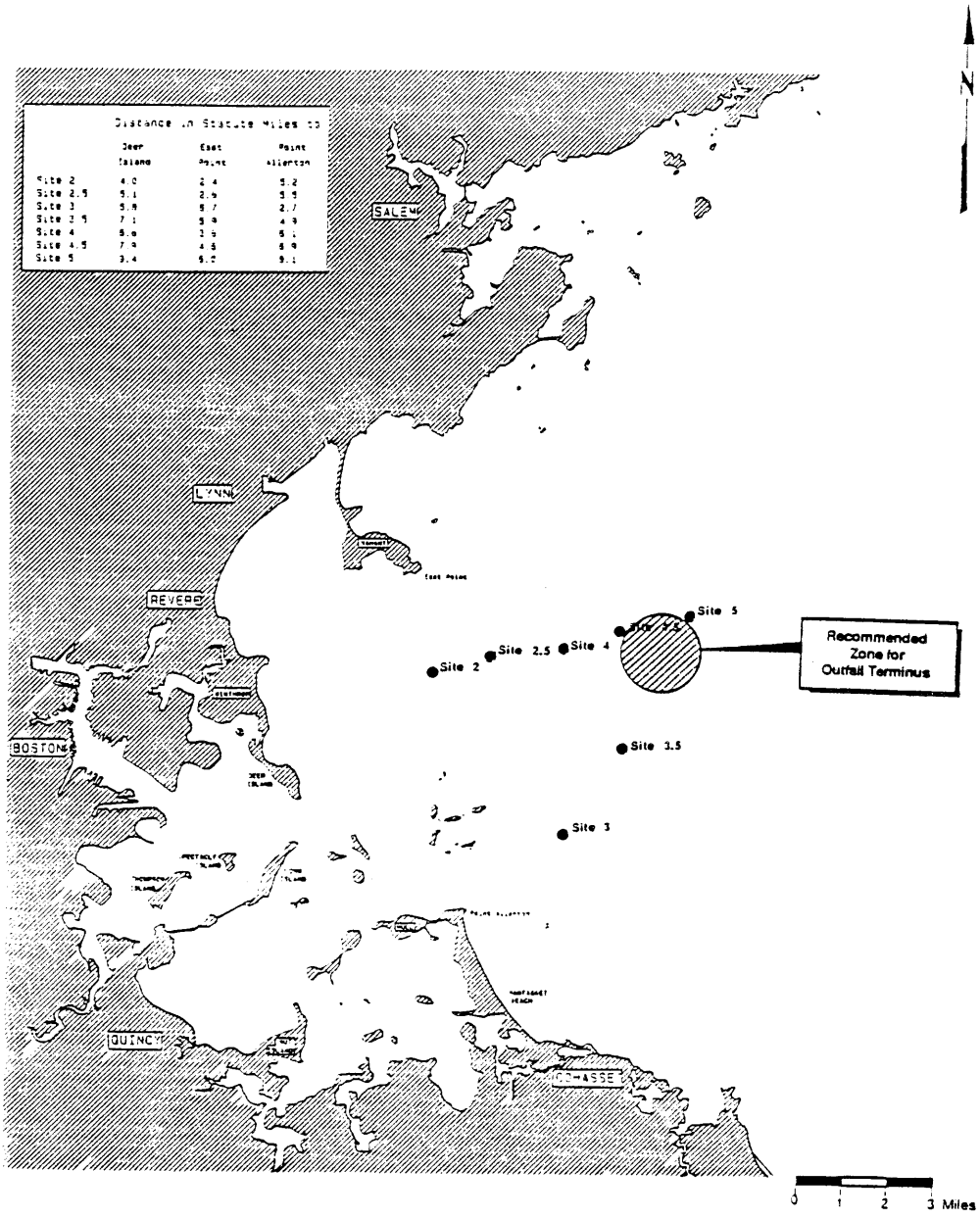


Figure 4-31: Possible Locations for the Outfall (MWRA V-V, 1988)

Table 4-12: Commonwealth of Massachusetts Water Quality Standards for Class SA Waters (MWRA V-III, 1988)

The following minimum criteria are adopted and shall be applicable to all waters of the Commonwealth, unless criteria specified for individual classes are more stringent.

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; 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 Supended 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 exeed the site-specific limits necessary to control accelerated or cultural eutrophication.

Table 4-12: Commonwealth of Massachusetts Water Quality Standards for Class SA Waters (Continued)

7. Other Constituents	Waters shall be free from pollutants alone or in 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.

Additional Criteria

The following additional minimum criteria are applicable to coastal and marine waters for Class SA waters.

Parameter	Criteria
1. Dissolved Oxgen	Shall be a minimum of 85 percent of saturation at water temperatures above 77° C (25° C) and shall be a minimum of 6.0 mg/l at water temperatures of 77° C (25° C) and below.
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 occurring range.
4. 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.

Beyond the initial mixing zone, the intermediate-field processes dominate. These processes are measured over several tidal cycles where tidal dynamics, wind drift, and stratification becomes important. They are important because they provide information on the possible transport of the plume into resource areas. The model used to evaluate the intermediate zone was Massachusetts Institute of Technology's Transient Plume Model (TPM). TPM is a three-dimensional quasi-analytical model that uses velocities obtained from moored current meters to simulate the discharge of a contaminate as a series of puffs so that the diffusion of the effluent plume can be determined (Adams, et al. 1975).

Two circulation patterns exist at the terminus locations. At the near-shore sites, these patterns trend east-west and are strongly influenced by tidal action. The offshore sites exhibit circulation patterns trending north-south, which are strongly influenced by the large-scale circulation of Massachusetts Bay. These differences result in variations in the direction of circulation and average dispersive capacity (largest at offshore sites).

The far-field mixing phase occurs over a much larger area and over a much larger time scale than any of the other phases. In this phase, the effluent will be carried passively by the ambient current. The plume is dispersed slowly due to ambient turbulence and is transported due to large scale circulation patterns in Massachusetts Bay. The model used to evaluate the far-field mixing was Massachusetts Institute of Technology's TEA and ELA. TEA (Tidal Embayment Analysis) is a two-dimensional, vertically averaged, finite element, frequency domain, circulation model that simulates water circulation in embayments in which the circulation pattern is predominantly tidal driven. ELA (Eulerian-Lagrangian Analysis) is a two-dimensional, finite element, mass transport model that uses the results of TEA as input in order to simulate the transport of a contaminate released into the embayment (Kossik, et al., 1986).

Results of these modeling efforts on the Boston Harbor were reported in MWRA Volume V, 1988 as follows:

- "A distinction between candidate sites cannot be drawn based on this criteria (EPA Ambient Water Quality Criteria); each site has a good ability to meet the criteria. No site meets all criteria for primary or secondary treatment. However, all sites have the ability to meet a majority of the criteria. Sites further out require less pollutant reduction to meet the criteria."

- "For each site, Water Quality Standards for temperature and pH will be met for both primary and secondary treatment. For the dissolved oxygen (DO) standard, all sites meet the standard for secondary treatment. For primary treatment, only Site 5 meets the DO standard under all conditions."

- "There is an obvious trend of less (sediment) accumulation as a function of distance offshore, with Site 5 receiving the lowest maximum sedimentation rate and lowest predicted metals accumulation."

- "For both primary and secondary treatment, no floatables will be discharged, therefore no floatables will reach the shorelines."

- "Effluent dilution at each shoreline is the average dilution of the effluent at the nearest land masses. ... Dilution from near-shore sites range from 40 to 120, depending on the site and the nearest shoreline. The offshore sites, particularly Sites 4.5 and 5, allow dilution at shorelines to reach between 139 and 232."

- "For primary and secondary treatment, the offshore sites minimize the amount of viruses at shores to some degree greater than near-shore sites."

Recommended Plan

The recommended plan for the new outfall will be a deep rock tunnel with a multi-port diffuser. Two alternatives are proposed for the outfall's terminus location - Sites 4.5 and 5. Differences in these site locations arise in the overall length of the outfall. Site 4.5 figures are shown because the actual location of the terminus will ultimately be determined by further geotechnical investigations. A conceptual view of the outfall location is shown in Figure 4-32.

The estimated cost of this project will be approximately $400-$500 million (1986 dollars). This cost estimate includes the vertical access shaft, outfall tunnel, lining of the tunnel, disposal of the spoils, drilling the risers for the diffuser, purchasing and installing diffuser heads, and a 35% allowance for engineering and other costs.

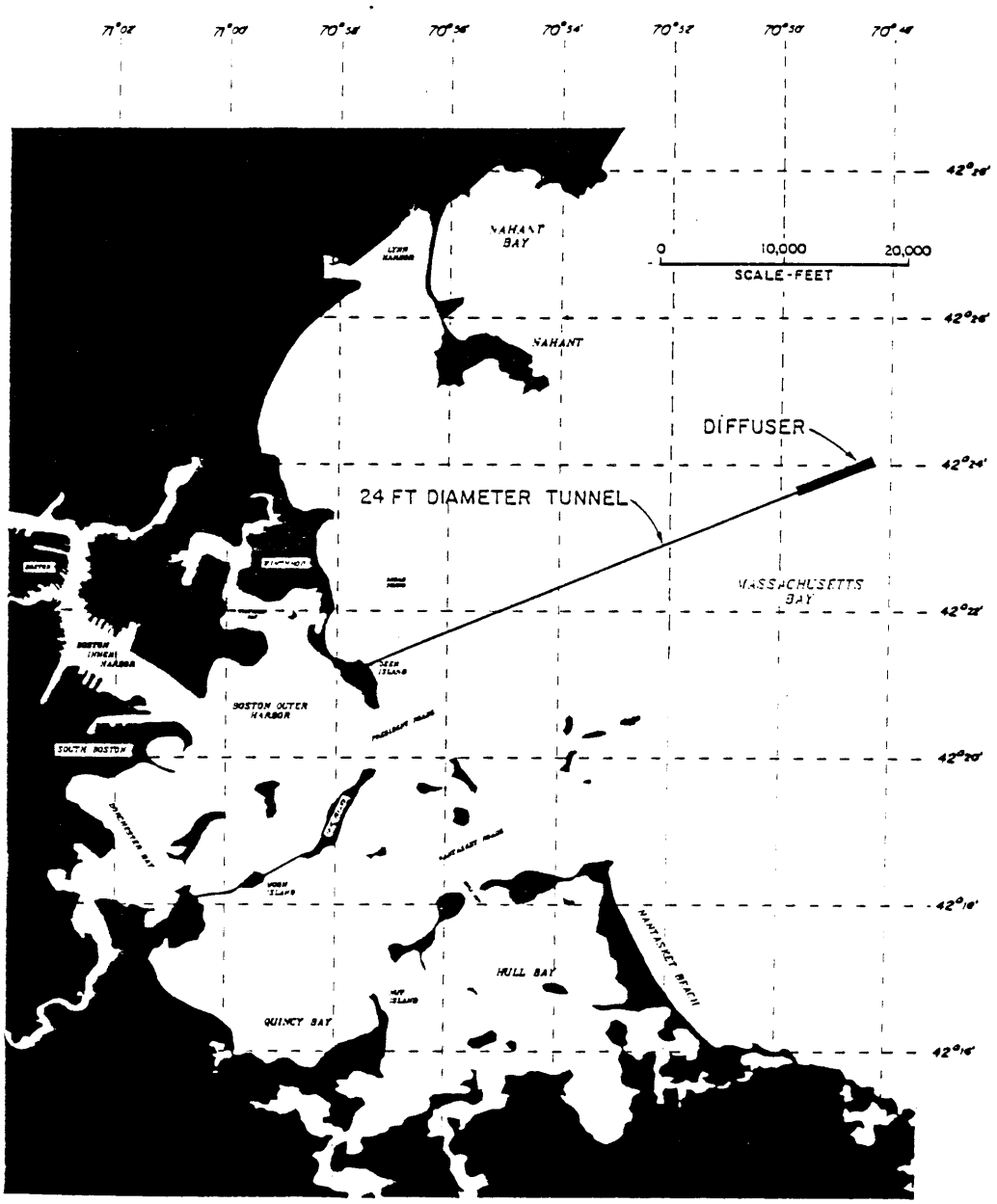


Figure 4-32: Conceptual View of Outfall Location (MWRA V-V, 1988)

Deep Rock Tunnel

The deep rock tunnel (Fig. 4-33) will begin with a vertical access shaft at Deer Island. This shaft will be approximately 9.1 m by 4.6 m and 91 to 122 m below sea level. Starting at the access shaft, a 7.3 to 7.6 m diameter tunnel will be drilled. The tunnel will be 14 to 17 km long with a concrete lining of a minimum thickness of 0.3 m. As a result of this tunneling, approximately 1.8 to 2.5 x 10^6 m^3 of material will need to be disposed of.

The predominate rock that the tunnel will be drilled through is Cambridge Argillite, a medium-hard rock. The tunnel is anticipated to be drilled using a tunnel boring machine, although, if harder rock is encountered, some portions of the tunnel may require conventional drill and blast techniques. The excavation rate for the tunnel boring machine is estimated to be 21 meters per day. The tunnel will be lined with either precast concrete sections or cast-in-place concrete. The lining will be placed at the same time as the excavation.

The concrete liner will be used to minimize friction head losses, hence, decreasing the size requirements of the tunnel. This relationship between the friction factor and the diameter of the tunnel can be shown in the following equation.

$$H_f = f(L/D)(V^2/2g)$$

where:
H_f = friction head loss, m
f = friction factor
L = length of tunnel, m
D = diameter of tunnel, m
V = velocity of flow, m/sec
g = acceleration due to gravity, m/sec^2

The friction factor is a function of the relative roughness of the tunnel and the Reynolds number. If a constant Reynolds number is assumed, the friction factor decreases as the tunnel becomes smoother. By lining the tunnel and making it smoother, we can minimize the head loss while using a smaller diameter tunnel.

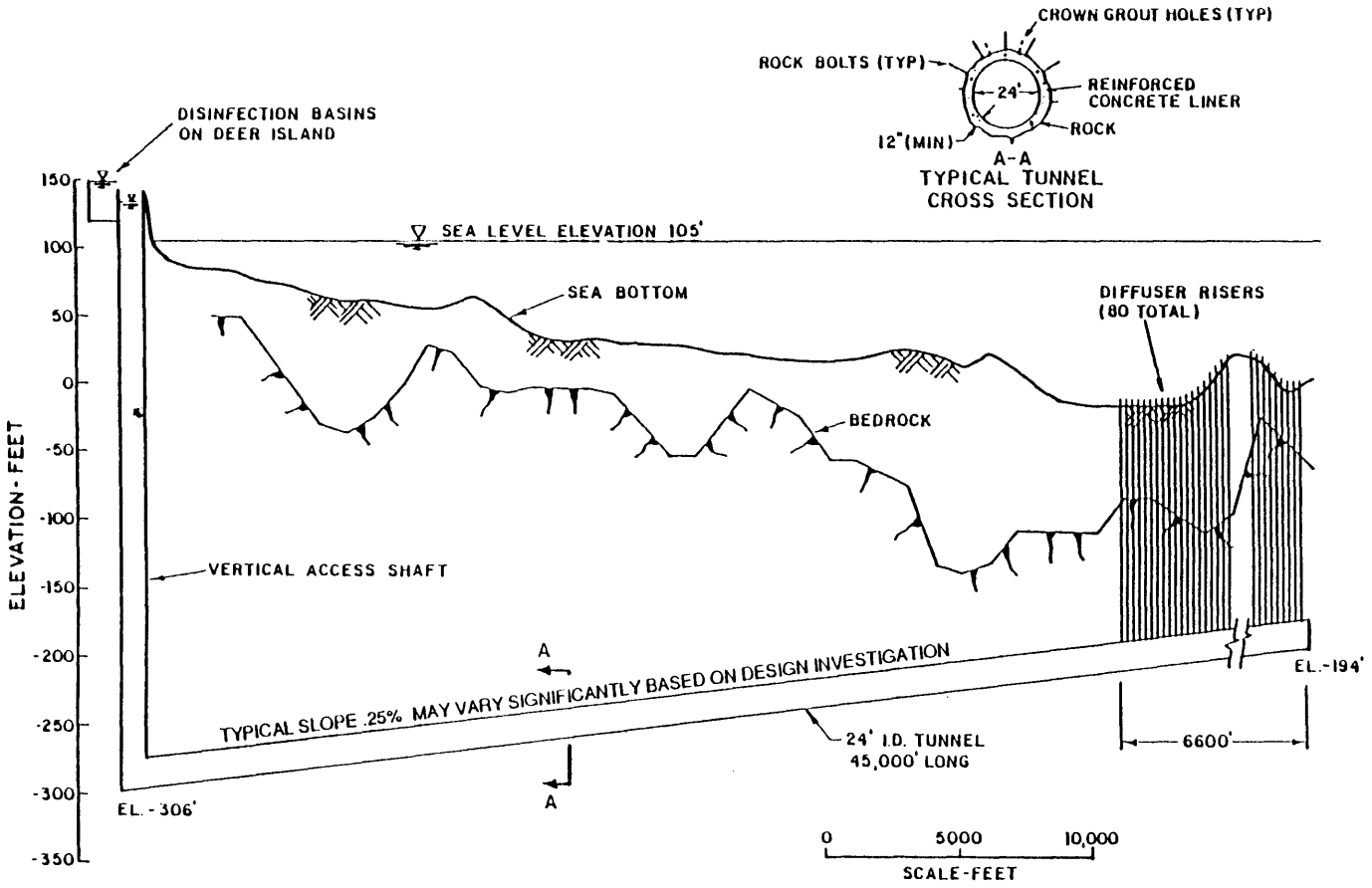


Figure 4-33: Deep Rock Tunnel - Sectional View (MWRA V-V, 1988)

Multi-port Diffuser

The multi-port diffuser is used to increase the mixing of the effluent with the receiving water. It is designed to maximize initial dilution. The parameters associated with the design of the diffuser are as follows:

- length
- orientation
- number and spacing of ports
- riser diameter
- construction technique
- purging of saltwater
- consideration of gravity versus pumped flow

Length

To determine the length of the diffuser, the following was assumed: an initial dilution of 50, an average flow of 480 mgd (21 m^3/sec), and stratified conditions (the plume is submerged) (MWRA Vol V-D, 1987). The initial dilution value is a conservative estimate. Actual values obtained by MWRA and EPA range between 75 and 388 (EPA, 1988). The initial dilution ratio equation provides the resulting length (Roberts and Snyder, 1987):

$$Id = [0.97(g\Delta\rho/\rho)^{2/3}L^{1/3}] / (NQ^{1/3})$$

where:

Id = initial dilution ratio
g = gravitational acceleration, m/sec^2
ρ = ambient density of the seawater, kg/m^3
$\Delta\rho$ = difference between effluent and ambient densities, kg/m^3
L = diffuser length, m
N = Brunt-Vaisala frequency, $(-(g/\rho)(d\rho/dz))^{0.5}$
Q = total discharge, m^3/sec

Under the assumptions that Id = 50, $g\Delta\rho/\rho = 0.265$ m/sec^2, N = 0.036, Q = 21 m^3/sec; therefore, L = 1914 m; or approximately 2000 m. MWRA has decided to use a length of 2000 m for all of its future diffuser calculations.

Orientation

The topography is more likely to dictate the ultimate location and orientation of the outfall and its diffusers, rather than the ambient current direction.

Number and Spacing of Ports

With a diffuser length of 2000 m, the maximum length of spacing required for each diffuser port to obtain an initial dilution of 50, is three meters (MWRA Vol V-D, 1987). This results in a total of 667 ports, which is not feasible. Although, it has been shown that ports can be clustered, up to eight per cluster (Isaacson, et al., 1983). MWRA has decided to use approximately 80 risers, each with a multi-port riser cap (eight risers per cap) to obtain essentially the same dilution as with 667 risers. Each riser will be equally spaced at 25 m apart.

Riser Diameter

The aggregate area of the riser pipes should not exceed the area of the tunnel. Therefore, if an interior tunnel diameter of 6.7 m is assumed, the riser diameter should not exceed 0.75 m. MWRA has decided to size the risers at 90% of this value, resulting in the riser diameter being 0.67 m. The equation used to calculate the riser diameter is as follows:

$$(\pi d_t^2/4) > (\pi d_r^2/4)80$$

where:

d_t = tunnel diameter
d_r = riser diameter

Construction Technique

The diffusers will be located in the last 2000 m of the tunnel. It is proposed that there will be 80 vertical risers, all equally spaced. The inside diameter of the risers will be 0.7 m and will be capped with a 3.0 m diameter multi-port discharge head (Fig. 4-34). It is estimated that the initial dilution will range from 75 to 388 (EPA, 1988). The multi-port diffusers will be flush with the ocean bottom and be at a minimum depth of 29 m.

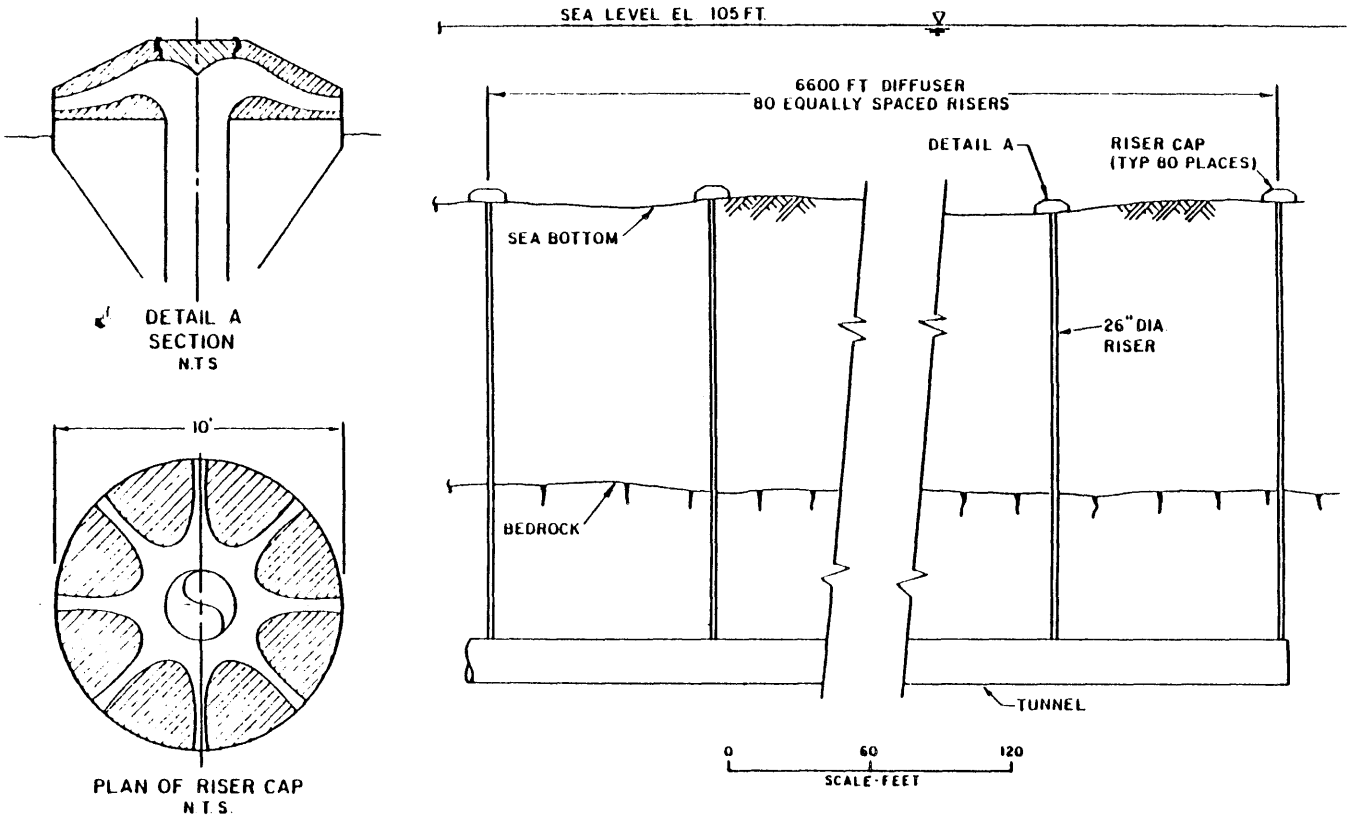


Figure 4-34: Diffuser with Mult-port Risers - Cross Section (MWRA V-V, 1988)

The diffuser risers will be constructed in open waters. Two types of offshore rigs are being considered for the construction: a Jack-up barge and a Semi-submersible drilling platform. Work from the Jack-up barge is performed from a platform above the ocean over each riser site. Therefore, the rig will need to be moved each time a new riser location is drilled. It is estimated that two barges will be needed to drill 80 risers in 30 months.

The Semi-submerged drilling platform is a self-propelled floating platform which is anchored over the site. Work is done above the ocean. It is estimated that one rig will be needed to drill 80 risers in 20 to 30 months.

Purging of Seawater

When the outfall and diffuser are constructed, they will be initially filled with seawater that is approximately 2.7% denser than the wastewater effluent. It is required that all of this seawater be purged from the system to achieve proper operation of the diffusers and to achieve the required dilution of the effluent. If purging is not achieved, inflow of seawater into some of the risers will occur at the same time the remaining risers are discharging effluent.

The minimum flow required to purge the system is Q_P and the minimum flow required to prevent intrusion is Q_I. Q_P is always greater than Q_I with Q_P/Q_I approximately equal to 15 (Brooks, 1988 - see Appendix B1).

There are three design criteria (Brooks, 1988) associated with seawater intrusion and purging of deep-rock tunnel outfalls with a large number of ports. These design criteria are as follows:

- To prevent intrusion, the Froude number must exceed unity when the ports are flowing full.

- To purge the tunnel, the tunnel pressure must exceed the seawater hydrostatic pressure.

- A buoyant front is established in a long sloping tunnel causing a seawater wedge. To expel seawater from a full-flowing tunnel, the hydraulic slope must be greater than $(\Delta r/r)S$, where S is the slope of the tunnel.

Gravity Versus Pumped Flow

The hydraulic profile (Fig. 4-35) is sufficient so that gravity flow is always possible. Problems may result from an excess of potential energy associated with the effluent flow during periods of low tide. Therefore, a mechanism to dissipate the energy before flow enters the tunnel has been recommended.

Presently, there are less than ten tunneled outfalls throughout the world with several either under construction or in the design stage. Several of these tunneled outfalls have not performed up to design expectations due to the effluent discharging through some -not all- of the risers. This incomplete discharge is due to: seawater intrusion in some risers, incomplete purging, and circulation blocking. An incomplete discharge causes a decrease in dilution, increase in head losses, biofouling, and collection of sediments in the tunnel.

Currently, Massachusetts Institute of Technology (Dr. E. E. Adams) is working on a hydraulic model to determine if gravity flow from the treatment plant is sufficient to completely purge the tunnel and risers.

Construction Schedule

The construction sequence for the new outfall will be as follows:

- Construct the vertical access shaft at Deer Island.
- Excavate the deep rock tunnel while removing the spoils through the access tunnels. Diffuser risers will be completed before the tunnel reaches them.
- Line the tunnel with concrete as the tunnel is being constructed.

The outfall is expected to be completed by May 1995. A comprehensive schedule for the outfall construction is given in Table 4-1.

Potential environmental impacts related to construction include habitat removal, increase in turbidity, resuspension of sediment, and disposal of the excavated material. These impacts can be imposed on three separate parts of the outfall: the tunnel, diffusers, and discharge. The environmental impacts caused by the outfall tunnel are essentially none. The impact caused by the diffusers are minor benthic disturbances. The discharge may have minor impacts on marine resources and may cause small amounts of bioaccumulation of toxic material in sediments.

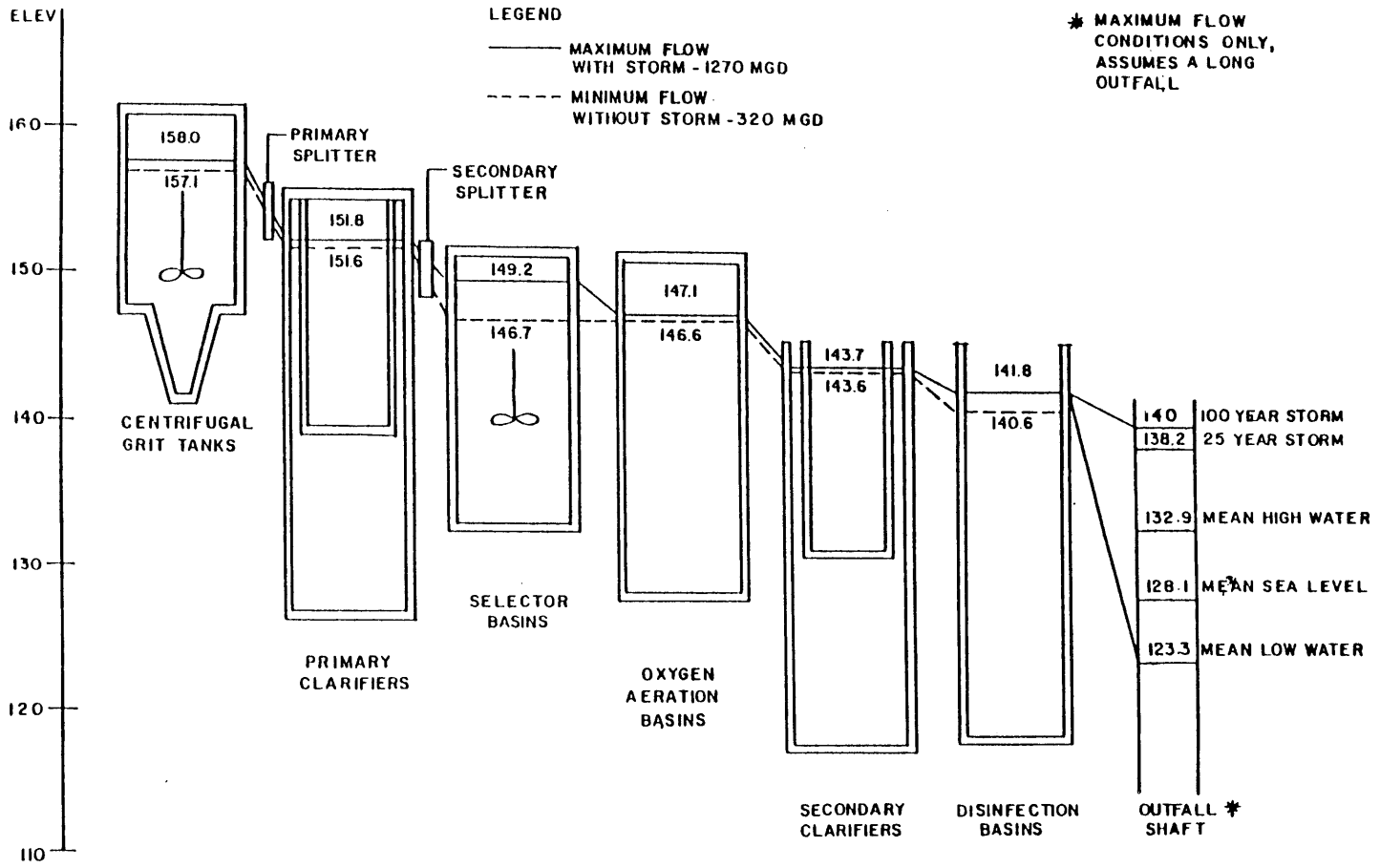


Figure 4-35: Deer Island Hydraulic Profile (MWRA V-III, 1988)

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United States Environmental Protection Agency (EPA). April 1988. Boston Harbor Wastewater Conveyance System, Volume II, Draft Supplemental Environmental Impact Statement Appendices.

The remainder of the material for this section was extracted from the following documents:

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Executive Summary, Final Report.

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Volume V: Effluent Outfall, Final Report.

Massachusetts Water Resources Authority (MWRA). April 1988. Secondary Treatment Facilities Plan, Volume V, Appendix A: Physical Oceanographic Investigations, Final Report.

Chapter 5

MWRA's Residual Management Plan

MWRA Residual Management Plan

Introduction

The two primary treatment plants on Deer and Nut Islands (Fig. 5-1) treat the raw wastewater by removing biochemical oxygen demand (BOD) and total suspended solids (TSS). The material removed from the wastewater is referred to as residuals. The Massachusetts Water Resource Authority (MWRA) defines residuals as "Substances separated from the wastewater during treatment. Residuals include sludge and minor residuals." Sludge consists of the organic solid material that settles out during primary and secondary treatment. Minor residuals are the substances, other than sludge, that are removed from the wastewater. Minor residuals include grit, screenings, and scum.

Up until December 1988, effluent, digested sludge, and digested scum had been discharged into the ocean through outfalls near Deer Island (Fig. 5-2). As of December 1988, the scum is being chemically fixed by mixing it with cement kiln (only at Deer Island) in order to bind the scum. This material is then used as a cover for landfills or is disposed of on land. Today, grit and screenings are hauled by truck and disposed of in Buffalo, New York at a cost of $2 million per year.

Currently, MWRA and the New York / New Jersey metropolitan area are the only communities in the U.S. that still dispose of sludge by discharging it into the ocean. Regulations established in 1988 prohibit ocean discharging through outfalls. MWRA sludge disposal through an outfall is not expected to stop until December 1991 when a new sludge processing facility is built.

Background

In 1977 the Federal Water Pollution Control Act (FWPCA) was amended and became known as the Clean Water Act (CWA). Under this act, a provision, section 301(h), was established allowing the Environmental Protection Agency (EPA) to waive the secondary treatment plant requirements for publicly owned treatment works (POTWs) discharging into marine waters. The Metropolitan District Commission (MDC) applied for a waiver and did not receive it. The events that lead up the the tentative denial of the waiver application can be found in Chapter 3 of this document. A few specific points are mentioned.

Figure 5-1: MWRA Service Area

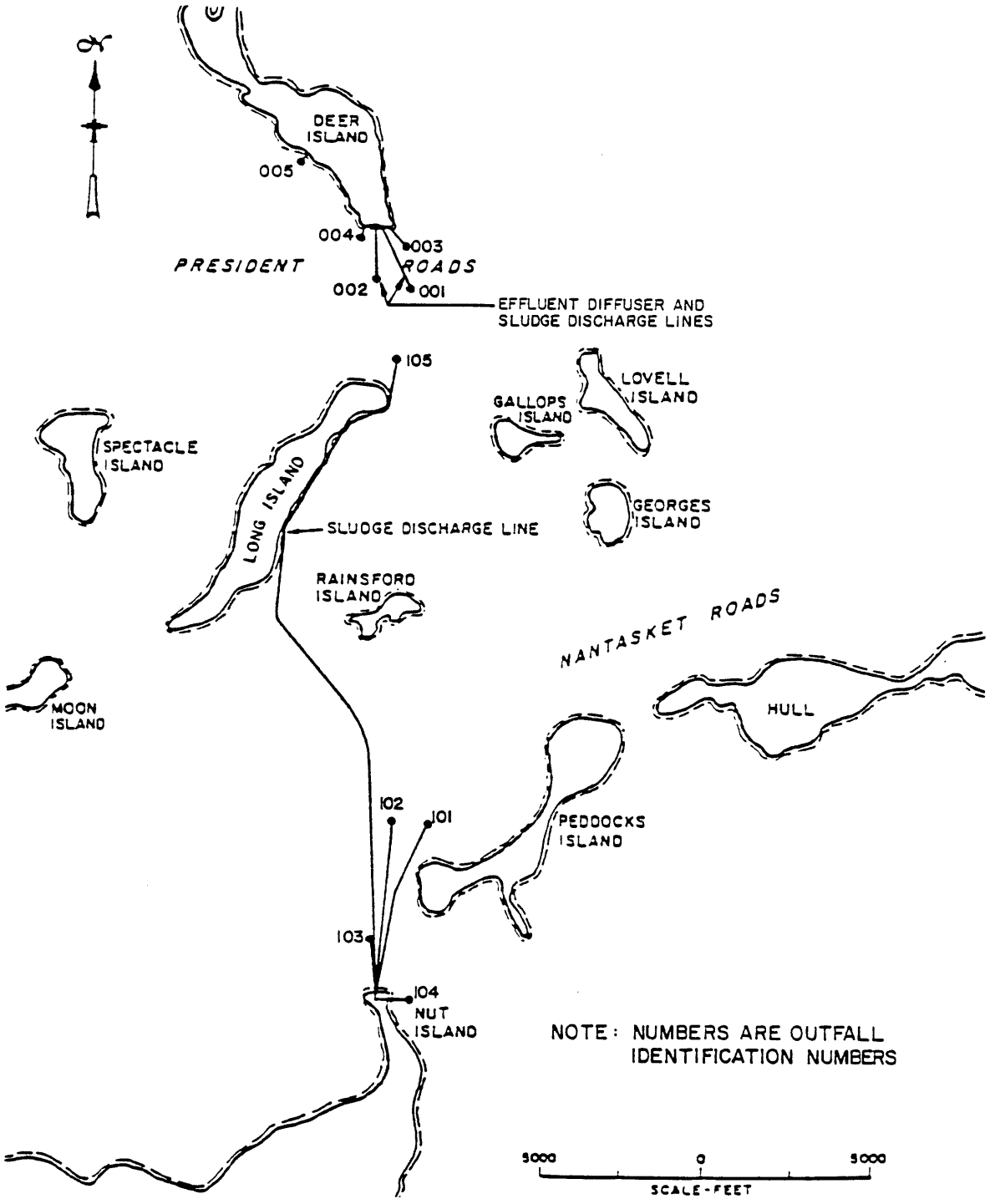

Figure 5-2: Location of MWRA's sludge and effluent outfalls (MWRA, 1988)

In December 1982, the City of Quincy sued the MDC for polluting the harbor. During court hearings, it was determined that MDC was unable to effectively solve the pollution problems associated with Boston Harbor. On January 1, 1985, the courts created the MWRA and charged it with the responsibility of providing wholesale water supply services and sewage interception, treatment, and disposal services.

Shortly after the creation of MWRA, EPA filed suit against MWRA for violation of the CWA, stating that a major source of pollution in the harbor was the discharge of sludge.

As a result of the suit filed by EPA, the courts have established a schedule for "cleaning up" Boston Harbor. This cleanup includes the construction of a secondary treatment plant and a land-based sludge disposal site. The schedule for a land-based sludge disposal site is summarized below :

- December 1991 is the latest allowable date for MWRA to commence land-based sludge disposal. This is the start-up date for the interim residual management facility.

- December 1995 is the anticipated start-up date of the long-term residual management facility. This is also the date of the complete consolidation of treatment at Deer Island.

- December 1999 is the start-up date of the secondary treatment facility on Deer Island.

- December 2020 is the end of the Residual Management Facility Plan (RMFP) planning period.

As a result of the court order mandating the MWRA to halt all ocean disposal of sludge by December 1991, MWRA began planning for land-based sludge disposal. The plan is known as the Residual Management Facility Plan (RMFP). The primary agencies in the siting process and their roles are shown in Table 5-1. A list of major events that have and will take place are given in Table 5-2.

