Water Distribution System Study
for the Town of
West Newbury, Massachusetts

Water Department Copy

COMMONWEALTH OF MASSACHUSETTS
PAUL
B.
HOWARD
REGISTERED
PROFESSIONAL ENGINEER

Paul B. Howard
10/15/03

Prepared by

TATA & HOWARD, INC.
125 Turnpike Road
Westborough, MA 01581

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TATA & HOWARD
INCORPORATED

March 12, 2009

Mr. Michael Gootee, Superintendent
West Newbury Water Department
381 Main Street
West Newbury, MA 01985

Subject: 2003 Water Distribution Study Capital Plan Update

Dear Mr. Gootee:

As requested, Tata & Howard has updated the 2003 Water Distribution Study to include the Capital Plan Update, which was completed in June 2008. The Capital Plan Update has been added to the Water Distribution Study as Appendix E. Ten copies are included for your use.

Should you have any questions, please do not hesitate to contact us.

Sincerely,

TATA & HOWARD, INC.

Shira A. McWaters

Ms. Shira A. McWaters, P.E.
Associate

Attachments

CONSULTING ENGINEERS

MAIN OFFICE
67 Forest Street, Marlborough, MA 01752
508-303-9400 Fax: 508-303-9500

OTHER OFFICES
Lakeville, MA • Meriden, CT • Nashua, NH • Goodyear, AZ
www.tataandhoward.com

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Executive Summary

Introduction

The Town of West Newbury contracted Tata & Howard, Inc., to perform a water supply and distribution system analysis. This study evaluated the Town's ability to meet current and estimated future demands and provided a phased program of recommendations to address identified deficiencies.

As part of this analysis, a computer hydraulic model was created. The model was used to determine the adequacy of the distribution system by simulating different demand conditions such as average day demands or fire flows. Although a conceptual model is a feasible option for developing improvements such as cleaning and lining or looping, they are often based on visual observation and not field data. A verified computer model provides more specific recommendations relative to the size of the new mains and the best locations for the improvements. In short, a verified model can identify the impacts a demand, new source or improvements can have on the distribution system prior to the implementation of that scenario or improvement.

Water Supply

The ability of the Town's existing water supply source to meet present and future demands was evaluated in this study. Because population has a direct correlation to water consumption, population projections for the Town through the year 2020 were reviewed and adjusted to reflect actual and planned growth within the Town. Average day demands (ADD) and maximum day demands (MDD) were calculated using historical water consumption data, Department of Environmental Management (DEM) water projections and population projections. These demands were then compared to available water supplies.

The Town has one existing supply source and, based on accepted engineering practices, is unable to meet the projected 2020 summer ADD. In addition, based on the existing capacity of the wellfield, a projected MDD deficit of 0.66 mgd exists for the year 2020. It is therefore critical for the town to develop new water supply options.

Water Storage

Required storage capacity in the water system was calculated using fire flow requirements based on available Insurance Service Office (ISO) data. The ISO data was used to determine the adequacy of the existing distribution system in meeting fire flow demands. Recommendations of the necessary reinforcements to the distribution system were also completed as part of this study.

The distribution system is comprised of two service areas, each of which is serviced by its own water storage tank. The results of the water storage evaluation completed in this study show that no additional storage is required in the low service area. However, additional storage is required in the high service area to meet demand fluctuations within the system and furnish an adequate reserve for fire fighting.

Distribution System

The distribution system was originally established in 1936. Approximately 87 percent of the mains in distribution are 8-inch in diameter or less. Due to the age and size of the mains in the transmission grid, distribution improvements are required to strengthen and improve flow throughout the system. The recommended improvements include replacing the smaller 8-inch diameter water mains on Main Street with a new 16-inch diameter water main. In addition, several infrastructure improvements are required to replace or rehabilitate 6-inch diameter or smaller mains and older unlined water mains in the system. It should be noted that although the Town's distribution system is deficient in providing adequate fire protection in some areas, in general minimum pressure requirements can be maintained during typical demand conditions such as average day and maximum day demands.

Recommendations

Recommendations to address the deficiencies noted in water supply, storage and distribution were developed and separated into six phases according to priority. Each phase, if the Town chooses, can be carried out over a period of years, depending on the availability of funds. It is anticipated that deficiencies will be addressed relative to the costs associated with these improvements as well as priority. A more detailed discussion of these improvements is provided in Section 4.

Phase	Estimated Improvement Costs
1 – Water Supply	$4,685,000
2 – Storage/Transmission	$1,039,000
3 – High/Low Service Area Trans.	$1,418,000
4 – High/Low Service Area Trans.	$ 677,000
5 – Secondary Transmission	$ 777,000
6 – Restrictions	$1,336,600

Section 2

Model Input Parameters

Section 2

Model Input Parameters

Introduction

The availability of an updated hydraulic computer model provides the Town with an important tool in evaluating system expansions or changes in the future, particularly when evaluating the impacts of proposed new residential developments or water main replacement. In short, a verified model can identify the impacts a demand, new source or improvements can have on the distribution system prior to the implementation of that scenario or improvement. In order to establish a hydraulic model that simulates existing system conditions, several input parameters are necessary so that the model can adequately represent the distribution system. These parameters include information relative to the water supply source(s) and water storage facilities, the sizes and condition of the water mains, as well as water demands such as projected maximum day demands and ISO fire flow requirements. As such, a general description of the existing distribution system including the water supply source and storage, as well as population and water demand projections are provided herein.

