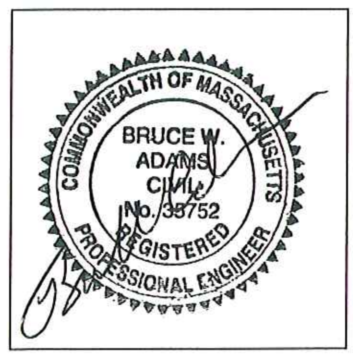
Barnstable, Massachusetts

Hyannis Water System Master Plan

April 2007

Weston&Sampson
ENGINEERS, INC.
Report



Weston & Sampson Engineers, Inc.
Five Centennial Drive
Peabody, MA 01960-7985

www.westonandsampson.com
Tel: 978-532-1900
Fax: 978-977-0100

Five Centennial Drive
Peabody, MA 01960-7985
tel: 978-532-1900 fax: 978-977-0100
www.westonandsampson.com

planning, permitting, design, construction, operation, maintenance, design/build, & equipment
Weston&Sampson®

Barnstable, Massachusetts
Weston & Sampson No. 2060316

April 9, 2007

Mr. Hans Keijser
Supervisor
Water Supply Division
Town of Barnstable
47 Old Yarmouth Road
Hyannis, MA 02601

Re: Water System Master Plan Final Report

Dear Mr. Keijser:

We are pleased to submit to the Hyannis Water Supply Division, Department of Public Works, Town of Barnstable, the attached Water System Master Plan. This document presents the results of our comprehensive study of the Hyannis Water System. A plan of improvements was prepared to ensure that the quality and quantity of water will meet the future needs of Hyannis.

We wish to acknowledge your assistance and that of Dave Condrey of Pennichuck White Water Inc. and all the staff who assisted the project team in gathering background information for the project. Their cooperation was essential in the completion of the report and is sincerely appreciated.

Jeffrey W. McClure, P.E. was the project manager in charge of the project, and Johanna D. Nagle was the engineer who assisted with this project. Thank you for this opportunity to be of assistance to the Town of Barnstable.

Very truly yours,

WESTON & SAMPSON ENGINEERS, INC.

Bruce W. Adams, P.E.
Vice President

COMMONWEALTH OF MASSACHUSETTS
BRUCE W. ADAMS
CIVIL
No. 35752
REGISTERED
PROFESSIONAL ENGINEER

O:\Barnstable MA\2060316 Water Master Plan\Corresp\Final Report letter.doc

1.0 EXISTING WATER SYSTEM

1.1 General

The Town of Barnstable is located southeast of Boston in Barnstable County on the southern side of Cape Cod. The town is bordered by Cape Cod Bay on the north, Nantucket Sound on the south, Sandwich and Mashpee on the west and Yarmouth on the east. The Town of Barnstable includes seven villages within its boundaries. The village of Hyannis, which this report will focus on, is primarily residential and includes the town's central business/commercial district, town offices and several shopping malls. It is more urbanized and commercialized than the remainder of Barnstable. [1]

The Hyannis Water Supply Division, Department of Public Works, Town of Barnstable (Hyannis Water System) water withdrawal points are all in the Cape Cod Basin. The Hyannis Water System is currently registered through the Department of Environmental Protection (DEP) to withdraw water from twelve groundwater wells, eleven of which are currently active, and zero surface water withdrawal points. Well locations are shown on Figure 1-1.

The Hyannis Water System includes one service area without any sub-systems of varying pressure. Groundwater is pumped into the system from eleven well pumping stations through four treatment plants and is distributed through a network of water mains approximately 107 miles long.

1.2 Water Supply and Treatment

1.2.1 Maher Wells No. 1, No. 2, and No. 3

The Maher Well No. 1 was installed in 1971 and is located in a block building set off the access road to the Maher Treatment Facility. The well has a safe yield of 972 gallons per minute (gpm) after surging in 2006, and consists of a 24-inch by 48-inch gravel packed well. The well is screened at two 10-foot intervals at depths of 37 to 47 feet and 70 to 80 feet. The pump was replaced in 2002 following a lightning strike. The pump is a Byron Jackson 12 GM 2 Stage pump with a 25 HP

[1] The Village of Hyannis has a population of approximately 16,000 (Census 2000, Cape Cod Commission). The entire water distribution system serves approximately 7,345 accounts (2005 Public Water Supply Annual Statistical Report).

Motor. Its intake is at a depth of 53 feet. Barnstable DPW reports that the Maher Well No. 1 was redeveloped, chemically cleaned and surged in June 2006, which increased capacity significantly from 17 gpm/ft at 517 gpm to 37.6 gpm/ft at 771 gpm. Its original capacity was 60 gpm/ft at 1002 gpm. Treatment includes chemical sequestering. C5 is pumped in at the station as a pretreater to sequester iron and manganese. All three Maher Well pumps turn on to maintain a 7-foot water height of clearwell as described in Section 1.2.7. The structure has an intrusion alarm.

The Maher Well No. 2 was installed in 1975 and is located behind the Maher Treatment Facility. The well has a safe yield of 869 gpm and consists of a 24-inch gravel packed well with a 10-foot shutter screen at a depth of 44-feet. The total depth of the well is 54 feet. The well has a Goulds 7CHC 1 Stage submersible pump with a pitless adapter, installed July 2001. (The pump is rated for 500 gpm at a design head of 85 feet and has a 15 HP Franklin Motor.) The pump intake is 33-feet 6-inches from the pitless adapter. The pump had an original specific capacity of 45.9 gpm/ft at 800 gpm. March 2006 flow tests show a specific capacity of 39.7 gpm/ft at 548 gpm. The pitless adapter fence/roof fence area has a sample tap, main electric disconnect, and a well level sensor.

Maher Well No. 3 was installed in 1976 and is located in front of the Maher Treatment Facility. The well has a safe yield of 663 gpm. It is an 18-inch well with a total depth of 48.7-feet. The well is equipped with a Goulds 8RJHC 1 Stage submersible pump with a pitless adapter, installed July 2001. The pump is rated for 650 gpm at a design head of 85 feet and has a 20 HP Franklin Motor. The pump intake is 26 feet from the pitless adapter. March 2006 flow tests show a specific capacity of 31.8 gpm/ft at 608 gpm. The pitless adapter fence/roof fence area has a sample tap, main electric disconnect, and a well level sensor.

1.2.2 Mary Dunn Wells No. 1, 2, 3, 4

The Mary Dunn Well No. 1 was installed in1976. It is located off of Mary Dunn Road. The well has a safe yield of 534 gpm and consists of a 24-inch by 48-inch gravel pack with a 10-foot screen. The well has a depth of 54.6 feet. The well has a Goulds 8RJLC 3-stage submersible pump rated for 500 gpm at a design head of 225 feet set in a heavy duty pitless adapter, with a 40 HP motor. The pump intake is 23-feet 6 inches from the pitless adapter, which is adjacent to the old pump building. Flow tests in 1976 showed the well having an original specific capacity of 39 gpm/ft at 749 gpm. D.L.

Maher reports that the specific capacity has declined 49 percent since the 2000 redevelopment of this well. This well has emergency power provided by a generator inside the Mary Dunn Water Treatment Plant.

The Mary Dunn Well No. 2 was installed in 1975 and is located behind the Mary Dunn Treatment Facility. It has a safe yield of 567 gpm and consists of a 24-inch by 48-inch gravel pack with a 10-foot screen and total depth of 50 feet. The pump installed is a Byron Jackson 10 H Six Stage vertical well pump rated for 700 gpm at a design head of 225 feet. The U.S. motor is a 50 HP motor running at 1780 rpm, replaced in the summer of 2005. D.L. Maher reports a 25.2 gpm/ft specific capacity at 472 gpm in March 2006. The specific capacity has declined 39 percent since the well was redeveloped in 2000. This well has an auxiliary power generator located adjacent to the building (not provided by a generator inside the Mary Dunn Water Treatment Plant). The well has an automatic transfer switch and a full load motor starter in the building.

In August 2006 a perchlorate release was reported to the DEP. The perchlorate eminated from road and marine flares and rockets on the grounds of the Barnstable Fire Training Academy (BFTA), upgradient of Mary Dunn Well No. 2. The concentrations of perchlorate were above the RCGW-1 and within the Zone II to the downgradient public water supply. The re-sampling of monitoring wells indicate that a sustained hot spot mass of perchlorate is migrating downgradient of the BFTA site. Currently, a monitoring and remediation plan is in place to abate or mitigate the elevated levels of perchlorate eminating from the release area. Mary Dunn Well #2 will be closed for the 18-24 month duration of the clean-up to help ensure that contaminated groundwater is not pulled into the distribution system.

The Mary Dunn Well No. 3 was installed in 1975 and is located off of South Flint Rock Road near the electric transmission line easement. This well is 10-inches in diameter and has a submersible 5 stage pump with a relatively new Emerson Motor Company 40 HP motor running at 1780 rpm. The pump is rated for 500 gpm at a design head of 225 feet. Flow tests performed by D.L. Maher in March 2006 show a specific capacity of 29 gpm/ ft at 442 gpm. The original specific capacity in 1975 was recorded as 25 gpm/ft at 503 gpm. Previous contamination from the Cape Cod Potato Chips facility's chloroform plume and a benzene, toluene, ethylbenzene, and xylene (BTEX) plume from the Barnstable Fire Training Academy forced the pump station to be used only during emergency conditions. However, the contamination has been mitigated and the pump was restored to active use in November 2004. Evidence indicates that dumping of rubbish has occurred on this site and is a continuous concern.

The Mary Dunn Well No. 3 is located inside a block building approximately 10-feet by 15-feet. The structure has a new asphalt roof and ceiling. The station has flooded in the past and there is not a sump pump installed. The station is in need of a new soft start motor starter. This well does not have an auxiliary power source.

The Mary Dunn Well No. 4 was installed in 1976 and is located off of Mary Dunn Road on the electric transmission line easement. The well has a safe yield of 296 gpm and consists of a 24-inch by 48-inch gravel pack with a 10-foot screen. The DEP Annual Statistical 2002 Report indicates a well pump capacity of 500 gpm and a depth of 46 feet which conflicts with D.L. Maher data, which reports a well depth of 51 feet. D.L. Maher performed flow tests in March 2003 and found the specific capacity to have declined 39 percent since 1992. This well is utilized for emergencies only since microscopic particulate analysis (MPA) testing in the past indicated that the water produced by this well may be under the influence of surface water. The well is currently monitored and tested due to the MPA concerns. The gate valve located outside the building is closed and water is currently drained from the header piping. The well is located inside a 10-foot by 10-foot small block building with a new plywood and asphalt shingle roof. The building is not enclosed by a fence and does not have an auxiliary power source.

1.2.3 Airport Well

The Airport Well was installed in 1971 and is located off of Mary Dunn Road. The flow from this well goes to the Mary Dunn Treatment Plant. The well has a safe yield of 459 gpm and consists of a 48-inch by 24-inch gravel pack with a 10-foot screen with a total well depth of 63 feet. The well is equipped with a Goulds 12CMC 4 Stage vertical pump and is rated for 1000 gpm at a design head of 255 feet with a 100 HP motor. The pump intake is at a depth of 56 feet. The structure was constructed between 1995 and 1996. March 2003 Flow tests performed by D.L. Maher indicate a 50 percent decline in specific capacity. Barnstable DPW reports that the Airport Well was redeveloped, chemically cleaned and surged in June 2006, which increased capacity significantly from 26.9 gpm/ft at 383 gpm to 51 gpm/ft at 770 gpm. Its original capacity was 60 gpm/ft at 1002 gpm. The well is treated with a C-5 chemical sequestering agent.

The Airport Well is located in a fairly new, block building 15-feet by 10-feet in size. The building features a sloped roof. The station is equipped with a full load motor starter and a Parco Surge Control Valve. There is no auxiliary power.

1.2.4 Simmons Pond Well

The Simmons Pond Well was installed in 1975 and is located off of Smith Road. The well has a safe yield of 792 gpm. It is an 18-inch well that consists of a 15-foot telescope screen with a depth of 74 feet. It has a Byron Jackson 10MQH six stage vertical pump rated for 600 gpm at a design head of 265 feet with a 50 HP motor. The pump intake is at a depth of 50-feet 6-inches. In 1975, the original specific capacity of this well was found to be 21.1 gpm/feet at 800 gpm. D.L. Maher performed flow tests in March 2003, which indicated a 24 percent decline. Chemical sequestering is used for treatment at this well. The pump station has an engine and a right-angle gear drive to supply auxiliary power. March 2006 flow testing performed by D.L. Maher found the specific capacity to be 23.3 gpm/ft at 442 gpm. The well runs on tank level controlled from the office building but has the ability to run off distribution system pressure in emergency situations. The well has C-5 chemical sequestering treatment.

The structure is built of concrete block and has recently received a new roof. A propane tank is located at the well, which fuels the direct drive engine right angle drive. The building has an electric unit heater. The well uses a Parco Surge Control Valve.

1.2.5 Hyannisport Well

The Hyannisport Well was installed in 1975 and is located off of Smith Road. The well has a safe yield of 715 gpm and consists of a 15-foot screen and a depth of 75 feet. It is equipped with a Goulds 10RJLO 7 Stage vertical pump rated for 500 gpm at a design head of 266 feet with a 40 HP motor. The pump intake is at a depth of 46-feet. The well's original specific capacity was recorded at 18 gpm/feet in 1975. Flow tests in March of 2003 show a 6 percent decline in specific capacity.

March 2006 flow testing performed by D.L. Maher found the specific capacity to be 19.9 gpm/ft at 517 gpm. Treatment at this well includes chemical sequestering and pH adjustment.

1.2.6 Straightway Wells No. 1 and No. 2

Straightway Well No. 1 was constructed in the early 1970's. Straightway Well No. 2 was constructed in April of 1998. They are both located off of Straightway Road. Well No. 2 has a safe yield of 1530 gpm and consists of a 24-inch by 18-inch gravel pack with a 25-foot long screen at a depth of 187 feet. The intake for the well pump is at 100 feet. Well No. 2 has a Goulds 12CHC 5-Stage pump rated for 1,100 gpm with a 125 HP motor. March 2006 flow testing performed by D.L. Maher found the specific capacity to be 14.5 gpm/ft at 704 gpm. The original capacity of the well was 60 gpm/ft at 1002 gpm. The well has high levels of manganese present.

Well No. 1 is inactive. It is capped and is not used due to iron and manganese problems. It consists of a 15-foot long screen and has a depth of 60 feet. The facility doesn't currently have a generator, but is equipped with a generator room for future installation.

1.2.7 Maher Treatment Facility

Maher Treatment Facility is located off of Old Yarmouth Road and was renovated in 1991. The facility treats the water from Maher Wells No. 1, No.2, and No.3. Variable frequency drives run the 25 HP motor for Maher Well No. 1 and the 15 HP motor for Maher Well No. 2 at 74 percent and 64 percent, respectively. Maher Well No. 3 runs off a 20 HP motor. The treatment facility has three generators, with automatic transfer switches, located inside the building to provide auxiliary power for the raw and finished water pumping. Two generators supply auxiliary power to Maher Wells No. 1, No. 2, and No. 3. The third, larger generator supplies power to the finish water pumps and the remaining load of the treatment facility.

Water is pumped from the wells to the air stripper, which is located outside of the building. The air stripper is 8 to 10 years old and removes volatile organic compounds (VOCs) found in the raw water. VOCs are presumed to be from Tier I sites located on Iyannough Road. The air blower used to feed the air stripper is located inside the new portion of the facility, adjacent to the air stripper. The blower appears oversized, likely for future contaminant load.

After treatment, water enters a 250,000-gallon clearwell from which water is pumped into the distribution system by three vertical turbine pumps. The finished water pumps are equipped with variable frequency drives and Parco Surge Control Valves. The clearwell is located in a submerged area of the facility below the vertical turbine pumps.

Additional chemicals are injected in the discharge piping post clearwell. Treatment includes chemical sequestering with C-5, pH adjustment, and disinfection with sodium hypochlorite. The chlorine is stored in the overhead door area, which previously contained a large chlorine tank that was oversized and removed. The sequestering chemicals are also located in the older portion of the facility. The sodium hypochlorite and zinc orthophosphate are located in a containment area adjacent to the submerged clearwell. The chlorine analyzer repeatedly plugs with iron due to the water quality. There is no air conditioning in the treatment area.

1.2.8 Mary Dunn Treatment Plant

Mary Dunn Treatment plant was constructed in either 1995 or 1996 and is located on Mary Dunn Road. The facility treats water from Mary Dunn Wells No. 1, No. 2, No. 3, and No. 4 as well as the Airport Well. Treatment includes C-5 and C-9 chemical sequestering and pH adjustment. Sodium hypochlorite is added for disinfection when flushing the distribution system or when bacteria hits are detected. The chemical feed is flow-paced. The plant is equipped with a propane-fired generator that serves Mary Dunn Well #1 and the treatment plant building.

1.2.9 Hyannisport Treatment Plant

The Hyannisport Treatment Plant was constructed in 1991 and treats raw water from the Hyannisport Well (located inside treatment facility) and the Simmons Pond Well. C-5 and C-9 chemical sequestering takes place in this facility. The facility is equipped with a generator located outside of the building, which supplies auxiliary power for lights and chemical pumping only. This generator does not supply auxiliary power for the Hyannisport Well. The facility has the ability to inject chlorine, however it is not actively needed or used. The Simmons Pond well has a right angle drive to run the well in the event of a power failure.

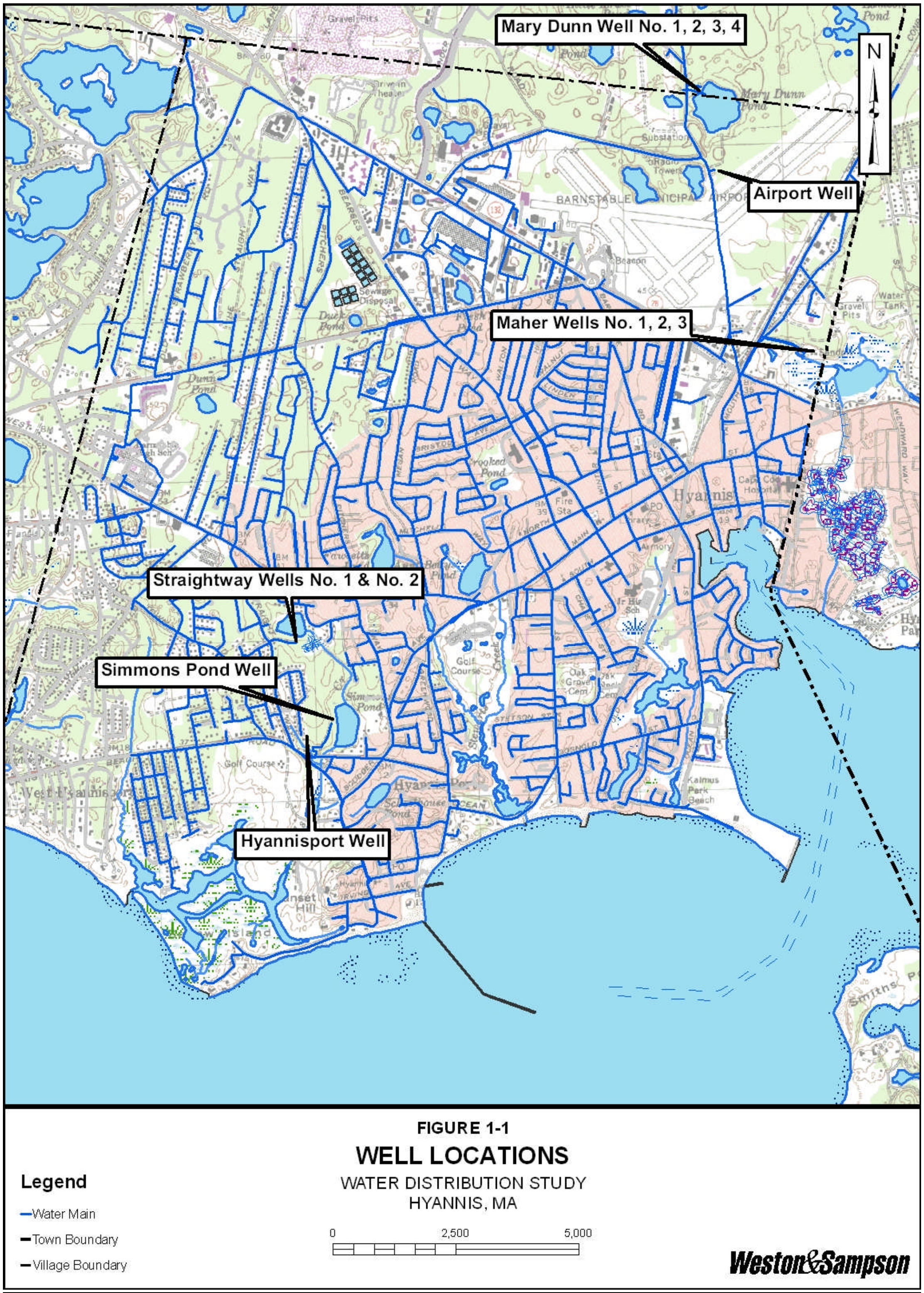


FIGURE 1-1

WELL LOCATIONS

WATER DISTRIBUTION STUDY
HYANNIS, MA

Table 1-1

Barnstable Groundwater Supply Facilities

Well No.	Depth (feet)	Diameter (inches)	Year Constructed	Safe Yield (gpm)	Chemical Addition+	Backup Power
Maher Well No. 1	80	24	1971	972	Calgon C-5	Yes
Maher Well No. 2	54	24	1975	869	Calgon C-5	Yes
Maher Well No. 3	48.7	18	1976	663	Calgon C-5	Yes
Mary Dunn Well No. 1	54.6	24	1976	534	Calgon C-5	Yes
Mary Dunn Well No. 2	50	24	*	567	Calgon C-5	Yes
Mary Dunn Well No. 3	*	*	1975	442	Calgon C-5	No
Mary Dunn Well No. 4 (Active, Emergency Only)	51	24	1976	296	Calgon C-5	No
Airport Well	63	24	1971	459	Calgon C-5	No
Simmons Pond Well	74	18	1975	792	Calgon C-5	Yes
Hyannisport Well	75		1975	715	Calgon C-5, C-9, NaOH	Partial^
Straightway Well No. 1	60	24	1988	Not Active		Not Active
Straightway Well No. 2	187	24	1988	1530	Calgon C-5, C-9, NaOH	No

* Unknown at this time
^ Backup for lights and chemical feed only, not for pump
+ All water receives C-9 chemical addition before distribution

1.2.10 Water System Safe Yield

Because there is a limited amount of groundwater within an aquifer, well fields are rated for the maximum amount of water which may be withdrawn over a long period of time so that the wells are not pumped dry. According to the Massachusetts DEP 2001 Guidelines and Policies for Public Water Supply Systems, the safe yield is defined as the yield that can be produced by pumping a well 24 hours per day throughout the year during a moderate drought. Although the maximum (short-term) yield is somewhat higher, it would not be good judgement to assume the maximum supply could be produced at the time of the maximum day usage. Supplies are usually maximized during the wet spring season; however, maximum usage normally occurs during the dryer summer months. The safe yield for Hyannis will be dependent on mutual interference between the wells and potential intrusion of salt water in this region. The DEP registered withdrawal volume for the Hyannis Water System is 1.49 mgd. The system-wide safe yield for the Village of Hyannis should be determined through an additional hydrogeologic study including prolonged pump tests and a computer model of groundwater flow such as MODFLOW, which is beyond the scope of this report.

1.2.11 Emergency Connections

The Hyannis Water System maintains four emergency backup connections with surrounding towns' water supply systems. The emergency connections are shown in Table 1-2. Locations are shown on Figure 1-3. The connection with the Centerville/Osterville (COMM) system at Craigville Beach does not exist anymore. There is also the possibility for an emergency connection with the Barnstable Fire District Water Department behind the Home Depot on Independence Drive, should the Water Supply Division wish to pursue a connection with the Barnstable Fire District Water Department.

Table 1-2

Emergency Water Service Connections

Town	Water Main Size	Location
Yarmouth	12-inch	Main Street
Yarmouth	6-inch	Willow Street
Centerville/Osterville	10-inch	Longview Drive (Hydrant Bypass)
Centerville/Osterville	10-inch	Old Town Road

1.3 Distribution System and Storage

1.3.1 Existing Water Mains

Water is transmitted through approximately 107 miles of 2-inch to 20-inch diameter water mains. Tuberculation problems have been found in water mains installed prior to 1930. Tuberculation is the corrosion of the inner surface of cast iron pipes, which causes the diameter to decrease and flow to become restricted. Tuberculation is more prevalent on unlined cast iron mains than cement lined, cast and ductile iron mains.

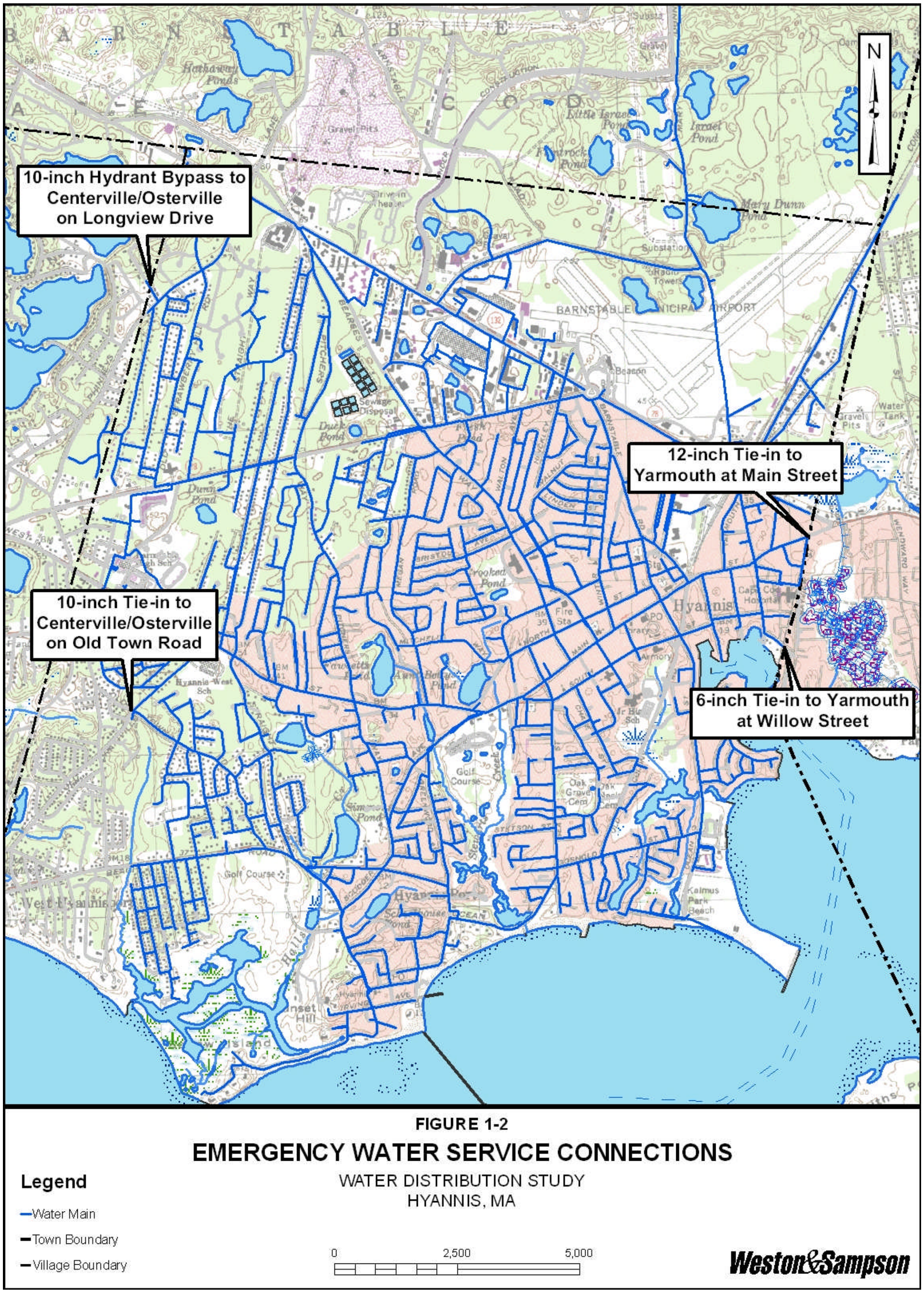


FIGURE 1-2

EMERGENCY WATER SERVICE CONNECTIONS

WATER DISTRIBUTION STUDY
HYANNIS, MA

Therefore, the unlined water mains are more in need of improvement. Of the 107 miles of water main in the distribution system, 20 miles (18.4-percent) of water mains are not cement lined. Table 1-3 shows the lengths of lined and unlined water main in the system for each pipe size.

Table 1-3

Summary of Existing System Water Mains

Main Size (inch)	Unlined Main (feet)	Unlined Main %	Cement-Lined Main[1] (feet)	Cement-Lined Main[1] %
<6	6,273	1.11	72,549	12.88
6	57,226	10.16	123,471	21.92
8	14,456	2.57	165,135	29.31
10	0	0	56,607	10.05
12	18,137	3.22	24,150	4.29
16	7,639	1.35	12,080	2.14
20	0	0	5,653	1.00
Total	**103,731**	**18.41**	**459,645**	**81.59**

(1) This table is based on the assumption that Barnstable began using cement lined pipe in 1933

The Distribution system includes one service area without any sub-systems of varying pressure. Groundwater can be pumped into the system through twelve well pumping stations, ten of which are currently active. The hydraulic gradient of the service system is maintained by two water storage tanks, which have overflow elevations of 223 feet.

1.3.2 Distribution System Storage Facilities

The village of Hyannis has two water storage tanks (see Table 1-4) that serve the main distribution system with a total capacity of 1.370 million gallons (MG). The storage tanks are located off of Mary Dunn Road, as shown in Figure 1-2. Mary Dunn No. 1 Storage Tank is a 370,000-gallon riveted steel tank with a wooden roof that was constructed in 1911. The tank is 25 feet in diameter and 98 feet tall. Emergency repairs have been made on this tank in 2002 following a water quality test, which indicated the presence of E. coli bacteria. Roof repairs were completed on this tank in the summer of 2003. Mary Dunn No. 2 Storage Tank is a 1,000,000-gallon welded steel tank that was constructed in 1963. The tank is 42 feet in diameter and 98 feet tall. The tank has two

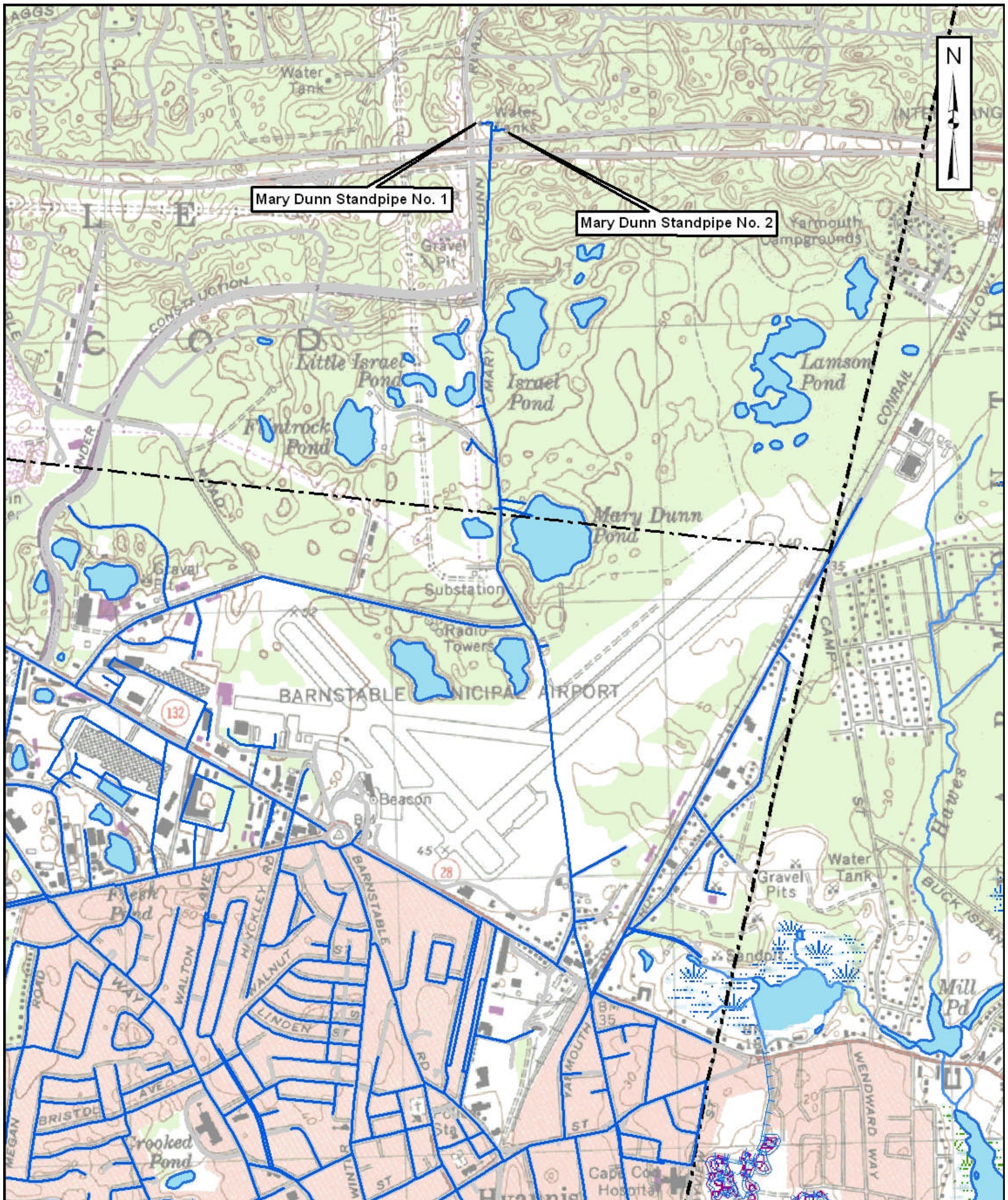


FIGURE 1-3

WATER STORAGE LOCATIONS

WATER DISTRIBUTION STUDY
HYANNIS, MA

Legend

- Water Main
- Town Boundary
- Village Boundary

0 1,600 3,200

Weston&Sampson

communications antennae located on the roof. Sediment in this tank was removed and the interior was painted in October/November of 2002.

