Chapter 5

Existing Water and Wastewater Infrastructure

CHAPTER 5

EXISTING WATER AND WASTEWATER INFRASTRUCTURE

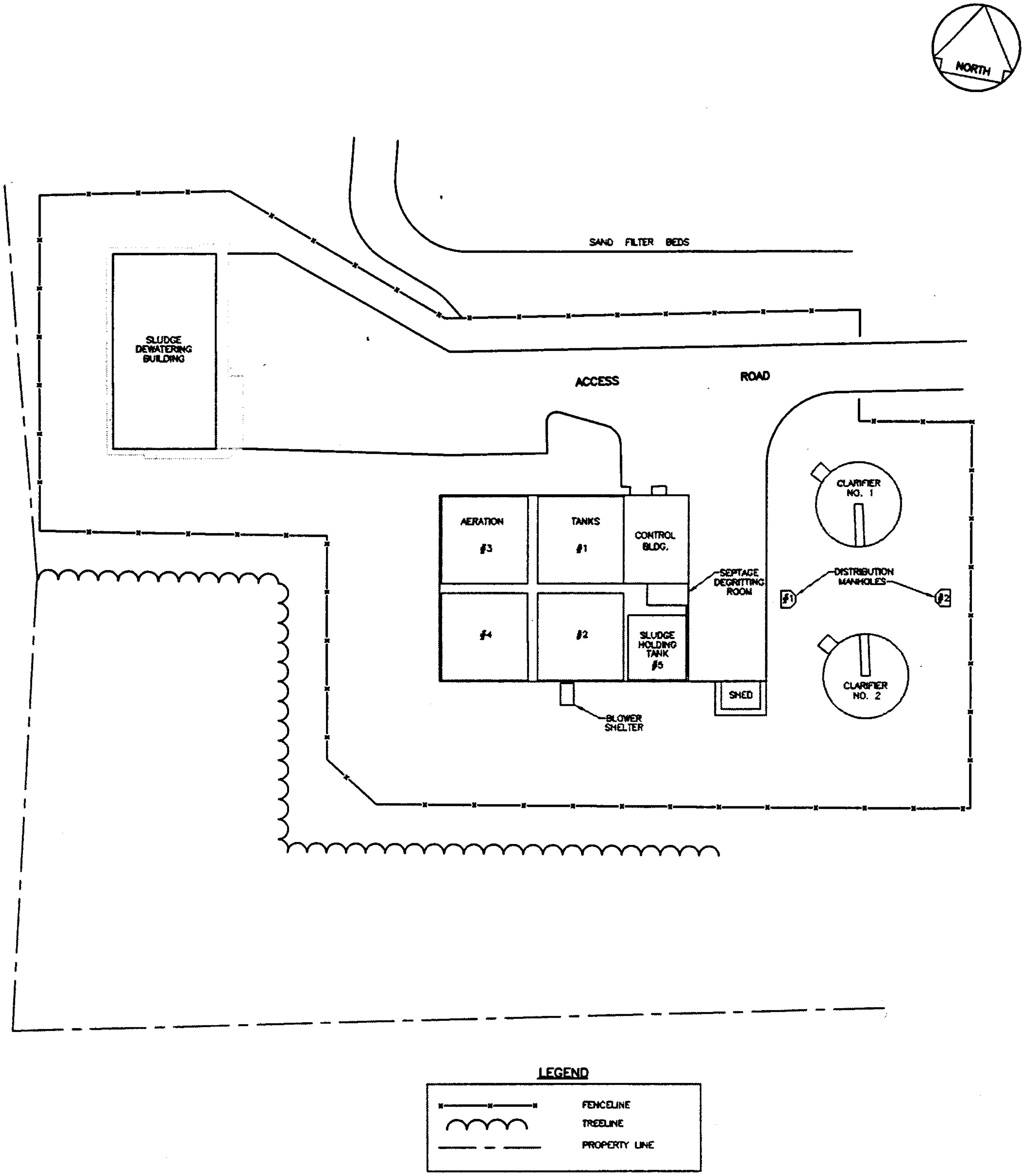
5.1 EXISTING CENTRALIZED WATER POLLUTION CONTROL FACILITY

A. History of Chatham Water Pollution Control Facility (WPCF). The existing water pollution control facility was constructed in 1972. Effluent discharge to groundwater via infiltration beds has been continuously practiced at this site since that time. Several additions and modifications have been made to the plant over the last 27 years. The majority of these facilities were constructed under three construction projects, which are described below. Figure 5-1 illustrates the present site layout.

1. Water Pollution Control Facility Construction. The present control building, headworks, pump gallery, aeration tanks, clarifiers, and infiltration beds were completed and placed in service in 1972.

2. Dewatering Building Addition. This addition of a sludge dewatering building was completed and put online in 1990. The building contains two belt filter presses; polymer feed system, odor control, and equipment garage.

3. Nitrogen Control and Septage Degritting Addition. This project modified the existing aeration tanks to the Modified Ludzack-Ettinger (MLE) nitrogen removal process, which was completed and on-line in 1996. Modifications included baffle walls in the aeration tanks, recycle pumps, and mixers. The project also provided septage degritting facilities and a structure to house degritting equipment.



Stearns & Wheler, LLC
ENVIRONMENTAL ENGINEERS & SCIENTISTS
HYANNIS, MA.

TOWN OF CHATHAM, MASSACHUSETTS

COMPREHENSIVE WASTEWATER MANAGEMENT PLANNING STUDY

FIGURE 5-1
EXISTING SITE PLAN

B. Summary of Existing Centralized Wastewater Flows and Loadings.

1. Analysis of Historical Treatment Plant Records. Water pollution control facility staff regularly sample and record the flows of wastewater, septage, grease, and sludge. Much of the sampled and recorded data is reported to the State in monthly reports, while other data is utilized by the WPCF staff as part of their monitoring operations. As part of this study, three years of data (from 1995 through 1997) were analyzed to determine the flows and loadings to the Chatham WPCF. Monthly averages were computed and are summarized in Appendix H. This data is the basis for evaluations of influent flows and loadings and plant performance.

2. Wastewater. Wastewater flows to the plant are pumped from the collection system to the headworks of the Chatham WPCF and discharged to the receiving channel. This flow passes through two bar screens and a 6-inch Parshall flume, where flows are measured. The rate of flow is then recorded and totaled. The average day, maximum day, minimum day, and total flows are recorded and reported to DEP in the monthly report. The influent flow represents the total flow for the day.

Table 5-1 summarizes the average annual, minimum month, maximum month, and peak day flows between 1995 and 1997. Table 5-2 summarizes these same flows for 1997, which will be used for the existing conditions.

The facility influent and effluent is sampled daily and analyzed for five day biochemical oxygen demand (BOD_5), total suspended solids (TSS), total Kjeldahl nitrogen (TKN), ammonia nitrogen (NH_4-N), nitrate nitrogen (NO_3-N), and total nitrogen (Total N).

BOD_5 is used to gauge the strength of wastewater, as it is an approximate measure of the quantity of oxygen that will be required to biologically stabilize the organic matter present in the wastewater. Influent municipal wastewater with BOD_5 values in the vicinity of 100 mg/l is considered weak; 200 mg/l is considered medium strength; and

TABLE 5-1

CHATHAM WPCF
FLOWS (1995 to 1997)
Comprehensive Wastewater Management Planning Study
Town of Chatham, Massachusetts

Flow Average (1)	Flow (gallons per day)	Time of Occurrence
Average annual	115,400	1997
Minimum month	77,300	Feb-95
Maximum month	175,200	Aug-97
Peak day	227,600	21-Oct-96
Peak hour	-	-

Notes:

1. Flows include sewage, septage, and grease

TABLE 5-2

CHATHAM WPCF
TOTAL 1997 FLOWS
Comprehensive Wastewater Management Planning Study
Town of Chatham, Massachusetts

Flow Average	Sewage Flow (gallons per day)	Septage Flow (2)	Grease Flow (2)	Total	Time of Occurrence
Average annual	112,500	2,300	600	115,400	1997
Minimum month	79,500	1,200	400	81,100	Mar-97
Maximum month	170,600	3,900	700	175,200	Aug-97
Peak day (1)	187,200	3,900	700	191,800	5-Aug-97
Peak hour	-	-	-		

Notes:

1. Peak day consists of the peak day sewage flow and the average septage and grease flows for that particular month.
2. Septage and grease flows for maximum and minimum months are not necessarily the maximum or minimum values for the year. They are the values that correspond to the months that the maximum and minimum sewage flows occurred. Maximum and minimum septage and grease flows are shown on Table 5-6.

BOD_5 values above 300 mg/l are considered high strength. Weak wastewater is often found in communities with significant infiltration and inflow where the pollutant concentration is diluted by groundwater and stormwater. Water pollution control facilities treating large volumes of septage or high-strength industrial wastewater can have influent BOD_5 values even higher than the normal range for high-strength municipal wastewater.

Influent BOD sampling at the Chatham WPCF indicate an average BOD concentration between 180 to 200 mg/l. Tables 5-3 and 5-4 summarize the influent and effluent flows and loadings prior to and following the implementation of the MLE process. Over this period, BOD concentrations ranged from a low of 135 mg/l to a high of 290 mg/l. These BOD concentrations classify this wastewater as medium strength.

Influent TSS concentrations during the same period averaged 150 to 170 mg/l, with a maximum value of 200 mg/l and a minimum of 95 mg/l. These TSS concentrations are typical of wastewater with medium strength.

Nitrogen is also sampled and analyzed to obtain values of TKN, NH4-N, NO3-N, and Total N. Influent TKN values averaged 33 mg/l, with values ranging from 16 to 45 mg/l. NH4-N values in the influent ranged from 15 to 45 mg/l and averaged 26 mg/l. NO3-N values averaged 0.15 mg/l with a peak at 1.47. These nitrogen concentrations are typical of wastewater with medium strength.

3. Septage. The Chatham WPCF receives and treats septage from haulers located in and around Chatham. The WPCF tracks the number of septage loads discharged to the system, the gallons of septage, and the average pH. The average annual flow, maximum month flow and the minimum month flow are 2,300 gpd, 3,900 gpd and 800 gpd respectively. A peak day flow of 12,750 gpd was experienced during late July of 1997. These flows are summarized on Table 5-5.

TABLE 5-3

SUMMARY OF CHATHAM WPCF PERFORMANCE
PRIOR TO MLE INSTALLATION
JANUARY 1995 THROUGH SEPTEMBER 1996
Comprehensive Wastewater Management Planning Study
Town of Chatham, Massachusetts

Parameter	Plant Influent		Plant Effluent		Average Removal
	Maximum Month	Average	Maximum Month	Average	
Flow, mgd	0.168	0.110	-	-	-
BOD, mg/l	290	180	21	10	
BOD, lb/day	350	170	14	8	95%
TSS mg/l	220	150	23	8	
TSS lb/day	260	140	15	7	95%
TKN, mg/l	42	34	5	3	
TKN, lb/day	51	30	6	2	-
NH4-N, mg/l	35	27	2	0.4	
NH4-N, lb/day	43	24	3	0.4	-
NO3-N, mg/l	0.3	0.2	19	11	
NO3-N, lb/day	0.4	0.2	21	10	-
Total N, mg/l	42	35	20	14	
Total N, lb/day	51	31	21	12	56%

TABLE 5-4

SUMMARY OF CHATHAM WPCF PERFORMANCE
FOLLOWING MLE INSTALLATION
JANUARY 1997 THROUGH DECEMBER 1997
Comprehensive Wastewater Management Planning Study
Town of Chatham, Massachusetts

Parameter	Plant Influent		Plant Effluent		Average Removal
	Maximum Month	Average	Maximum Month	Average	
Flow, mgd	0.175	0.116	-	-	-
BOD, mg/l	250	200	12	6	
BOD, lb/day	360	190	8	5	97%
TSS mg/l	220	170	12	6	
TSS lb/day	310	160	9	6	96%
TKN, mg/l	45	32	6	2	
TKN, lb/day	63	31	4	2	-
NH4-N, mg/l	35	24	4	1	
NH4-N, lb/day	49	24	3	1	-
NO3-N, mg/l	1	0.2	7	3	
NO3-N, lb/day	1	0.1	9	3	-
Total N, mg/l	45	32	9	5	
Total N, lb/day	63	32	12	5	83%

The Town Health Department also tracks the volume of septage and grease pumped from area septic tanks and grease traps. Table 5-6 summarizes this data from 1994 to 1997. It is noted that not all of the septage and trap grease is disposed of at the Chatham WPCF. Some is disposed at other regional facilities and the Town has minimal control where it is taken. Comparison of average annual flow data for 1997 on Tables 5-5 and 5-6 indicates that 3,200 gpd of septage was pumped from individual septic systems in 1997, and only 2,300 gpd was delivered to the Chatham WPCF.

Concentrations of septage are difficult to estimate, due to the variable nature of the septage received. Additional sampling and analysis was performed as part of this study by WPCF staff to provide further wastewater data, and is described in further detail in Section 5.1 (C). Three septage samples were collected and analyzed for BOD_5, TSS, TKN, NH_4-N, and NO_3-N, and an average of the three samples is reported on Table 5-7.

4. Trap Grease. The Chatham WPCF also receives trap grease for treatment and disposal. The average annual flow of grease in 1997 was 600 gpd, with a maximum month of 900 gpd, a minimum month of 300 gpd and a peak day of 8,000 gpd (on September 5, 1997). These flows are summarized on Table 5-5.

Additional analytical testing was also performed on the trap grease. Results of three analyses for BOD_5, TSS, TKN, NH_4-N, and NO_3-N were averaged for each parameter and are listed in Table 5-7.

5. Infiltration and Inflow. In 1988 an Infiltration and Inflow (I/I) study was performed to quantify the amount of infiltration and inflow entering the collection system. This study was followed by a 1992 Sewer System Evaluation Study to locate specific sources of infiltration and inflow and provide a cost effectiveness analysis based on the findings (M&E, 1992). Infiltration and inflow was detected, but below DEP acceptable limits. As a result only minor repairs to the system were recommended. A portion of the inflow was attributed to private sump pumps, and it was recommended that these be removed from the system.

TABLE 5-5

CHATHAM WPCF
1997 SEPTAGE AND GREASE FLOWS
Comprehensive Wastewater Management Planning Study
Town of Chatham, Massachusetts

Flow Average	Septage Flow (gallons per day)	Time of Occurrence	Grease Flow (gallons per day)	Time of Occurrence	Total
Average annual	2,300	1997	600	1997	2,900
Minimum month [1]	800	Feb-97	300	Dec-97	1,100
Maximum month [1]	3,900	Aug-97	900	Sep-97	4,800
Peak day	12,750	30-Jul-97	8,000	5-Sept-97 [2]	20,750

Notes:

1. Maximum and Minimum values for the month represent the average over the month, even though septage and grease is only discharged periodically during any given month.
2. Also occurred on September 16, 1997, and February 19, 1997

TABLE 5-6

TOWN OF CHATHAM
SEPTAGE AND GREASE PUMP OUTS (1994 TO 1997)
Comprehensive Wastewater Management Planning Study
Town of Chatham, Massachusetts

Year	Total Gallons	Gallons Grease(1)	Gallons Septage(2)	Total Number of Pump Outs	Grease Pump Outs	Septage Pump Outs
1994	1,224,485	208,525	1,015,960	867	93	775
1995	1,329,479	291,043	1,038,436	1,009	109	900
1996	1,344,197	241,592	1,102,605	849	79	771
1997	1,449,747 [3]	296,780 [4]	1,152,966 [5]	863	79	785

Note:

1. Identified as "Grease Trap" in data set plus half of those identified as "Septic, Grease"
2. Refers to all remaining system types.
3. Average annual flow of 4,000 gpd
4. Average annual flow of 800 gpd
5. Average annual flow of 3,200 gpd

TABLE 5-7

SUMMARY OF ANALYTICAL DATA
ADDITIONAL WASTEWATER, SEPTAGE, GREASE, AND SLUDGE SAMPLING
Comprehensive Wastewater Management Planning Study
Town of Chatham, Massachusetts

Parameter	Sludge Cake	RAS/WAS	Septage	Trap Grease	Decant Liquid	BFP Filtrate
Liquid Analysis (1)						
BOD_5, mg/l			2,200	3,800	5	43
TSS, mg/l			5,400	2,400	<10	210
TKN, mg/l			200	95	2	8
NH_4-N, mg/l			79	33	1	4
NO_3-N, mg/l			4	<0.2	0.3	35 (4)
Sludge Analysis (2)						
TSS, mg/l		5,800				
VSS,mg/l		4,750				
Volatile Fraction, %		82				
B, mg/kg	<550					
Cd, mg/kg	4.9					
Cr, mg/kg	28					
Cu, mg/kg (3)	1,200					
Hg, mg/kg	8.3					
K, mg/kg	5,600					
Ni, mg/kg	26					
NH_4-N, mg/kg	940					
Total N, mg/kg	57,000					
Pb, mg/kg	98					
Total P, mg/kg	21,000					
Solids, %TS (3)	17.5					
Volatile Fraction, %	76					
Zn, mg/kg	1,300					
PCB, ug/kg	<420					
As, mg/kg	6					
Se, mg/kg	6.2					

Notes:
1. Average of 3 analyses.
2. One analysis unless otherwise noted.
3. Average of 2 samples
4. Average of 3 samples that ranged from 0.2 to 50 mg/l. See discussion in the text.

These studies identified a peak infiltration rate of 61,000 gpd and a peak inflow rate of 62,000 gpd, which corresponds to a total I/I flow rate of 3,380 gallons per day per inch mile 9gpd/in-mi). Following the results of the Sewer System Evaluation Study, the Town of Chatham performed the recommended repairs on the collection system.

As a follow-up to the findings of peak I/I flows from previous studies, water use of the sewered properties was compared to the volume of wastewater received at the Chatham WPCF and observed to be very similar. The water use of sewered properties in 1997 was 40,670,000 gallons or 111,400 gpd on an average annual basis. This is very similar to the 1997 average wastewater flow at the Chatham WPCF of 112,500 gpd. It is noted that not all of this water usage will become wastewater due to the following water uses: outside showers, lawn watering, and swimming pool use. An average annual I/I value was calculated by estimating wastewater generation at 90 percent of water usage and subtracting this wastewater generation rate from the wastewater flow observed at the Chatham WPCF. Average annual I/I is calculated at 12,000 gpd.

6. Sludge (Biosolids). Since the addition of the Sludge Dewatering Building in 1990, the WPCF produced an average of 75,800 gallons per month of sludge or 2,500 gpd prior to the operation of the MLE process. Following the MLE installation in September of 1996, the sludge feed to the belt filter presses (BFP) averaged 52,200 gallons per month or 1,700 gpd. The sludge feed has an average percent solids of 2.6 percent, which is equivalent to 5,400 dry lbs. per month of sludge. See Appendix H for a summary of sludge production.

The BFPs dewater this sludge and increase the average solids content to 17 percent. This sludge is disposed of at the Yarmouth Septage Plant, in Yarmouth, Massachusetts. The WPCF generates an average of 24 wet tons of sludge per month or almost 300 wet tons per year. Disposal records were available from June 1996 to January 1998. August had the highest average of 42 wet tons per month, and January the lowest of 12 tons per month.

7. **Screenings and Grit.** Screenings are generated from 3 bar racks at the WPCF. Two bar racks are located in the influent channel and one is located at the septage receiving channel. Screenings are hand cleaned from the bar racks and combined with the grit for disposal. Grit is collected from the septage in the degritting room. Grit is removed from the self-dumping decanter on average four times per year. Combined grit and screenings are disposed at the Bourne Landfill. Average screenings and grit production is approximately 0.9 ton per month.

C. Additional Wastewater, Septage, Trap Grease, and Biosolids Sampling. Review of the available data revealed some data gaps, and, therefore, additional sampling and analysis was performed by WPCF staff. The sampled processes and the goals of the additional sampling are listed below:

- Septage: Obtain additional septage data.

- Trap Grease: Obtain additional trap grease data.

- Decant liquid: Determine BOD, TSS and nitrogen concentrations of the decant liquid that are returned to the MLE treatment process.

- Belt Filter Press (BFP) Filtrate: Characterize the filtrate from this process to evaluate performance of the BFP, and characterize the filtrate flow, which is reintroduced into the treatment process.
- Sludge Cake: Characterize the sludge for beneficial reuse or other disposal alternatives.

The results of these additional tests are summarized on Table 5-7. The following findings are noted based on these results.

- Large variations in BOD, TSS, TKN, and NH_4-N values were identified for the three samples each of septage and trap grease. This is expected and is a function of the length of time between septic tank or grease trap pumpings, and the type of wastewater generator. Average values are presented on Table 5-7.