Distribution System

Originally established in 1936, the Town of West Newbury's water distribution system consists of approximately 27 miles of water and service mains generally ranging from two to ten inches in diameter. Approximately 87 percent of the mains in the distribution system are 8-inch in diameter or less. Based on the Town's Water Department records, the material of construction of the water mains are unlined cast iron, cement lined ductile iron or asbestos cement (AC). The age and type of main is important when estimating the friction factor of the associated pipe in the computer model. In general, ductile iron and asbestos cement mains retain their original carrying capacity over time and are not as susceptible to tuberculation as cast iron mains.

The Town's distribution system is divided into high and low service areas. The low service area is comprised of the eastern portion of the distribution system from the Town's boundary with Newburyport to the Main Street booster pump station (BPS). There is one water storage facility, the Pipestave Tank, and one groundwater supply (the wellfield) within the low service area. Water from the low service area is boosted to the high service area through a primary BPS on Main Street (Route 113). The BPS is located across Route 113 from the entrance to the Page School. Additional water demands are met through an interconnection with the City of Newburyport.

The high service area is comprised of the western portion of the distribution system from the BPS located on Main Street to the western Town boundary. The majority of the Town's water users (approximately 80 percent) are serviced within the high service area. This percentage represents the total number of consumers in this service area and is not based on town-wide population. One water storage facility, the Brake Hill Tank, services this area. Since there are no water supply sources in this service area, water is supplied via the BPS located near the Pipestave Tank. The BPS pumps water from the wellfield, which includes Newburyport water, and water from the Pipestave Tank to the Brake Hill Tank.

Main Street Booster Pump Station (BPS)

The BPS is a precast station equipped with two pumps. Based on flow tests conducted at this station, the pumping capacity ranges from approximately 200 to 300 gallons per minute (gpm) with one pump operating to approximately 700 gpm with two pumps operating. This station has an emergency chlorinating feed system and flow metering, however there is no emergency power in this station. The pump turns on when the water level in the Brake Hill Tank falls below 54 feet and turns off when the water reaches a height of 59 feet.

Storage Facilities

As previously stated, there are two water storage facilities that serve the Town. They are the Pipestave Tank and the Brake Hill Tank. Originally constructed in 1982, the Pipestave Tank is a 50 foot diameter, 500,000 gallon capacity concrete tank located in the northeastern portion of the distribution system off Main Street (Route 113). It is approximately 40 feet high with an overflow elevation of 232 feet. The Pipestave Tank services the low service area in the system and the Page School fire suppression system. There are two pipes to the Pipestave Tank. There is one pipe that is the combined inlet/outlet for the distribution system and one pipe that is the outlet for the adjacent Page School fire suppression system. The combined inlet/outlet pipe for the distribution system is located approximately 6 feet above the floor elevation of the tank. The outlet pipe for the fire suppression system is approximately 13 inches above the bottom of the tank.

The Brake Hill Tank is located in the northwestern portion of the distribution system off Main Street (Route 113). It is a 30 foot diameter, 312,000 gallon riveted steel tank constructed in 1936. It is approximately 60 feet high with an overflow elevation of 300 feet. The tank was sandblasted and recoated in 1994. The Brake Hill Tank serves the high service area of the distribution system.

A summary of the water storage facilities is listed on Table No. 2-1.

Existing Water Supply Source

The Town has one groundwater supply source located off Main Street in the northeastern portion of the system. Originally constructed in 1991, Wellfield No. 1 consists of seven 2-1/2 inch diameter wells and one horizontal well. The horizontal well was drilled in 1994 in an effort to regain capacity due to the declining yield in the wellfield. The 2-1/2 inch wells range in depth from 32 feet to 47 feet. The wellfield pump station houses chemical feed equipment for pH adjustment, fluoridation and chlorination. In addition, water purchased from the City of Newburyport is pumped via a booster pump located in this station. There is also an emergency generator at this station.

The estimated safe yield for this supply is 0.20 mgd. However, in accordance with their Water Management Act (WMA) permit issued in April 1996, the Town currently is authorized to withdraw an average daily withdrawal volume of 0.16 mgd.

A summary of the existing water supply source is presented on Table No. 2-2.

Interconnections

Prior to the construction of the Town's wellfield in 1991, the water was supplied by interconnections with the City of Newburyport and the Town of Groveland. The Town's Water Department purchased water from Groveland from its inception in 1936 to 1979. As reported in the Water Master Plan prepared by Comprehensive Environmental, Inc. in February 2001, it is our understanding that one of the Groveland wells was found to be contaminated in 1979. As such, the Town abandoned the interconnection with Groveland and began purchasing water from the City of Newburyport. The Newburyport interconnection is a 10-inch diameter main located on Main Street, which enters the West Newbury distribution system through the Town's wellfield pump station.

Due to limited capacity at the Town's existing wellfield, the Town relies on the City of Newburyport to supply water during recharging of the aquifer and high demand periods. The Town utilizes the wellfield until drawdown levels reach a maximum drawdown depth. Once this depth is reached, the SCADA system automatically turns off the wellfield pump(s) and activates the Newburyport booster pump to supplement demands. The water from the City of Newburyport is pumped into the West Newbury system through a booster pump located in the wellfield pump station.

Currently, there is no agreement between the Town and the City of Newburyport. However, it is our understanding that the Town is in the process of negotiating an agreement with the City of Newburyport.

Population Projections

A Town's population has a direct correlation to water consumption. As such, projected average and maximum day demands were estimated by evaluating both current usage trends and population trends. A discussion of each is provided herein.