Table 1-4

Distribution Storage Facilities

Storage Facility	Service System	Year Built	Overflow Elevation (ft)	Nominal Capacity (MG)
Mary Dunn No. 1 Storage Tank	Main	1911	223	.370
Mary Dunn No. 2 Storage Tank	Main	1963	223	1.00

Both storage tanks were last inspected in October of 2001 by Extech, LLC and have had their outlets screened and overflow pipe replaced following the inspection. At the time of inspection, Mary Dunn No. 2 tank appeared to be in good structural condition. The inspection did find minor mildew and algae on the exterior of the lower rings of the tank. The coating was found to be in good condition and the concrete ring wall had no cracking or spalling. Minor corrosion was found on the anchor rods and on the interior weld seams. An ASTM adhesion test on the roof showed that the coating had failed down to substrate. Weston & Sampson recommends that the Water Supply Division have regular inspections performed for both tanks in order to ensure that they both remain in acceptable condition.

1.3.3 Typical Distribution System Water Pressure

The maximum and minimum ground elevations and corresponding potential water pressures, as well as the maximum hydraulic grade line (HGL), are shown in Table 1-5. The pressure in Table 1-5 was determined using the typical water tank operational range of 10 feet and the maximum and minimum ground elevations in the service area. Water pressures seen in the Hyannis Water System are within DEP guidelines.

Table 1-5

Typical Service System Pressure Ranges

Maximum Elevation(1) (feet)	Minimum Elevation(1) (feet)	Minimum Pressure (psi)	Maximum Pressure (psi)	Max. HGL (feet)
78	3	58	96	223

(1) Ground elevations referenced from USGS datum

1.3.4 Typical Distribution System operations

The Hyannis Water Supply Division generally operates their water supply sources starting with Maher Treatment Plant, Hyannisport Well, and Simmons Pond Well. Mary Dunn Well No. 1 & 2 and Straightway are also used as required throughout the year. Mary Dunn Well No. 3 & 4 and Airport Well are mostly used during peak demands and are limited by its capacity to pump at the site. The Hyannis Water Supply Division cleans an average of two wells per year.

O:\BARNSTABLE MA\WATER MASTER PLAN\REPORT\CHAP1.DOC

2.0 WATER SUPPLY REQUIREMENTS

2.1 General

In order to accurately forecast future water supply requirements, it is necessary to analyze water consumption records and to project long-term changes in consumption during the design period. The following estimates of future water supply demands are based upon projected population served, per capita water usage, categorical water use, unaccounted-for water, and maximum day to average day demand ratios.

2.2 Population

Population figures for Hyannis, as reported by the U.S. Census Bureau, are shown in Table 2-1. The 1950, 1960, 1970, and 1980 U.S. census included a breakdown of population for Hyannis including Hyannisport. The 1990 and 2000 census had a total population for the Town of Barnstable but not a separate population for the Village of Hyannis.

TABLE 2-1
HYANNIS POPULATION PER U.S. CENSUS BUREAU, 1950-2000

YEAR	Town of Barnstable	Hyannis Village	PERCENT INCREASE
1950	10,480	4,235	
1960	13,465	5,139	21.3%
1970	19,842	6,847	33.2%
1980	30,898	9,118	39.9%
1990	40,949	12,694*	32.5%
2000	47,989	14,877*	17.2%

*Assumes 31% of Barnstable population

Yearly population figures for Hyannis, as recorded by the town clerk are shown in Table 2-2.

TABLE 2-2
HYANNIS POPULATION FIGURES
AS RECORDED BY THE TOWN CLERK

YEAR	Town of Barnstable	Hyannis Village*
1980	31,334	9,714
1981	38,760	9,846
1982	31,303	9,704
1983	31,567	9,786
1984	31,526	10,393
1985	33,479	10,378
1986	33,295	10,321
1987	35,708	11,069
1988	36,304	11,254
1989	39,304	12,184
1990	31,439	9,746
1991	32,822	10,175
1992	38,037	11,791
1993	41,227	12,780
1995	43,184	13,387
1996	43,500	13,485
1997	Not Available	Not Available
1998	Not Available	Not Available
1999	Not Available	Not Available
2000	Not Available	Not Available
2001	Not Available	Not Available
2002	45,072	13,972
2003	40,126	12,439
2004	Not Available	Not Available
2005	39,690	12,304

*Assumes 31% of Barnstable population

Inquiries to the town clerk's office indicated that the town does not maintain record of the 1997-2001 population, nor is there record of the 2004 town census results. The town does not keep population data for each village and the town census data is available for some years, but not others. It should be noted that the geographical limits of the water system do not exactly match the village of Hyannis boundary lines. Some houses outside of the Hyannis political borders are served by the Hyannis Water System. Therefore the population figures should be considered an estimate.

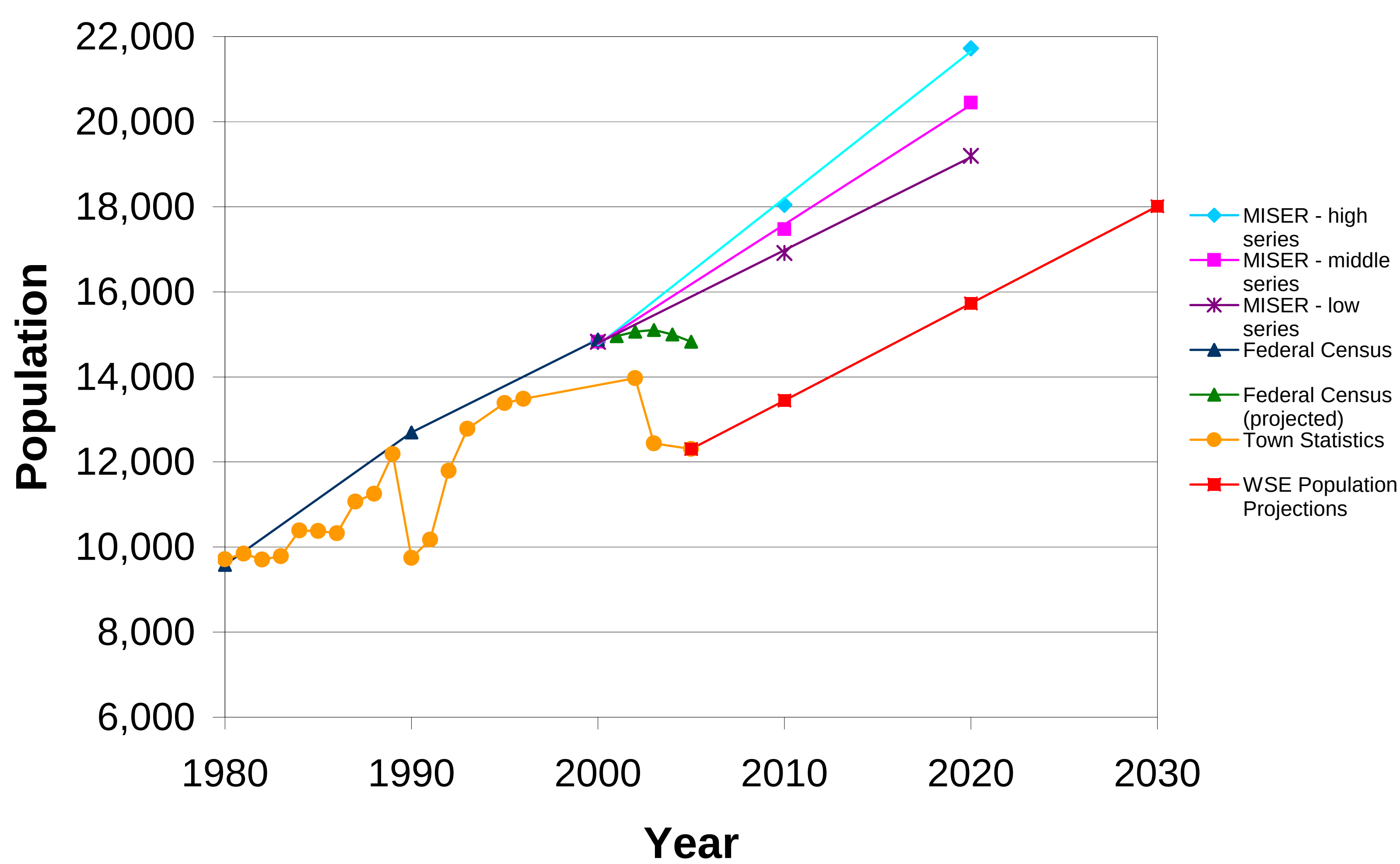
The estimates for the Village of Hyannis were based on 31% of the projected Barnstable population. This number most closely represents the percentage of the Barnstable population that is Hyannis based on the information received from the Town of Barnstable Growth Management Department. The Growth Management Department provided the population for the Town of Barnstable in the year 2000 as 47,821 and the population for the Village of Hyannis in 2000 as 14,924. Weston & Sampson was told by the Growth Management Department that any population increases since the year 2000 have been offset by about equal decreases therefore making 31% a relatively accurate number in expressing the population of Hyannis in terms of the entire Town of Barnstable.

Population projections for Barnstable for the years 2010 and 2020 were prepared by the Massachusetts Institute for Social and Economic Research (MISER). MISER is the Massachusetts State representative to the Federal/State Cooperative Program for Population Estimates, which distributes population estimates annually for the state. These MISER projections are listed in Table 2-3 along with the Weston & Sampson population projections. Figure 2-1 shows the historical population and population projections.

TABLE 2-3
HYANNIS POPULATION PROJECTIONS, 2010-2020

YEAR	MISER High Series POPULATION	MISER Middle Series POPULATION	MISER Low Series POPULATION	Weston & Sampson POPULATION
2010	18,042	17,474	16,915	13,446
2020	21,726	20,445	19,198	15,729

Figure 2-1
Population Projections



Weston & Sampson estimates that the future population of Hyannis will be less than the MISER low series population projections. By normalizing the 2005 population as given by the town clerk and using the same slope as the MISER Low Series projection, Weston & Sampson created projections that we believe are more accurate in representing the future population of Hyannis. This conclusion was based on the recent decline in Hyannis population, information received from the Barnstable Growth Management Department, and takes into account housing regulations newly adopted by the town.

Weston & Sampson obtained a copy of the *Hyannis Downtown Buildout Analysis*, dated September 27, 2005, which was prepared to estimate the potential for development and redevelopment within an area of downtown Hyannis. The analysis indicates that the current number of multi-family housing units is 678 and the 20-year buildout multi-family is 2,573 units. The amount of office space, restaurant space, and retail space is also expected to increase. However, the number of single-family units is the only category expected to decrease. The current number of single-family units in downtown is 309 units and the 20-year buildout indicates only 151 single family units. New single-family development in all areas of Hyannis has greatly decreased since the minimum lot size for a single-family new construction has been increased to 2 acres. This decision effects roughly two thirds of the town. The build-out projections for Hyannis are less than the build-out projections for the entire Town of Barnstable since the regulations for Hyannis are more restrictive. The downtown area of Hyannis will see an increase in population density as a result of new smart growth initiatives. The town of Barnstable is also subject to Chapter 40B regulations which could cause an increase in the population with more affordable housing units being constructed, mainly in the downtown area. However, any increase in water usage due to rapid growth in the downtown area is offset by the reduction in water use due to the loss of single family housing throughout Hyannis. In general, as housing density increases, per capita water consumption decreases. More water is normally consumed in single-family residences for such uses as lawn watering and car washing than in multi-family residences.

The Massachusetts Executive Office of Environmental Affairs (EOEA) also estimated population and build-out projections for the Town of Barnstable in 2002. This data states that the maximum population at full build-out would be 59,876 people. Using 31% of the total town of Barnstable population for consistency, this number is similar to the Weston & Sampson population projections of about 18,000 people in Hyannis by the year 2030.

2.3 Population Served

In many communities, a small percentage of residents within the confines of the public water system utilize private well water. The number of private wells in Hyannis is minimal and therefore 100 percent of the population is considered to be using the public water system for the purpose of this report. This study will assume that the same percentage of the population served by the water system will be maintained for all future conditions. The population from 1980 to 2005, as recorded by the town, is shown in Table 2-2.

2.4 Water Use Statistics

Water usage for the years 1996 through 2005 were obtained from the Massachusetts DEP Public Water Supply Annual Statistical Reports that were prepared by the Hyannis Water Department. The average day demand, maximum day demand, and unaccounted-for water are presented in Table 2-4 for the years 1996 through 2005. The water use statistics average day demands have ranged from 2.462 mgd to 2.941 mgd and maximum day demands have ranged from 4.464 mgd to 7.148 mgd. The last ten years show a typical ratio of average day to maximum day demands of 1.7 to 2.0 which is typical for a community like Hyannis.

TABLE 2-4

WATER USE STATISTICS, 1996-2005

Year	Average Day Demand (mgd)	Maximum Day Demand (mgd)	Maximum to Average Water Ratio
1996	2.462	4.506	1.83
1997	2.716	5.184	1.91
1998	2.941	4.860	1.65
1999	2.721	5.519	2.03
2000	2.486	4.752	1.91
2001	2.594	4.673	1.80
2002	2.616	5.121	1.96
2003	2.574	4.464	1.73
2004	2.655	7.148	2.69
2005	2.754	5.216	1.89
Average	2.652	5.144	1.94

2.5 Average Day Demand

Total water demand includes the following: residential, semi-residential, transient, and unaccounted-for water usage. Table 2-5 shows the total water usage in each category while Table 2-6 shows the percent of total usage in each category for the same time period. These tables illustrate that there is no single category of water usage that has varied greatly in the past five years. Because of the consistency of the usage data it is not necessary to develop water demand projections by individually projecting each category of water usage. Instead, demand projections can be made by evaluating total water usage.

TABLE 2-5

WATER CONSUMPTION BY CATEGORY, 2001-2005

CATEGORY	AVERAGE WATER CONSUMPTION (MGD)				
	2001	2002	2003	2004	2005
Residential	1.09	1.33	1.16	1.27	1.10
Semi-residential	0.06	0.04	0.31	0.07	0.14
Transient	0.36	0.37	0.06	0.31	0.00*
Other	0.79	0.65	0.78	0.78	1.27
Unaccounted-For	0.22	0.23	0.27	0.22	0.25

*2005 Statistical Report did not specify consumption in transient category.

TABLE 2-6

PERCENT DISTRIBUTION OF WATER USAGE, 2001-2005

CATEGORY	PERCENT OF TOTAL WATER USAGE				
	2001	2002	2003	2004	2005
Residential	43.3	50.7	45.0	47.7	39.9
Semi-residential	2.3	1.4	11.9	2.8	5.1
Transient	14.3	14.2	2.2	11.8	0.0*
Other	31.2	25.0	30.4	29.4	46.1
Unaccounted-For	8.8	8.7	10.5	8.5	9.0

*2005 Statistical Report did not specify consumption in transient category.

As indicated by Tables 2-5 and 2-6, most of the water consumption within Hyannis is for residential use. Residential demand is affected by factors such as:

- Standard of living - In general, as standard of living increases so does water use.
- Type of community - In general, as housing density increases, per capita water consumption decreases. More water is normally consumed in single-family residences for such uses as lawn watering and car washing than in multi-family residences.
- The number of water-consuming devices such as dishwashers and garbage disposals.

- The number of water conservation devices such as low-flow toilets, shower savers, faucet restrictors, and pressure-reducing valves.
- Metering techniques - In general, metered customers tend to use less water than unmetered customers. The presence of sanitary sewers that utilize the water meter data for billing purposes tend to cause a decrease in water usage.
- Water pricing - in general, the unit cost for water will affect the per capita usage. It is generally assumed that costs will be increased as new legislation creates more stringent water quality requirements. As Barnstable improves the infrastructure of the distribution system, the costs of new water quality treatment and distribution system improvements will be passed on to the customer and per capita water usage will most likely decrease as a result.

2.6 Maximum Day Demand

Maximum day demand is the highest 24-hour demand during the year, and is commonly expressed as a percentage of average day demand. Records from 1996 through 2005, as shown in Table 2-4, indicate that the average maximum day demand is 5.1 mgd and has varied from 165 percent to 269 percent of the average day demand. Based on these figures, the average ratio between maximum and average day demands is 1.94. To provide a conservative estimate for projected demands, a ratio of 2.0 will be used to determine the future maximum day water demands. Hyannis's current water supply facilities, shown in Table 1-1, are sufficient to meet the maximum day demands.

2.7 Per Capita Water Usage

Public awareness of water conservation, for both economic and environmental reasons, has increased dramatically in recent years. The Water Management Act Permitting Policy sets guidelines for residential per capita water use for a public water supplier. The Guidance Document (April 2004) states that the standard shall not be more than 65 gallons per day for residential gallons per capita day (rgpcd) for high and medium stressed basins. For public water suppliers, such as Hyannis, located in a low stress or unassessed basin, the residential per capita water use shall not be more than 80 gpcd.

For this reason it is unlikely that per capita water consumption will increase significantly above recent historical levels. The chances are greater that the per capita consumption may actually decrease. But for the purposes of planning for improvements to the water system, a degree of conservatism is warranted. Therefore, to project the future demand, an analysis of the average per capita consumption for the past four years was utilized.

Past records of total average day demand, population served, and per capita consumption are shown in Table 2-7. The residential per capita consumption rate has fluctuated between a low of 89 gpcd in 2005 to a high of 103 gpcd in 2004. The residential per capita consumption rate has fluctuated with no apparent pattern as population decreases during the years 2002 to 2005. However, the overall per capita consumption has increased regularly over the same period. This indicates that the per capita consumption is driven more by the semi-residential, transient, or other categories as opposed to the residential demands.

With an average per capita consumption of 95 gpcd, the town is not in compliance with the DEP Guidance Document which mandates a per capita consumption less than 80 gpcd for a town such as Barnstable located in a low stress or unassessed basin. The town should strive to ensure that the residential per capita water use is under 80 gpcd each year, in order to comply with the state DEP Policy. It should be noted however that the per capita consumptions listed are estimates and based on the populations given by the town clerk. If the population assumed for Hyannis is low, than the per capita consumption could actually be less than indicated.

TABLE 2-7
PER CAPITA CONSUMPTION

Year	Average Day Demand, mgd	Residential Demand, mgd	Population Served	Per Capita Consumption, gpcd	Residential Per Capita Consumption, gpcd (1)
2002	2.62	1.33	13,972	187.5	95.2
2003	2.57	1.16	12,439	206.6	93.3
2004	2.66	1.27	12,372 (2)	215.0	102.7
2005	2.75	1.10	12,304	223.5	89.4
Average	2.65	1.22		208.2	95.15

(1) Based on Residential Demand
(2) Estimated population since the town does not have record of the 2004 population

The per capita estimate and the Weston & Sampson projected population of the town was used to determine the projected average day consumption. The projected maximum day demands were determined using a 2.0 ratio between the maximum and average day water demands. The results for the projected average and maximum day demands for the period 2010 through 2030 are shown in Table 2-8.

Note that the Water Supply Division does not maintain records of the percent of the town population that is served by the water system versus supplied by private wells. The per capita consumption shown in Table 2-7 is based on total town population since the number of private wells is minimal. The water demand projections shown in Table 2-8 assume the same percentage of the population served by the water system versus private wells is maintained in future years.

TABLE 2-8
PROJECTED WATER DEMANDS, 2010 – 2030

YEAR	AVERAGE DAY DEMAND (mgd)	MAXIMUM DAY DEMAND (mgd)
2010	2.80	5.59
2020	3.27	6.54
2030	3.75	7.49

Weston & Sampson recommends that the town conduct a water supply study in about 15 years time. In this time, the newly adopted smart growth initiatives will have had been in affect and the town will gain a better understanding of future population trends and it's future water supply needs. We also recommend the town keep detailed population records to aid in the forecasting of water supply needs. Hyannis's current water supply facilities, shown in Table 1-1, are sufficient supply for the projected maximum day water demand shown in Table 2-8.

2.8 Peak Hour Demand

Peak hour demands are the highest hourly demands that occur during a 24-hour period and generally occur in conjunction with the maximum day demand. Because peak hour demands can vary anywhere from 1.5 to 6.0 times the maximum day demand, and are short-term demands, they can and should be met from distribution system storage rather than from supply facilities.

2.9 Unaccounted-For Water

The Water Management Act Permitting Policy sets guidelines for un-accounted for water for a public water supplier. The Guidance Document (April 2004) states that the standard shall not be more than 10% for high and medium stressed basins. For public water suppliers, such as Barnstable, located in a low stress or unassessed basin, the un-accounted for water use shall be lower than 15%. The amount of un-accounted for water takes into account normal leakage within a system, under-registration of meters, and the use of unmetered water for municipal purposes such as flushing and fire fighting. For the years 2001 through 2005 Hyannis's unaccounted-for water averaged 9%, as shown in Table 2-9. Hyannis, on average, has met the Water Management Act Guidance Document requirements for unaccounted-for water. The town should continue to try to minimize the amount of un-accounted for water in the system.

One way to help minimize the amount of un-accounted for water is to perform leak detection surveys in order to identify and then fix leaks within the system. A water leak detection survey was performed on the Hyannis Water System between May 3rd and May 14th, 2004. A total of seven leaks were located at this time consisting of five hydrant leaks, one service leak, and one main line leak. The total estimated flow from the seven leaks was found to be 6 gpm. The total loss from these leaks would be over 3 MG per year. Two running bleeders were also noted during the leak detection survey. One at Hamden Circle was running at 3 gpm and was metered. The second, at Whip O Will Drive was running at 9 gpm and was not metered. Weston & Sampson recommends that the town perform leak detection surveys every two years.

By exercising good control of the water system the town has been able to keep the percentage of unaccounted-for water relatively low. The town also tries to meter water that is run to waste for such things as: flushing, filling mains, blow offs, washing sanders, and other things of this nature in order to keep the unaccounted for percentage down.

TABLE 2-9
UNACCOUNTED-FOR WATER

YEAR	TOTAL WATER DELIVERED (mgd)	TOTAL WATER BILLED (mgd)	UNACCOUNTED WATER USAGE (mgd)	PERCENT UNACCOUNTED OF TOTAL
2001	2.526	2.372	0.223	8.8
2002	2.616	2.390	0.227	8.7
2003	2.574	2.305	0.269	10.5
2004	2.655	2.431	0.225	8.5
2005	2.754	2.507	0.247	9.0

2.10 Conclusion

The water usage forecast indicates that Hyannis's average daily water usage will remain fairly constant over the next 25 years. Although the population of Hyannis is expected to increase, most of the increase will be through multi-family housing units as opposed to single family housing. Residential per capita usage should be decreased over the design period in order to comply with the DEP Policy. The residential per capita water usage can be decreased by raising water rates, implementing water re-use strategies, setting up a water conservation bank, educating the public about water use, passing zoning bylaw changes for lawn irrigation, and implementing retrofit programs such as installing low flow fixtures to public buildings. The projected maximum day demand is less than the capacity of the wells. Therefore, it is anticipated that the town will have sufficient water supply through the year 2030.

O:\BARNSTABLE MA\WATER MASTER PLAN\REPORT\CHAP2.DOC

3.0 REGULATORY REQUIREMENTS

3.1 General

The purpose of this section of the report is to present information on pertinent regulations as they relate to Barnstable's water the Hyannis Water System. Information is presented on the Insurance Services Office (ISO) requirements as they relate to water systems, Massachusetts General Law (MGL) 310 CMR 22, and the Massachusetts DEP Guidelines and Policies for Public Water Systems.

3.2 ISO Recommendations

3.2.1 General

A water distribution system has two primary functions. The first is to supply water for domestic, commercial and industrial use, and the second is to provide adequate fire protection. The ISO has established certain standards by which the adequacy of a public water system to provide fire protection can be rated. When establishing fire flow recommendations for a community, the ISO considers the different types of development within the community and establishes a recommended hydrant flow for each type. The recommended rate and duration of flow is based on building structural conditions, type of occupancy, and the congestion of buildings in the district under consideration. The largest fire flow demands generally occur in the principal business and industrial districts of a community.

The degree of compliance with ISO recommended standards is used to set fire insurance rates within a community. In the design of waterworks, it is considered good practice to adhere to these standards to minimize fire insurance rates within a community and reduce the risk of human casualties and property damage resulting from fires.

3.2.2 Recommended Fire Flows

Recommended fire flows are defined as the recommended minimum fire flow rate from the distribution system. To meet the recommended fire flow, a minimum pressure of 20 pounds per square inch (psi) must be maintained in the water mains per MGL 310 CMR 22. In October 2006, the ISO evaluated the village of Hyannis within the town of Barnstable. Twenty-two fire flow tests were performed. Two of the 6 residential sites tested had inadequate fire flows. Of the 16 commercial sites tested, 13 had inadequate fire flows. The results of the Hyannis ISO evaluations are shown in Appendix A.

According to the Barnstable Water Supply Division, some town municipal buildings have sprinkler systems. In addition, eight new 4-story buildings are proposed on Main Street, all to have sprinklers. Buildings that do not contain sprinkler systems generally have fire flow requirements up to two times that for fully sprinklered buildings. Some of the larger buildings without sprinkler systems may have fire flow requirements larger than 3,500 gpm. The recommended fire flow protection in excess of 3,500 gpm becomes the responsibility of the property owner and impacts their individual insurance rates, not the community insurance rates. The property owner can take steps such as installing sprinkler systems to reduce the fire flow requirement of the property to less than 3,500 gpm.

3.2.3 Time Duration Requirements

In addition to setting recommended fire flow requirements, the ISO has established recommended time duration requirements during which the fire flow should be maintained. In general, fire flows up to 2,500 gpm should be available for two hours, while fire flows greater than 2,500 gpm should be maintained for three hours or more, depending on the flow. The ISO standards for time-duration for recommended flows are shown in Table 3-1.

TABLE 3-1
ISO FIRE FLOW DURATION RECOMMENDATIONS

Recommended Fire Flow (gpm)	Recommended Duration (hours)
2,500 and less	2
3,000	3
3,500	3
4,000 and greater	4

3.3 DEP Distribution System Requirements

3.3.1 General

The Commonwealth of Massachusetts DEP, Division of Water Supply, promulgates Massachusetts state regulations regarding water distribution system requirements as specified in Drinking Water Regulation 310 CMR 22. The DEP also specifies water distribution system guidelines in Guidelines and Policies for Public Water Systems.

3.3.2 Pressure Requirements

Drinking Water Regulation 310 CMR 22.19 promulgated by the DEP states that "all service connections shall have a minimum residual water pressure at street level of at least 20 psi under all design conditions of flow," including fire flows. This pressure is equivalent to 46 feet in elevation and will permit water to overcome the frictional resistance of house plumbing and rise to a height equivalent of about a three-story building.

Under average day conditions, DEP Guidelines recommend a normal working pressure in the distribution system of 60 psi and not less than 35 psi. The 35-psi minimum pressure allows a "factor of safety" so that a fire in the system will not reduce pressures elsewhere to below 20 psi. Drinking Water Regulation 310 CMR 22.19 has made these guidelines regulatory.

Although no regulation has been established, it is generally accepted that pressures at street level exceeding 110 psi are considered excessive and should not occur anywhere in the system. Such pressures result in rapid discharge of water at fixtures, thus wasted water, and also increase leakage throughout the system. Massachusetts plumbing code (248 CMR 2.14) requires pressure reducing valves in buildings where the pressure exceeds 80 psi, except where the water service pipe supplies water directly to a water pressure booster system, an elevated water gravity tank, or to pumps provided in connection with a hydropneumatic or elevated gravity water supply tank system.

3.3.3 Design Requirements

Chapter 9 of the DEP Guidelines addresses several requirements for distribution system design:

- Water-main diameter shall be designed to maintain the minimum residual pressure of 20 psi while providing adequate fire flow. Water mains designed to provide fire flow shall not be smaller than 8 inches in diameter.
- Fire flow design shall be in accordance with National Board of Fire Underwriters (NBFU) requirements.
- Fire hydrants shall not be provided on water mains not designed to provide fire flows. Hydrants should be located at street intersections and at spacings of 350 to 600 feet. The hydrant laterals shall have a valve and be a minimum of 6 inches in diameter.
- Valve spacing should not exceed 500 feet in commercial districts and 800 feet in other districts.
- Dead-ends should be minimized by looping of all water mains whenever practical.

3.3.4 Cross Connections

Drinking Water Regulation 310 CMR 22.22 promulgated by the DEP establishes regulatory requirements for cross-connection control. A cross connection is defined as an actual or potential connection between a potable and non-potable water supply. These connections constitute a serious public health hazard. Cross connections have been the cause of many well-documented public drinking water contaminations that resulted in the spread of diseases. The DEP requires that all actual and potential cross connections be protected through the use of approved backflow prevention devices.

The Town of Barnstable has administered a cross connection control program in order to protect the distribution system from potential hazards. However the 2005 Public Water System Annual Statistical Report states that all commercial, industrial, institutional, and municipal owned facilities have not been surveyed for cross-connections at least once. All surveying is expected to be completed as of December 31, 2006. In 2005 there were a total of 9 violations.

4.0 FIELD TESTING AND RESULTS

4.1 General

In order for a distribution system to provide adequate service, it must be able to meet demands during periods of peak consumption. The system must also be capable of delivering hydrant flows at adequate pressures for fire protection. Deterioration of the distribution system will affect the ability of the system to deliver these flows. Hydrant flow tests and C-value test results provide an indication of the distribution system's ability to meet these requirements.

The capacity of the system to deliver flows depends on the carrying capacities of the individual pipeline segments. A distribution system is comprised of many segments of pipe. An individual pipe segment's ability to transfer water, or its carrying capacity, is a function of several factors, including pipe inner diameter, length, material and inner surface condition. A significant number of pipelines with insufficient carrying capacity can seriously limit a distribution system's ability to deliver sufficient flows. The distribution system should be evaluated regularly to determine if there is sufficient capacity to meet peak demands imposed upon it, as well as fire flow availability. C-value tests and hydrant flow tests are utilized to assess distribution system capacity by determining individual pipe segment carrying capacity and fire flow capability.

C-value tests are a method to measure an individual pipeline's carrying capacity. The C-value is a coefficient used in the Hazen-Williams equation for determining the head loss from friction through a segment of pipe. The coefficient C is a function of the roughness of the internal surface of the pipe and is directly proportional to the carrying capacity of the pipe segment. It is generally accepted that the conservative C-value of a new cement-lined ductile iron pipe is approximately 120. Therefore, a test yielding a C-value of 60 indicates that the carrying capacity of that pipe has been reduced in half.