Table 5-1: Agencies and Roles Related to the Residual Management Facility Siting Process.

Agencies	Roles
MWRA	The agency trying to site the Residual Management facility.
MEPA	A unit of the Executive Office of Environmental Affairs (EOEA) that set-up the Special Procedure for conducting the siting process.
EPA	Sets national standards for hazardous and non-hazardous wastes. Ensures that a site is environmentally safe.
DEQE	Sets standards for hazardous and non-hazardous waste within the state of Massachusetts. Oversees MWRA's operations.
U.S. District Court	Mandated the schedule for "cleaning up" Boston Harbor.
City of Quincy	Site for the short-term and long-term sludge processing facilities.
Town of Walpole	Site for the minor residuals landfill.
Residents of MWRA service area	It is their waste that will be disposed of and processed. It is also their money that will finance the venture.

Table 5-2: Chronology of the Residual Management Facility Plan

- In 1952, Nut Island Primary Treatment Plant was constructed and today services the southern portion of the MWRA service area (Fig. 5-1).

- In 1968, Deer Island Primary Treatment Plant was constructed and today services the northern portion of the MWRA service area (Fig. 5-1).

- In 1972, the federal government mandated secondary treatment.

- In 1977, the federal government provided for coastal communities to have less than secondary treatment under certain conditions.

- In the fall of 1979, the Metropolitan District Commission (MDC) applied for a waiver from secondary treatment.

- In December 1982, the City of Quincy filed a suit against MWRA's predecessor MDC for violations of the CWA.

- In January 1985, EPA filed a suit against MWRA for violations of the CWA.

- By December 1991, a court order mandates that ocean disposal of sludge must be stopped.

- In 1985, MWRA begins their Residual Management Facility Plan (RMFP) as part of a court order to "cleanup" Boston Harbor.

- In 1986, Phase I of the RMFP is completed. This report determined projected sludge quantities for the design year 2020 and established 200 sites as potential candidates for facilities siting.

- In 1986, MWRA made a request to the EOEA that Phase II of the RMFP be designated as "Major and Complicated". With the RMFP being designated as "Major and Complicated", MWRA was able to perform an environmental review together with the facility planning process.

- In February 1986, EOEA determined that the RMFP was indeed "Major and Complicated" and established a "Special Procedure" for completing the project. This Special Procedure included breaking the overall task of determining a viable site for the disposal or reuse of the residuals into four separate tasks.

    - Residual Characterization
    - Technological Assessment
    - Transportation Assessment
    - Site Assessment

  The results of these tasks were combined into one task - the Candidate Options Alternative task.

- In February 1987, as a result of the Candidate Option Alternatives report, a list of 12 sites, three technologies, and two waste types were considered for further evaluation.

- In August 1987, the above tasks were completed and MWRA purchased Fore River Staging Area (FRSA) in Quincy (Fig. 5-1).

- MWRA chose FRSA as the site for their short-term sludge processing facility. The short-term facility is planned to operate between 1991 and 1995.

- In August 1987, the City of Quincy sued MWRA for choosing them as the site for the short-term sludge processing facility.

- In January 1988, Quincy agrees to host the short-term sludge processing facility and drop the lawsuit against the MWRA in exchange for certain commitments and conditions. A detailed explanation of these conditions can be found in the Memorandum of Understanding (MWRA, January,1988).

- In September 1988, MWRA completed its review of the candidates and narrowed the siting list down to two landfills for minor residuals and three sites for sludge processing facilities.

- In January 1989, MWRA chose Walpole as their preferred site for the minor residuals landfill.

- Walpole residents protest the decision and 600 residents form outside the statehouse and MWRA headquarters. Walpole also questions MWRA siting procedures and begins to investigate the environmental impacts associated with the landfill site.

- In February 1989, MWRA chooses FRSA as their preferred site for the sludge processing facilities.

- Quincy sues MWRA, stating that the reason that they where chosen as the long-term sludge processing site was because they agreed to host the short-term sludge processing facilities.

- MWRA's report on the Draft Facility Plan / Environmental Impact Report (DFP) is released for review by the public and the Massachusetts Environmental Policy Act (MEPA) unit of EOEA.

- In August 1989, MWRA will release its Final Facilities Plan (FFP) based on the comments made by the public and MEPA.

- In September 1989, the FFP report must be certified by EOEA. If this happens, the MWRA Board of Directors will approve the certified FFP by November 1989.

Need

The need for one or more facilities to process residuals has been well established and agreed upon by all the major players. The need is based on a court order and a desire to clean up the harbor. The court order is the driving force behind the schedule for siting the Residual Management Facility (RMF) as well as all other aspects of the harbor cleanup.

Public Participation

The MWRA developed an extensive public participation program that included:

- regional task forces comprised of members of each community involved in the siting process

- a Facility Planning Citizen Advisory Committee (FPCAC) that reviewed technical reports produced by MWRA

- approximately 180 meetings related to the siting process

The reason for this public participation was to inform the public of the project's progress and to allow the public to ask questions about the project.

Residual Characterization

The purpose of characterizing the residuals is to provide information on residual quality and quantities. Below is a summary list of findings by Black & Veatch, the main consultants to MWRA on the RMFP project. This summary was extracted from the Draft Report on the Characterization of Residuals, February 1987 and January 1988.

Table 5-3: Residual Characterization Summary of Findings

- Influent loadings of TSS and BOD_5 will increase slightly from 1995 to 2020 (Fig. 5-3) due to population growth and initiation of CSO treatment facilities.

- Raw primary, raw secondary, digested primary, and digested combined sludge production is expected to increase slightly from 1995 to 2020 (Fig. 5-4, 5).

- Design year (2020) annual average sludge production is as follows:

Raw primary sludge	154 dry tonnes per day
Raw secondary sludge	141 dry tonnes per day
Raw combined sludge	295 dry tonnes per day
Digested primary sludge	86 dry tonnes per day
Digested secondary sludge	163 dry tonnes per day

For details see Table 5-4

- Physical properties of the annual average sludge production in the design year 2020 is as follows:

	Raw Sludge			Digested Sludge	
	Primary	Secondary	Combined	Primary	Combined
Volatile Fraction	75	78	76	56	58
Non Volatile	25	22	24	44	42

- Influent concentrations of metals and toxic organic pollutants will not increase over the design period due to industrial pretreatment monitoring and treatment technology improvements.

- Projected quantity of minor residuals for design year 2020:

Grit	35 to 39 cubic meters per day
Screenings	19 to 22 cubic meters per day
Scum	3.4 dry tonnes per day

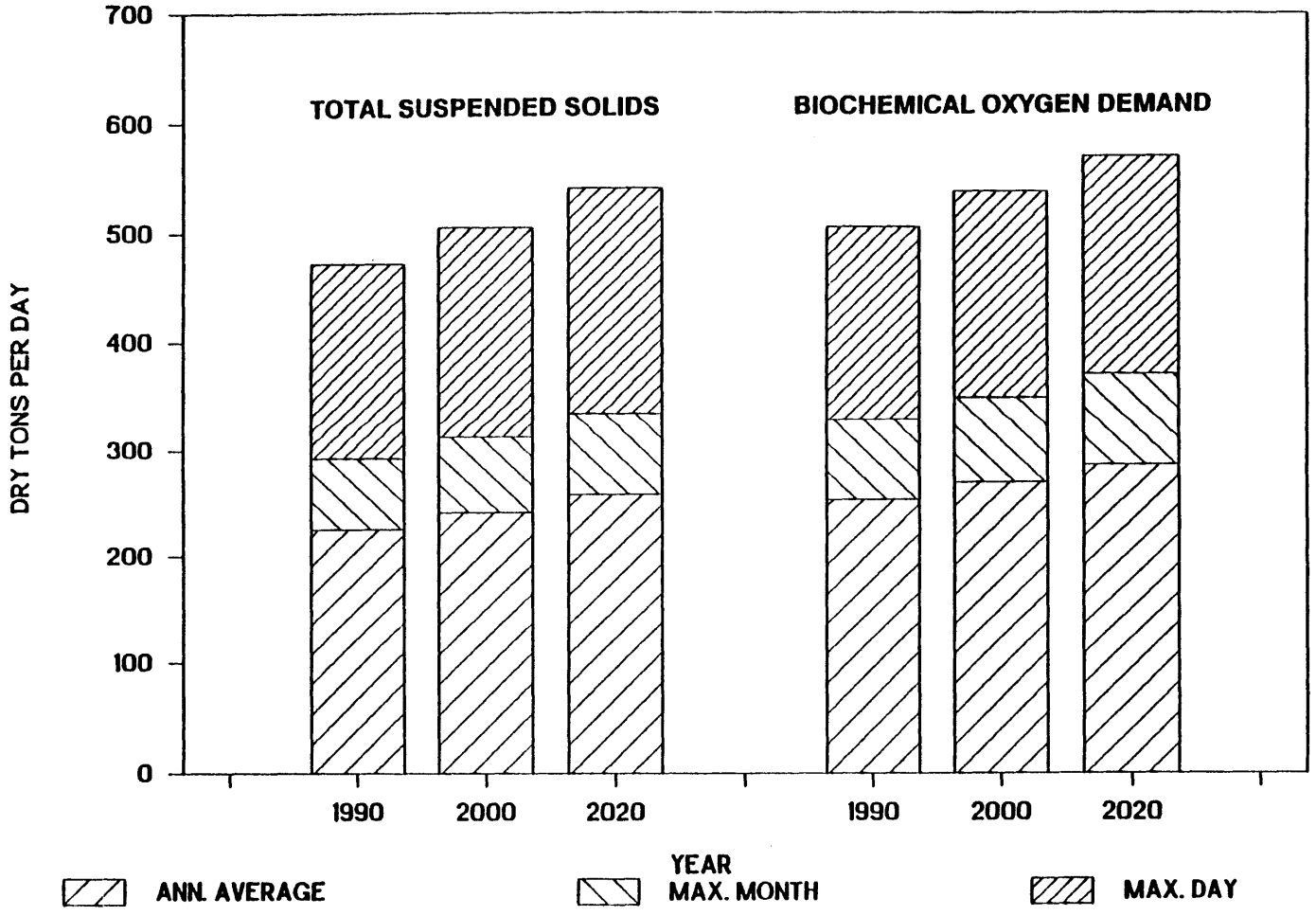

Figure 5-3: Projected TSS and BOD Loadings in Dry Tons (short) per Day (Black & Veatch Inc., January 1988)

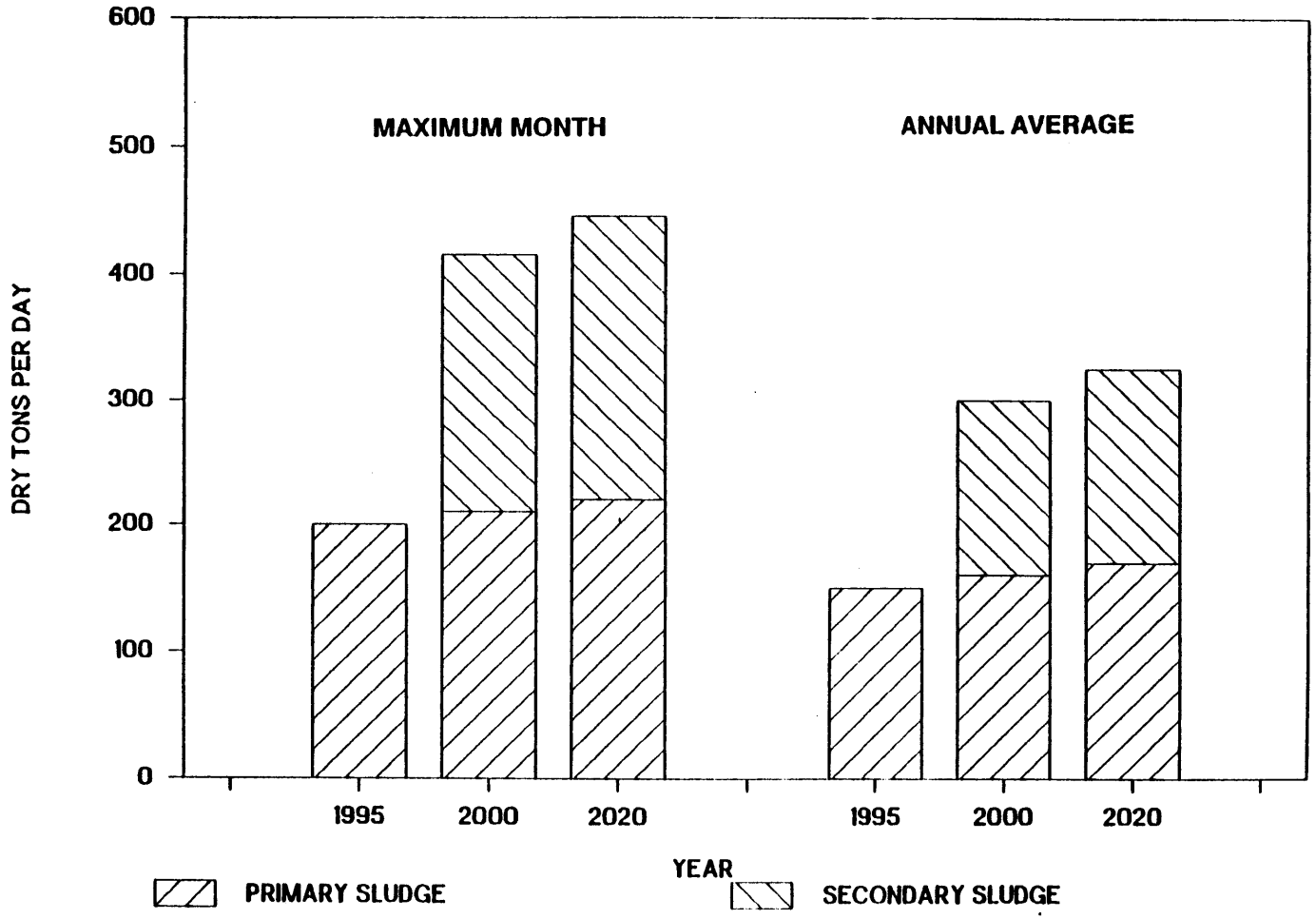

Figure 5-4: Projected Raw Sludge Production in Dry Tons (short) per Day (Black & Veatch Inc., January 1988)

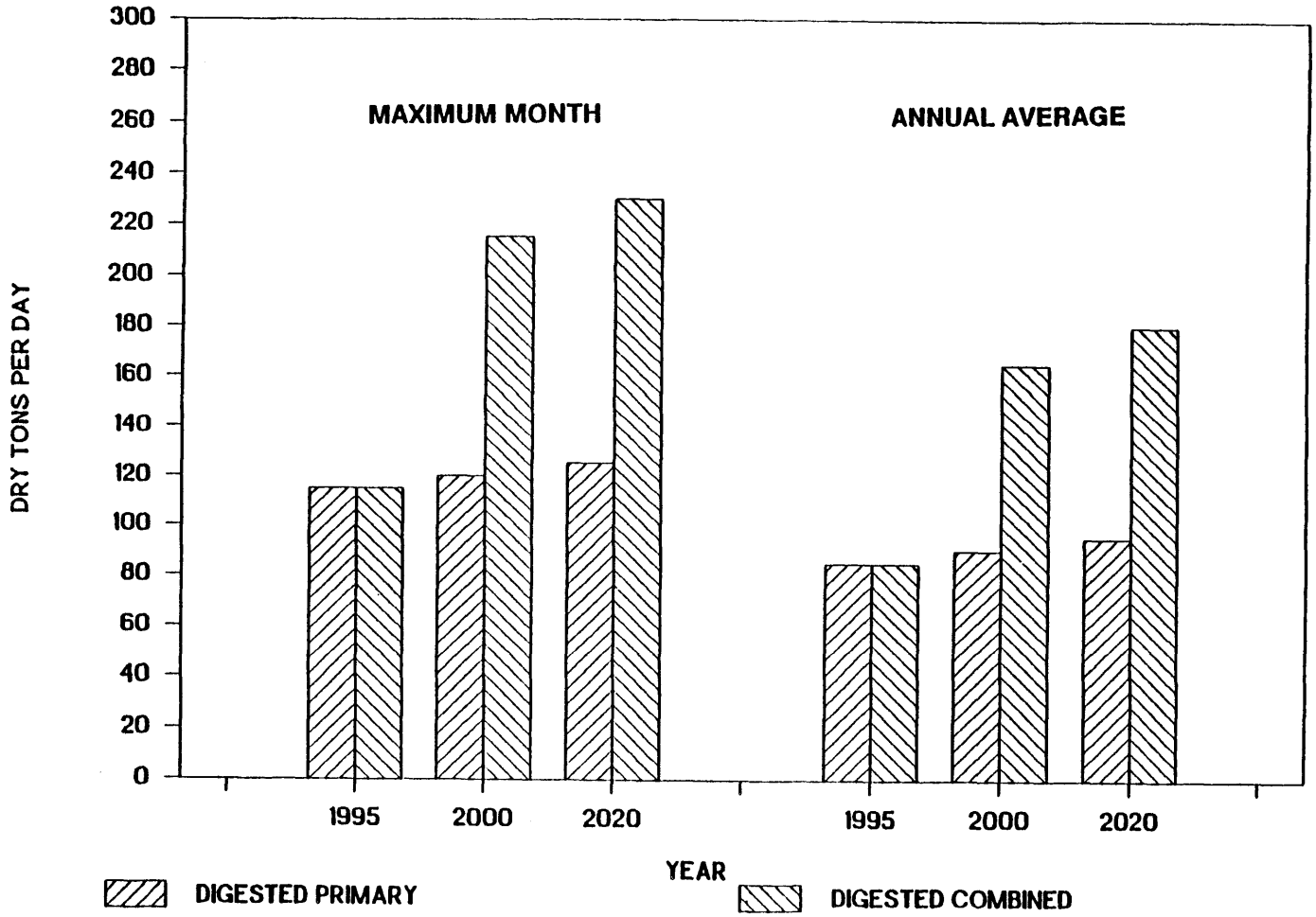

Figure 5-5: Projected Digested Sludge Production in Dry Tons (short) per Day (Black & Veatch Inc., January 1988)

Table 5-4: Projected Sludge Quantities in Dry Tons (short) per Day (Black & Veatch Inc., January 1988)

Item	Year 1995 (dtpd)	2000 (dtpd)	2020 (dtpd)
Raw Primary Sludge			
annual average	150	160	170
maximum month	200	210	220
peak day **	-	-	120
peak storm day **	-	-	115
Raw Secondary Sludge			
annual average	N/A	140	155
maximum month	N/A	205	225
peak day **	-	-	125
peak storm day **	-	-	180
Raw Combined Sludge			
annual average	N/A	300	325
maximum month	N/A	415	445
peak day **	-	-	245
peak storm day **	-	-	295
Digested Primary Sludge*			
annual average	85	90	95
maximum month	115	120	125
Digested Combined Sludge			
annual average	N/A	165	180
maximum month	N/A	215	230

N/A Denotes not applicable. Secondary sludge will not be generated until 1999.

* Figures presented assume all the primary sludge undergoes digestion.

** Peak day represents the increase in solids generated above maximum month quantities.

- Projected dry weight concentrations of the sludge are as follows:

Nitrogen	2.5 to 4.0%
Phosphorous	1.0 to 3.0%
Potassium	0.1 to 0.4%

These concentrations are typical of other wastewater sludge. The typical values found in most commercial fertilizers is approximately 10% of each constituent.

- Chloride content of the existing sludge and wastewater in the North system is high; although, it will not cause a problem in the agricultural or horticultural use of the sludge or sludge-based products. After tide gate rehabilitation has been completed, it is expected that chloride levels in the North system during dry weather periods will decrease to the levels typical of the South system.

- Influent concentration is not impacted significantly by industrial contributions. MWRA's influent is more typical of a municipality that has a small industrial component with respect to priority toxic pollutants (Table 5-5).

- Limited amounts of volatile organic priority pollutants are anticipated to partition to the primary or secondary sludges because the principal removal mechanisms are biodegradation and air stripping - not sorption to the sludge.

- No significant amount of pesticides or polychlorinated biphenyls (PCBs) are anticipated to be present in any of the residual waste streams from the MWRA treatment facilities. However, sporadic detection has occurred indicating that the source might be due to illegal dumping rather than from a continuous source.

- Metals and cyanide are projected to be generally higher in secondary than in primary sludge due to the expected greater removal efficiency of secondary treatment (Table 5-6).

Table 5-5: Comparision of MWRA Influent Concentration to Other POTW Influents (Black & Veatch Inc., January 1988)

		Commercial/Residential Sources		POTWs With Industrial Contributions	
Constituent	Projected * MWRA Influent (ug/l)	Typical Res/Com. Source (ug/l)	Less Than 4% Industrial Flow (ug/l)	40 POTW Study (ug/l)	Greater Than 4% Industrial Flow (ug/l)
cadmium	2.1	3	32	38	30
chromium	22	50	75	173	476
copper	100	63	151	226	489
lead	17	64	75	101	161
mercury	1.2	< 1	2	< 1	26
nickel	20	21	85	120	319
silver	4.5	4	7	9	29
zinc	220	198	417	723	640

* - Figures based on design year 2020 annual average flows and loads.

Table 5-6: Summary of Projected Raw Sludge Metals Quality (Black & Veatch Inc., January 1988)

Constituent	Raw Primary (mg/kg)	Raw Secondary (mg/kg)	Raw Combined (mg/kg)
antimony	14	16	15
arsenic	5.6	6.1	5.8
beryllium*	< 9	< 10	< 9
boron	92	150	120
cadmium	3.7	9.5	6.5
chromium	100	100	100
copper	410	610	500
lead	94	25	61
mercury	3.2	8.5	5.8
molybdenum	6.3	28	16
nickel	35	43	39
selenium	16	69	41
silver	16	35	25
thallium*	< 14	< 15	< 14
zinc	1,000	1,000	1,000
cyanide	33	180	100

All concentrations on dry weight basis.

* Projected concentrations were determined by multiplying the detection limits in the influent concentrations by the appropriate factors to obtain an upper limit for the sludge.

- Metal concentrations on a dry weight basis were calculated for composting (Table 5-7), combustion (Table 5-8), and digestion (Table 5-9). Because metals are conserved throughout the treatment processes, the concentration of the metals in the residuals is proportional to the reduction of solids in the effluent. Therefore, metals concentration in ash are greater than in raw sludge.

- DEQE and EPA have set a number of guidelines and regulations on the metal concentrations for land disposal of residuals (Table 5-10). DEQE regulations are divided into three categories: Type I, Type II, and Type III material with Type I material being the most desirable.

- The projected metal concentrations for primary and combined compost from composting of undigested sludge are approximately equal to the respective raw sludge concentrations. Both the primary-only compost and combined compost from undigested sludges are categorized as DEQE Type III materials (Fig. 5-6 and 5-7); however, the primary compost is of higher quality.

- Anaerobic digestion effectively reduces the volatile solids content of the raw sludge. This reduction in solids results in a proportional increase in the metals content of the digested sludge. For the primary-only digested sludge, DEQE regulations for a Type I material are exceeded by the projected cadmium, copper, zinc, and molybdenum concentrations. DEQE regulations for Type II material are exceeded by the projected zinc, copper, and molybdenum concentrations. For the combined digested sludge, cadmium and boron also exceed the DEQE regulation for a Type II material.

- The metals concentrations for the digested primary and digested combined compost are projected to be approximately equal to or slightly less than the respective digested sludge concentrations. The actual compost metals concentration will depend on the composting process incorporated, the extent of digestion, and the amount of compost amendment blended with the sludge. The primary-only digested compost is projected to be Type II or Type III material. The combined digested compost, which is similarly categorized as a Type II or Type III material, has higher concentrations of cadmium, copper, zinc, boron, and molybdenum. These high concentrations limit usability.

Table 5-7: Projected Ranges of Compost Metals Quality (Black & Veatch Inc., 1988)

Constituent	Reactor System			Aerated Static Pile	
	Raw Primary (mg/kg)	Digested Primary (mg/kg)	Digested Combined (mg/kg)	Digested Primary (mg/kg)	Digested Combined (mg/kg)
antimony	7.5 - 23	7.4 - 22	7.9 - 25	7.4 - 21	7.9 - 22
arsenic	2.9 - 8.9	2.9 - 8.7	3.1 - 9.9	2.9 - 8.2	3.1 - 8.8
beryllium	< 5 - < 14	< 5 - < 14	< 5 - < 15	< 5 - < 13	< 5 - < 13
boron	48 - 150	48 - 140	64 - 210	48 - 130	64 - 190
cadmium	1.9 - 5.9	1.9 - 5.8	3.4 - 11	1.9 - 5.5	3.4 - 9.8
chromium	55 - 170	54 - 160	55 - 180	54 - 150	55 - 160
copper	220 - 660	210 - 640	270 - 860	210 - 610	270 - 760
lead	49 - 150	49 - 150	32 - 100	49 - 140	32 - 89
mercury	1.7 - 5.2	1.7 - 5.0	3.0 - 9.8	1.7 - 4.7	3.0 - 8.7
molybdenum	3.3 - 10	3.3 - 9.8	8.6 - 28	3.3 - 9.3	8.6 - 25
nickel	18 - 56	18 - 54	21 - 66	18 - 51	21 - 59
selenium	8.2 - 25	8.1 - 24	22 - 70	8.1 - 23	22 - 62
silver	8.3 - 25	8.3 - 25	13 - 42	8.3 - 24	13 - 37
thallium	< 7 - < 22	< 7 - < 22	< 8 - < 24	< 7 - < 21	< 8 - < 21
zinc	530 - 1,600	530 - 1,600	530 - 1,700	530 - 1,500	530 - 1,500
cyanide	17 - 53	17 - 51	55 - 180	17 - 48	55 - 160

All concentrations given on dry weight basis.

Table 5-8: Projected Ash Metals Quality (Black & Veatch Inc., 1988)

Constituent	Sludge	
	Secondary[1] (mg/kg)	Digested Combined (mg/kg)
antimony	67	63
arsenic	26	25
beryllium	< 41	< 38
boron	600	510
cadmium	35	27
chromium	450	440
copper	2,500	2,100
lead	190	260
mercury	32	24
molybdenum	97	70
nickel	180	170
selenium	240	170
silver	130	110
thallium	< 63	< 60
zinc	4,400	4,300

All concentrations given on dry weight basis.

(1) Actual composition of feed sludge would be expected to be 75 percent secondary sludge and 25 percent primary.

Table 5-9: Projected Digested Sludge Metals Quality (Black & Veatch Inc., 1988)

	Projected Design Year 2020	
Constituent	Digested Primary (mg/kg)	Digested Combined (mg/kg)
antimony	26	27
arsenic	10	11
beryllium	< 16	< 16
boron	170	220
cadmium	7	12
chromium	190	190
copper	740	910
lead	170	110
mercury	6	11
molybdenum	11	30
nickel	62	70
selenium	28	74
silver	28	45
thallium	< 25	< 26
zinc	1,800	1,800
cyanide	59	190

All concentrations given on dry weight basis.

Table 5-10: Massachusetts DEQE and EPA Guidance Criteria for Land Disposal of Residuals (Black & Veatch Inc., 1988)

Constituent	US EPA High Quality Criteria (mg/kg)	DEQE Regulations Type I (mg/kg)	Type II (mg/kg)	Type III (mg/kg)
Metals				
cadmium	< 25	< 2	< 25	> 25
lead	< 1,000	< 300	< 1,000	> 1,000
nickel	NA	< 200	< 200	> 200
zinc	NA	< 2,500	< 2,500	> 2,500
copper	NA	< 1,000	< 1,000	> 1,000
chromium	NA	< 1,000	< 1,000	> 1,000
mercury	NA	< 10	< 10	> 10
molybdenum	NA	< 10	< 10	> 10
boron	NA	< 300	< 300	> 300
Organics				
PCBs	< 10	< 2 *	< 10	> 10

All concentrations given on dry weight basis.

* 2 for fertilizer or use on pasture land; 1 for soil conditioner.

NA - Not applicable

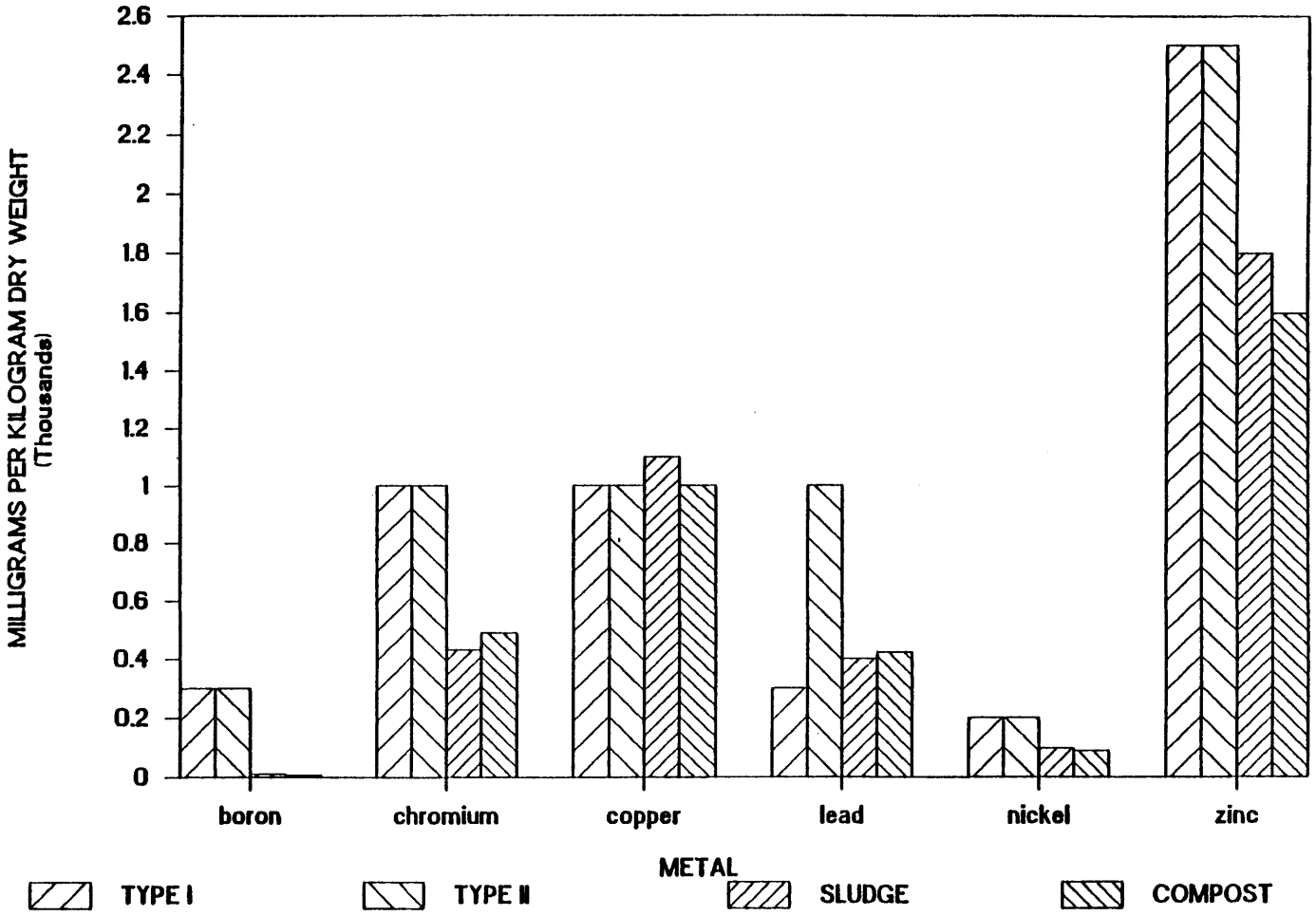

Figure 5-6: Composting Program Regulatory Evaluation (a) (Black & Veatch Inc., January 1988)

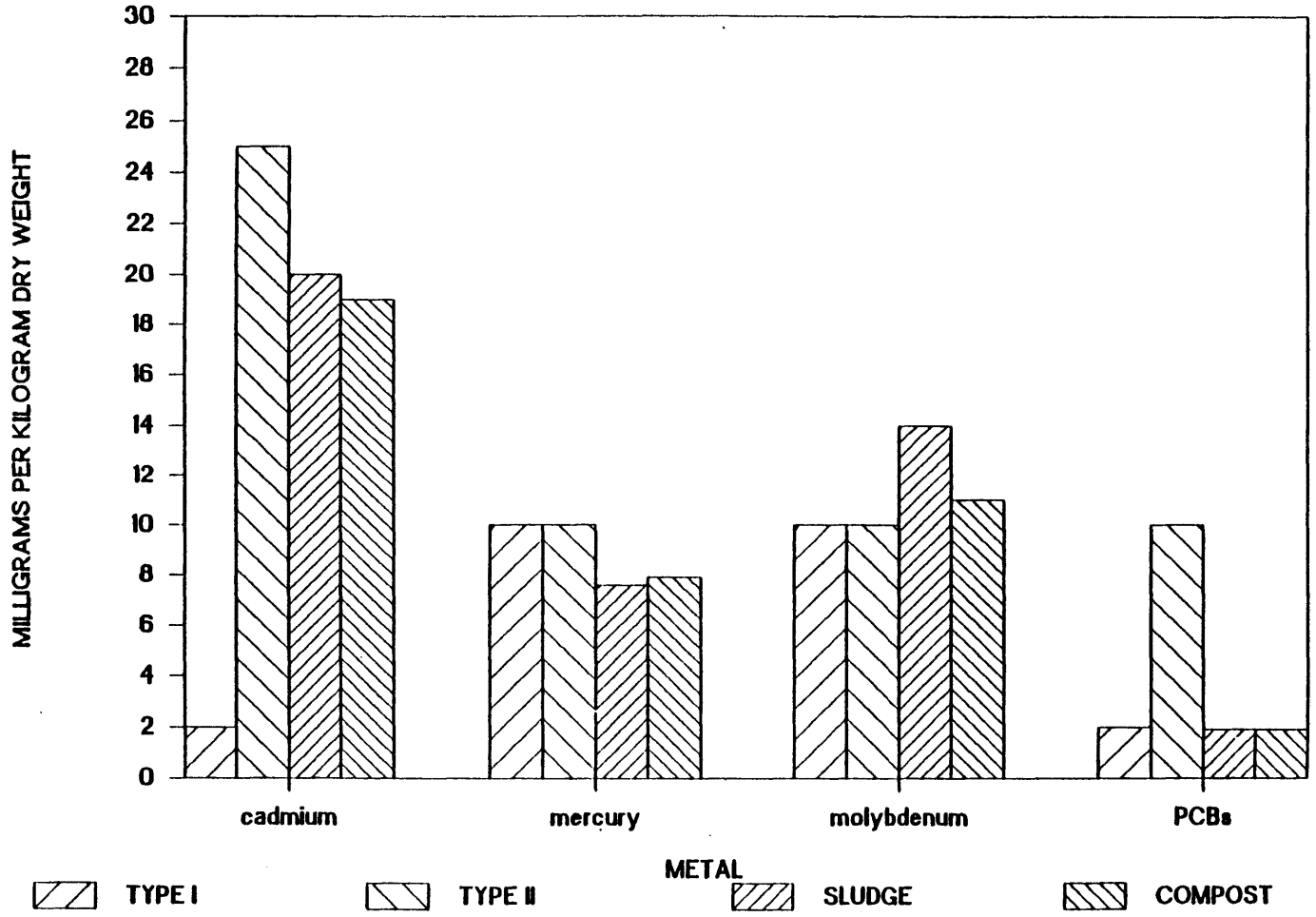

Figure 5-7: Composting Program Regulatory Evaluation (b) (Black & Veatch Inc., 1988)

- An additional monitoring procedure to assess the toxicity of solid wastes is the Toxicity Characteristics Leaching Procedure (TCLP). This method evolved as an update to the extraction procedure toxicity test in an attempt to characterize the leachability of toxic organics, particularly volatile organics, from residuals. Although no formal regulations specific to municipal sludge have been developed for TCLP, dewatered primary sludge from Deer Island was analyzed using this method. The results showed no positive occurrence for any of the organic priority pollutants.

- Currently, the differences in the primary and secondary sludge warrant the consideration of separate treatment of the sludges.

- Currently, there are not sufficient differences in the North and South system sludge to warrant the consideration of separate treatment of North and South system wastewaters.

- The MWRA currently has an industrial monitoring program for characterizing the waste discharges to the sewerage system from each industry and an established database on potential constituents from each industry. This program allows MWRA the opportunity to implement an ongoing sampling and analysis program to characterize the effluents from industry, the influent to the POTWs and the sludge streams at the POTWs. If any adverse changes in sludge composition are noted, the MWRA can identify the constituent of concern, then take necessary corrective actions to curb the discharge of that particular compound to the sewerage system to ensure that residual quality from the MWRA treatment facilities is not diminished.

- Copper and lead have been identified as pollutants of concern in MWRA sludge. Recent studies indicate that a significant fraction of the influent loadings of copper and lead result from corrosion in water supply systems caused by corrosive water and household piping made of copper joined with lead solder (Table 5-11).

Table 5-11: Greater Boston Area Residential Water Supply Metals Quanity (Black & Veatch Inc., 1988)

Constituent	First Flush Measured Concentration* (ug/l)	Projected Design Year 2020 Concentration (ug/l)
copper	100 - 150	100
lead	18	17
zinc	5	220

* Measured values taken from an unpublished study carried out by Region I EPA in the Greater Boston Area during 1985. First flush refers to water that has been in contact with household piping for more than several hours.

Technological Assessment

Technologies to evaluate wastewater residuals processing, utilization, and disposal options were considered independently of a site location. The three primary tasks of the technological assessment report were to review existing data, screen technology blocks, and to develop more detailed information on selected technologies for the candidate option phase. The residual processes will be classified as major technologies. The technologies which provide support to the major technologies will be classified as the support technologies (Black & Veatch Inc, January 1987a). A list of the technologies considered is shown in Table 5-12.

Each major technology block was evaluated and screened based on the following in order to determine which technology was viable for MWRA residual facilities:

- Regulatory Assessment
- Engineering Assessment
- Environmental Impact Potentional
- Mitigation of Environmental Impacted

It was determined that all support technologies would be considered for either long-term, short-term, or both as remedies for the RMPF. There were only three viable major technologies chosen to be evaluated any further, these are:

- Landfilling
- Composting and Compost Distribution
- Combustion, Energy Production, and Ash Disposal

These viable major technologies were further evaluated and screened in terms of development criteria for use in the candidate options phase of the project based on the following:

- Preliminary design criteria
- Cost
- Applicability to mid-term/long-term plan
- Site suitability criteria
- Minor residuals

Table 5-12: A List of Technologies Considered by MWRA.