- The decant liquid is a very clean and clear liquid with minimal BOD, TSS, and nitrogen compounds. Most NO_3-N is denitrified by facultative bacteria during the time that the aeration is turned off in the sludge holding tanks, and settling is occurring.

- The BFP filtrate is also quite clean with minimal BOD and TSS. One of the three samples had NO_3-N concentrations less than 0.2 mg/l. Two of the three analyses had high NO_3-N concentrations (> 50 mg/l). Inquiries to the analytical lab found no problems with the analysis. Discussions with the plant operator indicate that the sample with the low nitrate concentration probably resulted from BFP usage that used settled sludge as the process feed; therefore, the filtrate had a low nitrate concentration which was comparable to the nitrate concentration of the decant liquid. The two samples that had high nitrate concentration probably resulted from BFP usage that used aerated (and mixed sludge) as the process feed; therefore, the filtrate would be nitrified and have a high concentration of nitrates. The plant operator has recently changed dewatering operations to use aerated sludge instead of settled sludge as the process feed claiming that the aerated sludge dewaters to a higher solids concentration. The plant operator says that the filtrate is returned to the sludge holding tanks where it has no impact on the MLE wastewater treatment process. After it is added to the sludge holding tanks, it is mixed into the sludge and allowed to settle. This denitrifies the nitrates and precipitates any residual polymer in the filtrate. The clear denitrified supernatant is then decanted to the MLE wastewater treatment process. This processing of the filtrate appears to be additional work for the operations staff, but is worthwhile because it eliminates high nitrate loading on the MLE wastewater treatment process.

- The Return Activated Sludge/Waste Activated Sludge (RAS/WAS) has a volatile fraction of 82 percent. This indicates that 18 percent of the RAS/WAS is inert material and not active biological cell mass available for treating the wastewater.

- The sludge cake has a volatile fraction of 76 percent indicating a higher percentage of inert material probably contributed from the septage.

- The copper content of the sludge is the only parameter that exceeds the Massachusetts limit for sludge to be beneficially reused. It is believed that the copper is originating from household piping and fixtures. In the beginning of 1996, the Town completed their corrosion control program to raise the pH of the naturally acidic groundwater supply and reduce the amount of metals that are leached from piping. Since the corrosion control program began, the copper content of the sludge has decreased. It is believed that a storehouse of copper is still entering the WPCF from the septage, which was collected before the corrosion control program was completed. In an effort to identify the source and fate of the copper, the following copper analyses were performed in May 1998:

    - WPCF Influent copper content: 0.07 mg/l
    - WPCF Effluent copper content: < 0.025 mg/l
    - Septage copper content: 0.92 mg/l

These analyses indicate that the septage contributes more copper than the influent sewage.

D. Development of Total Flows and Loadings to the WPCF. The wastewater flow quantities at the Chatham WPCF have been further evaluated by researching the land uses defined by the tax assessor's office for the sewered properties in Town. These land uses, and associated wastewater flows have been grouped into the following categories: residential, commercial, industrial, and institutional. Wastewater loadings have been calculated for these flows using typical values for each category. The wastewater flows and loadings for these categories, and flows for the septage, grease, and infiltration/inflow are summarized on Table 5-8.

E. Summary of Overall WPCF Performance. Overall plant performance is indicated by the quality of the treated effluent pumped to the filter beds. Monthly averages (from June 1995 to November 1997) of plant effluent are plotted on Figure 5-2 for BOD_5, Figure 5-3 for TSS, and Figure 5-4 for nitrogen.

Figure 5- 2 indicates that the plant performs well at removing BOD on a monthly basis. The current groundwater discharge limit for the Chatham WPCF is 30 mg/l BOD on a daily basis and the effluent BOD concentration never exceeded the 30 mg/l limit during this period.

Figure 5- 3 indicates that the plant performs well at removing TSS on a monthly basis. The current groundwater discharge limit for the Chatham WPCF is 30 mg/l TSS on a daily basis and the effluent TSS concentration never exceeded the 30 mg/l limit during this period.

Figure 5-4 shows the monthly average total nitrogen and ammonia-nitrogen concentrations in the plant effluent. Figure 5-4 indicates that, on a monthly basis, the Chatham WPCF currently removes total nitrogen to less than 10 mg/l. Prior to the start of the MLE process, effluent total nitrogen levels routinely exceeded the 10 mg/l limit.

Following the startup of the MLE process, the average removal efficiency of BOD_5 is 97 percent, TSS is 96 percent, and total nitrogen is 81 percent.

F. Wastewater Treatment Facilities.

1. General. The wastewater treatment facilities are comprised of the following major components: pretreatment facilities, aeration tanks, and secondary clarifier facilities. The existing site plan, Figure 5-1, shows the arrangement of these

TABLE 5-8

CHATHAM WPCF
WASTEWATER FLOWS AND LOADINGS
Comprehensive Wastewater Management Planning Study
Town of Chatham, Massachusetts

Source	Average Annual	Minimum Month	Maximum Month
Residential			
Flow, gpd	35,000	25,000	53,000
BOD_5, lb/day	70	40	130
TSS, lb/day	60	40	110
TKN lb/day	10	0	30
Commercial & Industrial (1)			
Flow, gpd	54,000	38,000	82,000
BOD_5, lb/day	110	60	210
TSS, lb/day	90	60	190
TKN, lb/day	20	10	50
Institutional			
Flow, gpd	6,000	4,000	9,000
BOD_5, lb/day	10	10	20
TSS, lb/day	10	10	20
TKN, lb/day	0	0	10
Total Sewered Areas			
Flow, gpd	95,000	67,000	144,000
BOD_5, lb/day	200	110	360
TSS, lb/day	160	100	320
TKN, lb/day	30	10	80
WWTF Infiltration/Inflow (gpd) (2)	17,000	17,000	17,000
Septage (3)			
Flow, gpd	2,300	800	3,900
Grease (3)			
Flow, gpd	600	300	900
Total WWTF Influent			
Flow, gpd	112,000	80,000	171,000
BOD_5, lb/day	200	120	400
TSS, lb/day	160	90	310
TKN, lb/day	30	10	90

Notes:

1. Industrial flows are very small and make up a small percentage of the total commercial and industrial category.
2. Infiltration and Inflow can occur any time of the year. Peak values of I/I are discussed in the report text.
3. Septage and grease are treated in the sludge holding tanks and the decant liquid and belt filter press filtrate from these flows have minimal contribution to the wastewater treatment process. Typical values for these 2 flows are presented in Table 5-7.

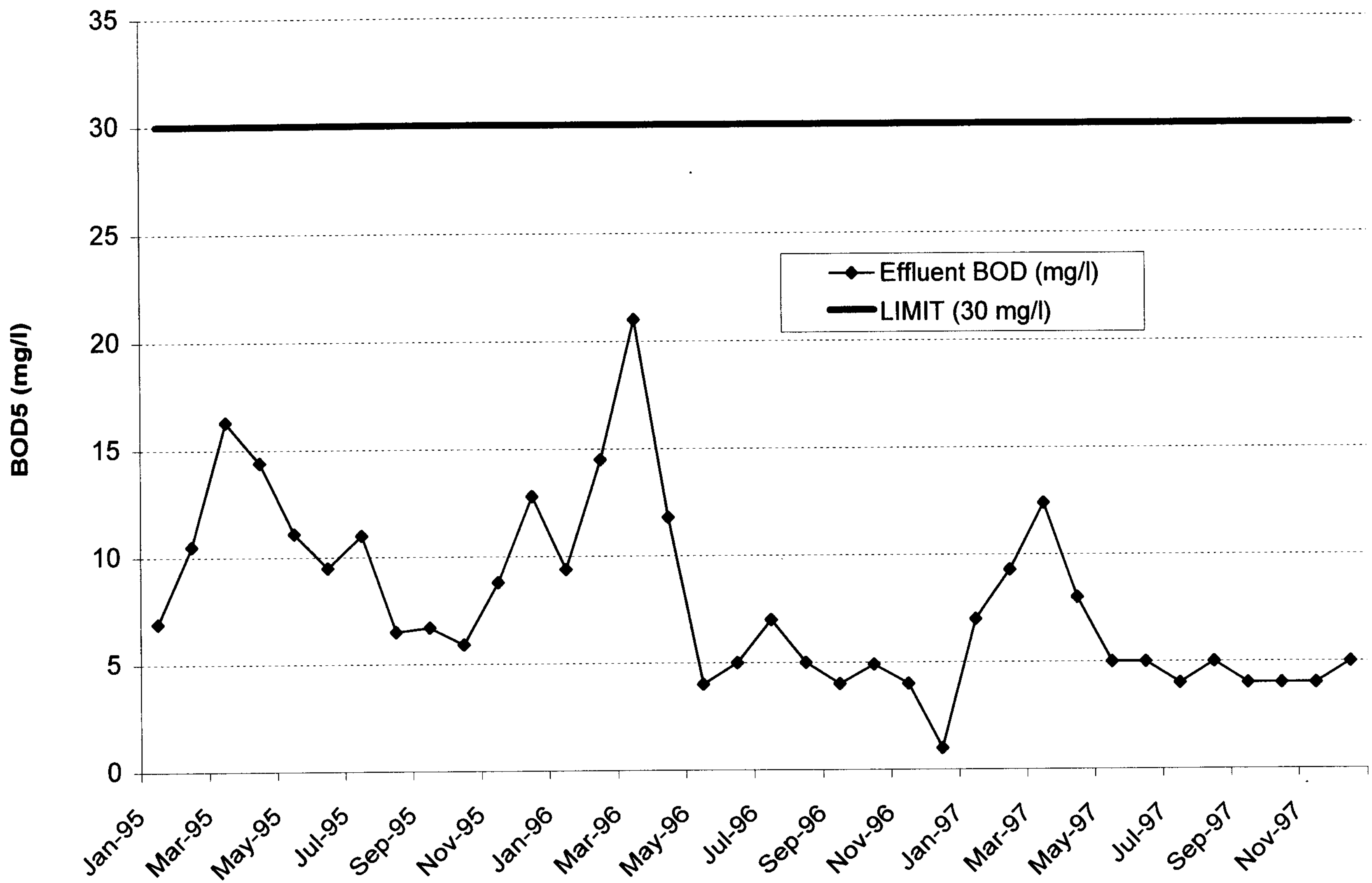


FIGURE 5-2
EFFLUENT BOD CONCENTRATIONS

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chat5figFig (BOD)

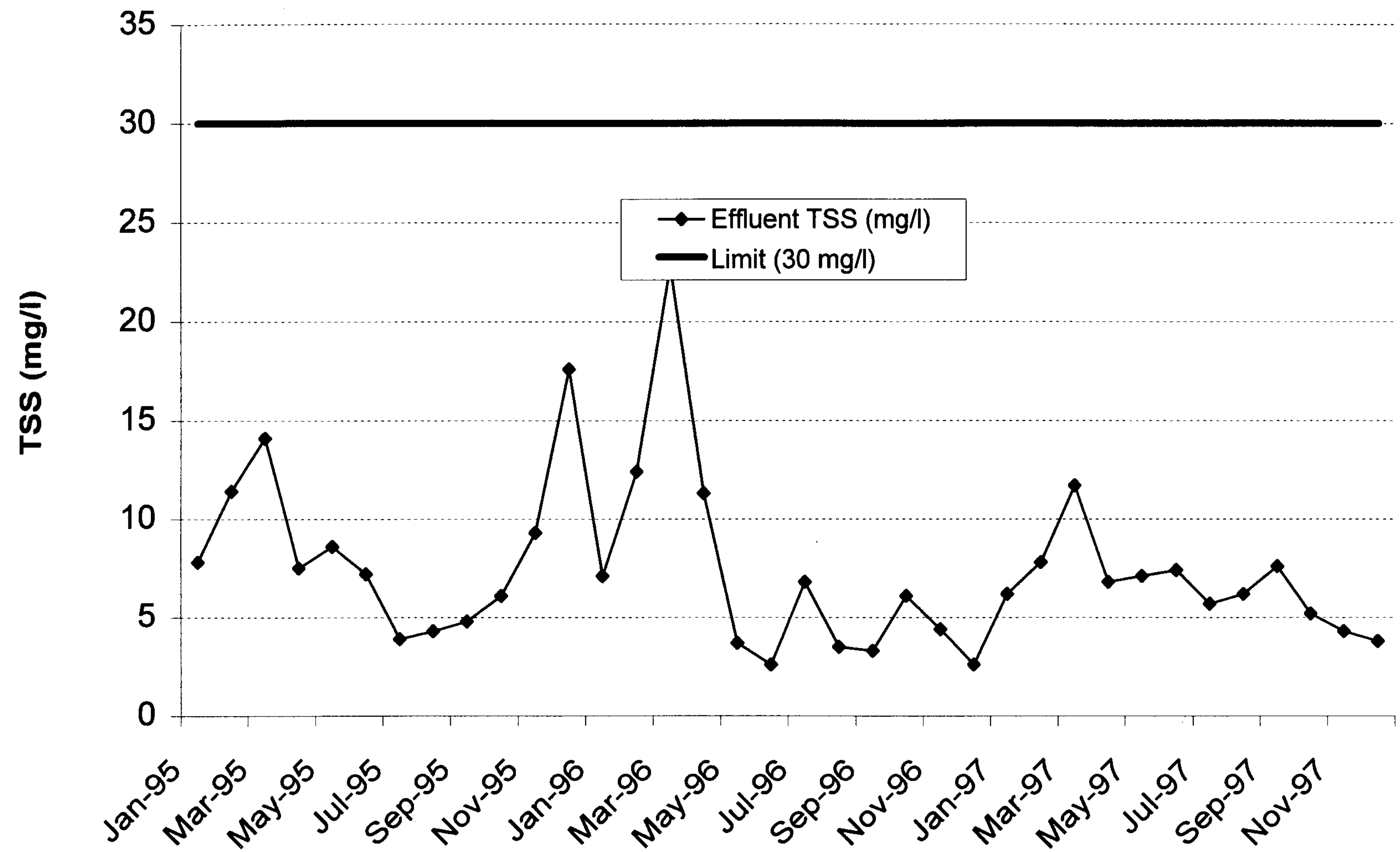

FIGURE 5-3 EFFLUENT TSS CONCENTRATIONS
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chat5figFig (TSS)

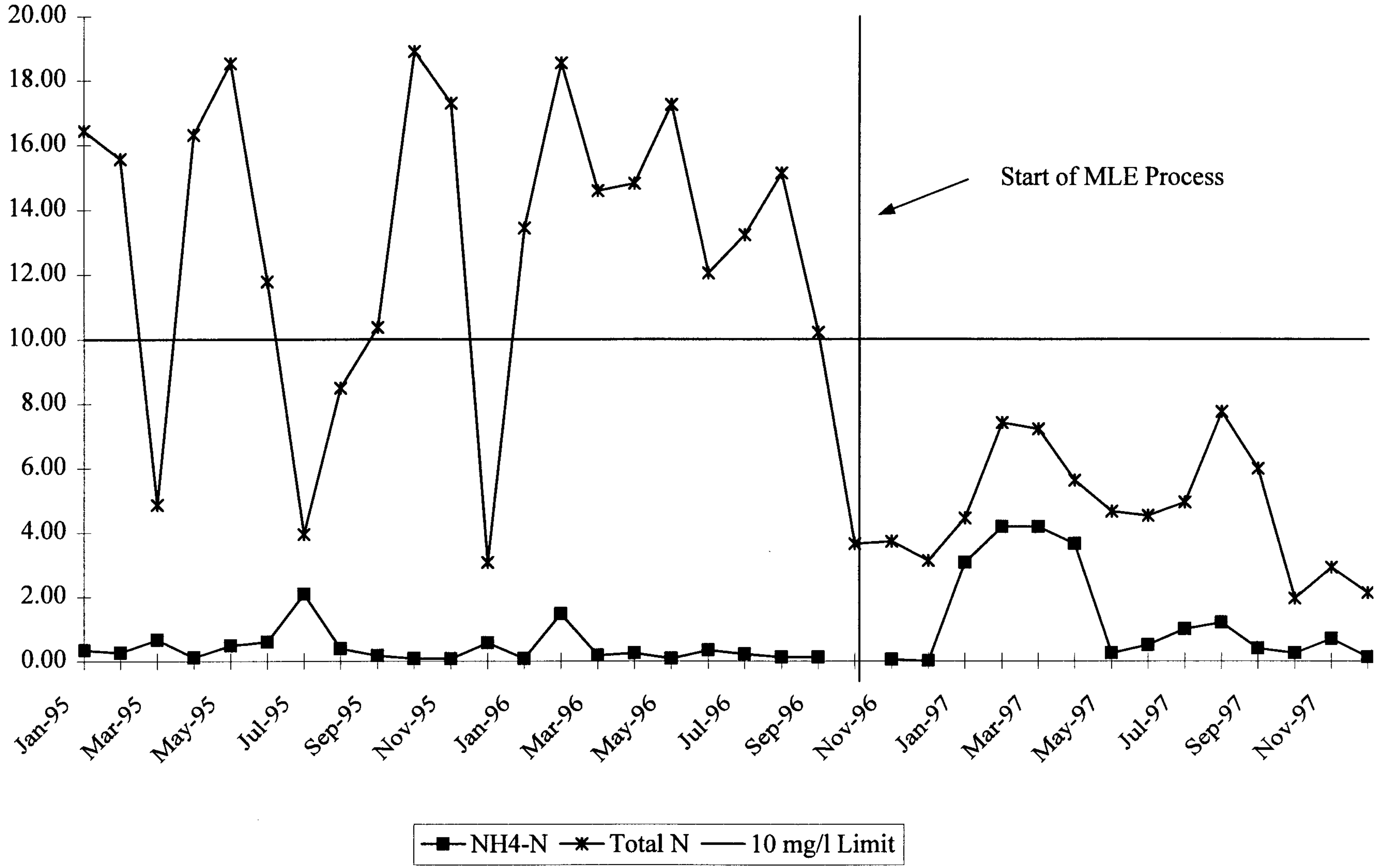

FIGURE 5-4 Nitrogen Concentrations Before and After MLE Startup
Comprehensive Wastewater Management Planning Study Town of Chatham, MA
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4/26/99
chat5fig.XLSFig (N-Plot)

facilities on the Chatham WPCF site. The existing wastewater flow schematic, Figure 5-5, provides a graphic presentation of the wastewater treatment and the flow of the wastewater through the Chatham WPCF.

An inventory of the treatment facilities and process equipment referenced in this and subsequent sections is attached to this report as Appendix I.

a. Pretreatment facilities. The pretreatment facilities consist of a coarse bar screen, and a fine bar screen and a Parshall Flume. Wastewater from the 6-inch and 8-inch force mains first flow through a 2-inch coarse bar screen. This bar screen is located inside the septage degritting room. The flow then passes through a ¾-inch fine bar screen and 6-inch Parshall Flume before flowing to the aeration tanks. The bar screens are cleaned manually on a daily basis, and the screenings are combined with the grit from the septage and disposed of at the Bourne lined landfill. A comminutor was located in parallel to the fine bar screen, but it has since been removed.

Following screening and degritting, the wastewater flows through a 6-inch Parshall flume, where the flow is metered. At the Parshall flume, an ultrasonic level sensor measures the water surface elevation and sends a flow signal, which is recorded at the control building.

b. Aeration facilities. The aeration facilities are comprised of four aeration tanks as shown on Figure 5-1 (No. 1, No.2, No. 3, and No. 4), each with its own surface aerator, and flow control gates located at the inlets and discharges of the tanks. Only tanks Nos. 3 and 4 are used for wastewater treatment as shown on Figure 5-5. Tanks Nos. 1 and 2 are used for sludge storage.