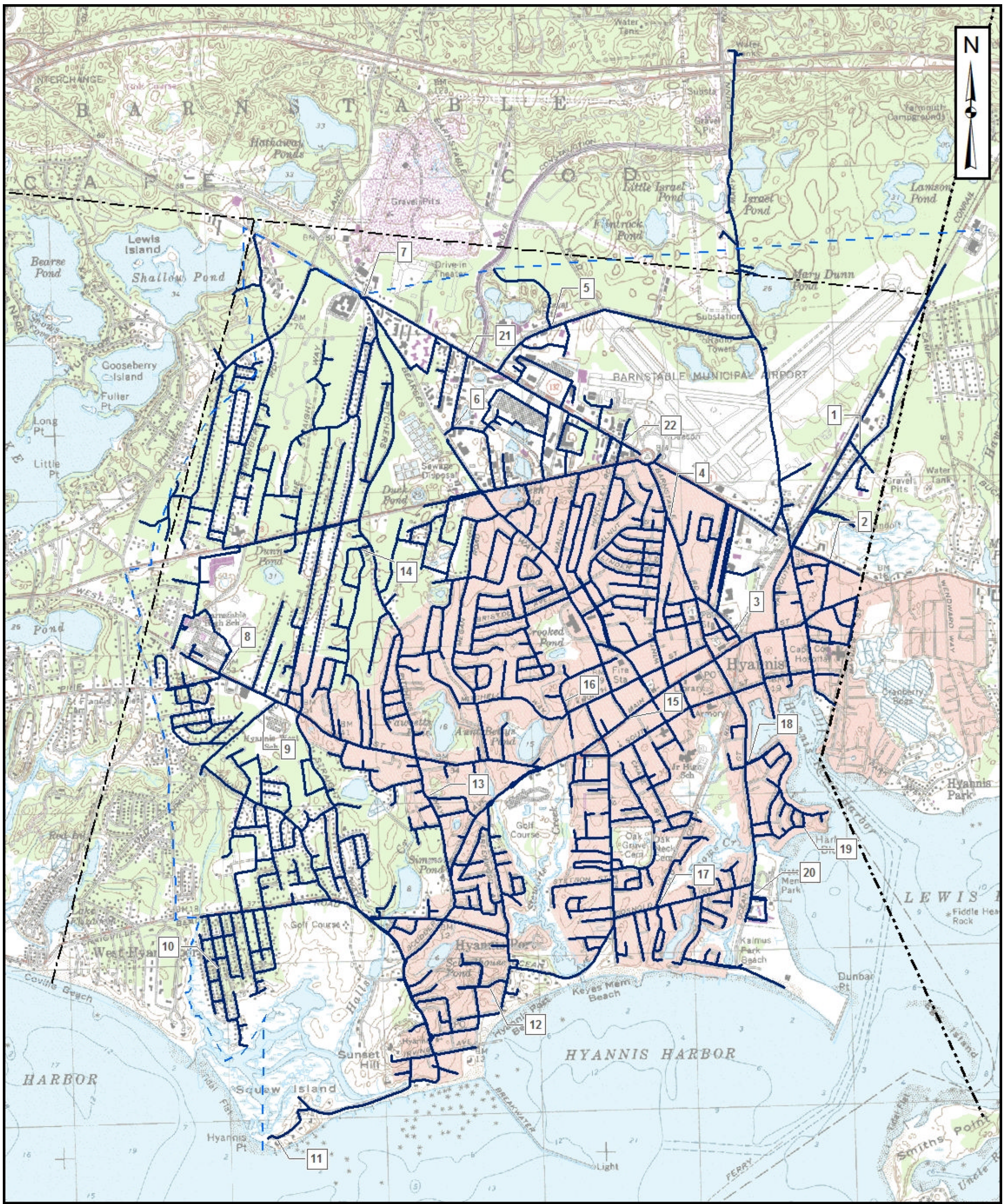
The primary cause for reduction of the C-value of a pipe is the corrosion and consequential deterioration of unprotected metal surfaces on the internal surface of the pipe. The internal corrosion increases the roughness of the interior wall of the pipe and forms a buildup called tuberculation, which in turn reduces the cross-sectional area of the pipe. These two effects of corrosion can reduce the C-value, or carrying capacity, of a pipe significantly. Cast iron pipe manufactured prior to and during the 1920s were generally constructed without internal cement linings and are therefore more susceptible to tuberculation. The exact year when the Hyannis Water System started using cement lined cast iron pipes remains unknown. However we estimate that Hyannis began using cement lined pipes in 1933 based on mapping received from the town.

It is generally accepted that when a pipeline's carrying capacity deteriorates to 50 percent of its original capacity, serious consideration should be given to replacement or rehabilitation of the pipeline to restore carrying capacities to near original levels. In addition to the seriously reduced carrying capacity, a pipeline at this stage has already built-up significant amounts of tuberculation on its inner surface. As the pipeline diameter decreases, the velocity of the water increases, which can result in the shearing/scouring of this material. This can yield significant water discoloration problems. As the pipeline ages, the deterioration of the inner surface gets worse and the problems become more severe.

The carrying capacity of a pipeline that is 100 years old can be reduced to 20 percent of its original capacity. At this point, the usefulness of the pipeline would be minimal, as the pipeline often cannot deliver peak demands or fire flows. For this reason, the useful lifetime of an unlined cast iron pipeline is generally accepted to be 75 to 100 years (although excessive tuberculation often occurs sooner, especially in 6- and 8-inch unlined pipes). Again, unlined pipes can significantly disrupt the ability of the distribution system to meet normal and peak system demands.

4.2 Hydrant Flow Tests

Hydrant flow testing and data from ISO Commercial Risk Services, Inc. fire flow testing was used during this study to evaluate the Hyannis Water System's ability to supply recommended fire flows and pressures. Figure 4-1 shows the locations of the ISO flow testing in Hyannis.



LEGEND

1 FLOWTEST LOCATIONS (#1-22)

FIGURE 4-1

FLOW TEST LOCATIONS

WATER DISTRIBUTION STUDY
HYANNIS, MA

0 2,800 5,600 Feet

Weston&Sampson

4.2.1 ISO Fire Flow Testing

The ISO conducted 22 hydrant flow tests in Hyannis in October 2006. Available and recommended fire flows as determined by the ISO are discussed in Chapter 3 and the test results are attached in Appendix A. It is apparent from the results of the ISO flow testing that, at the time of testing, the existing system did not provide flows adequate for fire protection in several areas of Hyannis. These locations are summarized in Table 4-1 and shown in Figure 4-1.

TABLE 4-1
LOCATIONS OF INSUFFICIENT ISO FIRE FLOWS (1)

Test Designation and Location	ISO Recommended Flow (gpm)	Flow Available at 20 psi (gpm)	Percent of Recommended Flow Available
1. Yarmouth Road @ Ferndoc Street	2000	1700	85
2. Iyannough Road @ Cedar Street	3000	2900	97
3. Ridgewood Avenue @ Center Street	4000/3500	2100	60(2)
4. Barnstable Road @ Baxter Road	5500/3500	2200	63(2)
7. Iyannough Road @ Bearses Way	5500	2800	51
8. West Main Street @ High School	5000/3000	2600	74(2)
9. Old Craigville Road @ Elementary School	3500	1800	51
18. Ocean Street @ Bay Street	5000/1500	1400	40(2)
20. Ocean Street @ Gosnold Street	3500	1100	31
22. Barnstable Road @ Hinckley Road	4000/3000	2700	77(2)

(1) As determined by ISO tests performed in Hyannis, October 2006.
(2) Based on recommended ISO flow of 3,500 gpm.

4.2.2 Weston & Sampson Flow Test Results

Weston & Sampson was present for the ISO Flow Tests conducted in October 2006. Using the data obtained at the hydrants, Weston & Sampson determined the flow available at 20 psi for all 22 flow test locations. The results of these tests are shown in table 4-2 and their locations are shown in Figure 4-1. Unlike the ISO test results as seen in Appendix A, Weston & Sampson corrected for the difference in elevations between the gage and the flow hydrants by shifting the static and residual pressures from the gage hydrant to the flow hydrant. Due to this, our flow available at 20 psi may differ from the ISO test results. Weston & Sampson also confirmed the gage calibration of the ISO gage at the first hydrant test location. During the October flow testing, only the Hyannisport, Simmons Pond, Mary Dunn #3, and Maher Treatment Plant wells were in service. Therefore the flow of water into the system was from the storage tanks and 4 well pumps during the testing periods. The levels of the storage tanks and the rates of the 4 wells in service during the testing are shown on the individual flow test data sheets in Appendix A. At the start of the flow testing, the storage tank levels were at approximately 97 feet. The Weston & Sampson flow test calculations were used to balance the computer model.

TABLE 4-2
WESTON & SAMPSON FIRE FLOW TESTS

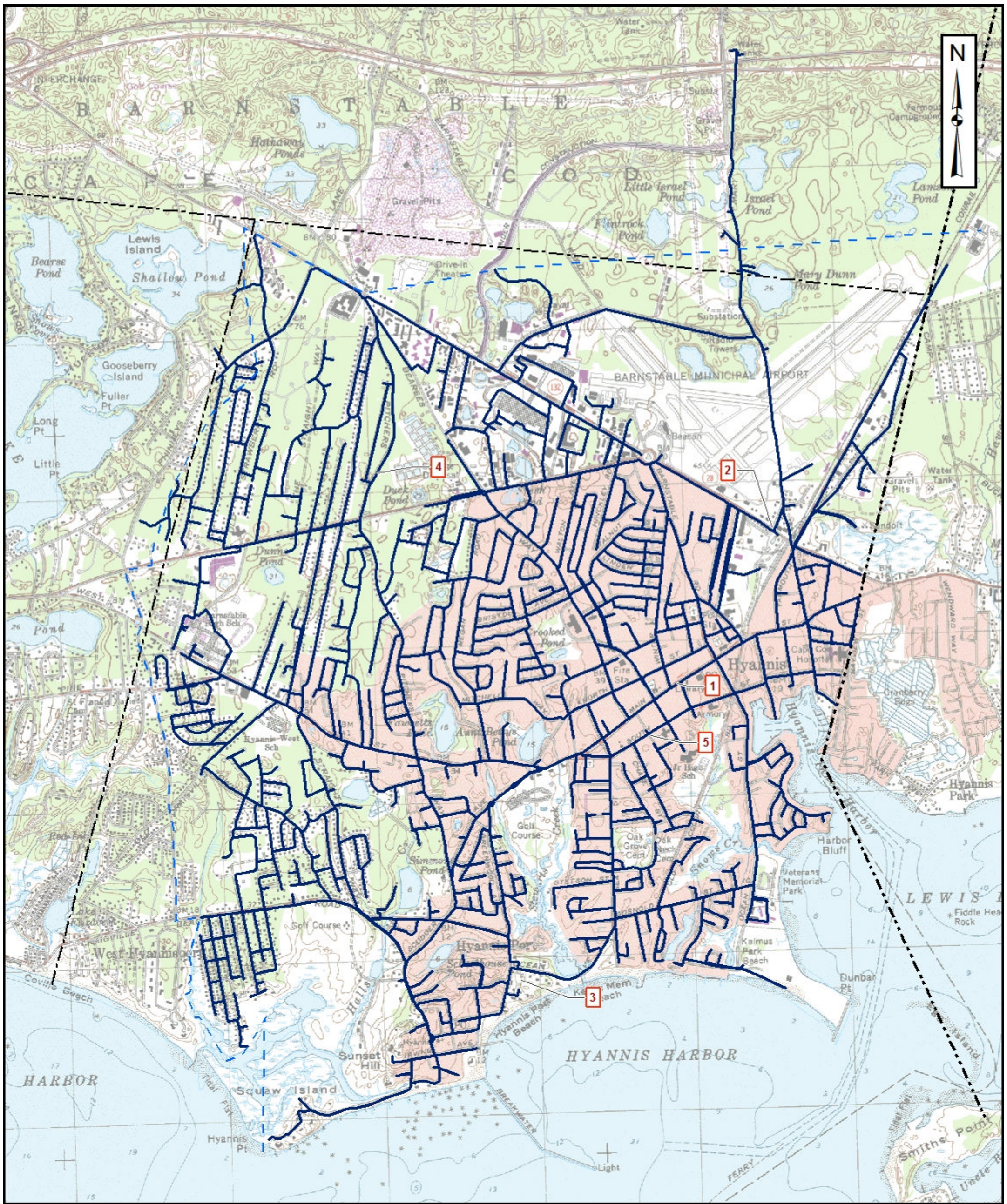
Test Designation and Location of Hydrant	Pipe Diam. (inches)	Test Discharge (gpm)	Available Flow @ 20 psi (gpm)
1. Yarmouth Road @ Ferndoc Street	6	1,300	1,720
2. Iyannough Road @ Cedar Street	6	1,240	2,420
3. Barnstable Road @ Hinckley Road	8	1,130	2,680
4. Attuck's Way @ Airport Road	20	2,720	4,330
5. Iyannough Road @ Festival Plaza	16	2,430	5,050
6. Iyannough Road @ Bearses Way	16	2,550	3,410
7. Corporation Street @ Enterprise Road	8	1,240	3,660
8. Barnstable Road @ Baxter Road	6	1,190	2,190
9. Ridgewood Avenue @ Center Street	6	1,540	2,450
10. Ocean Street @ Bay Street	8	1,125	1,370
11. Ocean Street @ Gosnold Street	6	840	1,130
12. Old Harbor Road @ Bay Shore Road	6	920	1,060
13. Old Colony Road @ Snow Creek Drive	8	1,060	1,570
14. Edgehill Road @ Mt. Vernon Avenue	6	800	1,170

Test Designation and Location of Hydrant	Pipe Diam. (inches)	Test Discharge (gpm)	Available Flow @ 20 psi (gpm)
15. Squaw Island Road	6	470	470
16. Main Street @ Bassett Lane	12	1,840	4,020
17. North Street @ Stevens Street	8	1,400	3,530
18. Pitchers Way @ Elizabeth Lane	8	1,210	3,550
19. Pitchers Way @ Wayland Road	10	2,370	3,640
20. West Main Street @ High School	10	2,430	3,070
21. Old Craigville Road @ Elementary School	8	1,720	2,220
22. Ocean Avenue @ Sixth Street	4	750	930

The results of the flow tests show that a few areas in Hyannis did not meet the ISO required flow at 20 psi. The flow test conducted on Squaw Island Road indicated that the available flow at 20 psi was only 470 gpm. This was the lowest available flow at 20 psi among the 22 flow tests. The tests conducted at both the High School and the Elementary School indicated that there is not enough flow available for adequate fire protection in those areas. Most residential areas of Hyannis did have adequate flows available for fire protection. However some of the more dense commercial areas of the village did not, such as Barnstable Road at Baxter Road and Ridgewood Avenue at Center Street.

4.3 C-Value Tests

WSE conducted five C-value tests on water main that was installed between approximately 1911 and 1973. The C-value test performed on the 12-inch water main in Main Street indicates that the pipe has a C-value of 49. The test performed on the 16-inch water main in Mary Dunn Road indicates that the pipe has a C-value of 70; and the test performed on the 12-inch water main in Ocean Avenue indicates that the pipe has a C-value of 80. One C-value test was conducted on AC pipe. This test was on Pitcher's Way at Beth Lane. The results indicated that this pipe has a C-value of 123. The final C-value test was conducted on a 6-inch, water main on South Street. This test confirmed that the pipe is unlined, and has a C-value of 42.



LEGEND

1 C-VALUE TEST LOCATIONS (#1-5)

FIGURE 4-2

C-VALUE TEST LOCATIONS

WATER DISTRIBUTION STUDY
HYANNIS, MA

0 2,800 5,600 Feet

Weston&Sampson

The sites in the distribution system that a C-value test was be performed were chosen due to their significance in the distribution system. Each of the five areas are key hydraulic links located between sources of supply and the areas of high water demand. By confirming the C-values of these key hydraulic paths and utilizing the results of the flow tests, we are able to successfully balance the computer hydraulic model. The locations of the five C-value tests are shown in Table 4-3 and Figure 4-2.

TABLE 4-3
WESTON & SAMPSON C-VALUE TESTS

Street Location & Test Designation	Water Main Size	Material	C-Value
Main Street @ Pearl Street	12"	CI	49
Mary Dunn Road @ Route 28	16"	CI	70
Ocean Avenue @ Hyannis Avenue	12"	CI	80
Pitcher's Way @ Beth Lane	10"	AC	123
South Street @ Pine Avenue	6"	CI	42

4.4 Conclusion

64-percent of the total footage of water mains in the Hyannis Water System are 6- and 8-inch water main. 18.7-percent of the water mains are unlined cast iron. The South Street C-value test indicates that the carrying capacity of Hyannis's small-diameter, 6- and 8-inch unlined cast-iron pipes, in particular, has been severely reduced by deterioration of the inner surface. There also appears to be difficulty in transporting the water efficiently from the Maher Treatment Plant area to the areas of high demand. By increasing the transmission capacity around the Maher Treatment Plant and Main Street area, providing alternate flow paths, and eliminating choke points in the system, the entire system will show an improvement in its ability to distribute water throughout the system. These items are discussed in detail in Chapter 6, Water Distribution System Deficiencies.

5.0 HYDRAULIC ANALYSIS

5.1 General

A hydraulic model of Hyannis's distribution system was developed to investigate the ability of the system to meet water demands during periods of peak consumption and fire flow demands. The model was created using the age (when available), material, and diameter of every active pipe segment throughout Hyannis's system. A computer-aided hydraulic analysis was performed on the distribution system using the hydraulic model. All pipes 2-inches and greater are included in the hydraulic model.

5.2 Hydraulic Analysis by Computer Modeling

The distribution system analysis utilized the computerized hydraulic analysis program H2ONet®. H20Net® version 6.1 is an AutoCAD add-in program developed by MWH Soft, Inc.

The distribution system was schematized into a system of pipe segments and pipe junctions (nodes). Each pipe segment and node was assigned an identification number. The hydraulic model developed for Hyannis contains approximately 2740 pipe segments and 2540 nodes. Information on pipe diameters, internal pipe condition (C-value), ground elevations, and water demands was collected and entered into the model database.

The H2ONet® program output includes: a summary of input data; headloss, flow, flow direction, and velocity for each pipe segment; and demand, pressure, and hydraulic grade line (HGL) at each node. The output data was reviewed for hydraulic deficiencies based upon predetermined criteria. The output data may be shown graphically using functions in H2ONet® that will build pressure, HGL and elevation contours onto the H2ONet® hydraulic map.

5.3 Hydraulic Modeling Data

The Hydraulic Model for the village of Hyannis was created by compiling many data sets into one usable model and checking for accuracy. Data sets included were the Boston University Graduate Student EPAnet data, Barnstable Water Co. Distribution maps: 1917, 1932, 1933, 1949, 1965, 1976, 1979, and Hyannis service cards. The digital data sets were combined into one by transferring the EPAnet material and year attributes for the pipes that had known values to their corresponding pipes on the CAD drawing. Missing material data in the distribution system was found using the old distribution maps. The Barnstable Water Co. distribution maps do not show material, therefore material had to be assumed from the install year of the pipe. All water pipes shown on the 1917, 1932 and 1933 distribution maps were assumed to be unlined cast iron. All water mains constructed between 1933 and 1976 were assumed to be Asbestos Concrete. Water mains that were constructed in 1979 or later were assumed to be PVC. To check for accuracy, the updated CAD drawing with defined material attributes for every pipe in the system was then compared with the service cards. The base map was imported into H2ONet® at full scale which allowed for pipe lengths to be calculated automatically.

To initiate the analysis process, updated data pertaining to the system characteristics was collected and incorporated into the water layer of Hyannis's GIS. Nodal elevations were obtained from the elevational information contained within MassGIS topographic mapping. Information on the type of water main and year installed was determined from the various mapping provided by the Barnstable Water Supply Division. Initial C-values were attributed to the pipe segments based on age and material of construction.

5.3.1 Water Demand Allocation

Hyannis's water demand was proportionally allocated among the model's junction nodes to simulate the varying demand that is seen across the distribution system. Each of the top 62 water users in the village of Hyannis were represented by their actual water demand through assignment of their demand to the nearest junction node in the hydraulic model. There are 7,345 service connections in the distribution system, each was assumed to be represented by a building in Hyannis. The remaining demands were summed and divided by the remaining number of buildings in Hyannis resulting in a demand of .216 gpm placed on the remaining 7,283 buildings in the model. The demands were automatically assigned to each junction node based on the building density in the area. The demands per building were combined with nearby buildings and the sum was placed on the closest corresponding node to the location of those buildings. As a result, the hydraulic model correctly represents the building density in the village through the nodal demands.

A first attempt was made to input all 7,345 individual user account water service connection demands using the data provided by the Water Supply Division. However, some of the parcel ID #'s from the consumption data did not match the parcel ID#'s in the GIS model that was provided by the Water Supply Division. Therefore we could not successfully input all 7,345 demands. In a second attempt, all the Tier II and Tier III user data was compiled representing a total of 217 top users. We attempted to input the consumption of each of these 217 users into the model, however, again many of the parcel ID #'s did not match the parcel ID #'s provided in the GIS model from the Water Supply Division. Next, the consumption data from the top 62 users, representing 15% of the total usage for Hyannis, were entered into the model. A few of the parcel ID#'s did not match, however we could search by the property address and determine which node to place the corresponding demand on. The remaining demands were summed and divided by the number of buildings in Hyannis. This average demand (excluding the top user demands) were placed on the remaining nodes in the model.

5.3.2 C-Value Assignment

C-value tests are a method to measure an individual pipeline's carrying capacity. The C-value is a coefficient used in the Hazen-Williams equation for determining the pressure drop through a segment of pipe. The coefficient C is a function of the roughness of the internal surface of the pipe and is directly proportional to the carrying capacity of the pipe segment. It is generally accepted that the C-value of a new cement-lined ductile iron pipe is approximately 120. Therefore, a pipe yielding a C-value of approximately 60 indicates that the carrying capacity of that pipe has been reduced by a factor of two.

The main cause of the reduction of the C-value of a pipeline is the corrosion and consequential deterioration of unprotected metal surfaces on the internal surface of the pipeline. The internal corrosion increases the roughness of the inner wall of the pipe and forms a buildup called tuberculation, which shrinks the cross-sectional area of the pipe. These two results of corrosion reduce the C-value or carrying capacity of a pipe significantly. Generally, unlined cast iron pipes manufactured during and before the 1920's do not have internal cement linings and are very susceptible to tuberculation.

The C-values were assigned for each pipe segment based on the age and material of the pipe from information received from the Barnstable Water Supply Division. The five materials used for the water mains in Hyannis are: unlined cast iron (CI); cement lined cast iron (CLCI); asbestos cement (AC); ductile iron (DI); and polyvinyl chloride (PVC). Table 5-1 shows the C-values used for the hydraulic model of the existing distribution system and the approximate years each type of water main was installed.

TABLE 5-1

C-VALUES USED IN HYDRAULIC MODEL OF EXISTING SYSTEM

DESCRIPTION	YEARS INSTALLED	C-VALUE
Unlined Cast Iron	1917-1932	Varied*
Cement Lined Cast Iron	unknown	105
Asbestos Cement	1933-1976	110
Ductile Iron	1979-present	115
Polyvinyl Chloride	1979- present	125

*See Figure 5-1 for Unlined Cast Iron C-Values

5.3.3 Wells & Tanks

The boundary conditions (operating conditions at the wells and tanks) during the fire hydrant flow tests were obtained from the Barnstable Water Supply Division. For calibrating the model, the boundary conditions at the wells and tanks represent the same flows and levels as that while performing the fire hydrant flow tests. A negative demand equivalent to the flow from a particular well was placed on a node representing the well in the model. The Hyannisport, Simmons Pond, Mary Dunn #3, and Maher Treatment Plant wells were in service during the fire hydrant flow tests.

For this analysis, the Mary Dunn Wells were represented by a single node, as was the Maher Treatment Plant. The wells were not modeled as pumps since well pump curve information could not be located within the Barnstable Water Supply Division's files.

Similarly, the tank levels also represent the same levels at the time of fire hydrant flow tests. The two tanks were modeled as cylindrical, flat-bottomed tanks using the actual tank capacity, overflow and land elevations.

5.4 Calibration of the Hydraulic Model

Calibration is the process of comparing field data to the modeling results and, if required, making changes to the parameters describing the system until the model-predicted results reasonably agree with field data under all operating conditions. The parameters required to be adjusted for calibration may include system demands, roughness coefficient of pipes (C-values), pump operating characteristics, and other model attributes that affect simulation results.

The model was calibrated by simulating the distribution system conditions using the twenty-two (22) fire hydrant flow tests performed by the ISO and as observed by Weston & Sampson. Results from the hydrant flow tests were simulated in the computer model, including static pressures, residual pressures, tank levels, pumping data and flows. The calibration of the computer model to simulate the hydrant flow test results required adjustment of individual pipe C-values. Pressure drops in the computer model mimic pressure drops in the distribution system measured during flow testing. If the pressure drops were not equal, adjustments were made to the computer model so that they were within 90% accuracy. The calibrated model results are shown in Table 5-1 and Figure 5-2.

TABLE 5-2

CALIBRATION RESULTS

Test Number	Location		Static Pressure psi	Residual Pressure psi	Pressure Drop psi
1	Yarmouth Road @ Ferndoc Street	Field	92	48	44
		Model	87	43	44
2	Iyannough Road @ Cedar Street	Field	86	72	14
		Model	88	74	14
3	Ridgewood Avenue @ Center Street	Field	85	58	27
		Model	83	54	29
4	Barnstable Road @ Baxter Road	Field	80	60	20
		Model	78	61	17
5	Attuck's Way @ Airport Road	Field	77	55	22
		Model	75	56	19
6	Corporation Street @ Cape Cod Mall	Field	79	71	8
		Model	76	66	10
7	Iyannough Road @ Bearses Way	Field	68	40	28
		Model	66	44	22
8	West Main Street @ High School	Field	75	40	35
		Model	74	41	33
9	Old Craigville Road @ Elementary School	Field	76	40	36
		Model	74	39	35
10	Ocean Avenue @ Sixth Street	Field	93	45	48
		Model	90	45	45
11	Squaw Island Road	Field	82	6	76
		Model	82	9	73
12	Edgehill Road @ Mt. Vernon Avenue	Field	85	53	32
		Model	84	50	34
13	Pitchers Way @ Elizabeth Lane	Field	82	72	10
		Model	82	70	12

Test Number	Location		Static Pressure psi	Residual Pressure psi	Pressure Drop psi
14	Pitchers Way @ Wayland Road	Field	74	50	24
		Model	74	49	25
15	Main Street @ Bassett Lane	Field	80	65	15
		Model	80	66	14
16	North Street @ Stevens Street	Field	87	75	12
		Model	87	74	13
17	Old Colony Road @ Snow Creek Drive	Field	88	54	34
		Model	88	51	37
18	Ocean Street @ Bay Street	Field	85	42	43
		Model	88	44	44
19	Old Harbor Road @ Bay Shore Road	Field	95	38	57
		Model	92	32	60
20	Ocean Street @ Gosnold Street	Field	95	50	45
		Model	92	53	39
21	Iyannough Road @ Festival Plaza	Field	80	65	15
		Model	77	63	14
22	Barnstable Road @ Hinckley Road	Field	70	60	10
		Model	71	64	7

Figure 5-1

C-VALUES FOR UNLINED CAST IRON WATER MAINS

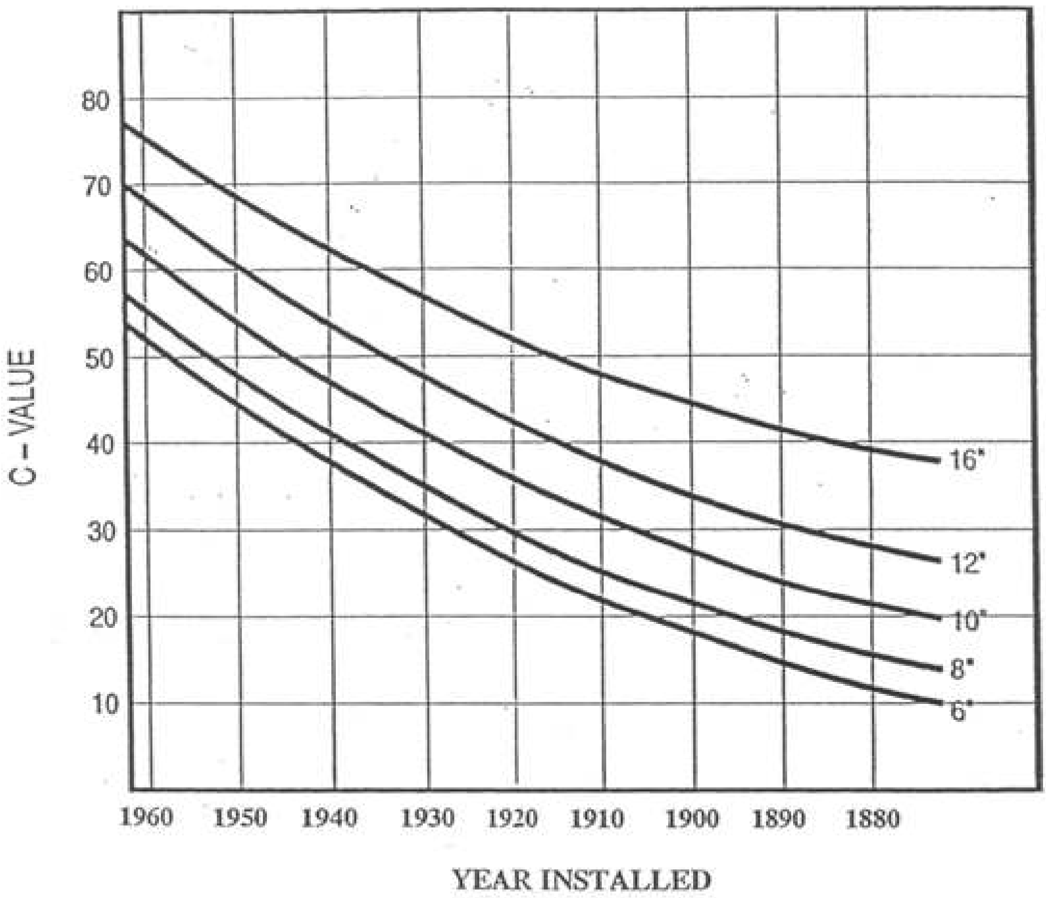
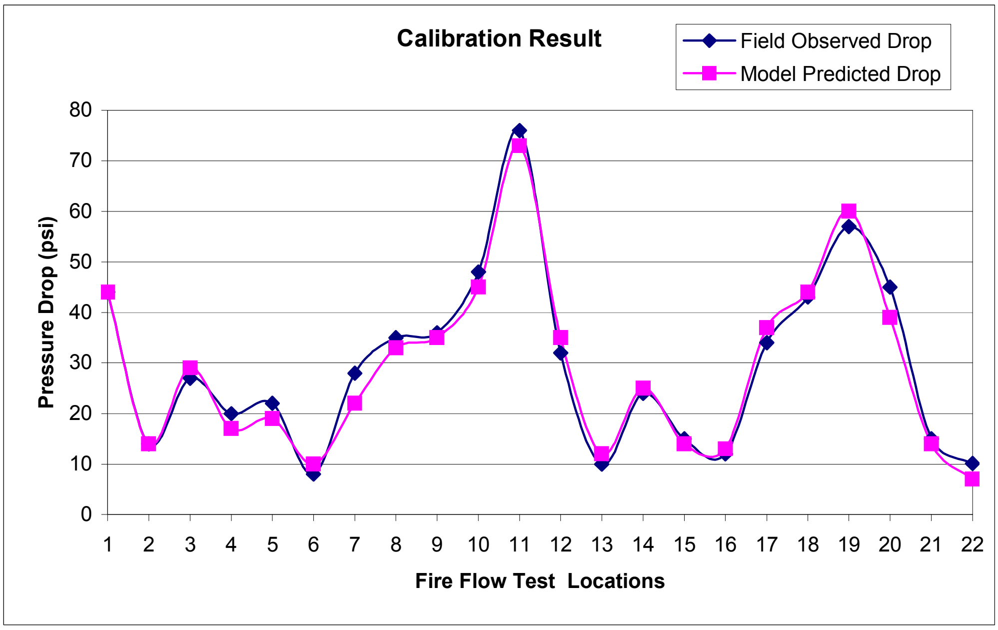


FIGURE 5-2

CALIBRATION RESULTS



There were two locations where simulations couldn't be performed by changing the C-values within acceptable limits. These locations are Ridgewood Avenue at Center Street (Test 3) and Old Craigville Road at Elementary School (Test 9). Trying various alternatives led to the possibility of a closed valve in the exiting distribution system. These locations were marked and a copy was given to the Barnstable Water Supply Division to verify the presence of a closed valve. Barnstable Water Supply Division inspected the locations and found no closed valves. However, these valves were kept closed in the model for calibration purpose only.

5.5 Adequacy of Existing Distribution System

To determine the adequacy of the existing distribution system, the system was analyzed under current and projected average day and maximum day water demands. The system was also analyzed with fire flow conditions under average day and maximum day demands. The overall results of the analysis show that the existing service system is able to provide a minimum of 35 psi to all customers on the distribution system under the current maximum day demand as well as the future (2030) maximum day demand. The existing distribution system is not able to provide sufficient fire flows as rated by the ISO to all customers. A total of 10 out of 22 locations were found to be below the required fire flow as per the ISO. A full synopsis of the adequacy of the water system is presented in Chapter 6.