The Major technology blocks are:

- *Landfilling* - is the disposal of material on land by burial. This is a final disposal technique with no beneficial reuse unless future recovery methods are planned. This disposal technique is widely practiced throughout the U.S, although it requires a lot of land and increased monitoring practices. If sludge is disposed of in this way, the sludge must be mixed with an agent, such as soil, to stabilize it before landfilling. Minor residuals usually require less preparation and are more stable than sludge. A primary consideration when designing a landfill is its potential for groundwater contamination.

- *Composting and Compost distribution* - is a form of stabilization where sludge is decomposed by microorganisms in the presence of oxygen. Amendments, such as sawdust or woodchips, are usually added to the sludge to increase porosity so that air can enter. This increases the total solids concentration. The ideal operating range of composters is between 55 and 60 degrees Celsius. There are two categories of composters: reactors or in-vessel, and non-reactors. The compost material is used as a soil conditioner where the market is available.

- *Co-composting and Compost distribution* - is a form of stabilization where sludge is mixed with municipal solid wastes and then decomposed by microorganisms in the presence of oxygen to produce a compost material. The municipal solid wastes must be preprocessed to remove metals, glass, hard-plastics, and any other non-biodegradable material before mixing it with the sludge. The ideal operating range of composters is between 55 and 60 degrees Celsius. There are two categories of composters: reactors or in-vessel, and non-reactors. This process often produces a nuisance odor. The compost material is used as a soil conditioner where the market is available.

- *Ocean disposal* - may be accomplished by discharging through ocean outfalls or by transporting the sludge by barge out into the ocean. The disposal of sludge through ocean outfalls will be prohibited as of December 1991; therefore, only

barging is considered. In barging sludge to the ocean, the vessel transports the sludge to a designated location and discharges the sludge while moving. This causes the sludge to be mixed in the wake of the vessel resulting in increased initial dilution.

- *Combustion, Energy production, and Ash disposal* - is the heating/burning of sludge in incinerators in order to reduce the volume of material for final disposal, destroy pathogens, reduce or destroy toxics, and generate energy to be used throughout the treatment plant. The ash produced by incineration is either disposed of in landfills or processed for its metals content.

- *Co-combustion and Ash disposal* - is the heating/burning of sludge and municipal solid waste in incinerators in order to reduce the volume of material for final disposal, destroy pathogens, reduce or destroy toxics, and generate energy to be used throughout the treatment plant. The ash produced by incineration is either disposed of in landfills or processed for its metals content.

- *Sludge Oxidation and Ash disposal (Vert. tube reactors)* - includes a Vertical Tube Reactor (VTR) process that is a proprietary, complete, wet-air oxidation sludge reduction process (Fig. 5-8). Its main advantages are that it requires a limited amount of land, has low process energy requirements, and has sludge stabilization capacity. Its principal disadvantage is that it has only been used in one location in the U.S.

- *Combustion-at-Sea and Ash disposal* - is a high temperature thermal oxidation process that is done in an incinerator on board a vessel at sea. This process reduces the volume of waste for disposal. To date, this technology is not being used in the U.S.

The Supporting technology blocks are:

- *Thickening* - removes water from the sludge which reduces the volume and increases the solids concentration which increases the treatment efficiency downstream. The types of thickening devices evaluated were: gravity thickeners, dissolved air flotation, gravity belt thickeners, and centrifuge thickeners.

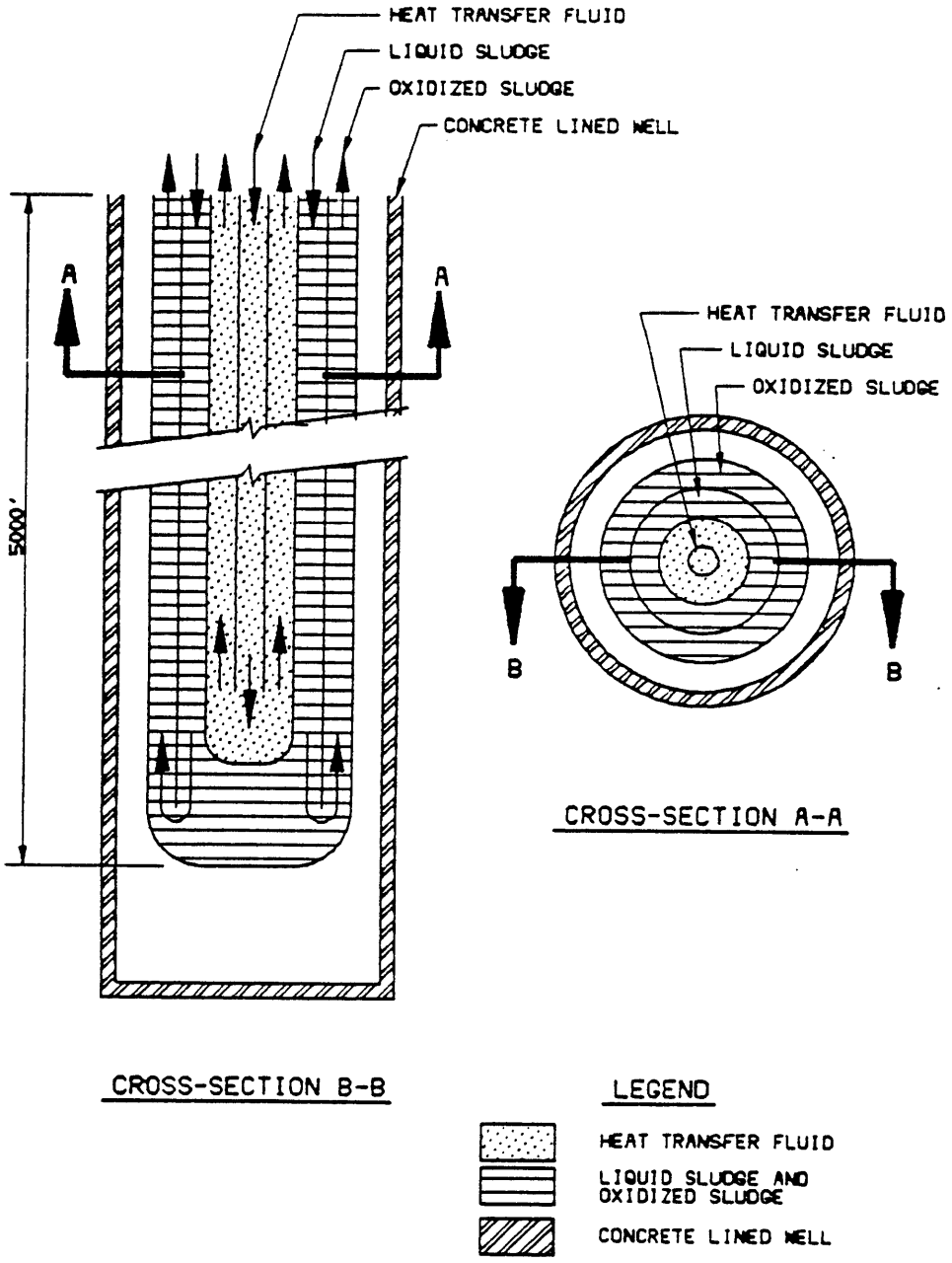

Figure 5-8: Vertical Tube Reactor (Black & Veatch Inc., 1987a)

•*Anaerobic digestion and gas utilization* - is a stabilizing process which reduces odors, pathogenic organisms, and the total mass of the residual solids. This process involves the biological degradation of organic substances in the absence of oxygen. The gas produced is approximately 65% methane, 35% carbon dioxide, and a trace of water. This gas has been used for sludge heating, building heating, and fueling gas engines to run equipment or generate electricity.

•*Dewatering* - removes water from the sludge which reduces the volume and increases the solids concentration. The purpose of dewatering is to reduce the cost and increase the ease of handling the sludge. Dewatering can increase the solids concentration up to 40%. The types of dewatering processes evaluated were: natural drying (open drying beds and lagoons), mechanical dewatering (belt filter press, centrifuges, and diaphram or plate-and-frame filter press), and thermal dewatering (heat dryers and rotary disc dryers). Thermal conditioning was also considered.

•*Chemical treatment* - is the addition of chemicals to improve the handling and physical characteristics of the sludge. Two types of chemical treatments were considered: lime treatment and chemical fixation. Lime is added to reduce odors, condition the sludge, adjust the pH, destroy pathogens, prevent bulking, and increase the percentage of solids removal. Chemical fixation decreases the surface area of the particles, inhibits the transfer or loss of contained pollutants, and limits the solubility of pollutants. An example of chemical fixation is MWRA's process of fixing cement kiln with scum and using it to reduce odors and kill pathogens within the scum. This mixture of scum and kiln is used as a cover for landfills.

Transportation Assessment

The feasibility and practicality of transporting a variety of residuals such as liquid sludge, thickened sludge, dewatered sludge, compost and its amendments, ash and minor residuals from the treatment plant at Deer Island to an inland, coastal, or island location were evaluated. Four basic transport modes were considered: barge, pipeline, rail, and truck (Black & Veatch Inc., January 1987b)

Site Assessment

A four-tier process was used to site the RMF (Black & Veatch Inc., August 1987b). In the first tier, sites were identified and evaluated without considering specific residual management technology. All 299 sites were identified based on the following criteria:

- geographic location (primarily within the MWRA service area)
- minimum acreage (five acres for coastal areas and eight acres for inland areas)
- development status (sites with non-active building or land use)

The 299 sites were then ordered based on site screening criteria. To specify the relative importance of each screening criteria, a numerical suitability scale was developed with a set of weighting factors in order. These weighting factors were determined based on information gathered from citizen advisory committees, a group of RMFP technical advisors, and a group of MWRA staff members.

In the second tier, sites were ordered according to a set of technology-specific requirements. This ordering incorporated the site requirements of a specific residual management technology. At this point, the MWRA recommended seven sites for further investigation.

In the third tier, the results from the residual characterization, technology assessment, transportation assessment, and site screening analysis were evaluated together resulting in 31 system combinations called "candidate options" (Black & Veatch Inc., August 1987c).

In the fourth tier, the sludge processing and minor residual alternatives were screened differently to determine preferred sites. Of the 31 candidate options, five were associated with the minor residuals. These five minor residual options went through a two-level screening process. In the first level, two sites were eliminated based on close proximity to superfund locations and permitting problems. In the second level, the remaining three sites were evaluated based on cost, environmental, institutional, and technical criteria. One site was eliminated.

This left two final options, which were then evaluated based on costs, capacity/life of site, transportation/traffic, water supply impacts, engineering considerations, impacts on abutting landowners/residents, and environmental impacts. Walpole was the most favorable site and is MWRA's preferred site for the minor residual landfill.

Candidate Option Alternatives

Due to advanced technologies, thermal processing is recognized as a viable technology to augment composting. Therefore, thermal processing (heat drying) was elevated from a support technology to a major technology and was considered in conjunction with composting.

The sludge processing candidate options also went through a two-level screening process (Black & Veatch Inc., August 1987a). In the first level, the options were evaluated based on flexibility and reliability. This narrowed the options down to 19 system combinations. In the second level, the remaining 19 options were evaluated based on cost, environmental, institutional, and technical criteria. This narrowed the options down to five (Table 5-13).

After reviewing the list of options, MWRA's Board of Directors decided that incineration was technically too difficult to operate and presented undesirable risks to the residents and the environment. Therefore, incineration was ruled out. This left the Fore River Staging Area (FRSA) in Quincy as the only option available for the sludge processing facility. This facility will process the sludge using two methods: composting and heat drying (which produces compost pellets).

Table 5-13: Options Selected for Final Review by MWRA

Options Selected For Final Review

	Quincy	Stoughton	Spectacle Island
*Option 1)	composting heat drying		
Option 2)	composting heat drying		incineration
Option 3)	heat drying	composting incineration	
Option 4)	heat drying		incineration
Option 5)			composting heat drying incineration

*Preferred Option

Currently

The MWRA has determined that for the short term (1991-1995), sludge processing will be done at the FRSA in Quincy, and Quincy has agreed to this. For the long-term (1995-2020), MWRA has chosen Walpole as the minor residual landfill site and FRSA as the site of the sludge processing facilities. To date, these long-term facilities are not yet finalized.

The compost and pellets that will be produced by MWRA will meet all EPA standards as non-hazardous material (Table 5-10) and Massachusetts Department of Environmental Quality Engineering (DEQE) standards for Type III material (Table 5-10). Although, DEQE is looking into their standards for Molybdenum so that the processed sludge will be classified as a Type II material. The primary metal that stops the sludge from being classified as a Type I material is copper. MWRA believes that the primary source of copper in the system results from copper pipes in residual areas.

DEQE's regulations regarding sludge application is as follows (Commonwealth of Massachusetts, 1983):

- "Type I - Sludge ... may (be) used, sold, or distributed or offered for use, sale, or distribution on any site without further approval of the Department, and which may be used for growing vegetation."

- "Type II - Sludge ... may be used, sold, or distributed or offered for use, sale, or distribution on a site only with prior approval of the Department, and which may be used for growing any vegetation."

- "Type III - Sludge ... may be used, sold, or distributed or offered for use, sale, or distribution for land application on a site only with prior approval of the Department, which may be used for growing vegetation not including direct food chain crops, and whose land application to a site must be recorded in the registry of deeds in the chain of title for such site."

MWRA has chosen to dispose of minor residuals in a landfill, and to reuse the sludge by composting and heat drying.

Minor Residuals

The landfill site being considered for the disposal of MWRA minor residuals is located in Walpole (Fig. 5-9). This site encompasses an area of approximately 94 acres and is owned by the Commonwealth of Massachusetts. Of the 94 acres, only 47 acres will be used in the landfill operation. A cross-section of the landfill is shown in Figure 5-10. The landfill will have two liners, two leachate collection systems, a cover, and groundwater monitoring wells. The landfill will be designed using proven technologies and will isolate the minor residuals from the surrounding environment.

Under normal conditions, about four truckloads per day of minor residuals will be disposed of in the landfill. Over the 25-year life of the landfill, only 28% of the original capacity of the landfill will be used under normal conditions. According to EOEA, the landfill site should also be capable of holding six months of sludge as an emergency backup. Currently, the MWRA feels that this is not necessary because the marketing reports on the reuse of sludge indicate that there will be enough of a demand for the composted and pelletized sludge. The estimated lifetime and operational cost of the Walpole landfill is $23.8 million (1988 dollars).

Sludge

The sludge processing facilities, which include composting and heat drying, will be located at the Fore River Staging Area (FRSA) in Quincy (Fig. 5-1). This site encompasses approximately 150 acres and was purchased by MWRA in August of 1987. Of the 150 acres, only 50 acres will be used in the composting and heat drying processes. Construction is expected to begin in 1989, with the New England Fertilizing Co. beginning operation of the short-term heat drying facility by 1991. Completion of the long-term facilities is not expected to occur until 1999. The short-term facility will dewater, heat dry, and pelletize the liquid sludge barged from the Deer and Nut Islands wastewater treatment plants. The long-term facility will also compost the sludge.

Heat drying involves heating the sludge in tank-like dryers. This process removes moisture through evaporation and greatly reduces the volume of sludge. Heat drying is done at temperatures that kill harmful viruses and bacteria. After heat drying, the sludge will be formed into pellets for ease of transport.

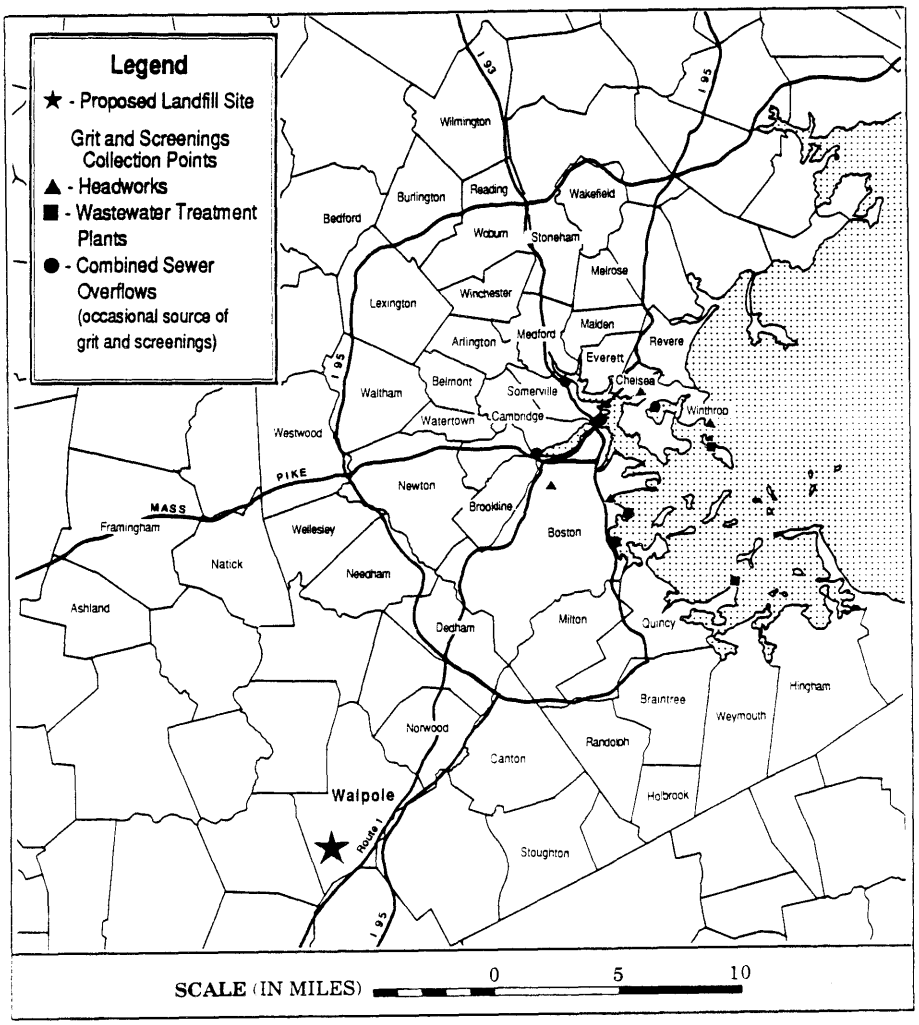

Figure 5-9: Map of Boston and Vicinity

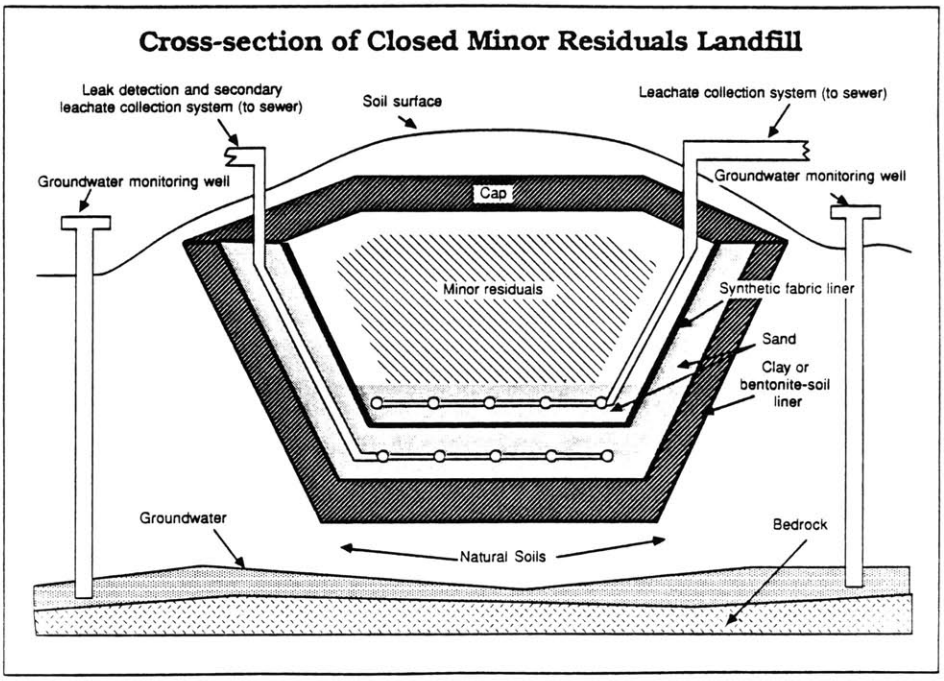

Figure 5-10: Cross-section of Closed Minor Residual Landfill

Pellets formed by heat-drying sludge can be used as fuel for an energy-recovery incinerator and as a partial substitute for commercial fertilizer in areas such as:

- golf courses
- sod farms
- horticultural uses
- agricultural uses
- forest areas

Some advantages that heat drying provides are:

- it is a proven technology
- a permit system is already in place
- it is a flexible technology
- facilities can be built rapidly
- it converts waste into a resource

There are some problems associated with heat drying; although, these problems can be mitigated through either source reduction or technologies. Some of the problems associated with heat drying are as follows:

- increased concentration of contaminates in dried sludge
- production of offensive odors
- high energy cost

Composting involves forcing air through a mixture of sludge and an amendment, such as woodchips or sawdust. The addition of an amendment accomplishes two objectives. First, it reduces the concentration of contaminates, and second, it increases the porosity of the sludge, which allows more air to enter the mixture. In the presence of air, microorganisms multiply by breaking down the organic material in the sludge, causing heat, reducing the volume, and conversion of a waste into a resource. Compost piles will reach temperatures of 50° to 70° C, which are hot enough to kill harmful viruses and bacteria.

The compost can be used as a soil conditioner in areas such as:

- landscaping
- enhancing median strips and highways
- greenhouses and nurseries
- land reclamation

Some advantages that composting provides are:

- it is a proven technology
- a permit system is already in place
- pollution control technologies already exist
- facilities can be built rapidly
- it converts waste into a resource

There are some problems associated with composting; although, these problem can be mitigated through either source reduction or technologies. Some of the problems associated with composting are as follows:

- increased concentration of contaminates in composted sludge
- production of offensive odors
- threat to groundwater
- generation of compost leachate

Marketing

It is projected by the year 2020, with secondary treatment operational, MWRA will produce 163 dry tonnes of digested sludge per day. MWRA plans to compost and pelletize their sludge and sell it as a soil conditioner. The April 1989 issue of the TBHA newsletter entitled "But Sewage Sludge!" addresses the marketability of composted and heat-dried sludge. Below is a summary of the questions addressed in that article (TBHA, 1989).

- Can MWRA sell approximately 40 truckloads of processed and dried sludge per day?

- Will anyone buy sludge with the concentration of toxics taken from a wastewater treatment plant?

- Will anyone be willing to put on their land a material that EPA says is too dangerous to put into the ocean?

- How will the proposed revisions to EPA regulations regarding toxics in sludge products, ie. copper standards, affect the marketability of the sludge?

- What will happen when competition, from municipalities such as New York, New Jersey, and Philadelphia enter the market?

- What will happen to the sludge if no one wants it?

Based on their marketing analysis, MWRA believes that there is a large demand for sludge products. They believe that their production of "heat dried pellets may be only 3% of the demand in the New York and New England area" (TBHA, 1989). MWRA also has initiated a research program in conjunction with the University of Massachusetts / College of Food and Agriculture, that will obtain information to develop a database on the usability of MWRA sludge produces. This information will be used to evaluate management criteria and safety issues associated with MWRA sludge products.

MWRA has recently demonstrated the use of the compost material in the following incidences:

- in the flower beds of several traffic circles in South Boston
- for ornamental shrubs in a new traffic island in Winthrop
- for trees at the new MWRA Fox Point CSO project

These products have been successful and MWRA plans to continue with similar projects in the future.

MWRA plans to pelletize about two-thirds of the sludge and compost the remaining one-third. They believe that the sludge will be classified as a Type II sludge (Table 5-10) by DEQE; although the current levels of toxics indicate a Type III sludge. This would severely limit the market distribution of the sludge product in the state of Massachusetts.

It is yet to be determined if EPA's newly proposed regulations will affect the classification of MWRA sludge products. These proposed regulations are stricter than DEQE's regulations; although, they are not finalized as of yet. If these regulations are imposed, there would severely inhibit the beneficial reuse of sludge products throughout the country and would force many municipalities to rely on landfilling and incineration.

It is essential for MWRA to tighten their control over source reduction in order to ensure more marketable products.

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Chapter 6

Combined Sewer Overflows (CSO)

Combined Sewer Overflows (CSO)

Introduction

Combined sewers are designed to carry both wastewater and stormwater through a single pipe to the wastewater treatment plant. If the maximum capacity of these pipes, or some other aspect of the sewerage or treatment system is overloaded, flow is diverted and discharged as overflow at locations close to the shore. This overflow, termed Combined Sewer Overflow (CSO), results in raw wastewater and stormwater being discharged into water bodies. This discharge visibly pollutes the water body, carries harmful viruses and pathogens, degradation of the water quality, and usually results in beach and shellfish bed closures.

Current regulations require all locations where CSOdischarge to have a National Pollution Discharge Elimination System (NPDES) permit. These permits establish limitations on the amount of pollution discharged from a point source (ie. CSO). In MWRA service area, 87 of the 101 CSOs (Fig. 6-1) do not meet the limitations set forth in their permits. MWRA has accepted the responsibility for the management of the CSOs in the locations where they exist, namely: Boston, Cambridge, Chelsea, and Somerville. Although, Boston and Cambridge already have begun to separate their sewers.

The CSO problem occurs during both wet and dry periods. During wet periods, an excess loading on all parts of the sewerage system occurs and flow is purposely diverted. This occurs during storm events where the first flush from the storms contribute a majority of the suspended solids. During dry periods, there are three major causes of CSOs: blockage and sedimentation, inadequacies in piping and mechanical devices, and inadequate capacity at the Deer Island treatment plant. Inadequate capacity at Deer Island treatment plant is the primary cause of Dry Weather Overflows (DWO).

Once the fast-track improvements (see Chapter 4) are made in the Deer Island headworks facilities and pumping stations, the total CSO flow is expected to decrease by 45%, from 39.7 to 22.0 billion liters per year. This reduction in flow through CSOs is projected to reduce the number of overflows to approximately 45 to 55 times per year without additional controls.

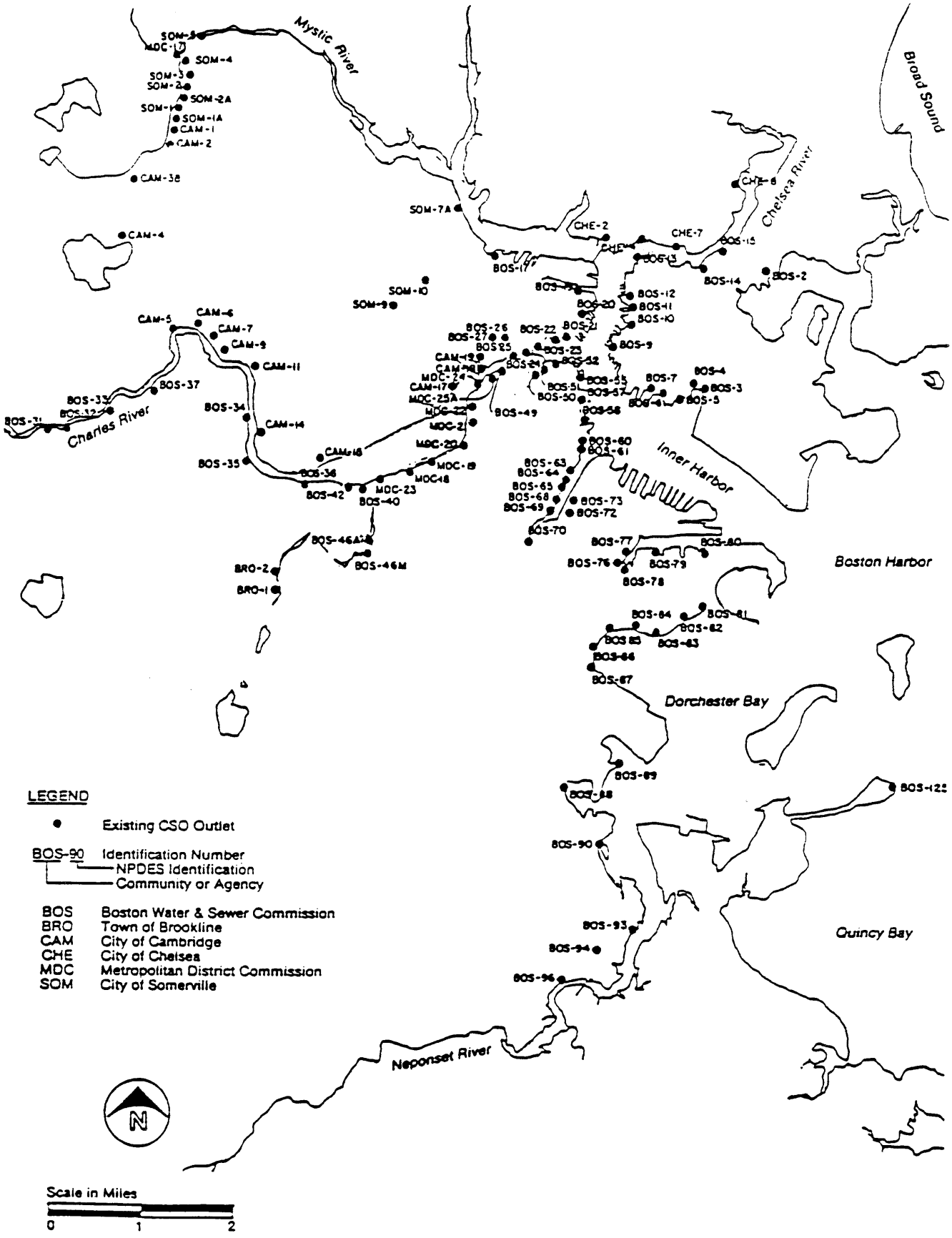


Figure 6-1: Locations of existing Combined Sewer Overflows (CH2M HILL, 1988)

Initially, CSO were considered a solution to a communities wastewater and stromwater problems because they prevented backup into residents homes. Today, CSO are considered a problem. As the cities and population have increased, the amount of wastewater in the overflow has increased to a point of significantly imparing the receiving water. EPA has recently established regulations (Federal Register, 1988) on stormwater discharges which would require urban runoff to meet the same regulations as other point sources. These regulations are expected to take effect in 1991. After 1991, CSO may once again be considered an advantage from a control stand point.

To date, there are only preliminary reports available on CSO control for MWRA's service area. These reports were based on an annual flow of 22.0 billion liters.

Preliminary Screening of Long-term CSO Control

CH2M HILL, under contract to MWRA, evaluated alternative technologies for controlling MWRA's CSO problem. These technologies were rated as favorable or unfavorable in regard to there probable success. The study divided the service area into five sub-areas based on the receiving waters. The following is a list of these areas:

- Dorchester Bay / Inner Harbor
- Lower Mystic River (Marine portion)
- Charles River Basin
- Alewife Brook / Upper Mystic River (Freshwater portion)
- Neponset River

Dorchester Bay / Inner Harbor

The Dorchester Bay / Inner harbor area encompasses portions of Quincy Bay and Boston Harbor including the Inner Harbor and Chelsea Creek CSOs. This area represents 46% of the total CSO study area, the largest surface drainage area. It also represents 48% of the flow, 50% of the SS, 72% of the BOD, and 76% of the fecal coliform entering the harbor from CSO (Table 6-1). It represents 10% of the flow, 6% of the SS, 19% of the BOD, and 29% of the fecal coliform entering the harbor from stormwater discharges (Table 6-2). Eighty percent of the flow into the Boston Harbor from the Dorchester Bay / Inner Harbor area is from CSO (Table 6-3).

Table 6-1: Average Annual CSO Flows and Loadings for the Year 2020

Location	Area (acres)	% of toal Area	Flow (MG)	% of total Flow	SS (1000 lbs)	% of total SS	BOD (1000 lbs)	% of total BOD	Fecal Coliform (10^{15} MPN)	% of total Fecal Coliform	Copper (lbs)	% of total Copper
Dorchester Bay / Inner Harbor	4928	46	2770	47.6	3900	50	2745	71.6	312	75.7	2077	47.6
Lower Mystic River	531	5	160	2.8	225	3.0	159	4.1	18	4.4	120	2.8
Charles River Basin	3735	35	2810	48.3	3488	45	854	22.3	70	17	2107	48.3
Alewife Brook / Upper Mystic	894	8	16	0.3	20	0.3	10	0.3	1	0.2	12	0.3
Neponset River	597	6	60	1.0	129	1.7	65	1.7	11	2.7	45	1.0

Table 6-2: Average Annual Stormwater Flows and Loadings for the Year 2020

Location	Area (acres)	% of toal Area	Flow (MG)	% of total Flow	SS (1000 lbs)	% of total SS	BOD (1000 lbs)	% of total BOD	Fecal Coliform (10^{15} MPN)	% of total Fecal Coliform	Copper (lbs)	% of total Copper
Dorchester Bay / Inner Harbor	4928	46	675	10	770	6	203	18.6	28.9	39.4	2077	47.6
Lower Mystic River	531	5	50	1	57	0.5	15	1.4	2.1	2.9	120	2.8
Charles River Basin	3735	35	4590	68	5710	45	570	52.3	19	25.9	2107	48.3
Alewife Brook / Upper Mystic	894	8	635	9	3030	23.5	83	7.6	15.3	20.8	12	0.3
Neponset River	597	6	790	12	3200	25	220	20.1	8.1	11	45	1.0

Table 6-3: Comparision of Average Annual CSO and Stormwater Flows for the Year 2020

Location	Area (acres)	% of toal Area	Total Flow (MG)	% of total Flow	CSO Flow (MG)	% of Total Flow	Stormwater Flow (MG)	% of Total Flow
Dorchester Bay / Inner Harbor	4928	46	3445	27	2770	80	675	20
Lower Mystic River	531	5	210	2	160	76	50	24
Charles River Basin	3735	35	7400	59	2810	38	4590	62
Alewife Brook / Upper Mystic	894	8	651	5	16	2	635	98
Neponset River	597	6	850	7	60	7	790	93

Lower Mystic River

The Lower Mystic River area represents 5% of the total CSO study area. It also represents 3% of the flow, 3% of the SS, 4% of the BOD, and 4% of the fecal coliform entering the harbor from CSO (Table 6-1). It represents 1% of the flow, 1% of the SS, 1% of the BOD, and 3% of the fecal coliform entering the harbor from stormwater discharges (Table 6-2). Seventy-five percent of the flow into the Boston Harbor from the Lower Mystic River area is from CSO (Table 6-3).

Charles River Basin

The Charles River Basin area represents 35% of the total CSO study area. It also represents 48% of the flow, 45% of the SS, 22% of the BOD, and 17% of the fecal coliform entering the harbor from CSO (Table 6-1). It represents 68% of the flow, 45% of the SS, 52% of the BOD, and 26% of the fecal coliform entering the harbor from stormwater discharges (Table 6-2). Thirty-eight percent of the flow into the Boston Harbor from the Charles River Basin area is from CSO (Table 6-3).

Alewife Brook / Upper Mystic River

The Alewife Brook / Upper Mystic River area represents 8% of the total CSO study area. It represents 1% of the flow, 1% of the SS, 1% of the BOD, and 1% of the fecal coliform entering the harbor from CSO (Table 6-1). It represents 9% of the flow, 24% of the SS, 8% of the BOD, and 21% of the fecal coliform entering the harbor from stormwater discharges (Table 6-2). Two percent of the flow into the Boston Harbor from the Alewife Brook / Upper Mystic River area is from CSO (Table 6-3).

Neponset River

The Neponset River area represents 6% of the total CSO study area. It also represents 1% of the flow, 2% of the SS, 2% of the BOD, and 3% of the fecal coliform entering the harbor from CSO (Table 6-1). It represents 12% of the flow, 25% of the SS, 20% of the BOD, and 11% of the fecal coliform entering the harbor from stormwater discharges (Table 6-2). Seven percent of the flow into the Boston Harbor from the Neponset River area is from CSO (Table 6-3).

For each of the five areas studied, four major CSO control approaches were considered. These control approaches are as follows:

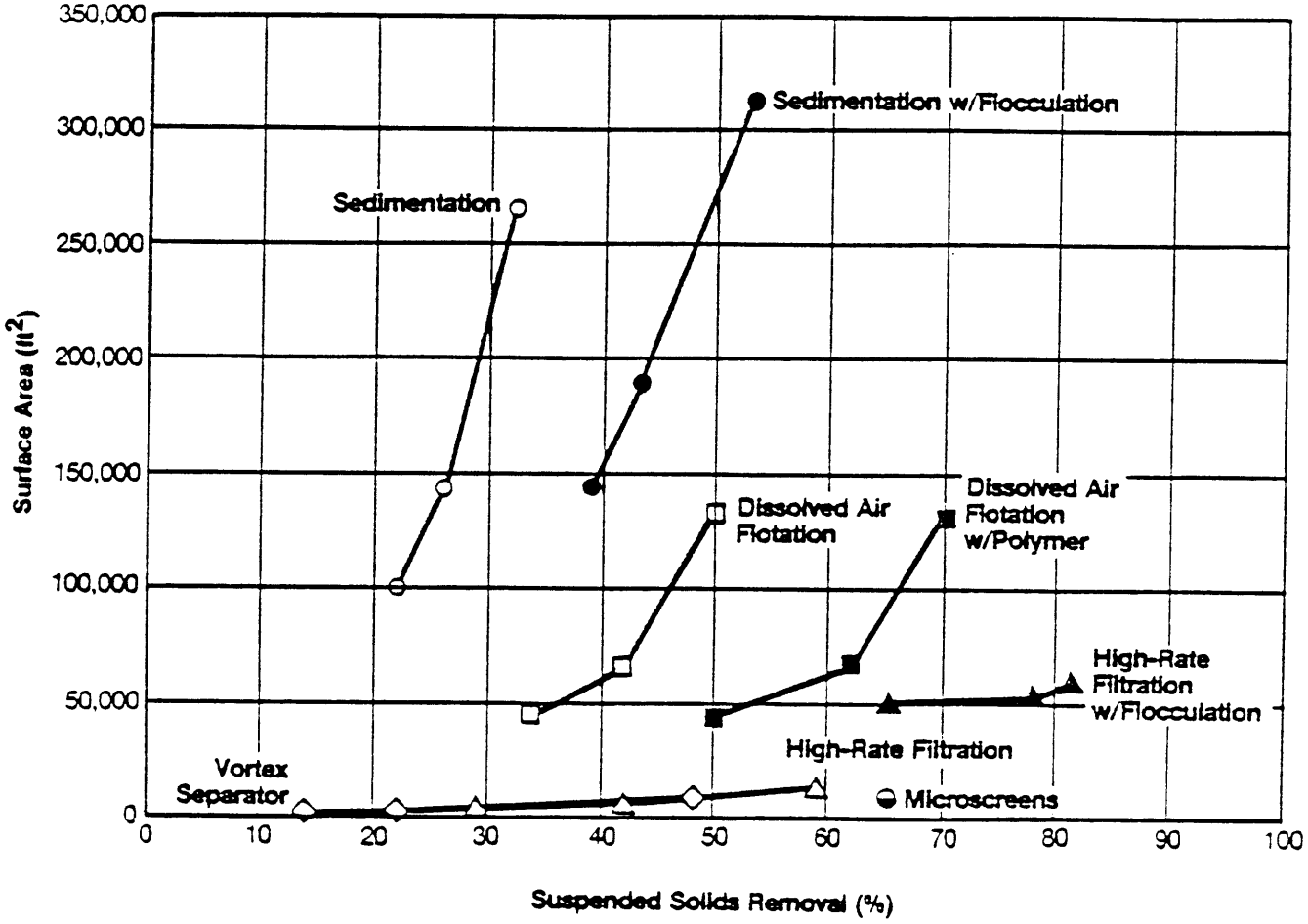
- Satellite Treatment
- Near-Surface Storage
- Deep tunnel Storage
- Sewer Separation

Satellite Treatment Technologies

Satellite treatment facilities are localized treatment facilities that treate wastewater at an individual location. These technologies are direct, end-of-pipe treatments. The primary concern with this type of treatment is that the flow will be highly intermettent and variable; therefore excluding biological treatment of the waste stream. Some form of storage for the flow will also be required at these satellite facilities. Nine satellite treatment technologies were evaluated as possible control approaches. These are as follows:

- Vortex Separators
- Coarse Screening
- Microscreening
- Sedimentation
- Sedimentation with Polymer Addition
- Dissolved Air Flotation
- Dissolved Air Flotation with Polymer Addition
- High-Rate Filtration
- High-Rate Filtration with Polymer Addition

These technologies were evaluated based on unit suspended solids removal costs and surface area requirements (Fig. 6-2). It was determined that for all five areas, satellite treatment would not provide adequate pollution reduction (except for fecal coliform due to disinfection) because of the inflexibility in adjusting to changes in flows, loadings and future expansion. Most treatment technologies need substantial surface area, and there are continual operating and maintenance costs to be considered.