Each of the four aeration tanks is 13,636 cubic feet with dimensions of 37 feet square by 10.2 feet deep. The total volume of these four tanks is 54,544 cubic feet. Only two of the four aeration tanks (Nos. 3 and 4) are used for the activated

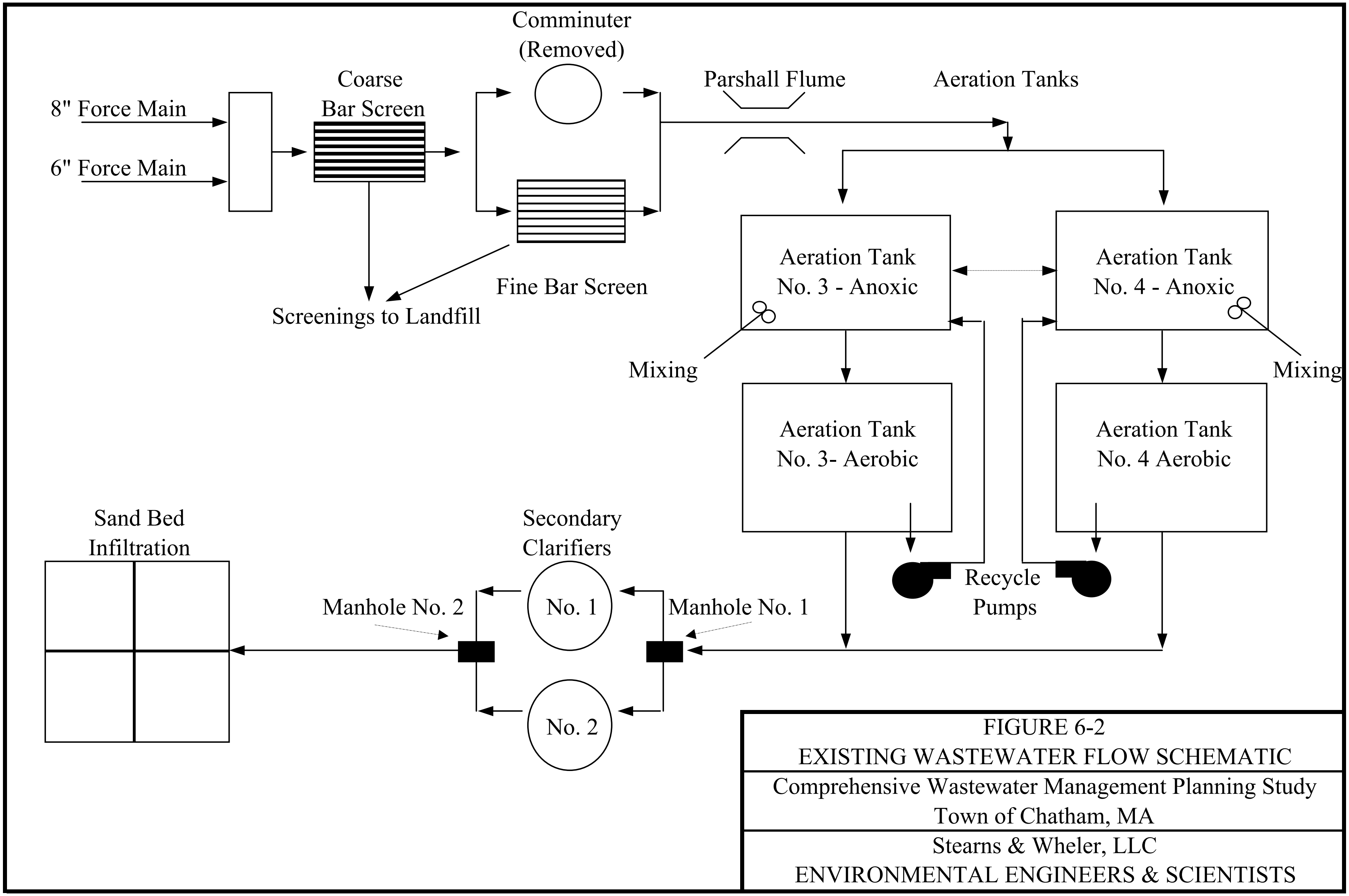

FIGURE 6-2
EXISTING WASTEWATER FLOW SCHEMATIC

Comprehensive Wastewater Management Planning Study
Town of Chatham, MA

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sludge wastewater treatment process. These two tanks were modified, in 1996, for nitrogen removal. Concrete baffle walls were installed in these two tanks for the nitrogen removal process. The baffle walls divide these tanks into an anoxic and an aerobic side. Each anoxic zone is 4,010 cubic feet and each aerobic zone is 9,625 cubic feet.

The remaining two aeration tanks (Nos. 1 and 2) are currently used for septage, sludge and trap grease pretreatment and are discussed in further detail in following sections.

As described in Section 5.1, the Chatham WPCF was modified in 1996 to utilize the modified Ludzack-Ettinger (MLE) nitrogen removal process. This process utilizes a dedicated aerobic zone (a zone that is constantly aerated and mixed) for biological treatment of the Biochemical Oxygen Demand (BOD) and conversion of the ammonia nitrogen to nitrate nitrogen (nitrification). It also utilizes a dedicated anoxic zone (a zone that is mixed but not aerated) for conversion of nitrate nitrogen to nitrogen gas (denitrification) which is released to the atmosphere.

During typical operation the two treatment trains of the MLE process receive wastewater and return activated sludge (RAS) from the influent channel. This combined flow is split into anoxic tanks Nos. 3 and 4 by sluice gates on opposite sides of the channel. Each anoxic tank is equipped with a submersible mixer to provide a completely mixed condition. Flow between the anoxic and the aerobic tanks is controlled by sluice gates in the baffle walls.

The aerobic zones of tanks No. 3 and 4 receive the mixed liquor suspended solids (MLSS) from the anoxic tanks. The aerobic tanks are constantly aerated and mixed by the mechanical surface aerators, and the ammonia nitrogen is converted to nitrate nitrogen (nitrification). Recycle pumps located in each aerobic zone recycle MLSS flow back to the anoxic zone of the tank where it is denitrified.

Flow from the aerobic zone of the tank flows over the effluent weir, down the effluent channel and into a 12-inch collection pipe. This 12-inch pipe carries the flow to Manhole #1, and into one of the two secondary clarifiers. This treatment flow is illustrated in Figure 5-5.

Occasionally, during winter operation, the overall process is modified to keep only one MLE treatment train is in operation. Flexibility in the system allows for both anoxic tanks to be in service during the winter months. With both anoxic zones in operation, influent flow enters into the anoxic zone of tank No. 4. The return activated sludge also enters this tank from the RAS distribution box, in order to prevent the distribution box from freezing. The MLSS flows from anoxic tank No. 4 to anoxic tank no. 3 and finally to the aerobic zone of tank No. 3. The recycle pump then pumps the MLSS back to anoxic tank No. 4. The mechanical aerator during the winter is cycled 30-minutes on and 15-minutes off during the day, and 15-minutes on and 15-minutes off at night to save electricity and to prevent excessive cooling of the process liquid.

c. Secondary clarifiers. The secondary clarifier facilities consist of Manhole #1, Manhole #2, two secondary clarifiers, and two return sludge pumps. Manhole #1 is located upstream of the secondary clarifiers, and manhole #2 is located downstream of the secondary clarifiers. The secondary clarifiers are 30 feet in diameter with a seven foot side water depth. Mixed Liquor Suspended Solids (MLSS) flows from the center of the clarifiers radially to the effluent weir, while activated sludge solids settle to the tank floor. Return and waste sludge are collected in a central hopper. Scum is continuously skimmed from the tanks and discharged to the scum pits. Two RAS pumps are used for returning and wasting sludge. The pumps either return sludge to the aeration tanks (Nos. 3 and 4), or waste sludge to tanks nos. 1, 2, and 5 (septage/sludge holding tank). Only one clarifier is used, and this clarifier is covered during the winter to prevent freezing problems.

Clarified effluent overflows the effluent weirs and is conveyed via two 12-inch pipes to manhole #2. From this manhole the effluent flows by gravity to the infiltration beds through a 12-inch pipe.

2. Performance. The overall plant performance is discussed in Section 5.1E. The performance of the major individual wastewater treatment components is discussed in this section.

The combined performance of the aeration tanks and final clarifiers is indicated by the effluent BOD and total nitrogen, which were discussed in Section 5.1E. The aeration tanks perform very well at removing BOD and the effluent has not exceeded the 30 mg/l limit in the past three years. Since the installation and operation of the MLE process in September 1996, the WPCF has achieved a total nitrogen content below the 10 mg/l limit. On two separate occasions, in March and April of 1997, the total nitrogen did exceed the 10 mg/l limit but the monthly average concentration remained below 10 mg/l.

The performance of the secondary clarifiers is indicated by the effluent TSS and was discussed in Section 5.1E. From 1995 to 1997, there have been no occurrences of TSS exceeding the 30 mg/l limit. The secondary clarifiers have consistently produced acceptable effluent.

3. Capacity. The capacity of each major component of the wastewater treatment facility was assessed. Capacity determinations were based on hydraulic calculations, review of current performance, comparison with accepted design standards, and process calculations. The resulting capacity determinations for WPCF components are summarized on Table 5-9. Discussions of the assessment of various components are presented below.

The major components of the pretreatment facilities are the bar racks and Parshall flume. The capacity associated with the Parshall flume is sufficient for the facility. The six-inch

TABLE 5-9

CAPACITY SUMMARY OF EXISTING WPCF COMPONENTS
Comprehensive Wastewater Management Planning Study
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Component	Capacity (mgd)
Parshall flumes (peak hour) (1)	2.5
Surface Aeration for MLE Process Tanks (2)	0.2
MLE process tanks (2)(3)	0.3
Secondary clarifiers (2)	
Average annual (4)	0.3
Peak flow (5)	0.8

Notes:
1. Value from Isco Open Channel Flow Measurement Handbook (4th Edition)
2. Both tanks in service, maximum month conditions
3. The aeration tanks would have this capacity if diffused aeration was installed
4. Based on 240 gpd/ft2 overflow rate due to a 7 foot side water depth
5. Based on 560 gpd/ft2 overflow rate due to a 7 foot side water depth

Parshall flume has a peak hour capacity of 2.5 mgd, which is greater than the 0.2 mgd peak day flows currently, experienced during the summer months.

The biological treatment process capacity was evaluated using methods developed by Stearns & Wheler and others as documented by the USEPA Nitrogen Control Manual. Capacity of the aeration system was evaluated, as well as the volume of the aeration tanks. The four aeration tanks were originally rated at a capacity of 0.44 mgd (0.11 mgd per tank) for secondary treatment. Stearns & Wheler's evaluation indicates that the surface aeration system can provide sufficient oxygen transfer to treat a flow of 0.1 mgd per tank at the maximum month BOD and TKN concentrations of 250 and 45 mg/l, respectively. If diffused aeration was installed in the tanks, the aeration capacity and tank capacity could be increased to 0.15 mgd per tank or 0.3 mgd for two tanks. The winter capacity is approximately equal to the summer capacity due to the ability to carry a higher mixed liquid suspended solids in winter and a higher oxygen solubility at colder temperatures. These two factors tend to offset lower biological activity rates that occur at colder temperatures.

The capacity of the secondary clarifiers is assessed based upon surface overflow rate and solids loading rate. Facilities that maintain the longer solids retention times necessary for nitrification often produce a slower settling sludge than facilities with shorter solids retention times. Surface overflow rates for Chatham' clarifiers should generally be maintained at less than 240 gpd/ft^2 average and 560 gpd/ft^2 peak due to the low side water depth (seven feet) of the clarifiers and occasional poor settling characteristics of the sludge. Solids loading rates for Chatham's clarifiers should generally be maintained at 25 lbs/ft^2/day or less. Based upon the above criteria, the existing secondary clarifier capacity is approximately 0.3 mgd at annual average flow and 0.8 mgd at peak hour flow with two clarifiers in service.

The RAS pumps are each rated for 50 to 150 gpm, according to the nameplate information. Each of the MLE recycle pumps have a capacity 320 gpm.

4. Current Problems and Planned Solutions. A list of current problems with the wastewater treatment facilities has been developed through assessment of the existing facilities and discussion with the WPCF staff. Solutions for several of the problems have already been identified. Others may require further study to identify the proper solution. The list of identified problems and proposed solutions is presented below.

a. **Grit removal**. The WPCF currently has no influent grit removal facilities. Grit has become a big problem at the WPCF because it settles to the bottom of tanks. The WPCF was originally equipped with a Dorr Oliver cyclone degritter called a Dorr Clone. This cyclone degritter was located outside, and was plagued with freezing problems. The unit was removed 10-15 years ago and never replaced.

b. **Screenings removal**. Removal of screenings (large solids that could potentially be removed with a fine screen) in the plant influent is a problem at the WPCF. Currently the facility has two bar screens in the influent channel, which do not remove all the screenings from the influent flow. Originally a comminutor (device for shredding screenings) was located in the 18-inch channel adjacent to the fine bar screen. This was removed due to operational problems. A new comminutor has been requested for FY 2000.

c. **Aeration tanks**. Since the implementation of the MLE process, nitrogen effluent concentrations have been within the facilities permit limits. In March and April of 1997, there were two separate occurrences where the total nitrogen exceeded the 10 mg/l limit. These each occurred in separate months, so the monthly averages for March and April were still below the 10 mg/l limit. The total nitrogen exceedances are presumed to have occurred because the MLE process was operating below the recommended pH levels. One recommended solution would be the installation of an alkalinity feed of sodium hydroxide or other form of metered chemical, which would automatically regulate the pH of the system.

The aeration tanks have no dissolved oxygen (DO) controls.

The mechanical aerator's gear drives are approximately 25 years old and original equipment. These drives are old and need to be rebuilt. The motors for the mechanical aerators are also original equipment and have been rewound and refurbished since their original installation.

Aeration tanks Nos. 3 and 4 have submersible mixers. There is no backup for this equipment. The same is true for the recycle pumps used for the MLE process. Both of these pumps had problems following their installation in 1996, and have since been rebuilt. An uninstalled standby mixer and pump should be available as a backup. A flow meter was also installed on each of the pump discharge lines. They were calibrated to the pump flow when they were first installed but the pumps were improperly wired and ran backwards. The pumps were repaired, but the flow meters are no longer calibrated properly, and indicate flows greater than 150 percent of the design flow. These flow meters should be recalibrated.

The aeration tank effluent weirs are adjustable, but currently are unused due to missing parts. The couplings for the weir operators should be replaced and these weirs should be exercised regularly as preventative maintenance.

Each of the aeration tanks is equipped with a drain located at the center of the tank. The MLE process divided tanks 3 and 4 so that the anoxic side of each has no drain. To drain these tanks, a dewatering or sump pump is lowered into the anoxic side and the MLSS is pumped over the baffle wall to the adjacent tank.

The lack of influent degritting has resulted in grit buildup in the aeration tanks. On average, a total of five to six cubic yards of grit is removed from the four tanks per year. This grit build up reduces the effective volume and impacts the efficiency of the MLE process, reducing the retention times. To clean the tanks, each tank is emptied and the grit is removed manually. This cleaning process is preformed once every year. The installation of a grit removal system in the influent channel and tank drains would help reduce the grit buildup, provide easier methods to remove the grit and provide a more effluent use of the aeration tanks.

The aeration tanks experience foaming. Grease in the wastewater is suspected to be a major contributor to the generation of this foam. The foam, possibly a result of Nacardia bacteria present in the wastewater, can also be a result of extended solids retention times, a common result of nitrification/denitrification processes. Limited chlorination of the return activated sludge is a typical method to help reduce the foam. Also, the solids retention time should be closely monitored, and the MLSS concentration should be kept as low as possible.

d. **Return activated sludge equipment**. The two RAS pumps are rated at 150 gpm and run at approximately 90 percent or 135 gpm. The pumps are equipped with speed reducers, but operation below 70 percent results in clogging from rags. Communitors are proposed to solve this problem. The RAS pumps are also recommended for replacement with similar pumps equipped with variable frequency drives (VFDs) instead of the mechanical drives.

The RAS lines are equipped with a polysonics flow meters, which were installed eight years ago. The operational staff are not satisfied with the performance of these meters and new magnetic flow meters are recommended as a replacement.

The RAS pumps are also used to waste activated sludge because there is only one line from the pump gallery to each clarifier. This is seen as a limitation as one line must be used to pump RAS, WAS and scum from the clarifiers. Scum and floatables clog the RAS pumps and air binding occurs when the scum is drained from the lines. The RAS line also requires flushing following any pumping of scum from the clarifiers, to avoid residual scum from entering the aeration tanks. Currently the scum boxes are emptied by a septage pumping truck, and discharged into tanks No. 1, 2, or 5 at a cost of $50 per pumping. The scum pit is pumped once per month in the winter and twice per month in the summer.

The return sludge distribution box was designed to operated with four V-notch weirs, controlling flow to each of the four aeration tanks. The operators for the V-notch weirs are rusted and are not functioning. The operators are also missing pieces and all of the V-notch weirs are missing. Flow only discharges through one weir; the remainder have been blocked with wood to keep flow from the other tanks. This structure should be repaired and missing parts replaced. This would provide increased flexibility for returning sludge to tanks nos. 3 and 4, and provide an easier means to waste sludge to tanks nos. 1 and 2.

e. **Secondary clarifiers**. Only one of the two secondary clarifiers is currently used. The weir elevations on the two clarifiers are not the same, and this causes problems when both clarifiers are used at the same time. Depending on the elevation difference, weirs can either be adjusted or replaced to be at the same elevation. The clarifiers also experience freezing problems in the winter time. One clarifier is covered by a tent, but a similar covering system is required if winter use of the second clarifier becomes necessary.

The clarifiers also experience hydraulic overloading in the mornings, resulting in the sludge blanked rising. This is suspected to be a result of several commercial establishments that discharge directly to the force mains. Timers could be installed on these pumps to discharge during times of low flow.

G. Septage Handling Facilities.

1. Description. The septage handling facilities are comprised of the following main components: septage receiving station, holding tanks, degritting equipment, and grit pump. These components are shown schematically in Figure 5-6.

Septage is discharged directly from the haulers' tank trucks through a 6-inch pipe to the septage holding tank. The septage passes through a rock trap and a coarse bar rack. The rocks and screenings are manually raked following each discharge by a hauler, and disposed of in a covered container as required.

The septage flows by gravity to a 600 cubic foot septage holding tank located below the septage degritting room. Any overflow from this tank flows to the trap grease holding tank, adjacent to the septage holding tank. A recessed impeller centrifugal grit pump transfers the septage to the degritting room.

The raw septage is pumped through a teacup solids classifier, and the grit settles out in the bottom of the cyclone. The teacup must be blown down one time every half-hour with effluent water to remove the grit. The fluidized grit flows into a decanter, which allows the supernatant to drain. Periodically, the decanter is drained through a screened opening, and the decanter is tipped to the front to allow the grit to be emptied into a bobcat wheel loader. The grit is then delivered to a rolloff container where it is covered with lime, and taken to a lined landfill.

Typically, the degritted septage then flows to aeration tank no.2, where it is held and aerated. Occasionally, the aeration is turned off, and supernatant is decanted to the MLE process. The diffuser at the bottom of tank no. 2 is a four inch perforated PVC pipe. This diffuser was originally installed as an air sparger for use in association with a mixer, which has been removed. Air is supplied to the sparger by a positive displacement blower located in a small shed adjacent to the aeration tank.

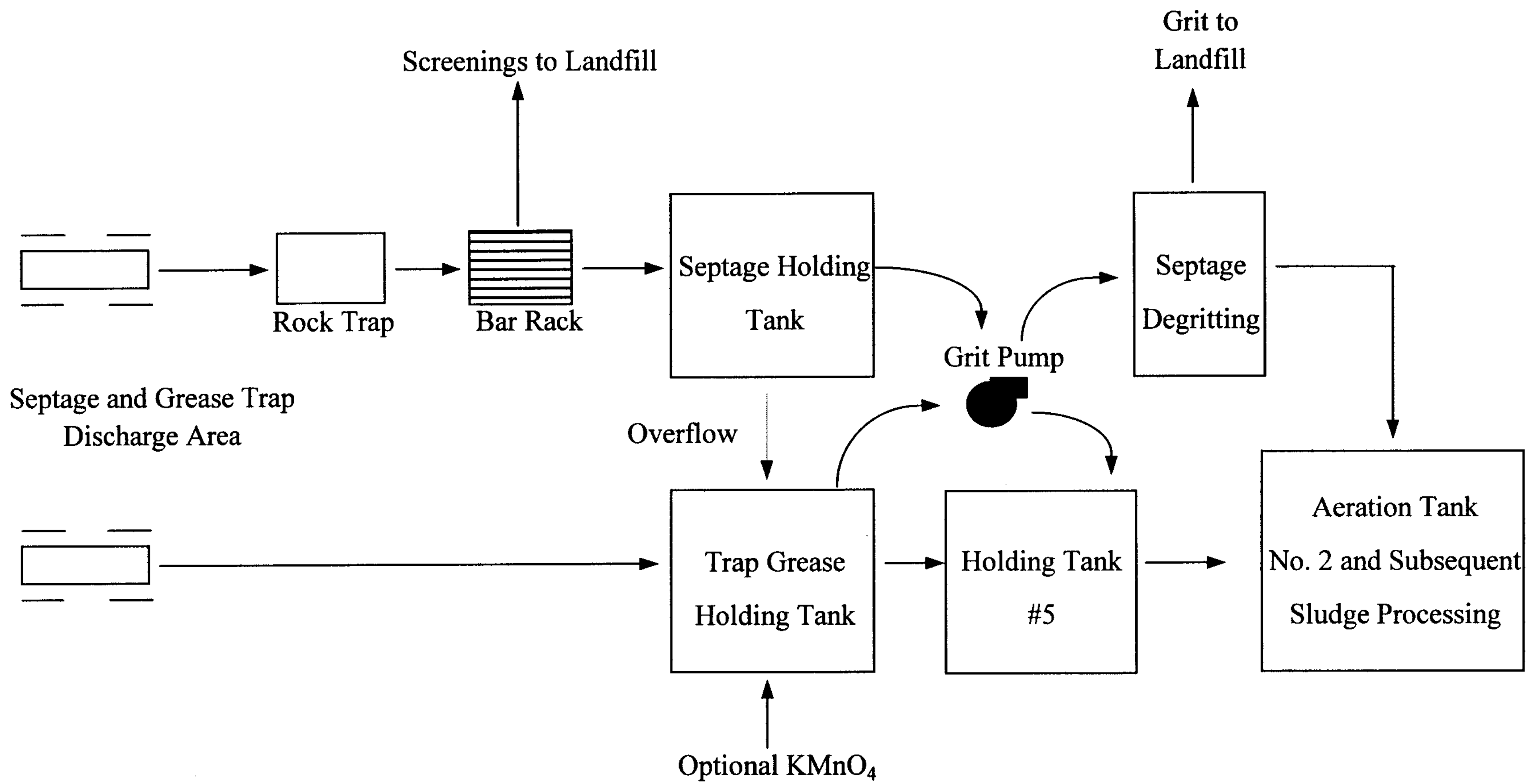

FIGURE 5-6
EXISTING SEPTAGE AND TRAP GREASE FLOW SCHEMATIC

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The degritted and settled septage is then pumped through the teacup to aeration tank no. 1 by the grit pump. The material is aerated and settled in tank no. 1. When it reaches a concentration of 0.8 to 1 percent total solids, it is typically ready for dewatering. Tank No. 1 has a surface aerator, which is periodically used for mixing and aerating.