5.5.1 Hydraulic Capacity Evaluation

The service system is capable of supplying adequate flow to meet the current 2.75 mgd average day demand and the current 5.22 mgd maximum day demand. Under present day average demand and maximum day demand conditions, the distribution system has adequate capacity to provide all areas with pressures greater than 35 psi.

Overall it was found that the eastern side of distribution system to be deficient in providing fire flows. The system has sufficient capacity to maintain a minimum pressure of 35 psi during maximum day conditions, but does not maintain the capacity to provide sufficient fire flows to all areas. Several of the areas this was exhibited include Ocean Street, South Street, North Street, Barnstable Road, and Winter Street. These aging water mains are unlined cast iron 6- and 8-inch diameter and the hydraulic capacity has therefore greatly diminished over time due to tuberculation. The ISO flow tests performed in October-November 2006 demonstrated the low fire flows area in the eastern side of the distribution system. See Chapter 6 for a more detailed account on this issue.

O:\Barnstable MA\2060316 Water Master Plan\Report\CHAP5.doc

6.0 WATER DISTRIBUTION SYSTEM DEFICIENCIES

6.1 General

The standard for identifying and prioritizing the improvements recommended in this chapter are based on the Barnstable Water Supply Division's goal of providing high quality water at adequate pressure to homes and businesses in Hyannis at reasonable cost, as well as providing the village with sufficient fire flows.

In this chapter, the distribution system is evaluated based on different types of deficiencies that exist in the present distribution system. As the system ages, these deficiencies will grow in magnitude and frequency and new problems may develop. This chapter includes recommended improvements to significantly reduce or eliminate these deficiencies. The Recommended Improvements map in Appendix C shows these recommendations graphically, with each pipe segment colored according to the recommended improvement. The recommended improvement as well as the estimated construction cost of each improvement is listed in Chapter 9.

General deficiencies addressed within the water distribution system evaluation fall into several categories including:

- small-diameter unlined mains
- transmission main deficiencies
- deficient fire flows
- parallel mains to be abandoned
- areas of frequent water main breaks
- dead end water mains
- water main obstructions
- pressure deficiencies
- supply deficiencies
- lead goosenecks
- vinyl lined asbestos cement water mains

A series of recommended maintenance practices is also presented in Chapter 10, which will help to keep the system in good working order.

6.2 Small Diameter Unlined Water Mains

Fourteen percent of the Hyannis Water System consists of pipes smaller than 6-inch diameter. The majority of these smaller diameter pipes are formed by 2-inch pipes. These 2-inch pipes being smaller in diameter are incapable of providing sufficient flow to the larger mains beyond, thereby causing deficiencies in the distribution system. One such example is the 2-inch pipe on Greenwood Avenue that is connected to 6-inch main at both ends forming a weak loop and acting almost as a dead end.

Locations causing similar problems are:

- Lantern Lane
- Fernwood Avenue
- Fourth Street (should be looped around Forest Street and Seventh Street)
- Potter Avenue
- Oak Street
- Chestnut Street
- Louis Street

The majority of the unlined cast iron mains are in the east side of the distribution system as this is the oldest section of the system. The unlined water mains, as a result of tuberculation, reduce the effectiveness of the distribution system to provide fire flows to the portions of the water main beyond the unlined sections. Unlined 6-inch and 8-inch diameter water mains are a problem for the same reason as large diameter unlined water mains. They reduce the fire flow capacity and cause water quality problems. The unlined cast iron water mains installed circa 1917 have approximate C-values of about 35, which indicates the mains have a carrying capacity one-fourth of its original capacity. To put this in perspective, an unlined 8-inch main has the capacity less than a new 2-inch main.

One such tuberculated main is the 6-inch main on South Street. This main provides water to all the homes, businesses, and municipal building along South Street. South Street consists of a 6-inch unlined cast iron water main installed circa 1917. Due to its age and material, it has tuberculated to an extent that restricts its hydraulic capacity since its initial installation. The C-Value Test performed on South Street at Pine Street revealed a C-Value of 40.

The locations with unlined cast iron main are:

- Ocean Street
- North Street
- Main Street
- Winter Street
- Barnstable Road
- Gosnold Street
- Center Street
- Yarmouth Road
- Spring Street
- Cedar Street
- Park Street
- Parkway Place
- Pleasant Street
- Lewis Bay Road
- School Street

Appendix C includes a map showing all the unlined cast iron water mains in the Hyannis Water System. It should be noted that the 6-inch diameter water main on Goose Point Road is shown unlined cast iron as per the Connecticut Water Map obtained from the Hyannis Water Supply Division. However, we suspect that this would only be accurate if the pipe was previously connected to Centerville Osterville Water System and later deeded to Hyannis. The area surrounding Goose Point Road was developed after the time unlined pipe was used and it is suspicious as the only unlined cast iron pipe in that location. However, we could not find information to document its pipe type and are relying on the original Connecticut Water mapping.

DEP Guidelines and industry standards state that the smallest new pipe that should be installed in a distribution system is 8-inch. Since it is generally not economical to clean and line 6- and 8-inch diameter pipe, it is more effective to replace them with new 8-inch water mains. It is, therefore, recommended that all unlined 4, 6, and 8-inch diameter water mains be replaced with 8-inch ductile iron water mains. It should be noted, however, that cleaning and lining is a possible option with regard to improving existing large diameter water mains and small diameter water mains where construction of a new main is too disruptive to traffic or construction costs are unreasonably high. In total, there are approximately 78,000 linear feet of unlined 2-inch, 4-inch, 6-inch, and 8-inch water main comprising 14-percent of the Hyannis Water System.

6.3 Transmission Main Deficiencies

The transmission system is the foundation for the entire distribution system and is comprised of the network of large diameter water mains. These mains provide water to the smaller diameter neighborhood pipes throughout the system. Since large diameter water mains provide the majority of water transfer throughout the distribution system, the maintenance, and when appropriate, improvement of all large diameter water main segments is of high priority to the overall effectiveness of the distribution system. The elimination of a few water main deficiencies within the transmission system can positively impact large areas of the distribution system. For this reason, these deficiencies should be addressed early in the improvement program. An analysis was done for the following two water supply scenarios under current maximum day demand:

1) Stressing North Water Supply Sources.
2) Stressing South Water Supply Sources.

Table 6-1 shows flow assumed at the supply sources for the Hyannis Water System. In scenario 1 the North supply sources were stressed and for scenario 2 the south supply sources were stressed with the tanks being full.

TABLE 6-1

SCENARIOS FOR WATER SUPPLY SOURCES

Supply Sources	Scenario 1 Flow (gpm)	Scenario 2 Flow (gpm)
Maher Treatment Plant	1500	1000
Mary Dunn Road Wells	1000	800
Straightway Well	500	800
Hyannisport Well	300	500
Simmons Pond	400	600

6.3.1 Stressing North Water Supply Sources:

The critical deficient water main in the Hyannis Water System is the 6-inch water main on Yarmouth Road and 8-inch water main on Maher Road off the Maher Treatment Plant. There is an 8-inch unlined cast iron pipe and a 12-inch AC pipe providing water into the distribution system from the treatment plant. The 8-inch unlined cast iron pipe installed circa 1917 is causing a headloss of 4.45 feet per 1000 feet at a flow of 180 gpm. The headloss through the 12-inch AC pipe is 5.44 feet per 1000 feet. Similarly, 6-inch unlined cast iron pipe on Yarmouth Road has a headloss per 1000 feet of 10.40 feet at a flow of 130 gpm. We recommend replacing the existing 8-inch main on Maher Road and 6-inch main on Yarmouth Road with a new 12-inch water main thereby reducing the headloss and increasing the flow. The improvement drops the headloss on Maher Road to 2 feet and increasing flow to 820 gpm. Also, the headloss through 12-inch AC pipe is reduced to 2 feet. On Yarmouth Road the headloss drops to 2 feet and flow is increased to 815 gpm. Since Maher Treatment Plant is operated most of the time throughout the year and has been supplying maximum of approximately 1500 gpm for the past 4 years, this improvement will tremendously help the distribution system by reducing the initial headloss and increasing the flow through 16-inch main on Camp Street.

Another deficient water main found during this hydraulic analysis is the 12-inch water main on Main Street, as it is the only large diameter water main carrying flow from the Maher Treatment Plant to the Northern, Central, and Southern parts of the water distribution system. The flow through Main Street is 655 gpm when Maher Treatment Plant is supplying 1500 gpm. It should be noted that Main Street is closer to Maher Treatment Plant and a major transmission main to the Downtown Area has less than 50 percent flow of that at the source. The majority of the flow enters the distribution system through Old Mary Dunn Road causing lower flows in Main Street and its adjacent streets. In order to increase the flow through Main Street, upgrading Main Street to a larger diameter pipe would be a good idea. However, Main Street being a busy street and has been recently reconstructed, we recommend replacing the 6-inch water main on Lewis Bay Road, South Street and Ocean Street (between Main Street and Channel Point) with a 12-inch water main connecting to Main Street at Ocean Street. This improvement results in a flow of 425 gpm through Main Street and 400 gpm through South Street. A flow increase of 170 gpm in the Downtown location under current maximum day demands condition. Also, South Street was found to have C-value of 40 during the C-value test, this improvement will eliminate a 6-inch unlined cast iron main and alleviate the low fire flow issues on South Street. Upgrading Ocean Street will further bolster the available fire flow that was found to be below the 2006 ISO requirement. This upgrade aids in providing more flows in the Downtown area adjacent to Main Street. This improvement also reduces flow through the Mary Dunn Road and forces more flow through the Main Street, and South Street.

6.3.2 Stressing South Water Supply Sources:

From the analysis of this scenario it was found that 6-inch water main on Scudder Avenue has a headloss per 1000 feet of 2 feet and flow of 52 gpm. We recommend replacing the existing 6-inch water main with a 12-inch water main (between Craigville Beach Road and Greenwood Avenue) and at the same time connect a 12-inch water main on Craigsville Beach Road to 6-inch water main on Smith Street. This improvement helps drop the headloss to 0.45 feet and increases the flow to 375 gpm on Scudder Road. Also, builds a strong 12-inch water main loop for providing easier path for water into the central part of the distribution system from the Hyannisport and Simons Pond wells and reducing the stress on Hyannis Avenue 12-inch water main and 2-inch main on Greenwood Avenue.

6.4 Comparative Analysis of the Distribution System: Pre- and Post-Improvements

6.4.1 General

This analysis will focus on the specific portions of the distribution system that exhibited sub-standard performance prior to the improvements (excessive headloss or velocities, restricted flows, low pressures, etc.) and where improvements were recommended, and compare those results to the hydraulic analyses performed after the recommended improvements.

Table 6-2 summarizes the results of the Hyannis Water System's hydraulic analysis before and after recommended improvements. Upgrading Maher Road and Yarmouth Road shows significant increase in flow and lower initial headloss. Lewis Bay Road and South Street increases flows in the Downtown Area. Although there isn't significant increase in flow on Ocean Street, the improvement helps increase the available fire flows in the Southeastern parts of the distribution system. The improvement on Scudder Avenue with a 12-inch water main and connecting to the 12-inch water main at Greenwood Avenue provides a strong loop in the water distribution system and reduces initial headloss for flow from Simmons Pond and Hyannisport wells.

TABLE 6-2

EXISTING AND POST-IMPROVEMENT CONDITIONS*

LOCATION	**EXISTING CONDITIONS**			**POST IMPROVEMENT**		
	Original Pipe Size (in)	Flow (gpm)	Headloss (ft/Kft)	Improved Pipe Size (in)	Flow (gpm)	Headloss (ft/Kft)
Maher Road from Treatment Plant to Yarmouth Road	8	180	4.45	12	818	2
Yarmouth Road from Maher Road to Camp Street	6	133	10.38	12	815	2
Lewis Bay Road from Camp Street to South Street	6	19	0.45	12	370	0.45
South Street from Lewis Bay Road to Sea Street	6	46	3.50	12	395	0.50
Ocean Street between Main Street and Channel Point Road	6	110	1.30	12	161	0.10
Scudder Avenue from Craigville Beach Road to Greenwood Avenue	6	52	0.45	12	363	0.45

* Current Maximum Day Demand Scenario

There is currently an 8-inch and a 12-inch water main on Maher Road from the Treatment Plant to Yarmouth Road. The improvement stated above refers to replacing the existing 8-inch main with a new 12-inch main so two 12-inch parallel mains will be in service. Similarly, two 12-inch parallel mains will be in service on Yarmouth Road from Maher Road to Camp Street. The recommendation stated above refers to replacing the existing 6-inch main with a new 12-inch water main on Yarmouth Road. The existing section of 16-inch water main on Lewis Bay Road is to remain when replacing the 6-inch water main with a new 12-inch water main from Camp Street to South Street.

6.5 Deficient Fire Flows

Deficient fire flows in the system are caused by pipes with inadequate carrying capacity (small-diameter or low C-values) that result in high pressure losses through the pipelines at high flows. Also, customers located at far end, presence of closed or partly closed valves, and inadequate pumps are reasons of low fire flows. These deficient fire flows can be increased by replacing unlined cast iron mains, increasing the size of major transmission mains and/or looping existing water mains into nearby water mains. Looping of existing water mains during the replacement program will increase the flow to the area by providing better circulation and multiple supplies to the particular segment of pipe.

Table 6-3 shows the deficient fire flow locations based off of the Weston and Sampson Fire Flow test calculations. A total of 10 out of 22 locations were found to have deficient fire flows in the Hyannis Water System. For example, the areas around the Barnstable Road did not meet the ISO required fire flows.

TABLE 6-3

DEFICIENT FIRE FLOW LOCATIONS

Test Designation and Location	ISO Recommended Flow (gpm)	Flow Available at 20 psi as per Weston & Sampson Flow test calculations (gpm)
1. Yarmouth Road @ Ferndoc Street	2000	1700
2. Iyannough Road @ Cedar Street	3000	2900
3. Ridgewood Avenue @ Center Street	3500/4000	2100
4. Barnstable Road @ Baxter Road	3500/5500	2200
9. Old Craigville Road @ Elementary School	3500	2200
10. Ocean Street @ Sixth Street	1000	950
11. Squaw Island Road	1250	474
18. Ocean Street @ Bay Street	1500/5000	1400
20. Ocean Street @ Gosnold Street	3500	1100
22. Barnstable Road @ Hinckley Road	3000/4000	2700

The calibrated model was used to identify the best feasible upgrade to increase the available fire flows in the location of deficient fire flows. The recommendations made in the transmission main deficiencies in addition to headloss also help increase available fire flows in 2 locations. These locations are Iyannough Road at Cedar Road, and Ocean Street at Bay Street.

Barnstable Road at Baxter Road and Ocean Street at Gosnold Street are also areas of deficient fire flows with fire flow requirement of 3500 gpm. We recommend upgrading both Barnstable Road and Gosnold Street to a 12-inch water main in order to increase the available fire flows in these locations. These upgrade increase the available flows by approximately 1000 gpm at the ISO fire flow test location. Upgrading Barnstable Road will also provide more flows on 2-inch mains on Chestnut Street, Oak Street, Elm Street, Maple Street, Linden Street, Mulberry Street and Cherry Road.

Another low available fire flow location is Ocean Drive at Sixth Avenue in Hyannisport. We recommend replacing the 2-inch water main on Forest Street with an 8-inch water main. Also, eliminating dead end water mains on Fourth Avenue, Fifth Avenue, and Forest Street as recommended in the Dead End Water Mains section will bolster the available fire flows in that location.

As per the flow test results, the Ridgewood Avenue at Center Street and Old Craigville Road at the Elementary School were found to have low fire flows. However, from our modeling analysis we suspect the presence of closed valves or other obstructions in those locations. These locations have been mentioned in the Water Main Obstruction section and need a thorough inspection or field test to identify the problem.

The three other locations of low fire flows are Squaw Island Road, Yarmouth Road at Ferndoc Street, and Iyannough Road at Festival Mall. They are a result of customers being located at the far end of and being serviced from a small diameter pipe. These locations are of low priority for upgrade at this moment.

6.6 Parallel Mains to be Abandoned

In Hyannis there are some old, unlined water mains that run parallel to newer cement lined or large diameter water mains which are still in service. These smaller, older mains are a detriment to the system for many reasons. They can collect debris and sediment because the larger parallel water mains carry the majority of the water flow. The extensive amount of tuberculation in the small diameter, unlined mains often results in poor water quality to the residents and plugged service to customers that are still connected to these older mains. These older mains are also difficult to flush because many cross connections exist to the parallel mains and because the hydrants can be connected to the large diameter mains. Chlorine residuals are harder to maintain and bacterial outbreaks can occur in the parallel mains. These older mains also tend to experience more leakage and breakage.

For these reasons, we recommend that these older mains be abandoned and all services and hydrants be transferred to the newer main in the street. Abandonment should include removing the tee at the connection with the newer main. The removal of the tee will eliminate water quality problems that may arise from stagnant water in a capped tee. A capped tee is also very susceptible to breakage during excavation at some future date. The overall effect of the abandonment procedure will be an improvement in water quality and simplified system maintenance. The following table presents the parallel mains to be abandoned. These improvements should be performed when funds are available but are considered a low priority compared to transmission main and fire flow deficiencies. We did not show the 8-inch mains parallel to a larger sized main since many of them were supplementing the larger sized mains ability to pass water through the particular area where they were located.

TABLE 6-4

PARALLEL WATER MAINS TO BE ABANDONED

STREET	MAIN DIAMETER	LENGTH
Spring Street	2”	1800’
Ridgewood Avenue	2”	1100’
Lincoln Road	2”	850’
Quail Lane	2”	1000’
Hawes Avenue	2”	650’
Bacon Road	2”	1600’
Falmouth Road between Bearses Way and Garden Lane	2”	700’
Nightingale Lane	2”	600’
Woodland Avenue	2”	500’
Pine Street	2”	600’
Channel Point Road	2”	600’
Scudder Avenue	6”	1800’

6.7 Areas of Frequent Water Main Breaks

The Water Supply Division has kept record of the locations where water main breaks have occurred due to sewer construction. These locations are on Barnstable Road, South Street, Center Street, and Bearses Way. Water mains with multiple breaks are considered to have a high priority for replacement because valuable resources are used during a break, including lost water, water main shutdowns and repair costs, and street repair costs.

6.8 Dead-End Water Mains

There are three types of water main dead-ends: dead-end roads, unrepaired broken pipelines, and dead-ends formed by a division between service systems (i.e. closed valves). Water will stagnate in these mains because the demands are usually small and there is little or no circulation through the mains. This will often result in poor water quality. It is recommended that, where possible, these mains be looped into a nearby main to promote circulation in the main. When looping of the water main is not possible, a 1-inch blow off should be constructed at the end of the water main and brought to the surface.

6.8.1 Dead-End Road Mains

There are some dead-end roads in Hyannis resulting from the geography of the village and its proximity to the ocean. For example, the 6-inch water main on Squaw Island Road is a dead end because of its location on an island and was constructed to reach development on the island. It cannot be connected to any other water main in the system without a high cost.

Many of the roads with dead-end mains are located adjacent to another street and can be looped. The 6-inch water main on Bearses Way can be connected to Corporation Street by adding 100 feet of new water main. Similarly, replacing the 2-inch water main on Highland Street with 8-inch water main connecting to Cook Circle in order to eliminate dead ends and create a loop. Weston & Sampson recommends a new 8-inch water main on Forest Street from Third Avenue to Sixth Avenue to eliminate 2-inch dead ends and bolster the available fire flows in that area.

Some dead end mains are relatively short in length and are not located adjacent to another street and therefore cannot be looped. The Barnstable Water Supply Division should maintain a list of the complaints from dead-end water mains to evaluate which mains pose a threat to the quality of water and require frequent flushing.

6.8.2 Water Main Breaks

When there are broken pipelines which have not been repaired, they create dead-ends on both sides of the break. These breaks should be repaired to provide circulation through the main. The typical type of pipe breaks that are not repaired occur at railroads or bridges and will, therefore, be expensive to repair. Hyannis currently has one broken water main on Harvard Street at Pine Avenue. This is a 2-inch water main and should be repaired as soon as possible.

6.8.3 Service System Dead-Ends

In areas where the service systems border each other, common practice is to close a valve on the single pipeline traversing the boundary. This often results in two dead-ends, causing poor water quality. Since Hyannis only has one service system, there are no service system boundary dead-ends present.

Weston & Sampson evaluated the Hyannis Water System for possible locations to construct new water main that will loop the distribution system to eliminate dead-end water mains (Table 6-5). Construction of these water mains is not considered a high priority at this time.

TABLE 6-5
WATER MAINS TO LOOP DISTRIBUTION SYSTEM

Street Name	Feet	Size
Highland Street to Cook Circle	250	8”
Forest Street	650	8”
Nightingale Lane	90	8”
Bearses Way	100	8”
Route 28 to Route 132 at Rotary	140	8”
Route 28 at Engine House Road to Old Mary Dunn Road	200	12”
TOTAL	**1430**	

6.9 Water Main Obstructions

Water main obstructions may be due to closed valves, debris such as a rock lodged within the pipe, or the accumulation of air. Performing an annual town-wide flushing program will help identify problematic areas where obstructions have developed. The Water Supply Division should also compile a list of closed broken valves in the distribution system. These valves restrict flow and cause pressure losses through the system. The Water Supply Division staff should fix these valves at times convenient to them. Due to discrepancy between the Weston & Sampson C-value and flow tests data and the model results, we suspected water main obstructions (closed valve) locations in the water distribution system. The suspected closed valve locations are Center Street at Main Street and

Old Craigville Road at West Main Street. These locations were marked and given to the Barnstable Water Supply Division for field inspection. However, no closed valves were found during field inspection by Barnstable Water Supply Division personnel. We recommend a thorough inspection of these locations so as to locate any closed valves or valves opening right, as opposed to the Hyannis Water System standard of opening left.

6.10 Pressure Deficiencies

DEP requirements for distribution system pressures include a minimum working pressure of 35 psi measured at street level. DEP also recommends an average working pressure of 60 psi measured at street level. Although there are no regulatory requirements regarding the upper limit for distribution system pressures, it is generally accepted that a pressure at street level of 110 psi should be the highest pressure allowed. There are no areas in Hyannis that are known to have insufficient pressures.

All of the static pressures in the service system remain at or above 35 psi during average and maximum day demands, including the areas of higher elevations. Analysis of future maximum day demands also showed pressures to remain at or above 35 psi in all areas of Hyannis. Under present maximum day conditions, pressures in Hyannis range from 41 psi to 90 psi. The lowest pressure in the system is located on Phinney's Lane where the pressure can drop to 41 psi during a maximum day event. This is located where the highest elevations in the distribution system are, in the northwest section of Hyannis. No houses above 78 feet are currently connected to the system.

6.11 Supply Deficiencies

The current maximum day demand and the future (year 2030) maximum day demand as estimated by Weston & Sampson are 3700 gpm and 5200 gpm respectively. Table 6-5 shows the current maximum pumping capacity for pumps at all water supply sources for the Hyannis Water System. It is evident from the Table 6-6 that the total pumping capacity is higher than that the current and future water supply demand and therefore we don't anticipate water supply deficiencies for Hyannis Water System. A standard set forth by the DEP states that Hyannis must be able to meet maximum daily demands with the largest pump being out of service. Hyannis currently meets this standard since the current maximum daily demand is approximately 3,620 gpm.

TABLE 6-6

PUMPING CAPACITY AT SUPPLY SOURCES

Well No.	Safe Yield (gpm)	Pumping Capacity (gpm)
Maher Well No. 1	972	771
Maher Well No. 2	869	548
Maher Well No. 3	663	608
Mary Dunn Well No. 1	534	400
Mary Dunn Well No. 2	567	472
Mary Dunn Well No. 3	442	500
Mary Dunn Well No. 4	296	500
Airport Well	459	770
Simmons Pond Well	792	442
Hyannisport Well	715	517
Straightway Well No. 1	*	*
Straightway Well No. 2	1530	704
Total		**6,232**

* Well is not in use

6.12 Lead Goosenecks

Lead goosenecks are flexible connecting pipes that were installed between the water main and all iron water services. Galvanized iron and cement lined iron water services were standard in Hyannis for all services early in the twentieth century. Because of the extensive corrosion of iron services, their estimated life is 50 years. Most iron water services in Hyannis have already been replaced with copper services. The remaining iron services and lead gooseneck services should be identified and replaced in future water main replacement projects.

6.13 Vinyl Lined Asbestos Cement Water Mains

In the late 1960s, vinyl lined asbestos cement (VLAC) pipe was developed because conventional asbestos-cement pipe was found to produce high alkalinity and poor-tasting water. VLAC pipe was manufactured by thinning a resin with tetrachloroethylene (PCE) and then spraying the mixture onto the inside surface of asbestos-cement pipe. In the late 1970s, it was discovered that VLAC pipe was capable of leaching PCE into the water carried by the pipes. This problem was found to be most pronounced in pipes that are flushed infrequently (e.g., dead end or low-flow pipes). Following this discovery, the manufacture of VLAC pipe ceased in 1980. In 1980, the Massachusetts Department of Environmental Quality Engineering (now the Department of Environmental Protection or

MADEP) made recommendations to public water suppliers in Massachusetts about how to control PCE concentrations in VLAC pipes. The most effective, but most expensive, solution to the problem was to replace the VLAC pipes entirely. If replacement was not possible, a number of other measures could be implemented including installing bleeders on dead end pipes to increase water flow and looping dead end water mains. The PCE concentrations in VLAC pipes should decrease over time as more of the PCE is depleted from the lining and should be highest during seasons of low water demand (e.g., during the winter). Due to the age of the VLAC pipe in the Hyannis water system, the presence of PCE is most likely not a current safety concern. The Water Supply Division should replace all VLAC pipe as shown in Table 9-3, Phase B Improvements, after the Phase A improvements have been made.

O:\BARNSTABLE MA\WATER MASTER PLAN\REPORT\CHAP6.DOC

7.0 DISTRIBUTION STORAGE EVALUATION

7.1 General

The purpose of this section is to assess the distribution system storage with respect to Hyannis's present and future storage requirements and to identify any deficiencies that might exist. Information on the existing system storage facilities, pump stations, and HGL were presented in Chapter 1.

Physical inspections of the Mary Dunn No.1 and Mary Dunn No. 2 storage tanks were conducted in October 2001 to determine their condition and the extent of rehabilitation required. As a result of the inspections, both tanks had their outlets screened and overflow pipe replaced. The Mary Dunn No. 1 tank was last painted in November 1996. Emergency repairs were made on the Mary Dunn No. 1 storage tank in 2002 following a water quality test, which indicated the presence of *Escherichia coli (E. coli)* bacteria. Roof repairs were also completed on this tank in the summer of 2003. The Mary Dunn No. 2 storage tank was repainted in October/November of 2002 and sediment was also removed.

7.2 Service System Storage Requirements

7.2.1 General

Distribution storage is provided to meet peak demands of short duration, minimize pressure fluctuations during periods of demand changes in the distribution system, and furnish a reserve for fire fighting. Storage may also serve to provide an emergency supply in case of temporary breakdown of pumping facilities. Equalization, fire, and emergency storage are typically allocated at specific levels within a storage facility to ensure the storage volume will be available at a hydraulic gradient adequate for the intended purpose. Equalization storage is provided within the top portion of the tank, with fire storage positioned immediately below. Emergency storage is located within the lowest portion of the tank.

Hyannis's distribution storage has a combined capacity of 1.370 MG. This is the total storage capacity that will be used in determining the adequacy of the storage facilities. The following presents an analysis of the distribution storage of Hyannis.

TABLE 7-1
TANK OVERFLOW ELEVATIONS AND DIMENSIONS

Storage Facility	Overflow Elevation (ft)	Capacity (gal)	Height (ft)	Diameter (ft)	Capacity per Foot of Height (gal)
Mary Dunn No. 1 Storage Tank	223	362,768	98	25	3,672
Mary Dunn No. 2 Storage Tank	223	1,023,875	98	42	10,363

7.2.2 Equalization Storage

The AWWA Manual of Water Supply Practices - Distribution Network Analysis for Water Utilities, Manual M32, offers guidelines on the amount of equalization storage required for a municipality stating it is a function of the pumping capacity, distribution piping capacity, and system demand characteristics. Manual M32 states that equalization storage makes up 20 to 25 percent of the average day demand, although these percentages are not recommended as rule-of-thumb design criteria. While equalization storage of 25 percent of the average day demand is acceptable for communities with large total water use and significant commercial and industrial demands, Weston & Sampson recommends a primarily residential community of Hyannis's size provide equalization storage of at least 25-percent of the maximum day demand for the area served by the tank. This will provide additional storage for the peak demands that arise from uses such as lawn irrigation, pool filling, etc.

The maximum day demand for Hyannis is currently 5.22 mgd and was estimated to be 7.49 mgd by the year 2030. When a factor of 25-percent is applied, the current volume recommended for equalization storage is approximately 1.3 MG and is estimated to increase to 1.9 MG in 2030.

Although the existing storage facilities for the service system have a combined capacity of 1.370 MG, all of this storage cannot be considered usable for daily operations. According to the DEP

Guidelines, a minimum pressure of 35 psi (81 feet) should be provided to customers under normal demand conditions. Thus, only the volume of water within a tank that will provide a pressure of 35 psi to the highest house elevation can be considered usable as equalization storage. A review of the USGS map for the village of Hyannis indicates the highest house elevation served in the system is approximately 78 feet (USGS). This elevation is obtained on Phinney's Lane.

The volume required for equalization storage would need to be provided above an elevation of 159 feet (78 feet + 81 feet). Both Mary Dunn tanks have an overflow elevation at 223 feet. Therefore, each tank has an available equalization storage height of approximately 64 feet (223 feet – 159 feet)[1]. The total volume of equalization storage available is 900,335 gallons. This amount is 404,665 gallons less than the recommended equalization storage of at least 25-percent of the maximum day demand. Based on the required equalization storage estimated using future water demands, a deficiency of 1.0 MG will exist by the year 2030.

7.2.3 Fire Storage

Municipalities are responsible to provide up to a maximum fire flow of 3,500 gpm for a 3-hour duration, while maintaining pressures above 20 psi throughout the distribution system. Any fire flow requirement above 3,500 gpm is the responsibility of the owner of the establishment, although a municipality could decide to provide additional fire flow capacity beyond 3,500 gpm to attract more business development. Based on a maximum fire flow of 3,500 gpm at duration of 3 hours, a minimum volume of approximately 0.63 MG would be required.

From the DEP Guidelines, usable fire storage is defined as the amount of water within a storage tank that will provide a pressure of 20 psi (46 feet) to the highest house elevation in the system. For a highest house elevation previously noted of 78 feet (USGS), the volume of water required for fire storage would need to be provided above an elevation of 124 feet (78 feet + 46 feet) to be considered usable. Based on the tanks' diameter and overflow elevation of 223 feet (USGS), approximately

1 DEP Guidelines state the maximum variation between high and low levels in storage structures providing pressure to a distribution system should not exceed 30-feet.

486,307 gallons is available as usable fire storage. This is 143,693 gallons less than the 0.63 MG required for fire protection.

7.2.4 Emergency Storage

Any storage provided within a tank below the elevation required to maintain the 20 psi pressure for fire storage is considered emergency storage, and would be used for water main breaks, equipment failures, or raw water contamination. The volume required is a function of risk and the desired system dependability with respect to an interruption of supply and is typically estimated as a percentage of the combined equalization and fire storage volumes. Typically, up to 2 days of average day demand is recommended. Since the base elevation of both tanks is at 125 feet and all usable storage above 124 feet is used for either equalization or fire storage, there is no volume of emergency storage available. The current recommended volume of emergency storage based on 2 days of average day demand is 5.51 MG. In 2030 the recommended emergency storage will be 7.5 MG. The Hyannis Water System will remain deficient in emergency storage unless another tank is constructed in Hyannis.

It should be noted that providing alternative power sources at the well sites may reduce the total amount of emergency storage required, but would not prove useful in the event of a raw water contamination occurrence.

TABLE 7-2
PRESENT AND FUTURE STORAGE AVAILABILITY (MG)

TYPE OF STORAGE	STORAGE AVAILABLE	CURRENT NEEDS (YEAR 2006)	FUTURE NEEDS (YEAR 2030)
EQUALIZATION	0.90	1.31	1.87
FIRE	0.49	0.63	0.63
EMERGENCY	0.00	5.51[1]	7.50[1]
TOTAL	**1.39**	**7.45**	**10.00**

(1) Represents 2 days of average day demand, which is recommended as the necessary volume of emergency storage.

7.2.5 Conclusion

The results of the analysis indicate Hyannis can adequately meet the DEP Guidelines for water pressure under all flow conditions. However, there is not enough storage available to meet the equalization or fire storage needs, and there is no emergency storage available within the storage tanks. As the average and maximum day demands increase with time, the storage deficiency will increase. Therefore Weston & Sampson recommends that a new storage tank be constructed in the Hyannis Water System. The construction of a new tank will substantially strengthen Hyannis's distribution storage capabilities. A new storage tank should ideally be located at a higher elevation and relatively close to downtown in order to reduce pressure fluctuations seen throughout Hyannis. The location of the existing tanks is hydraulically remote and far from the locations of most water demand. An exact location for a new storage tank should be determined through further study.