The Town of West Newbury has experienced a growth rate of approximately 35 percent since 1980. During this time, the population has increased from approximately 2,861 in 1980 to approximately 4,227 in 2001. Population projections from the area's planning agencies, the Town, and the US Census were reviewed and used to estimate growth within the Town through the year 2020.

Population projections to the year 2010 were provided by the Massachusetts Institute for Social and Economic Research (MISER). The MISER population projections to the year 2010 were reviewed and considered consistent with the projected curve from the historic and current populations. This information, as well as historic populations from the Town and US Census data, were plotted graphically to determine growth trends and the projected population for the design year 2020. Since MISER estimates were slightly lower than the Town's recorded population, the recorded population data was used to linearly estimate the Town's future population. Based on this criteria, the projected population for West Newbury is estimated to be approximately 5,540 for the year 2020 and is presented on Figure No. 2-1.

Projected Water Demands

DEM follows specific guidelines when projecting water usage for communities in conjunction with the Massachusetts Department of Environmental Protection Water Management Act. These guidelines incorporate trends in the use of water conservation devices in homes and industry, and emphasize the importance of monitoring the distribution system through water audits and leak detection surveys to reduce unaccounted-for water. These guidelines include an average residential per capita consumption of 80 gallons per capita per day (gpcd) for water planning purposes and an unaccounted-for water percentage of between 10 and 15 percent. It should be noted that DEM is considering lowering the average residential per capita consumption to 70 gpcd.

It is important to note that the DEM has a key role in the water management permit approval process. Water demand projections must be approved by DEM before the DEP will approve development of a new water supply source or authorize the withdrawal of additional volume from existing sources under the WMA program. Typically, DEM guidelines are used to estimate demand projections for water distribution system studies so that the projected water demands can be submitted for a WMA permit application.

Average Day Demand

Historical water records for the past six years were used to determine recent water usage trends. Although historical water data from 1988 to 2001 is available, using older data might inadequately represent recent water trends which includes the installation of water saving devices and impacts to summer demands from irrigation systems. In the past six years, outdoor water use has increasingly become a significant component of the maximum day demands due to the increase in home irrigation systems. As such, the last six years of data was selected to evaluate recent water use trends. Based on the Town's records for the past six years, the ADD ranged from approximately 0.189 in 1996 to approximately 0.24 mgd in 2001.

The breakdown of service connections in each water use category has been consistent over the past six years. Review of the Town of West Newbury Zoning Map indicates that there are limited parcels within Town zoned for commercial and industrial use. In addition, the industrial zoned area is located beyond the extent of the water distribution system. As such, it is assumed that the percentage of commercial/industrial water use will remain constant over the next twenty years.

The summer average day demand represents the average day demand during the summer months of May through August. The summer ADD is typically higher than the yearly ADD since outdoor water use is higher during the summer months. Over the last six years, the summer ADD has ranged from a low of 0.23 mgd in 1996 to a high of 0.31 mgd in 1999. The summer ADD for the year 2001 was 0.29 mgd. Upon comparison of the summer ADD to the yearly ADD, the ratio, which is called the peaking factor, ranges from a low of 1.21 to high of 1.34. The average summer ADD to yearly ADD peaking factor over the past six years is 1.26.

Based on the 2001 Annual Water Statistical Report, there are approximately 877 service connections in the Town of West Newbury. Of the 877 connections, there are approximately 845 residential service connections, 7 municipal service connections, 5 industrial/agricultural service connections and 20 commercial service connections. Assuming that there are 3.1 people per service connection as estimated in the Comprehensive Plan, the service population for the year 2001 is approximately 2,700 people, which is approximately 64 percent of the Town's population. Additionally, the Town's current residential per capita consumption is approximately 63 gpcd, which is well below DEM's 80 gpcd guideline. The average per capita consumption rate is approximately 69 gpcd over the past six years. The Town's current and average residential consumption compares favorably to DEM's proposed estimate. As such, a residential consumption of 70 gpcd was used in this study.

Based on annual statistical reports, the amount of unaccounted-for water in the system over the past six years ranges from a low of approximately 5 percent to a high of approximately 23 percent in 2001 with an average of 13 percent. It should be noted that the unaccounted-for water in the system in 2001 was significantly higher than historical data due to problems associated with the telemetry between the BPS and tank causing the tank to overflow. The wiring problem has been corrected.

Unaccounted-for water consists of unmetered water used for street, pipe flushing, meter losses, unauthorized water uses, fire fighting and leakage in the distribution system. It is expressed as a percentage of the total water supplied to the system and can be estimated by taking the difference between the total amount of water supplied and the total water billed divided by the total water supplied. Typically a water audit is performed to determine the various components of unaccounted-for water.

The following criteria were used to project water demands for the year 2020: (1) year 2020 population of 5,540 (Based on conversations with the Hydraulic Study Committee, it is estimated that the service population is assumed to remain constant at approximately 65% of the Town population); (2) an average consumption rate of 70 gpcd; (3) 8 percent for commercial/industrial/agricultural/municipal users; (4) 15 percent unaccounted-for water and (5) a summer ADD to yearly ADD peaking factor of 1.26. Based on this criteria, the estimated ADD for the year 2020 is approximately 0.33 mgd, and the estimated summer ADD for the year 2020 is approximately 0.42 mgd both are presented in Figure No. 2-2.