2. Performance. The teacup classifier performs fairly well. The grit is well cleaned, but this equipment does require operator attention for peak performance. The septage holding tank is sufficient in size and has the flexibility of overflow to the trap grease holding tank, which has a larger volume. The tank also performs well because it has steep (45 degree) slopes on the bottom to facilitate the removal of solids to the grit pump.

3. Capacity. The WPCF receives and treats an average of 80,000 gallons per month or 2,600 gallons per day of septage. With the flexibility of five separate tanks (septage holding tank, trap grease holding tank, and tank nos. 1, 2, and 5) for storage and treatment, for a total capacity of 250,000 gallons or 35,000 cubic feet, the current capacity is more than sufficient to handle current septage flows.

The grit pump used to pump both septage and trap grease has a capacity of 200 gpm. The teacup classifier also has a 200 gpm capacity. The self dumping decanter has a capacity of 1.5 cubic yard. This equipment and associated capacities are summarized in Appendix I. The WPCF uses a 9 cubic yard hopper for grit and screening storage for disposal. Typically, this material is disposed of when the hopper reaches a quarter of its capacity.

4. Current Problems and Performance Limiting Factors. The grit pump used to transfer septage to the degritting room is part of the original equipment installed in 1972, and is approaching or surpassed its design life. The Town of Chatham has recommended this pump for replacement in fiscal year (FY) 2000.

The remainder of the septage degritting equipment was installed during the 1996 WPCF upgrade. Operators have expressed concerns with the efficiency of the equipment's ability to remove grit, especially sand. The decanter is only emptied 4 times per year and mostly during the summer months. Cyclone degritters with screw classifiers have been identified to produce a drier grit and more reliable results. The self-dumping decanter clogs during decanting if there is not sufficient sand in the bottom to filter the screenings and other filamentous materials removed during the degritting process. The operators are required to "probe" the bottom of the decanter prior to use, to check for sufficient material depths. This is a high maintenance system.

H. Trap Grease Handling Facilities.

1. Description. The WPCF receives and treats trap grease. Septage haulers discharge the trap grease into a 6-inch pipe adjacent to the septage discharge pipe. The discharge pipe carries the grease directly to a 1,100 cubic foot trap grease holding tank. The tank is located immediately west of the septage holding tank, below the teacup classifier. Periodically, the holding tank is discharged into Tank No. 5. This tank is a 5,600 cubic foot tank located south of the degritting room and was originally designed as a sludge storage tank.

The grease is mixed in the larger tank and treated with potassium permanganate prior to being discharged into Tank No. 2 with the septage and waste sludge. Solids from this tank are pumped to the dewatering building for processing on the belt filter presses. Following dewatering, the sludge cake is disposed of at the Yarmouth Septage Treatment Plant.

2. Performance. This method of treating trap grease is working well, and it is successful in isolating grease from the MLE wastewater treatment process. This process requires careful operator attention, and is flexible to allow longer retention time if it is needed.

3. Capacity. The WPCF receives and treats 19,000 gallons of trap grease per month or 600 gallons per day. Similar to the septage, trap grease has the flexibility of using five separate tanks (septage holding tank, trap grease holding tank, and tank nos. 1, 2, and 5) for storage and treatment, with a total capacity of 250,000 gallons or 35,000 cubic feet. This is sufficient capacity for the current flows.

The capacity of the grit pump was discussed in the previous section on Septage Treatment.

I. Sludge Handling Facilities.

1. Description. The sludge handling facilities are comprised of the following major components: secondary sludge handling facilities, dewatering facilities, and related pumping facilities. The existing sludge handling facilities are shown schematically on Figure 5-7.

Return Activated Sludge (RAS) is pumped from the secondary clarifiers to the aeration tanks, where it is mixed with WPCF influent flow. Waste Activated Sludge (WAS) is also pumped from the secondary clarifiers to either tank no. 5 for holding and blending with trap grease, or to tank nos. 1 and 2. Sludge from tanks 1 and 2 is run through a grinder before being pumped to the dewatering building by Belt Filter Press (BFP) feed pumps for dewatering.

A chemical feed system is used to meter polymer to the sludge as it enters the dewatering building and is transferred to the BFPs. There are two 1-meter BFPs, which dewater the sludge and discharge it to a hopper for disposal at the Yarmouth Septage Treatment Facility. Typically only one BFP is used during the dewatering process.

Sludge is pressed when the concentration of the liquid sludge in tank No. 1 or 2 reaches a minimum of 0.8 to 1 percent total solids. Sludge is dewatered typically once per week and fills a bin following six to eight hours of operation. Sludge production averages five wet tons per week during the year. Occasionally, there is enough sludge to press two hoppers per week. Each full hopper weighs approximately five or six tons.

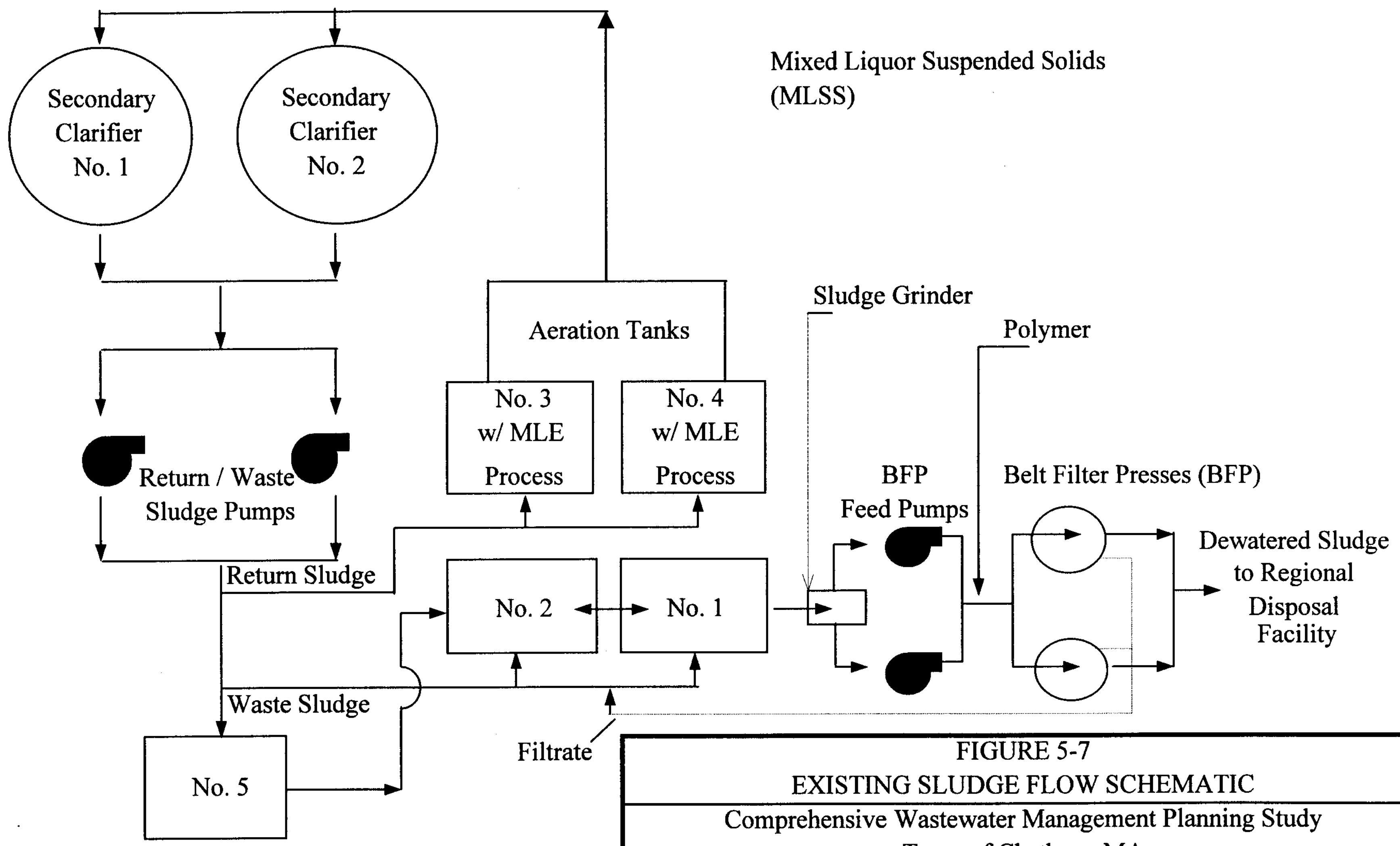

FIGURE 5-7
EXISTING SLUDGE FLOW SCHEMATIC

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2. Performance. Sampling and analysis was performed on the BFP feed and waste steams and is summarized on Table 5-7 and Appendix 5-1. The average feed concentration was 2.6 percent, and the average concentration for the dewatered sludge cake was 17 percent. Typical performance of BFPs for activated sludge is 12- 20 percent solids for the sludge cake; therefore, this represents good performance.

3. Capacity. The capacities of the sludge handling equipment are compiled in Appendix I.

Sludge is fed through a grinder prior to the pump. The grinder's rated capacity is 200 gpm at 0.75 percent solids. Currently, one positive displacement sludge feed pump is used to feed one belt at 100 gpm.

Each belt filter press has a one-meter belt, and a capacity as stated in the operation and maintenance manual of 100 gpm. Typical hydraulic loading capacities for machines of this type range from 25 to 100 gallons per meter per minute.

The RAS pumps and their capacities are discussed in the Wastewater Treatment Facilities section of this chapter.

4. Current Problems and Performance Limiting Factors. The Town currently has to rely on the Yarmouth Septage Treatment Facility and Sludge Composting facility for their sludge disposal needs. When the sludge composting facility went down for a short period of time, the WPCF was forced to store liquid sludge and dewatered sludge for an extended period. The Town would like to examine viable sludge disposal options, which are not dependent on a regional facility. Disposal at the regional facility costs an average of $23,000 per year, or approximately $2,000 per month.

The sludge feed pumps are positive displacement rotary lobe pumps manufactured by Lobeflo – MGI Pumps Inc. Each pump is capable of pumping 100 gpm, but with only one pump functioning, there is no backup. The BFPs are each capable of handling 100 gpm of sludge flow, so typically only one press is used. Both pumps were placed in service in 1990 but have experienced operational difficulties recently.

J. Effluent Disposal.

1. Description and Capacity of Existing Discharge Beds. The effluent from the secondary clarifiers flows by gravity through a 12-inch pipe to the four sand infiltration beds. Routing of the effluent is controlled through manual operation of gate valves and shear gates at two manhole locations. Each sand infiltration bed is fed from two six-inch pipes at the manholes.

Each infiltration bed has a leaching area of 41,400 square feet for a total of approximately 166,000 square feet. The infiltration beds contain three feet of sand, and the side slopes are at a two to one ratio and covered with four inches of 1.5 to two inch diameter stone.

Based on an average design hydraulic loading of the sand beds at five gpd/square foot of bed area, and resting of half the beds at one time, the plant's disposal capacity is approximately 410,000 gpd. The ACO administered by the MADEP has currently limited the WPCF to 150,000 gpd, and this is well below the above stated hydraulic capacity for these beds. This limit is based on potential impacts to drinking supply wells in the area and the impact of the effluent on the local groundwater elevations.

2. Description of Existing Groundwater Mound. The WPCF is located adjacent to the Chatham Sanitary Landfill, residential and commercial properties. The facility is also within 1.5 miles of five of the drinking water supply wells for Chatham, including: Indian Hill Well, the Town Forest Wells, and the South Chatham Wells.

The operation of these drinking water supply wells creates a cone of influence around the wellheads, which draws the natural groundwater velocity to the wells. The discharge from the WPCF has a potential of migrating to the supply wells. As part of the ACO issued in 1987, extensive groundwater modeling studies have been developed and quarterly sampling reports are produced each year characterizing the groundwater mound created by the WPCF.

The quarterly reports show no migration towards the South Chatham or Town Forest Wells. Groundwater modeling indicates that effluent flows exceeding the 150,000 gpd limit may push groundwater flow toward the Indian Hill Well. The groundwater flow direction is normally south toward Cockle Cove Creek, away from all drinking supply wells. The infiltration beds discharge the effluent to the upper aquifer system (as described in Chapter 4). The drinking water supply wells draw from the lower aquifer, and although there may be some leakage between the two aquifer systems, the effluent has negligible effects on the lower system based on the information gathered from previous groundwater studies at the Chatham WPCF.

3. Description of Existing Groundwater Plume. In conjunction with all the groundwater mounding studies, groundwater quality samples have been collected to examine the effluent impacts on the groundwater. Samples were analyzed for nitrate-nitrogen, ammonia-nitrogen, TKN, alkalinity, sodium, and chloride. Five wells were also analyzed for VOCs, and field parameters were taken for each well including specific conductance, temperature, and pH.

Results showed all samples were below the MCLs for drinking water, except sodium concentrations in wells MW-1-45 and SW-7. These wells are south of the WPCF and inside the assumed effluent plume.

K. Residuals Disposal.

1. Biosolids. The monthly average sludge generation from the State monthly reports and disposal invoices are summarized in Appendix H and include WAS flow given in 1000 gallons per month, sludge feed in 1000 gallons per month, and sludge cake disposal in wet tons per month. Sludge is disposed at the Yarmouth Septage Treatment Facility. From this facility, Waste Stream Environmental (WSE) trucks the treated sludge to either a landfill or an incineration facility.

The 1997 production of sludge is summarized below:

Sludge Stream	Average Annual	Maximum Month
WAS (1000 gal/mo.)	67	105
BFP Feed (1000 gal/mo.)	54	108
Sludge Cake (wet tons/mo.)	24	47

2. Screenings and Grit. Screenings and grit are generated by operation of the hand-cleaned bar screens and the septage degritting facilities in the septage degritting room. The screenings are combined with the grit and lime is applied. The grit and screenings are then taken to the Bourne Landfill. Average screenings and grit production is approximately 0.9 ton per month.

L. Comments and Recommendations on Operation and Maintenance and Interim Improvements.

- Operation and Maintenance costs for Fiscal year 1997 are summarized on Table 5-10.

- Capital improvement projects for the Chatham WPCF and collection system are summarized on Table 5-11.

TABLE 5-10

CHATHAM WPCF
1997 O & M COSTS [1]
Comprehensive Wastewater Management Planning Study
Town of Chatham, Massachusetts

Item	Cost
Salaries (2)	$60,500
Services	$35,500
Electricity (WPCF)	$27,000
Electricity (Pump Stations)	$10,000
Water	$5,000
WPCF Maintenance	$40,000
Solid Waste Disposal	$30,000
Contract Services (3)	$187,100
Supplies	$9,100
Chemicals	$10,000
Expense	$7,000
Groundwater Monitoring	$30,000
Debt Service	$282,300
Queen Anne Pump Station (4)	$25,000
Total Cost	$785,500

Notes: 1. Based on 1998 budget appropriations
2. Town office personel
3. Salaries for 4 operations personel
4. Auxillary generator installation capital improvement

TABLE 5-11

CHATHAM WPCF AND COLLECTION SYSTEM
CAPITAL IMPROVEMENT PROJECTS FOR FISCAL YEAR 1998-2000 (1)

Comprehensive Wastewater Management Planning Study
Town of Chatham, Massachusetts

Fiscal Year	Project Title	Amount Requested
1998	Queen Anne Sewage Lift Station Generator	$34,020
1998	Sewer Safety Improvements	$8,308
1998	Comprehensive Wastewater Management Plan and Data Gap and Admin.	$100,000
1999	Queen Anne Station Ultrasonic Liquid Level Pump Control	$8,000
1999	Replace Return Activated Sludge Pumps	$20,500
1999	Replace Pump #2 at Stage Harbor Pump Station	$35,000
1999	Replace Pumps at the Queen Anne Sewage Lift Station	$19,200
1999	Install Security Fence Around Sand Infiltration Beds	$35,000
2000	Install WPCF Influent Comminutter	$19,000
2000	Replace Stage Harbor Station's Communitor	$19,000
2000	Upgrade Portable Auto. Composite Samplers with Stationary Refrig. Units	$8,000
2000	Replace WPCF Grit Pump	$23,000
2000	Replace Sewer Jetwasher	$35,000

Notes: 1. Based on Town of Chatham's Capital Improvement Program for fiscal year 1999

- The Chatham WPCF has exceeded the 10 mg/l total nitrogen limit a couple of times in the winter and early spring of 1997. Ammonia nitrogen values were observed to reach elevated levels as illustrated in Figure 5-4. These high values have been attributed to low pH and an inability to monitor and control alkalinity. The Town may want to add automatic alkalinity feed facilities and operate both aeration tanks in the winter.

- Several pumps have exceeded their design life as identified in previous sections. These pumps should be rebuilt or replaced in the near future as allowed by the operating budget.

- The production of scum and foam in the aeration tanks is a problem when surface aerators are used for wastewater treatment processes with a long solids retention time. It is further aggravated if the wastewater has a high grease component. Work at other treatment plants indicates that the foam can be minimized by reducing the solids retention time and reducing the MLSS. The reduction in foam production will create easier operations and improved effluent quality.

M. Summary. Overall, the existing water pollution control facility is operating well and is well maintained. The preceding summary of the existing facility will serve as a reference and provide a basis for evaluation of treatment alternatives in future phases of this project. Current problems have been described and discussed so that they can be addressed earlier individually or as part of a future plant upgrade.

5.2 EXISTING WASTEWATER COLLECTION SYSTEM

A. System History. The majority of the Town's sewer system was constructed in downtown Chatham in 1971. The downtown area is sewered with 8, 10, and 12-inch diameter asbestos-cement (AC) pipe, and all branches eventually feed the Stage Harbor Pump Station. The collection system has been slightly expanded since 1972.

A sanitary sewer typically has a rated design life of 50 years. However, with proper system maintenance and appropriate repairs, it is possible to extend the life of a sewer significantly. The age and lengths of various lines in the system are detailed in the Table below.

Overview of Town of Chatham Sewer System

Time Period	Approximate Length of Sewers Installed (feet)	Predominant Material	Percentage of Overall System
1969 –1971	19,511	Asbestos cement	80%
1972-1988	3,300	Unknown (1)	13%
1988-Present	1,650	Unknown (1)	7%

Note (1): Record drawings not available.

Most of the sewers in the Town of Chatham are well under 50 years old. The earliest any of Chatham's collection system will reach the 50-year design life is in 2021. Also, based on the information received from the Chatham WPCF operators, these sewers have been operating with minimal problems. A further discussion of operation and maintenance is contained in Section 5.2 (F).

B. Summary of Existing Facilities.

1. Gravity Sewers. The Town has approximately 24,000 linear feet of gravity sewers, or approximately 4.5 miles. They range from 8-inch to 12-inch in diameter. The 8-inch sewers account for over 70-percent of the total collection system. A breakdown of the gravity system by size is presented in the Table below.