7.3 Storage Facility Inspection Findings

7.3.1 General

Both storage tanks were last inspected in October of 2001 by Extech, LLC and have had their outlets screened and overflow pipe replaced following the inspection. The following presents a summary of the recommended improvements for the tanks.

7.3.2 Mary Dunn No. 1 Storage Tank

The Mary Dunn No. 1 storage tank was found in good structural condition. This tank was last painted in November 1996 and emergency repairs were made in 2002 following a water quality test, which indicated the presence of E. coli bacteria. Weston & Sampson recommends that this tank be drained in the fall of 2007, when demands are low, and inspected. It is also recommended to clean the tank as 3" to 5" of sediment has accumulated at the bottom of the tank.

7.3.3 Mary Dunn No. 2 Storage Tank

The Mary Dunn No. 2 storage tank was found in good structural condition. The inspection did find minor mildew and algae on the lower exterior rings of the tank. The coating was found to be in good condition and the concrete ringwall had no cracking or spalling. Minor corrosion was found on the anchor rods and on the interior weld seams. An ASTM adhesion test on the roof showed that the coating had failed down to substrate. The exterior of the tank was coated and the interior roof was painted in 1997. The interior shell is currently in poor condition and may need to be painted. Sediment in this tank was removed and the interior was painted in October/November of 2002. This tank should also be drained in the fall of 2007 in order to be inspected and cleaned

7.4 Conclusions

The DEP Guidelines mandate a minimum of 35 psi to all customers, empowered by 310 CMR 22.00. The evaluation of the system storage capabilities indicates that the existing two storage tanks adequately serve the area with regard to water pressure. However there is not enough equalization or fire flow storage provided with the existing tanks and no emergency storage provided. A new tank should be constructed in Hyannis in order to provide sufficient storage in all categories. A sufficient location for a new storage tank can be determined through further study. Considerations for locating a new tank should include elevation, distance from the existing tanks, and proximity to areas of high water demand.

The two existing tanks are in good structural condition, however each have a typical usable life span of about 100 years. The Mary Dunn No. 1 storage tank is a riveted tank approaching the 100 year mark and serious consideration should be given to construction of a new tank that will better meet the future storage needs of Hyannis. Chapter 9 details the recommended improvements program and the approximate years each improvement should be constructed.

O:\BARNSTABLE MA\2060316 WATER MASTER PLAN\REPORT\CHAP7.DOC

8.0 EVALUATION OF EXISTING FACILITIES

8.1 General

The following is an evaluation of the existing facilities that are part of the Hyannis Water System. The evaluation is primarily based on the insight of Weston & Sampson Services personnel. The evaluation includes all well buildings, treatment facilities, and storage tanks. Recommendations are also provided on how to improve upon each facility, if warranted.

8.2 Supervisory Control and Data Acquisition, (SCADA)

The Telekey SCADA software is a DOS based system located at the water department office and is at least 20 years old. The system was recently upgraded and contains backup capabilities to prevent loss of historical data.

The current SCADA system is physically large and contains outdated technology. As compared to the newer technology this unit has limited capabilities. A large uninterrupted power source (UPS) battery system is located in a back room and serves as the backup power supply for the system. The UPS runs for a half hour then a 10KW generator is used for backup power. The SCADA system must be run by on-site operation only and cannot be controlled remotely.

While the current SCADA System is operable and does function on a daily basis, the system is proprietary by design. Specific components and/or programs contained within the system remain the property of the manufacturer. Obtaining technical assistance for repair or operation can be very limited and will have increased limitation in the future. The same problem will apply to obtaining various system components. The current system has exceeded the anticipated life span and has been modified to maintain operation.

A new SCADA system would require less room and maintenance. The newer programs are significantly improved in function and efficiency. They can be configured to provide countless control strategies and have better data acquisition and superior historical archiving capabilities. We recommend that a new SCADA system upgrade be designed and installed. The new upgrade should have web based remote access and a modular system.

8.3 Office Building

The building is metal sandwich panel construction and has a limited lifespan. It is an adequate facility at present, however, it should be replaced by a longer lived building when the existing building starts to show signs of failing. The office area appears to be very functional with adequate space, although additional space could be made available through a reconfiguration of the existing placement of current office equipment. A change in the current SCADA system, (discussed above), can also potentially provide more room. The office space is well lit and appears to provide a relatively comfortable environment with adequate heat and air conditioning.

Currently the office does not have a backup electrical source. Typically during a catastrophic event the office will become the Operations Control Center to control and monitor emergency operations. Without an emergency power source, this may become impossible. The office contains many computers that rely on electricity. An emergency generator will minimize their failure as well.

The Maher Treatment Facility contains three standby generators. The two smaller generators provide emergency power to the wells and the larger generator provides emergency power to the treatment facility. Typically emergency power systems are greatly oversized and their full potential is not used. There may be a possibility of utilizing one of these power sources to provide emergency power to the office. If this is not a realistic way to provide emergency power to the office due to back feed issues, it is our recommendation that an alternate power source be designed and installed.

8.4 Maher Well #1

The building appears to be in good condition. The electrical system also appears to be in good condition. The Well was re-developed and the pump rebuilt in 2006 resulting in a 100% increase in the well's yield. The motor was replaced in 2005 due to a lightning strike. The overall condition of this facility is very good. Hyannis may consider adding an HAVC system to the facility including a heater and dehumidifier.

When the Maher Treatment Plant was constructed, the Maher wells were no longer required to pump into the distribution system. The existing Maher wells only need to have sufficient head to pump water over the air stripper. Each Maher well should be evaluated to determine if pump bowls may be removed and the motor can be downsized in order to save energy.

8.5 Maher Well #2

The well area is protected by a chain link fence and is covered with fencing material on the roof as well. Inside the fence is a sample tap, main electrical disconnect circuit breaker, and a well level sensor. The pump is small and is suspended within the well with a pitless adaptor. The well appears to be operating as designed. As described in Section 8.4, the bowls on this pump can also be removed and the motor downsized in order to conserve energy.

The brush and grass should be cut in the fenced interior and approximately two feet away from the outer perimeter of the fence. This practice insures accessibility to the well head and protects the metal fence from accelerated deterioration.

8.6 Maher Well #3

This well is the same as Maher Well #2 with regards to equipment and set-up. The pump is small and is suspended within the well with a pitless adaptor. The well appears to be operating as designed.

The brush and grass near this well should also be cut in the fenced interior and approximately two feet away from the outer perimeter of the fence. This practice insures accessibility to the well head and protects the metal fence from accelerated deterioration.

8.7 Maher Treatment Facility

This facility appears to be in very good condition. The Motor Control Center and electrical supply has been modified and well maintained. The emergency power generators for the treatment facilities and well fields are contained within this facility. Once again, they are well maintained and neat. The emergency electricity transfer system is automated and initiates automatic exercising of the generator and transfer of the power source from the normal utility power to the emergency power source.

Lighting throughout the facility is very good. Weston & Sampson did note that there were several areas that are void of convenience electrical outlets. We did not observe if any of the electrical outlets had ground fault protection.

The chemical feed systems appeared to be in very good condition. The large bulk tanks are protected from sun and weather as well as accidental punctures. The containment area exceeds the 110 % of the maximum chemical volume that is stored. All chemical piping is supported properly and in some instances is sheathed to protect employees from chemical leaks. Due to gasification concerns, the chlorine storage should be moved to the treatment area adjacent to the wetwell.

Weston & Sampson obtained latest inspection report for the clearwell at Maher Treatment facility. As per the report, Extech Representatives last inspected the 250,000 gallons clearwell on September 5th, 2002. The inspection report mentioned clearwell being in good condition overall, with some minor defects and recommended cleaning of the clearwell within the next few years. Also, AWWA recommends inspections be done every five years. We recommend inspection and cleaning of the clearwell.

The three clearwell vertical turbine pumps are outfitted with Parco surge protection valves. The Parco valves are redundant since there are VFDs on the motors. The pumps are well maintained and operate properly. The area around the pumps is very neat and well kept.

The facility is designed to remove volatile organic compounds from the ground water via an air stripper. This is accomplished by pumping the untreated water to the top of strutted packing material. Atmospheric air is pumped into the bottom of the packing material. As the water pours down across the struts and the air is forced upwards, the water begins to spread into a thin sheet formation. The interface between the thin water sheet and pumped air causes the contaminants in the water to volatilize, turn into a gas, and be carried off by the air. Two out of the three Maher wells are always in operation in order to keep the air stripper in service. There is a carbon filter located next to the air stripper.

Table 8.1 shows the recommendations and improvement cost for the air stripper listed in the October 12, 2005 inspection report by Haley and Ward, Inc.

TABLE 8-1

AIR STRIPPER RECOMMENDATIONS

	Description	Year 2005 Cost[1]	Year 2007 Cost[2]
1	**Air Tower Recommendation**		
	Inlet Pipe	$2,300	$2,427
	Ditribution Tray-Cleaning	$2,800	$2,954
	Distribution Tray-Painting	$6,000	$6,330
	Packing Replacement	$32,000	$33,760
2	**Air Tower Ductwork and Blower Recommendation**		
	Intake Duct Replacement	$1,400	$1,477
	Exhaust Duct Replacement	$2,000	$2,110
	Blower Replacement	$7,800	$8,229
3	**Carbon Filter Recommendation**		
	Pressure Indicator	$1,000	$1,055
	Carbon Replacement	$35,000	$36,925
		TOTAL	**$95,267**

1 Engineering cost not included
2 January 2007 Cost obtained from ENR Boston indices

The operator reports that the process is working well and removing approximately 90% of the volatiles or better. The operator did express his concern that this system does not contain any redundancy. Weston & Sampson fully agrees with his concern. There is only one packed tower and only one blower. In the event the blower fails and requires extensive repairs or replacement, this system cannot operate. It could take as long as two weeks to obtain replacement parts for the blower plus the time to perform the repairs. If a new blower is required, the lead-time averages 6 to 10 weeks. Currently, even preventative maintenance on the blower requires a process shutdown.

The same problem exists in relation to the pack tower and the packing it contains. If the tower requires repair, or if the packing requires treatment or removal, the process must be shut down. This shortfall could have a very serious impact on the department's ability to produce an adequate volume of water, especially during high demand periods. The average maximum day demand is 5.1 mgd; with the Maher Treatment Facility out of service, the maximum amount of water that can be pumped into the system is greatly reduced. The water main feed to the air stripper currently needs maintenance. The galvanized exhaust duct also needs some cosmetic maintenance.

The Water Supply Division may consider installing a dehumidifier and air conditioning in the finish pump/treatment room. They may also consider installing a variable frequency drive (VFD) on the blower motor. This installation would be beneficial in reducing the electrical power consumption of the blower. It will also allow the operator to have the ability to adjust the blower speed to meet the well flows rendering the process more efficient and less costly.

8.8 Hyannisport Well and Treatment Facility

The building interior is well lit and heated. The Motor Control Center and Control Panels appear to be in very good condition. The Hyannisport well has a pressure relief valve that can be used to control distribution system pressure if the tanks are off-line or fail due to an ice blockage. The ground water is treated for iron and manganese via sequestering. The chemical storage and feed systems are very basic and in good repair. The 5000 gallon hydroxide storage tank is also in good shape. The tank is well protected from accidental damage and well ventilated. The containment area is abundant and appears to exceed the standard of 110 % of maximum chemical stored requirement. This facility currently treats with C-5 and C-9, however the Hyannis System Operators would like to replace this with polyphosphate treatment. Using a polyphosphate, such as zinc orthophosphate, would eliminate the need to use both C-5 and C-9 for treatment. The zinc orthophosphate is typically added in the distribution line before the water enters the system. The emergency power backup system is automated and exercised on a scheduled basis; however the generator is undersized and cannot be used to power the well pump. The building and grounds are neat, clean and in good condition.

8.9 Simmons Pond Well Station

This well was redeveloped and the pump was rebuilt in 2003. The well runs on tank level controlled from the office building but has the ability to run on pressure in emergency situations. The pump is driven with a 50 HP electric motor under normal conditions. A new screen guard should be installed on the motor. During emergency periods the pump can be operated by an LP gas-fired 8 cylinder engine with a right angle drive. The transition is a manual operation requiring the operator to adjust the motor coupling. The electric and control panels are of an older style but appear to have been well maintained. A new motor starter should be installed at this facility. The Water Supply Division is planning to refurbish some cosmetic aspects of this facility.

The well pump is outfitted with a Parco surge control valve. Although this valve has hydraulic actuation, normal operation requires electricity to operate solenoids that direct pressurized water to actuate the valve. Under emergency conditions, without electricity, this system has a manual control to operate the system. The Water Supply Division may consider natural gas to feed the generator as opposed to propane tanks. Cooling water from the pump currently discharges from the back wall of the facility. DEP Guidelines mandate a wetwell 100 feet from the facility for the discharge of water from a well pump station facility.

A small chemical feed system applies phosphate (C-5) to the water to arrest elevated manganese levels. The overall facility has been properly maintained and is in good condition. However a step is needed for better access to the building and crushed stone or gravel should be added for better drainage of the site, as the parking area gets muddy during rain events. There is a possibility that the facility may flood during heavy rains.

8.10 Straightway Well Station

This facility is in very good condition with adequate heat, light and ventilation. The building interior and exterior are clean and well maintained. The MCC, electrical and control panel boards are neat and well maintained. A variable frequency drive has been installed with the pump. The facility is surrounded by a fence, however there is no barbed wire on the fence.

A 5000 gallon bulk storage tank is utilized to store hydroxide and is fully contained. The Hyannis System Operators believe the other bulk chemical tanks are too big. The chemical feed systems also provide two phosphate products to sequester iron and manganese. The chemical feed pumps appear to be well maintained. The only negative note with regard to the chemical feed system is the size of the day tanks. Day tanks are typically sized to provide chemical to the chemical feed pumps for a period of 24 to 48 hours, under maximum dosage conditions. This sizing of these tanks provides the operator the ability to obtain accurate usage of chemical and minimizes the potential of over dosage. There are no provisions for emergency power at this facility.

8.11 Mary Dunn Tanks

This water storage facility is comprised of two cylindrical steel standpipes. Both tanks were last inspected in 2001. Refer to Chapter 7, Distribution Storage Evaluation, for more information. The smaller tank holds 370,000 gallon and the larger tank holds 1,000,000 gallons of water. Both tanks have a checkerboard paint pattern, due to their proximity to the Hyannis Airport.

The 370,000 gallon tank is a riveted tank. The roof of the tank has steel support trusses and a wood base with recently replaced asphalt roofing. The exterior paint appears to be in good condition. There has been some loss of metal on the rivet heads on this tank, however overall the tank appears to be in good condition.

The 1,000,000 gallon tank is of welded construction with a steel dome. Two 24-inch inspection manways are located on the base of the tank and an airport warning light is located on top of the tank. The manway cover is gasketed and bolted onto a flange. The paint on this tank also appears to be in good condition and a protective coating has been applied to the tank. There is adequate reveal on the ringwall of the tank. There are cell phone antennas attached to this tank, which impair general maintenance of this tank. The antenna company's ground equipment installation does not allow for complete access around the tank and the antenna cables impair access to the shell ladder. The Water Supply Division may consider having the cell phone companies relocate their ground equipment to outside the fenced area prior to the next repainting of the tank.

The control valves for both tanks are located in the pathway between the tanks. The valve boxes are raised several inches above the ground. It is recommended that markers be placed in the ground near each valve box. An identifier, (tank fill, tank drain, etc.), can be placed on the markers. This action can facilitate valve location and use in the dark and during snow events.

The overall condition of the storage tanks is good. The manway on the larger tank is covered with algae. This should be removed to prevent the acceleration of oxidation. There are spots on both tanks that either the paint has been scraped or flaked off. These spots should be cleaned, primed and painted to protect the integrity of the existing paint. As per Massachusetts DEP, the exterior and interior of tank should be cleaned and/or inspected annually by qualified personnel. A thorough structural and coating inspection should be conducted every 5 years.

8.12 Mary Dunn Well 1

The well contains a submersible pump with a pitless adapter and is located adjacent to the old Mary Dunn Station. Power to this well comes from the Mary Dunn Well Treatment Plant. The flowmeter for this well is located in the Mary Dunn Treatment Plant. The well head is fenced in at this site, however there is an unprotected sample tap located outside of the fenced in area. This should be moved inside the fenced in area or a fence should be installed around the tap. Although the operating wheel has been removed from the tap, it is still of critical concern. The outer well casing is rusting and should receive a protective coating of paint.

Hyannis needs to take more positive measures in order to prevent dumping at this site. A barrier gate is needed at this site in order to keep unauthorized vehicles from entering. Gravel should also be brought to this site in order to create an access road.

8.13 Mary Dunn Well 2

This is an older and small station. The building appears to be in good condition. The electrical panel is older but in relatively good condition. The station has emergency power and the generator which runs on propane is automatically exercised and well maintained. A small chemical feed system applies phosphate to sequester iron. This well runs off tank level but has a reserve system that can run on pressure only.

8.14 Mary Dunn Well 3

The entry to this facility is conducive to the accumulation of snow, snow melt, and excessive water run off from the road side of the building. There isn't any drainage relief to prevent excessive water build up. The water level can overflow into the facility entrance resulting in flooding of the station interior. There is no sump pump at this facility. It is recommended that this situation be corrected. This could be an electrical hazard and/or a health hazard to the operators.

The building appears to be in good condition. The electrical panel is mounted on a wooden stand approximately 18 inches from the front wall. The panel is old but appears to be in fair to good condition. Weston & Sampson recommends supplying emergency power to this site. There are trees growing on top of the discharge pipe outside of this facility. The trees should be cleared allowing for a 10-foot wide path centered over the pipe.

A small chemical feed system applies phosphate to sequester iron. The chemical system is in good repair. The well pump is also in good repair and utilizes a reduced voltage magnetic starter. The Water Supply Division may consider replacing the reduced voltage magnetic starter with a soft starter or a variable frequency drive. Both devices will reduce the electrical power consumption at the station. These devices also extend the life span of the pump by reducing the initial stress placed on the equipment during start-up.

We also recommend installing a fence around the facility for security and a gravel access road to allow for better drainage.

8.15 Mary Dunn Well 4

This station is utilized for emergency only as microscopic particulate analysis (MPA) testing in the past indicated that the water produced by this well may be under the influence of surface water.

Since Mary Dunn Well No. 4 is under the influence of surface water and is not currently in use, the Water Supply Division should consider constructing a replacement satellite well adjacent to the existing well. This well would be at a sufficient distance to avoid the influence of surface water and would eliminate the need to treat the existing well. The Department of Environmental Protection, Standard Operating Procedure for Replacement Wells, allows a replacement well to be situated within 250 feet of the original well. Replacement wells shall not significantly alter the existing groundwater hydraulics or Zone II boundaries. The Water Supply Division will need to submit a Request for Site/Exam Pumping Test Proposal – Letter Report to the DEP requesting permission for the replacement well.

The building is adequate for a well that is rarely used. The electrical system is old an in marginal condition. Backup power should be supplied to this facility. The building is not fenced and brush is becoming excessive. The metal door to the facility is rusting.

8.16 Mary Dunn Treatment Facility

The building interior and exterior appear to be in good condition with adequate lighting, heat and ventilation. The MCC, electric and control panel boards also appear to be in good condition. The station is also outfitted with an emergency generator to provide power when normal utility power is not available. The generator is in good condition and well maintained.

The facility has chemical feed systems to apply two different phosphate products and sodium hydroxide. The storage tank and day tanks, along with the chemical feed pumps are in good condition.

8.17 Old Mary Dunn Station

The building is out of service, boarded up, and in disrepair. There is an old 12-inch tubular well field adjacent to the building. The building is becoming unsightly with vines growing up the sides of it, which will lead to deterioration of the cement mortar. The vent in the gable has also been left open allowing animals and birds to get in. This should be boarded up. There is an old piston driven diaphram pump engine located within this building and there appears to be dumping of leaves and debris around this building as well. The dumping of leaves is of concern, especially given the close proximity of the Mary Dunn 1 Well.

We recommend that the department consider either repairing the building or demolishing it. We also suggest the installation of a barrier to minimize access to the area and prevent dumping.

8.18 Airport Well

This well is located in a fenced area on the grounds of the Barnstable Municipal Airport. There is no separate gate for the Water Supply Division to enter. Heightened security at this site mandates that the gate must be unlocked and then locked immediately behind personnel when accessing the well. The building interior and exterior are in good condition. The pump and motor appear to be in good condition. A Parco surge control valve has been installed on the pump. The electrical panels and control panels are in very good condition. Power is supplied to this site underground. The station does not have an emergency power source.

8.19 Conclusion

Weston & Sampson was impressed with the condition of the overall facilities. Throughout the tour it was obvious that regardless of the age of the building or equipment, priority was given to perform maintenance.

Maintenance is an investment and a commitment to maximize the use and dependability of properties and equipment. The level and detail of maintenance is directly related to the dependability and reliability of the properties and equipment that is currently operating. To this end, the Water Supply Division has done an outstanding job and has a very solid and reliable production system that their customers can depend on.

There are several items that the Water Supply Division may wish to consider for improvement in their capital improvements program (CIP). These items include emergency power to all of the stations, the office, and the garage.

Emergency power is currently provided to the majority of the well-fields and treatment facilities. This alone does not guarantee a station can be operated during a crisis situation. Weather events can result in flooding of a portion of the facilities while other stations can be severely damaged by wind damage or falling trees or poles. In these situations the operator may require the availability of those stations that do not currently have an emergency power source.

During a crisis situation the office typically becomes the command center. The office typically contains all of the data, (e.g.; drawings, locations, contact lists), that is required to direct emergency personnel. Much of this information is stored in the department computer system(s).

Power to the garage is necessary to provide lighting to find parts and/or tools for emergency repairs. It also provides a safe dry environment for workers to prepare their vehicles before departing for assignment. Electrical power also assures that the heating systems are available in both buildings, allowing outside personnel the opportunity to dry off and/or warm up during weather events.

Another consideration for the CIP is replacement of the SCADA system. As previously discussed the existing SCADA system is operational but old. Many of the electro-mechanical devices used have been replaced within the newer software programs. The new programs also eliminate the need for chart recorders and their associated charts and storage. All the data provided to the program is archived and can be presented in whatever format the user desires. Configuration of these programs can include a design to fill out and print state monthly report forms. With the appropriate hardware, the control function will allow the operator to start/stop well pumps and chemical feed systems. If the equipment has variable speed controls, the operator will have the ability to control those speeds as well.

The newer systems can also incorporate an alarm system that provides local notification at the monitor, as well as, paging and remote visual alarms. The alarms can be incorporated to include systems other than process such as high/low temperatures, security, and operator emergency. Alarms can be prioritized for notification purposes. The system can print and display on the monitor, all alarms, when they were activated, when they were acknowledged and when they were cleared.

The capital expenditure to procure a newer system offers savings in many other ways. A newer system requires fewer electro-mechanical devices, fewer chart recorders and charts, lower maintenance costs through the elimination of the aforementioned and increased efficiency of operations staff.

Lastly, Weston & Sampson noted that many of the facilities did not have fencing or security systems such as cameras and motion detectors. Fencing is a priority as it minimizes vandalism to properties. While fencing is not foolproof, it does provide an improved level of security. Intrusion alarms are provided through the SCADA system to notify personnel of unauthorized entry.

O:\BARNSTABLE MA\2060316 WATER MASTER PLAN\REPORT\CHAP8.DOC

9.0 RECOMMENDED IMPROVEMENTS AND PHASED IMPROVEMENT PROGRAM

9.1 General

In the preceding chapters, we have described the various improvements for the water supply and distribution system to eliminate current system deficiencies and maintain the system in good working order. The recommended improvements suggested in the previous chapters are estimated to cost a total of $33,330,00 (year 2007 costs). This total cost is broken down into $1,110,000 segments as described in the 30 year phased improvement program.

9.2 Summary of Recommended Improvements

The following recommended improvements are intended to eliminate current system deficiencies and maintain the system in good working order. The facilities without backup emergency power should be supplied with generators. These facilities include:

- Mary Dunn Well No.3
- Airport Well
- Hyannisport Well – backup needed for pump only
- Straightway Well No. 2

The existing generator at the Hyannisport Well should be moved to the Mary Dunn Well No. 3 facility; and a new generator capable of supplying power for the entire Hyannisport Well facility should be purchased. Energy saving improvements should also be made to the Maher Treatment Facility. This can be accomplished by removing bowls from the pumps and downsizing the motors. HVAC equipment such as a dehumidifier and air conditioning should also be installed in the Maher No. 1 facility and the Maher Treatment Facility.

There is currently not enough tank storage available to meet the equalization or fire storage needs, and there is no emergency storage available within the storage tanks. As the average and maximum day demands increase with time, the storage deficiency will increase. Weston & Sampson recommends that a new storage tank be constructed in Hyannis. The construction of a new tank will substantially strengthen the Hyannis's distribution storage capabilities. Considerations for locating a new tank should include elevation, distance from the existing tanks, and proximity to areas of high water demand. The location of the existing tanks is hydraulically remote and far from the locations of most water demand. The Hyannis Water Supply Division should identify the land availability and any political concerns in the neighborhood of potential locations. A location for a new storage tank will be determined through further study.

Table 9-2 and 9-3 lists the recommended improvements described in this chapter.

9.3 Estimated Construction Costs

The estimated costs specified for each group of improvements include construction costs, engineering costs, and contingencies. The estimated costs were developed in part by using recent construction costs for towns with similar development and geographic location to Barnstable. These costs were updated to an Engineering News Record (ENR) Boston index of 9200. Other sources include the Means "Building Construction Cost Data" and manufacturers' quotations. It should be noted that the estimated costs listed in this report reflect 2007 construction costs and engineering costs only, and no steps were taken to inflate the costs to reflect future construction costs. The per foot construction costs used in this chapter are shown in Table 9-1.

Water main construction projects should be bid prior to February in order to receive competitive pricing. If possible, smaller projects should also be combined so the total construction length is increased and more potential bidders will be attracted. Following these general guidelines will ensure that the Water Supply Division can make necessary improvements to the system in the most cost efficient manner.

TABLE 9-1
PRESENT DAY COSTS BY RECOMMENDATION

Recommendation	Cost (per lin. ft)
Replace with 8-inch Ductile Iron Water Main	$160.00
Replace with 12-inch Ductile Iron Water Main	$185.00
Clean and Line 12-inch Water Main	$130.00
Clean and Line 16-inch Water Main	$145.00
Abandon Water Main	$55.00

The costs for replacement of water main used in this study are for year 2007 construction and engineering costs and include the following:

- Design, construction administration, and resident inspection engineering.
- 10% contingencies.
- Abandonment of the old water main and replacement with new water main.
- Hydrants at 500 feet spacing and gate valves at 500 foot maximum spacing.
- Replacement of non-copper water services (including corporation stop, copper tubing and curb stop).
- 2-inches of trench binder course pavement.
- Trench-width top-course pavement 1½-inches thick.

9.4 Phased Improvement Program

The improvements to the distribution system were split into two priorities, A and B and are listed in Table 9-2 and Table 9-3. Phase A improvements are designed to improve the hydraulic capacity of the distribution system by providing additional direct, large sized transmission mains from the Maher Treatment Facility towards the downtown area. These improvements are also intended to eliminate the existing fire flow deficiencies in the eastern section of Hyannis and improve water quality and supply to the downtown area. These improvements are recommended to be completed in the first years of the phased program.

The goal of the Phase B improvements is to eliminate other deficiencies within the system and to make non-critical improvements in order to strengthen various areas of Hyannis as well as the system in general. These improvements include replacing all unlined cast iron water mains and the vinyl lined asbestos cement water mains in the town and to provide the looping water mains as shown in Table 6-4. The cost for these improvements is based on construction in year 2007 and have not been adjusted for inflation.

TABLE 9-2

PHASE A IMPROVEMENTS

Phase	Description	Year 2007 Cost
A-1	Air Stripper Improvements	$100,000
	Relocate Mary Dunn Well No. 4	$440,000
	Provide 3 new generators to well facilities and other building improvements	$610,000
A-2	Replace existing 8-inch water main on Maher Road from Treatment Plant to Yarmouth Road with a new 12-inch ductile iron water main adjacent to existing 12-inch water main	$190,000
	Replace existing 6-inch water main on Yarmouth Road from Maher Road to Camp Street with a new 12-inch ductile iron water main adjacent to existing 12-inch water main	$180,000
	Replace existing 6-inch water main on Lewis Bay Road from Camp Street to South Street with a new 12-inch ductile iron water main	$130,000
A-3	Replace existing 6-inch water main on South Street from Lewis Bay Road to Sea Street with a new 12-inch ductile iron water main	$890,000
A-4	Construction of new 1.25 MG elevated storage tank	$3,000,000
A-5	Tank Painting	$430,000
	SCADA system upgrade	$440,000
A-6	Replace existing 6-inch/8-inch water main on Ocean Street between Main Street and Channel Point Road with a new 12-inch ductile iron water main	$440,000
	Replace existing 6-inch water main on Barnstable Road from Main Street to Winter Street with a new 12-inch ductile iron water main	$630,000
A-7	Replace existing 6-inch water main on Scudder Avenue from Craigville Beach Road to Greenwood Avenue with a new 12-inch ductile iron water main	$740,000
A-8	Looping Highland Street to Cook Circle with 2,000 feet of new 8-inch ductile iron water main	$340,000
	TOTAL	**$8,560,000**

Table 9-4 presents the improvements that should be made after the completion of the Phase A improvements.

TABLE 9-3

PHASE B IMPROVEMENTS

Description	Total Length (feet)	Year 2007 Cost
Replace all 6-inch and 8-inch unlined cast iron water main with 8-inch ductile iron water main	71,680	$12,040,000
Clean and line 12-inch unlined cast iron water main	18,140	$2,360,000
Clean and line 16-inch unlined cast iron water main	7,640	$1,110,000
Abandonment of parallel water mains	12,200	$670,000
Looping water mains with 8-inch ductile iron water main	1,230	$210,000
Looping water mains with 12-inch ductile iron water main	200	$40,000
Replacement of existing 6-inch and 8-inch Vinyl AC water main with 8-inch ductile iron water main	30,440	$5,110,000
Replacement of 10-inch and 12-inch Vinyl AC water main with 12-inch ductile iron water main	17,465	$3,230,000
TOTAL	**157,695**	**$24,770,000**

9.5 Schedule

WSE recommends that the Barnstable Water Supply Division implement an 30-year phased improvement program. The estimated construction cost of up to $33.5 million will require approximately $1,110,000 per year on improvements. There are several options for the implementation of Phase A and Phase B improvements. Some important factors that were kept in mind while designing the phasing plan were:

- It is more efficient and cost effective to keep the improvement program ahead of system deterioration.
- Work should be scheduled and grouped together so that all utility work needed within a certain neighborhood, can be completed all at once.

These guidelines will insure that the oldest pipes in the system are replaced as they reach the end of their useful life of 100 years.

O:\BARNSTABLE MA\2060316 WATER MASTER PLAN\REPORT\CHAP9.DOC

10.0 MISCELLANEOUS MAINTENANCE

10.1 General

In addition to the water main replacement program recommended in Chapter 9, we recommend that certain maintenance efforts be continued on a yearly basis. This includes adopting a set of minimum standards for piping materials, appurtenances and construction methods, systematic flushing of the distribution system, inspecting hydrants and gate valves, testing and replacement of water meters, and record keeping practices.

10.2 Minimum Standards

It is recommended that the following minimum design standards be adopted by the Hyannis Water Supply Division to provide consistency in system design and construction. These are consistent with DEP guidelines.

Water Main Material:	Ductile Iron
Minimum Depth of Cover:	4.5 feet
Maximum Gate Valve Spacing:	500 linear feet
Gate Valve Type:	Resilient Seat
Maximum Hydrant Spacing:	500 linear feet
Hydrant Open Direction:	Left (counterclockwise)
Valve Open Direction:	Left (counterclockwise)
Hydrant Barrel Color:	Red
Hydrant Bonnet and Cap Color:	White
Service Piping Material:	Type K Copper or Polyethylene
Corporation Stops:	1-inch
Minimum Service Diameter:	1-inch

This listing provides a base listing of water system standard materials. We recommend that the Water Supply Division expand this list as necessary and add standard construction requirements and details to insure consistency throughout the distribution system.

The Massachusetts DEP recommends hydrant spacing to range from 350 to 600 feet depending on the area and valves to be placed at 500 feet for commercial area and not more than one block or 800 feet in more rural areas. The Water Supply Division uses 500 feet for hydrant spacing and 500 feet for valves spacing, which is in compliance with DEP standards.