Maximum Day Demand

Table No. 2-3 presents the historical pumping data for the years 1996 to 2001 and includes the total amount of water supplied each year and respective ADD and MDD. This information was used to establish existing water usage trends and a peaking factor showing the relationship of maximum day demands to average day demands. As shown, the maximum day demands range from 0.367 to 0.511 mgd. The MDD/ADD ratio has ranged from a high of 2.57 in 1996 to a low of 1.76 in 2000. The average MDD/ADD ratio is approximately 2.12. The average historical peaking factor over the past six years was used to estimate the projected MDD. It should be noted that the MDD/ADD ratio could be reduced by encouraging water conservation through mandatory water bans and pricing. Using the average peaking factor of 2.12 and an ADD of 0.33 mgd, the projected MDD for the design year 2020 is estimated to be 0.7 mgd.

Figure No. 2-2 presents the existing ADD and MDD and the projected ADD and MDD through the year 2020.

Peak Hour Demand

Since Town records of peak hourly demands were not available, the peak hour/ADD ratio for the design year 2020 was estimated from the Merrimac Curve. It is common engineering practice to use the Merrimac Curve to estimate peak water consumption when historical data is not available. The Merrimac Curve is based on statistical analysis of sanitary sewage flows and water consumption for many towns and communities in New England, relating peak hour flows to average day flows. The average historical MDD/ADD peaking factor (2.12) for the Town was used in the Merrimac Curve. Based on a peak hour/ADD ratio of 3.7 and an ADD of 0.33 mgd, the projected peak hour demand for 2020 is estimated to be 1.22 mgd.

Table No. 2-1
Water Storage Facilities
West Newbury, Massachusetts

Tank	Height (ft)	Diameter (ft)	Capacity (mg)	Overflow Elevation (ft)
Pipestave Tank	40	50	0.50	232
Brake Hill Tank	60	30	0.31	300

Table No. 2-2
Existing Water Supply Source – Wellfield No. 1
West Newbury, Massachusetts

Source	Type of Well	Approximate Depth (ft)	Top of Casing Elevation (ft)
Well No. 1	2-1/2-inch Tubular	36	16.38
Well No. 2	2-1/2-inch Tubular	42	15.99
Well No. 3	2-1/2-inch Tubular	35	15.53
Well No. 4	2-1/2-inch Tubular	32	15.50
Well No. 5	2-1/2-inch Tubular	38	15.95
Well No. 6	2-1/2-inch Tubular	41	15.64
Well No. 7	2-1/2-inch Tubular	47	18.41
Well No. 8	Horizontal Well	15	18.50