Overview of Town of Chatham Gravity Sewer System

Diameter (inches)	Length (feet)	Percentage of Overall System
8	17,824	73%
10	1,472	6%
12	215	< 1%
Unknown (1)	4,900	20%

Note (1) Record drawings not available.

The extent of the wastewater collection system is illustrated in Figure 4-4.

The MADEP issued an administrative consent order (ACO) in 1988, limiting the discharge at the WWTF. As part of the order, the Town was prohibited from creating additional connections except where the Chatham Board of Health finds (and certifies in writing to DEP) that the connection is necessary to abate an imminent hazard to public health caused by inadequate sewage disposal. Since the issuing of this ACO, the Town has extended its sewer system to include the High School property, and a development on Nonatum Lane, Elkanah Street, and Konuhasett Way. These additions have accounted for approximately 1,600 linear feet of sewer.

2. Pumping Stations and Force Mains. In addition to the gravity sewers, the Town of Chatham operates and maintains the following four pumping stations:

- Stage Harbor
- Chatham Housing Authority
- Queen Anne
- Mill Pond

There are also four pumping station outside the responsibility of the Town's Water and Sewer Department at the High School, and three commercial establishments (The Corn

Field, Frog Pond Laundry, and Old Harbor Fish Market). These three pumping stations have direct connections to the 6 and 8-inch force mains, which run to the WWTF.

The High School and the Chatham Housing Authority pumping stations collect wastewater and discharge it via force main to gravity sewers which feed the Queen Anne pumping station. Queen Anne and Mill Pond discharge their wastewater via force main to gravity sewers, which feed the Stage Harbor pumping station. In addition to these flows, the Stage Harbor pumping station also collects wastewater from other gravity sewers near the downtown area of Chatham.

There is approximately three miles of force main in Chatham, ranging from 4-inch to 8-inch in diameter. The following table presents information on the force mains and Town owned pumping stations.

Summary of Pumping Stations and Force Mains

Pump Station	Year Built	Force Main Size (inches)	Approximate Length (ft)	Force Main Material	Discharge Location
Mill Pond	1971	4	565	AC	8-in. gravity sewer on Mill Pond Road
Queen Anne	1971	6	450	AC	8-in. gravity sewer on Queen Anne Road
CHA (2)	Unknown (1)	Unknown (1)	960	Unknown (1)	8-in. gravity sewer on Crowell Rd.
Stage Harbor	1971	6 and 8	13,700	AC	WPCF

Note (1) Record drawings not available

(2) Chatham Housing Authority

The Mill Pond lift station is an ejector pot type lift station. The station has a 100-gallon capacity per pot, and two pots. The station was constructed in a wetland immediately adjacent to Little Mill Pond, and recently underwent a $165,000 upgrade.

The Chatham Housing Authority pumping station is a converted septic tank, equipped with two 5-hp submersible Myers grinder pumps and float level controls. The station is also equipped with a trash basket because of the frequent clogging of the grinder pumps.

The Queen Anne pumping station is a wet well/dry well installation. The station is equipped with two Fairbanks Morse, 5-horsepower, vertical, closed coupled, dry pit pumps. The Town plans to install an ultrasonic level control with float backups, to replace the existing bubbler system, which performs poorly. Several upgrades are proposed for FY 1998 and 1999 including a new generator (completed in August 1998), and pump replacement (with identical model pumps).

The Stage Harbor pumping station is the centralized collection point for the existing sewer collection system. The station, is a wet well/dry well installation. It collects wastewater and pumps it to the WPCF via six inch and eight inch force mains. The station is equipped with three pumps: a 30-horsepower Flygt dry pit submersible pump and two 60-horsepower Allis Chalmers vertical, centrifugal pumps. This station is also equipped with an old Worthington comminutor, which the Town plans to replace with a new comminutor. The station also has a channel, which is used to facilitate grit removal. The station has recently undergone extensive renovations to the valving, piping, backup generator system, and level controls. One of the Allis Chalmers pumps is scheduled for replacement in FY 1999 with another 30-hp dry pit submersible Flygt pump.

In addition to the stations maintained by the Town, there are four pumping stations outside the responsibility of the Town's Water and Sewer Department. No specific records are kept on these stations, and a listing is presented below.

Listing of Private Pumping Stations

Location	Location of Discharge
The Corn Field (AKA Fancy's Farm)	Connects to 6 and 8 in. Town force mains
Frog Pond Laundry	Connects to Town force mains
Old Harbor Fish Market	Connects to Town force mains
High School	Discharges to gravity line on Crowell Road

C. Hydraulic Capacity of Sewers. It is important to know the full pipe capacity of all major sewer segments for planning and evaluation purposes. This knowledge combined with existing average and peak flows, helps to determine the available sewer capacity for growth and to plan for upgrades and expansions to the system. This section summarizes the hydraulic capacity evaluation of the Town's sewer system.

1. **Methodology.** The full-pipe hydraulic capacity of each section in Chatham's sewer system was computed using the Manning's equation. Pipe size and slope information were taken from the 1969 sewer design record drawings. Pipe roughness (Manning's "n" value) was based on the pipe material. Chatham's existing sewer system was constructed using AC pipe almost exclusively. An "n" value of 0.013 was used in the equation, which is typical of this type of material and pipe age.

In 1988 and 1992, evaluations of the infiltration and inflow (I/I) were performed (M&E, 1989 and 1992). These studies utilized comprehensive infiltration/inflow evaluations, which included continuous flow metering, rainfall measurement, and groundwater level measurement. The studies divided the existing collection system into two subareas ("A" and "B"). For further evaluation, smaller areas were identified ("A1" and "B1") for each main subarea to further isolate infiltration and inflow to the system. These subareas are identified in Appendix J.

The continuous flow data from the M&E reports was used to determine existing flow conditions within the sewers. This is a conservative flow estimate as the Town of

Chatham has performed repairs to the existing collection system; (per the recommendations of the 1988 and 1992 I/I studies), reducing these extraneous flows.

The instantaneous peak flow value for each sewer subarea over the entire metering period was compared with the average value over the same period to see whether the peaking factor was comparable to those observed at the treatment facility. In addition, flows were updated to 1998 values. The adjusted flow number was compared to the full-capacity of each sewer section in the subarea.

Continuous metering data or record drawings were not available for all parts of Chatham's sewer collection system. The specific sections not included are: the areas serviced by CHOPS, Henshaw Drive, Depot Road, and sections connecting manholes No.126 through No. 140, and No. 152 through No.158.

2. **Results.** In all cases, peak values were less than the pipe capacity for each sewer subarea. All the sewers installed before 1972 appear to have adequate capacity to handle current flows. The following table presents a summary of the capacity analysis results, listing the subarea, 1988 peak flow, 1998 projected peak flow, and half and full capacity of the most restrictive line in that subarea.

Summary of Capacity Analysis Results				
Subarea	**1988 Peak Flow (gpm)**	**1998 Peak Flow (gpm)**	**Half Pipe Capacity (gpm)**	**Full Pipe Capacity (gpm)**
A	80	120	172	344
B	80	120	172	344
A1	70	105	172	344
B1	60	90	172	344

The capacity analysis was performed to obtain an indication of sewer flow capacity under existing conditions. Based on the results, the current sewer sizing is adequate to handle current flows.

D. Infiltration/Inflow (I/I). In 1988 and 1992, Infiltration and Inflow (I/I) analyses were performed. The sewered sections of Chatham were divided into 2 subareas for this analyses. The 1988 study identified problem areas, and the 1992 study developed a recommended plan (M&E, 1988, 1992). Background information gathered from these reports is used in Section 5.2 (C) for the hydraulic capacity study.

Inflow to a sewer system is a direct result of a storm event or direct connection from an illicit connection. Runoff from gutters, roadways, yards drains, sump pumps, and manhole covers can result in a direct increase in sewage flows during and immediately following these events. Inflow is typically identified by measuring flow rates prior to, during and following a storm event.

Infiltration is a steady 24-hr flow that results from groundwater entering a sewer system through defects in pipes and manholes. Infiltration rates are normally measured in terms of gallons per day per inch-mile (gpd/in-mi). It is recognized that not all infiltration is cost effective to eliminate, and the Massachusetts Department of Environmental Protection (MADEP) established a criteria of 10,000 gpd/in-mi for infiltration rates to justify grant eligibility, and for performing further investigative and rehabilitation work.

A review of the 1988 and 1992 Infiltration and Inflow Report findings was conducted to estimate the I/I occurring in the collection system. A peak infiltration of 61,000 gpd was estimated based on the continuous flow monitoring, resulting in a 2,312 gpd/in-mi infiltration rate which is below the 10,000 gpd/in-mi standard set by MADEP. The peak inflow rate for the design storm was estimated at 62,000 gpd (M&E, 1988).

As a follow up to the findings of peak I/I from previous studies, the water use of the sewered properties was compared to the volume of wastewater received at the Chatham

WPCF. These two flows were found to be very similar. The water use of sewered properties in 1997 was 40,700,000 gallons or 111,400 gpd on an average annual basis. This is very similar to the 1997 average wastewater flow at the Chatham WPCF of 112,500 gpd. It is noted that not all of this water usage will become wastewater due to the following water uses: outside showers, lawn watering, and swimming pool use. An average annual I/I value was calculated by estimating wastewater generation at 90 percent of water usage and subtracting this wastewater generation rate from the wastewater flow observed at the Chatham WPCF. (This 90 percent factor is based on the ratio of the Title 5 design wastewater generation rate of 110 gpd/bedroom to the American Water Works Association water consumption rate of 120 gpd/bedroom, and discussions with the Chatham Water Department.) Average annual I/I is calculated at 12,000 gpd. This average value is reported on summary of current flows and loadings to the Chatham WPCF (Table 5-8).

As part of the Sewer System Evaluation Survey (M&E, 1992), a cost effectiveness analysis was performed to compare the cost of repairs to the cost of treating the I/I at the WPCF. Based on this report, in most cases it was determined not to be cost effective to remove the I/I. The report recommended that periodic monitoring of early morning flow rates at the WPCF be performed to identify the occurrence of high infiltration rates. Manholes that were identified as in need of repair should be renovated to help reduce any current and future infiltration. Lastly, it was recommended that the Town initiate a program to remove private sump pumps from the sewer system (M&E, 1992) to reduce inflow.

E. Exfiltration from Wastewater Collection Systems. The WPCF staff had expressed concerns about exfiltration occurring in the force main between the Stage Harbor pump station and the WPCF. The operational staff investigated the leak, which appeared to occur between pumpings from the Stage Harbor pumping station. The water level at the outlet to the WPCF was observed to drop following the completion of the pumping cycle. The staff replaced a check valve at the pump station and the water level has remained constant at the outlet following this repair. The operational staff do not feel that there is any exfiltration from the force main.

Exfiltration is defined by the National Association of Sewer Service Companies as "The leakage or discharge of flows being carried by sewers out into the ground through leaks in pipes, joints, manholes, or other sewer system structures; the reverse of infiltration." While much attention has been given to infiltration and inflow in literature written by various parties, little information exists on the location and measurement of exfiltration.

Water normally flows along the path of least resistance. Therefore, in order for significant exfiltration to occur, two conditions must exist concurrently. There must be a defect in the sewer line which can allow sewage to escape into the surrounding soil (open joints, broken pipe), and the sewage must be under enough pressure so that it is easier for the sewage to pass through the defect, passing into the soil and having a flow path away from the defect, rather than continuing down the sewer line. This pressure could come from either high flow conditions within the pipe, or surcharged conditions created from a sewer blockage.

No single method is widely used to locate and quantify exfiltration rates, although it could potentially be measured in several ways. One method would be flow metering at upstream and downstream manholes of individual sewer sections. If the downstream reading is less than the upstream, the net difference represents the exfiltration rate. However, obtaining consistent and accurate readings that would reflect these changes is difficult. Flow results could also be affected from flows entering the line from lateral connections.

If exfiltration does occur in a sewer section, the resulting flow should eventually reach the groundwater table. If so, any bacteria or nutrients in the wastewater could possibly be traced by groundwater sampling.

However, sewers subject to exfiltration would tend to act almost as a leaching field pipe from a septic system. Since these sewers could be above the groundwater level, a bio-mat could develop outside the pipe, forming a barrier that would filter out most of the bacteria prior to contact with the groundwater table. If the bio-mat becomes thick enough, it could also help to prevent the exfiltration from occurring in the first place. In addition, if any pollutants were discovered in the groundwater table, it would be very difficult to determine if they were caused by exfiltration from the sewer system, or by failing septic systems, if there are any in the area.

A third method, which could be used to measure exfiltration rates, is a hydrostatic test on an individual sewer segment. An entire sewer section between two manholes could be placed under a head of water, and the water level monitored over a set period. Any drop in the level represents a maximum potential exfiltration rate. It should be noted that this rate, if measured, would be representative of a surcharged sewer, not free-flow conditions.

A fourth method, which could be used to locate exfiltration, is enforced monographic pipeline leak detection. The Water and Sewer Department is considering this technology for water main leakage.

A final option to locate exfiltration sources would be to perform selected television inspection of sewer lines. This could be used to locate defects, which have the possibility of allowing sewage to leave the pipe.

None of the methods discussed above is inexpensive or easily performed. Of them, hydrostatic testing and television inspection appear to be the methods which would give the most reliable results. Before any inspection work is performed, sewer lines with potential for exfiltration must first be identified. These lines would consist of those sewers, which are, (1) above the groundwater table; and (2) flowing at high capacity conditions, or subject to blockages. Further study would be required to identify those sections of sewer lines below the groundwater table.

The remainders of the gravity sewers experience no surcharging, have limited blockages, and have plenty of capacity, thus would not be pressurized to induce exfiltration. The Town also has a limited number of force mains, the two largest of which, as discussed above, were evaluated and do not appear to suffer from exfiltration. Exfiltration from the Town's collection system does not appear to be a problem.

F. Comments and Recommendations on Operations and Maintenance. A well-operating sanitary sewage collection system depends upon adequate scheduled operation and maintenance of pumping stations, force mains, and gravity sewers; and proper equipment, spare parts, manuals, safety programs, and trained people available to perform routine and emergency repairs. With proper preventative maintenance and routine observations of all components, emergency responses are minimized, and downtime or overflows and backups are virtually eliminated.

The Town of Chatham has established such a system. The pumping stations and sanitary sewers are checked, inspected and maintained on a routine basis, and gravity sewers are flushed and maintained so as to help prevent problems.

The WPCF staff jet wash the entire collection system two times per year. A pumper truck is brought in, and any grit collected in the manholes is removed. Sodium hydroxide is often added during the jet washing to dissolve grease and other materials.

Grease has been identified as the collection system's biggest problem. It tends to foul the gravity sewer pipes and pumping stations making maintenance of these systems more difficult. Grease trap inspections are performed monthly and a service charge of $25.00 is charged to the proprietor of the establishment. Grease also causes problems in the air release valves in the system. There are approximately 18 of these valves, and each valve is maintained annually.

Since 1996, the collection system has experienced only two blockages. The blockages were a result of large amounts of root material, and were repaired by excavating the pipe sections and resetting the pipe. Only minimal root problems have been reported by the operational staff. The collection system has also not had any pump station overflows, except for a small overflow at the Mill Pond Pumping Station during construction.

The collection system has a cross-over valve connecting the six inch and eight inch force mains, which originate from the Stage Harbor Pumping Station. This valve is exercised once a year, but appears to be leaking because the force mains can not be isolated. This valve should be replaced.

G. Collection System O&M Equipment. Major equipment used for collection system maintenance and operation included the following equipment.

- 25 kW portable generator sets for Mill Pond.
- 9 kW portable generator sets for Chatham Housing Authority Pumping Station.
- Three-inch portable trash pump, which is used to connect to Queen Anne Pumping Station to pump into the force main.
- One sewer jetter which has 300-gallon capacity at 40 gallons per minute, which is trailer mounted. The Town of Chatham plans to buy a new sewer jetter with a 700-gallon capacity.

H. O&M and Planned Capital Improvement Program Costs. Operation and Maintenance costs for the sewer department (including the Chatham WPCF) are summarized on Table 5-10. Planned improvements to the collection system and presented on Table 5-11, which lists the Sewer Department's, Capital Improvement Plan for Fiscal Year 1998 – 2000.

I. Summary. Overall, the existing wastewater collection system is operating well and is well maintained. The preceding summary of the existing facilities will serve as a reference and provide a basis for evaluation of collection system alternatives in future phases of this project. Current problems have been described and discussed so that they can be addressed earlier individually or as part of a future plant upgrade.

5.3 EXISTING ON-SITE SYSTEMS

A. Description of Systems. Although centralized wastewater treatment technologies will have to be considered in the comprehensive wastewater management planning process, it is likely that many of the existing and anticipated future wastewater management problems can be handled through the use of on-site systems. On-site systems are used to treat wastewater from individual residential or commercial lots, and these systems, defined in Section 3.2B as Title 5 systems, may take advantage of a combination of innovative and alternative technologies. Wastewater flows less than 10,000 gpd (previously 15,000 gpd) are regulated by Title 5, while flows greater than 10,000 gpd require a State issued groundwater discharge permit. With these factors in mind, the objective of this section is to summarize available information on the on-site systems in the Town of Chatham.

The state, regional, and local regulations governing the use of these systems were discussed in Chapter 3. The key local regulations for on-site systems are listed and briefly identified below. More detailed descriptions are located in Chapter 3. (The designation BOH indicates that the regulation was enacted by the Chatham Board of Health.)

- BOH 4-88: All proposed wastewater flows (subdivision comprised of single family houses are exempt) greater than 2,000 gpd must receive a Town Sewage Discharge Permit, and demonstrate that the discharge will not cause the groundwater to exceed specified limits of nitrogen and phosphorus.

- BOH 89-2: Wastewater discharges from new development must be limited to groundwater nitrogen loading less than 10 mg/l. This regulation is currently (April 1999) being revised to apply to all properties in Town and to decrease the loading limit to 5 mg/l.

- BOH 91-1: This regulation provides standards on design, operation, and maintenance of small wastewater treatment plants.

- BOH Advisory Letter #20, May 1992: All cesspools must be upgraded to systems allowed by Title 5 regulations at the time of property transfer.

- BOH 95-1: This regulation provides monitoring requirements of alternative septic systems.

There are several types of on-site systems in Chatham including Title 5 systems, cesspools, tight tanks, commercial systems, and alternative systems.

Title 5 systems receive their name because they were designed based on the Title 5 regulations described in Chapter 3. They are composed of three main elements: septic tank, distribution box, and soil absorption system. Septic tanks remove floatable and settable solids from the waste stream, and can act as an anaerobic digester to digest (remove) solids, as well as a flow equalization tank. The tank is usually constructed of concrete and consists of baffled chambers, or it has inlet and outlet tees designed to isolate the solids in the tank and eliminate short circuiting of floatables. The distribution box receives the effluent from the septic tank and distributes it evenly throughout the leaching system. The distribution box is typically a small watertight concrete structure with one inlet and several outlets. The soil absorption system is used to further treat the septic tank effluent while infiltrating the treated effluent into the ground. Soil absorption systems come in many forms including leaching trenches, leaching pits, leaching galleries, and leaching fields. The selection of a particular type of soil absorption system for a particular design will depend upon the specific site considerations and costs.

Cesspools are tanks with open joints or holes in the walls and bottom through which the wastewater percolates into the ground. Solids collect in the bottom of the tank where they decompose or can be removed as septage. They are considered a substandard septic system in Chatham, and must be upgraded to a system allowed by Title 5 regulations at the time of property transfer.

Tight tanks are non-discharge systems that collect and store the wastewater until it can be removed. All the wastewater goes directly into the tight tank. The tank has a level indicator with an alarm, and a signal is transmitted when the liquid level reaches a certain height. When the tank is full, a septage hauler empties the tank and transports the contents to a treatment facility. There are two tight tank installations in Chatham. Tight tanks are usually approved by DEP as an interim measure to meet a health risk.

Communal systems are Title 5 systems that treat and dispose wastewater from more than one property. They can use common septic tanks, as well as common soil absorption systems. There are two communal systems in Chatham.

Alternative systems are systems that use advanced technology to provide a higher level of treatment than regular Title 5 systems. The Title 5 regulations allow a smaller soil absorption system when they are used. They can be used to reduce the Biochemical Oxygen Demand (BOD) and nitrogen in the septic tank effluent. There are six alternative systems in Chatham: two Bioclere systems, and four FAST systems. These systems often have more monitoring requirements than a regular Title 5 system. An additional alternative system is planned for a retirement home that may be developed along Route 28. The system will be an Amphidrome system designed for additional nitrogen and BOD removal. If constructed, it will operate under a Town Sewage Discharge Permit in accordance with BOH 4-88 because it will be designed for a wastewater flow greater than 2,000 gpd. Alternative systems are usually requested and approved when a property owner has minimal space for a soil absorption system or when the property is located in a nitrogen sensitive area.