10.3 System Flushing

Systematic flushing of the system should be scheduled a minimum of once per year to help prevent rust and sediment accumulations in the distribution system. Piping frequently builds up deposits that must be removed to maintain the carrying capacity of the line and to prevent water quality problems. By flushing, higher than normal velocities are obtained, scouring the side of the pipe and removing scale and debris. For optimal scouring, flushing should be performed in a manner that produces flow directions opposite of normal directions. Flushing is especially important in older and dead-end mains. Flushing is also a means of locating obstructions in the distribution system by alerting the operators of insufficient flows.

10.4 Valve Inspection

All hydrants and gate valves should be inspected periodically. Both should be opened and closed to establish their condition. By checking and operating each valve it can be determined whether a valve was left in the correct position, and the condition of the valves, stems, and operators can be established. It is recommended that the Water Supply Division adopt a valve exercising program during which each hydrant and gate valve should be inspected at least twice each year, preferably once in the spring and once in the fall. It is advisable to check all hydrants after any usage. The system flushing and valve and hydrant inspection can be performed consecutively to maximize efficiency.

10.5 Water Meters

A program should be established for the regular testing and maintenance of water meters. This is recommended for three reasons. First, to ensure that the cost of water service is equitably distributed among all customers; secondly, loss of revenue to the Water Supply Division may occur if the meters are not maintained at a reasonable level of efficiency; and finally, because it is a requirement of the Water Supply Division's Water Management Act Permit.

It is advisable to provide for more frequent tests of large meters, on the logical premise that an error in its registration affects revenue to a much greater extent. Older meters and those carrying the heaviest volume should be given priority in a testing program. In addition, displacement meters, which are the type most commonly used, may seriously under-register for long periods without complete stoppage. Turbine meters are poor for applications where low or variable flow rates will be experienced.

It is necessary to test meters periodically to minimize loss of revenue. The accuracy of meters in service are subject to change and may either under- or over-register. The period of time for which water meters retain overall accuracy is variable and depends mainly on the characteristics and quality of water being measured. The rates charged for water service also have a distinct bearing on how frequently meters should be tested. Meter testing should be in accordance with AWWA standards. While it is difficult to determine the economic balance between the cost of more frequent testing and potential loss in revenue caused by meter under-registration, meter testing is necessary.

10.6 Record Keeping

It is important that the Water Supply Division maintain accurate and detailed records of work performed throughout its system including new water mains and service installation, water main and service repairs and upgrades, flushing and exercising programs, and new meter installation, existing meter repair, and replacement and meter maintenance. The Water Supply Division should computerize all record keeping to prepare for the water distribution system's future inclusion into Hyannis's future GIS mapping.

The Water Supply Division should also request that Hyannis maintain better records of its population. As the DEP places more importance on the residential per capita water consumption, it will be important for the Water Supply Division to know its full-time population as well as its seasonal population.

O:\BARNSTABLE MA\2060316 WATER MASTER PLAN\REPORT\CHAP10.DOC

Appendix A

ISO and Weston & Sampson Flow Test Data Sheets

&

Weston & Sampson C-Valve Test Data Sheets

INSURANCE SERVICES OFFICE, INC.

HYDRANT FLOW DATA SUMMARY

City Hyannis

County Barnstable State MA Witnessed by: Insurance Services Office, Inc. Date October 31, 2006

TEST NO.	TYPE DIST.*	TEST LOCATION	SERVICE	FLOW - GPM Q=(29.83(C(d²)p^0.5))				PRESSURE PSI		FLOW -AT 20 PSI		REMARKS***
				INDIVIDUAL HYDRANTS			TOTAL	STATIC	RESID.	NEEDED **	AVAIL.	
1	Comm	Yarmouth Road @ Ferndoc Street	Main	650	650		1300	92	48	2000	1700 I	
2	Comm	Iyannough Road @ Cedar Street	Main	1240			1240	86	72	3000	2900 I	
3	Comm	Ridgewood Avenue @ Center Street	Main	1280			1280	85	58	4000	2100 I	
3A	Comm	Ridgewood Avenue @ Center Street	Main	1280			1280	85	58	3500	2100 I	
4	Comm	Barnstable Road @ Baxter Road	Main	1190			1190	80	60	5500	2200 I	(A)-(3950 gpm)
4A	Comm	Barnstable Road @ Baxter Road	Main	1190			1190	80	60	3500	2200 I	(A)-(2950 gpm)
5	Comm	Attuck's Way @ Airport Road	Main	2260			2260	77	55	3500	3800	(A)-(2950 gpm)
6	Comm	Corporation Street @ Cape Cod Mall	Main	1240			1240	79	71	4000	3600 I	
6A	Comm	Corporation Street @ Cape Cod Mall	Main	1240			1240	79	71	2500	3600	
7	Comm	Iyannough Road @ Bearses Way	Main	2120			2120	68	40	5500	2800 I	
7A	Comm	Iyannough Road @ Bearses Way	Main	2120			2120	68	40	1000	2800	
8	Comm	West Main Street @ High School	Main	2020			2020	75	40	5000	2600 I	
8A	Comm	West Main Street @ High School	Main	2020			2020	75	40	3000	2600 I	
9	Comm	Old Craigville Road @ Elementary School	Main	1430			1430	76	40	3500	1800 I	
10	Res	Sixth Ave @ Pine Way	Main	750			750	93	45	1000	950	
11	Res	Last Hydrant on Squaw Island Road	Main	480			480	82	6	1000	450	

THE ABOVE LISTED NEEDED FIRE FLOWS ARE FOR PROPERTY INSURANCE PREMIUM CALCULATIONS ONLY AND ARE *NOT* INTENDED TO PREDICT THE MAXIMUM AMOUNT OF WATER REQUIRED FOR A LARGE SCALE FIRE CONDITION. THE AVAILABLE FLOWS ONLY INDICATE THE CONDITIONS THAT EXISTED AT THE TIME AND AT THE LOCATION WHERE TESTS WERE WITNESSED.

*Comm = Commercial; Res = Residential.

**Needed is the rate of flow for a specific duration for a full credit condition. Needed Fire Flows greater than 3,500 gpm are not considered in determining the classification of the city when using the Fire Suppression Rating Schedule.

***(A)-Limited by available hydrants to gpm shown.

INSURANCE SERVICES OFFICE, INC.

HYDRANT FLOW DATA SUMMARY

City Hyannis
County Barnstable State MA Witnessed by: Insurance Services Office, Inc. Date October 31, 2006

TEST NO.	TYPE DIST.*	TEST LOCATION	SERVICE	FLOW - GPM $Q=(29.83(C(d^2)p^{0.5}))$ INDIVIDUAL HYDRANTS			FLOW - GPM TOTAL	PRESSURE PSI STATIC	PRESSURE PSI RESID.	FLOW -AT 20 PSI $Q_R=Q_F(h_R^{0.54}/h_F^{0.54})$ NEEDED **	FLOW -AT 20 PSI AVAIL.	REMARKS***
12	Res	Edgehill Road @ Mt. Vernon Avenue	Main	810			810	85	53	1000	1200	
13	Res	Pitchers Way @ Elizabeth Lane	Main	1240			1240	82	72	750	3300	
14	Res.	Pitchers Way @ Wayland Road	Main	1970			1970	74	50	750	3100	
15	Comm	Main Street @ Bassett Lane	Main	920	920		1840	80	65	4500	3900	
15A	Comm	Main Street @ Bassett Lane	Main	920	920		1840	80	65	3500	3900	
16	Comm	North Street @ Stevens Street	Main	1400			1400	87	75	4500	3500	(A)-(3900 gpm)
16A	Comm	North Street @ Stevens Street	Main	1400			1400	87	75	3000	3500	
17	Comm	Old Colony Road @ Snow Creek Drive	Main	1060			1060	88	54	1500	1500	
18	Comm	Ocean Street @ Bay Street	Main	1130			1130	85	42	5000	1400	(A)-(2950 gpm)
18A	Comm	Ocean Street @ Bay Street	Main	1130			1130	85	42	1500	1400	
19	Res	Old Harbor Road @ Bay Shore Road	Main	920			920	95	38	1000	1100	
20	Comm	Ocean Street @ Gosnold Street	Main	840			840	95	50	3500	1100	
21	Comm	Iyannough Road @ Festival Plaza	Main	2020			2020	80	65	4000	4300	
21A	Comm	Iyannough Road @ Festival Plaza	Main	2020			2020	80	65	2250	4300	
22	Comm	Barnstable Road @ Hinckley Road	Main	1130			1130	70	60	4000	2700	(A)-(1950 gpm)
22A	Comm	Barnstable Road @ Hinckley Road	Main	1130			1130	70	60	3000	2700	

THE ABOVE LISTED NEEDED FIRE FLOWS ARE FOR PROPERTY INSURANCE PREMIUM CALCULATIONS ONLY AND ARE *NOT* INTENDED TO PREDICT THE MAXIMUM AMOUNT OF WATER REQUIRED FOR A LARGE SCALE FIRE CONDITION. THE AVAILABLE FLOWS ONLY INDICATE THE CONDITIONS THAT EXISTED AT THE TIME AND AT THE LOCATION WHERE TESTS WERE WITNESSED.

*Comm = Commercial; Res = Residential.

**Needed is the rate of flow for a specific duration for a full credit condition. Needed Fire Flows greater than 3,500 gpm are not considered in determining the classification of the city when using the Fire Suppression Rating Schedule.

***(A)-Limited by available hydrants to gpm shown.

City ınnis
County ɒarnstable State MA 2006

613 & 614 HYDRANT FLOW DATA (MC(1) & HD(1))

Test No.	Location	Needed Fire Flow	Service Level	Pressure (PSI) Static	Residual	Pitot	Orifice	Flow GPM	Flow GPM@ 20 psi	Hydrant Condition
1	Yarmouth Road @ Ferndoc Street	2000	1	92	48			1300	1700	OK
						15	2.5	650		
						15	2.5	650		
2	Iyannough Road @ Cedar Street	3000	1	86	72			1240	2900	OK
						55	2.5	1240		
3	Ridgewood Avenue @ Center Street	4000	1	85	58			1280	2100	OK
						8	4.5	1280		
3A	Ridgewood Avenue @ Center Street	3500	1	85	58			1280	2100	OK
						8	4.5	1280		
4	Barnstable Road @ Baxter Road	5500	1	80	60			1190	2200	OK
						50	2.5	1190		
4A	Barnstable Road @ Baxter Road	3500	1	80	60			1190	2200	OK
						50	2.5	1190		
5	Attuck's Way @ Airport Road	3500	1	77	55			2260	3800	OK
						25	4.5	2260		
6	Corporation Street @ Cape Cod Mall	4000	1	79	71			1240	3600	OK
						55	2.5	1240		
6A	Corporation Street @ Cape Cod Mall	2500	1	79	71			1240	3600	OK
						55	2.5	1240		

7	nough Road @ Bearses Way	5500	1	68	40			2120	2800	OK
						22	4.5	2120		
7A	Iyannough Road @ Bearses Way	1000	1	68	40			2120	2800	OK
						22	4.5	2120		
8	West Main Street @ High School	5000	1	75	40			2020	2600	OK
						20	4.5	2020		
8A	West Main Street @ High School	3000	1	75	40			2020	2600	OK
						20	4.5	2020		
9	Old Craigville Road @ Elementary School	3500	1	76	40			1430	1800	OK
						10	4.5	1430		
10	Sixth Ave @ Pine Way	1000	1	93	45			750	950	OK
	Ocean Avenue @ Sixth Ave					20	2.5	750		OK
11	Last Hydrant on Squaw Island Road	1000	1	82	6			480	450	OK
						8	2.5	480		
12	Edgehill Road @ Mt. Vernon Avenue	1000	1	85	53			810	1200	OK
						23	2.5	810		
13	Pitchers Way @ Elizabeth Lane	750	1	82	72			1240	3300	OK
						55	2.5	1240		
14	Pitchers Way @ Wayland Road	750	1	74	50			1970	3100	OK
						19	4.5	1970		

15	n Street @ Bassett Lane	4500	1	80	65			1840	3900	OK
						30	2.5	920		
						30	2.5	920		
15A	Main Street @ Bassett Lane	3500	1	80	65			1840	3900	OK
						30	2.5	920		
						30	2.5	920		
16	North Street @ Stevens Street	4500	1	87	75			1400	3500	OK
						70	2.5	1400		
16A	North Street @ Stevens Street	3000	1	87	75			1400	3500	OK
						70	2.5	1400		
17	Old Colony Road @ Snow Creek Drive	1500	1	88	54			1060	1500	OK
						40	2.5	1060		
18	Ocean Street @ Bay Street	5000	1	85	42			1130	1400	OK
						45	2.5	1130		
18A	Ocean Street @ Bay Street	1500	1	85	42			1130	1400	OK
						45	2.5	1130		
19	Old Harbor Road @ Bay Shore Road	1000	1	95	38			920	1100	OK
						30	2.5	920		
20	Ocean Street @ Gosnold Street	3500	1	95	50			840	1100	OK
						25	2.5	840		
21	Iyannough Road @ Festival Plaza	4000	1	80	65			2020	4300	OK
						20	4.5	2020		

21A	Iyannough Road @ Festival Plaza	2250	1	80	65			2020	4300	OK
						20	4.5	2020		
22	Barnstable Road @ Hinckley Road	4000	1	70	60			1130	2700	OK
						45	2.5	1130		
22A	Barnstable Road @ Hinckley Road	3000	1	70	60			1130	2700	OK
						45	2.5	1130		

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: Barnstable Water Model Date: 10/31/06 Time: 9:10 AM

Test Number: 1 Inspector: JDN

City: Hyannis State: MA

Location

Zone:

Streets: Yarmouth Road and Ferndoc Street

Weather: Sunny, Warm

Sources of supply in operations and rates: ~~SEE ATTACHED~~

Tank levels:

Type of development in the area: Commercial

Required Flow:

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
Gage	Static:	~~90~~ 92	Static:	94
	Residual:	48	Residual:	50
Flow	Static:	94	Static:	94
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
Flow	2x 2.5	.9	6"		2x 15	~~920~~

Flow available at 20 psi ~~[illegible]~~ 1721 gpm.

$Q = 29.83(.9)(2.5)^2\sqrt{15} = \cancel{735.23}$ $\cancel{3676.13}$ 649.86; x2 = 1299.

Sketch & Remarks

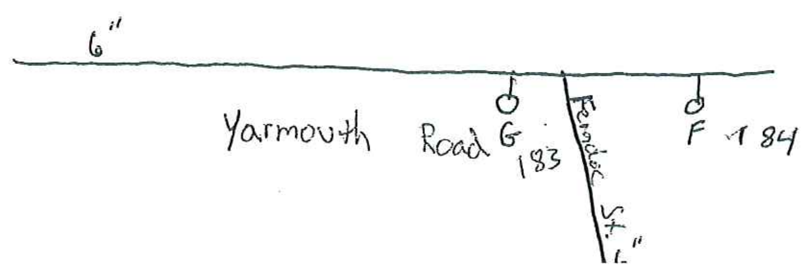
Q = 920

ΔS = 2

R_A = 50

~~Q = 920 ΔS = +4 R_A = 52~~

$$Q_{20} = \underset{1300}{\cancel{920}}\left[\frac{(94-20)^{.54}}{(94-50)^{.54}}\right] = 1721$$



Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project:
Date: 10/31/06
Time: 9:25 AM
2 Test Number: 2
Inspector:
City:
State:
Location
Zone:
Streets: Iyanough Rd + Cedar Street
Weather: Sunny, warm
Sources of supply in operations and rates:
Tank levels:
Type of development in the area: Heavy Commercial
Required Flow:

Hydrant No.		Observed Pressure (psi)		Corrected Pressure (psi)
Gage	Static:	86	Static:	68
	Residual:	72	Residual:	54
Flow	Static:	~~85~~ 68	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
Flow	2.5	.9	[illegible]		55	

Flow available at 20 psi ~~968~~ 2420 gpm.

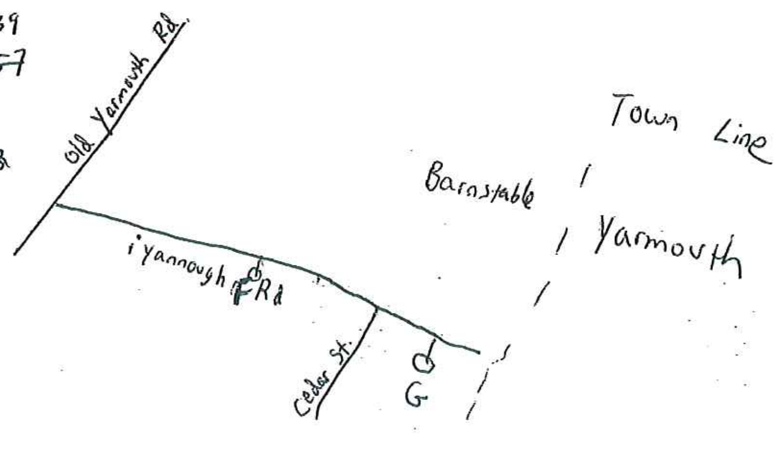
$\Delta s = -18$

Dirty Water ≈ 10 min to Clear

Sketch & Remarks

$$Q = 29.83\,(.9)\,(2.5)^2\sqrt{55} = \cancel{497.757}\ 1244.39$$

$$Q_{20} = \cancel{497.757}\ 1244.39 \left[\frac{(68-20)^{.54}}{(68-54)^{.54}}\right] = \cancel{968.23}\ 2420$$



Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ______ Date: 10/31/06 Time: 9:50 AM

3 Test Number: 22 Inspector: ______

City: ______ State: ______

Location

Zone: ______

Streets: Barnstable Road + Hinckley Road

Weather: Sunny, warm

Sources of supply in operations and rates: ______

Tank levels: ______

Type of development in the area: Heavy Commercial

Required Flow: ______

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
Gage	Static:	70	Static:	70
	Residual:	60	Residual:	60
Flow	Static:	70	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

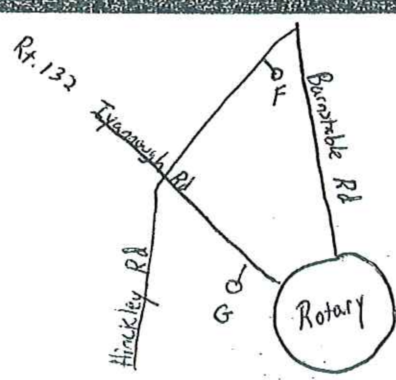
Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
Flow	2.5	.9	8"		45	

Flow available at 20 psi 2684 gpm.

Sketch & Remarks

$$Q = 29.83\,(.9)(2.5)^2\sqrt{45} = 1125.59$$

$$Q_{20} = 1125.59\left[\frac{(70-20)^{.54}}{(70-60)^{.54}}\right] =$$



Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ______ Date: 10/31/06 Time: 10:15 AM

4 Test Number: 5 Inspector: ______

City: ______ State: ______

Location

Zone: ______

Streets: Attuck's Way + Airport Road

Weather: ______

Sources of supply in operations and rates: ______

Tank levels: ______

Type of development in the area: ______

Required Flow: ______

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
Gage	Static:	77	Static:	72
	Residual:	55	Residual:	50
Flow	Static:	72	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

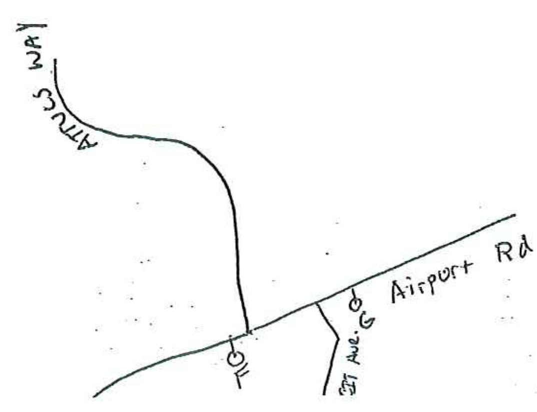
Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	~~2~~ 4.5	.9	8"		25	

Flow available at 20 psi 4325 gpm. $\Delta S = -5$

Sketch & Remarks

$Q = 29.83 \times .9 \times (4.5)^2 \sqrt{25} = 2718.26$

$Q_{20} = 2718.26 \left[\frac{(72-20)^{.54}}{(72-50)^{.54}} \right] =$



Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: Barnstable Water Model Date: 11/3/06 Time:
Test Number: 5A Inspector:
City: Hyannis State: MA
Location
Zone:
Streets: Attuck's Way + Airport Road
Weather:
Sources of supply in operations and rates:
Tank levels:
Type of development in the area:
Required Flow:

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
Gage	Static:	72	Static:	
	Residual:	51	Residual:	
Flow	Static:	~~81~~	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5		~~8"~~		45	

Flow available at 20 psi 1837 gpm.

Sketch & Remarks

$$Q = 29.83\,(.9)\,(2.5)^2\sqrt{45} = 1125.59$$

$$Q_{20} = 1125.59\left[\frac{(72-20)^{.54}}{(72-51)^{.54}}\right] = 1837$$

* Info from Deputy Fire Chief Melanson After ISO tests Conclude

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ______ Date: 10/31/06 Time: 10:40 AM

Test Number: 21 Inspector: ______

City: ______ State: ______

Location

Zone: ______

Streets: Iyannough Rd + Festival Plaza

Weather: ______

Sources of supply in operations and rates: ______

Tank levels: ______

Type of development in the area: ______

Required Flow: ______

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
Gage	Static:	80	Static:	78
	Residual:	65	Residual:	63
Flow	Static:	78	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

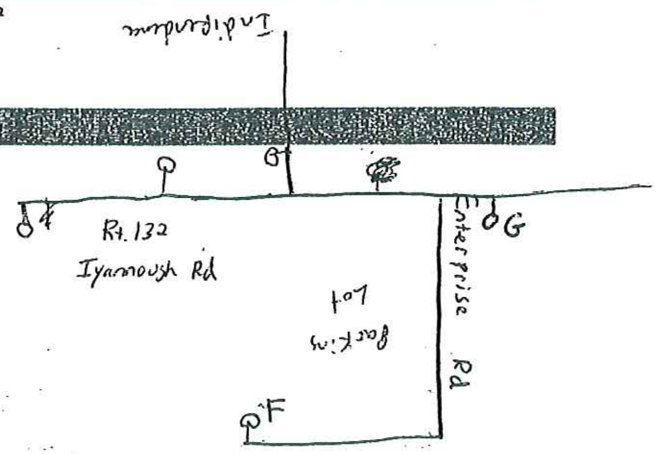
Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
Flow	4.5		[illegible]		20	

Flow available at 20 psi 5047 gpm.

Sketch & Remarks

$$Q = 29.83\,(.9)(4.5)^2\sqrt{20} = 2431.28$$

$$Q_{20} = 2431.28\left[\frac{(78-20)^{.54}}{(78-63)^{.54}}\right] =$$



Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ______________ Date: 10/31/06 Time: 11:00 AM

6 Test Number: 7 Inspector: ______________

City: ______________ State: ______________

Location

Zone: ______________

Streets: Iyannough Rd + Bearses Way

Weather: ______________

Sources of supply in operations and rates: ______________

Tank levels: ______________

Type of development in the area: Heavy Commercial

Required Flow: ______________

Hydrant No.	Observed Pressure (psi)	Corrected Pressure (psi)
	Static: 68	Static: 68
	Residual: 40	Residual: 40
	Static: 68	Static:
	Residual:	Residual:
	Static:	Static:
	Residual:	Residual:

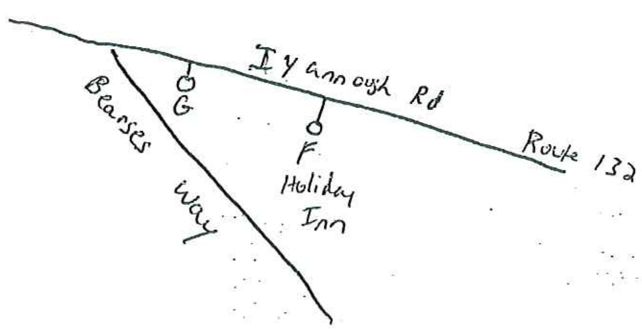
Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	4.5	.9	8"		22	

Flow available at 20 psi 3411 gpm.

Sketch & Remarks

$$Q = 29.83 \times (.9)(4.5)^2\sqrt{22} = 2550$$

$$Q_{20} = 2550\left[\frac{(68-20)^{.54}}{(68-40)^{.54}}\right]$$



Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ____________ Date: 10/31/06 Time: 11:15 AM
7 Test Number: 6 Inspector: ____________
City: ____________ State: ____________
Location
Zone: ____________
Streets: Corporation St + Enterprise Road
Weather: ____________
Sources of supply in operations and rates: ____________
Tank levels: ____________
Type of development in the area: ____________
Required Flow: ____________

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
Gage	Static:	79	Static:	
	Residual:	71	Residual:	
Flow	Static:	79	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

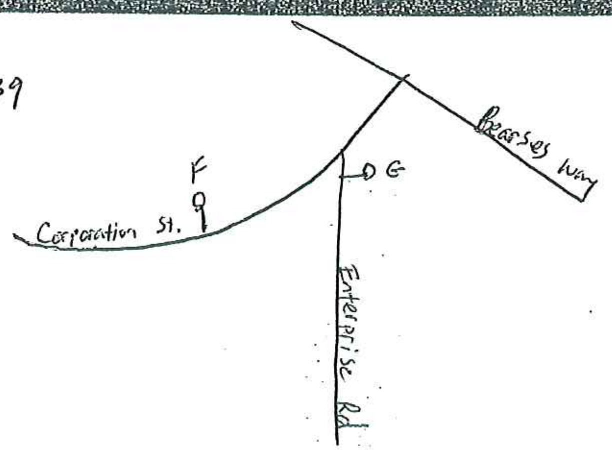
Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5		[illegible]"		55	

Flow available at 20 psi 3661 gpm.

Sketch & Remarks

$Q = 29.83(.9)(2.5)^2\sqrt{55} = 1244.39$

$Q_{20} = 1244.39\left[\frac{(79-20)^{.54}}{(79-71)^{.54}}\right] =$



Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ______ Date: ______ Time: 11:35 AM
8 Test Number: 4 Inspector: ______
City: ______ State: ______
Location
Zone: ______
Streets: ______
Weather: ______
Sources of supply in operations and rates: ______
Tank levels: ______
Type of development in the area: ______
Required Flow: ______

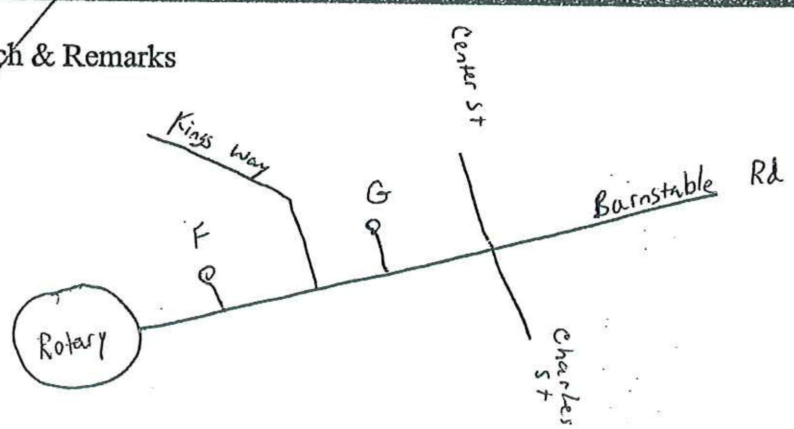
Hydrant No.

Observed Pressure (psi)	Corrected Pressure (psi)
Static:	Static:
Residual:	Residual:
Static:	Static:
Residual:	Residual:
Static:	Static:
Residual:	Residual:

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
			8"			

Flow available at 20 psi ______ gpm.

Sketch & Remarks



Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ______ Date: 10/31/06 Time: 11:45 AM
8 Test Number: 4 Inspector: ______
City: ______ State: ______
Location
Zone: ______
Streets: ______
Weather: ______
Sources of supply in operations and rates: ______
Tank levels: ______
Type of development in the area: ______
Required Flow: ______

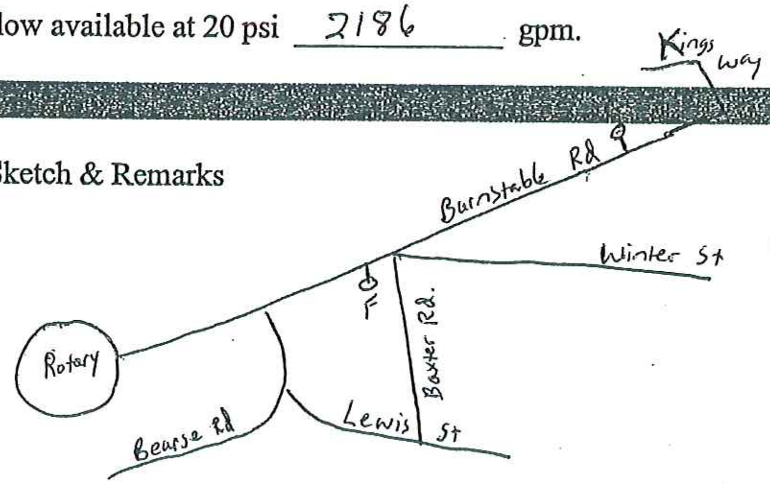
Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
	Static:	80	Static:	82
	Residual:	60	Residual:	62
	Static:	82	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5		8"		50	

$\Delta S = 2$

Flow available at 20 psi 2186 gpm.

Sketch & Remarks



$Q = 29.83\ (.9)(2.5)^2\sqrt{50} = 1186$

$Q_{20} = 1186.4\left[\frac{(82-20)^{.54}}{(82-62)^{.54}}\right]$

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ____________ Date: ____________ Time: ____________

9 Test Number: 3 Inspector: ____________

City: ____________ State: ____________

Location

Zone: ____________

Streets: ____________

Weather: ____________

Sources of supply in operations and rates: ____________

Tank levels: ____________

Type of development in the area: ____________

Required Flow: ____________

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
	Static:	85	Static:	84
	Residual:	58	Residual:	57
	Static:	84	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	4.5		8"		8	

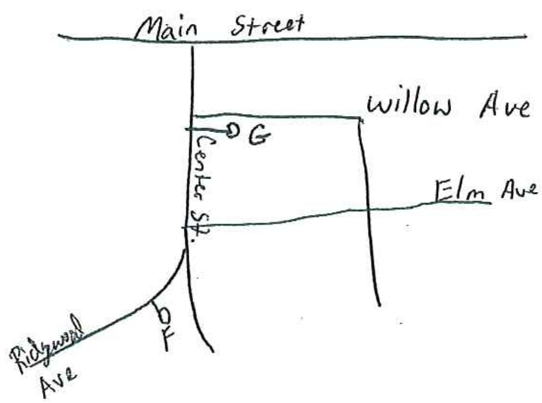
Flow available at 20 psi 2451 gpm. $\Delta s = -1$

Dirty Water

Sketch & Remarks

$Q = 29.83(.9)(4.5)^2\sqrt{8}$

$= 1538$



$$Q_{20} = 1538\left[\frac{(84-20)^{.54}}{(84-57)^{.54}}\right]$$

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ______ Date: 10/31/06 Time: 12:40 pm
10 Test Number: 18 Inspector: ______
City: ______ State: ______
Location
Zone: ______
Streets: ______
Weather: ______
Sources of supply in operations and rates: ______
Tank levels: ______
Type of development in the area: ______
Required Flow: ______

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
	Static:	85	Static:	82
	Residual:	42	Residual:	39
	Static:	82	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

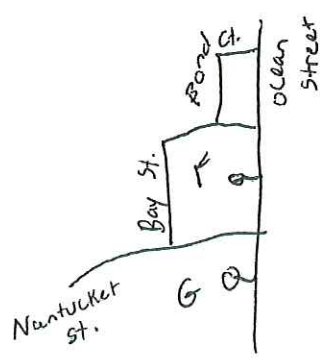
Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5		8"		45	1130

Flow available at 20 psi ≃ 1400 (1371.52) gpm.