Figure No. 2-1
Population Projections
West Newbury, Massachusetts

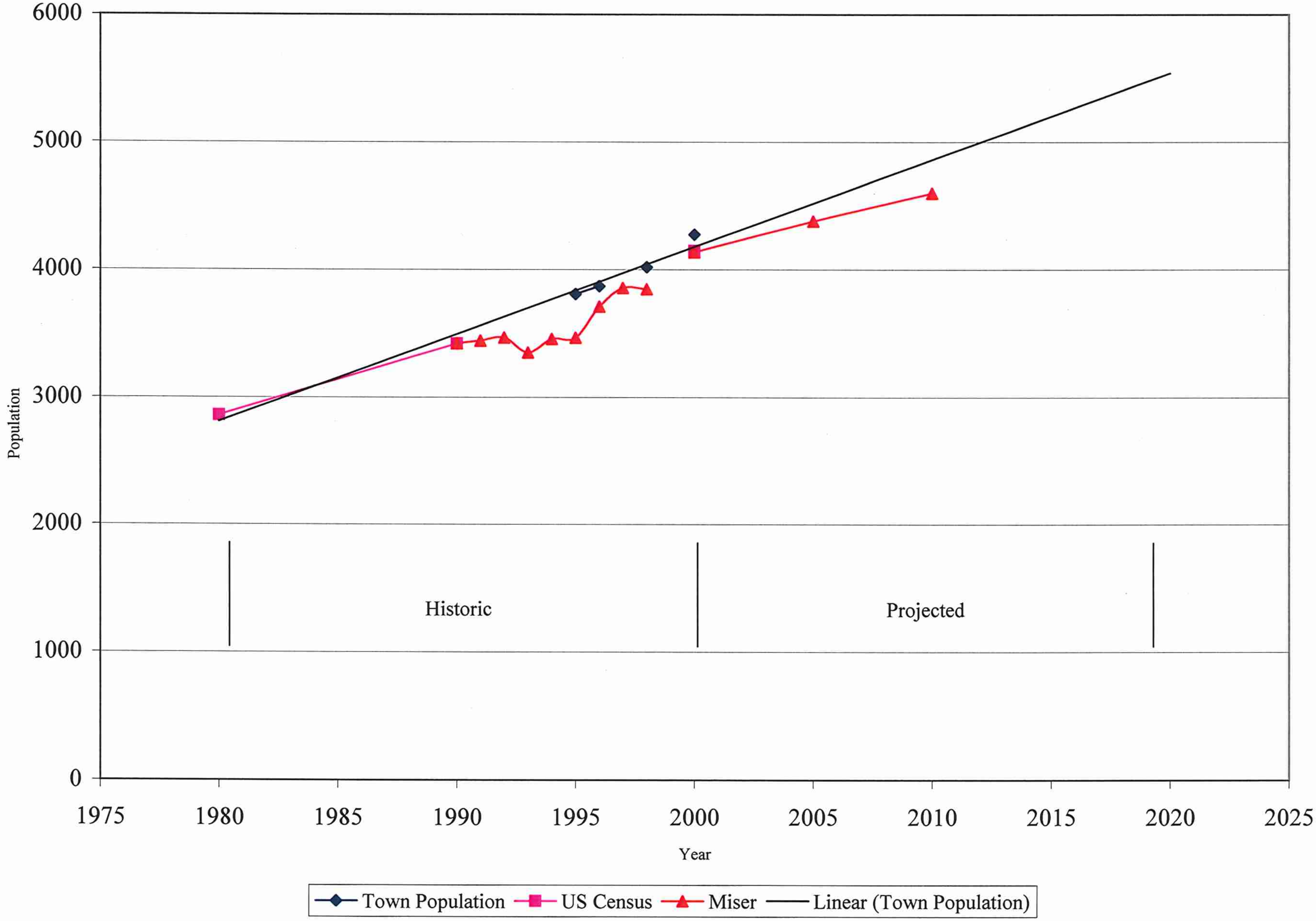


Figure No. 2-2
Water Demand Projections
West Newbury, Massachusetts

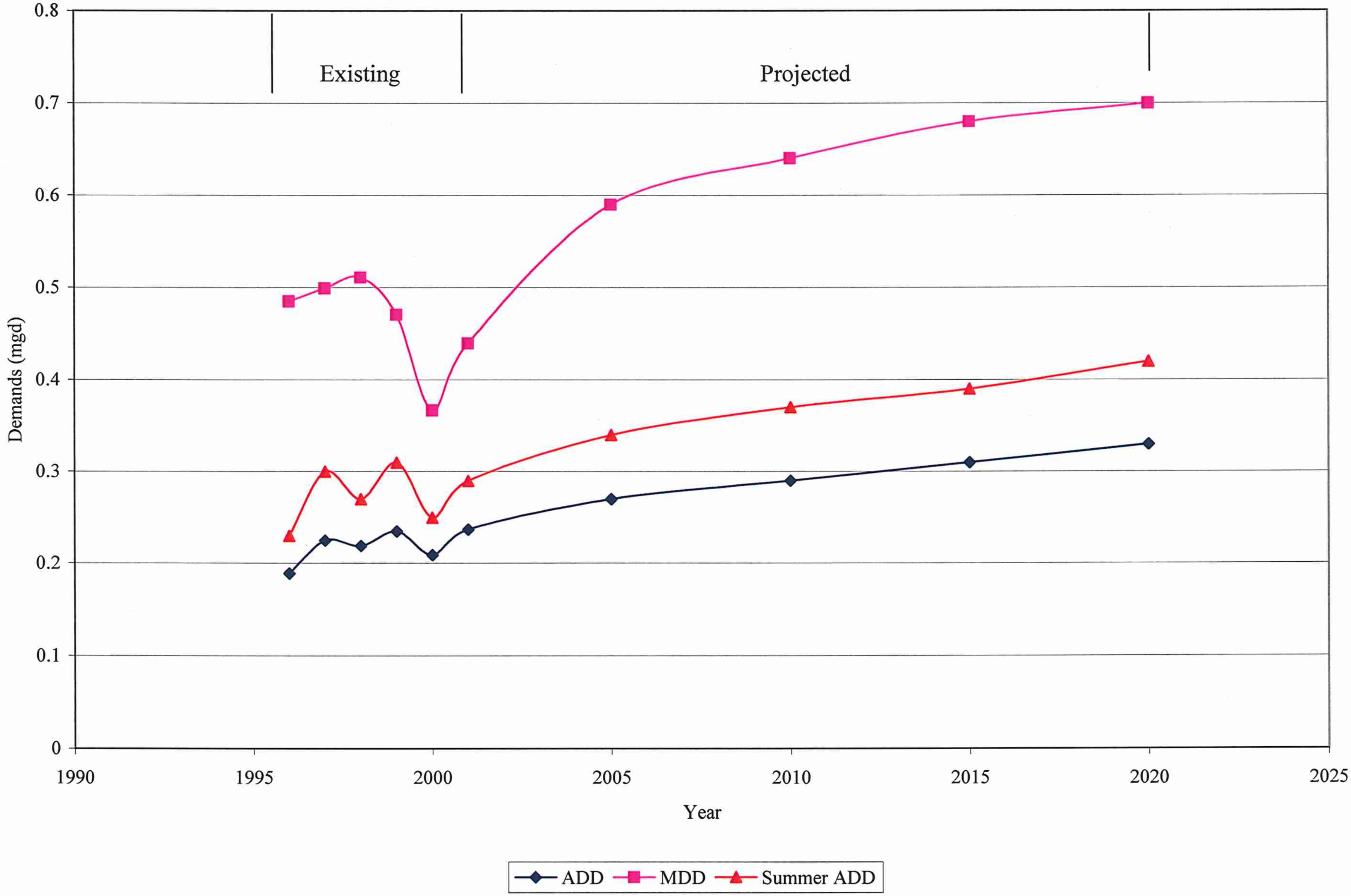


Table No. 2-3
Historical Pumping Data
West Newbury, Massachusetts

Year	Total Pumped (gallons)	ADD (mgd)	Summer ADD (mgd)	Summer ADD/ ADD P.F.	MDD (mgd)	MDD/ADD P.F.
1996	69,020,600	0.189	0.23	1.21	0.485	2.57
1997	82,206,300	0.225	0.30	1.34	0.499	2.22
1998	79,830,200	0.219	0.27	1.25	0.511	2.33
1999	85,721,200	0.235	0.31	1.3	0.471	2.00
2000	76,361,390	0.209	0.25	1.20	0.367	1.76
2001	86,568,500	0.237	0.29	1.23	0.440	1.86
Ave.	79,951,365	0.219	0.27	1.26	0.462	2.12