NOTE: Flow rate = 200 million gallons per day.

Figure 6-2: Area vs. Suspended Solids Removal for Satellite Treatment Technologies (CH2M HILL, 1988).

Near-Surface Storage

Near-surface storage consists of a number of individual facilities connected to existing sewer lines. These surface storage facilities would hold excess flow until the hydraulic capacity of the system could transport the wastewater to the treatment plant. The storage system considered (Fig. 6-3) would be capable of holding approximately 330 ML - 320 ML along the Charles River, 3.4 ML along the Neponset River, and 5.7 ML along the Alewife Brook - at a cost of $173 million (1988 dollars). This system would prohibit all flow that was captured from reaching Boston Harbor. Although, only 60% of the flow and 70% of the pollutants would by eliminated due to CSO in the areas. Since all of this flow will receive treatment at the new Deer Island treatment facilities, near-surface storage is considered a viable alternative and will be considered on a site-specific basis. Difficulties may arise from the cost and acquasition of the surface sites.

Deep-Rock Tunnel Storage

Deep-rock tunnel storage consists of a long tunnel connected to existing sewer lines. This storage facility would hold excess flow until the hydraulic capacity of the system could transport the wastewater to the treatment plant. The deep tunnel facility would consist of five main components:

- Near-surface conduit to consolidate the flow from several tunnels.
- Vertical vortex-type drop shafts to convey the consolidated flow to the main tunnel.
- Deep tunnels to store the flow and connect the drop shafts.
- One or more pump stations to transfer the stored flow to the treatment plant.
- Dewatering facilities to flush the tunnels in order to control sedimentation.

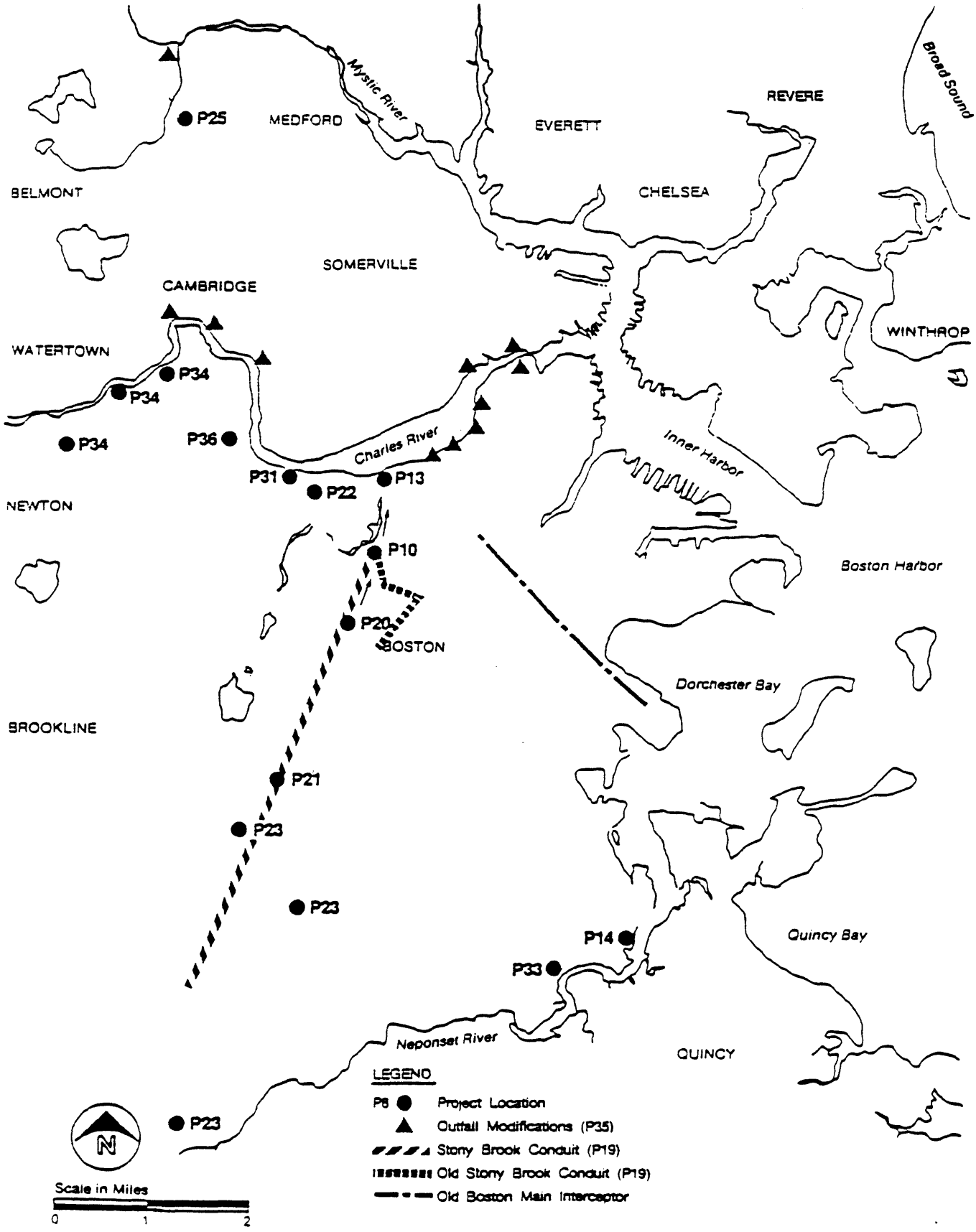


Figure 6-3: General Location of Near-Surface Storage Facilities (CH2M HILL, 1988).

The deep-rock tunnel system (Fig. 6-4) would service the entire combined sewer overflow area with a storage capacity of 1620 ML at a cost of approximately $800 million (1988 dollars). The tunnel is anticipated to have a diameter of 7.6 m and a linear extent of 32 to 40 km. The tunnel will capture 87% of the flow and 96% of the pollutants before it enters the harbor. Environmentally, deep-rock tunnel storage is highly desirable because of the high capture rates. Land requirements for deep-rock tunnels are small; although, it would require a major construction effort and take several years to construct. This is a highly reliable alternative and would eliminate DWO. Since all of the flow will receive treatment at the new Deer Island treatment facilities, this is considered a viable alternative and will be considered on an area-wide basis.

Sewer Separation

Currently, approximately 5% of the MWRA service area has combined sewers (Fig. 6-5). Sewer separation involves placing new sewer lines so that wastewater can flow through one pipe line and the stormwater can flow through another pipe line. This is very disruptive, especially in highly populated areas, and would only partially solve the loading problems in Boston Harbor (Tables 6-2,3). The cost of separating all sewers is $1.2 billion (1988 dollars).

Sewer separation is very reliable in transporting wastewater to the treatment plant; although, the stormwater will not be treated. Not treating the stormwater is a problem due to the new stormwater regulations established by EPA (Federal Register, 1988) expected to take effect in 1991. The regulations will require stormwater discharges, if classified as a point source, to have the same NPDES permit that the CSOs have. This is an undesirable alternative and not considered for any of the areas.

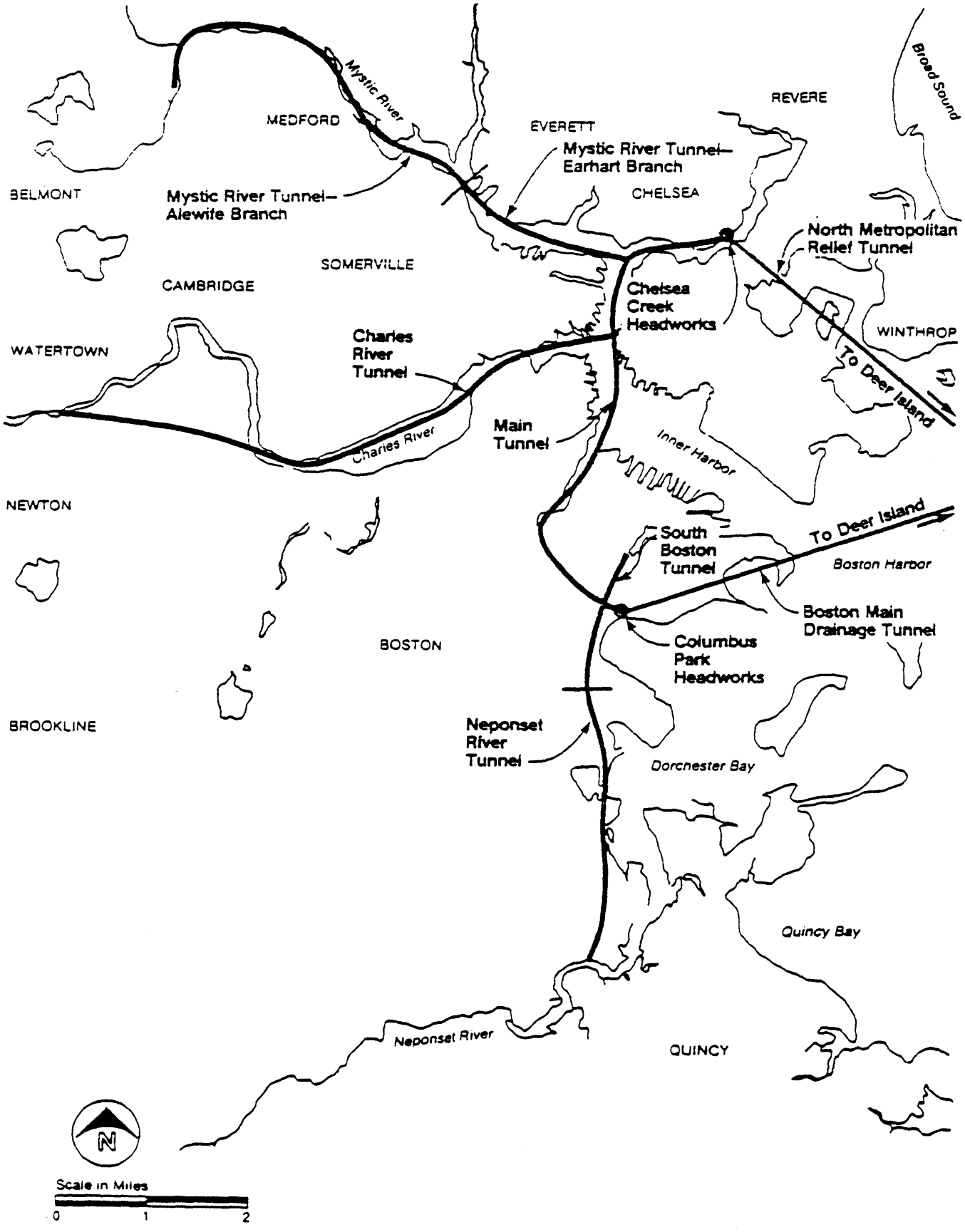


Figure 6-4: Alignment of Major Deep Rock Tunnels (CH2M HILL, 1988).

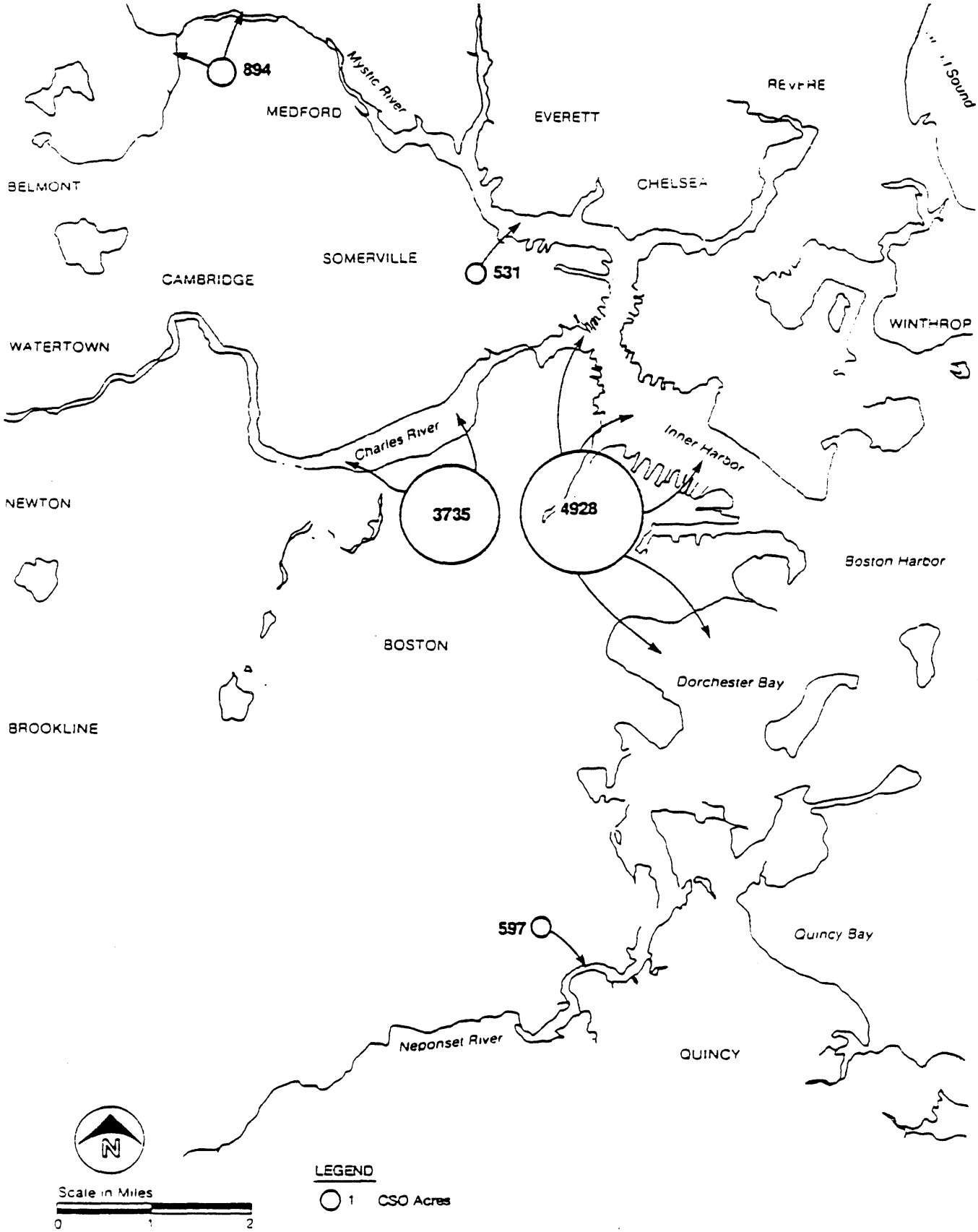


Figure 6-5: Distribution of CSO areas (CH2M HILL, 1988).

Recommended Plan

It has been recommended that storage is the best means of controlling CSOs. The following is a list of the service areas and the recommended alternatives to be considered in proceeding studies.

Dorchester Bay / Inner Harbor	Deep-Rock Tunnel
Lower Mystic River	Deep-Rock Tunnel
Charles River Basin	Deep-rock Tunnel / or Near-Surface Storage
Alewife Brook / Upper Mystic River	Deep-Rock Tunnel / or Near-Surface Storage
Neponset River	Deep-Rock Tunnel / or Near-Surface Storage

If the deep-rock tunnel alternative is chosen for the entire area, 87% of the flow will be captured at a cost of $800 million. If both the deep-rock tunnel and near-surface storage alternatives are used concurrently, 73% of the flow will be captured at a cost of $650 million.

The draft CSO facilities plan is due in December 1989 and the final facilities plan is due in May 1990.

References

Federal Register. Volume 53(235):49416 December 7, 1988.

All other material for this section was extracted from the following documents:

CH2M HILL, Boston Massachusetts. December 1988. Combined Sewer Overflow Facilities Plan, Technical Memorandum 5-1A, Interim Water Quality Impact Evaluation. Prepared for Massachusetts Water Resource Authority.

CH2M HILL, Boston Massachusetts. December 1988. Combined Sewer Overflow Facilities Plan, Technical Memorandum 7-6, Preliminary Screening ofCSO Control Alternatives. Prepared for Massachusetts Water Resource Authority.

CH2M HILL, Boston Massachusetts. February 1989. Combined Sewer Overflow Facilities Plan, Technical Memorandum B1-1, Chronic Dry Weather Overflow Problem Areas. Prepared for Massachusetts Water Resource Authority.

CH2M HILL, Boston Massachusetts. February 1989. Combined Sewer Overflow Facilities Plan, Final Dry Weather Overflow Control Program. Prepared for Massachusetts Water Resource Authority.

Chapter 7

Modeling and Monitoring

Modeling and Monitoring

Introduction

Mathematical models are simplified analytical descriptions of the real world. The models do not attempt to incorporate all the physical phenomena relevant to the problem under consideration – only those that are relevant. If properly constructed, these models are valuable diagnostic and predictive tools used in management decisions. The validity of the prediction is often determined by how well the model approximates field observations. Good field data is essential when using the model in a predictive mode. The field data is obtained by systematically monitoring parameters that describe the modeled system.

When developing a model, the real world is broken down into physical systems that can be mathematically described in terms of differential equations with boundary conditions and initial conditions. The equations are represented discretely in order to use the computer in long simulations. The models require input parameters, which are obtained from field observations or published data, to solve the equations. The physical system can be represented in zero, one, two or three spatial dimensions and one time dimension.

The models discussed in this report addresses the hydrodynamics and water quality in the coastal zone. These types of models can be used in the following ways:

- As a predictive tool for management.
- To develop pollution reduction goals.
- To develop water quality standards.
- To determine ambient water quality and hydrodynamic conditions.
- To predict sediment deposition and transport.
- To predict the fate of toxic material.
- To determine the relationship between nutrient loading and eutrophication.
- To determine the critical nutrients to control eutrophication and anoxia.
- To determine how and the degree to which nutrient loading should be controlled.
- To set priorities for control strategies.
- To determine the time required to improve the water quality of a specific area.
- To determine the circulation in a specific area.

Whenever effluent is discharged into a body of water, the effluent must meet the water quality limits specified in a permit. In an effort to comply with the regulations set forth by the permit, municipalities have established monitoring programs. The monitoring programs measure a variety of water quality parameters on a daily or weekly basis. Although this information is essential in determining the quality of the receiving water, information that would aid in predicting and understanding the conditions associated with the hydrodynamics of the receiving water (such as data on currents and density profiles) is not often collected. The absence of this information makes it difficult to predict when a discharge might not comply.

A good monitoring program should measure conventional and non-conventional pollutants and flow rates associated with the effluent, water quality, flow field, and benthic conditions in the vicinity of the discharge and at nearby shores. Following is a list of parameters that need to be monitored in order to properly model the physical system.

- Temperature
- Salinity concentrations
- Heavy metals concentrations in effluent, water column and sediments
- Nutrient loading (point and non-point sources)
- Nutrient concentrations (N, P, etc.)
- Dissolved Oxygen
- pH
- Toxic compounds in effluent, water column and sediments
- Turbidity (suspended particles)
- Coliform bacteria
- Plankton in the vicinity of the discharge (both phytoplankton and zooplankton)
- Benthic fauna and flora
- Flow field
- Density profiles

In this chapter, the modeling efforts on the Boston Harbor will be discussed in four parts: the modeling done before the 301(h) waiver application, during the waiver, after the waiver, and what should be done in the future.

Pre-Waiver (before 1979)

One of the first efforts to model Boston Harbor was by Hydroscience Inc., in 1971. This model attempted to describe the effects of discharges from Deer Island (effluent and sludge) and discharges from Combined Sewer Overflows (CSO). The model represented the hydrodynamics of Boston Harbor as two open channels with the channels axes along President Roads and Nantasket Roads (Fig. 1-2). The mass fluxes associated with these channels were represented by freshwater advection and tidally averaged dispersion. Segments adjacent to the channels received mass fluxes only by dispersion.

In order to overcome some of the shortcomings of their first model, Hydroscience Inc. developed another model used on Boston Harbor in 1973. This model used a formal (Leendertse's) two dimensional, finite difference, approach to describe the hydrodynamics of Boston Harbor. The model was calibrated using the National Ocean Survey of 1971. The water quality portion of the model was developed only to investigate the temporal and spatial variations of coliform. The major drawbacks of this model are high computational expenses to run the model and very coarse discretization.

Also in 1973, the New England Aquarium used a model to simulate the aquatic ecosystem of Boston Harbor. Formal hydrodynamics were not incorporated into this model, but rather it used an estimated flow field as input to the mass conservation equation.

In 1974, Process Research Inc. used the same model Hydroscience Inc. used in 1971 to evaluate the pollution of beaches in Dorchester Bay (Fig. 1-2) by CSO.

In the early 1970's, researchers at Massachusetts Institute of Technology (MIT) developed a set of finite element computer models - CAFE (Circulation Analysis by Finite Elements) and DISPER (Pollution Dispersion based on CAFE). These models were applied to Massachusetts Bay for the New England Offshore Mining Environmental Study (NOMES) project (Connor and Wang, 1973). This modeling project made no specific effort to model Boston Harbor.

Waiver (1979 to 1986)

In Boston's attempt to obtain a 301(h) waiver from secondary treatment, they were required to model both the near-field and far-field effects of the effluent discharged through the proposed deep tunnel outfall. The near-field model will be discussed first followed by the far-field models.

Near-field

The near-field was modeled by Metcalf & Eddy Inc. in 1979 as part of the 301(h) waiver application. They evaluated five near-field mixing zone models: PLUME, OUTPLM, DKHPLM, Fan and Brooks Method, and Abrahams Method and concluded that DKHPLM was the best model to use. DKHPLM, developed by EPA, analyzes the discharge of a round buoyant jet in a stratified, flowing, ambient environment. It conserves mass and momentum and always predicts the plume width larger than observed.

In 1984, Metcalf & Eddy Inc. used UDKHDEN to recalculate the initial dilution for Boston's revised 301(h) waiver application. UDKHDEN, based on work done by Davis (1975), is a model developed by EPA and accepts inputs of seawater and effluent densities, current velocity, effluent flow rate, depth of effluent discharge, and diffuser size and alignment. It then calculates the initial dilution at the edge of the mixing zone and the trapped height of rise of the plume for stratified conditions.

Far-field

The far-field was modeled by Metcalf & Eddy Inc. in 1979 as part of the 301(h) waiver application. They used CAFE and DISPER to predict the effects of effluent discharges at the proposed outfall. In 1982, Metcalf & Eddy Inc. again used CAFE and DISPER to better describe the effects in the outer harbor. In 1984, EG&G used CAFE and DISPER in Boston Harbor and attempted to better resolve the complex geometry of the harbor. Due to the extent to which CAFE and DISPER have been used in Boston Harbor, a brief description of each is given.

CAFE (Circulation Analysis by Finite Elements)

CAFE is a two-dimensional, finite element, single layered, vertically integrated, circulation model. It simulates tidal elevations and the velocity field (tidal currents). This model solves the continuity equation and the momentum equation. The continuity equation is based on the conservation of mass principle which states that the mass of a moving fluid particle remains constant. The momentum equation is based on the conservation of

momentum principle which states that the time rate of change of momentum of a moving fluid particle is equal to the sum of the forces acting on the particle.

CAFE, being a finite element model, uses triangular elements to better define the area of interest. Temporal variations are determined by a time integration technique, which is similar to the central difference method and produces good stability. Boundary conditions can be given as fluxes of tidal amplitudes. Bottom friction and eddy viscosities (due to internal turbulence) are inputted as constant coefficients. The model is usually run for only one tidal cycle because of the large computational expense and precludes the use of finely spaced grids.

DISPER (Pollution Dispersion based on CAFE)

DISPER is a two-dimensional, vertically integrated, water quality model. DISPER uses input from CAFE to solve the conservation of mass equation for a dissolved constituent. Results are obtained as pollutant concentrations. DISPER uses the same grid as CAFE. Temporal variations are determined by an implicit iterative scheme. Boundary conditions and sources and sinks of the material can be given as fluxes or concentrations. Dispersion and decay (or settling rates) are inputted as constant coefficients.

Post Waiver (1986 to present)

In 1986, computer models developed at Massachusetts Institute of Technology (MIT) were used to measure volatile organic compounds associated with the effluent discharged into Boston Harbor (Kossik et al., 1986). The models used were TEA (Tidal Embayment Analysis) and ELA (Eulerian-Lagrangian Analysis). These models allowed for better resolution of complex geometry and permitted long-term simulation without incurring large computer expenses.

In 1988, MWRA was required to model both the near-field and far-field effects of the effluent discharged through the proposed outfall for their Secondary Treatment Facilities Plan and Environmental Impact Report. To determine the near-field effects, EPA's ULINE model was used. To determine the far-field effects, MIT's TEA and ELA were used. Each of these models are discussed below along with the findings associated with each modeling effort.

Near-Field Modeling with ULINE

The near-field, or zone of initial dilution (ZID), is associated with the impacts of the effluent in the immediate vicinity of the outfall and has a time scale of a few minutes. The major driving forces are momentum and buoyancy. In the ZID, large quantities of ambient water are entrained with the effluent causing the effluent plume to grow and become dilute. Because the effluent density is smaller than the ambient density, the plume rises towards the ocean surface. If the ambient density is uniform (unstratified conditions) the plume will grow until it reaches the ocean surface causing a large amount of initial dilution. If the ambient density is not uniform (stratified conditions) the plume may become trapped in a lower layer below the thermocline. This represents the most critical period for constituent concentration and settling. The level the plume rises to is often referred to as the trapped level. If the plume becomes trapped below the ocean surface, not as much ambient water will be entrained into the plume resulting in a lower initial dilution. Most state and federal water quality standards are required to be met at the end of the process. Near-field mixing depends on the following:

- Multi-port diffuser (location, port spacing, size, and orientation)
- Effluent properties (velocity and density)
- Receiving water characteristic (depth, stratification, velocity, and currents)

The model used by MWRA was EPA's ULINE which was based on work done by Roberts (1979). ULINE assumes that the diffuser acts as a line source within a three-dimensional flow field created by a diffuser of finite length in a moving current. ULINE accepts the following parameters: seawater and effluent density, current velocity, effluent flow rate, depth of effluent discharge, and diffuser size and alignment. It then calculates the initial dilution at the edge of the mixing zone and the trapped height of rise of the plume for stratified conditions.

The assumption that the effluent can be approximated as a line source are most appropriate under the following conditions (Roberts, 1979):

- low discharge rates
- weakly stratified conditions
- large density differences in effluent and seawater
- closely spaced risers
- discharges into deep water

A limitation of ULINE is that the assumption of a line source is not always valid particularly in the case of strongly stratified conditions. As in all mathematical models, the main limitation is in the quality of the input parameters.

ULINE's predicted monthly average dilution (Fig. 7-1) is greater than 100 for all months at sites 4 and 5 (Fig. 4-31). The predicted monthly height of rise of the effluent plume (Fig. 7-2) is above 15 m for 50% of the values calculated at sites 4 and 5.

The major conclusions that can be drawn from the near-field analysis are (MWRA V-A, 1988):

- The majority of the constituents of concern are not expected to exceed the water quality criteria at any of the sites for primary or secondary treatment.

- The deeper the site, the greater the dilution.

- pH requirements are met at all sites at all times.

- DO standards will seldom if ever fail to meet current standards of 6.0 mg/l (Table 7-1) at site 5.

- Coliform standards will always be met at sites 4 and 5 after chlorination (Table 7-2).

- Temperature standards will be met at all sites at all times.

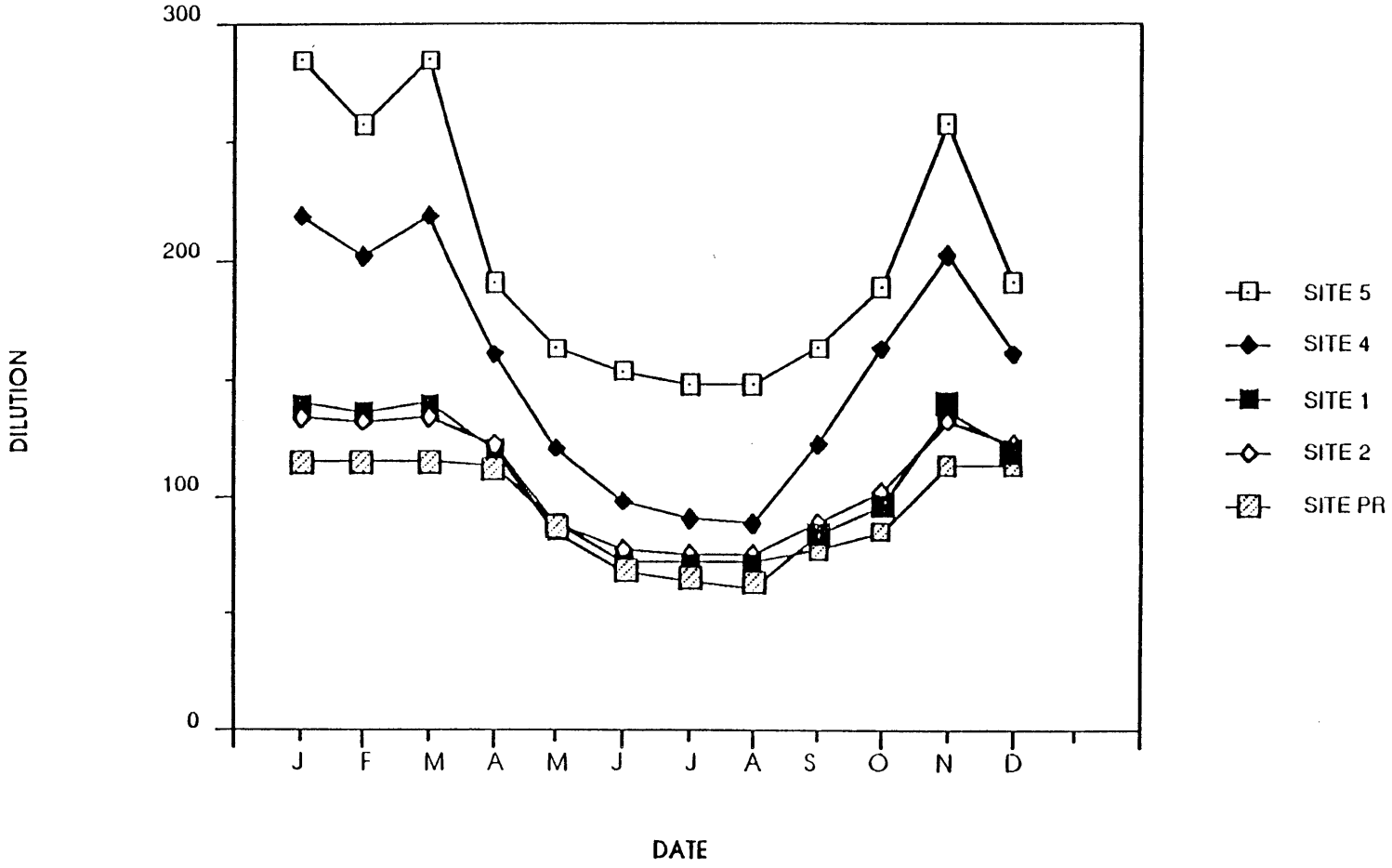


Figure 7-1: Predicted Monthly Average Dilutions for Sites President Roads (PR), 2, 3, 4, and 5 (MWRA Vol V-A, 1988)

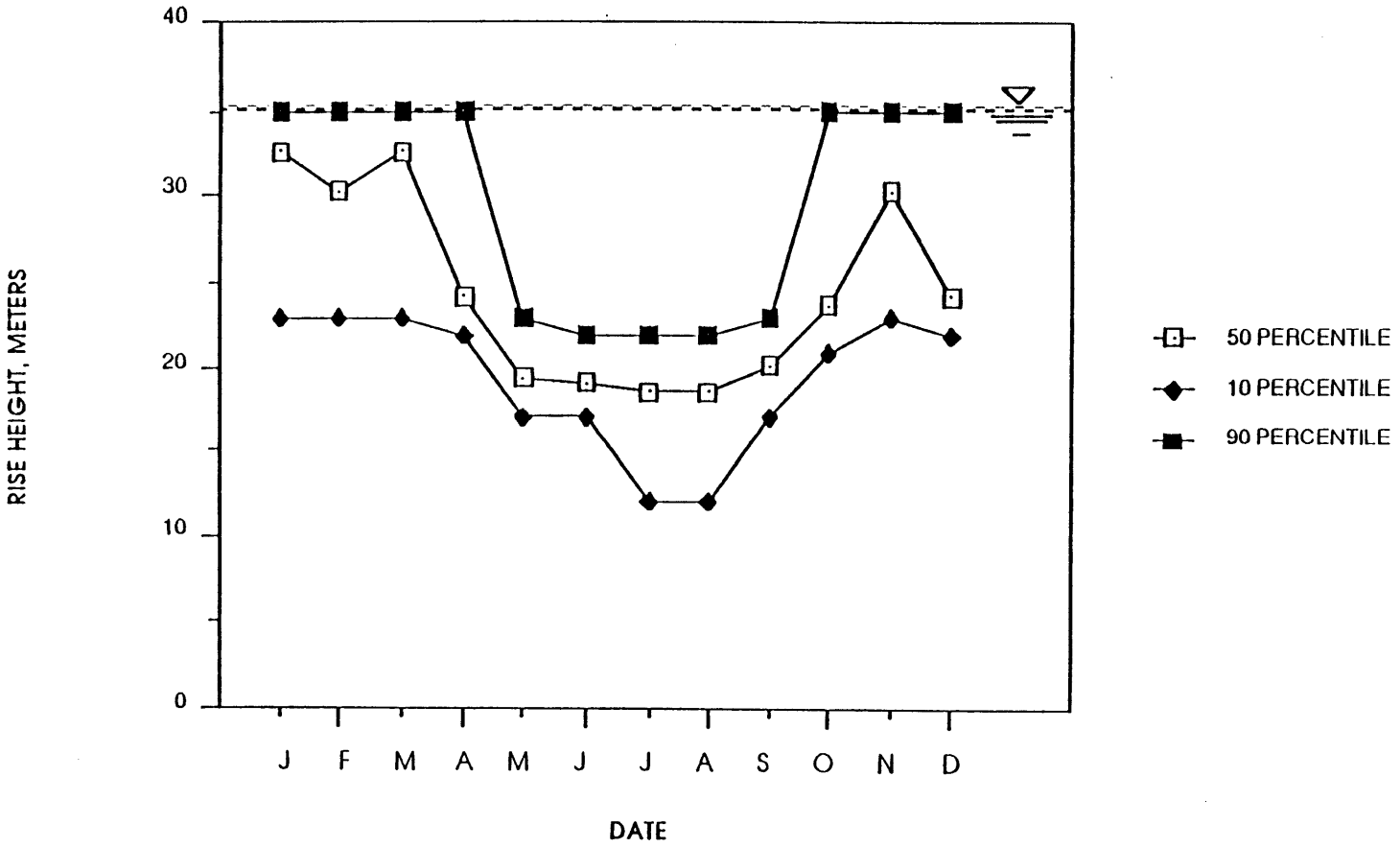


Figure 7-2: Predicted Monthly Plume Rise Heights at Site 5 (MWRA Vol V-A, 1988)

Table 7-1: Summary of Dissolved Oxygen Predictions (MWRA Vol V-A, 1988)

SUMMARY OF DISSOLVED OXYGEN PREDICTIONS

	Ambient (mg/l)	Far-field DO Deficit[1] (mg/l)	Minimum Water Column DO (mg/l)	Resuspension DO Deficit (mg/l)	Combined Worst Case DO (mg/l)
Primary Treatment Effluent					
Site 2	7.5	2.33	5.17	0.50	4.67
Site 3	7.5	2.60	4.90	0.40	4.50
Stie 4	7.5	2.10	5.40	0.32	5.08
Site 5	7.5	1.21	6.29	0.14	6.15
Secondary Treatment Effluent					
Site 2	7.5	1.09	6.41	0.15	6.26
Site 3	7.5	1.20	6.30	0.13	6.17
Site 4	7.5	0.97	6.53	0.10	6.43
Site 5	7.5	0.54	6.96	0.05	6.94
Alternate Plan Treatment Effluent					
Site 4	7.5	1.75	5.75	0.24	5.51
Site 5	7.5	1.15	6.35	0.13	6.22

Note: [1] Far-field DO deficit is due to CBOD, NBOD, and SSOD

Table 7-2: Comparison of Predicted to Required Dilution to Achieve Coliform Bacteria Standard (MWRA Vol V-A, 1988)

COMPARISON OF PREDICTED TO REQUIRED DILUTION TO ACHIEVE COLIFORM BACTERIA STANDARD

Site	Dilution Factor Required to Meet Standard (1)	Predicted Minimum Initial Dilution (2)	Probability of Achieving Necessary Dilution (2)
PR	28.6	24.87	99.93%
2	28.6	38.09	100%
3	28.6	26.99	99.91%
4	28.6	33.62	100%
5	28.6	40.08	100%

Notes: (1) Dilution required to reduce bacterial concentration from 400 to 14 per/100 ml

(2) Predicted by ULINE

Far-Field Modeling with TEA and ELA

The far-field mixing occurs over a much larger area and over a much larger time scale than the near-field mixing zone. In this mixing zone, the effluent is carried passively by the ambient current. The plume is dispersed slowly due to ambient turbulence and is transported due to large scale circulation patterns in Massachusetts Bay. The models used by MWRA to evaluate the far-field effects of the effluent discharged through the proposed outfall were MIT's TEA and ELA.