B. Failed Systems. Several septic systems in Chatham have failed inspections at the time of property transfer or have been pumped four or more times in a year. Between 1994 and 1997, 26 properties were identified as being pumped four or more times in one year, three of which have been identified twice in this four year period. Between 1995 and 1997, 41 properties were identified as requiring upgrades as a result of inspection for impending Real Estate transactions. Failed systems are typically upgraded to Title 5 systems. The two tight tank installations are the only systems that have not been able to up grade to Title 5 systems.

C. Properties with Large Wastewater Discharge. Properties with design flows greater than 15,000 gallons per day are required to obtain a groundwater discharge permit or connect to a sewer. (This limit has been reduced to 10,000 gpd for new installations.) Properties with design flows between 10,000 and 15,000 may be required to obtain a State groundwater discharge permit or connect to a sewer. The Chatham Bars Inn (CBI) has a high Title 5 design flow, and has agreed to construct a wastewater treatment plant to allow their treated wastewater to be discharged into the ground. CBI has received a groundwater discharge permit from DEP for a discharge of 35,000 gpd.

D. Properties with Minimal Land for Title 5 Systems. Two properties in Chatham have tight tanks; have experience failed systems, and do not have sufficient space for Title 5 systems. These systems have been allowed to protection public health from the failed systems.

Stearns & Wheler has used a computerized database connected to the Geographic Information System (GIS) to identify additional non-sewered properties that may not have sufficient space for a Title 5 system. The Title 5 design flow was calculated for each property based on the number of bedrooms, number of restaurant seats, and commercial floor space. The required land area (for a Title 5 system) was then calculated for each property based on Title 5 design flow and other engineering considerations. The available land area (for a Title 5 system) was calculated for each property by subtracting the area occupied by buildings from the total property area. If the required area is greater than the available area, the property was researched by the Health Department to identify if it could fit a Title 5 system. No additional properties have been identified as not being able to fit a Title 5 system.

E. Properties with High Groundwater Conditions. Several portions of Chatham have ground-surface elevations between zero and ten feet above MSL. These surface elevations in combination with 100 year flood zones and high groundwater conditions result in areas where septic systems must be elevated to provide sufficient separation between the top of the groundwater and the bottom of the soil absorption system.

Chatham's "Minimum Requirements for the Subsurface Disposal of Sanitary Sewage", December 1982 has the following regulation on groundwater separation and raised systems.

> Section 2.18 SEWAGE DISPOSAL SYSTEM IN FLOOD PLAIN ZONE
>
> Any sewage system located in a Flood Plain Zone, as delineated on the Town of Chatham Flood Insurance Rate Map, shall be so located that the bottom of the leaching portion of the system is at a minimum elevation of nine (9) feet above mean sea level (USGS Datum), and in no case shall a variance be granted to this regulation which would allow the elevation of the finished grades over the system to be less than one (1) foot above the flood elevation for that area. It is Board of Health policy that this section applies only to new construction and not for the replacement of septic systems for existing buildings.

Title 5 regulations require four feet of separation between the top of maximum high groundwater elevation and the bottom of the soil absorption system. This distance is increased to five feet when the soils have a percolation rate of less than two minutes per inch. The ten-foot surface contour is used as the perimeter of the low elevation area where a septic system would need to be raised to meet these regulations. The low elevation areas below the ten-foot contour are shown on Figure 4-3. The 100 year flood zones (A zones and V zones) are shown on Figure 4-5.

Discussions with the Town Health Agent indicate that there are several properties near Mill Pond that are at low elevation and have minimal area to site a Title 5 system. These properties are identified as the Eliphamets Lane Area of Concern on Figure 4-3. The

Health Department believes that this area has wastewater disposal problems, and should be evaluated for acceptable wastewater facilities.

F. Septic System Records. The Health Department has a computer file, which lists all of the septic system permits issued since the beginning of 1997. This database lists the plan date, the history of the septic system, whether there is a well on the property, percolation rate, and other pertinent information.

The Health Department has numerous files containing septic system permits and design plans for most of the properties in Town. In addition, there is a group of five Rolodex files summarizing much of the permit and design information for these properties.

If the Town Health Department organized septic system information on a computerized database, this would allow for easy referencing and updating, and linking it to the Town's new Geographic Information System (GIS) through map and parcel identification numbers. If undertaken, the following information should be summarized for the septic systems in Town:

- System type
- Design data
- Installation date
- Design flow
- Leaching system type
- Approval type
    - general use
    - piloting
    - remedial
- Approval type
    - groundwater separation reduction
    - size reduction
    - distance to well

- distance to water course

- Monitoring requirements
- Title 5 code type
  - precode
  - 78 code
  - 95 code

This information would be valuable for wastewater management planning, and it could have great value to the Town as the wastewater management plan is implemented. It will allow the Town to know the type of wastewater systems being used, and estimate the wastewater treatment performance of these systems. It is noted that the DEP has recently revised the SepTrac program (developed by the Buzzards Bay Project). They have produced a program called BOH 2000. It is currently in the testing phase and a beta version of the program is available from DEP.

5.4 WATER SUPPLY AND USAGE

The Town of Chatham utilizes four wellfields to supply water to public water supply users. Set atop the Monomoy Lens, the four wellfields are referred to as the South Chatham Wellfield (Well Nos. 1, 2, & 3), Indian Hill Well (Well No. 4), the Training Field Well (Well No. 5), and the Chatham Town Forest Wells (Wells No. 6 & 7). The Indian Hill Well has not been used since 1988, due to PCE contamination problems. These wells, the overall Zone II area, and the Wellhead Protection District are illustrated on Figure 4-4.

The water distribution system has recently been expanded and is serves approximately 90 percent of the developed properties. This chapter section provides an evaluation of the existing water consumption and water capacity in Chatham.

A. **Analysis of Annual Pumpage and Consumption.** Data describing daily (1997), monthly (1994-1997), and annual (1994-1997) volumes of water pumped from the four wellfields (pumpage) have been obtained and analyzed for this report. Water consumption data (water metered for individual water accounts) has also been obtained and analyzed for this report. Total annual pumpage and consumption from 1994 to 1997 is presented below and is depicted in Figure 5-8.

ANNUAL PUMPAGE AND CONSUMPTION (1) (Million Gallons)				
YEAR	**1994**	**1995**	**1996**	**1997**
Total Pumpage	348	353	343	368
Total Consumption[2]	296	301	252	307
Notes: 1. Data obtained from Town of Chatham Public Water Supply Annual Statistical Reports prepared for DEP 2. Excluding process flushing and fire fighting use.				

Pumpage has remained steady since 1994. The total pumpage for the 1997 period was 368 million gallons. Total water consumption in Chatham mirrors the well pumpage, which indicates that the unaccounted for water has remained constant during this period. Total consumption has remained relatively flat with only a slight decrease in 1996. The total water consumption for 1997 was 307 million gallons.

B. **Analysis of Monthly Pumpage.** Monthly pumpage was analyzed for 1994 through 1997 to investigate monthly trends. It is noted that the water meters at individual households are read on a quarterly or monthly basis (depending on the property); therefore, the pumpage records are used to investigate monthly and daily variations. As shown in Table 5-12 and Figure 5-9, monthly flows were consistent over the four year period. Flows measured in 1995 and 1997 were highest during the month of July. Flows in February, March, April, May, August, November, and December were relatively constant over the four year period. Peak monthly flows ranged from 60 million gallons in 1994 to 70 million gallons in 1997. Peak pumpage rates for 1995 through 1997 occurred

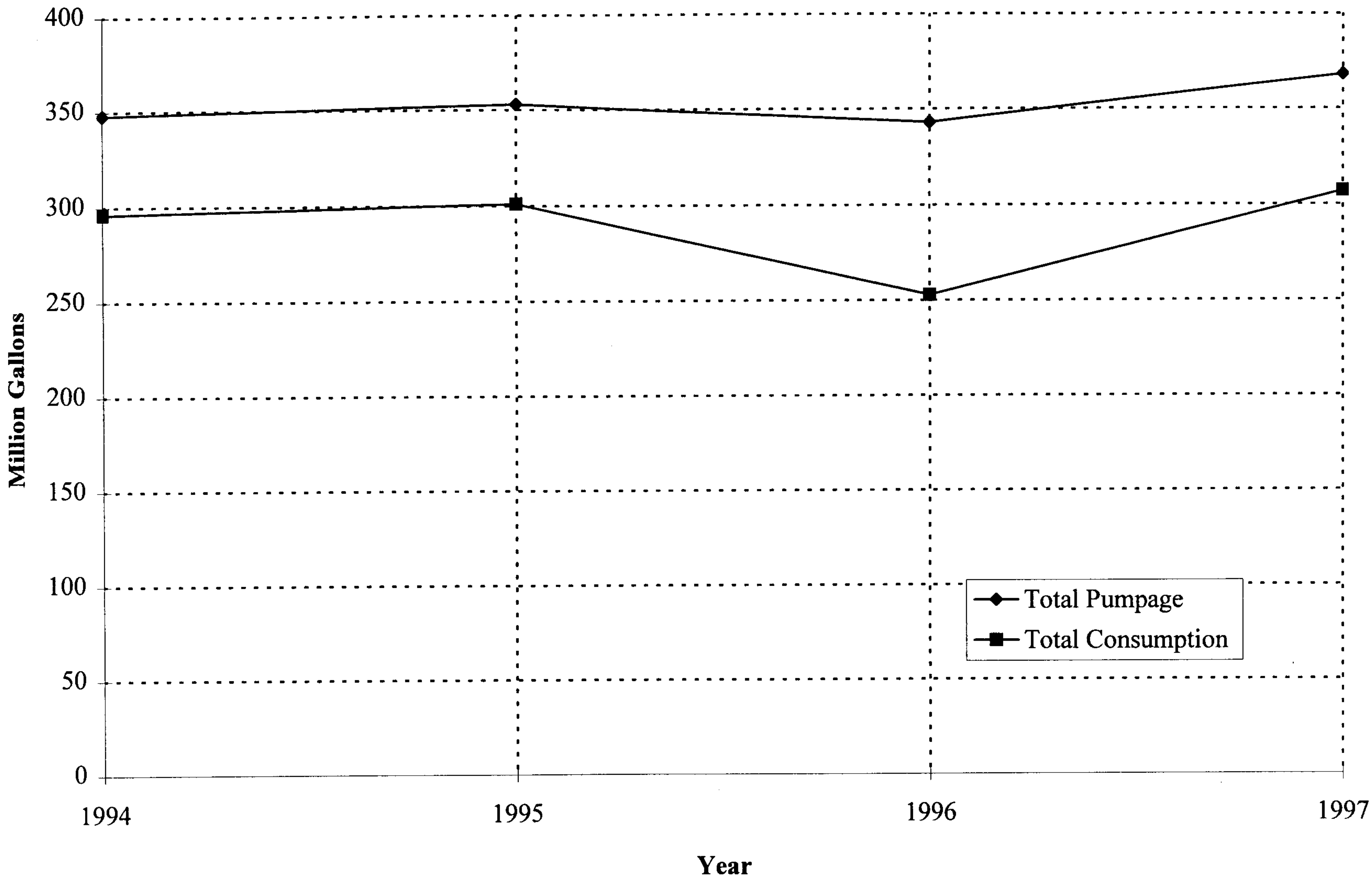


FIGURE 5-8 TOTAL ANNUAL PUMPAGE & CONSUMPTION, 1994-1997
ComprehensiveWastewater Management Planning Study Town of Chatham, MA
Stearns & Wheler, LLC ENVIRONMENTAL ENGINEERS & SCIENTISTS

Watrpump Fig.(Annual)
4/16/99

TABLE 5-12

TOTAL MONTHLY PUMPAGE, ANNUALLY
ComprehensiveWastewater Management Planning Study
Town of Chatham, MA

Total Montly Pumpage (Million Gallons Per Month) [1]				
Month	1994	1995	1996	1997
Jan	25.2	17.2	17.6	18.2
Feb	18.9	15.2	16.6	13.9
Mar	15.1	14.2	17.0	17.2
Apr	16.1	16.6	18.1	18.6
May	26.3	28.0	28.0	26.1
Jun	35.8	32.1	45.6	43.0
Jul	57.3	67.6	55.4	70.6
Aug	59.7	55.2	54.3	56.6
Sep	29.6	43.2	30.6	33.5
Oct	22.7	29.1	23.1	29.1
Nov	21.9	17.7	18.8	22.1
Dec	19.2	16.5	17.9	19.1

Notes:
1. Source - Town of Chatham Public Water Supply Annual Statistical Reports prepared for MADEP

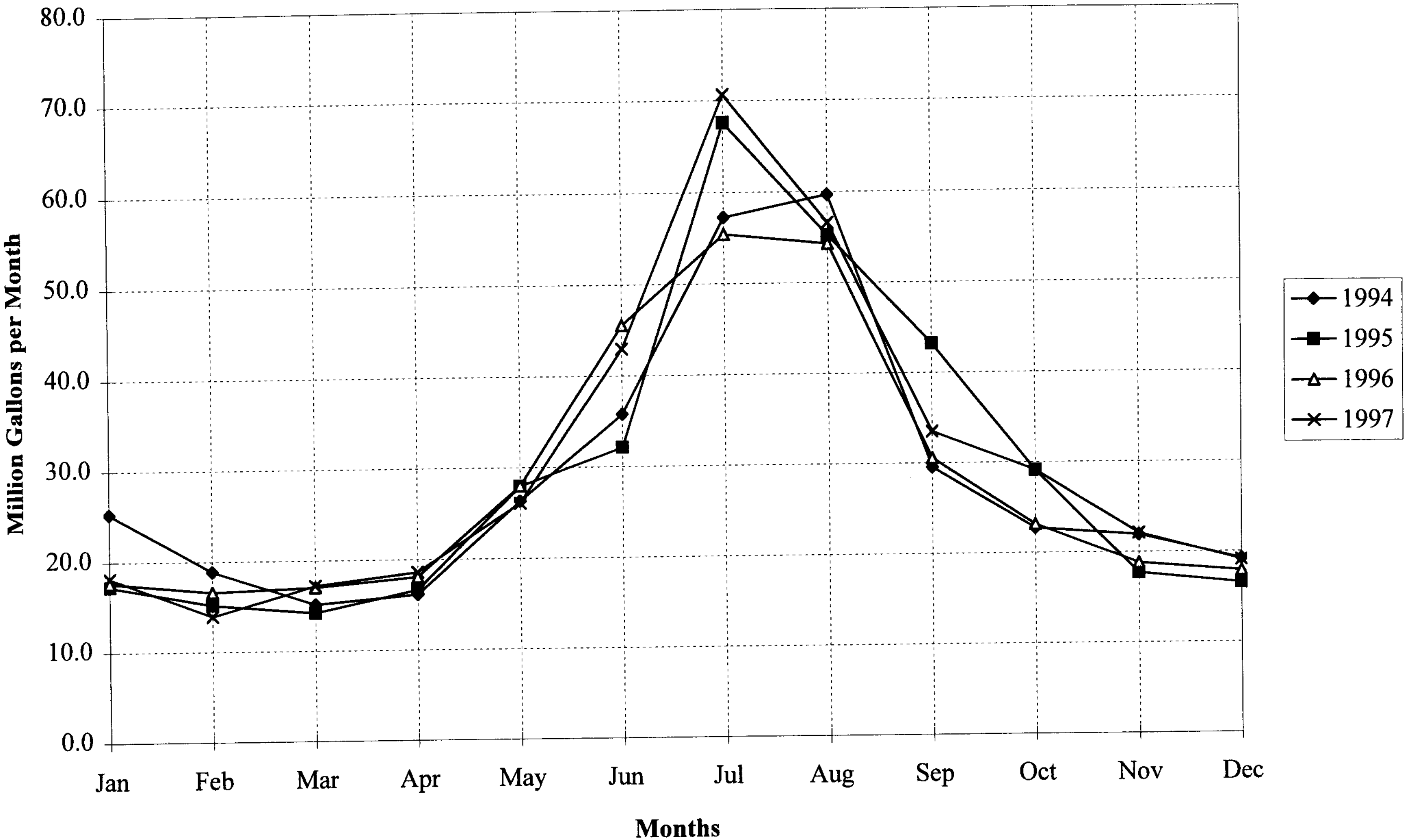


FIGURE 5-9 TOTAL MONTHLY PUMPAGE
Comprehensive Wastewater Management Planning Study Town of Chatham, MA
Stearns & Wheler, LLC ENVIRONMENTAL ENGINEERS & SCIENTISTS

4/16/99
WatrpumpFig (Month)

in July, except 1994's peak pumpage, which occurred in August. Minimum monthly flows occurred in February and March and ranged from 14 million gallons to 16.6 million gallons.

C. **Analysis of Daily Pumpage.** Daily pumpage data was reviewed for 1997, and is presented in Table 5-13. The daily pumpage ranges from 0.45 million gallons per day in February to 2.64 million gallons per day in August. The daily pumpage fluctuates during the week for the months of June, July, August, September, October, and November as the population fluctuates with weekend visitors. The daily pumpage for the months of January, February, March, April, May and December are shown in Figure 5-10, and indicate relatively constant consumption.

D. **Analysis of Water Account Data.** All of the properties in Chatham served by public water supply have water meters and water accounts with the Chatham Water and Sewer Departments. In 1997, there were 5,660 water accounts in Chatham at 5,397 properties. There were 5,709 water meters as detailed below.

- One , 4-inch meter (high school)
- Four, 3-inch meters (commercial and industrial)
- 17, 2-inch meters (commercial and industrial)
- Ten, 1 ½ inch meters (commercial)
- 182, 1-inch meters (commercial)
- Three, 5/8 inch meters (commercial)
- 5,492, ½ inch meters (residential)

The metered water consumption data was analyzed for 1997 to estimate annual water consumption for properties in the Town of Chatham. The data indicates that 307 million gallons were metered in Chatham.