ADD – Average Day Demand
Summer ADD – Summer Average Day Demand
Summer ADD/ADD P.F. – Summer Average Day Demand/Average Day Demand Peaking Factor
MDD – Maximum Day Demand
MDD/ADD P.F. – Maximum Day Demand/Average Day Demand Peaking Factor

Section 3

Distribution System Analysis

Section 3

Distribution System Analysis

General

In order to evaluate the Town's existing water distribution system and to obtain a basis for recommending water system improvements, a comprehensive computer model was utilized to mathematically simulate the water distribution system. The computer model, which was verified using field flow test conditions, provides the Town with a planning tool with which evaluations of new and proposed development, storage and supply capacity can be done relative to their potential impacts to the water distribution system.

The hydraulic computer model utilized for this study is called WaterCad, which utilizes AutoCad for its mapping component. The existing AutoCad map of the distribution system developed under a previous study was revised to incorporate discrepancies noted by the Water Department. The map was color coded to reflect different main sizes and was used to establish the distribution system layout required to conduct the hydraulic analysis.

A hydraulic analysis, using available data on the water distribution system and fire flow test results, provides an indication of the distribution system's ability to meet the criteria described in this chapter.

WaterCad

There are several hydraulic models available for use in water system analysis. The most commonly used model is WaterCad distributed by Haested Methods. WaterCad allows the user to conduct hydraulic simulations using mathematically based algorithms, while in an AutoCad environment. As indicated by the Town, GIS mapping is currently being completed for the Town's utilities. The AutoCad format will allow the distribution system mapping to be saved and incorporated into the Town's GIS format.

WaterCad has a variety of modeling capabilities some of which are listed below:

- fire flow analyses to determine how the system will react under stressed conditions,
- steady state analyses of water distribution systems with pumps, tanks and control valves,
- extended period simulations to analyze the piping system's response to varying supply and demand schedules,
- water quality simulations to determine the water source and age, or track the growth or decay of a chemical constituent throughout the distribution system,
- establishment of a unidirectional flushing program that will outline the sequence of valves to close and hydrants to flush,
- evaluations of the impacts a proposed development has on the system, which includes evaluation of the pressure and flow requirements of the new development.

Since the mapping and modeling is AutoCad based, the system and hydraulic model can be updated frequently to incorporate new streets and/or subdivisions or developments.

Many clients inquire about the "in-house" purchase and use of this hydraulic model. However, it is cost prohibitive for smaller communities to purchase both AutoCad and WaterCad, as the initial software and equipment costs is approximately $15,000. Therefore, it is more cost effective for smaller communities to hire consultants to run model simulations. These simulations include impacts on portions of the distribution system from new developments or infrastructure improvements, the hydraulics associated with the development of a new water supply source, or the construction of a new water storage tank.

Model Construction and Verification

The computer model was completed in three phases. In the first phase water mains in the AutoCad system map were color coded and line typed. A draft map was provided to the Town for review and comment. The distribution map provides information on sizes of water mains, major roadways, and a general layout of the distribution system. A color coded map of the Town's water distribution system can be found in Appendix A and at the back of this section (Drawing No. 3-1). Many communities find this map useful for easy reference of water main sizes in streets, for leak detection system maintenance DEP encourages the development of a water distribution map.

In the second phase of the model development, 22 fire flow tests were conducted in various locations throughout the distribution system. A copy of the fire flow test data can be found in Appendix B. Table No. 3-1 presents a summary of the fire flow tests completed under this study. The fire flow tests were done to determine available fire flows in the distribution system and to provide data for computer model verification. The locations of the tests were coordinated with the Town and conducted at night to minimize impacts to the Town's consumers.

Verification of the computer model was completed in the third phase. Verification is an important component in the model process so that the model closely simulates the existing distribution system. To properly evaluate the system and model it on a computer, the various data collected during the study was used to verify the model. This data included water levels in the tanks, pumping rates at the water supply source, static and residual pressure readings and measurement of flows from hydrants obtained during the fire flow tests, and estimated pipe friction factors based on the type and ages of mains. The initial data collected such as the annual statistical reports, Insurance Services Office (ISO) data, information on the wells including safe yield, type and depth and tank information was used either to verify the model or as supporting material for this report.

The above mentioned hydraulic input data, which is provided in Appendix C, provides information on system water demands, length of water mains, estimated friction factors of the mains, and the overflow elevation of the tanks. During each simulation, it was important to reflect actual field conditions. Actual field conditions included current demands in the system as well as varying tank elevations.

When the results of the computer runs compared to within five percent of the hydraulic data collected from the field flow tests, the computer model was considered verified and mathematically represents the physical operating condition of the Town's supply and distribution system.

Once the model has been verified, hypothetical conditions can be simulated on the model and the effects can be observed without actually creating the conditions in the field. Effects of increased demands, changes in pipeline friction factors, required fire flows, pipeline additions and/or changes and chemical constituent transport can be analyzed.

It is common engineering practice to evaluate a system using ISO fire flow requirements. In order to provide adequate service, a distribution system should be able to meet demands during periods of peak consumption and during emergencies such as a fire. For the purposes of this study the model was stressed under projected maximum day demand conditions with a coincident fire flow. If the system can satisfactorily provide the water under this demand condition, it is considered reasonable that the system can handle all other demand conditions.