TEA (Tidal Embayment Analysis)

TEA is a two-dimensional, vertically integrated, finite element, circulation model that simulates water circulation in embayments in which the circulation patterns are predominantly tidal driven. The computer model works within the frequency domain as opposed to the time domain in order to take advantage of the periodic nature of the tide. This allows a much finer spatial resolution without the expense of small timesteps to satisfy stability conditions, making TEA cheaper to run than a program like CAFE. TEA solves the depth averaged forms of the Navier-Stokes equation and the continuity equation (Kossik et al., 1986). These equations are of the form:

$$\frac{\delta u}{\delta t} + \frac{\delta[u(h+\eta)]}{\delta x} + \frac{\delta[v(h+\eta)]}{\delta y} = 0$$

$$\frac{\delta u}{\delta t} + g\left(\frac{\delta h}{\delta x}\right) - fv - \frac{\tau_x^s}{\rho(h+\eta)} + \frac{\tau_y^s}{\rho(h+\eta)} + \left[u\frac{\delta u}{\delta x} + v\frac{\delta u}{\delta y}\right] = 0$$

$$\frac{\delta u}{\delta t} + g\left(\frac{\delta h}{\delta y}\right) - fv - \frac{\tau_x^b}{\rho(h+\eta)} + \frac{\tau_y^b}{\rho(h+\eta)} + \left[u\frac{\delta v}{\delta x} + v\frac{\delta v}{\delta y}\right] = 0$$

u(x,y,t) is the x component of the depth-averaged velocity
v(x,y,t) is the y component of the depth-averaged velocity
h(x,y,t) is the surface elevation relative to the mean sea level
h is the mean sea level
ρ is the water density
τ^s_x and τ^s_y are applied surface stresses
τ^b_x and τ^b_y are bottom stresses
g is the gravitational acceleration, f is the Coriolis factor

The boundary conditions associated with these equations are of two types: elevations and normal fluxes.

The limitations of TEA are:

- It is depth averaged; therefore, it can not adequately model strongly stratified conditions and wind induced surface circulations.
- It ignores lateral viscosity; therefore, flow separation near land boundaries is not properly modeled.
- A constant density is assumed; therefore, it precludes simulation of density-driven currents.
- It assumes a hydrostatic pressure; therefore, short or intermediate length waves can not be modeled.

TEA should only be used to model well mixed, tidally dominated coastal embayments.

The major advantage of TEA is that it works in the frequency domain instead of the time domain taking advantage of the periodic nature of the tide. This allows for much finer spatial resolution without the expense of computational time, hence making it cheaper to use.

Input parameters for TEA are the geometry and bathymetry of the area of interest, forcing functions such as tide, wind, and steady currents, bottom friction factor, wind drag coefficient, and boundary conditions. It outputs the circulation as a function of space and time.

ELA (Eulerian-Lagrangian Analysis)

ELA is a two-dimensional, vertically integrated, finite element, mass transport model that uses the results from TEA as input in order to simulate the transport of a contaminant released into the embayment. ELA solves the depth averaged form of the mass conservation (advection-dispersion) equation for a passive pollutant in a turbulent flow (Kossik et al. 1986). It uses the Euler-Lagrangian method to solve these transport

equations by decoupling it into pure advection and pure dispersion components. This equation is of the form:

$$\frac{\delta c}{\delta t} + u\frac{\delta c}{\delta x} + v\frac{\delta c}{\delta y} = \frac{1}{h}\frac{\delta}{\delta x}\left[h D_{xx}\frac{\delta c}{\delta x} + h D_{xy}\frac{\delta c}{\delta y}\right] + \frac{1}{h}\frac{\delta}{\delta y}\left[h D_{yx}\frac{\delta c}{\delta x} + h D_{yy}\frac{\delta c}{\delta y}\right] + Q$$

c(x,y,t) is the depth-averaged concentration
u(x,y,t) is the x component of the depth-averaged velocity
v(x,y,t) is the y component of the depth-averaged velocity
h(x,y,t) is the depth
D_{xx}, D_{xy}, D_{yx}, and D_{yy} are dispersion coefficients
Q represents sources, sinks, and vertical boundary fluxes

The boundary conditions can be concentrations and normal fluxes.

Limitations of ELA are the same as those of TEA because of its two-dimensional nature. The advantages of ELA are:

- numerical diffusion is greatly reduced due to the Euler-Lagrangian approach
- increased resolution near the source is possible due to a "puff" algorithm
- there is a large computational savings.

Input parameters for ELA are the geometry and bathymetry of the area of interest, advective velocity field from TEA, dispersion coefficients, decay rate for pollution of interest, source location and strength, initial dilution, boundary conditions, timestep, and length of simulation. It outputs the concentration at every node for each timestep as a function of space and time. ELA can also be used to calculate sedimentation rates.

TEA uses a grid which consists of 1044 triangular elements and 628 corner nodes for the Secondary Treatment Facilities Plan (Fig. 7-3). TEA uses linear basis functions in its finite element solution. Whereas, ELA uses quadratic basis functions, which have three corner nodes and three side nodes associated with each element. ELA uses a grid which consists of 1044 triangular elements and 2039 nodes for the Secondary Treatment Facilities Plan.

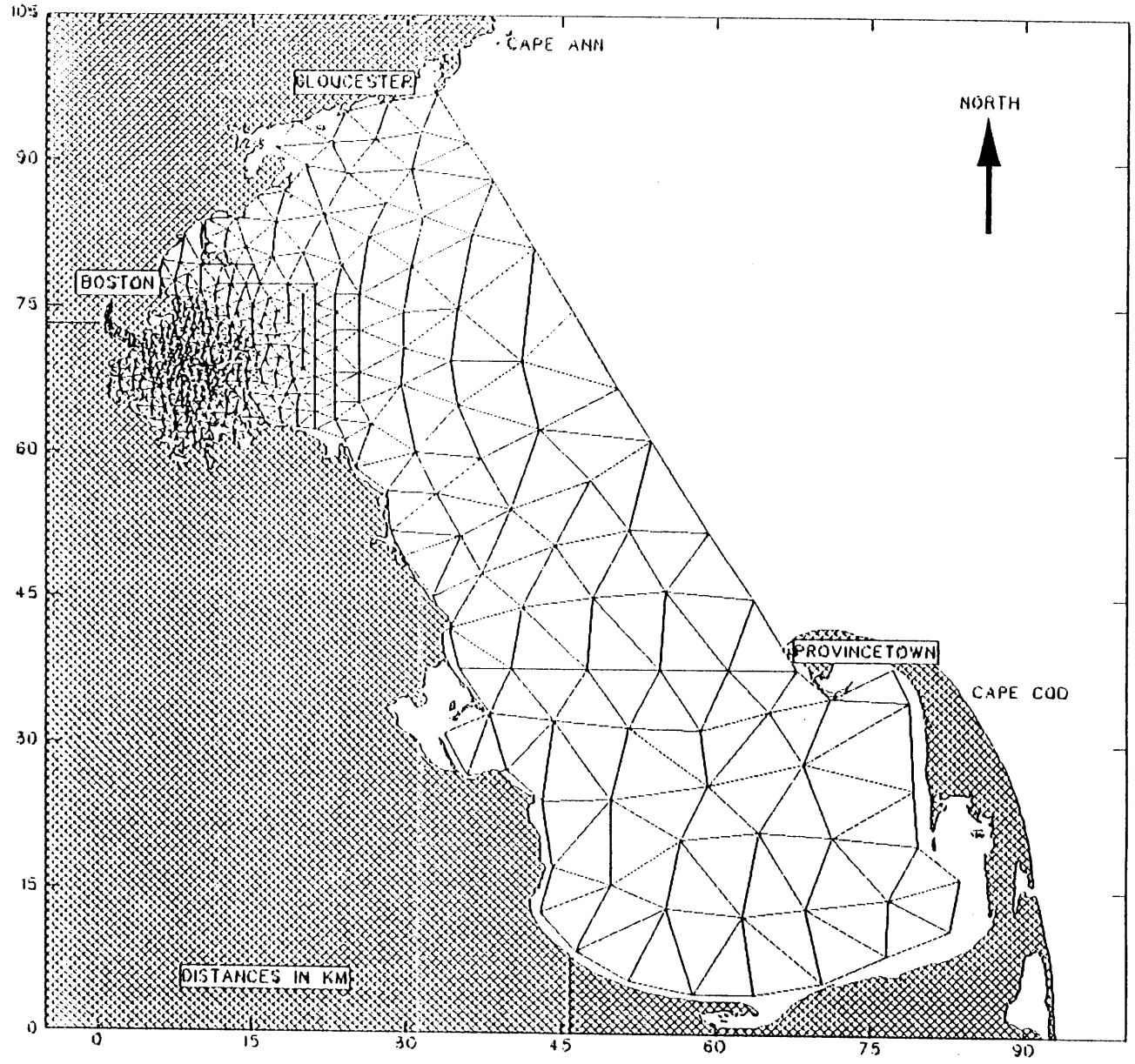


Figure 7-3: Finite Element Grid of Massachusetts and Cape Cod Bays (MWRA Vol V-A, 1988)

The major conclusion drawn from this modeling effort was that different circulation patterns exist near the outfall terminus and that there is a significant reduction in the concentration and sedimentation rates as the outfall moves offshore. Circulation patterns at the near-shore sites trend east-west and are strongly influenced by tidal action. The offshore sites exhibit circulation patterns trending north-south, which are strongly influenced by the large-scale circulation of Massachusetts Bay (Fig. 7-4). These differences result in variations in the direction of circulation and average dispersion capacity (largest at offshore sites).

Future

Sources of pollution in Boston Harbor can be attributed to wastewater effluent, wastewater sludge, combined sewer overflows, river inflows, and accumulation within the bottom sediments. In order to properly account for these sources of pollution, data must be collected within the area of interest along the complete vertical profile and within the sediments at specific times. This data needs to be collected for two purposes: to fulfill requirements in the NPDES permits and to be used as input into a predictive mathematical model. The data required for use in the predictive model is much more extensive than that required by a permit. Therefore, it is essential to understand the parameters necessary to run the model and the interactions occurring among the important parameters in the area. Understanding the interactions is important because they aid in determining the model's sensitivity to changes.

Hydrodynamic Model

In the coastal zone, net transport in the horizontal direction is accomplished by tidal movement, waves, currents, wind, freshwater inflows, and local topography. These transport processes also affect vertical transport or restriction of transport within the water column. Other factors that affect vertical transport are salinity and temperature gradients. These gradients affect density which in turn drives vertical circulation (Chapter 1). Besides affecting the hydrodynamics, temperature is also a key exogenous driving force in the behavior of phytoplankton. As seen in Chapter 1, the water column in Boston Harbor is strongly stratified in the summer. Wind plays a major role in the stratification and circulation processes in the Boston area.

In previous two-dimensional modeling efforts in the Boston Harbor (TEA and ELA), stratified conditions were modeled by decreasing the nodal depths to include only the depth below the thermocline. This did not allow for exchange between the epilimnion

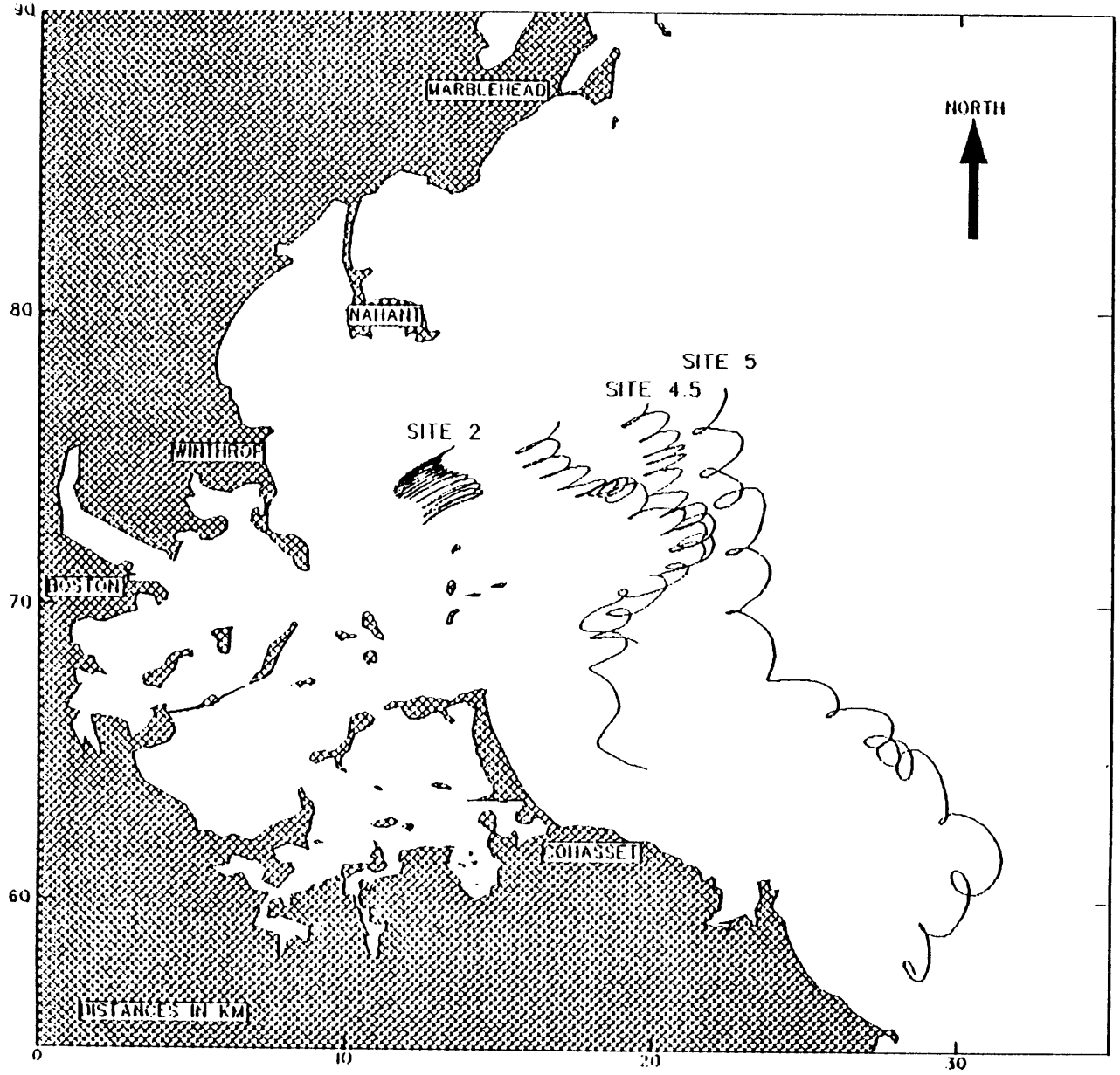


Figure 7-4: Simulated Particle Tracks Along Northern Transect (MWRA Vol V-A, 1988)

and hypolimnion and the effects of wind and freshwater inflows were not considered. These factors, which were ignored, are significant in driving the vertical density variations in the harbor. To properly describe the complex hydrodynamics of Boston Harbors coastal ecosystem, a three-dimensional, time-varying, hydrodynamic and water quality model should be used.

The hydrodynamics model should be coupled with a water quality (eutrophication) model in order to describe the temporal and spatial variations in the water quality parameters of Boston Harbor. The water quality model should consider nutrient kinetics, oxygen dynamics, anoxia, productivity, point and non-point loading, and the water/sediment interface.

Water Quality Model

Previously, only the fate and transport of specific nutrients and bacteria has been modeled in the harbor. To date, no attempt has been made to model eutrophication. Eutrophication is associated with:

- increased productivity
- structural simplification of the biota
- reduced stability (ie. a reduction in the ability of certain plants and animals to adapt to imposed changes)
- high nutrient levels
- high organic productivity

A eutrophication model would show year-round variations in nutrients, phytoplankton, and dissolved oxygen. Primary productivity changes near the vicinity of the outfall could be determined. Most importantly, the beneficial versus detrimental effects of primary versus advanced primary versus secondary treated effluent discharges could be assessed. The eutrophication model would provide information on the limiting nutrient discharged into the system, such as nitrogen or phosphorus. Then control measures can be focused on these nutrients.

Currently, there is a project to develop a coupled, three-dimensional, time-varying, hydrodynamic and water quality model for Chesapeake Bay (Hydroqual, Inc., 1987: U.S. Army Engineering Waterways Experiment Station, 1988). The hydrodynamic model is a finite difference model that allows the vertical turbulent eddy coefficient to be entered in

several ways. It also incorporates hydrographical and meteorological information such as freshwater inflow, tide, current, wind, and atmospheric pressure. The model then calculates the water elevation, three-dimensional flow velocity, salinity, and density. Linked to this model is a sediment transport model which calculates sediment concentration in the water column, sediment entrainment, and sediment deposition.

The hydrodynamic model is coupled to a water quality model which will include the following 23 state variables:

- Diatoms
- Cyanobacteria
- Other phytoplankton
- Dissolved organic phosphorus
- Particulate labile organic phosphorus
- Particulate refractory organic phosphorus
- Dissolved inorganic phosphorus
- Dissolved organic nitrogen
- Particulate labile organic nitrogen
- Particulate refractory organic nitrogen
- Ammonium
- Nitrate + Nitrite
- Available silica
- Unavailable silica
- Dissolved organic carbon
- Particulate labile organic carbon
- Particulate refractory organic carbon
- Dissolved oxygen
- Total zooplankton
- Chemical oxygen demand (sulfide)
- Temperature
- Salinity
- Inorganic suspended solids

In order to incorporate all of these state variables into a model, a clear understanding of the interactions of these variables and a detailed monitoring program to accompany the model is essential.

The experience that has been and will be gained on Chesapeake Bay can be transferred to the Boston Harbor. Although, a proper monitoring program must be implemented for Boston Harbor in order to obtain the necessary data for the model and to determine the role of each of these state variables in the harbor's water quality.

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Chapter 8

Alternatives to Secondary (Activated Sludge) Treatment

Alternatives to Secondary (Activated Sludge) Treatment

Introduction

This section addresses viable and innovative treatment alternatives, the construction schedule imposed on MWRA, and the EPA imposed requirement for full secondary treatment of MWRA wastewater.

Construction Schedule

In April of 1988, MWRA and EPA issued facilities planning reports and priorities for the $6.1 billion "cleanup" of Boston Harbor. The planned schedule for the "cleanup" is as follows:

- By 1991, complete the short-term residual management facility.
- By 1995, complete the new primary treatment facility plus a 13 km to 16 km deep-rock ocean outfall and multi-port diffuser.
- By 1999, complete a secondary treatment facility.
- By 1999, complete a long-term residual management facility.

This construction schedule includes the elimination of sludge disposal into the ocean by December 1991. This will be accomplished by completing the short-term residual management facilities at the Fore River Staging Area in Quincy, which will eliminate one of the major sources of pollution in the harbor. Although, combined sewer overflows (CSO) causing raw wastewater to pour into the harbor near the shorelines are the primary cause of beach and shellfish bed closures and are a major cause of pollution (Resource Analysis, Inc., 1976). This schedule does not include the control of CSO; however, CSO are currently being studied by MWRA (see Chapter 6) and other interested parties, such as the Massachusetts Institute of Technology (Lee et al., 1989).

The present primary treatment plants at Deer and Nut Islands are not designed to handle the volume of flow that they receive. Therefore, the construction of a new primary

treatment plant is essential. The new plant and the associated improvements will eliminate much of the raw wastewater flow that reaches the ocean.

The 13 km to 16 km ocean outfall with the multi-port diffusers will allow the effluent to be carried far from the shores, and dilute the concentration so that the ocean environment can assimilate the effluent. From the data collected, it is unclear that the outfall would need to be extended this far offshore (Barker, 1989) with secondary treatment imposed.

With an 13 km to 16 km outfall, secondary treatment offers a minimal improvement in water quality over conventional primary treatment (Harleman, 1989a – see Appendix A1). Therefore, it seems reasonable to redirect the money that would be used on secondary treatment facilities into controlling the CSO problem. A more reasonable schedule might be as follows (Harleman, 1989b – see Appendix A2):

- By 1991, complete the short-term residual management facility.
- By 1995, complete a new primary treatment facility plus a 13 km to 16 km deep-rock outfall.
- By 1999, control CSOs.
- By 1999, complete a long-term residual management facility.
- Beginning in 1995, re-assess of the water quality in Massachusetts Bay to determine the need for additional treatment.
- If additional treatment is indicated, use advanced primary.

This schedule represents a better use of the ratepayers' money, a possible savings of $2.5 billion, and a cleaner harbor. If MWRA continues on its present schedule, the control of CSO will be delayed until the next century. This delay ultimately delays Boston Harbor's ability to cleanse itself. There are two drawbacks to the schedule set by Harleman. First, the court has already set forth and approved the schedule proposed by MWRA and EPA for the Boston Harbor "cleanup". Second, the right to appeal the tentative denial for a waiver from secondary treatment has expired. Therefore, it would

take an act of Congress to reverse the requirements of full secondary treatment of MWRA's wastewater (Folley, 1989).

Advanced Primary

Advanced primary treatment is a process that uses chemicals and/or polyelectolytes (polymers) in primary sedimentation basins to enhance the removal of the biochemical oxygen demand (BOD) and total suspended solids (TSS).

Chemicals are used to reduce the electrical field surrounding a particle, making it possible for these particles to come together and flocculate during mechanical agitation. This mechanical agitation must occur in order to properly mix the chemicals with the influent. Usually, this mixing is done with high speed propellers or turbines in mixing tanks with retention times of only a few minutes.

Polyelectrolytes are water soluble polymers which allow particles to readily adsorb onto them. One polymer can have several particles adsorbed to it forming a bridge between the particles. This produces stable flocs. These flocs settle carrying with them additional particles that become trapped below the flocs. Polymers can be anionic, cationic, or nonionic. From previous studies (Wilson et al., 1975; Ferguson and Vråle, 1984; Ooten and Heinz, 1986) the typical concentration of polymers used during advanced primary treatment ranges from 0.1 ppm to 1.0 ppm.

The choice of chemicals and/or polymers to be added to the wastewater greatly affects the quality of the effluent and the quantity of the sludge. For example, if a mixture of lime and ferric chloride is added to the sedimentation tanks, removal rates for TSS (87%) and BOD (80%) are similar to the 90% removal rates obtained using secondary treatment. A large amount of lime is required to obtain these removal rates and the process produces as much, if not more, sludge than secondary treatment. During the past several years, the use of polymers has increased. It has been shown that, with a very small amount of polymers, TSS removal rates as high as 90% can be obtained. Although, BOD removal rates are not nearly as high for advanced primary (50%) as for secondary treatment (90%), but more than primary treatment (35%). Due to the small dosage of polymers required, the quantity of sludge produced increases only slightly over primary treatment. The fact that TSS removals are high, BOD removals are moderately high, and sludge quantities are low, makes advanced primary treatment using polymers an attractive alternative to secondary treatment in coastal environments.

The remainder of this chapter addresses advanced primary treatment assuming the addition of ferric chloride and polymers.

Conventional pollutant removal rates (Fig. 8-1,2) indicate that advanced primary treatment has TSS removal rates comparable to secondary treatment (80% for advanced primary and 90% for secondary treatment) and has greater BOD removal rates than primary treatment (50% for advanced primary and 35% for primary treatment). These removal rates are based on ideal conditions. Below is a comparison of TSS and BOD removal rates for MWRA's new treatment facilities based on flows (mld) for primary, advanced primary, and secondary treatment:

	Low Groundwater (8 months)	High Groundwater (4 months)	Storm Events
Flow	1440	2540	4800
Primary[1]			
TSS	60	58	40
BOD	36	31	22
Advanced Primary			
TSS	80	80	80
BOD	50	50	50
Secondary[1]			
TSS	90	77	68
BOD	91	79	61

1 From MWRA Vol III, 1988.

As indicated above, primary and secondary treatment efficiencies are dependent on the flow or more specifically the overflow rate. The overflow rate is a design parameter that is defined as the flow rate divided by the surface area of the sedimentation tanks. Overflow rates for a selected number of coastal municipalities are shown in Figure 8-3. MWRA's overflow rates range from 3.1×10^4 lpd/m^2 to as high as 8.1×10^4 lpd/m^2. The higher overflow rate is similar to the West coast treatment facilities. This wide range of overflow rates makes operating a secondary treatment facility difficult, as indicated from the above table. Whereas, for an advanced primary treatment facility, in order to maintain a high

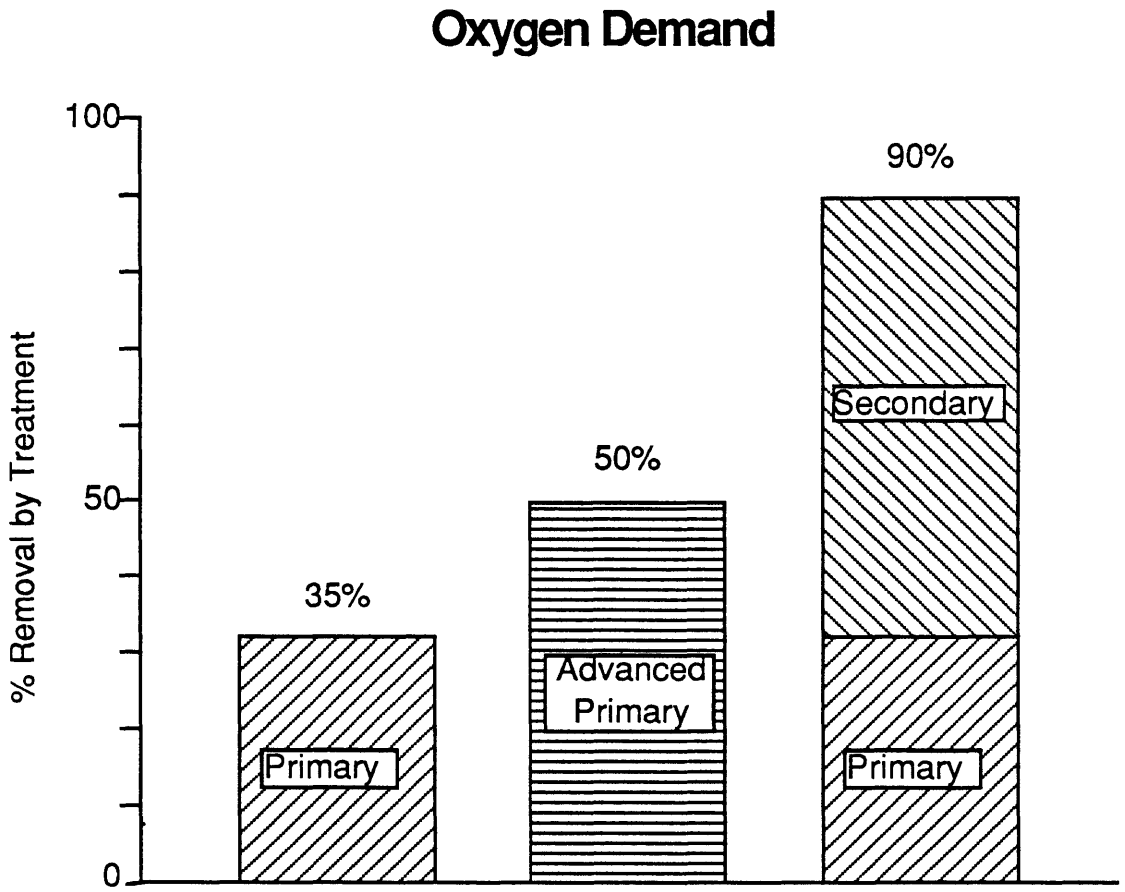

Figure 8-1: Idealized biochemical oxygen demand (BOD) removal rates for various treatment processes.

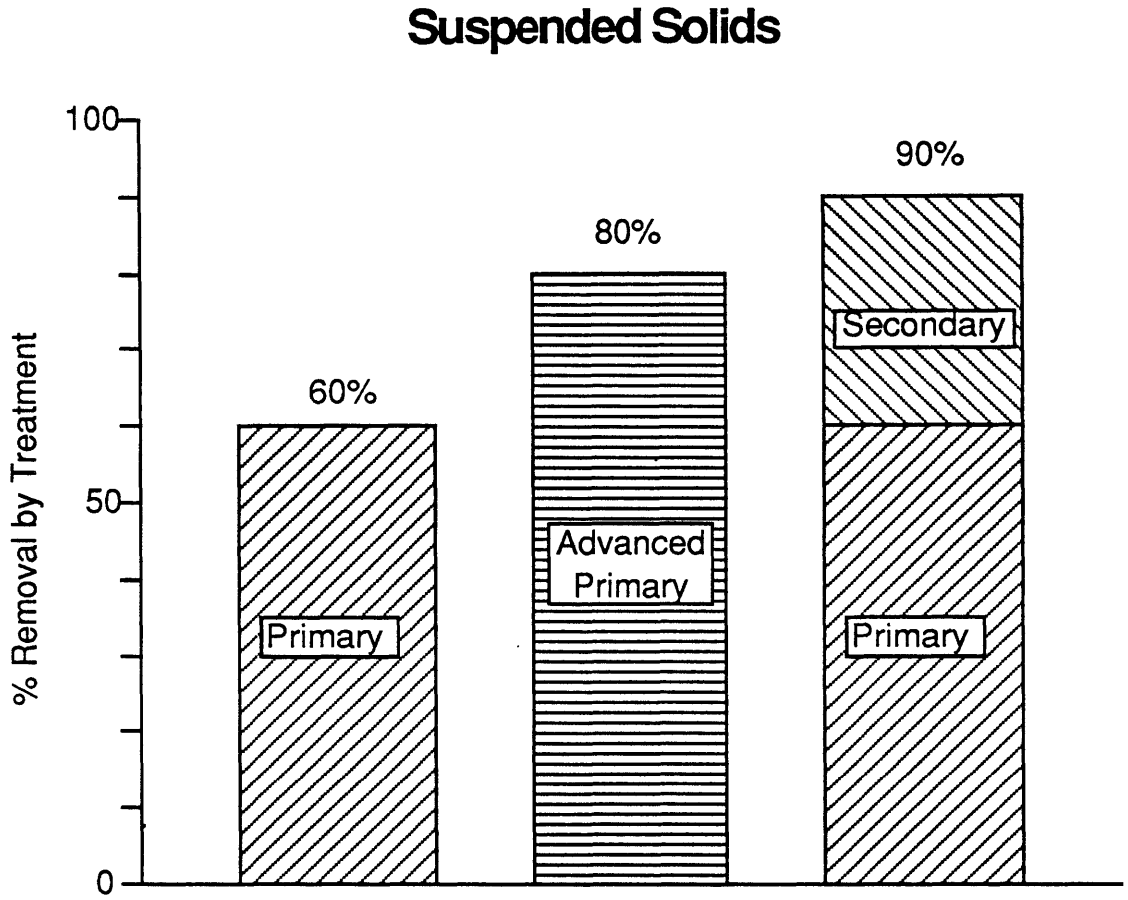

Figure 8-2: Idealized total suspended solids (TSS) removal rates for various treatment processes.

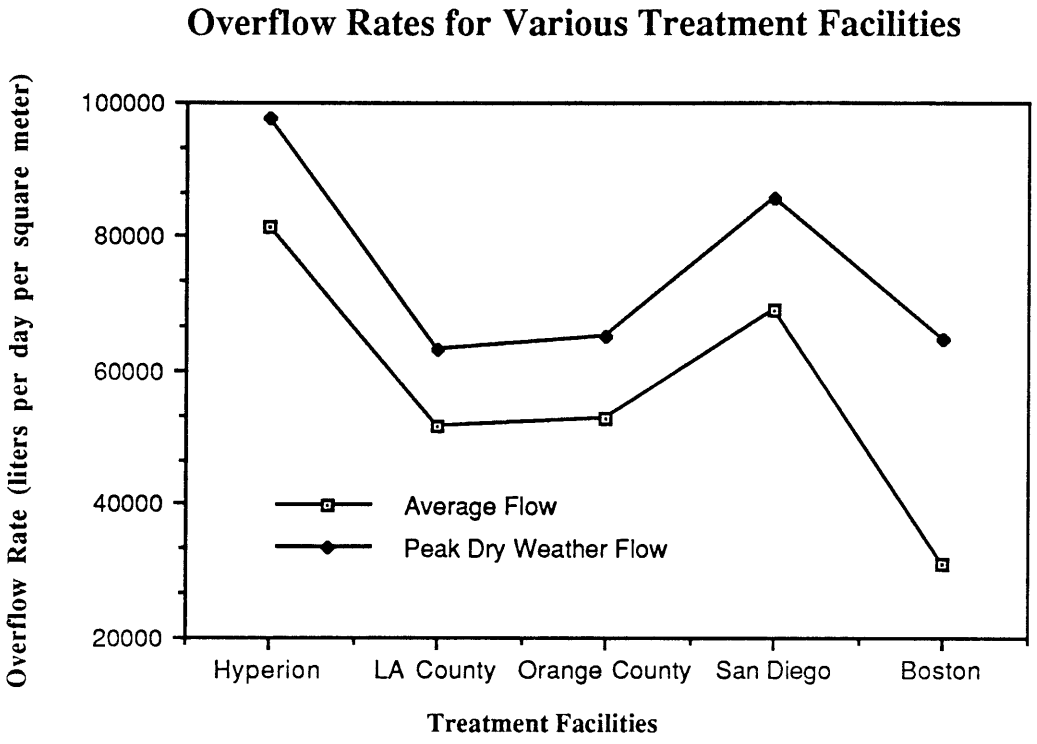

Figure 8-3: Overflow Rates Associated with Various Treatment Facilities.

removal rate, more chemicals/polymers would need to be added. In San Diego, where they have overflow rates of 5.3×10^4 lpd/m^2, they have removal rates of 80% TSS and 56% BOD.

The sludge produced (Fig. 8-4) from advanced primary treatment is much less than the sludge produced from secondary treatment. The sludge volume for secondary treatment is approximately twice that of primary treatment and about 60% to 70% greater than advanced primary treatment.

A comparison of the removal rates and sludge production indicates that advanced primary treatment is a viable alternative to secondary treatment. This is especially the case for effluent ocean disposal where BOD removal is not a major concern. The replenishment of oxygen occurs rapidly in the ocean due to large surface areas. Therefore, BOD removal is not critical in ocean environments. Another important difference in advanced primary versus secondary treatment is the quantity/quality of sludge produced. This has become especially important due to the onset of the newly proposed EPA regulations on sludge disposal (Federal Register, 1989). These regulations were set forth to protect public health from the adverse effects of certain pollutants that may be present in the sewage sludge. Requirements were established for final use and disposal of sewage sludge when it is applied to land, landfilled, incinerated, or distributed and marketed. Ocean disposal of sewage sludge is prohibited as of December 31, 1991 due to the Ocean Dumping Ban Act of 1988. Pollutant standards are given in terms of limits or equations to calculate the limits. Monitoring, record keeping, and reporting requirements will also be established. These proposed regulations are much more restrictive than the current EPA regulations on sludge disposal.

Facilities such as Los Angeles, Orange County, San Diego, and other West coast plants have used advanced primary treatment as an alternative to secondary treatment (Morrissey and Harleman, 1989 – see Appendix A4). Removal rates, chemical additives (type, amounts, and time durations), and flow rates for a number of plants are shown in Table 8-1.

Studies

Orange County conducted a detailed study (Ooten and Heinz, 1986) to determine the optimal type and dosage of chemicals to use for the enhancement of primary treatment. Costs associated with each of these chemicals are shown in Table 8-2. They determined

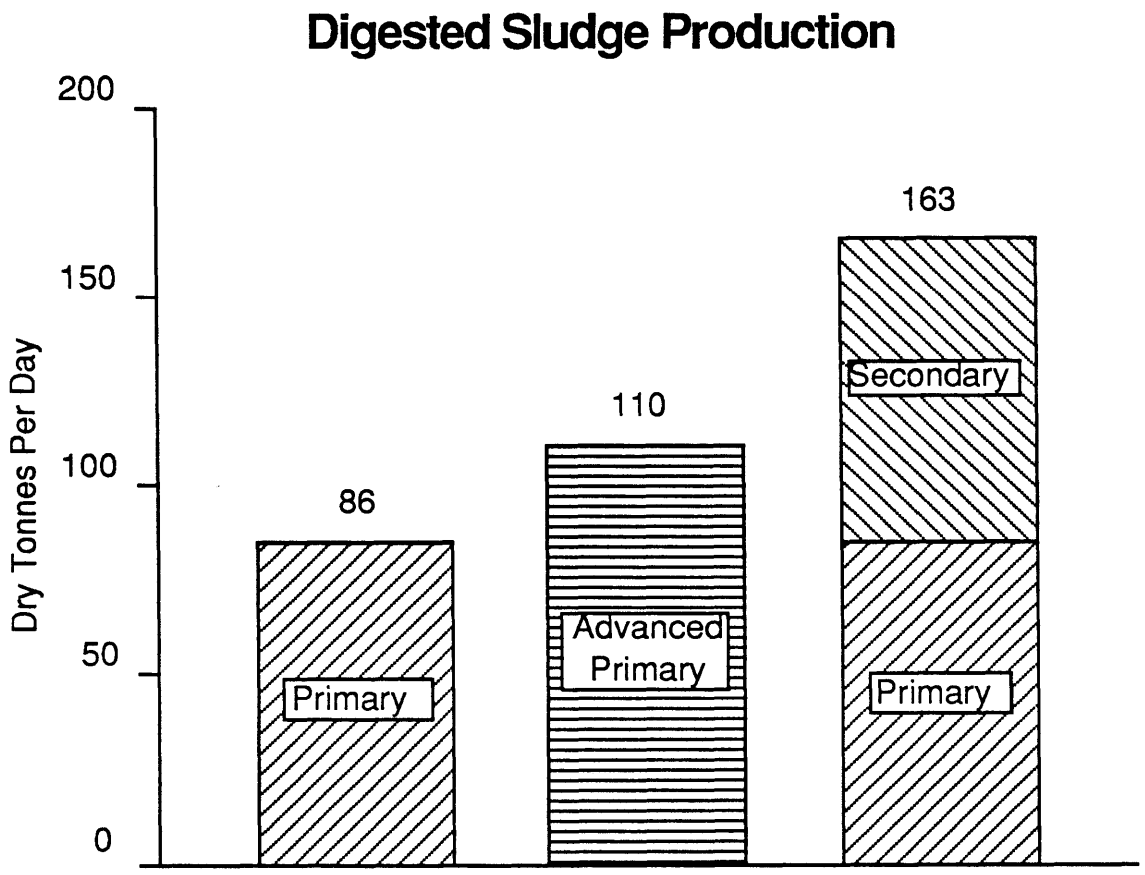

Figure 8-4: Digested Sludge production for the proposed treatment facility on Deer Island (Black & Veatch, 1987).