The total metered use of 307 million gallons is less than the 368 million gallons pumped from the wells during the corresponding period. The difference is attributed to hydrant

TABLE 5-13

AVERAGE DAILY PUMPAGE
Comprehensive Wastewater Management Planning Study
Town of Chatham, MA

1997 Average Daily Pumpage (Million Gallons per Day)							
Month	Sunday	Monday	Tuesday	Wednesday	Thursday	Friday	Saturday
Jan	0.63	0.60	0.58	0.61	0.58	0.54	0.57
Feb	0.52	0.50	0.50	0.47	0.45	0.49	0.52
Mar	0.56	0.56	0.54	0.56	0.53	0.51	0.61
Apr	0.69	0.66	0.56	0.62	0.59	0.54	0.69
May	0.92	0.84	0.81	0.83	0.79	0.82	0.89
Jun	1.60	1.43	1.35	1.33	1.29	1.28	1.69
Jul	2.64	2.34	2.14	2.13	2.20	2.16	2.43
Aug	1.99	1.83	1.78	1.60	1.71	1.82	1.98
Sep	1.31	1.27	1.11	1.13	1.15	1.16	1.38
Oct	1.09	0.90	0.91	0.88	0.98	0.83	1.02
Nov	0.97	0.62	0.64	0.67	0.71	0.72	0.76
Dec	0.69	0.64	0.62	0.56	0.64	0.54	0.63

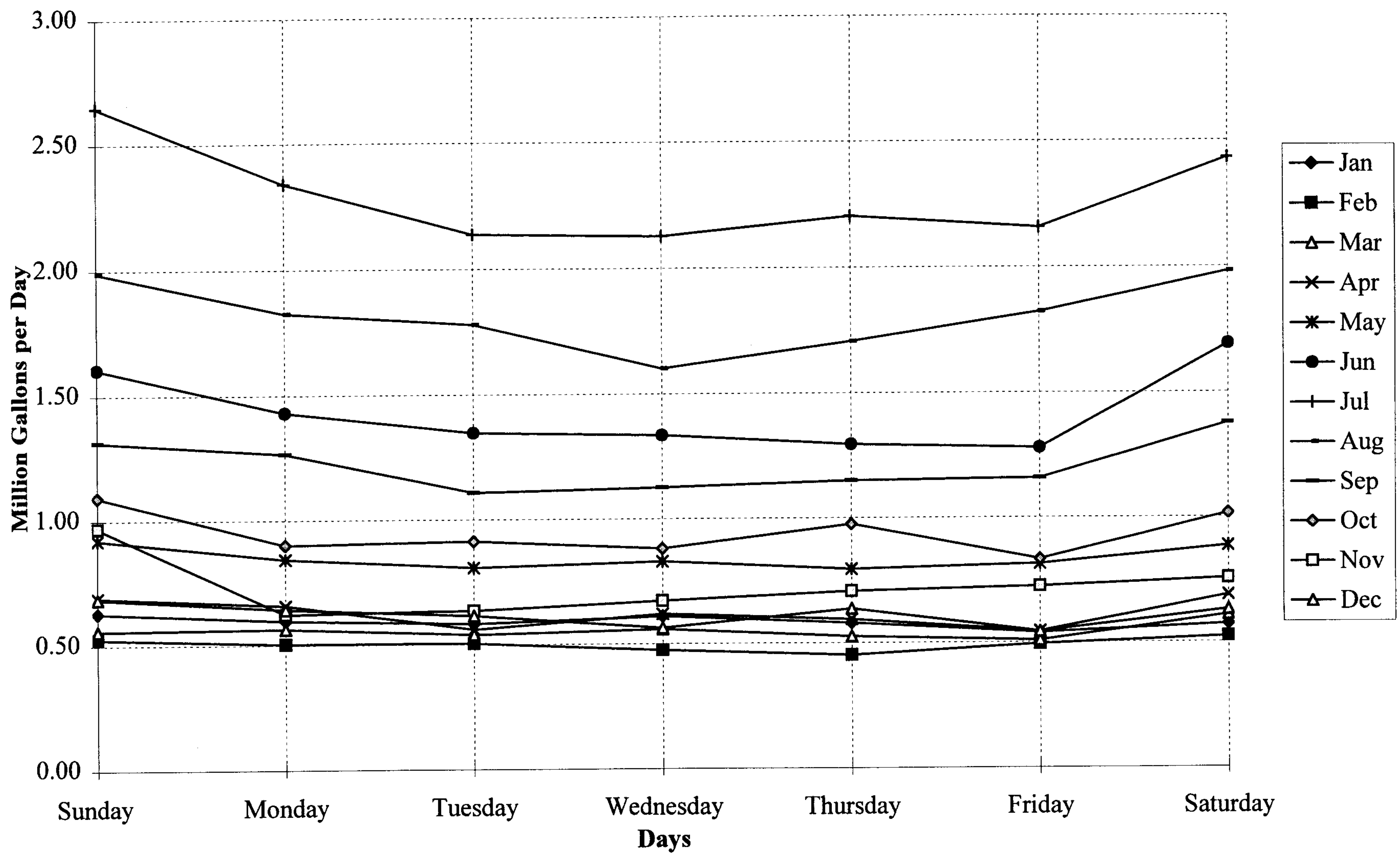


FIGURE 5-10
AVERAGE DAILY PUMPAGE (1997)

Comprehensive Wastewater Management Planning Study
Town of Chatham, MA

Stearns & Wheler, LLC
ENVIRONMENTAL ENGINEERS & SCIENTISTS

4/16/99
WatrpumpFig.(Ave. Daily)

flushing, fire flows, street cleaning, etc; and leakage in the distribution system. During 1997, the Public Water Supply Annual Statistical Report indicated that of this 368 million gallons 63.3 percent was billed residential, 16.5 percent commercial, 2 percent municipal, 1 percent industrial, 0.7 percent other PWS, 3.9 percent flushing, 10.9 percent unaccounted for, and 0.1 percent other. These percentages were developed by the Town based on service meter size and the type of land usage (residential, commercial, industrial, etc.) assumed for the various size services. It is noted that this land usage is different than the property land usage assigned by the Tax Assessor.

It is noted that system leaks are difficult to locate because of sandy soils in Chatham. The leaked water drains quickly to the water table and does not make itself evident as a wet spot on the land surface.

E. **Development of Water Flows for Properties Served by Public Water Supplies.** Water flows for the properties served by public water supplies were developed using the annual metered water consumption and the pumpage volumes discussed in previous sections. These flows were developed for the following seasonal periods, which characterize water consumption and resultant wastewater generation: average-annual, maximum-month, minimum-month and peak-day periods. The water flows developed for these periods all have the units of million gallons per day (mgd) and are summarized in Table 5-14.

The development of these water flows followed the following steps:

- The 1997 total pumpage volumes for these periods were selected from Tables 5-12 and 5-13.

- The average annual flow of total pumpage (1.01 mgd) was based on the 1997 water pumpage data. The Maximum month was derived from July 1997 data, minimum month from February 1997, and peak day was based on July 13, 1997.

TABLE 5-14

SUMMARY OF CURRENT WATER FLOWS TO
PROPERTIES SERVED BY PUBLIC WATER SUPPLIES
Comprehensive Wastewater Management Planning Study
Town of Chatham, MA

	Water Flows (mgd) (1)			
	Average Annual	Maximum Month	Minimum Month	Peak Day
Total Pumpage (2)	1.01	2.29 (3)	0.5 (4)	3.07 (5)
Unmetered (6)	0.17	0.17	0.17	0.17
Total Metered (7)	0.84	2.12	0.33	2.9
Residential (8)	0.67	1.69	0.26	2.31
- Single Family	0.56	1.41	0.22	1.93
- Multi Family	0.11	0.28	0.04	0.38
Commercial (8)	0.14	0.35	0.05	0.48
Industrial (8)	0.003	0.01	0.00	0.01
Institutional (8)	0.01	0.03	0.00	0.03

Notes:
1. Water flows are expressed as million gallons per day (mgd)
2. Flows based on 1997 water pumpage data
3. July 1997
4. February 1997
5. July 13, 1997
6. Based on Town of Chatham 1997 Public Water Supply Annual Statistical Report and includes Process usage, system flushing, unaccounted and other flows. This flow is estimated at being constant throughout the year due to leakage.
7. The Average Annual flow is the sum of all water meter readings. The flows for other averaging periods are obtained by subtracting the unmetered flow from the Total Pumpage.
8. The Average Annual flow is the sum of the water meter readings for the properties with this grouping of land use codes as assigned by the Tax Assessor. The flows for the other averaging periods are calculated based on the peaking factors observed for the Total metered flow.

- The unmetered flow rate is based on information provided in the Town of Chatham 1997 Public Water Supply Annual Statistical Report and includes process usage, system flushing, unaccounted and other flows. It is estimated at being constant throughout the year because a large portion of it appears to be due to leakage, which is constant throughout the year. This information was calculated in the Statistical Report by subtracting the total metered flow from the total pumpage.

- The average annual total metered flow was calculated by summing the total water consumption as indicated by all the water meters in Chatham. The total metered flow for the other averaging periods was calculated by subtracting the unmetered flow from the total pumpage. Total metered flow peaking factors were then calculated by dividing the flow for the averaging period by the average annual flow. The following metered flow Peaking Factors (PF) were identified.

  - Average Annual PF: 1.00
  - Maximum Month PF: 2.52
  - Minimum Month PF: 0.39
  - Peak Day PF: 3.45

- The Average Annual Flow was distributed to the listed land uses by matching individual water usage to individual properties and their assessed land use. This was accomplished using a Geographic Information System (GIS) database.

- The land use flows for the other averaging periods (maximum month, minimum month, and peak day) were calculated by multiplying the average annual flow for the land use by the associated Peaking Factor calculated in Step No. 4.

- Small adjustments were made to Table 5-14 to balance the values based on an understanding of the Town's demographics and water distribution system.

Average water flow rates for the land use categories were calculated by dividing the average annual water flow for a particular land use grouping by the total number of properties with that land use. These average water flow rates are listed below.

Land Use Category	Average Water Flow (gpd/property)
Residential	140
Single Family Residential	130
Multi-Family Residential	280
Commercial	530
Industrial	320
Institutional	340
All Properties	170

F. Development of Per Capita Water Consumption Values. Per capita water consumption was calculated based on the seasonal populations presented in Chapter 4; the metered flows analyzed in this chapter; and the estimate that 90 percent of the Town's developed properties are served by public water. The following table summarizes the development of per capita water values.

Data Type	Average Annual	Maximum Month	Minimum Month
Water Data (mgd)			
Total Pumpage	1.01	2.29	0.50
Total Metered	0.84	2.12	0.33
Unmetered Flow	0.17	0.17	0.17
Total Residential	0.67	1.69	0.26
Population Data			
Town Population(1)		25,000	6,000
90% Population	11,000 (2)	22,500	5,400
Total Per Capita Water Consumption (gal/cap/day)	92	102	92
Residential Per capita Water(3) Consumption, (gal/cap/day)	60	75	48

Note: 1. The Town year-round population in 1996 was 6,930 according to the US Census Bureau. The maximum month and minimum month populations were provided by the Chatham Chamber of Commerce.
2. Weighted average
3. Residential flow divided by population.

G. Development of Town-Wide Water Flows. Town-wide water flows (average annual) were developed by multiplying the average water consumption rates for the residential, commercial, and industrial properties by the total number of properties of those types in Chatham.

Town-wide water flows for the other averaging periods were calculated by multiplying the average annual flows by the metered flow peaking factors calculated in the preceding section.

The Town-wide water flows are summarized the following table.

EXISTING TOWN-WIDE WATER FLOWS (mgd) (1)				
	Average Annual	**Maximum Month**	**Minimum Month**	**Peak Day**
Residential	0.72	1.8	0.28	2.5
Commercial	0.19	0.48	0.07	0.66
Industrial	0.02	0.05	0.008	0.07
Institutional	0.01	0.03	0.01	0.04
Existing Non-Metered Flow (2)	0.17	0.17	0.17	0.17
Total	**1.11**	**2.53**	**0.54**	**3.44**

Notes: 1. These are estimated Town-wide flows based on 1997 water pumpage and consumption, and the total number of developed properties in Town. Flows are expressed as million gallons per day (mgd).

2. This is the existing non-metered flow that is pumped from the municipal supply wells but is not metered and consumed at individual properties. It includes process usage, system flushing, unaccounted and other flows.

H. Existing Water System Capacity. The water system capacity is based on the acceptable well yield and the capacity of pumps installed in the water supply wells. The following table summarizes the capacities of the Town's existing wells based on information in the Water Management Act Water Withdrawal Permits (W&H, 1990).

EXISTING WATER SYSTEM CAPACITY			
Well Name	**Well Number**	**Year in Service**	**Capacity (gpm)**
S. Chatham Well #1	01 G	1945	250
S. Chatham Well #2	02 G	1949	500
S. Chatham Well #3	03 G	1966	700
Indian Hill Well #4	04 G	1970	700
Training Field Well #5	05 G	1989	450
Town Forest Well #6	06 G	1992	700
Town Forest Well #7	07 G	1993	700
TOTAL:			**4,000**

This pumping capacity in combination with the storage capacity provided by two water storage tanks represents the capacity to meet a peak day's flow. The 4,000 gpm capacity equates to a daily capacity of 5.76 mgd.

The Indian Hill Well is currently not used due to PCE contamination. If the capacity of that well is removed, the system capacity becomes 4.75 mgd.

This system capacity exceeds the current water peak day demand of 3.07 mgd recorded for July 5, 1997.

I. Water Conservation.

1. Water Conservation Measures. The Town developed a Water Conservation Plan in November 1990 as part of the Water Management Act. This Water Conservation Plan has been revised and updated to July 1998, and is contained in Appendix K.

The Town expends approximately $625,000 per year on leak detection, system rehabilitation through capital improvement, and system rehabilitation through the operating budget. All water services are metered, and the Town conducts an annual water audit to account for all of the water pumped from the wells and the water delivered to the water system users. Leaks are repaired as soon as they are found. A five year capital program has been established to replace all small diameter water mains and services that are over 20 years old. Many of these old services are constructed of steel and galvanized steel which result in many leaks. System leakage is difficult to detect in Chatham due to the sandy soils, which allow the leaks to drain to the water table and not come to the ground surface where it is easily noticed. The September 23, 1998 memo on unaccounted water in the Chatham water system is included in Appendix L.

The Town uses an increasing block rate structure, which charges a higher rate for higher water usage. The Town also uses a seasonal pricing structure to charge a higher rate during the summer when the demand is highest. Summer rates are approximately 40 to 80 percent higher than winter rates depending on the size of the water service. The current water rate schedule is summarized below.

WATER RATE SCHEDULE		
Service Charge [1]	**Winter**	**Summer**
5/8” Meter (1,000 cf)	$21.00	$33.00
¾” Meter (1.700 cf)	$36.00	$57.00
1” Meter (2,300 cf)	$48.00	$83.00
1 ½” Meter (3,700 cf)	$81.00	$132.00
2” Meter (5,500 cf)	$124.00	$201.00
3” Meter (9,300 cf)	$218.00	$350.00
4” Meter (13,600 cf)	$324.00	$516.00
Metered Rates	**Winter**	**Summer**
1st Step (1,001-3,000 cf)	$2.00	$3.50
2nd Step (3,001-5,000 cf)	$2.35	$3.70
3rd Step (over 5,000 cf)	$2.50	$3.90
Notes: 1. Billed quarterly in areas and includes the minimum usage.		

Increases to this rate structure are currently being reviewed by the Town.

The Town Water and Sewer Department has several public education efforts to promote water conservation. Printed newsletters and brochures are sent to all water customers in the spring and fall with water conservation information. Users are notified that water conservation kits are available through the Barnstable County Water Utilities Association and Commonwealth Electric. The Water and Sewer Department continues to work with the school system to establish curriculum on water conservation, and has distributed water conservation pamphlets in the school, and has had a water conservation poster contest.

2. Evaluation of Water Conservation Practices. The Certificate of the Secretary of Environmental Affairs (contained in Appendix A) requested an analysis of the Town’s water conservation practices and the development of a preliminary water demand management and conservation plan. A Water Conservation Study prepared in May 1995 for the Town of Plymouth was used as an example of such a plan. The Plymouth Water Conservation Study identified the following as major components of a water conservation plan:

- public education programs,
- indoor residential water use management, and
- institutional and commercial water use management.

Water Conservation plans were classified by the Town of Plymouth as passive or aggressive depending on the level of community participation and Town action. An aggressive water conservation program takes all measures possible to reduce water demand while a passive program includes minimal activity by the Town and the community. Billing inserts, media campaigns, and school curriculum supplements as preformed by the Chatham conservation program are all cited as components of an aggressive public education program. Of the measures cited for implementing conservation in residential water use, distribution of water conservation devices by mail or at a local distribution point are considered least aggressive, while door-to-door delivery and direct installation by the local water department are considered most aggressive.

The Plymouth Study developed costs for various conservation scenarios and determined that a program in the middle of the passive-aggressive range would be most appropriate, and would include the following:

- public education with billing inserts and brochures,
- providing water conservation devices by mail to interested consumers, and
- financing programs through the local water and wastewater departments.

Based on the classification system established in the Plymouth Water Conservation Study and the current water conservation practices of the Town, Chatham's Water conservation

Plan is considered moderately aggressive. The Town's current plan includes a public education program and notifies users where to obtain water conservation kits. Public participation in water conservation could increase if the Town chose to distribute water conservation devices by mail, door-to-door delivery, or by having the Town Water and Sewer Department perform installations.

5.5 TOWN-WIDE WASTEWATER FLOWS AND LOADINGS

The Town-wide wastewater flows are calculated based on the total number of residential, commercial, industrial, and institutional properties in Town and the existing water usage estimated for these properties in Section 5.4.

The loadings are calculated based on the analysis of loadings received at the Chatham WPCF as estimated in Section 5.1.

These flows and loadings are summarized on Table 5-15. This table provides much detail based on estimates of seasonal variations, average flows for various land uses, and average pollutant concentrations. Review of the table indicates the following findings.

- Approximately 10 percent of the Town's wastewater flow (0.095 mgd on average) is treated at the Chatham WPCF. This wastewater is treated to a high level and the effluent is discharged to the groundwater system that flows south through the Cockle Cove Creek Watershed. The wastewater component in the effluent that is the main concern to the environment is nitrogen at an average concentration of 5.3 mg/l. Approximately 5 lb/day of nitrogen is being discharged at the Chatham WPCF based on an effluent nitrogen concentration of 5.3 mg/l and an average flow of 0.12 mgd.

- Approximately 90 percent of the Town's wastewater (0.82 mgd on average) are treated in individual septic systems at a lower level of treatment. The organic component of the wastewater (BOD and TSS) is treated to a level that allows the

TABLE 5-15

CURRENT TOWN-WIDE WASTEWATER FLOWS AND LOADINGS
Comprehensive Wastewater Management Planing Study
Town of Chatham, Massachusetts

Source	Average Annual	Minimum Month	Maximum Month
Residential			
Flow, gpd	650,000	250,000	1,620,000
BOD_5, lb/day	1,360	420	4,060
TSS, lb/day	1,080	370	3,380
TN lb/day	220	50	720
Commercial			
Flow, gpd	170,000	60,000	430,000
BOD_5, lb/day	350	100	1,080
TSS, lb/day	280	90	990
TN, lb/day	60	10	180
Industrial			
Flow, gpd	18,000	7,000	45,000
BOD_5, lb/day	40	10	110
TSS, lb/day	30	10	100
TN, lb/day	10	0	30
Institutional			
Flow, gpd	9,000	4,000	27,000
BOD_5, lb/day	20	10	70
TSS, lb/day	20	10	60
TN, lb/day	<5	<5	10
Total Town			
Flow, gpd	847,000	321,000	2,122,000
BOD_5, lb/day	1,770	540	5,310
TSS, lb/day	1,410	470	4,710
TN, lb/day	280	70	930

water to flow to the groundwater system without plugging the soil absorption system. A small portion of the nitrogen is typically removed in a septic system but most is passed to the groundwater system at a typical concentration of 35 to 40 mg/l (USEP Design Manual, On-Site Wastewater Treatment and Disposal Systems). The effluent from a cesspool would have a higher nitrogen concentration. Approximately 240 lb/day of nitrogen is being discharged through individual on-site systems based on an effluent nitrogen concentration of 35 mg/l and an average flow of 0.82 mgd.

5.6 FLOW AND LOADING REDUCTION OPPORTUNITIES

A. Introduction. The purpose of this section is to identify and review alternatives for reducing wastewater flows (water conservation) and pollutant loadings. The MEPA Certificate (attached in Appendix A) requested that measures that have the potential to reduce wastewater volume be identified and discussed in the Phase I report. Potential wastewater loading reduction opportunities are also identified. Water conservation is important because it reduces groundwater withdrawal and wastewater flows, and could potentially reduce the size of wastewater treatment facilities. The reduction of pollutant loadings also could potentially reduce the size of wastewater treatment facilities because there would be less waste in the water to treat. The following methods could be used by the Town to promote water conservation and reduce pollutant loadings:

- modification of current water pricing policies,
- use of low flow fixtures,
- use of waterless toilets (composting and incinerating),
- reuse and recycling of wastewater, and
- prohibited use of kitchen garbage grinders.

B. Pricing Policies. The preceding report section (5.4,H) presented the Town's water rate structure. This is an increasing block rate structure which tends to increase conservation because the customer is charged a higher rate the more water that the customer uses. The customer is also charged a higher rate in the summer when the water demand is the greatest and the wastewater flows are the greatest. The wastewater pricing is structured on a similar basis with increasing block rates and higher rates (100 percent higher) in the summer.

This pricing could be increased as a further economic incentive to reduce water consumption and wastewater generation. This rate increase could have an impact on retired people with low fixed incomes; therefore, an abatement program could be considered with a potential increase.

The Town is currently reviewing options to raise water billing rates to cover costs for operations and recent expansion of the distribution system, and to promote further conservation. Four options were reviewed at a March 1998 Water Rate Hearing. These options represented an approximate 34 percent increase and (as of August 1998) are still being evaluated.

C. Low Flow Fixtures. Water consumption and wastewater flow can be reduced through the use of household water saving devices. Approximately 70 percent of the total residential wastewater volume is generated by toilet, laundry, and bath use. Use of low flow fixtures in these areas can reduce water consumption (and subsequent wastewater generation) by 15 to 20 percent.

Commonly used low flow fixtures include low flow showerheads, toilet dams, faucet aerators, reduced flush toilets, vacuum flush toilets, flow limiting valves, and pressure reducing valves. Current State plumbing codes encourage and require the use of low flow fixtures in new residential and commercial construction. Plumbing codes also require the use of flow control devices for hot water showerheads and public lavatories. The use of low flow fixtures should be further encouraged in Chatham.

D. Waterless Toilets. Water consumption, wastewater flow, and pollutant loadings can be reduced through the use of waterless toilets. Waterless toilet systems operate by separating black wastewater and gray wastewater. Black wastewater is toilet waste and gray wastewater is generated from non-sanitary sources such as washing clothes and dishes, and bathtub and shower use. Black wastewater is treated in the waterless toilet unit, and gray wastewater is discharged to a septic system with potential size reductions.