Sketch & Remarks

$Q = 29.83(.9)(2.5)^2\sqrt{45} = 1125.59$

$Q_{20} = 1125.59\left[\frac{82-20^{.54}}{82-39^{.54}}\right] =$



Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ____________________ Date: __________ Time: 1:05 PM

Test Number: ~~RS~~ 20 Inspector: __________

City: ____________________ State: __________

Location

Zone: ____________________

Streets: ____________________

Weather: ____________________

Sources of supply in operations and rates: ____________________

Tank levels: ____________________

Type of development in the area: ____________________

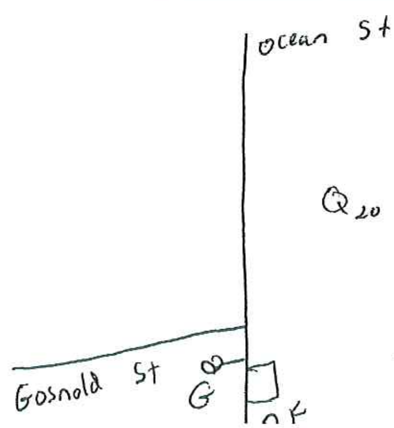
Required Flow: ____________________

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
Gage	Static:	95	Static:	98
	Residual:	50	Residual:	53
Flow	Static:	98	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5		8		25	

Flow available at 20 psi 1129 gpm. $\Delta s = 3$

Sketch & Remarks



$$Q = 29.83\,(.9)(2.5)^2\sqrt{25}$$
$$= 838.969$$

$$Q_{20} = 838.969\left[\frac{98-20^{\,.54}}{98-53^{\,.54}}\right]$$

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ______ Date: ______ Time: 1:20 PM

12 Test Number: ~~12~~ 19 Inspector: ______

City: ______ State: ______

Location

Zone: ______

Streets: ______

Weather: ______

Sources of supply in operations and rates: ______

Tank levels: ______

Type of development in the area: Residential

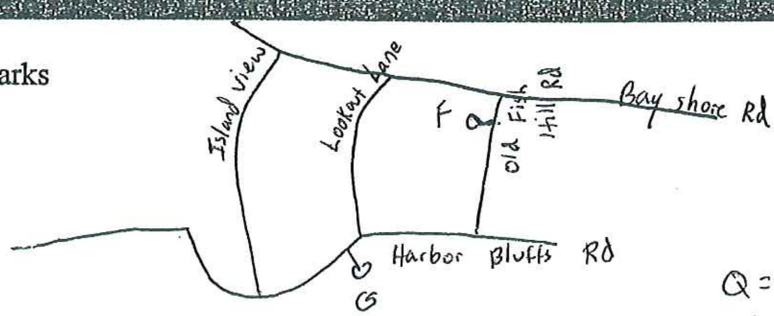
Required Flow: ______

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
	Static:	95 ~~[illegible]~~ ~~[illegible]~~	Static:	94
	Residual:	38	Residual:	37
	Static:	94	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5		8"		30	

Flow available at 20 psi 1058 gpm.

Sketch & Remarks



$$Q = 29.83(.9)(2.5)^2\sqrt{30}$$
$$= 919.04$$

$$Q_{20} = 919\left[\frac{94-20^{.54}}{94-37^{.54}}\right]$$

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ______ Date: ______ Time: 1:35 PM

13 Test Number: 17 Inspector: ______

City: ______ State: ______

Location

Zone: ______

Streets: ______

Weather: ______

Sources of supply in operations and rates: ______

Tank levels: ______

Type of development in the area: Residential

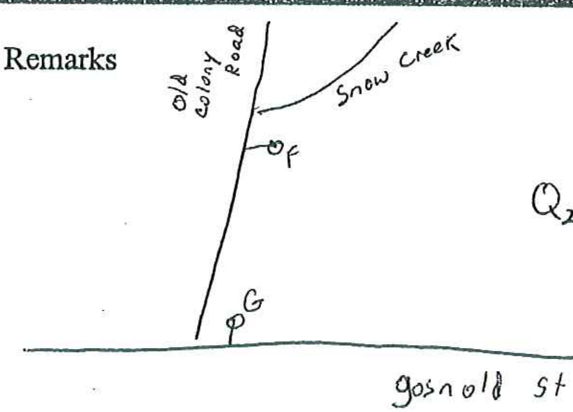
Required Flow: ______

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
	Static:	88	Static:	90
	Residual:	54	Residual:	56
	Static:	90	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5		8"		40	

Flow available at 20 psi 1567.33 gpm.

Sketch & Remarks



$$Q = 29.83(.9)(2.5)^2\sqrt{40}$$
$$= 1061.22$$

$$Q_{20} = 1061\left[\frac{90-20^{.54}}{90-56^{.54}}\right]$$

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ______ Date: ______ Time: 1:50 PM

14 Test Number: 12 Inspector: ______

City: ______ State: ______

Location

Zone: ______

Streets: ______

Weather: ______

Sources of supply in operations and rates: ______

Tank levels: ______

Type of development in the area: Residential (Light)

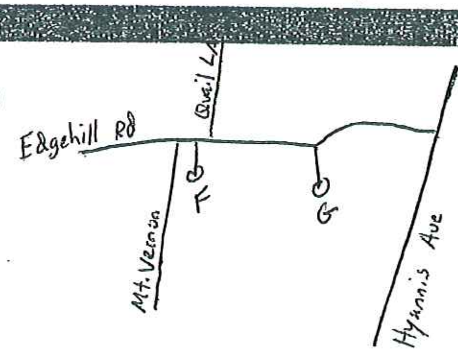
Required Flow: ______

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
______	Static:	85	Static:	84
______	Residual:	53	Residual:	52
______	Static:	84	Static:	
______	Residual:		Residual:	
______	Static:		Static:	
______	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5		4"		23	

Flow available at 20 psi 1170 gpm.

Dirty Water
Sketch & Remarks



$$Q = 29.83\,(.9)\,(2.5)^2\sqrt{23}$$

$$\approx 804.71$$

$$Q_{20} = 805\left[\frac{84-20^{.54}}{84-52^{.54}}\right]$$

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: 1 Date: 11/1/06 Time: 8:50 AM

16 Test Number: 15 Inspector:

City: State:

Location

Zone:

Streets: Main Street and Bassett Lane

Weather: Rain

Sources of supply in operations and rates:

Tank levels:

Type of development in the area: Commercial

Required Flow:

Hydrant No.		Observed Pressure (psi)		Corrected Pressure (psi)
Gage	Static:	~~80~~	Static:	84
	Residual:	65	Residual:	69
Flow	Static:	84	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2x 2.5		~~8~~"		30 x 2	

5034

Flow available at 20 psi ~~662~~ 4023 gpm. $\Delta s = 4$

Sketch & Remarks

$Q = 29.83\,(.9)\,(2 \times 2.5)^2 \sqrt{60}$ 30

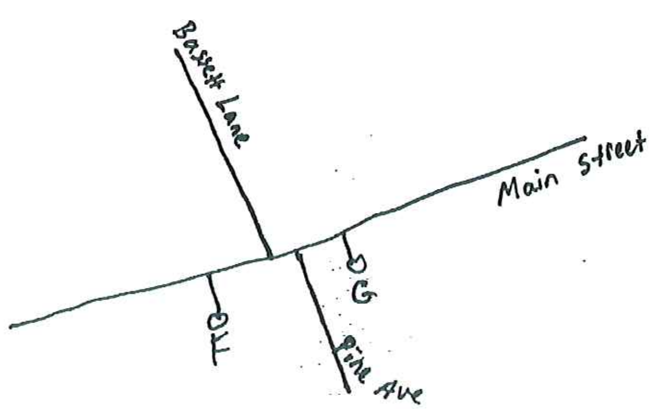
$= \cancel{5198.9}$

$919 \times 2 = 1838$

$Q_{20} = \cancel{5199} \left[\dfrac{84-20^{.54}}{84-69^{.54}} \right]$ 9,448 4,316

~~919~~

1838



Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ______ Date: 11/1/06 Time: 9:00 AM

17 Test Number: 16 Inspector: ______

City: ______ State: ______

Location

Zone: ______

Streets: North St. and Stevens Street

Weather: Rain

Sources of supply in operations and rates: ______

Tank levels: ______

Type of development in the area: Commercial

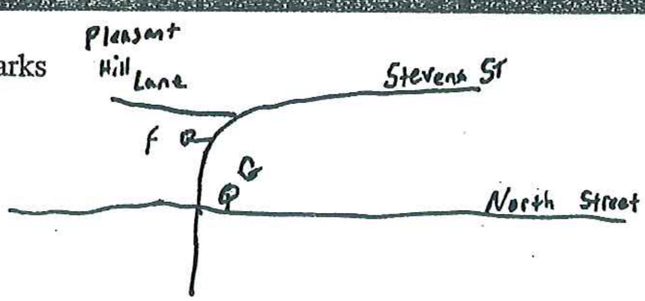
Required Flow: ______

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
Gage #14	Static:	87	Static:	86
	Residual:	75	Residual:	74
Flow	Static:	86	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5		~~8"~~		70	

Flow available at 20 psi 3525 gpm.

Sketch & Remarks



$Q = 29.83(.9)(2.5)^2\sqrt{70}$

$= 1403.86$

$Q_{20} = \left[\frac{86-20^{.54}}{86-74^{.54}}\right]$

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ______ Date: 11/1/06 Time: 9:10 AM

18 Test Number: 13 Inspector: ______

City: ______ State: ______

Location

Zone: ______

Streets: Pitchers Way and Elizabeth Lane

Weather: Rain

Sources of supply in operations and rates: ______

Tank levels: ______

Type of development in the area: Commercial

Required Flow: ______

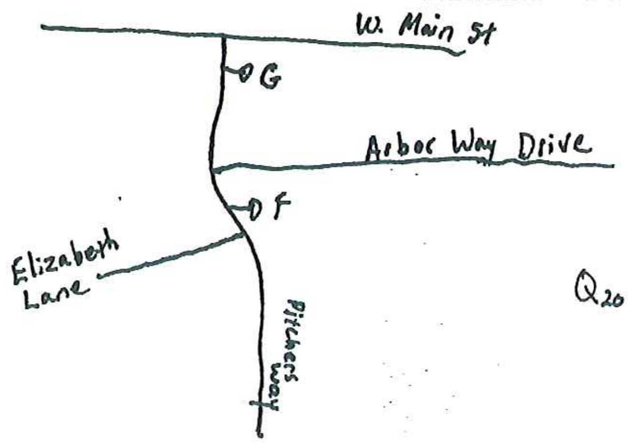
Hydrant No.		Observed Pressure (psi)		Corrected Pressure (psi)
Gage 237	Static:	82	Static:	86
	Residual:	72	Residual:	76
Flow	Static:	86	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5		8"		52	

Flow available at 20 psi 3352 gpm.

Dirty Water

Sketch & Remarks



$Q = 29.83(.9)(2.5)^2\sqrt{52}$

$= 1210$

$Q_{20} = 1210\left[\frac{(86-20)^{.54}}{(86-76)^{.54}}\right]$

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ____________ Date: 11/1/06 Time: 9:25 AM

19 Test Number: 14 Inspector: ____________

City: ____________ State: ____________

Location

Zone: ____________

Streets: Pitchers Way and Wayland Road

Weather: Rain

Sources of supply in operations and rates: ____________

Tank levels: ____________

Type of development in the area: Residential

Required Flow: ____________

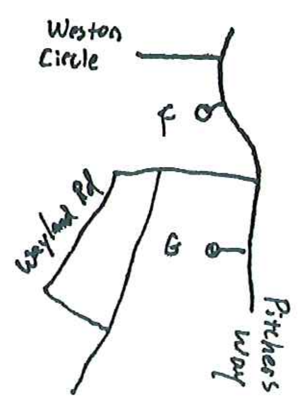
Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
Gage 541	Static:	74	Static:	73
	Residual:	50	Residual:	49
Flow	Static:	73	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	4.5		8"		19	

Flow available at 20 psi 3635 gpm.

Dirty Water

Sketch & Remarks



$Q = 29.83\,(.9)(4.5)^2\sqrt{19}$

$= 2370$

$$Q_{20} = 2370\left[\frac{73-20^{.54}}{73-49^{.54}}\right]$$

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ____________ Date: 11/1/06 Time: 9:50 AM

20 Test Number: 8 Inspector: ____________

City: ____________ State: ____________

Location

Zone: ____________

Streets: W. Main St and High School

Weather: Clear, wet

Sources of supply in operations and rates: ____________

Tank levels: ____________

Type of development in the area: High School / Commercial

Required Flow: ____________

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
Gage 702	Static:	75	Static:	74
	Residual:	40	Residual:	39
Flow 56	Static:	74	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

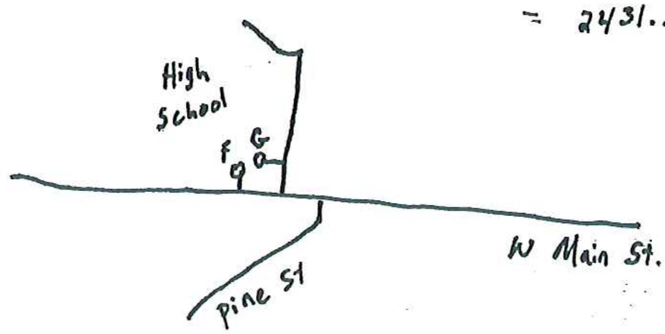
Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	4.5		8"		20	

Flow available at 20 psi 3073 gpm.

Sketch & Remarks

$Q = 29.83\,(.9)(4.5)^2\sqrt{20}$
$= 2431.28$

$Q_{20} = 2431\left[\frac{74-20^{\,.54}}{74-39^{\,.54}}\right]$



Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ______ Date: 11/1/06 Time: 10:10 AM

21 Test Number: 9 Inspector: ______

City: ______ State: ______

Location

Zone: ______

Streets: Old Craigville Road and Elementary School

Weather: Sunny, wet

Sources of supply in operations and rates: ______

Tank levels: ______

Type of development in the area: Elementary School / Light Commercial

Required Flow: ______

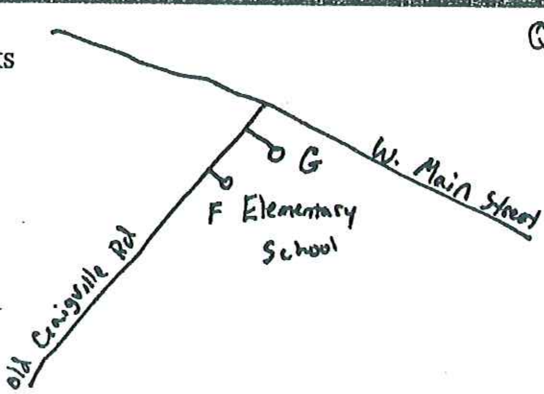
Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
Gage 258	Static:	76	Static:	78
	Residual:	40	Residual:	42
Flow 257	Static:	78	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	4.5		8"		10	

Flow available at 20 psi 2224 gpm.

Dirty Water

Sketch & Remarks



$Q = 29.83(.9)(4.5)^2\sqrt{10} = 1719.18$

$Q_{20} = 1719\left[\frac{78-20^{.54}}{78-42^{.54}}\right]$

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: ______ Date: 11/1/06 Time: 10:30 AM

22 Test Number: 10 Inspector: ______

City: ______ State: ______

Location

Zone: ______

Streets: Ocean ~~Street~~ Ave. and Sixth Street

Weather: Sunny

Sources of supply in operations and rates: ______

Tank levels: ______

Type of development in the area: Residential

Required Flow: ______

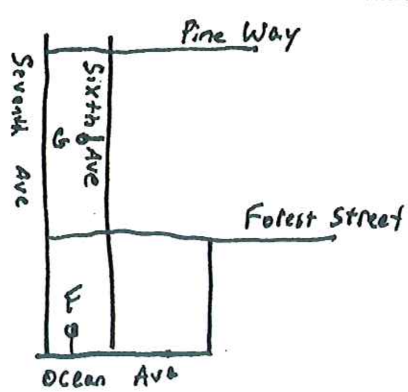
Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
Gage # 28	Static:	93	Static:	92
	Residual:	45	Residual:	44
Flow 27	Static:	92	Static:	
	Residual:		Residual:	
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5		8"		20	

Flow available at 20 psi 934 gpm.

Dirty Water

Sketch & Remarks



$$Q = 29.83(.9)(2.5)^2\sqrt{20} = 750.3$$

$$Q_{20} = 750\left[\frac{92-20^{.54}}{92-44^{.54}}\right]$$

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: BARNSTABLE Date: 11/29/06 Time: 11:00
Test Number: Inspector: YKJ
City: BARNSTABLE State: MA
Location
Zone: –
Streets: Main Street @ Pearl Street
Weather:
Sources of supply in operations and rates:
Tank levels:
Type of development in the area:
Required Flow:

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
1	Static:	80	Static:	
Gauge 2	Residual:	69 (64)	Residual:	
2	Static:	80	Static:	
Gauge	Residual:	61	Residual:	61
	Static:		Static:	
	Residual:		Residual:	

10.77

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5"	0.9	12"			

Flow available at 20 psi ____________ gpm.

$Q = 1235$

length 302

Sketch & Remarks

$\Delta R = 3$ psi
$= 6.93$ feet

Headloss $G = 6.93 \times \frac{1000}{302 \text{ (length)}} = 22.94$

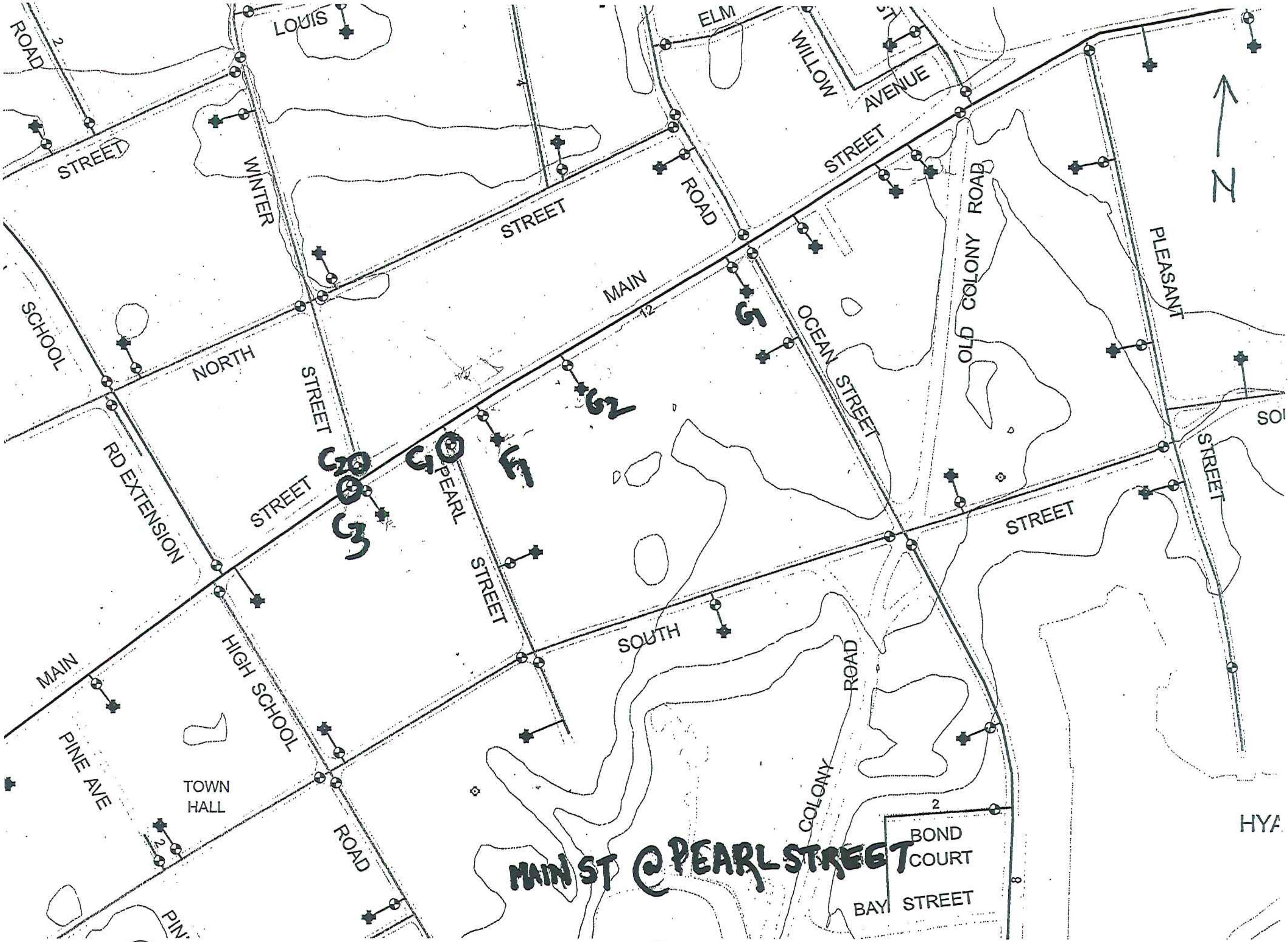
~~4437.14 per 1000'~~

183558.05
3744.82
= 49

$C = \frac{148.63\,(1235)\ Q}{(22.94)^{0.5405}\,(12)^{2.6298}} = 49$

headloss

G1 Mark
G2 YJ
f Eric



N
MAIN ST @ PEARL STREET
MAIN STREET
PINE AVE
TOWN HALL
HIGH SCHOOL ROAD
SCHOOL RD EXTENSION
NORTH STREET
WINTER STREET
LOUIS
PEARL STREET
SOUTH STREET
COLONY ROAD
BOND COURT
BAY STREET
OCEAN STREET
ELM
WILLOW AVENUE
OLD COLONY ROAD
PLEASANT STREET

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: BARNSTABLE Date: 11/29/05 Time: 9:50
Test Number: Inspector: YKJ
City: BARNSTABLE State: MA
Location
Zone:
Streets: Mary Dunn Rd @ Route 28
Weather:
Sources of supply in operations and rates:
Tank levels:
Type of development in the area:
Required Flow:

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
1	Static:	78	Static:	78
) Gauge	Residual:	66	Residual:	68
2	Static:	78	Static:	78
gauge	Residual:	67	Residual:	67
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5	0.9	16			

Flow available at 20 psi ____________ gpm. Q = 1220 gpm

Should be high C-value

Sketch & Remarks

$\Delta P = 1$ psi $= 2.31$ feet

Length = 798

bet hydran 182

YKJ

Main

1.77 349.31

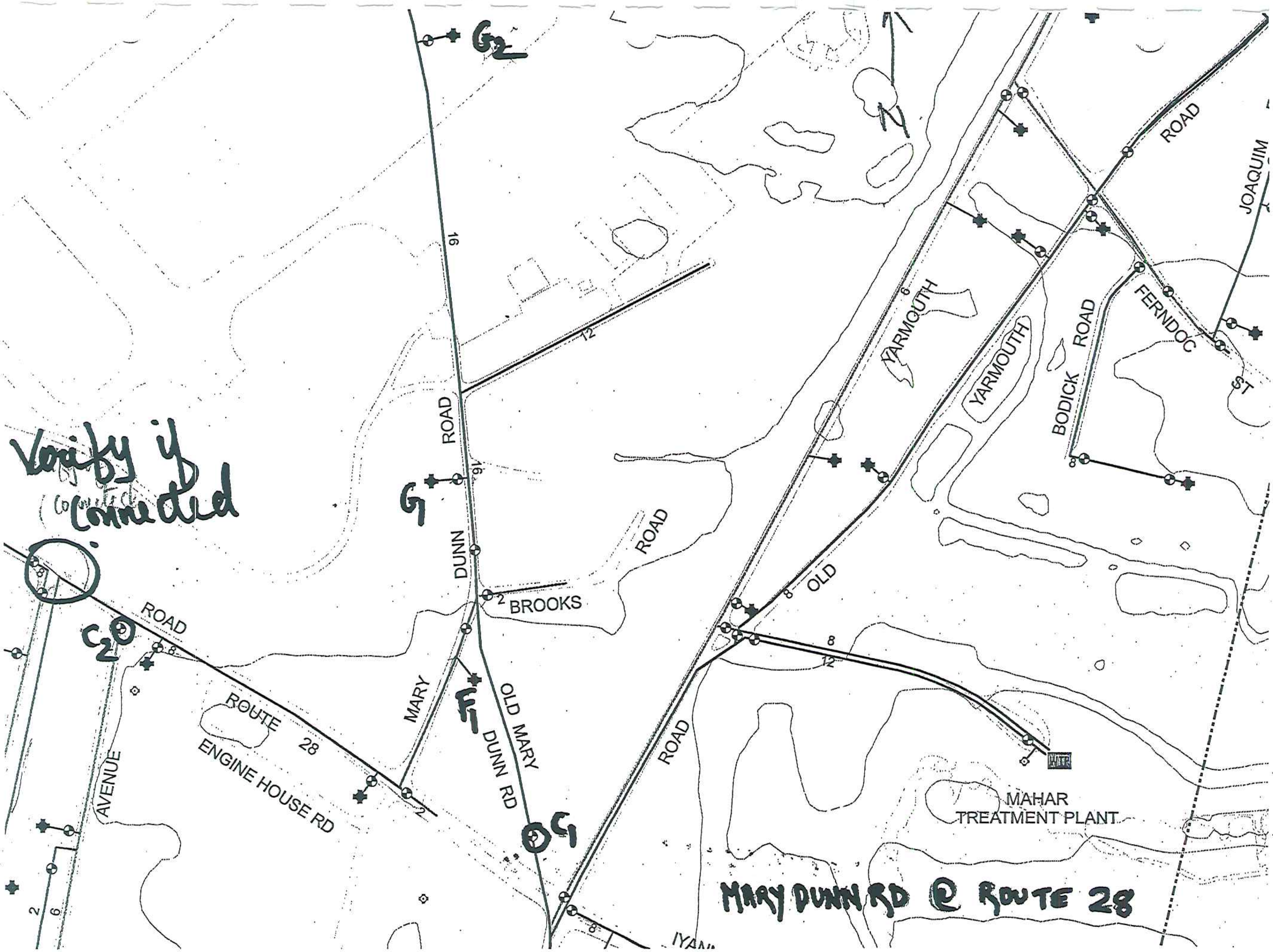
2.58

181328.6

260672

Headloss per 1000' $S = \frac{2.31 \times 1000}{798' \text{ (length)}} = 2.89$

$C = \frac{148.63 \times (1220)}{(2.89)^{0.5405} \times (16)^{2.6298}} = 70$



G2
ROAD
JOAQUIM
16
12
YARMOUTH
6
YARMOUTH
BODICK ROAD
FERNDOC
ST
ROAD
G1
DUNN
Verify if connected
8
ROAD
BROOKS
2
OLD
8
8
12
C2
ROUTE
28
ENGINE HOUSE RD
AVENUE
MARY
F1
DUNN RD
OLD MARY
C1
ROAD
MAHAR TREATMENT PLANT
MARY DUNN RD @ ROUTE 28
2
6

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: BARNSTABLE Date: 11/29/06 Time: 2:20
Test Number: Inspector:
City: BARNSTABLE State:
Location
Zone: —
Streets: Ocean Avenue @ Hyannis Ave
Weather:
Sources of supply in operations and rates:
Tank levels:
Type of development in the area:
Required Flow:

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
1	Static:	88	Static:	88
2	Residual:	54	Residual:	54
	Static:	88	Static:	88
	Residual:	52	Residual:	52
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5	0.9	12"			

Flow available at 20 psi ______ gpm.

Sketch & Remarks

$\Delta R = 2$ psi $= 4.62$ feet

length $= 647'$

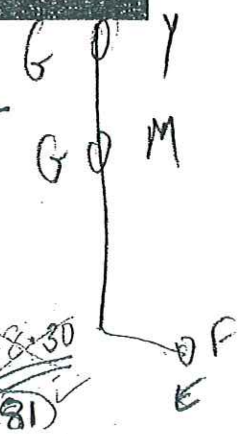
$Q = 1085$

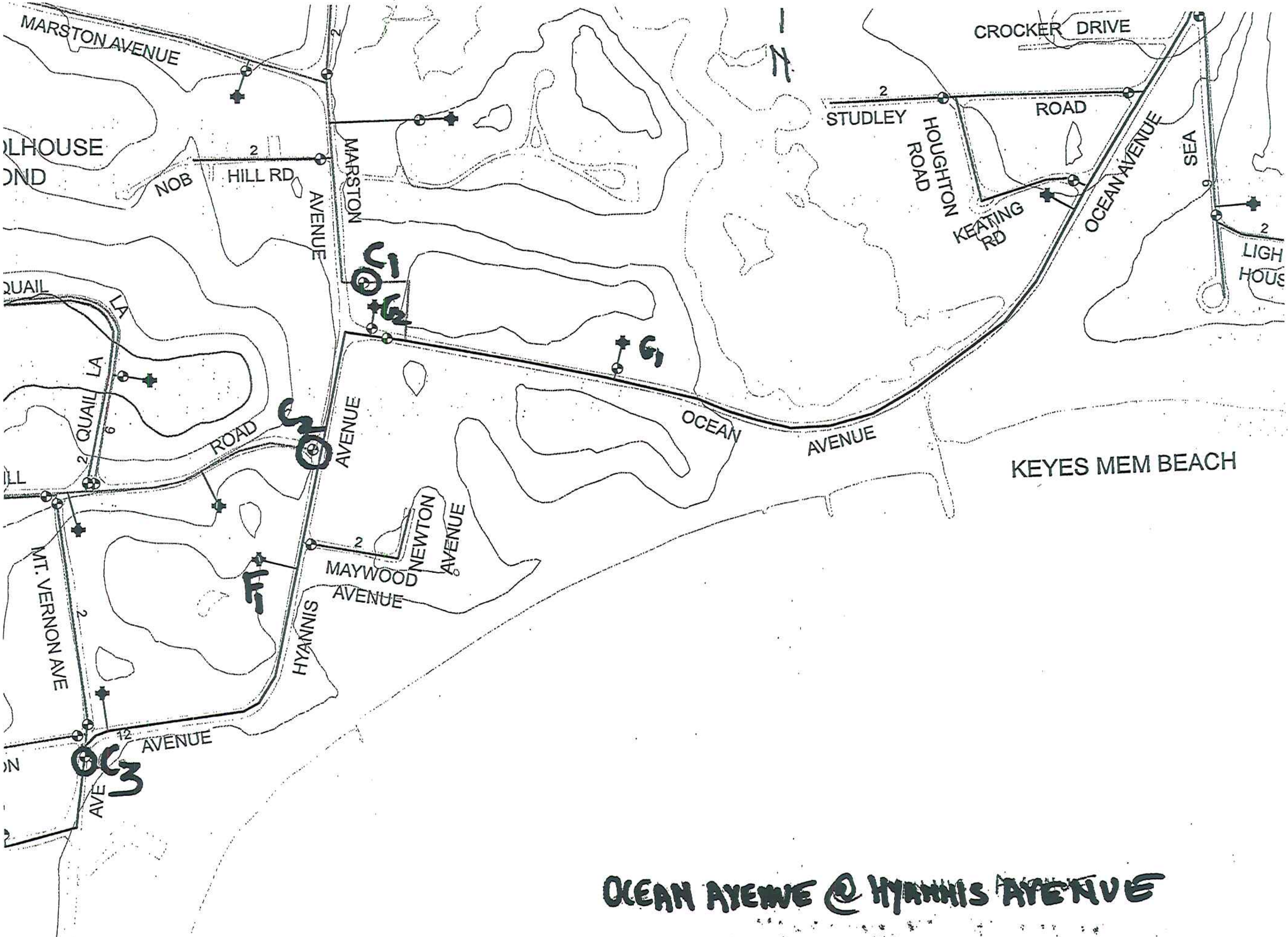
Headloss per 1000' $G = 4.62 \times \frac{1000}{647 \text{ (length)}} = 7.14$

$C = \frac{148.63 \times (1085)}{(7.14)^{0.5405} \times (12)^{2.6298}} = \cancel{68.30}$ (81)

$\frac{161263.55}{1992.85} = 80.92$

$\frac{161263.55}{2360.89} = 68.30$





OCEAN AVENUE @ HYANNIS AVENUE

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: BARNSTABLE Date: 11/29/06 Time: 11:45
Test Number: Inspector:
City: BARNSTABLE State:
Location
Zone:
Streets: Pitcher's Way @ Beth Lane
Weather:
Sources of supply in operations and rates:
Tank levels:
Type of development in the area:
Required Flow:

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
1	Static:	68	Static:	68
Gauge 2	Residual:	48	Residual:	48
	Static:	68	Static:	68
Gauge	Residual:	46	Residual:	46
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5"	0.9	10"			

Flow available at 20 psi ____________ gpm.