Table 8-1

Flow Rates, Removal Rates, and Chemical Additives Used at a Number of Advanced Primary Treatment Plants in California

Location	Flow	Removal Rates		Chemical Additives		Duration
	(mld)	BOD_5	TSS	Type	Amount	
City of Los Angeles (Hyperion)	1510	50%	83%	Ferric Chloride Anionic Polymers	20-25 ppm 0.25 ppm	continuous
Los Angeles County (JWPCP)	1360	47%	80%	Anionic Polymers	0.15 ppm	continuous
Orange County (Plant #1)	230	38%	65%	Ferric Chloride Anionic Polymers	20-25 ppm 0.25 ppm	8 hrs
Orange County (Plant #2)	760	47%	71%	Ferric Chloride Anionic Polymers	20-25 ppm 0.25 ppm	12 hrs
City of San Diego (Point Loma)	720	51%	80%	Ferric Chloride Anionic Polymers	35 ppm 0.26 ppm	continuous

Table 8-2: A comparison of costs and dosages for chemical used to enhance solids removals in primary sedimentation tanks

	LIME	IRON SALTS	POLYMER	ALUM
Dosage	100 ppm	20 ppm + polymer	0.1 ppm	100 ppm
$/Unit	.03/lb	.09/lb	2.64/lb	.09/lb
$/MGD	$25.00	$15.00	$2.20	$75.00
Effect on Odors	Undetermined	Positive	Unknown	Adverse
Capital Requirements	Major	Moderate	Minimal	Moderate
Maintenance Requirements	Major	Moderate	Minimal	Moderate
Effect on Operations Work Load	Major	Moderate	Minimal	Moderate
Effect on Sludge Digestability	Adverse	Unknown	Improved	Unknown
Present Volume Increase	1.7	1.5-2	Unknown	Unknown

that the addition of 20-25 ppm ferric chloride and 0.25 ppm anionic polymer increased the BOD removal to 48% and TSS removals to 83% in the primary clarifier. Besides the increased removal rates, benefits of chemical additions are: only a slight increase in sludge production, sludge digested normally, and a significant decrease (97%) in the production of sulfide gases during digestion.

A study conducted by the city of Tampa, Florida (Wilson et al., 1975) indicated similar results using 100-150 ppm alum and 0.6-0.7 ppm anionic polymer. This mix of chemicals increased the BOD removal to greater than 40% and TSS removals to as high as 90% in primary clarifiers.

In Seattle, Washington a study was conducted using a mixture of seawater and lime to enhance the removal of TSS, BOD, and phosphates (Ferguson and Vråle, 1984). This study indicated similar results using 200-2300 ppm lime and 10% seawater. This mixture increased the BOD removal to greater than 70% and TSS removals to as high as 90% in primary clarifiers. The principal disadvantages of this method are the high pH values in the sludge and effluent, and the large quantity of sludge produced.

As illustrated by these three examples, the choice of chemicals used influences the removal rates and the amount of sludge produced. All these studies indicated an increased removal of BOD and TSS with only a slight increase in sludge production. TSS removal rates are comparable to those obtained with secondary treatment. BOD removals increase over primary treatment but are significantly lower than those obtained using secondary treatment. Therefore, if effluent is to be disposed of in rivers or lakes where the surface area is relatively small and BOD levels are of greater importance, then advanced primary treatment as stated here may not be a viable choice for wastewater treatment. In ocean environments, on the other hand, it is quite acceptable (Harleman, 1989b; Balint, 1989).

Another aspect of treatment is the ability to remove heavy metals from the influent. Data from Los Angeles Hyperion plant (Shao, 1989), indicate that for 10 heavy metals, advanced primary treatment is able to remove 50% or more of seven out of ten of these metals (Table 8-3). This is similar to removal rates obtained for secondary treatment. Therefore, only slightly better sludge quality can be expected for advanced primary treatment over secondary treatment. Quantity may become a more important issue if EPA's proposed sludge regulations become law.

Table 8-3

Effects of Treatment Process on Metals Removal
Los Angeles California Hyperion Treatment Plant

Metal	Influent kg/ML	Adv Prim. Effluent kg/ML	% Removal	Sludge Conc. kg/dry tonnes	Secondary Effluent kg/ML	% Removal	Sludge Conc. Kg/dry tonnes
Arsenic	0.048	0.039	19	0.013	0.012	69	0.017
Cadmium	0.051	<0.029	~100	0.073	<0.029	–	–
Chromium	0.198	<0.058	~100	0.284	<0.058	–	–
Copper	0.687	0.268	61	0.600	0.047	83	0.140
Lead	0.315	0.210	33	0.149	0.186	11	0.014
Mercury	0.002	0.001	50	0.001	0.001	50	0.001
Nickel	0.198	0.146	26	0.074	0.128	12	0.011
Selenium	0.010	0.003	67	0.010	0.002	33	0.001
Silver	0.111	0.047	58	0.092	0.010	80	0.023
Zinc	0.926	0.402	57	0.755	0.233	42	0.106

All influent receives advanced primary treatment. Forty percent receives secondary treatment.

A Comparison of Secondary versus Advanced Primary Treatment

A comparison of secondary treatment and advanced primary treatment indicates that secondary treatment provides no significant advantages over advanced primary treatment especially when costs are considered in the coastal zone. This is substantiated as follows (Tegner, 1989). "I know of no evidence that secondary treatment will improve environmental conditions or public health standards where the discharge is into deep water, open coastal habitats." If a detailed comparison was performed, it might indicate that secondary treatment is more detrimental to an ocean environment than advanced primary treatment. This idea is expanded upon in the following paragraphs.

Secondary treatment is a biological process that reduces the biochemical oxygen demand (BOD) and total suspended solids (TSS). Microorganisms (bacteria) are used in secondary treatment to convert dissolved solids and small particulate organic material in the wastewater to suspended solids, which settle out in sedimentation tanks. The removal of BOD is the primary goal of secondary treatment.

The bacteria that aids in the removal of BOD propagate and feed on the waste material that enters the treatment plant. They are able to extract the available food energy, by oxidation, leaving behind oxidized particle material (often called ash because if lacks nutritional value) and simple dissolved compounds (often called mineralized compounds) (Frautschy, 1989). What is left behind extracts little oxygen from the water column (because the bacteria have already oxidized the organic matter). Although, it may have an effect on the benthic environment.

Simple dissolved compounds have no-food value for animals because, during the activated sludge process of secondary treatment, bacteria extracted the available food energy. Although, simple plants such as algae and plankton can readily use this mineralized form of nutrients.

Secondary treatment provides less nutrients for the propagation of fish (Revelle, 1989) and more mineralized compounds causing primitive plants such as algae and plankton to flourish. These primitive plants tend to accumulate causing an increased likelihood of blooms. The blooms inhibit the replenishment of oxygen to the water column, inhibit the ability of light to penetrate a given depth, and cause esthetic impairment to the receiving water.

Most organisms in the ocean feed opportunistically on small particulate organic material - most of which comes from the land. This is illustrated by the fact that the most productive areas of the ocean are in coastal waters and the deep ocean is desolate.

The use of advanced primary treatment would allow more particulate organic material to reach the ocean providing nutrients for animals.

A secondary treatment plant is more expensive to build and operate than an advanced primary treatment plant. To illustrate this point, the capital costs and operations and maintainence (O & M) costs for conventional primary, advanced primary, and secondary treatment processes are compared below. The data was based on information from MWRA's Secondary Treatment Facilities Plan (STFP), West Coast treatment facilities, and published articles on advanced primary treatment. Values are based on a treatment plant having the following characteristics:

- Flow 1820 mld
- TSS 150 mg/l
- BOD 150 mg/l

Capital costs associated with the processing of liquid and sludge are shown in Figure 8-5. There is a slight increase in capital costs associated with advanced primary over primary treatment. Although, the cost increases considerably for secondary treatment. It is reasonable to assume that the capital costs associated with sludge processing are proportional to the quantity of the sludge produced (Mueller and Anderson, 1983). For the liquid process, advanced primary treatment costs the same, if not less than, primary treatment. The increased costs associated with the small chemical tanks required to mix the chemicals with the effluent (with retention times of a few minutes) are offset by the ability to reduce the size of the sedimentation tanks while obtaining comparable removal rates.

The capital costs associated with the secondary treatment portion of the treatment facility for MWRA are estimated to be more than twice as much as for primary treatment alone. Whereas, the increase in capital costs associated with advanced primary treatment over primary treatment is only due to the increase in the quantity of sludge produced. The overall increase in capital costs associated with advanced primary is less than 10%. Dollar amounts for the liquid processing of wastewater are based on values found in MWRA's

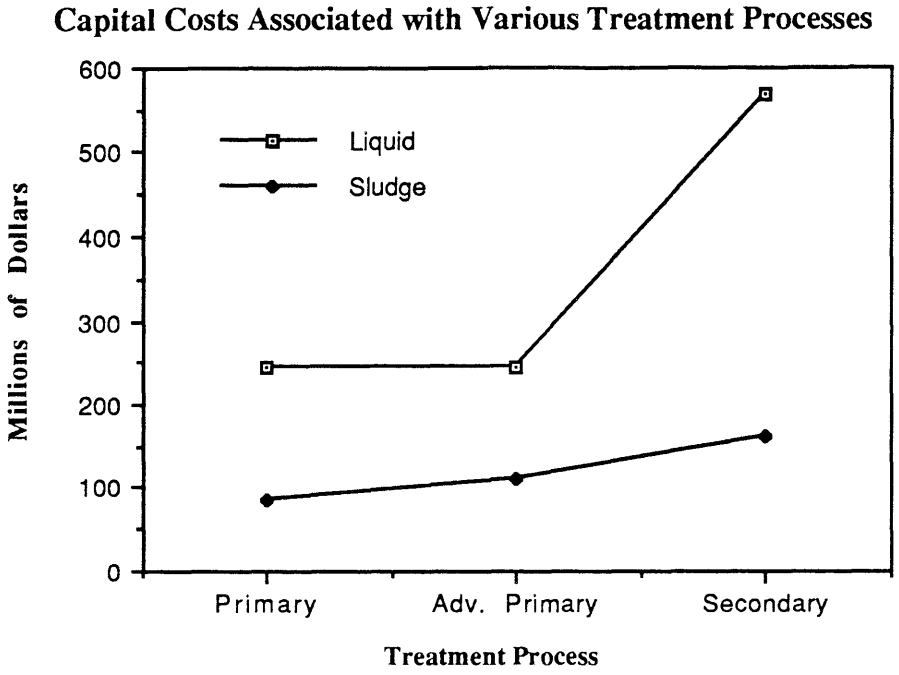

Figure 8-5: Capital Costs Associated with Various Treatment Processes.

STFP and reflect 1987 dollars including 35% for engineering and contingencies. The dollar amounts used for the sludge processing portion are based on a cost of $1 million per dry tonnes per day (dtpd) of sludge produced and reflect values found in a staff summary submitted by Paul Levy.

O & M costs associated with the processing of liquid and sludge are shown in Figure 8-6. For advanced primary treatment, the O&M costs for the liquid portion of the wastewater are directly related to the amount of chemicals/polymers used. Costs and dosages for different types of chemicals are shown in Table 8-2. The difference in O&M costs between advanced primary and primary treatment are about 50%, if we assume that the addition of chemicals constitutes the only difference in the costs (assuming that the dosage cost is $6.8 / mld (Duffy, 1988)). The O&M costs for secondary treatment for MWRA are estimated to be $7.35 million more than primary treatment. Cost estimates are based on 1987 dollars (MWRA, 1988) and include salary and wages, power, and equipement maintainence.

Secondary treatment affects a much larger land area (about twice the amount) than primary treatment. This area increase is due to the treatment plant facilities, sludge handling facilities, and sludge disposal requirements if a market is not available for the by-product. For the liquid processing, advanced primary treatment affects essentially the same land area as primary treatment. If we assume that land requirements for processing the solid portion of the wastewater are directly proportional to the amount of sludge produced, land requirements would increase 30% for advanced primary and 100% for secondary over conventional primary.

Both processes require that the effluent be chlorinated before releasing it to the ocean; hence, reducing the coliform count (a health standard indicator). There is a minimal difference in the coliform count from advanced primary to secondary treatment after chlorination.

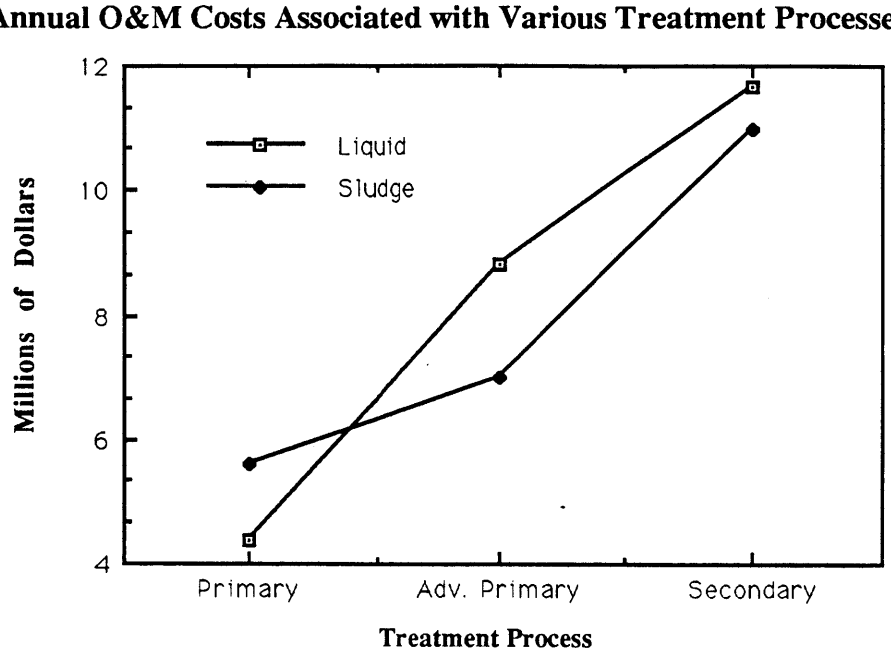

Figure 8-6: Annual O & M Costs Associated with Various Treatment Processes.

Mandating and enforcing uniform secondary treatment is administratively easier for the EPA; however, the following questions need to be addressed:

- Do the benefits of increased sludge production (assuming a beneficial reuse of all the sludge) outweigh the benefits of an increased fish population (for consumption)? In either case, a toxic waste source reduction program needs to be implemented.

- Is the ocean bottom area environment that is adversely effected by the effluent significantly different than the land area that is completely eliminated by the implementation of secondary treatment?

- How are the social effects of increased traffic associated with hauling the secondary residuals justified? How are the environmental impacts these trucks have on the air we breath justified? How are the environmental impacts of emissions associated with increased sludge production and the activated sludge process justified?

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Appendix A1

A Commentary on U.S. Environmental Protection Agency's Draft Supplemental Environmental Impact Statement (SEIS) on Boston Harbor Wastewater Conveyance System April 1988.

D. R. F. Harleman, May 25,1989.

A Commentary

on

U. S. Environmental Protection Agency's Draft Supplemental Environmental Impact Statement (SEIS) on Boston Harbor Wastewater Conveyance System April 1988

by

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May 25, 1988

1. Introduction

The U.S. EPA, Region I is to be congratulated for providing in the April 1988, Draft Supplemental Environmental Impact Statement (SEIS) on Boston Harbor Wastewater Conveyance System the technical information on the environmental impacts of effluents in Boston Harbor from both primary and secondary treatment under identical ocean outfall conditions. This opportunity arose, fortuitously, because of the construction time-table that provides for operation of the nine-mile ocean outfall, diffuser and the new primary treatment plant beginning in 1995; whereas, the new secondary treatment stage is not scheduled to go into operation until 1999. EPA explicitly considers the environmental impacts of the primary effluent only during the period 1995-1999. EPA also considers the environmental impacts of secondary effluent starting in 1999 but never makes the obvious comparison of the two treatment levels as options. Apparently, the excuse for ignoring this opportunity is that EPA requires both a primary and a secondary treatment plant be in operation by 1999. EPA has studiously imposed the regulatory uniformity of secondary treatment as a substitute for a general environmental assessment of various options for cleaning up Boston Harbor. The background for EPA's regulatory decision is concisely stated as follows:

> "The Clean Water Act requires that wastewater treatment plants be constructed which will provide 'secondary' treatment unless EPA, under strict statutory guidance, grants a waiver, under Section 301(h) of the Clean Water Act, permitting a less 'primary' degree of treatment with a deep ocean discharge. EPA has twice denied the MDC/MWRA request for such a waiver but final rights of appeal have not expired. EPA believes it is highly unlikely any such appeal, even if pursued, would prevail on the merits, or that the discharge of primary effluent into Massachusetts Bay would ultimately be permitted over the opposition of the Governor and other officials."[1]

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[1]Massachusetts Water Resources Authority. Secondary Treatment Facilities Plan, Volume V, Effluent Outfall, Final Report, March 31, 1988. Footnote, p3-37.

The present leadership of EPA, MWRA and the Office of the Secretary of Environmental Affairs of the Commonwealth have refused to discuss the question of whether there will be any environmental benefit of increasing the treatment of the effluent from the nine-mile ocean outfall from the primary to the secondary level. The basis for this refusal is the claim that the issue has been settled by EPA's 1985 denial of the secondary treatment waiver. What they have forgotten, or do not know, is that the waiver application was flawed from the outset by EPA's prejudice in favor of uniform secondary treatment. I quote from the rules and regulations establishing EPA's criteria for applying for the waiver of the secondary treatment requirement for discharges into marine waters:

> "Since the enactment of the 1972 amendments and the promulgation of EPA's secondary treatment regulations, a number of municipalities ... argued to both Congress and the EPA that secondary treatment is not necessary to protect the marine environment or to assure the attainment and maintenance of water quality in ocean waters. Those same municipalities contended that secondary treatment traditionally has been defined in terms of pollutant parameters and levels of pollutant reduction which are important for freshwater ecology where the discharge of oxygen-demanding waste and sedimentation of suspended solids results in distinct environmental degradation, but which have little significance for oceanic and saline estuarine waters where wastes are rapidly assimilated and dispersed by strong currents and tidal action. ... On this basis, these municipalities have maintained that they should be exempted from the Act's secondary treatment requirements, and the associated capital, maintenance and operating costs."[2]

EPA's bias against primary treatment is shown by the following:

> "EPA ... considers that primary treatment alone, with minimal suspended solids removed - even with a source control program - is environmentally inadequate for most municipal marine discharges. Thus a section 301(h) applicant seeking a modification based on only primary treatment will bear a particularly heavy burden in demonstrating to EPA that such treatment is sufficient to protect marine waters."[3]

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[2]EPA. Modification of Secondary Treatment Requirements for Discharges into Marine Waters. Federal Register Volume 44, No. 117, June 15, 1979. p.34784.

[3]EPA. Ibid., p 34797.

The most interesting section of the waiver rules and regulations is the following EPA response to public comments on the proposed regulations:

> "One commentator proposed that the demonstration required under this section be based on a comparison of the impact of the applicant's less-than-secondary discharge with a secondary effluent. EPA disagrees. Consistent with the Act (the Clean Water Act), its legislative history, and other sections of these regulations, the regulations promulgated today, like the proposed regulations, require the assessment under Section 301(h)(2) to be made on the basis of the actual or projected impact of the applicant's discharge, not in comparison to the impact of a secondary discharge."[4]

I can paraphrase this by saying that EPA, in not considering a comparison of the environmental impacts of primary and secondary effluent through the same ocean outfall, did not wish to be confused by the facts. In the years since the MDC submitted the waiver application in 1979 and the EPA's second denial in 1985, EPA was able to prevent this logical comparison. Even more damaging is the fact that EPA, in denying the waiver, applied water quality criteria that were much more restrictive than those used in the present environmental impact statement. Had the same water quality criteria been applied to the effluent from the secondary treatment plant, it too would have failed the EPA waiver test.

2. Environmental Impact Criteria

The EPA SEIS, published in April 1988, considers the consequences of primary effluent discharged through a 6600 ft. multi-port diffuser at the end of an ocean outfall during the interim period 1995-1999 and secondary effluent beginning in 1999 for various outfall lengths. It is the purpose of this commentary to make the comparison of the two treatment levels, as options, for site 5 which is approximately nine miles out in

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[4]EPA. Modification of Secondary Treatment Requirements for Discharges into Marine Waters. Federal Register Volume 44, No. 117. June 15, 1979. p34806.

the ocean from the present Deer Island outfall in about 100 feet depth. Site 5 is within the area recommended by EPA and MWRA for the diffuser location.

The SEIS presents in Table 1[5], a comparison of the data for worst case conditions regarding each of the criteria for evaluating the compliance of primary and secondary effluents. A modification of this table is presented in Table DH-1. The only difference between Table DH-1 and EPA's Table 1 is that data for the shorter outfalls (Sites 2 and 4) are omitted. In addition, criteria which showed no difference between primary and secondary effluents at Site 5 are omitted.

The criteria may be grouped in three categories:

1. Water quality dissolved oxygen standards for conventional pollutants.
2. Aquatic life and public health criteria for non-conventional pollutants.
3. Sediment enrichment and sediment toxicity for non-conventional pollutants.

The SEIS indicates that "impacts [of interim primary discharge] at Sites 4 and 5 are comparatively small and similar to each other and more importantly, they are reversible and therefore acceptable over the five-year period."[6]

On the face of it, the above statement would seem to be an endorsement of primary effluent, especially since secondary treatment waivers, if granted, require extensive monitoring to insure compliance and must be renewed at five-year intervals.

The worst case data summarized in Table DH-1 shows the primary effluent in a less favorable light in comparison with secondary; however, it is relatively easy to use the data contained in the EPA SEIS to analyze whether these differences are in fact environmentally significant.

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[5]Table 1. Outfall Site Comparison, EPA. SEIS. Executive Summary.

[6]EPA. SEIS. Executive Summary. p8.

Table DH-1

Comparison of Primary and Secondary Effluents

for Ocean Outfall at Site 5*

CRITERIA measure	SECONDARY	PRIMARY
Water Quality Standards		
min. DO (mg/l)	6.4	5.7 (sediment resuspension)
Aquatic Life Criteria		
number exceeding	0	4 (pesticides (2) + PCB, mercury)
Public Health Criteria		
number exceeding 10^{-5} risk	2 (arsenic, PCB)	4 (pesticides (2) + arsenic, PCB)
Sediment Enrichment		
km^2 degraded	0	0.05**
km^2 changed	3	12
Sediment Toxicity		
km^2 of effect	0	2
PCB area > .1 ppm	4	10

* adapted from Table 1: Boston Harbor Wastewater Conveyance System. Executive Summary, EPA Draft SEIS, April 1988.

** This is incorrectly given as 1 km^2 in Table 1 of the SEIS. The correct value of 0.05 km^2 is given on page 5-36 of SEIS, Volume I.

2.a Settling Velocities for Effluent Solids

The EPA's evaluation of primary and secondary effluents for three of the five criteria in Table DH-1 (water quality, sediment enrichment and sediment toxicity) are heavily influenced by their assumption of the distribution of discharged solids fall velocities for primary and secondary effluents.

Table DH-2

Comparison of Discharged Solids

Fall Velocity Distributions Used by EPA and MWRA

Percent of Total Solids Settled

	EPA[7]		MWRA[8]	
Fall Velocity cm/sec	Primary	Secondary	Primary	Secondary
0.1	5%	0%	0%	0%
0.01	20%	16%	20%	16%
0.001	35%	34%	30%	34%
≤.0001[9]	40%	50%	50%	50%

Table DH-2 compares the fall velocity distributions used in the recent effluent modeling studies by EPA (1988) and MWRA (1988). For secondary effluents, EPA and MWRA are in total agreement for the percentage of solids having fall velocities in the range of 0.1 to 0.0001 cm/sec. EPA and MWRA disagree for the primary effluent. The significant difference being that EPA assumes that 5% of the primary effluent solids have fall velocities of 0.1 cm/sec, whereas MWRA assumes 0% for this category, i.e. all

[7]EPA. SEIS Volume I, 1988. p 5-5.

[8]Massachusetts Water Resources Authority. Secondary Treatment Facilities Plan, Volume V, Effluent Outfall, Final Report, March 31, 1988. Appendix A. 1988. p3-62.

[9]Solids in this range do not settle.

particles have smaller fall velocities. EPA gives three references[10] to justify its fall velocity distribution for the primary effluent. Only the first reference, EPA (1982b)[11], which recommends fall velocities when settling tests are not performed gives any support for the 5% fall velocity of 0.1 cm/sec. The second reference, Cardoni, et al (1986)[12], conducted tests at Metcalf and Eddy on both sludge and primary effluent from Deer Island. Their extrapolation of the primary effluent test results to field conditions (Figure 8) shows that all fall velocities are less than 0.1 cm/sec. There is no valid argument that the higher fall velocities are a result of flocculation after discharge from the diffuser. The suspended solids concentration in the primary effluent is 55 mg/l, assuming a diffuser dilution of the order of 150, the concentration of suspended solids at the edge of the near-field would be 0.4 mg/l. This is only 10% of the average ambient suspended solids concentration of 4.5 mg/l.[13] Flocculation is not effective at these low concentrations. The third reference is the MWRA STFP report[14] which assumes no particles in the 0.1 cm/sec range.

The fact that EPA has overestimated the high fall velocity range for the primary effluent is further verified by calculating the overflow rate of the primary clarifier proposed for the new Deer Island primary treatment plant. The annual average treatment plant flow rate is 480 mgd and the surface area of the new primary clarifier is

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[10]EPA SEIS, Volume I. 1988. p5-5.

[11]United States Environmental Protection Agency. Revised Section 301(h) Technical Support Document. Government Printing Office, U.S. EPA. Washington, D.C. 1982b.

[12]Cardoni, J. J., D. R. Bingham, and N. D. Baratta. Determining Settling Velocities in Seawater, presented at the Water Pollution Control Federation Conference. 1986.

[13]EPA. SEIS. Volume II. 1988. p A-35.

[14]Massachusetts Water Resources Authority. Secondary Treatment Facilities Plan, Volume V, Effluent Outfall, Final Report, March 31, 1988.

636,500 ft^2. This is an overflow rate of 754 gal/day-ft^2 or 0.035 cm/sec. According to generally accepted primary clarifier theory[15], all particles having fall velocities greater than .035 cm/sec are removed. (Even at the peak flow rate of 1270 mgd., particles having fall velocities of 0.1 cm/sec will be removed.) It is therefore concluded that EPA's inclusion of 5% of solids having fall velocities of 0.1 cm/sec cannot be supported by the facts. The effect of this small difference on maximum sediment deposition rates is large as shown in Table DH-3. The fact that EPA's maximum sediment deposition rate is six times larger than MWRA's is primarily due to EPA's higher fall velocity distribution. It is interesting to observe that EPA's DO deficit due to sediment resuspension is also six times larger than MWRA's for the same reason.

2.b Water Quality Standard - Minimum Dissolved Oxygen (See Table DH-1.)

EPA's worst case DO impact for a primary effluent is identified in Table DH-1 as 5.7 mg/l in contrast to 6.4 mg/l for secondary effluent. Inasmuch as the Commonwealth standard for DO is 6.0 mg/l, this would appear to be a significant "black-mark" against primary effluent. However, EPA's bias in favor of secondary treatment again comes to the fore in setting the minimum ambient DO at the outfall site. As shown in Table DH-3, MWRA (1988) states that the minimum ambient DO is 7.5 mg/l, whereas EPA sets it at 6.5 mg/l. EPA's violation of the DO standard is calculated by subtracting the inflated resuspension DO deficit of 0.8 mg/l from the ambient of 6.5 to yield 5.7 mg/l (see Table DH-1).

The final blow to EPA's claim that the primary effluent will violate the DO standard is contained in the following quotation which describes the worst case scenario that EPA constructed for the crucial sediment resuspension event:

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[15]Metcalf & Eddy, Inc. Wastewater Engineering Treatment: Treatment, Disposal, Reuse. 2nd Ed. McGraw-Hill. 1979. p 205.

"The smallest minimum DO concentration is generally obtained for primary effluent, in stratified, worst net drift conditions, after a resuspension event. These values can be obtained only once per year, at the end of the summer and assume the following combination of events: no resuspension event during 90 days in the summer, followed by a 10 day period of zero net drift, followed by a resuspension event. Given the limited expected duration of primary discharge, it is likely that such an event will never be experienced."[16]

Table DH-3

Comparison of Sediment Deposition Rates and Sediment Resuspension DO Deficits for Primary Effluent at Site 5.

	EPA	MWRA
max. sediment deposition rate (g/m^2 -day)	1.9 (SEIS, p A-77)	0.34 (STFP, V.A. p6-29)
DO deficit due to sediment resuspension (mg/l)	0.8 (SEIS, p5-22)	0.14 (STFP, V.A. p5-108)
min. ambient DO (mg/l)	6.5 (SEIS, p5-22)	7.5 (STFP, V.A. p5-108)

[16]EPA. SEIS. Vol. I. 1988. p 5-23.

2.c Aquatic Life Criteria (See Table DH-1.)

The aquatic life criteria are based on a screening of toxic effects of more than 50 non-conventional pollutants. Of these, four failed the test for the primary effluent. Of the four, two are pesticides and one is PCB, none of which are presently detected in the influent to the Nut and Deer islands treatment plants. The one remaining, mercury, was predicted by MWRA to exceed criteria for both primary and secondary effluents.[17]

2.d Public Health Criteria (See Table DH-1.)

The public health criteria are less significant than the aquatic life criteria. Arsenic and PCB fail the criteria for both primary and secondary effluents, the remaining two that fail for primary only are the same pesticides that are not presently detected in the Boston influents.

2.e Sediment Enrichment and Sediment Toxicity (See Table DH-1.)

The differences in primary and secondary effluents for sediment enrichment and sediment toxicity are insignificant. EPA's values are biased against the primary effluent because of the inclusion of the higher particle fall velocities for the primary effluent.

3. Conclusions

My conclusion, based on the analysis of EPA's SEIS, is that there is no difference in environmental impacts for the Site 5 ocean outfall between primary and secondary effluents that could justify the construction of the secondary treatment stage.

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[17]MWRA. STFP, Vol. V., 1988. p8-22 and p8-31.

We must now consider the fact that whatever enters a treatment plant leaves the treatment plant, either as effluent through the ocean outfall or as sludge. The EPA SEIS conveniently considers only the environmental impacts of the effluent. Given that the impacts on Massachusetts Bay, whether of primary or secondary effluents, are not significantly different, we are forced to consider the fact that the addition of the secondary treatment stage will generate twice as much sludge[18] and that this presents an additional major political, social, and economic land disposal problem. It is obvious that the overall land/water environmental impact of the secondary treatment facility is negative.

My question to the EPA is: **How can you justify the expenditure of more than a billion dollars of Massachusetts tax-payer's money on a secondary treatment stage and its associated sludge disposal facility?**

This question cannot be satisfactorily answered by stating that it was settled by the EPA's denial of the secondary treatment waiver in 1985. The waiver application never allowed a comparison of primary and secondary effluent through the same ocean outfall. In addition, different but more restrictive criteria were used for that denial.

EPA's Summary of Findings included only one quantitative measure that led to their denial of the secondary treatment waivel:

> "The proposed discharge [of primary effluent] is expected to violate the Commonwealth of Massachusetts' water quality standard for dissolved oxygen during summer resuspension events, but is not expected to violate the Commonwealth's standard for suspended solids."[19]

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[18]MWRA. STFP, Vol. III, 1988. p11-48.

[19]EPA. Region I. Analysis of the Revised Section 301(h) Application of the MDC. Boston, MA. Signed by Michael Deland, March 29, 1985. Summary of Findings. p 4.

In order to justify its denial of the waiver on the basis of a DO standard violation, EPA Region I requested its national waiver review contractor[20] to assess impacts against a hypothetical ambient DO profile. This profile was obtained by selecting the worst DO levels at each depth regardless of the date or location at which the measurements were made. The DO measurements made near the outfall site in the summer of 1978 were the lowest and these were chosen to construct the hypothetical "ambient" DO profile in the following manner:

> "Using the worst case 1978 profile, with the associated ambient bottom dissolved oxygen concentration of 6.2 mg/l over the bottom 10.5 meters [and] the vertical average ambient dissolved oxygen concentration for the top 25.5 meters of the profile of 6.0 mg/l...."[21]

Thus, the weighted average hypothetical "ambient" DO for the entire 36 meter water column was calculated to be 6.06 mg.l! With this ambient DO both the primary and secondary effluents would violate the standard of 6.0 mg/l. This approach is technically unsound and inconsistent with wavier review methodology used by EPA in evaluating other waiver applications nationwide. It should be noted that the Boston's original waiver application was for an average flow of 485 mgd, and a 9.2 mile ocean outfall having a 6900 ft. diffuser, essentially the same as the facility described in the 1988 EPA SEIS and MWRA STFP documents.

The fact that EPA is presently operating under the secondary treatment mandate in no way relieves it of the responsibility to assess the environmental and economic consequences of its actions. Between now and 1995, the scheduled time for beginning

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[20]Tetra Tech, Inc., Addenda to the Technical Review of Boston's Deer Island and Nut Island Sewage Treatment Plants Section 301(h) Application for Modification of Secondary Treatment Requirements for Discharge into Marine Waters for U.S. Environmental Protection Agency. March 1985.

[21]EPA. Region I. Analysis of the Revised Section 301(h) Application of the MDC. Boston, MA. Signed by Michael Deland, March 29, 1985. Summary of Findings. p 17.

the construction of the secondary treatment stage, there is ample time for mandates to be changed provided that there are compelling reasons to do so.

Fortunately, EPA has left the door open for the appeal of the waiver. If EPA will not recognize the folly of insisting on secondary treatment, then MWRA, armed with the available evidence, should enter the open door and exercise its right of appeal. The money saved by eliminating the unnecessary secondary treatment stage can be better spent in solving the combined sewer overflow problem in the inner harbor and by improving the control of toxic and hazardous materials at their sources.

Appendix A2

Boston Harbor Cleanup: Use and Abuse of Regulatory Authority? Journal of the Boston Society of Civil Engineers Section/ASCE. Vol 4, No 1. pp. 25-32 D. R. F. Harleman, 1989.

BOSTON HARBOR CLEANUP:

USE OR ABUSE OF REGULATORY AUTHORITY?

by

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Boston Harbor is frequently described as the "dirtiest" in the country. During the presidential campaign the Harbor was the subject of heated, if not enlightened, debate on whether its condition and the delay in cleaning it up was the fault of federal or state and local government. Regardless of who is at fault, the public has been led to believe that a logical set of plans and priorities for the cleanup of Boston Harbor is finally in place. The present court–ordered schedule calls for the construction of a new primary treatment plant, interim sludge disposal facilities and a nine–mile ocean outfall by 1995. Additional treatment in a secondary stage as well as facilities for the land disposal of all sludge is required by 1999. The estimated cost of the new facilities is 6.1 billion dollars. Boston area residents will pay the highest water and sewer rates in the nation largely because EPA's construction grant program, which previously would have paid 75% or more of the cost, has now been phased out of the federal budget.

Do the present court–ordered facilities and the schedule for the cleanup represent an environmentally sound solution for Boston Harbor? The answer is no. To understand the reasons, it is necessary to go back to the passage of the federal Clean Water Act of 1972, and beyond that to the evolution of the harbor pollution problem and the non–federal plans to remedy it.

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* Honorary Member, Boston Society of Civil Engineers Section/ASCE.

1972 Clean Water Act

The Clean Water Act of 1972 required all publicly–owned treatment plants, without regard to the nature or location of the water into which they discharge, to achieve secondary treatment by 1977. EPA defined treatment in terms of the amount by which two quantities, biochemical oxygen demand (BOD) and suspended solids, are reduced in the treatment plant effluent. BOD is of concern because it tends to reduce dissolved oxygen levels in the receiving water. The severity of the depletion depends on a replenishment process, known as reaeration, in which oxygen is transferred from the air through the water surface. If the treatment plant effluent is discharged to a fresh water stream, the dissolved oxygen may drop to a level that is harmful to aquatic life because reaeration is limited by small water surface areas. In such cases high levels of BOD removal are desirable. In the ocean, water surface areas are large and oxygen is replenished readily, hence BOD removal is less critical. Suspended solids are of concern because their removal in the treatment process improves clarity and reduces formation of bottom deposits in the receiving water. In addition, hazardous and toxic substances tend to be adsorbed onto suspended solids.

A primary plant, the first stage of any treatment system, removes about 40% of the incoming BOD and 60% of the suspended solids in a series of sedimentation tanks. The secondary treatment stage is a biological process which provides additional treatment to the effluent of the primary plant with the result that the overall removal of BOD and suspended solids are increased to about 85%. The additional removal of BOD in the secondary stage is accomplished by adding oxygen to treatment tanks to speed the growth of bacteria that feed on, and thereby oxidize dissolved organic, oxygen demanding material in the wastewater. The bacteria, which continually grow and die, and other suspended solids settle out in secondary sedimentation tanks.

Sediment residue from the primary and secondary treatment stages is called sludge. The sludge contains everything that has been removed from the raw wastewater as well as new biomass generated in secondary treatment. Consequently, the amount of sludge produced by the combined primary-secondary stages is about twice as much as is produced by the primary stage alone. Current regulations prohibit the ocean disposal of sludge.

Soon after the 1972 act was passed, many municipalities argued to Congress and EPA that secondary treatment was not universally necessary for protection of the coastal marine environment. They contended that large reductions in BOD, while important for inland freshwater streams and lakes, were of little benefit to the coastal ocean where treatment plant effluents are mixed and dispersed by tidal currents and aerated by large water surface areas. They also pointed out that long outfall pipes could reach coastal areas of significant depth and tidal flushing action. Furthermore multi–port diffusers, thousands of meters in length, could be attached to the outfalls to reduce the concentration of treated effluents by more than a hundredfold through jet mixing. A number of Pacific coast communities that had been discharging primary effluent through ocean outfalls had accumulated evidence to demonstrate the scientific merit of their claims for exemption from secondary treatment requirement. Congress was persuaded and, in a special provision of the 1977 Clean Water Act, Section 301(h), directed EPA to allow municipal marine dischargers to test their case in the administrative process.

Boston Harbor Pollution Background

Boston began discharging untreated waste into the harbor more than a hundred years ago. It was not until 1968 that all dry weather sewage began receiving primary treatment at Nut and Deer Islands. However, that treatment has been essentially negated by the fact that the primary effluent and the sludge is discharged, after some digestion and chlorination, through short pipes near the entrance to the harbor. Although, sludge

discharge is supposed to occur mainly during the outgoing tide, much of Boston's present problem is due to this long–banned but continuing practice.