In addition to using ISO data, estimated fire flows based on AWWA fire protection requirements were incorporated into the model and evaluated. In this manner, not only current problems can be evaluated, but potential problems can be identified and mitigated before they develop. The simulation of these conditions provided the opportunity to identify system deficiencies and to develop necessary improvements using the hydraulic model of the distribution system.

<u>Fire Flow Demands</u>

The required fire flows in any community are established by the ISO. The ISO determines a theoretical flow rate that is needed to combat a major fire at a specific location taking into account the building structure and use, floor area, the building contents, and the availability of fire prevention systems. In general, the flows required for proper fire protection are based on maintaining a residual pressure of 20 psi. This residual pressure is considered necessary to maintain a positive pressure on the suction side of a fire department pumper truck with an allowance for frictional losses in the hydrant and fire hoses.

The Town's system was last inspected for fire insurance ratings by the ISO in November 1988. Six of the fire flow tests conducted under this hydraulic study will be used by ISO for their upcoming fire insurance rating. The results of the 1988 ISO inspections and fire flow testing are shown on Table No. 3-2. The test results indicate the available flow and the estimated needed fire flow in various sections of the distribution system. The difference between available and needed fire flows is as follows: an available flow is based on actual data obtained from conducting flow tests in the field. The flow recorded represents what is currently available at that particular location in the distribution system. The estimated needed fire flow is a flow rate needed to provide adequate fire protection to that area. The estimated needed fire flow is based on a specific structure's size, its materials of construction, use (i.e. school, chemical warehouse, etc.) and neighboring structures. At times the available flow is equal to or exceeds the estimated needed fire flow. When this occurs the infrastructure in that area is considered adequate in size and condition and improvements are not required. However, if the available flow is less than the estimated needed flow improvements may be warranted to mitigate the deficiency.

Three of the fire flow tests conducted under this hydraulic study were conducted at the approximate location of three ISO flow tests conducted in 1988. These tests include No. 2, 16 and 17 listed on Table No. 3-1. If these tests are generally compared to the corresponding 1988 ISO fire flow tests (presented in Table No. 3-2), the estimated available flow from the hydraulic study tests are approximately 200 to 500 gpm less than that reported in 1988 from ISO. The only exception is the estimated flow rate on Main Street near Garden Street in which the estimated available flows for both tests are approximately equal. The differences between the two sets of data most likely can be attributed to the operating conditions of the distribution system (i.e. tank elevations, BPS, wellfield). Overall, the results from the ISO study and this hydraulic study suggests that there are deficiencies relative to fire protection in the distribution system.

Adequacy of the Existing Distribution System

A distribution system should be able to provide adequate pressures during varying demand conditions. For the purposes of this study a minimum pressure of 35 psi at ground level was required during average day, maximum day, and peak hour demand conditions. An upper limiting pressure of 120 psi is generally recommended, as older fittings in the system are typically rated at 125 – 150 psi. Pressures above this level can result in increased water use from fixtures and also increased leakage throughout the distribution system. During fire flow conditions, a minimum pressure of 20 psi was required at ground level throughout the distribution system. This 20 psi of pressure is equivalent to 46 feet in elevation and will allow water to overcome frictional resistance in house plumbing and rise to a height equivalent to about a three story building. In summary, the system pressure criteria shown below was used for the distribution system hydraulic analysis:

Demand Condition	Minimum Pressure (psi)
Year 2020 ADD @ 0.33mgd	35
Year 2020 MDD @ 0.73 mgd	35
Year 2020 Peak Hour @ 1.22 mgd	35
Year 2020 MDD @ 0.73 mgd + Coincident ISO Fire Flow	20

1. Minimum Pressures - Maximum Day and Peak Hour

During normal year 2020 average day, maximum day and peak hour demand conditions (no coincident fire flow), the recommended minimum pressures requirement of 35 psi is met in the majority of the distribution system for the Town's two pressure services zones. Areas where 35 psi is not maintained are shown on Figure No. 3-1. The inability of the system to meet pressure requirements in these areas is primarily due to elevation, not necessarily due to restrictions in the distribution system. In general, residences located at elevations greater than 147 feet in the low service area and 215 feet in the high service area will experience pressures less than 35 psi during normal operating conditions.

2. Fire Flow Requirements - ISO

The estimated needed fire flows as recommended by the ISO were simulated on the computer model. The flow tests noted with an asterisk in Table No. 3-2 represent those areas which did not meet the ISO estimated needed fire flow requirement when field tests were conducted by ISO personnel in 1988. The results of the six fire flow tests conducted under this study for ISO are similar to the ISO results in 1988. In small communities such as West Newbury, it is not uncommon for a system to require several infrastructure improvements in order to assist in meeting the higher ISO requirements. The inability of the system to meet these flow requirements is primarily due to the diameter of the mains. Since approximately 87 percent of the Town's distribution system are 8-inch in diameter or less, many of the mains do not have the inherent carrying capacity to meet the higher flow requirements. Using the hydraulic computer model, improvements were developed to meet these flow requirements by either adjusted friction factors to simulate cleaning and lining of a pipe or increasing the pipe diameter to simulate the construction of a larger main. A description of the proposed improvements is presented in Section 4.

It should be noted that although the Town's distribution system is deficient in providing adequate fire protection in some areas, in general minimum pressure requirements can be maintained during typical demand conditions such as existing and projected average day and maximum day demands.