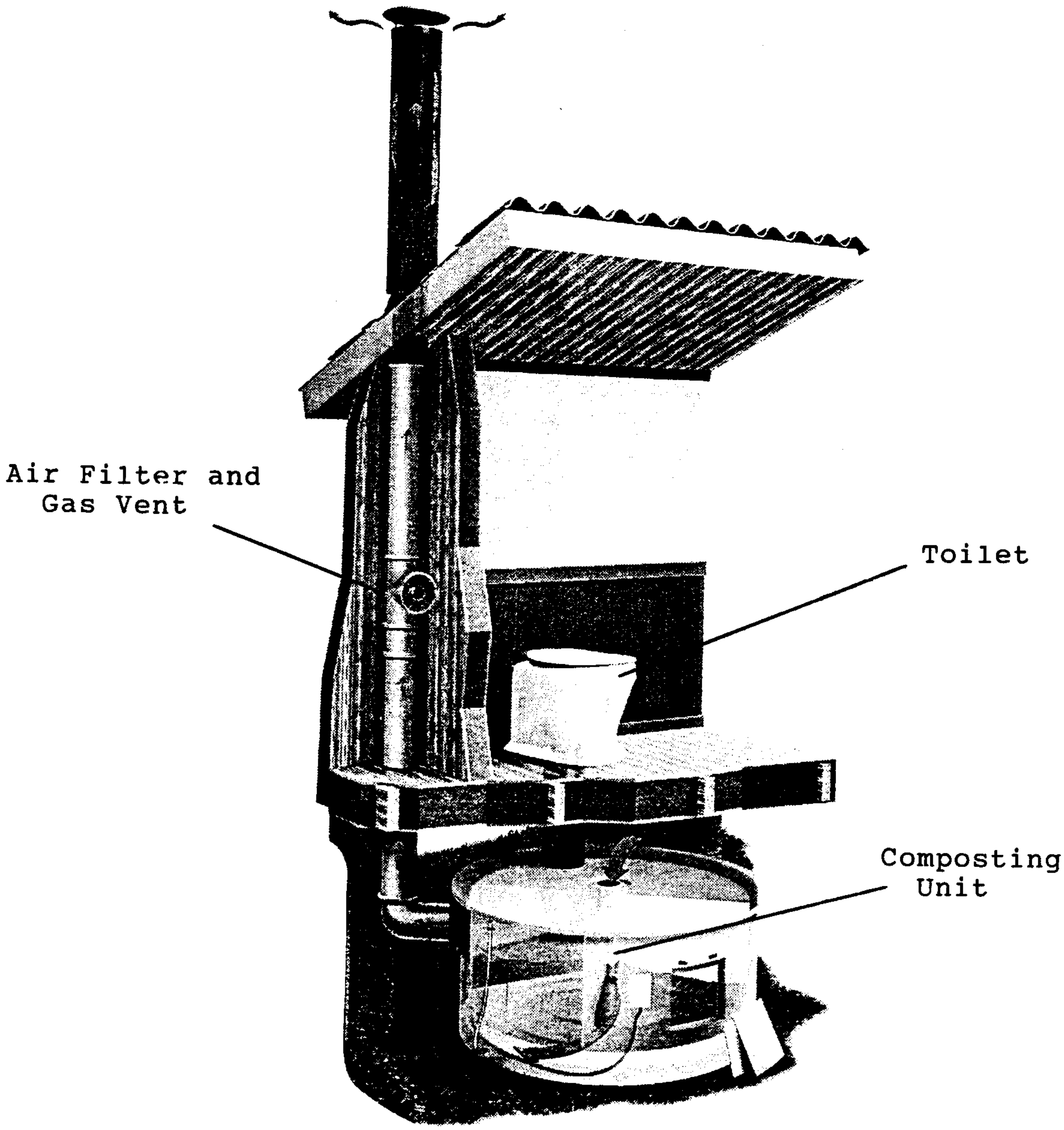
The two most common waterless toilet systems are composting toilets and incinerating toilets.

Composting toilets recirculate the black wastewater over accumulated solids to promote a natural decomposition process. Incinerating toilets burn black wastewater and generate a small quantity of ash and gas. Composted material and ash are periodically removed from the respective systems, and air filters and exhaust units are used to minimize odors. Public acceptance of waterless toilet systems is often low due to the composting, incinerating, and handling of human waste within living spaces. A potential use of waterless toilets is in public restrooms and convenience stations. This option eliminates the need for individual users to handle human waste, and would remove the composting process, odors, and the incinerating process from residential areas. Diagrams of composting and incinerating toilets are included as Figures 5-11 and 5-12, respectively.

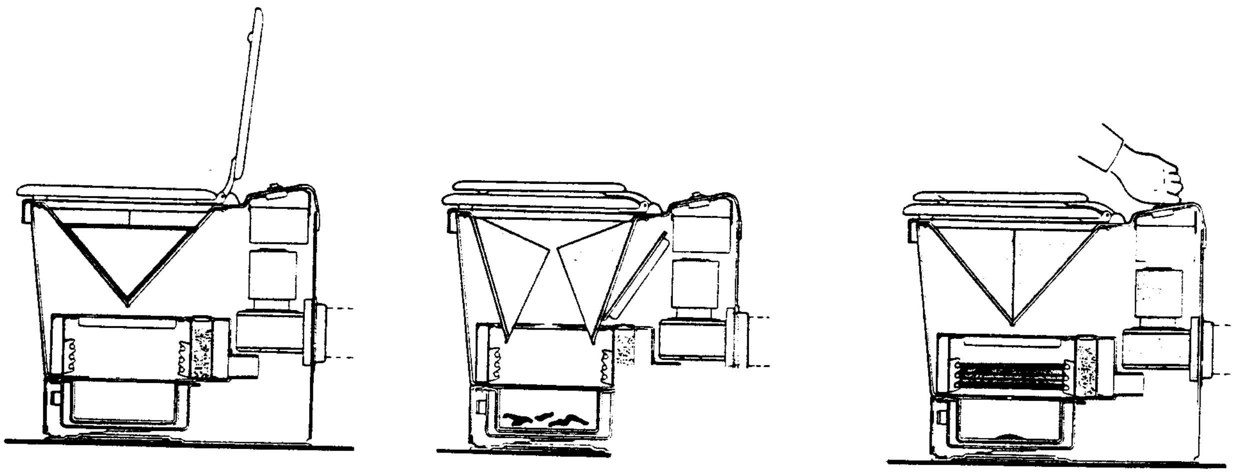
Waterless toilets have the following advantages.

- Wastewater flows and loads are reduced if properly designed and installed.
- Water consumption is significantly reduced.
- Minimal environmental concerns occur when properly sited and designed.
- Composting toilets require minimal energy use.
- Size of standard septic system can be reduced to treat only gray wastewater.
- Routine maintenance is minimal and requires no special training.
- Nitrogen loading to the environment is greatly reduced.

Waterless toilets have the following disadvantages:



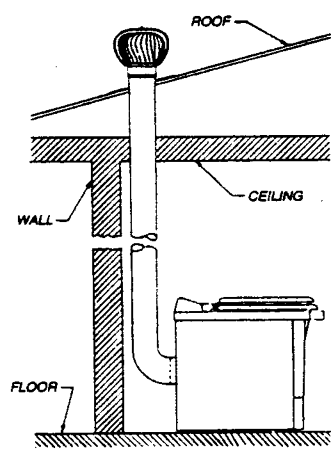
Source: Ecos-Water Conservation Systems, Inc.	FIGURE 5-11 COMPOSTING TOILET DIAGRAM
	Comprehensive Wastewater Management Planning Study Town of Chatham, MA
	Stearns & Wheler, LLC ENVIRONMENTAL ENGINEERS AND SCIENTISTS



1. Drop bowl liner into toilet bowl. Bowl liner catches and contains all waste plus paper.

2. Flush bowl by stepping on foot pedal.

3. Push start button to incinerate waste automatically.



Source: INCINOLET, 1/96	FIGURE 5-12 INCINERATING TOILET DIAGRAM
	Comprehensive Wastewater Management Planning Study Town of Chatham, MA
	Stearns & Wheler, LLC ENVIRONMENTAL ENGINEERS AND SCIENTISTS

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- Public acceptance is generally low.
- Some incinerating toilets require high-energy use.
- Handling of composting toilet contents can be objectionable.
- Incineration units are likely to generate odors if not vented properly.
- Composting toilets are not well suited to high seasonal peak loading.

E. Wastewater Reuse and Recycling. The identification of wastewater reuse opportunities is important because wastewater reuse can reduce both water consumption and the overall volume of wastewater that must be disposed. Wastewater sources that could be reused include gray wastewater from individual homes and treated wastewater effluent from the Chatham WPCF or from a new wastewater treatment facility.

The following wastewater reuse methods are considered:

- watering lawns with household gray wastewater,
- using treated effluent from wastewater treatment facility as industrial boiler makeup water or process water, and
- irrigating golf courses and Town-owned property with treated effluent from a wastewater treatment facility.

The use of reclaimed water must meet interim guidelines developed by DEP (Draft 6 – October 1998) in addition to the requirements of the Groundwater Discharge Permitting Program.

Reuse of household gray wastewater for lawn watering would be expensive to individual homeowners because it requires the construction of a separate gray wastewater collection, storage, and pumping system. Reuse of treated wastewater effluent from the Chatham WPCF by an industry, golf course, or municipal property would be expensive, difficult to administer, and would require additional wastewater treatment due to the health risks associated with potential human contact. In addition to monetary and health concerns, there are limited numbers of industries with a consistent year-round demand.

Chatham does have a golf course and municipal properties that could potentially utilize irrigation with treated effluent. This type of irrigation would have a high cost for piping and protection against freezing, and it could only be used periodically.

F. Prohibition of Kitchen Garbage Grinders. Kitchen garbage grinders grind food scraps and send them down the drain to collect, and be treated in a septic tank or to be treated at a wastewater treatment plant. They are convenient because they reduce the generation of food wastes that are often wet and messy. They increase the organic and nitrogen loading on wastewater treatment systems.

Septic systems are typically designed with a larger capacity when a kitchen grinder is used, and must be pumped out more frequently. A wastewater treatment plant needs to process a higher loading when kitchen grinders are used. Therefore, this use adds capital and operation costs to wastewater treatment processes. These grinders could be prohibited in a Board of Health regulation or bylaw.

G. Installation of Non-Potable Private Wells for Lawn Watering. Lawn watering in the summer uses a large quantity of potable water just to make the grass green. This water could be drawn from non-potable wells at the property, and distributed to an irrigation system through a pump and pressure tank. These types of systems should be feasible in Chatham, and would tend to pay for themselves over a short time period.

Large water users that irrigate their lawns with the public water supply could be identified by the Water Department by reviewing seasonal water billing records and comparing them to Title 5 design flows.

This type of non-potable water supply for lawn irrigation would reduce revenues at the Water Department. Also, this water use would be outside the Town's control, and may impact the groundwater level or quantity on which the public water supply depends.

5.7 STORMWATER DISCHARGES AND IMPACTS

The State, Town, and private roads in Chatham collect storm runoff (stormwater) which is then discharged to surface water bodies. The stormwater often contains dirt, fecal material from domestic and wild animals, nitrogen compounds, and phosphorus compounds. The fecal material and its associated coliform bacterial content can force shellfish closures in Chatham’s coastal waters. Nitrogen and phosphorus can fertilize surface water bodies, and promote the production of algae and other aquatic plants. These aquatic plants can further impact surface water quality when they die and settle to the bottom.

This section briefly identifies the main stormwater discharge locations, their impacts, and ongoing efforts to modify or remediate the discharge. Information for this section was obtained from Robert A. Duncanson, Ph.D., Chatham Water Quality Laboratory Director.

This information is presented to provide complete discussion of the major pollutant sources to surface water bodies. Evaluation and recommendation of solutions to mitigate stormwater impacts is beyond the scope of this Study.

A. Oyster Pond Furlong Discharge. This discharge is a pipe outfall which receives stormwater from the Five Corners area: portions of Crowell Road, Queen Anne Road, Main Street, Depot Road, drainage from a wetland system located to the West of Oyster Pond Furlong Road which has been ditched for mosquito control, and portions of Route 28. This discharge flows continually due to the flow from the ditched wetland.

Remediation is planned in the future as part of the Massachusetts Highway Department (MHD) Route 28 Highway Reconstruction Project. This discharge has a significant impact, and is believed to be one of the reasons why portions of Oyster Pond are permanently closed to shellfishing.

B. Stage Harbor Road Discharge. This discharge is a piped outfall which receives stormwater from approximately 1,200 feet of Route 28/Old Harbor Road, portions of Main Street, the Town parking lot, and overflow from Dugans Pond. In the past, this discharge received flow from the Main Street School area, which was recently remediated with a stormwater treatment system installed at the school.
The Main Street School remediation project removed approximately 20 percent of the total Stage Harbor Road discharge. The remediation project utilizes a 2,000-gallon oil/water separator to pretreat the stormwater before it is infiltrated into the ground. The project made use of a grant from the Massachusetts Coastal Zone Management (CZM).

This stormwater discharge generally flows all of the time due to overflow from Dugans Pond.

Further remediation of this stormwater discharge is planned as part of the Main Street Highway Reconstruction Project. This discharge has significant impact on the Oyster Pond water quality and is believed to be one of the reasons why portions of Oyster Pond are permanently closed to shellfishing.

C. Pond Street Parking Lot Discharge. This discharge is from the Pond Street parking lot and receives some flow from Queen Anne Road down to the parking lot. The discharge is directly from the parking lot and from two small headwalls located along Queen Anne Road. The discharge is relatively small.

D. Piped discharged located North of Emery Field Road. This discharge which receives overflow from a small pond located South of Cross Street. The discharge also receives flow from catch basins along Stage Harbor Road and discharges into Oyster Pond North of Emery Field Road. It generally flows all of the time due to the overflow of the pond.

E. Barn Hill Landing Discharge. This discharge is generated by runoff flow approximately 1,200 feet of Barn Hill Road and discharges it down the boat ramp into Oyster Pond River. It has minimal impact.

F. Mitchell River Bridge. This discharge receives stormwater flow from approximately 1,500 feet of Bridge Street and discharges at the bridge abutments. It has minimal impact on Mitchell River.

G. Mill Pond Town Landing Discharge. Flow from this discharge originates from Mill Pond Road and Homestead Lane and empties into Little Mill Pond at the Town landing. This discharge has significant impact on Little Mill Pond and funding to remediate it is planned in the FY 1999 Capital Budget. Remediation of this discharge will be challenging because there is minimal space to site infiltration facilities.

H. Chatham Harbor Fish Pier Discharge. This discharge receives stormwater from the Fish Pier and Shore Road areas. It has minimal impact due to the high flushing of the Chatham Harbor area at this location.

I. Cow Yard Landing Discharge. This discharge receives flow from the Route 28/Shore Road intersection, as well as portions of Old Harbor Road. The discharge is through a three-foot diameter culvert. It has minimal impact due to the high flushing in this portion of Chatham Harbor.

J. Frost Fish Creek/Route 28 Bridge Discharge. This discharge receives stormwater from portions of Route 28 and impacts both sides of the bridge. Remediation is planned during the Route 28 reconstruction project.

K. Ryders Cove Town Landing Discharge. This discharge is mainly from the parking lot but may receive flows from portions of Route 28. It empties into the Cove and has a small impact. This area has high groundwater conditions and will be difficult to mitigate with conventional groundwater infiltration technologies.

L. Route 28 Discharges along Ryders Cove. Stormwater is discharged from Route 28 at two or more locations along Ryders Cove. These discharges are planned to be remediated as part of the Route 28 reconstruction project.

M. Muddy Creek/Route 28 Bridge Discharge. This discharge receives stormwater from Route 28 at the Muddy Creek crossing. The bridge itself also creates a problem for Muddy Creek because it restricts tidal flushing of Muddy Creek due to a relatively small box culvert. The Route 28 reconstruction project is planned to remediate these problems.

N. Mill Creek Road Discharge. This discharge receives stormwater from approximately 500 feet of Mill Creek Road and flows into Mill Creek. The discharge results in a small water quality impact, as well as a larger erosion impact. Remediation is planned in the fall of 1998.

O. Taylors Pond Landing Discharge. This discharge was at the Taylors Pond Landing and was remediated in early 1998. Some stormwater flow continues and further remediation is planned when a decision is made on how to rebuild the boat ramp.

P. School House Pond Town Landing Discharge. This discharge originates from stormwater generated by the parking lot and portions of the road. It empties into a wetland, which is adjacent to the pond and has a minimal impact.

Q. White Pond Town Landing Discharge. This discharge receives stormwater flow from the road to the pond and portions of Wilfred Road. This discharge has minimal water quality impact but does cause an erosion problem. Remediation is planned in the near future.

R. Old Comers Road Discharge. This was a discharge into the Herring Run from Old Comers Road, which has been corrected.

5.8 MARINE PUMPOUT FACILITIES

Chatham has several harbors, coves, bays, and estuaries where boats are used and moored. Discharges from toilet facilities (marine sanitary devices) on these boats can greatly impact shellfish resources in these coastal embayments. The fecal material from these toilets has a large concentration of fecal coliforms, and if it is discharged into an embayment that is poorly flushed, it will raise the coliform concentration and force closure of the shellfish beds. It can also contribute pathogens, which can lead to public health problems. Stage Harbor has been designated as a No Discharge Zone.

In 1996, a total of 189 vessels were estimated to have marine sanitary devices in the Stage Harbor Complex (Duncanson, 1996). No data was located on the number of boats with marine sanitary devices in Chatham's other coastal waters.

The Town has two public pumpout facilities and both are located in the Stage Harbor Complex. The following text is based on the "Application for a Federal No Discharge Areas Designation for the Stage Harbor Complex" (Duncanson, 1996).

The first unit is a fixed, shore-based pumpout located at the Town-owned Old Mill Boat Yard (OMBY) facility. This pumpout became operational in the summer of 1995. It is centrally located in the Stage Harbor Complex near the entrance to Nantucket Sound and on a Federally maintained channel. This pumpout facility consists of a vacuum unit with a remote stand located at the loading/off-loading float at OMBY. This unit has a 60 gallon per cycle capacity with discharge to a 2,000 gallon tight tank. The OMBY float provides access for vessels up to 50 feet in length and a draft of five feet at mean low water. This pumpout is equipped to remove waste from portable toilets.

The second unit is a trailer mounted portable pumpout with a 225 gallon capacity. This unit was purchased by the Town in 1993, and has been operated under a contract by Stage Harbor Marine (SHM). This unit is located on the Mitchell River, on the southside of the Bridge Street Bridge. The SHM unit is accessible via the fuel dock, which provides services to vessels of up to 40 feet and draft of six feet at MLW. This unit, while principally located at Stage Harbor Marine, is available for pumpouts in all waters throughout Chatham. This pumpout is also equipped to remove waste from portable toilets.

Both pumpout facilities have been designed to require minimal maintenance throughout their operational life. Both units are serviced according to the manufacturer's recommendations. No fees are charged for use of either pumpout facility. Costs of the pumpout facilities are being funded through mooring fees, boat excise taxes, and a grant. Pumpouts at the OMBY location are self-service with oversight provided by personnel from the Chatham Harbormaster's office.

Pumpout waste from the OMBY facility is collected and stored in an existing, Department of Environmental Protection approved 2,000-gallon tight tank. This tank is equipped with all necessary alarms to indicate when pumping is required and to prevent overflow. The tight tank was pumped three times in 1994 and four times in 1995. Following activation, 80 gallons were pumped from six vessels. Waste is removed from the tight tank under existing procedures by a licensed waste hauler, and transported to the Chatham Water Pollution Control Facility for treatment.

Pumpout waste from the portable 225-gallon pumpout unit is discharged directly at the Chatham Water Pollution Control Facility for treatment. In 1994, six vessels were pumped with 100 gallons removed. In 1995, four vessels were pumped with 50 gallons removed. As a result, the unit has only been emptied once per year to date (1996).

The Pleasant Bay Plan (Pleasant Bay TAC, 1998) recommended that Pleasant Bay be designated a No Discharge zone. According to USEPA requirements, this designation would require that a marine pumpout facility be located in that area. It is unknown if the portable unit could meet the requirement. According to discussions with the Town's Water Quality Laboratory Director, the pumpout demands of Pleasant Bay need to be evaluated before a pumpout facility is dedicated to the Ryders Cove area. Most of the boats in this area are relatively small, day sailors, and do not have toilet facilities. The Pleasant Bay Plan considered recommendation of a pumpout facility for the Fish Pier but decided that a pumpout facility would not be needed because the boats that use the pier are offshore commercial boats which typically discharge their waste offshore where it is not a problem to shellfish and estuary areas.