The first serious water quality study of Boston Harbor was completed in the late 1960s. The findings were that primary treatment was satisfactory, disposal of sludge to the harbor should be stopped and the major problem for the harbor was combined sewer overflows. These overflows, from about 90 sources on the perimeter of the harbor, result from the collection of storm water and sewage in the same pipes throughout much of the older inner city area. Raw sewage is discharged from these sources during wet weather about 60 times a year when the treatment plant capacity is exceeded. It is generally agreed that combined sewer overflows are responsible for the frequent closing of shellfishing and bathing areas within the harbor.

The only positive thing to be said about the water quality situation in Boston Harbor is that the depletion of dissolved oxygen has never been a problem, except near the shoreline. It was therefore natural that the Metropolitan District Commission (MDC), the responsible state agency, should apply for a waiver of the secondary treatment requirement so that it could focus cleanup efforts on stopping sludge discharges and combined sewer overflows.

The Waiver Process

In response to the 1977 Congressional directive, EPA published preliminary criteria and procedures in spring 1978 and final guidelines in summer 1979 by which municipalities could apply for waivers of the secondary treatment requirement for discharges into coastal waters. Boston's application was one of 70 filed prior to the 1980 deadline. Subsequently the deadline was extended to the end of 1982 and 137 additional applications were filed. An EPA bias against the waiver procedure, in favor of the more easily enforceable uniform

secondary treatment, was suspect from the beginning. Their guidelines stated that applicants

> "will bear a particularly heavy burden in demonstrating to EPA that such (less–than–secondary) treatment is sufficient to protect marine waters."[1]

In addition, EPA, contradicting accepted principles of environmental impact analysis, refused to allow applicants to compare the environmental impacts of less–than–secondary and secondary effluents through the same outfall. That incremental benefits of secondary treatment might be negligible or unjustifiably costly was of no interest to EPA.

Boston's Waiver Plan

In 1967, well before the major federal water acts, Boston's MDC had concluded that the benefits of secondary treatment were minimal and proposed a three-part plan: an ocean outfall 7 miles offshore of Deer Island for the discharge of primary treatment effluent, the cessation of ocean sludge discharge and combined sewer overflow controls. In 1976, in the process of setting priorities, MDC determined that providing secondary treatment ranked fortieth in a list of about 50 projects that would be required to improve Boston harbor.

These earlier studies became the basis for the waiver application to EPA in the fall of 1979. MDC proposed upgrading the existing primary treatment plants, which were severely deteriorated, and constructing a combined ocean outfall and multi–port diffuser with a total length of 9 miles terminating in water more than 100 feet in depth. Boston was required to demonstrate that the waiver plan would meet state water quality standards for marine waters. The major standard was that dissolved oxygen not fall below 6 parts per million (ppm), about 80% of its saturation value under summer temperature conditions.

The estimated cost of the proposed facilities including sludge disposal on land was

The estimated cost of the proposed facilities including sludge disposal on land was 480 million in 1979 dollars, about half of this was the cost of the 9–mile ocean outfall and diffuser. At that time EPA was funding 75% of capital costs. MDC settled back to await EPA's verdict having no inkling that a decision on the waiver would drag on for the next five–and-one–half years.

EPA's Response

EPA, overwhelmed by the mountain of waiver applications, hired a consulting firm to assist in the review process. In mid 1981, EPA requested additional information from MDC including a sensitivity analysis of the water quality model, additional sampling and assessments of sediment deposition and resuspension. MDC responded in the fall of 1982 with new monitoring data and the analyses requested.

In the summer of 1983, EPA issued a tentative denial of Boston's waiver. The denial focused on potential violations of the state dissolved oxygen standard and excessive solids deposition; however, EPA left the door open by stipulating that MDC could submit a revised application by July 1984. There was a significant amount of interaction between MDC and EPA on the nature of new information to be submitted. The revised application was submitted and six months later EPA's consultant issued its technical review.

One point of contention between MDC and EPA's consultant was the proper value of the background or ambient dissolved oxygen in the vicinity of the 9–mile outfall. MDC said 7.4 ppm was reasonable for late summer conditions when dissolved oxygen was observed to be at its minimum.[2] The consultant recommended a more stringent value of 6.5 ppm, recalculated dissolved oxygen impacts in four separate analyses and concluded that "the Massachusetts dissolved oxygen standard will be met."[3] MDC assumed that the major issue had been resolved and awaited its waiver. Its optimism was short–lived

because in March 1985, the EPA regional administrator issued a "tentative decision" that the revised waiver application be denied.[4]

EPA's denial was based on seven findings. Six were non–quantitative or procedural in nature such as deficiencies in the monitoring program to assess future impacts and future source control programs to reduce toxics.[5] The single quantitative finding reversed the conclusion of EPA's consultant in one of the four impact analyses carried out to check the state's 6.0 ppm dissolved oxygen standard. This analysis involved the calculation of a dissolved oxygen change due to the resuspension in a storm event of organic particles deposited on the bottom after 90 days of uninterrupted deposition. EPA calculated a dissolved oxygen concentration of 5.5 ppm (a violation of 0.5 ppm) by means of two "adjustments." The first involved an arbitrary one–third increase in the rate of organic sediment accumulation in the vicinity of the outfall diffuser. The second and more serious adjustment reduced the ambient dissolved oxygen concentration for the resuspension event from 6.5 to 6.1 ppm,[6] a value only 0.1 ppm above the standard, which even the effluent from a secondary treatment plant would have violated. EPA made no effort to defend its arbitrary adjustments in this critical instance, yet proceeded to rest its case for secondary treatment on them.

There were valid reasons for challenging EPA's tentative denial, but other events had by this time removed MDC as the responsible state agency.

New Agency, New Plans

Early in 1983 the city of Quincy, on the southern portion of Boston Harbor, sued MDC for polluting its beaches. The case was heard by State Supreme Court Judge Paul Garrity who appointed Professor Charles Haar of Harvard Law School as special master. Haar's report adopted by the Court in the fall of 1983 included (1) a strict time–table for stipulated remediation measures, which, incidentally did not include secondary treatment,

and (2) recommended the creation of a new state agency with the power to issue bonds outside the control of the State legislature.

Following a year of MDC failure to meet Haar's schedule, Judge Garrity threatened to stop sewer connections for new buildings in Boston. In the last hours of 1984, the legislature created the Massachusetts Water Resources Authority (MWRA) and gave it bonding authority. In January 1985 EPA sued MWRA in Federal District Court for polluting the harbor. Under threat of a huge retroactive fine, MWRA has been operating under a 1986 federal court-ordered planning and construction schedule designed to carry out EPA's insistence on full secondary treatment by 1999. MWRA made the judgment that any attempt to reopen the waiver issue was doomed to failure.

It is easy to look back over the past ten years and to say what should have been done. The existing primary treatment plants are beyond rehabilitation and, in fact, never performed satisfactorily. Design and construction of new state–of–the–art primary plants and land–based facilities for the disposal of the primary sludge should have begun in 1979. No one ever questioned the need or priorities for these facilities regardless of the waiver decision. Work would have begun in 1979 but for the fact that EPA would not approve an application for a federal grant while a ruling on the waiver application was pending.

In retrospect, the most serious flaw in the waiver process was EPA's refusal to consider a comparison of the environmental impacts of primary treatment effluents and secondary treatment effluents through the same outfall. As a result the incremental environmental benefits of secondary treatment for the harbor were never balanced against the negative environmental impacts of disposing of twice as much sludge on land or by incineration in the air. Through a fortuitous set of circumstances in the spring of 1988, the data necessary for such a comparison became available.

In March 1988, MWRA published a comprehensive primary and secondary treatment facilities plan, and a few weeks later EPA issued a draft Environmental Impact

Statement for Boston Harbor based on this plan. The astonishing thing about the 1988 facilities plan is that the length of the outfall and diffuser, about nine miles, is the same as in the 1979 waiver plan. So it is now planned to discharge secondary effluent at essentially the same location as was originally suggested for the primary effluent. This is the longest outfall and diffuser ever designed specifically for secondary treatment effluent.

According to the court–ordered schedule, the new primary plant and the outfall and diffuser are to be completed by 1995 and the secondary treatment stage by 1999. Because of the five-year construction lag, MWRA and EPA were required to predict water quality conditions for primary as well as primary plus secondary effluents. Separate consulting firms were employed to carry out the technical analyses for the two agencies.

The MWRA facilities report provides a detailed analysis of all state and federal water quality criteria for conventional as well as hazardous wastes. MWRA concluded that standards, including dissolved oxygen, would be met by the primary effluent and that in general

> "secondary treatment discharge impacts are not expected to be significantly different from primary impacts."[7]

Despite these findings, MWRA again made no effort to reopen the issue of the marginal benefit of secondary treatment. When this was explicitly pointed out during the public comment period on the facilities plan, the MWRA response was

> "it is not necessary to justify secondary treatment in the Facilities Plan as the MWRA is mandated by Federal law and court agreements to provide this level of treatment."[8]

EPA made separate impact analyses of the facilities plan for primary and secondary effluents through the proposed 9–mile outfall and diffuser. Non–compliance with standards were cited in three instances for primary effluent. The first was based on a predicted dissolved oxygen violation of 5.7 ppm during a 90–day sediment resuspension

event. In contrast to the 1985 waiver denial based on a similar event, EPA now assumed the ambient dissolved oxygen at 6.5 mg/l, thereby backing away from its earlier indefensible ambient of 6.1 ppm.[9] A search was made for reasons why EPA predicted a dissolved oxygen violation while MWRA's analysis of the same event did not. It was found that EPA calculated a six–fold increase compared to MWRA in the average rates of sediment deposition in the ocean area surrounding the multi–port diffuser. This was caused by EPA's determination that a small fraction of sediment in the primary effluent would be in the size range having a fall velocity of 0.1 centimeters per second.[10] On the other hand, MWRA determined that the maximum fall velocity of sediment in the effluent would be 0.01 cm/sec.[11] A calculation of the sediment removal capability of the new primary treatment plant indicated that all particles having a fall velocity of 0.1 cm/sec would be removed in the primary treatment process even when the primary plant was operated at its maximum capacity of 1.2 billion gallons per day, which is two and a half times its average flow rate. EPA's use of the incorrect size fraction in the effluent resulted in higher calculated rates of sediment deposition and consequently an overestimation of the dissolved oxygen depression due to resuspension.

The second point in which EPA indicated that primary effluent impacts were less satisfactory than secondary was in the areal extent of bottom sediment enrichment and toxicity. Again this was a result of overestimating sediment deposition rates.

The third point, aquatic life criteria, is based on a screening of toxic effects of more than 50 non–conventional pollutants. Of these, the only violations indicated were for mercury and three compounds (two pesticides and PCB) not presently detected in the inflow to Boston's treatment plants.[12]

The above issues were pointed out during the public comment period and EPA's responses were given in the final environmental impact statement (FEIS) of July 1988. On the major non–compliance issue which revolves around the disagreement on sedimentation

rates, EPA acknowledged that the faster settling sizes would be removed in the primary treatment process. However, EPA justified retaining the high settling rate in order to

> "account for potential aggregation of the effluent particles in the marine waters, which would cause the aggregate particles to fall faster."[13]

The suspended solids concentration in the primary effluent is only about 50 ppm before undergoing a hundred–fold reduction in concentration through the multi–port diffuser. Even at the undiluted value there is no scientific basis for assuming that aggregation is effective at such low particle concentrations. The only evidence for aggregation in marine waters is in the discharge of sludge from outfalls where particle concentrations are more than a thousand parts per million.

EPA summed up the level of treatment issue in its FEIS

> "Some commentators questioned the need for secondary treatment, particularly for a discharge as far off–shore and as deep as the outfall location recommended. The suggestion was made that EPA was over–conservative in its analysis, and that money required to construct and operate the new secondary treatment facilities could better be used to address other pollution problems such as discharges of raw sewage from combined sewer overflows."[14]

EPA continued by saying that because the waiver was denied and MWRA is now committed to secondary treatment

> "the need for secondary treatment of MWRA wastewater was not a question addressed by this FEIS, and a comparison of the impacts of primary effluent versus secondary effluent is not required."[14]

Regulatory zeal here runs counter to the spirit of environmentalism that led to the creation of EPA and to the concept of environmental impact analysis. EPA's "tentative" waiver denial of 1985 has become incontrovertible law. It is as if the Food and Drug Administration, having tentatively kept a questionable drug off the market, used that tentative denial as an excuse not to consider later evidence that the drug had over–riding beneficial effects.

Priorities

Vitally important components are not included in the present federal court construction schedule for the harbor cleanup. They are the combined sewer overflow (CSO) control facilities. Preliminary plans indicate that more than 20 miles of deep-rock tunnels, 25 feet in diameter will be needed to store wet weather sewer flows so they can be fed into the treatment system in subsequent dry periods. The cost, certain to be a billion dollars or more, will ultimately be added to the 6.1 billion dollar price tag for scheduled facilities. While MWRA has accepted responsibility for the CSO problem, its placement in the construction schedule will not be resolved until after final CSO plans are submitted in mid 1990. It is very apparent that MWRA's financial and management capabilities will be stretched to the limit to complete the secondary treatment facilities plan by 1999 and that CSO construction will probably extend well into the next century. It will come as a rude shock to rate payers that the 6 billion dollar plan, when completed, will not make Boston harbor fishable or swimmable.

The cost of the secondary treatment and secondary sludge disposal facilities in the present schedule is about 2.5 billion dollars. This is a very high price to pay for the marginal environmental benefits of secondary treatment especially when the negative environmental impacts of disposing of twice as much sludge have yet to be evaluated. EPA's claimed benefits of secondary treatment relate to the removal of suspended solids rather than to the purpose for which it was designed, that is the removal of soluble organic material (BOD).

An innovative treatment process capable of levels of suspended solids removal comparable to secondary treatment but without high BOD removal and sludge production has not been considered in any of MWRA's post–waiver planning. This process, known as advanced primary treatment, consists of adding very small amounts of polymer chemicals to primary treatment tanks to cause aggregation of particles and increased sedimentation.

Los Angeles County, Orange County and the City of San Diego have successfully used advanced primary treatment for more than 10 years. Advanced primary treatment has achieved 80 percent suspended solids removal with only a 30 percent increase in sludge production over conventional primary treatment, as compared to a 100 percent increase in sludge production with secondary treatment. Orange County received a waiver of the full secondary treatment requirement from EPA in 1985. Los Angeles County's waiver application is still pending; however they are expecting a favorable decision that will allow them to continue this sensible practice. The City of San Diego waiver application has had a checkered history. It was tentatively approved in 1981; however in 1986, EPA announced a reversal of its decision with an option for the City to submit a revised application. In 1987 in response to public pressure, the City decided not to resubmit.

A logical set of priorities for Boston Harbor would follow the current schedule through the completion by 1995 of the new primary treatment plant, its sludge disposal facility and the 9–mile ocean outfall and diffuser. At that time, the cleanup effort should be directed away from secondary treatment in favor of facilities for the control of combined sewer overflows. Upon completion of the new primary plant and the ocean outfall and CSO remediation measures, an intensive monitoring program in Boston Harbor will indicate whether additional treatment is necessary. If so, the sensible step would be to implement advanced primary treatment.

Fortunately during the next six years, there is time to bring scientific and political pressure to force the priority issue through the new EPA administration and Congress. When EPA was footing three–fourths of the bill and threatening massive retroactive fines, there was little incentive to argue. Now there is every reason to insist that local funds be used to achieve an optimal environmental solution rather than one that adheres to a narrow and outmoded regulatory viewpoint.

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Appendix A3

Discussion on the Hundred Fold Difference in the Settling Accumulation Rate Calculated by MDC and EPA.
S. P. Morrissey and D. R. F. Harleman, 1989.

Discussion on the Hundred Fold Difference in the Settling Accumulation Rate Calculated by MDC and EPA

Shawn P. Morrissey and Donald R. F. Harleman

Tetra Tech, Inc. reported an almost two orders of magnitude difference in the settling accumulation rates calculated by Metropolitan District Commission (MDC) in its 1984 revised 301(h) waiver application to the EPA (Tetra Tech, 1984). This difference results almost entirely from the selected values for settling velocity distribution. The selection of 0.1 cm/sec by Tetra Tech as opposed to 0.01 cm/sec by MDC resulting in a decrease in affected area size by a 100 fold. MDC stated that all particles with a fall or settling velocity of 0.1 cm/sec are removed in the primary sedimentation tanks prior to discharge. This can be substantiated by assuming discrete particle settling.

The settling of discrete, nonflocculating particles can be analyzed by Stokes' law:

$$V_c = \frac{g\,(\rho_s - \rho)\,d^2}{18\,\mu}$$

where: V_c = fall or settling velocity
ρ_s = density of particle
ρ = density of fluid
g = acceleration due to gravity
d = diameter of particle
μ = dynamic viscosity

In designing sedimentation tanks, it is customary to determine a fall velocity, V_c, so that all particles with a fall velocity greater than V_c will be removed (Fig. 1). The rate at which clarified water is produced is then:

$$Q = A\,V_c$$

where: Q = flow rate
A = surface area of sedimentation tanks

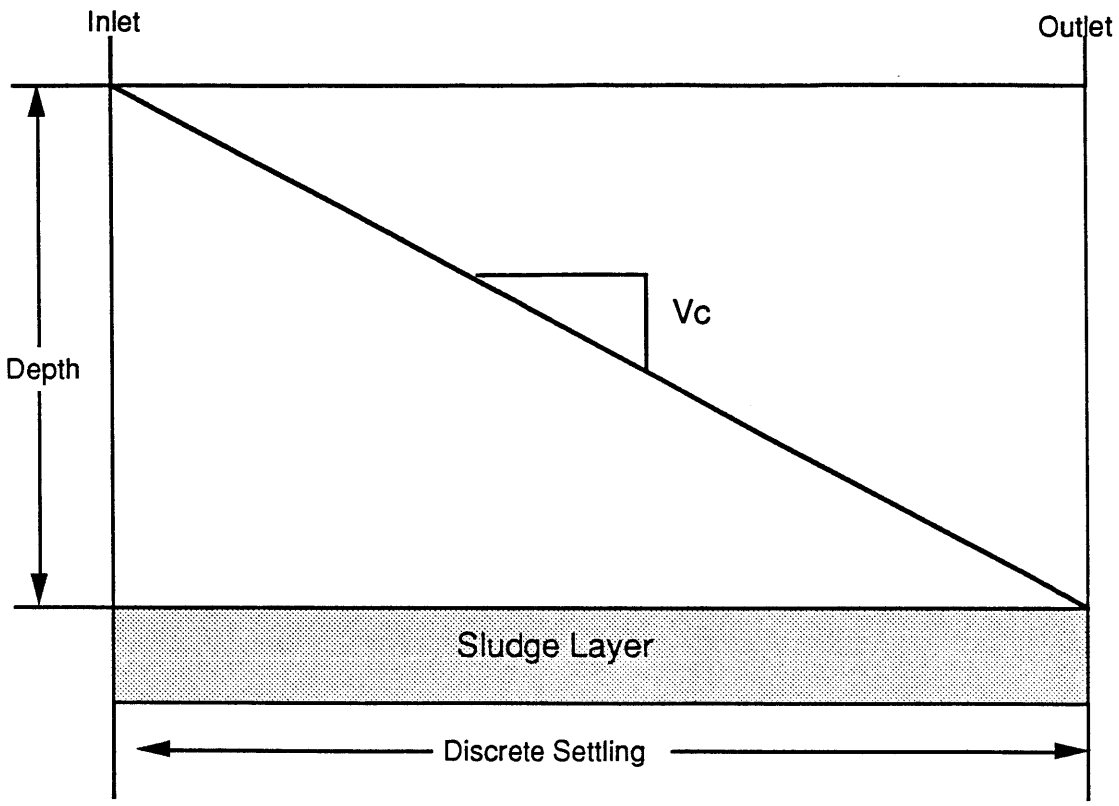

Figure 1 - Illustration of discrete particle settling.

Rearranging the above equation yield the overflow rate which becomes the design parameter.

$$V_c = \frac{Q}{A} \qquad V_C = \text{overflow rate}$$

Below is a table that compares the expected flow rates and fall velocities for Massachusetts Water Resources Authority (MWRA) proposed treatment facilities. The surface area is 59,000 m^2.

	Avg Annual Flow	Avg. High GW Flow	Maximum Daily Flow
Flow Rate (mld)	1820	2540	4810
Fall Velocity (cm/sec)	0.04	0.05	0.09

* All particles larger than the indicated fall velocity will be removed.
* GW = Groundwater

These calculations indicate that all particles which have fall velocities larger than 0.1 cm/sec will settle for all flow conditions.

The selection of a smaller fall velocity resulted in a decrease in the affected area of sediment accumulation by a hundred fold. As an example, the following formulas were used to calculate the area of sediment accumulation:

$$A = (2\,D)^2 \qquad \text{and} \qquad D = \frac{V_a\,H_t}{V_s}$$

where:
A = the area of deposition for a 90 day event
D = max horizontal distance a particle travels
V_a = ambient current (8 cm/sec)
H_t = plume height of rise (9.1 m)
V_S = settling velocity

The results are:

At $V_s = 0.01$ cm/sec, D = 7.3 km, A = 212 km^2
MDC predicted A = 166 km^2
At $V_s = 0.1$ cm/sec, D = 0.73 km, A = 2.1 km^2
Tetra Tech predicted A = 2.5 km^2

MDC used this formula and Tetra Tech used one very similar.

The different selection of settling velocity distributions directly effects the sediment accumulation rate as shown in the example below (EPA, 1988):

$$S = \frac{M P}{A K_d} (1 - \exp(-90 K_d))$$

where: S = 90 day organic accumulation rate
M = effluent organic solids loading rate (9.67×10^7 g/day)
P = percentage of M for a given setting velocity group
A = area of deposition of 90 day event (2.5 km^2)
Kd = decay rate constant (0.01 /day)

At V = 0.01 cm/sec

% of M for settling group	S (g/m^2)
1	0.35
2	0.69
3	1.04
4	1.38
5	1.73

At V = 0.1 cm/sec

% of M for settling group	S (g/m^2)
1	22.9
2	45.9
3	68.8
4	91.8
5	115

There is almost a two order of magnitude difference.

The affect of the selected values for settling velocity distribution can also be seen in MWRA's facilities planning report where they predicted a maximum sediment deposition rate which was six times smaller than EPA's prediction in their Draft Supplementary Environmental Impact Statement (DSEIS). In this instance, MWRA used the worst possible scenario for particle setting, which states that all particles will settle within an one hour tidal excursion of either side of the 2,000 m diffuser, or an area of approximately 4 km^2. This resulted in MWRA's choice of 25 g/m^2 to be used as the organic accumulation rate. Using the value of 25 g/m^2 and an area of 4 km^2, it was determined, based on contour maps generated using ELA (Fig. 2), that the maximum sediment deposition rate was 0.34 g/m^2-day.

Whereas, EPA choose a value of 90 g/m^2 to use as the organic accumulation rate based on five percent of the particles have a settling velocity of 0.1 cm/sec. Using the value of 90 g/m^2 and an area of 5 km^2, it was determined, based on contour maps generated using ELA (Fig. 3), that the maximum sediment deposition rate was 1.9 g/m^2-day.

The difference in settling velocity distribution values also effected the dissolved oxygen (DO) concentration due to resuspension events. The resuspension DO demand value was calculated using the following formula (EPA, 1988):

$$RDOD = \frac{S}{1.6\,(E_z\,t)^{\frac{1}{2}}}\,(1 - \exp(-K_r t))$$

where:
- RDOD = resuspension DO demand
- S = 90 day organic accumulation rate
- E_z = vertical diffusion coefficient (5×10^{-4} m^2/sec)
- t = elapsed time following resuspension (24 hours)
- K_r = decay rate of resuspended sediment (0.01 /day)

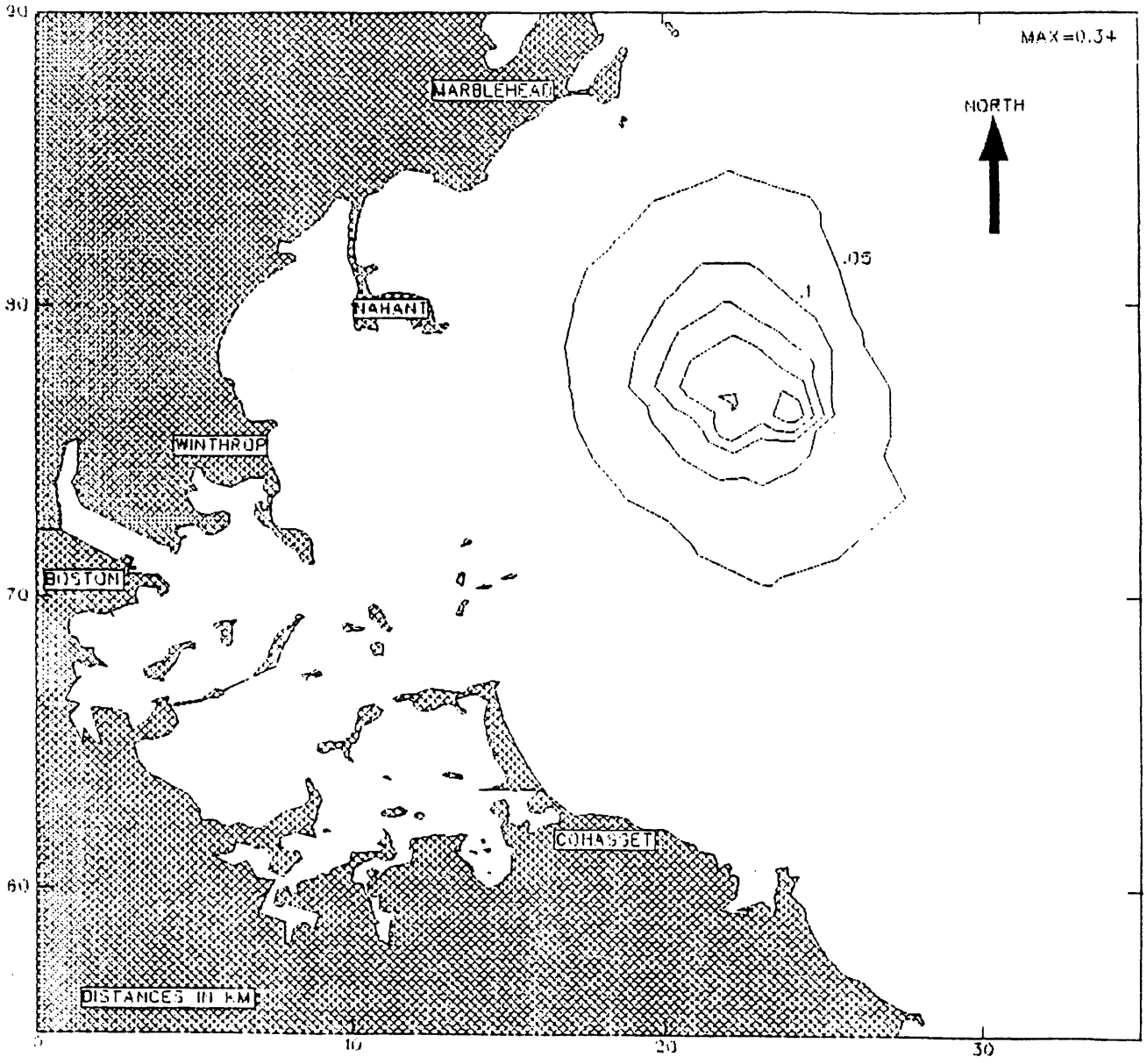

Figure 2 - Average Simulated Sediment Deposition Rates (g/m^2/day) for Primary Effluent at Site 5 (MWRA, 1988).

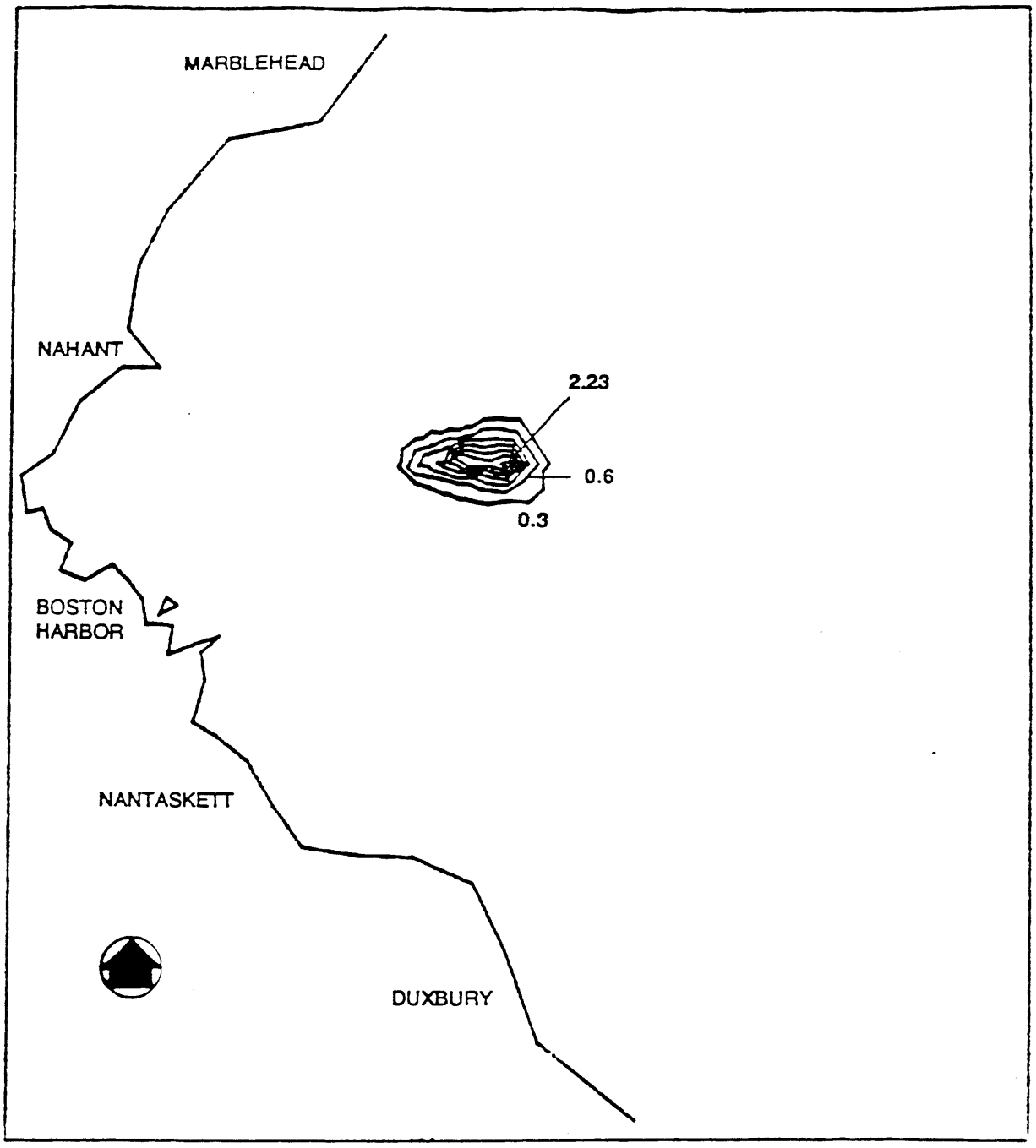

Figure 3 - Average Simulated Sediment Deposition Rates ($g/m^2/day$) for Primary Effluent at Site 5 (EPA, 1988).

As seen, RDOD is dependent on the organic accumulation rate which is ultimately dependent on the settling velocity distribution values selected. EPA predicted a RDOD of 0.82 mg/l and the MWRA predicted a RDOD of 0.14 mg/l. Even with this six fold discrepancy in DO demand due to resuspension, Tetra Tech concluded that no DO violations would occur (Tetra Tech, 1984).

At this point, EPA requested Tetra Tech to reassess the DO impacts based on a worst case hypothetical DO profile. The hypothetical profile (Fig. 4) was determined from subsets of all DO data representing the lowest DO concentrations at each depth (Tetra Tech, 1985). Based on this worst-case profile and an abrupt resuspension event after a critical 90 day accumulation period of sedimentation, Tetra Tech concluded that this would result in a DO violation (fall below 6.0 mg/l) of the Commonwealth of Massachusetts water quality standard. Therefore, the discrepancy in the settling velocity distribution and the choice of a hypothetical DO profile were the ultimate cause of the waiver application denial (Harleman, 1989).

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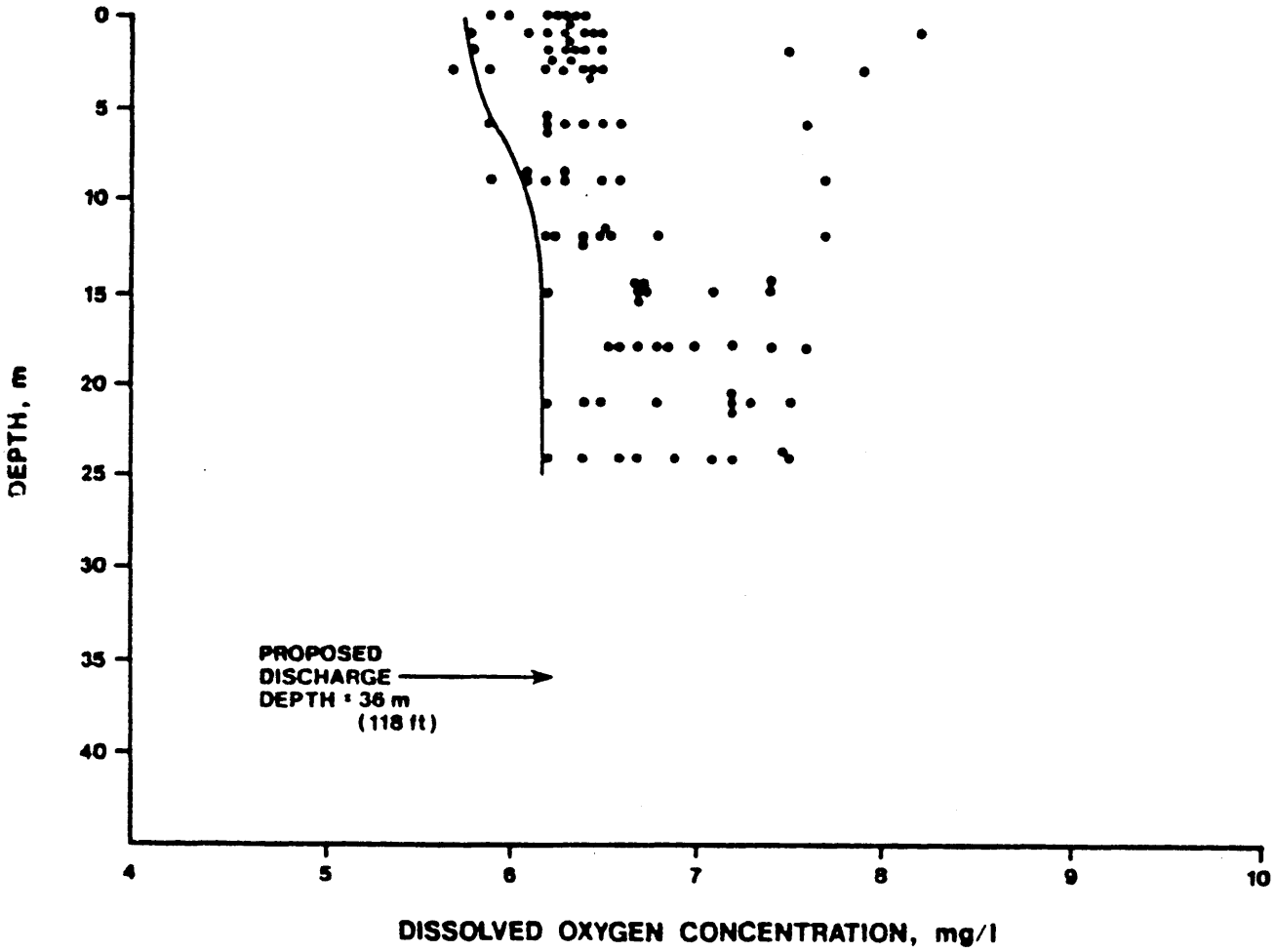

Figure 4 - Selected Dissolved Oxygen Concentration Measured Near the Proposed Discharge Site During August, 1978 (Tetra Tech, 1985).

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Appendix A4

Summary of Cailfornia's Experience with Urban Waste Management in the Coastal Zone.
S. P. Morrissey and D. R. F. Harleman, 1989.

Summary of California's Experience with Urban Waste Management in the Coastal Zone

California Ocean Plan

The California Ocean Plan, Water Quality Control Plan for Ocean Waters of California, was published by the state of California, State Water Resources Control Board and adopted on September 22, 1988.

This plan was set forth to ensure that the discharge of waste into the ocean does not interfere with the beneficial use and protection of California's ocean waters. Effluent quality standards for total suspended solids (TSS) are, for a 30-day average, 75% removal or 60 milligrams per liter (mg/l) of TSS, whichever is larger, from the influent stream before discharging effluent into the ocean. This plan has no effluent requirement for biochemical oxygen demand (BOD).

503 Sludge Regulations

The proposed ruling on the Standards for the Disposal of Sewage Sludge,was established by the United States Environmental Protection Agency (EPA) on February 6, 1989 and published in the Federal Register Vol 54 No. 23 pp 5746. This document is referred to as EPA's 503 sludge regulations.

This regulation was set forth to protect public health from the adverse effects of certain pollutants that may be present in the sewage sludge. Requirements were established for final use and disposal of sewage sludge when it is applied to land, landfilled, incinerated, or distributed and marketed. Ocean disposal of sewage sludge is prohibited as of December 31, 1991 due to the Ocean Dumping Ban Act of 1988. Pollutant standards are given in terms of limits or equations to calculate the limits. Monitoring, record keeping, and reporting requirements will also be established. These proposed regulations are much more restrictive than the current regulations on sludge disposal.

City and County of San Francisco

The City and County of San Francisco, servicing a population of 230 thousand, currently treats an average of 22 million gallons per day (mgd) of wastewater at its Richmond-Sunset plant. The peak flow is two times the average flow. All the flow receives primary treatment. The sedimentation tanks, with an average overflow rate of 925 gallons per day per square feet (gpd/ft^2), achieve the following removal rates: 32% BOD_5 and 65% TSS. This process includes the addition of a small amount of polymers before primary sedimentation.

Of the raw sludge, 65% is volatile solids. The sludge is anaerobically digested resulting in 28 dry tons per day (dtpd). The digested sludge is conditioned with ferric chloride and then dewatered using belt presses. Approximately 40% of San Francisco's sludge goes to beneficial reuse and 60% to landfills.