Sketch & Remarks

$\Delta R = 2$ psi
$\Delta R = 4.62$ feet

Length = 625

Q = 1040

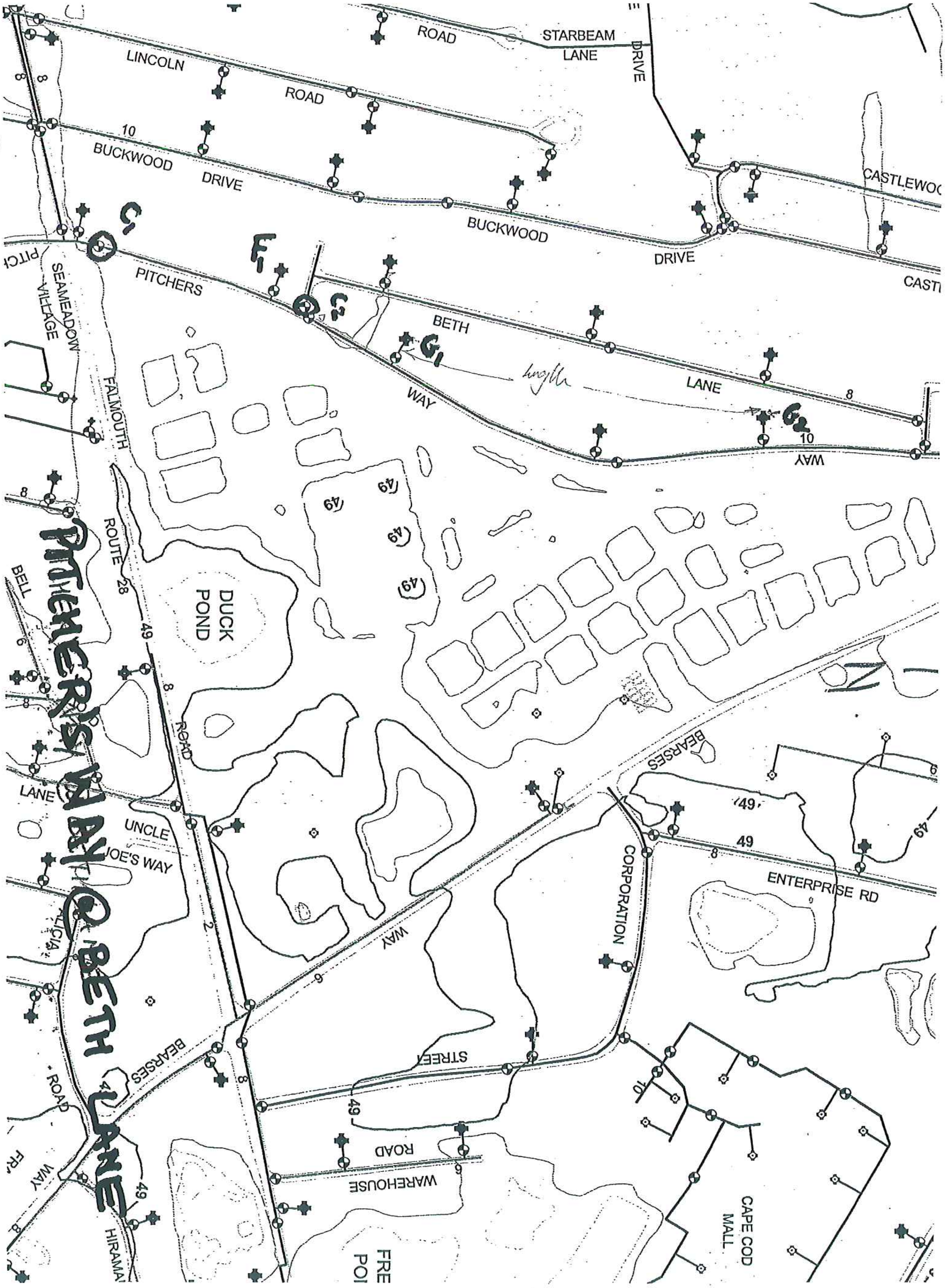
Head loss per 1000' $S = \frac{4.62 \times 1000'}{625 \text{ (length)}} = 7.4$

1471.70

$C = \frac{148.63 \, (1040)}{(7.4)^{0.5405} \, (10)^{2.6298}} = 122.89$

$\frac{154575.2}{1257.82} = 122.89$

G Y.
G M
F ?



STARBEAM LANE
LINCOLN ROAD
BUCKWOOD DRIVE
CASTLEWOOD
PITCHERS WAY
BETH LANE
SEAMEADOW VILLAGE
FALMOUTH ROUTE 28 ROAD
DUCK POND
BELL LANE
UNCLE JOE'S WAY
BEARSES WAY
CORPORATION STREET
ENTERPRISE RD
WAREHOUSE ROAD
CAPE COD MALL
PITCHER'S WAY @ BETH LANE
C1
C2
E1
G1
G2

Weston & Sampson Engineers, Inc.
5 Centennial Drive
Peabody, Massachusetts 01960-7906
Tel: (978-532-1900 Fax: (978)977-0100

Hydrant Test Report

Project: BARNSTABLE Date: 11/29/06 Time: 12:45
Test Number: Inspector: YKJ
City: BARNSTABLE State: MA
Location
Zone:
Streets: South Street @ Pine Avenue
Weather:
Sources of supply in operations and rates:
Tank levels:
Type of development in the area:
Required Flow:

Hydrant No.	Observed Pressure (psi)		Corrected Pressure (psi)	
1	Static:	80	Static:	
2	Residual:	72	Residual:	
2	Static:	80	Static:	
	Residual:	34	Residual:	34
	Static:		Static:	
	Residual:		Residual:	

Hyd. No.	Outlet Diam	Coeff.	Main Size	Static Pressure	Pitot Pressure	Flow (gpm)
	2.5	0.9	6"			

Flow available at 20 psi ____________ gpm.

Sketch & Remarks

$\Delta_B = 38$ psi
$\Delta_R = 87.78$ feet

Length = 850'

$Q = 390$

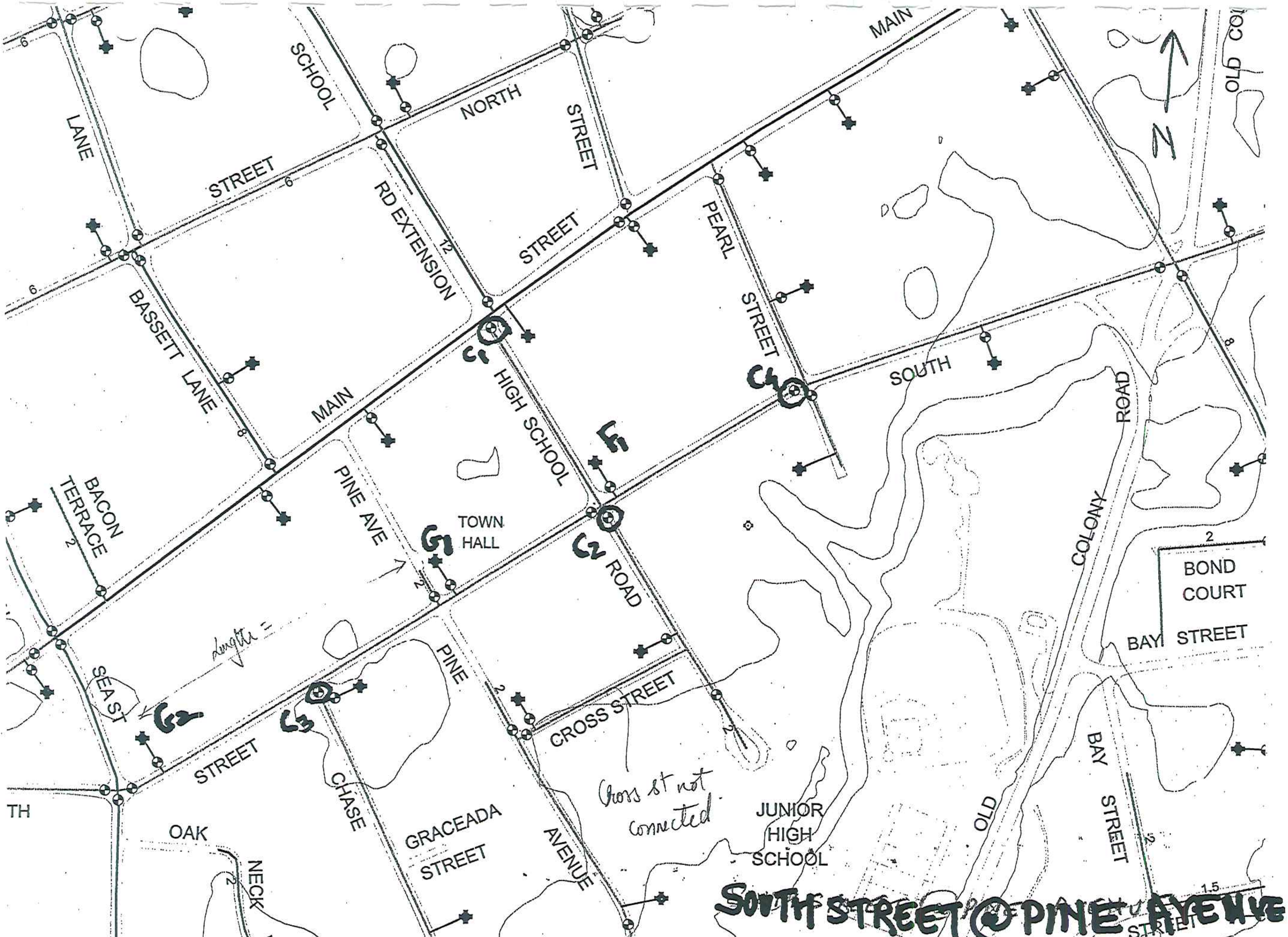
Headloss per 1000' $= 87.78 \times \frac{1000}{850 \text{ (length)}} = 103.27$

$C = \frac{148.63 \times (390)^{s}}{(103.27)^{0.5405}(6)^{2.6298}} = 42.48$

$\frac{57985.7}{1364.37}$

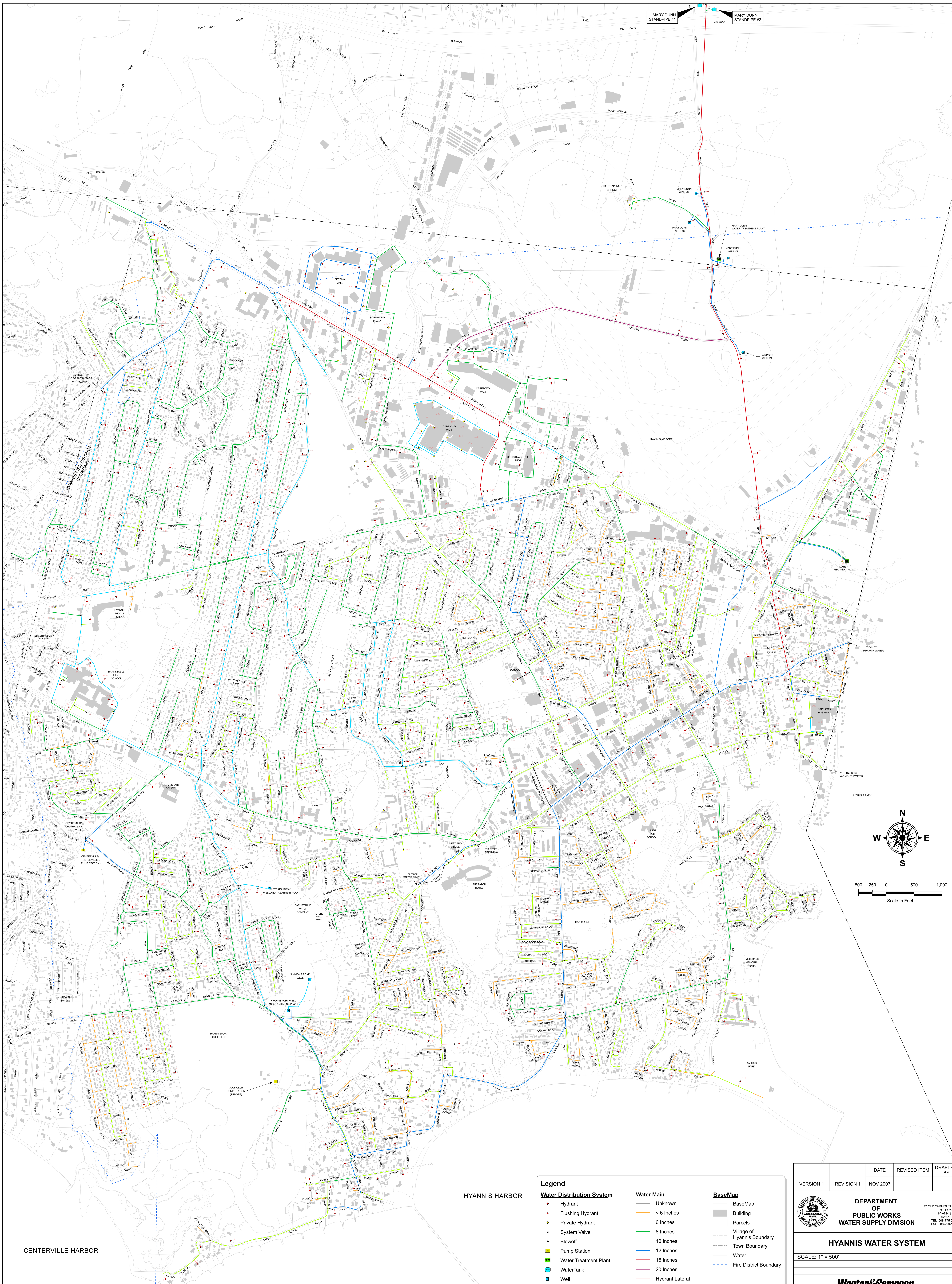
$\frac{57965.7}{1541.72}$

HL


SOUTH STREET @ PINE AVENUE
MAIN
MAIN
SCHOOL
STREET
LANE
NORTH
STREET
STREET
RD EXTENSION
HIGH SCHOOL
ROAD
BASSETT
LANE
PEARL
STREET
SOUTH
STREET
PINE AVE
PINE
AVENUE
TOWN
HALL
BACON
TERRACE
SEA ST
CROSS STREET
CHASE
GRACEADA
STREET
OAK
NECK
JUNIOR
HIGH
SCHOOL
OLD
COLONY
ROAD
BAY
STREET
BAY STREET
BOND
COURT
OLD CO
N
C1
C2
C3
C4
G1
G2
G5
Length =
Cross st not connected

Appendix B

Existing Distribution System Map



		DATE	REVISED ITEM	DRAFTED BY
VERSION 1	REVISION 1	NOV 2007		

DEPARTMENT OF PUBLIC WORKS WATER SUPPLY DIVISION

47 OLD YARMOUTH RD
P.O. BOX 326
HYANNIS, MA
02601-0326
TEL: 508-775-0063
FAX: 508-790-1313

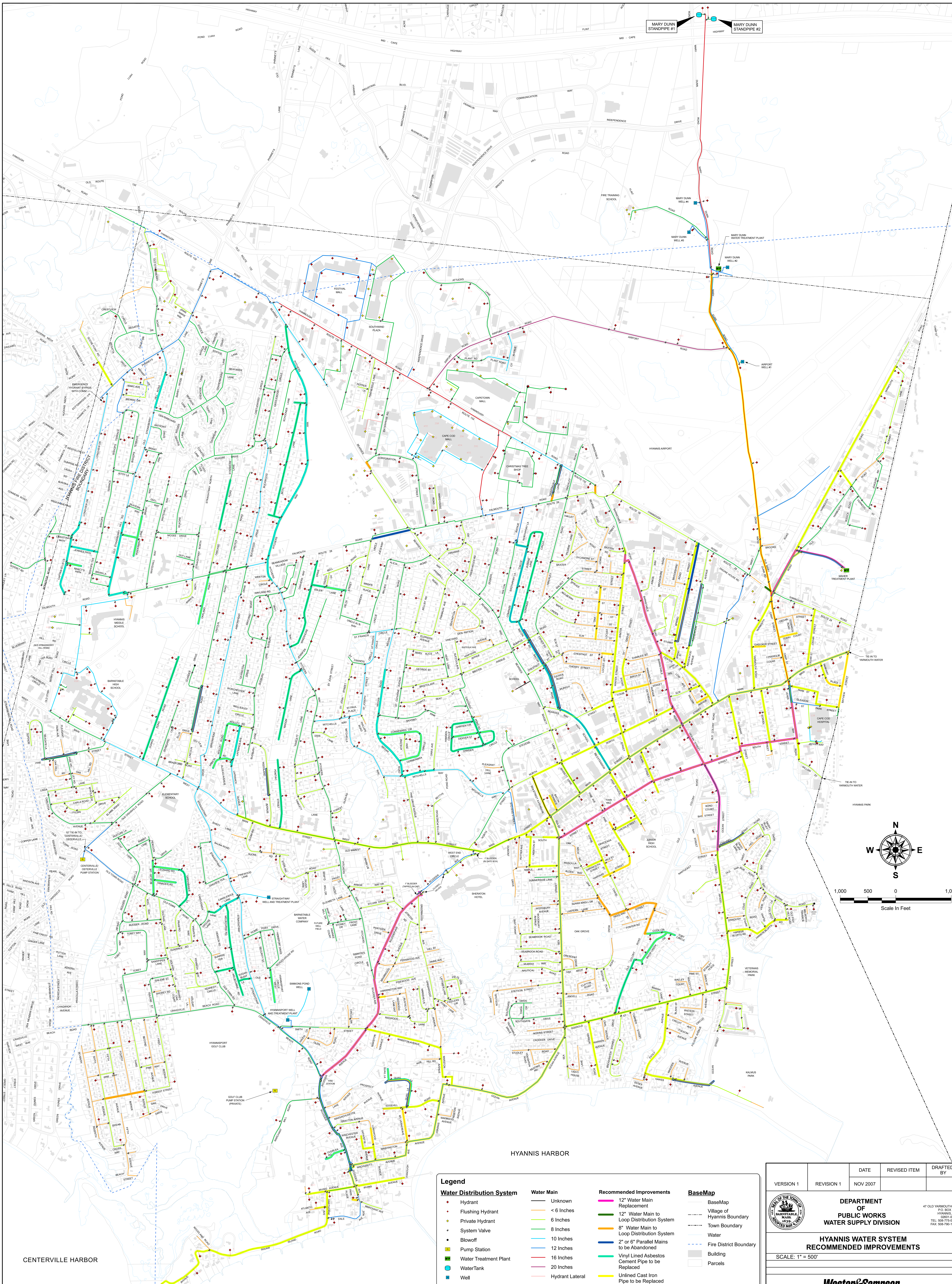
HYANNIS WATER SYSTEM

SCALE: 1" = 500'

Weston&Sampson®

Appendix C

Recommended Improvements Map



		DATE	REVISED ITEM	DRAFTED BY
VERSION 1	REVISION 1	NOV 2007		

DEPARTMENT OF PUBLIC WORKS WATER SUPPLY DIVISION

47 OLD YARMOUTH RD
P.O. BOX 326
HYANNIS, MA
02601-0326
TEL: 508-775-0063
FAX: 508-790-1313

HYANNIS WATER SYSTEM RECOMMENDED IMPROVEMENTS

SCALE: 1" = 500'

Weston&Sampson®

Appendix D

DEP Guidance Document for Water Management Act Permitting Policy

COMMONWEALTH OF MASSACHUSETTS
EXECUTIVE OFFICE OF ENVIRONMENTAL AFFAIRS
DEPARTMENT OF ENVIRONMENTAL PROTECTION
ONE WINTER STREET, BOSTON, MA 02108 617-292-5500

MITT ROMNEY
Governor

KERRY HEALEY
Lieutenant Governor

ELLEN ROY HERZFELDER
Secretary

ROBERT W. GOLLEDGE, Jr.
Commissioner

Guidance #BRP/DWM/DW/G04-1

Guidance Document for Water Management Act Permitting Policy

Permit and Permit Amendment Applications and 5-Year Reviews

April 2, 2004

I. DESCRIPTION OF STANDARDS AND CONDITIONS

1. RESIDENTIAL PER CAPITA WATER USE

The standard will be not more than 65 gallons per day for residential gallons per capita day (rgpcd) water use for High or Medium stressed basins as defined with the Stressed Basin Report.[1] The rgpcd standard for High and Medium stressed basins reflects the Water Resources Commission's performance standard for Interbasin Transfer Approval for effective water conservation for public water suppliers (PWS). For those PWS not meeting this requirement at the time of permit issuance, the Department will allow the PWS adequate opportunity to meet this requirement by extending the period for achieving compliance with this condition until the filing of the 2nd Annual Statistical Report (ASR) from the permit issuance date. For those PWS in basins identified as being either Low stressed or Unassessed, the performance standard to be included in new permits or permit amendments for residential per capita water use will be not more than 80 rgpcd. The Department reserves the right to require the standard of 65 rgpcd in Low or Unassessed basins in new permits, permit amendments or permit renewals where site specific impacts warrant the more stringent performance standard.

2. UNACCOUNTED-FOR WATER

The standard will be not more than 10% unaccounted–for water for High and Medium stressed basins. This requirement also reflects the Water Resources Commission's performance standard for Interbasin Transfer Approval for effective water conservation for PWS. The Commission defines unaccounted-for water as the difference between water pumped or purchased and water that is metered or confidently estimated. Unaccounted-for water should include: master meter

[1] Stressed Basins in Massachusetts, Water Resources Commission, December 13, 2001

inaccuracies, domestic and non-domestic meter under registration, errors in estimating for stopped meters, over-registering revenue meters, unauthorized hydrant openings, unavoidable leakage, recoverable leakage, illegal connections, standpipe overflows, data processing errors, and fire protection, unless this water is metered or confidently estimated. Timelines for achieving compliance with this condition will be consistent with those outlined for residential per capita water use, above. For those PWS in basins identified as being either Low stressed or Unassessed, the performance standard to be included in new permits or permit amendments for unaccounted-for water will be not more than 15%. The Department reserves the right to require the standard of not more than 10% in Low or Unassessed basins in new permits, permit amendments or permit renewals where site specific impacts warrant the more stringent performance standard.

3. SUMMER WITHDRAWAL CAP

The Department will require that a summer withdrawal cap be included in permits issued in High or Medium stressed basins to reduce the difference between summer (May through September) and previous winter (November through March) withdrawals. The withdrawal cap will be derived from each PWS's average summer to winter withdrawal ratio for the last three seasons as reported in the ASRs. Those PWS with an average summer to winter ratio of 1.4 or greater will be required to reduce the summer-winter difference by 50% from the highest summer water use during that period, which will be reflected in the permit as a seasonal summer water usage not to exceed a specified volume. The PWS that has an average summer to winter withdrawal ratio of less than 1.4 and greater than 1.2 will be required to reduce that summer-winter withdrawal difference by 25% from the highest summer water use during that period. For PWS with an average ratio of 1.2 or less, the summer limit will not be a specified volume cap but a requirement to maintain a summer to winter ratio of 1.2 or less. The Department will provide adequate opportunity to meet this condition by extending the compliance deadline until the filing of the second ASR after permit issuance. The Department may require reporting with sufficient frequency to obtain more immediate data on seasonal use.

4. LIMITS ON NONESSENTIAL OUTDOOR WATER USE

Based on the aquatic habitat needs of the stream segments potentially impacted by a specific withdrawal, the Department will require in permits issued to both PWS and other type of uses in High and Medium stressed basins, that specific demand management strategies be implemented under identified streamflow conditions. For purposes of this Policy the relevant stream flow thresholds triggering mandatory restrictions will be the US Fish and Wildlife's New England Aquatic Base Flow (ABF) default value of 0.50 cubic feet per second per square mile (cfsm), until such time that USGS, DCR, WRC, or DFW publish new stream flow standards that the Department adopts, or unless the Department had previously adopted a stream flow threshold that continues to apply to the basin. For the purpose of this condition, stream flow shall be measured at a suitable in-basin USGS stream gauge, or other monitoring point/measure as deemed appropriate by DEP. For permitted withdrawals, this will mean that at the identified stream flow threshold, mandatory restrictions on outside nonessential water use will be required.

The permit condition will require a PWS, or the municipality in which the water is used, to implement and enforce mandatory restrictions on nonessential outside water use whenever streamflow falls below the value identified by the Department for three consecutive days in the period from May 1st through September 30th. Those restrictions will include, at a minimum, limiting outside water use to hand-held hoses only and prohibiting outside watering between 9 AM and 5 PM, when evapotranspiration is typically the highest. Notwithstanding the foregoing, irrigation of public parks and recreational fields by means of automatic sprinklers equipped with moisture sensors or similar control technology may also be permitted if not between the hours of 9AM to 5 PM. Such mandatory restrictions must include the authority to assess penalties or impose fines for violations.

For permits issued to users other than a PWS, a plan restricting nonessential outside water use may need to be developed for the Department's review and approval for inclusion as a condition of the permit.

The term "nonessential outside water use" includes uses that do not have health or safety impacts, are not required by regulation, and are not needed to meet the core functions of a business or other organization. Examples of nonessential outside water uses include irrigation of lawns and ornamental plants; washing of vehicles unless necessary for operator safety, washing of building exteriors, outside structures, streets, sidewalks, and parking lots; the filling of swimming pools and hot tubs; and the operation of decorative pools and fountains. Examples of essential outside water uses may include water use for the production of food and fiber and the maintenance of livestock and poultry; outside water use by plant nurseries to maintain their stock; the watering of golf course greens; the washing of vehicles by commercial car washes, maintenance facilities, and dealers; and the washing of exterior building surfaces including windows, parking lots, driveways or sidewalks, prior to application of paint, preservatives, or stucco, or for the preparation of the surface prior to paving or repointing of bricks, or if required by health and safety regulations.

The Department has determined that restrictions by a PWS on nonessential outside water use may result in an increase in the number and use of private wells used partially or entirely for irrigation. Since these wells typically draw ultimately on the same water source as the public water system, installation and use of unregulated private wells can undermine efforts to reduce overall water use and protect water resources. In an attempt to protect water resources where private well use is not controlled, DEP may include a provision in the permit that requires a nonessential outside water use ban on customers of the PWS, rather than just mandatory controls, when specified streamflow thresholds occur. Such a provision will be considered for communities where private well proliferation has or may occur, and may significantly affect the water resource protection benefits of the permit, and would be imposed to offset the water losses that such uncontrolled use of private wells may cause. Communities that control private well use to the same extent as PWS use would not be subject to this additional provision because such communities can better control nonessential outdoor water use when stream flows reach low levels.

5. REPORTING REQUIREMENTS

To accurately assess compliance with all Standards and Conditions, the Department will condition all permits to require standard and consistent reporting of data. Permits will be conditioned to require that all permit holders report the raw water volumes withdrawn from individual sources and that a finished water volume for the entire system also be provided. In addition, for PWS the ASR will specifically require a detailed account of how the unaccounted-for water was calculated and specific information on how residential gallons per capita day was calculated.

6. Streamflow monitoring

Streamflow monitoring will be performed through the USGS stream gage network throughout the Commonwealth. However, where monitoring of specific sensitive receptors may already be a permit condition, or where site-specific monitoring of streamflow, pond elevations or wetlands may be appropriate, such additional monitoring may be used and where appropriate, required as a permit condition to help evaluate impacts for required mitigation as noted herein, including installation of additional stream gages, along with monitoring and reporting requirements on stream flow.

II. Review of permit and permit amendment applications

Permit or permit amendment applications seeking new sources and/or requesting increased authorized withdrawal volumes through a permit application must evaluate those sources and increased withdrawals based on the method described in the "Site Screening Process for Siting a New or Expanded Source of Public Water – February 5, 2001." For a description of the site screening process see DEP webpage: *www.state.ma.us/dep/brp/wtrm/sitescr.htm.* The Site Screening process is a desktop screening tool to evaluate a withdrawal impact from a groundwater source on an unregulated stream, and will also be used to evaluate authorized withdrawal increases from multiple existing sources.

The withdrawal volumes proposed by a permit application for an increase in volumes from an existing source(s), or through adding a new source through a permit or permit amendment, will be compared to natural streamflow. Where the water withdrawal volume is equal to or greater than 50% of natural August Median flow in a nearby stream as evaluated with USGS STREAMSTATS within the Site Screening method, applicants will be advised that the application will receive additional scrutiny. Withdrawals equal to or greater than 50% of natural August Median flow are considered to have significant flow impact and will encounter more rigorous review by the Department, including more detailed instrumentation for pumping test design, needs assessment, and alternatives analysis.

Where the natural August Median flow cannot be determined through STREAMSTATS, a surrogate method acceptable to the Department, such as a reference basin transform or other streamflow analysis, may be submitted for the Department's review and approval.

III. Compensating for new or increased withdrawals

1. Operational restrictions and management plan

New withdrawal sources may be subject to specific operational restrictions reducing the capacity and/or limiting the period of operation of a source, based upon site-specific impacts. Such issues are now and will continue to be identified during the permit application review process. Permit holders with restrictions limiting the use or capacity of an individual source(s) will be required to complete a management plan that identifies how the permit holder will meet demand should the new source's availability be limited.

The Department will also consider allowing a PWS that would otherwise be required to limit or cease withdrawing from a new source to limit or cease pumping instead at an existing source, should it be demonstrated to the Department's satisfaction that such an approach is equally or more protective of the water resource and is necessary for good management of the system.

2. OFFSETS FOR INCREASING AUTHORIZED WITHDRAWAL VOLUMES

The Department has determined that to minimize the impacts of increasing authorized withdrawal volumes in High and Medium stressed basins, it will require that an offset feasibility study be performed and submitted to the Department for approval. The Department also reserves the right to require offset feasibility studies in Low stressed or Unassessed basins where site-specific impacts warrant such offsets. The offset study shall evaluate the feasibility of reducing water losses to the basin, and identify the most feasible means of maintaining local water balance. The Department has determined there are a wide variety of activities that can return water to the basin or prevent water loss in the basin, such as reduced infiltration and inflow, recharge of stormwater, and retrofit of existing development using low impact development principles. The policy is intended to encourage communities to use the most cost effective and locally appropriate method for "keeping water local."

The amount of water to be offset will be based on the extent to which the additional water withdrawn is lost to that basin. Additional water withdrawn is the difference between the requested withdrawal volume and current use (volumes used for the prior calendar year or the average for the prior 3 years, whichever is higher).

For the purpose of identifying the volumes necessary to be offset for this General Condition and item 3 below, the Department will define basin as that area upstream of the nearest USGS Gauge used for determining that basin's stress classification as identified in the WRC Stressed Basin Report. The ratio that should be applied to the offsets will be based on the classification of stress in which the basin of withdrawal is located and the feasible means to decrease water loss to the basin.

3. OFFSETS FOR EXISTING AUTHORIZATIONS DURING THE 5-YEAR REVIEW PROCESS

The Department will require permit holders in High and Medium stressed basins to provide, where appropriate, a detailed evaluation of their need for an increase in their allocated withdrawal volumes as authorized during the next 5 Year Review Period, and to provide an offset feasibility study, evaluating offset opportunities, including identification of the most feasible means of providing offsets.

The volume to be offset for 5 Year Reviews of existing permits will be based upon the difference between current use, provided said use is in compliance with the Water Management Act, and those volumes authorized in the next 5 Year permit period. Current use is defined as those volumes used for the prior calendar year, or the average for the prior three years, whichever is higher. The Department also reserves the right to require offsets in Low or Unassessed basins, or in other circumstances where site-specific impacts warrant such offsets.

The offset measures proposed by the applicant for a new permit with increasing withdrawals, or the feasibility study provided by an existing permit holder during a 5 Year Review, must be submitted for the Department's review and approval and may include offsets such as, Infiltration/Inflow (I/I) removal, wastewater return, stormwater management, and the water savings achieved by retrofitting existing development with Low Impact Development methods, or other methods of water return or preventing water loss that the applicant or existing permit holder can demonstrate will offset their increased volumes. The Department will also consider granting offset credits to PWS that choose to regulate private well water use in the same way as PWS water use.

The Department will, where determined feasible, require offsets for those volumes lost from the basin(s) of the proposed or existing withdrawal(s). Volumes lost are those volumes being removed from the basin in question via wastewater removal, an industrial/commercial process, evapotranspiration, or any other use that results in a reduction in the volumes returned to that basin. For the purposes of this policy the Department will assume that for PWSs, municipalities or other entities returning water to the basin of withdrawal via on-site septic systems or other wastewater disposal system, the percentage being credited as returned reflects only those volumes returned upstream of the USGS gage that determined the stressed designation. Those volumes being used seasonally for irrigation or other outside uses will be identified as water lost to the basin. To calculate the percentage for return, the Department will use the ratio established for determining a summer withdrawal cap to identify the percentage of loss expected between summer use (May-September) and previous winter use (November-March).

Additional water conservation, over and above what is required in the Policy can also be an effective means of avoiding the need for additional withdrawal volumes. While conservation is not an offset that allows additional withdrawals, it clearly is an effective means for reducing demand from existing users, allowing for additional growth without needing new sources or increased withdrawals of water.

The offsets required for increasing withdrawals – or those identified as a water "bank" – have historically achieved a savings by having the new user identify a potential water savings and then implement such a plan to achieve those savings. Alternatively, a new user may contribute financially to a dedicated account that can be used by the PWS/community to achieve the water savings. However, the Department may require the adoption of a water bank in systems that have difficulty meeting demand because of specific conditions that prevent a permitted increase in withdrawal volumes.

The Department reserves the right to use its discretion to vary from the standards and conditions outlined in this guidance when there is a public health and safety issue, environmental emergency or as otherwise appropriate.

As more detail is developed the Department will update and supplement this guidance.