The effluent is discharged into the ocean through an outfall (equipped with a 1,450 ft diffuser) approximately 22,300 ft long at a depth of 80 ft, with a minimum initial dilution greater than 100. This outfall is designed to handle a maximum flow of 145 mgd. The outfall design provides for future expansion of the treatment facilities and as a means of transporting overflow out into the ocean.

On July 7, 1988, EPA approved San Francisco's 301(h) waiver to discharge effluent through an outfall into the ocean. The waiver was approved for 50% secondary and 50% primary treatment with an average monthly combined effluent concentration of 60 mg/l TSS and 193 mg/l BOD_5. This treatment is required to meet California Ocean Plan Standards for TSS. There is no requirement for BOD_5. Currently, San Francisco only provides primary treatment of its wastewater.

Although San Francisco has been granted a waiver, environmental groups have filed Requests for Evidentiary Hearings on the draft 301(h) waiver. These filings postpone the effective date of the waiver, thus the waiver process is not yet complete for San Francisco.

Information on San Francisco's experience with urban waste management has been provided by David Jones, planning and design coordinator, and Donald Munakaba, project manager for the Richmond-Sunset plant.

The City of Los Angeles (Hyperion)

The City of Los Angeles Hyperion Treatment Plant, servicing a population of three million, currently treats an average of 370 mgd of wastewater. The peak flow is 1.25 times the average flow. Sixty percent of the flow receives advanced primary treatment and 40% receives secondary treatment with a combined effluent concentration of 30 mg/l TSS and 90 mg/l BOD_5. The advanced primary treatment, with an average overflow rate of 2,000 gpd/ft^2, achieves the following removal rates: 50% BOD_5 and 80-85% TSS. These removal rates are obtained by applying 20-25 ppm ferric chloride and 0.25 ppm anionic polymers continuously to the wastewater. The addition of these chemicals has increased TSS removals from 65% to as high as 85%.

The sludge is thickened in centrifuges producing a mixture of 4.5% solids. Approximately 400 dtpd of raw sludge at 74% volatile solids is anaerobically digested. The anaerobic digestion destroys 54% of the volatile solids producing methane and carbon dioxide. The digested sludge is then conditioned with anionic polymers and dewatered using centrifuges to produce a cake of 20% solids.. At present, 20-30% of the 260 dtpd of digested sludge produced at the Hyperion plant is incinerated and used to generate energy for the plant. The remaining sludge is landfilled, composted, or chemically fixed and used as a landfill cover.

The effluent is discharged into the ocean through an outfall (equipped with a 4,000 ft diffuser) approximately 26,400 ft long at a depth of 320 ft, with a minimum initial dilution greater than 60. Until January 1988, the City of Los Angeles sludge was being disposed of through a seven mile ocean outfall.

The waiver application submitted by the City of Los Angeles was to upgrade the existing treatment to 60% secondary, 40% advanced primary, and to stop ocean sludge disposal. While waiting for approval, the Hyperion treatment plant began to deteriorate and the City of Los Angeles was fined twice for discharging sludge into the ocean at their seven mile outfall. On March 10, 1986, the City of Los Angeles was denied its request for a secondary treatment waiver and is now required to provide full secondary treatment by the year 1995.

Information on the City of Los Angeles experience with urban waste management has been provided by J. Y. Shoa, production engineer for the Hyperion plant.

Los Angeles County (JWPCP)

The Joint Water Pollution Control Plant (JWPCP),servicing a population of four million, currently treats an average of 360 mgd of wastewater with a combined effluent concentration is 90 mg/l of TSS and 90 mg/l BOD_5. Forty percent of the flow receives advanced primary and 60% receives secondary treatment. The advanced primary treatment, with an average overflow rate of 1,270 gpd/ft^2, achieves the following removal rates: 47% BOD_5 and 80% TSS. These removal rates are obtained by applying 0.15 ppm of anionic polymers continuously to the wastewater. The addition of these chemicals has increased TSS removals from 65% to as high as 83%.

Currently, there are five upstream tertiary treatment plants that provide relief to the JWPCP and treat an average of 150 mgd. All the raw sludge produced (650 dtpd) at the upstream plants and the JWPCP plant is combined and returned to a centralized facility at JWPCP. The sludge is then thickened in dissolved air flotation thickeners producing a mixture of 2.7% solids. Raw sludge at 74% volatile solids is anaerobically digested, which destroys 54% of the volatile solids producing methane and carbon dioxide. Retention time in the digesters is 16 days on average. The digested sludge is then conditioned with anionic polymers at 5.5 pounds per dry tons of sludge and dewatered using centrifuges to produce a cake of 23% solids. At present, 110 dtpd of the 390 dtpd of digested sludge is used as a soil conditioner (composted). The remaining 280 dtpd of sludge is landfilled. The composted sludge meets all of California's regulations for sludge composting. Although, with the onset of the new EPA 503 sludge regulations, the composted sludge is not expected to meet requirements for cadmium.

The combined effluent is discharged into the ocean through two outfalls. One outfall (equipped with a 4,400 ft diffuser) is 7,400 ft long and the other outfall (equipped with a Y shaped diffuser with each arm extending 2,200 ft) is 8,000 ft long. Both outfalls discharge effluent at a depth of approximately 190 ft, with a minimum initial dilution of 165.

Los Angeles County requested a waiver from full secondary treatment because they showed that TSS removal due to advanced primary treatment (80%) was essentially equal to the results that could be obtained with secondary treatment. They also believed that the added benefits of BOD_5 removal by secondary treatment was not significant enough in ocean waters to warrant the costs associated with a secondary treatment plant. Currently, JWPCP's waiver application is pending approval by EPA.

Information on Los Angeles County's experience with urban waste management has been provided by Blair Hanson, general superintendent at the JWPCP facility.

Orange County

Orange County Sanitation District consists of two treatment plants: Plant #1 in Fountain Valley and Plant #2 in Huntington Beach. These plants service approximately 1.8 million people with an expected increase to 2.8 million by the year 2020.

Plant #1 treats 60 mgd of wastewater, all of which receives advanced primary treatment before receiving secondary treatment. The advanced primary treatment, with an overflow rate of 1,300 gpd/ft^2, provides the following removal rates: 38% BOD_5, 65% TSS, and 48% oil and grease. These removal rates are obtained by applying 20-25 ppm ferric chloride and 0.25 ppm anionic polymers to the wastewater during peak diurnal BOD loading, which averages about eight hours a day. The addition of these chemicals has increased BOD_5 removal from 28% to 38%.

All of the wastewater at Plant #1 receives secondary treatment. Of the 60 mgd, 18 mgd passes through a trickling filter, with a recirculation ratio of 1.5:1. The effluent from these filters averages 30 mg/l BOD_5, 37 mg/l TSS, and 4.4 mg/l oil and grease. The remaining 42 mgd is treated using an oxygen activated sludge process. The effluent from this process averages 11 mg/l BOD_5, 6 mg/l TSS, and 1.2 mg/l oil and grease. Of the 42 mgd, 3.9 mgd receives tertiary treatment and is injected into the groundwater for protection against saltwater intrusion.

Plant #2 treats 197 mgd of wastewater, all of which receives advanced primary treatment and 40% receives secondary treatment. The advanced primary treatment, with overflow rate of 836 gpd/ft^2, provides the following removal rates: 47% BOD_5 and 71% TSS. These removal rates are obtained by applying 20-25 ppm ferric chloride and 0.25 ppm anionic polymers to the wastewater during peak diurnal BOD loading, which averages about 12 hours a day. In Plant #2, chemicals are applied for 12 hours a day as opposed to 8 hours a day in Plant #1. Applying the chemicals for a longer period of time has resulted in a higher removal rate of BOD and TSS. Forty percent of the flow receives secondary treatment using an oxygen activated sludge process. The combined effluent averages 6 mg/l BOD_5, 11 mg/l TSS, and 2.6 mg/l oil and grease.

The 110 dtpd of sludge produced at both plants is thickened in dissolved air flotation thickeners producing a mixture greater than 2.9% solids. To increase removal efficiency and produce a dense float, polymers are added to the waste activated sludge (WAS) during the thickening process at 91 pounds of wet polymers per ton of WAS solids.

Raw sludge at 75% volatile solids is anaerobically digested. The anaerobic digestion destroyed more than 56% of the volatile solids producing methane and carbon dioxide. Retention time in the digesters is 30 days for Plant #1 and 14 days for Plant #2 on average. The digested sludge is then conditioned with anionic polymers at 6 pounds per dry tons of sludge and dewatered using a belt press to produce a cake of 22% solids. At present, all sludge produced at Plant #1 is hauled offsite and composted, while only a portion of the sludge produced at Plant #2 is hauled offsite and composted. The rest is disposed of in landfills. Orange County has no problem meeting current regulations for sludge use; although, they believe they will have problems meeting the new EPA 503 regulations for sludge disposal.

The combined effluent of Orange County's two treatment plants is discharged into the ocean through an outfall (equipped with a 5,200 ft diffuser) approximately 26,400 ft long at a depth of 200 ft, with a minimum initial dilution of 148.

On February 22, 1985, EPA approved Orange County's 301(h) waiver to discharge effluent through an outfall into the ocean. The waiver was approved for 50% advanced primary and 50% secondary treatment with a combined effluent concentration of 47 mg/l suspended solids and 71 mg/l BOD_5. In Orange County's estimation, 25% more sludge is produced during advanced primary treatment than during conventional primary treatment.

There are two noteworthy comments:

- The wastewater entering the Orange County plants meets EPA's effluent standards for toxics.
- Their secondary treatment waiver is up for renewal in 1990.

Information on Orange County's experience with urban waste management has been provided by Robert Ooten, operations engineer.

The City of San Diego (Point Loma)

The City of San Diego, servicing a population of 1.7 million, currently treats an average of 186 mgd of wastewater at the Point Loma Plant. The peak flow is 1.2 times the average flow. All of the flow receives advanced primary treatment with an effluent concentration of 70 mg/l TSS and 116 mg/l BOD_5. This treatment process, with an average overflow rate of 1,700 gpd/ft^2, achieves the following removal rates: 75-80% TSS, 51% BOD_5, and 42% oil and grease. These removal rates are obtained by applying ferric chloride at 35 mg/l and anionic polymers at 0.26 mg/l to the wastewater. The addition of these chemicals has increased TSS removals from 60% to almost 80%.

The Point Loma plant produces 170 dtpd of raw sludge. The raw sludge is thickened using gravity thickeners to 6% solids. The sludge, at 74% volatile solids, is then anaerobically digested. The anaerobic digestion destroys 57% of the volatile solids producing methane and carbon dioxide resulting in 98 dtpd. The digested sludge is conditioned with polymers, dewatered in open air drying beds, and disposed of in landfills or sold as a soil conditioner. San Diego's advanced primary treatment plant produces approximately 28% more sludge than a conventional primary treatment plant.

The effluent is discharged into the ocean through one outfall (equipped with 1,350 ft diffusers) approximately 11,500 ft long at a depth of 200 ft, with a minimum initial dilution of 114.

After a comprehensive monitoring program, San Diego requested a waiver from full secondary treatment because the City concluded that the advanced primary effluent currently being discharged into the ocean was creating virtually no adverse impacts on the ocean and secondary treatment was not necessary at Point Loma.

On September 23, 1981, EPA issued a tentative approval of San Diego's waiver application. On September 30, 1986, EPA announced its decision to reverse its tentative approval and the City had until March 30, 1987 to submit a revised application. On March 12, 1987, San Diego decided to withdraw their waiver application due to negative responses from the public and the regulatory agencies in charge.

The City of San Diego is now one year into a three-year facility planning process. The cost of the secondary treatment project of 360 mgd capacity is estimated at $1.6 billion.

Information on San Diego's experience with urban waste management has been provided by Walter Konopka, chief chemist for the Point Loma plant.

Summary of California's Experience with Urban Waste Management in the Coastal Zone

Location	Waiver Status	Population (million)	Flow (mgd)	BOD_5 (% removal)	TSS (% removal)	Effluent Outfall length / diffuser / depth (m)	Initial Dilution	Treatment Process
San Francisco	Expecting Approval	0.23	22	32	65	22,300 / 1,450 / 80	100	100% Primary
City of Los Angeles (Hyperion)	Denial (3/10/86)	3.0	400	50	83	26,400 / 4,000 / 320	60	40% Secondary 60% Adv. Primary
Los Angeles County (JWPCP)	Pending	4.0	360	47	80	800 / 2@2,200 / 190 7,400 / 4,400 / 190	165 165	60% Secondary 40% Adv. Primary
Orange County	Approved (2/22/85)	1.8	260	40	70	26,400 / 5,000 / 200	148	50% Secondary 50% Adv. Primary
City of San Diego (Point Loma)	Withdrawn (3/12/87)	1.7	190	51	80	11,500 / 1,350 / 200	114	100% Adv. Primary

Note: - Los Angeles waiver request for 60% secondary and 40% primary treatment was denied.

- Orange County's 301(h) waiver is up for renewal in 1990. They are concerned that it may be denied.

Appendix B1

Seawater Intrusion and Purging in Tunnel Outfalls, A Case of Multiple Flow States. Schweizer Ingeneur und Architekt, Nr. 6. pp 156-160. N. H. Brooks, 1988.

Seawater Intrusion and Purging in Tunnelled Outfalls

A Case of Multiple Flow States

Wastewater outfalls tunnelled under the ocean floor terminate in a series of vertical shafts and risers to bring the flow up to the sea bottom for discharge through special jet manifolds. Since this system is hydrostatically unstable, there are special problems of seawater expulsion. This paper gives a simple analysis of the fresh-water flow rate required for purging such a system, and compares it with the much smaller discharge needed to control intrusion after purging has been accomplished. If purging is not achieved at peak discharges then seawater inflow through some risers will occur at less than peak discharges, while other risers still have outflow; in such a case, multiple flow configurations are possible. Results are presented in a parametric way to assist the designer in adjusting component sizes to achieve the desired purging and intrusion-prevention characteristics.

Introduction

Generally in hydraulic structures it is desired to have the flow uniquely determined by the imposed flows or heads. For example, sharp-crested weirs are ventilated so that the lower

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edge of the nappe may freely detach from the weir, and the head-discharge relation is unique or single-valued. An example of non-uniqueness is the liquid discharge from a tank into air through a short tube (length: diameter = 1:1) with a sharp entrance from the tank; in this case, the jet may either be a free contracted jet springing from the sharp-edged tube entrance, or it may fill the whole tube cross section.

In this paper, we shall examine some aspects of the purging hydraulics of a multiple-port ocean outfall diffuser, and see how hydraulic designers must be aware of non-unique flow situations resulting from the 2.7%-density difference between the ambient seawater and the wastewater effluent (essentially the same density as fresh water).

Problem Definition

We shall restrict our attention to tunnelled outfalls where the purging problem is more acute than for buried or surface-laid pipelines. Figs. 1 and 2 show the essential features of a tunnelled outfall system with risers and multiple ports in each riser head. Three such systems are currently under construction for Sydney, Australia, and designers are studying a similar option for discharge from the Boston metropolitan area into Massachussetts Bay. We will not present the analysis for any particular system but instead give an idealized example to illustrate the hydraulics.

After the initial decline tunnel (or vertical shaft), the tunnel under the sea floor slopes up slightly (slope S) so that any leakage during construction will drain back toward shore for safety. The risers (N = number) are connected at the invert of the tunnel to facilitate the expulsion of seawater and any accumulated sediments or settled solids. At the head of each riser a special manifold discharges through n nozzles directed horizontally in a radial pattern at angular separations of $360/n$ degrees. The total number of ports is then nN. This arrangement can give the equivalent of a line source for obtaining high dilution while keeping the number of risers reasonable [1]. The height of the riser from the top of the offtake from the tunnel to the centerline of the discharge ports (Fig. 2) is designated H.

Although the port diameters may vary slightly in order to equalize the flow, assume a representative diameter d_p such that the total discharge area is $nNd_p^2/4$. The area ratio R_1 (to-

Fig. 1. Schematic cross section of tunnelled multiport ocean outfall

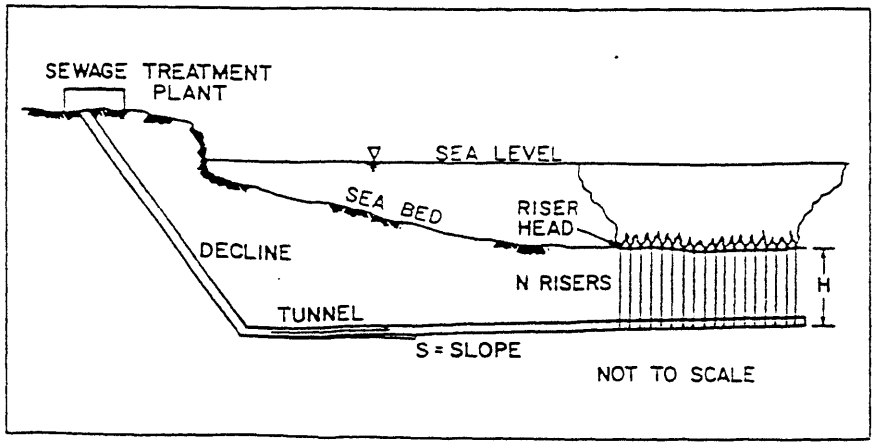
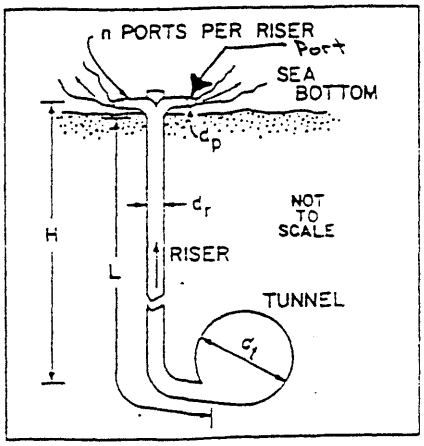


Fig. 2. Schematic cross section of a riser with bottom takeoff from the tunnel, and multiport riser head containing n ports arranged in a radial pattern



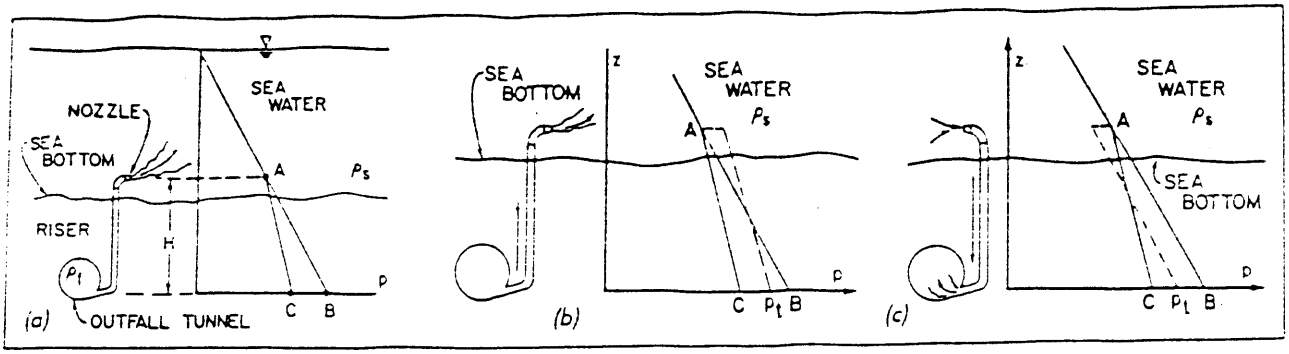


Fig. 3. Hydraulics of a single riser:
(a) Hydrostatic pressure distribution: AB for riser full of seawater; AC for riser full of effluent
(b) Pressure distribution (-----) for outflow (with pipe friction and nozzle loss) compared to hydrostatic pressures ($P_C < P_t < P_B$)
(c) Pressure distribution (-----) for inflow at some tunnel pressure

tal ports: tunnel) is nNd_p^2/d_t^2; the area ratio R_2 (ports: risers) is nd_p^2/d_r^2; and the area ratio R_3 (risers: tunnel) is Nd_r^2/d_t^2. Therefore, $R_1 = R_2 \cdot R_3$. For good manifold design all these ratios should be less than unity, for example $R_1 = 0.4$, $R_2 = 0.8$, $R_3 = 0.5$. The designer selects appropriate values based on manifold design, available head, range of discharges, costs, and purging problems, discussed here. (Other aspects of the manifold design are discussed in [2, 3, 4].)

For proper operation, the effluent flow should be capable of expelling all the seawater out of the tunnel and risers whenever the discharge is started from the condition of seawater flooding the entire system (such as the inital startup, or restart after a period of shutdown long enough for total seawater intrusion). The minimum total discharge required to purge the system (as described) is designated Q_p. After purging, the subsequent intrusion of seawater should be prevented at all operating flows. The minimum flow which will prevent intrusion is called Q_I.

Intrusion Criterion

It has been well established [4] that when a discharge port is flowing full, intrusion can be prevented by requiring the port densimetric Froude number to exceed unity, or for safety against perturbations and allowing for various geometries

$$(1) \quad F_p = \frac{V_j}{\sqrt{g' d_p}} > 2$$

where V_j = port velocity = $q/(\pi d_p^2/4)$, q = port discharge, $g' = (\Delta\varrho/\varrho)g$, $\Delta\varrho = \varrho_a - \varrho$, ϱ_a = ambient seawater density, ϱ = discharge density. By continuity we find that the required flow Q_I is:

$$(2) \quad Q_I = 2nN(a_p)\sqrt{g' d_p}\,, \text{ or}$$

$$(3) \quad Q_I = 2R_1(\pi d_t^2/4)\sqrt{g' d_p}$$

where a_p = port area = $\pi d_p^2/4$. This criterion has been used successfully to control *intrusion*, but without much attention until the 1980's to the fact that the *purging* flow Q_P may be much larger than Q_I, especially for tunnelled outfalls. For example, some tunnelled outfalls in Great Britain [5, 6, 7] had an operating range (Q_{min} to Q_{max}) such that $Q_I < Q_{min}$, but $Q_{max} < Q_p$; since the outfalls never were fully purged, the intrusion could not be prevented by the criterion of Eq. (1).

Purging Criterion-Risers

We consider first the hydraulics of a single riser (Fig. 3). If the riser is filled with seawater (with no flow), then the hydrostatic pressure distribution would be AB. The pressure represented by B will be considered the reference pressure. If the riser is filled with fresh water (no flow), then the pressure is represented by AC. The difference in pressure $\Delta p = p_B - p_C = \Delta\varrho g H$. If the operating pressure in the tunnel at the riser is between p_C and p_B, then seawater can flow down some risers into the tunnel at the same time that fresh water is discharging from other risers [6, 8].

For bottom takeoff risers (large number N, with $d_r \ll d_t$) the criterion for starting outflow is that the tunnel pressure exceeds p_B, the salt water hydrostatic value. The excess pressure will push the riser fluid slowly up and out, and depletes any seawater wedge in the tunnel. When fresh water begins to be drawn into the riser, the mean density of the water column in the riser starts dropping, thereby *decreasing* the hydrostatic pressure and increasing the dynamic pressure leading to a progressive increase in flow rate until the normal fresh-water flow in a riser is established.

Using Bernoulli's equation for the normal riser outflow, energy in the tunnel at the riser entrance equals the sum of the entrance loss, friction loss in the riser pipe, bend loss, losses in the top manifold and the energy of the discharge (assuming bellmouth ports with no jet contraction, C_D = discharge coefficient):

$$(4) \quad E = p_t + \varrho\frac{V_t^2}{2} = \Delta E_e + K_b\varrho\frac{V_r^2}{2} + f\frac{L}{d_r}\varrho\frac{V_r^2}{2} + (1/C_D^2 - 1)\varrho\frac{V_r^2}{2} + \varrho\frac{V_t^2}{2} + p_A + \varrho g H$$

Since the entrance loss is small it may reasonably be approximated as

$$(5) \quad \Delta E_e = \varrho\frac{V_t^2}{2} + K_e\varrho\frac{V_r^2}{2}$$

Letting $p_s = p_A + \varrho g H$ = hydrostatic pressure at tunnel level for fluid ϱ (effluent, seawater or mixed), the dynamic pressure is

(6) $p_d = p_i - p_s = \varrho \frac{V_j^2}{2}$

$\cdot [1/C_D^2 + R_1^2(K_e + K_b + fL/d_r)]$, or

(7) $p_d = \alpha \varrho \frac{V_j^2}{2}$,

where α = value of [ ] in Eq. (6).
Typical values of α, which is a system parameter, are derived as follows:

$C_D = 0.93 - 0.97$
$R_1 = 0.2 - 0.5$
$K_e = 0.1 - 0.5$
$K_b = 0.2 - 0.5$
$f = 0.015 - 0.03$
$L/d_r = 40 - 100$

$\alpha_{max} = 1/0.93^2 + 0.5^2(0.5 + 0.5 + 3)$
$= 2.16 \sim 2.2$

$\alpha_{min} = 1/0.97^2 + 0.2^2(0.1 + 0.2 + 0.6)$
$= 1.10$

Returning to Eq. (7), we see that it is a reasonable approximation to give the density in the $\varrho V_j^2/2$ term, a single value ϱ_o (according to the Boussinesq assumption). Finally, we get an overall discharge equation for a single riser as:

(8) $V_j = \sqrt{\frac{2(p_i - p_s)}{\alpha \varrho_o}}$

To find the purging criterion for the diffuser, we must consider a scenario. If the startup is slow, the effluent flow will establish itself in successive risers one-by-one starting with the offshore end, because of the slope of the tunnel. As Q increases an additional riser starts up whenever $p_i > p_B$, the hydrostatic seawater value. When $(N-1)$ risers have been started, the final riser will be purged when $p_i \geq p_B$. Just before purging, there will be a slight reverse seawater inflow in the last riser which blocks the entry of fresh water to the riser until the threshold is reached. The system purging flow Q_p may be found from Eq. (8); noting that p_s = hydrostatic pressure = p_C for fresh-water discharge:

(9) $p_i - p_s = p_B - p_C = \Delta \varrho g H$

(10) $Q_P = (N-1)\, n\, a_p\, V_j = (N-1)\, n\, a_p$

$\cdot \sqrt{\frac{2}{\alpha} \frac{\Delta \varrho}{\varrho_o} gH}$

$= \frac{N-1}{N}(nNa_p)\sqrt{\frac{2}{\alpha} g' H}$, where $g' = (\Delta \varrho / \varrho_o) g$

Just after purging is completed in N risers, the dynamic tunnel pressure will drop back slightly to $\Delta \varrho g h\,(N-1)^2/N^2$ because the riser velocity is reduced by the factor $(N-1)/N$.

The purging flow criterion Eq. (10) may also be written like a Froude number as follows:

(11) $\frac{Q_P}{nNa_p\sqrt{g' H}} = \frac{V_j}{\sqrt{g' H}} = \frac{N-1}{N}\sqrt{\frac{2}{\alpha}}$

Note that the factor $\sqrt{2/\alpha} \sim 1.0$–1.35 and $(N-1)/N \sim 0.9$–0.99, so overall

(12) $\frac{V_j}{\sqrt{g' H}} \sim 0.9 \div 1.3$

However, the value should be worked out for each system design, and the above values are intended only to show the magnitude.

This useful result shows the relationship of the required final jet velocity (the key variable) to the vertical height of the riser. For example, if $h = 50$ m, $N = 30$, $\alpha = 1.3$, $g' = 0.027$ g, then

$V_j = (29/30)\sqrt{2/1.3}\ \sqrt{g' H}$
$= 1.20\,(3.64)$
$= 4.36$ m/s

If purging is desired at $Q_p = 5.0$ m³/s, then nNa_p = total port area = $5.0/4.36 = 1.15$ m²; for n = 6 ports per riser,

$a_p = \frac{1.15\ \text{m}^2}{6(30)} = .0064\ \text{m}^2$

$d_p = 9.0$ cm

We may now examine the ratio of the critical flow for intrusion Q_I to the critical flow for purging by dividing Eq. (10) by Eq. (2):

(13) $\frac{Q_P}{Q_I} = \frac{N-1}{N}\sqrt{\frac{2}{\alpha}}\sqrt{\frac{H}{d_p}} \cdot \frac{1}{2}$

For the above example

(14) $\frac{Q_P}{Q_I} = \frac{1.20}{2}\sqrt{\frac{50}{0.09}} = 14.1$

This ratio is surprisingly large, considering that for a sewerage system Q_{max}/Q_{min} rarely exceeds 10; it shows why designing to make $Q_{max} > Q_P$ is likely to be more stringent than $Q_{min} > Q_I$ for a tunnelled outfall.

The above analysis is not sufficiently detailed for final design but it can be helpful for scaling or preliminary decisions.

Munro [9] first suggested a condition in the form of Eq. (12), namely:

(15) $\frac{V_j^2}{2g} = \frac{\Delta \varrho}{\varrho} H$, or

(16) $\frac{V_j}{\sqrt{g' H}} = \sqrt{2}$

This was based on the approximation that the total dynamic head for the riser was just the velocity head of the discharge jets. Since the other losses and the factor $(N-1)/N$ reduce the required purging head it is worth taking them into account.

The previous discussion presumed a slow startup, leading to a conservative estimate of the purging flow (called the Munro condition). It may be possible to purge an outfall at a lower flow by concurrent purging of a group of risers as the last step. The procedure would be to start slowly to expel most of the seawater and establish flow in ½ to ¾ of the risers. Then a rapid rise to a higher discharge within just a few minutes could force *concurrent* startup of all the risers in the remaining group. This may be expected because the time of flow establishment in a single riser is probably of the order of a few minutes. This would probably only work if the wedge of residual seawater in the tunnel is small and confined to the diffuser section. This procedure has not yet been demonstrated. The required purging discharge would be lowered approximately in proportion to $(N-m)/(N-1)$, where m is the number started concurrently (see Eq. [10]).

Purging Criterion-Tunnel

There is a second purging criterion, related to the expulsion of water from the long sloping tunnel. When fresh water is introduced into the upward sloping tunnel, it will establish a buoyancy front, and since the channel is long, an inverted open channel flow may be established. The seawater wedge can only be driven out if the discharge is increased to the point where the tunnel must flow full with a hydraulic slope $S_t > S' = (\Delta\varrho/\varrho_o)\, S$, where S is the tunnel slope. Using the Darcy-Weisbach friction factor f, the critical flow to achieve tunnel purging is then:

$$(17)\quad Q'_P = (\pi d_T^2/4)\sqrt{\frac{8 g d_t S'}{4f}} = (\pi d_T^2/4)\sqrt{\frac{2 g' d_t S}{f}}$$

The ratio of this flow to the flow required to purge the risers is found by dividing by Eq. (10), and using the area ratio R_1 (ports to tunnel):

$$(18)\quad \frac{Q'_P}{Q_P} = \frac{N}{N-1}\,\frac{1}{R_1}\sqrt{\frac{\alpha d_t S}{Hf}}$$

This flow ratio will normally be less than unity, indicating that the seawater is driven from the tunnel (given enough time) before the purging is complete in the diffuser section. However, the following factors will *increase* the ratio of tunnel Q'_P to the riser Q_P: increase in tunnel diameter, slope, and/or riser losses (α); decrease in port areas, riser height, and/or tunnel friction factor; and reduction in Q_P by special slow/fast startup ($N-1$ replaced by $N-m$). For example, given $R_1 = 0.15$, $d_t = 4.0$ m, $H = 30$ m, $\alpha = 1.2$, $f = 0.020$, $S = 0.005$, $N = 30$, then $Q'_P/Q_P = 1.38$. In this case, the diffuser section could purge but a residual seawater wedge would be left behind in the tunnel to be gradually removed by entrainment; in the meantime, the dead zone would be an undesirable sediment trap.

Reverse Flow in Risers

For the complete picture, we now examine the hydraulics of inflow from the sea back through the discharge ports and risers. This situation may occur (1) if: the outfall diffuser has not been completely purged; or (2) if the discharge drops below the critical value Q_I for intrusion through the ports (given by Eq. [2]), after the diffuser has previously been completely purged.

One significant difference between inflow and outflow hydraulics is that the ports will have a high energy loss for backward flow, which we describe as $K_p\, V_p^2/2g$, where V_p is the nominal port velocity not considering the severe contraction of flow at the entrance; the value of K_p may be expected to be about 3, including all the losses in the top manifold and elbow. Also at the exit from the riser into the tunnel, the full velocity head of the riser flow is assumed to be lost.

The driving force is the negative dynamic pressure in the tunnel—the amount the tunnel pressure is below the hydrostatic pressure for seawater, $p_E - p_t$. Including the intake loss, the pipe friction, bend loss, and exit loss, we obtain:

$$(19)\quad p_E - p_t = \varrho_o \frac{V_p^2}{2}\left[K_p + R_1^2\left(fL/d_r + K_b + 1\right)\right]$$

or defining β to be the sum of the coefficients in [ ]:

$$(20)\quad p_E - p_t = \beta\, \varrho_o \frac{V_p^2}{2}$$

For typical values of $K_p = 3$, $R_1 = 0.5$, $fL/d_r = 2$, $K_b = 0.3$, we obtain $\beta = 3.8$, showing the predominance of the inlet loss. For the same values, the corresponding coefficient for outflow would be about $\alpha = 1.8$ by Eqs. (6, 7). Thus the head loss for inflow is more than twice as large, or for a given driving pressure the reverse flow would be only 0.7 times as much as the normal outflow.

We can now define the possible system states when the tunnel pressure is between the hydrostatic values for fresh water and seawater ($p_C < p_t < p_E$ in Figs. 3b and 3c). If k risers have reverse flow q_i and $N-k$ have outflow q_o, and Q is the wastewater discharge, then by continuity

$$(21)\quad Q + kq_i = (N-k)q_o$$

The sum of the dynamic pressures driving the two types of flows is obtained by adding Eqs. (7) and (20) and equal to the constant difference between hydrostatic pressures for sea and fresh water (and using the Boussinesq assumption):

$$(22)\quad \alpha\, \varrho_o \frac{V_j^2}{2} + \beta\, \varrho_o \frac{V_p^2}{2} = p_E - p_C = \Delta\varrho g H$$

The velocities are respectively equal to the individual riser discharges (outflow and inflow) divided by the area of the ports na_p for each riser. It is convenient to normalize these equations by $q_\infty = na_p V_{jo}$, where V_{jo} is the outflow port velocity corresponding to $p_t = p_E$, or no seawater inflow ($V_p = 0$ and $V_{jo} = \sqrt{2g'H/\alpha}$). The corresponding dimensionless riser flows will be designated q_o^* and q_i^* for "out" and "in" respectively. The resulting dimensionless forms of Eqs. (21) and (22) are:

$$(23)\quad Q/q_\infty + kq_i^* = (N-k)q_o^*$$

$$(24)\quad q_o^{*2} + (\beta/\alpha)\, q_i^{*2} = 1$$

There are three unknowns, q_o^*, q_i^*, and k, but only two equations. Provided that $Q/q_\infty < N$, there are separate solutions for all non-negative values of k satisfying the contraint $Q/q_\infty < N-k$ or

$$(25)\quad 0 \leq k < N - Q/q_\infty$$

The relationship between the tunnel head, the outflow and the inflow is shown in Fig. 4. The ordinate in Fig. 4 is the dimensionless pressure difference $p^* = (p_t - P_C)/\Delta\varrho g H$, which is 0 at fresh-water hydrostatic and 1 at seawater hydrostatic pressure.

The reader is cautioned that this analysis is limited by the assumption that the outflow is at fresh-water density p_f, whereas it is likely that seawater inflows to the tunnel will be partially or possibly fully mixed with the fresh water and discharged back out. This phenomenon has been described by Wilkinson [7]. In Fig. 4, the middle curve shows the head-discharge relationship for the outflow density $\frac{1}{2}(\rho_f + \rho_s)$, a mixture of half fresh water and half seawater.

From Fig. 4 it is interesting to note: (a) that even a small decrease in q_o^* from 1 to 0.90 will cause an increase in inflow of q_i^* from 0 to 0.31; (b) that under the same head condition ($p^* = 0.81$), the outflow which is a mixture of 1:1 effluent and recirculated seawater would have an outflow $q^* = 0.56$. Thus, the possibility of tunnel mixing during recirculation increases still further the range of possible flow states.

The question remains: how many risers will have reverse flow? This depends on the past history of the system, and *cannot* be answered by a specification of *only* the present head or the net discharge as one might expect! If the outfall has been fully purged (the desired condition of operation), then no ris-

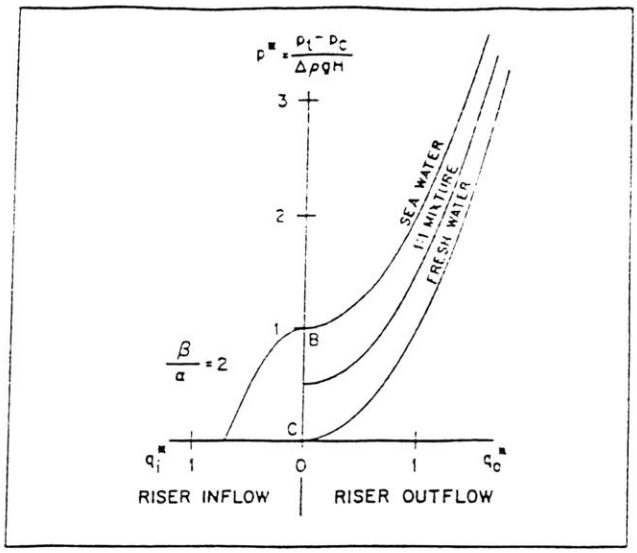


Fig. 4. Outflow and inflow as a function of tunnel pressure (for the case $\beta/\alpha = 2$). Note the possibility of either outflow of fresh water or inflow of seawater for tunnel pressures in the range $0 < p_t^* < 1$. (Based on Eq. (7) for outflow and Eq. (20) for inflow, with normalization)

ers will have reverse flow ($k = 0$) as long as the intrusion is prevented at each individual nozzle (Eq. [3]). If not previously purged, there will be inflow in those risers in which outflow was never established, and probably nearby risers due to tunnel mixing (not included above).

Closing Discussion

A simple hydraulics problem involving risers and discharge ports is not so simple when seawater and fresh water are involved in conditionally stable flows. Multiple states of flow are possible, with the previous flow history determining what actually happens. The analysis here has been simplified to show the role of the main outfall features.

The problem of starting a multiple riser outfall system in the ocean may be very roughly compared to starting up many fireplaces in a large cold house. When each fire is started, the fireplace reduces the room pressure due to the buoyancy in the chimney, which then starts cold air flowing *down* the chimneys not yet started. As more fireplaces are started, the downdrafts in the last chimneys become quite strong! But how can you stop this inflow long enough to get hot air into the last chimney to make it draw? (Open the front door!) Although the analogy is inexact (there would be no net flow into the house, just buoyancy flux generated inside), the comparison may help to make the confusing outfall behavior more intuitive.

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