3. Fire Flows Requirements – Extremities and Developed Residential Areas

Estimated fire flows of 500 to 1000 gpm were simulated at distribution system extremities, which are the outer limits of the system generally located along dead ends, and developed residential areas, such as subdivisions as part of a system wide evaluation. The estimated fire flow requirements for these areas were based on ISO criteria, which includes the size and use of the structures, their spacing and the material of construction. For example, a residential area composed primarily of single family homes spaced 50 feet apart would have an estimated needed fire flow of approximately 750 gpm. Using this criteria, estimated fire flows of 500 to 1000 gpm were simulated throughout the system.

The hydraulic analysis indicates that there are locations in the Town's distribution system that cannot adequately meet these smaller estimated fire flow requirements. This is common in systems such as West Newbury's in which there are restrictions in the pipes such as "bottlenecks", which occur when smaller water main are the sole means of transporting water between larger mains. Additionally, there are several long lengths of 6-inch or smaller diameter main in the system where looping is not possible. This creates friction losses in the pipes which, in turn, limits the amount of flow through the pipe and reduces pressure. Although these restrictions generally do not impact the adequacy of the system to provide minimum pressures during average day or maximum day demand conditions, these restrictions reduce the inherent carrying capacity of the mains that are needed to meet fire flow requirements. Therefore, additional distribution improvements are required for fire protection in these areas and are presented in Section 4.

Adequacy of Existing Water Storage Facilities

Distribution storage is provided to meet peak consumer demands such as peak hour demands and, additionally, to furnish a reserve for fire fighting. Storage may also serve to provide an emergency supply in case of temporary breakdown of pumping facilities, or for pressure regulating during periods of fluctuating demands.

There are three components that are considered when evaluating storage requirements. These components include equalization, fire flow requirements, and emergency storage. Since the distribution system is comprised of two service areas, the storage must be evaluated separately for each service area. Based on the current configuration of the high and low service areas, approximately 80 percent of the service population is in the high service area, while approximately 20 percent of the service population is in the low service area. For the storage evaluation, it is assumed these percentages will remain constant over the design period.

Equalization storage provides water from the tanks during peak hourly demands in the system. Typically, this quantity is a percentage of the maximum day demands. The percentages can range from fifteen to twenty-five percent, with fifteen percent used for a large system, twenty percent for a mid-sized system and twenty five percent used for a small system. A system is considered small if it has less than 3,300 customers, while a system is considered large if it has more than 50,000 customers. The Town's distribution system would be considered a small system. As a result, twenty five percent of projected maximum day demands was used for the equalization storage calculations.

A "Basic Fire Flow" is established for use in estimating the fire flow storage component. In general, a basic fire flow is somewhere in between the highest and lowest recommended ISO flow requirements, and ISO defines as "a fire flow indicative of the quantities needed for handling fires in important districts", and usually serves to mitigate some of the higher specific fire flows. Based on previous ISO work conducted within West Newbury the estimated needed fire flows varied from 500 to 4,000 gpm. The 500 gpm flow is considered too low to meet the higher flow requirements, while the 4000 gpm flow is too high and most likely represents a single fire flow requirement. For the purposes of this study, the basic fire flow requirement is estimated to be 2,000 gpm for a duration of 2 hours. This basic fire flow of 2,000 gpm was selected based on ISO data and quantities needed for handling fires in the Town's more important districts such as Main Street near the Town Hall or High School.

The emergency storage component is typically equivalent to an average day demand. However, if there is emergency power available at the pumping stations or an interconnection with a nearby community, the emergency storage component of an average day demand can be waived. In the low service area, the wellfield is equipped with a back-up generator and an interconnection with the City of Newburyport. Therefore, the emergency component is waived for the low service area. In the high service area, there is no emergency generator at the booster pump station. As such, we recommend the construction of an emergency generator at this station so that the emergency storage component can be waived. Further discussion of this recommendation is provided in Section 4.

Based on the above criteria, the three components of the storage evaluation for the low service and the high service area are presented on Table No. 3-3.

As shown on Table No. 3-3, the estimated required storage for the low service area for the design year 2020 is approximately 0.28 million gallons (MG). Under existing and projected ADD, MDD, and peak hour demands, a minimum pressure of 35 psi can be maintained up to a topographic elevation of approximately 147 feet. Standard waterworks practices allow for a drop in pressures to a minimum of 20 psi during MDD with a coincident fire flow demand. In order to maintain a minimum pressure of 20 psi at elevations of 147 feet, the hydraulic gradeline in the system can drop from 232 feet to approximately 200 feet. Under this scenario, the usable storage of the water storage tank would be equivalent to 100 percent of the total volume of the Pipestave Tank. Although the Pipestave Tank has a capacity of 500,000 gallons, the bottom six feet of the tank is used for fire protection for the Page School. Therefore, the usable storage of this tank is approximately 425,000 gallons. Based on the storage evaluation herein, no additional storage is required in the low service area.

Based on the criteria listed on Table No. 3-3, the total required storage for the high service area for the design year 2020 is approximately 0.39 MG. In the high service area, under existing and projected ADD, MDD, and peak hour demands, a minimum pressure of 35 psi can be maintained up to a topographic elevation of approximately 215 feet. In order to maintain a minimum pressure of 20 psi at elevations of 215 feet during a fire flow demand, the hydraulic gradeline in the system can drop from 300 feet to approximately 265 feet. Under this scenario, the usable storage of the water storage tank is equivalent to 58 percent of its total volume of 0.31 MG or 0.18 MG. Therefore, the Town's Brake Hill Tank does not have adequate storage capacity to meet the projected 0.39 MG storage requirement. Therefore, a new storage tank with a minimum usable capacity of 0.21 MG is recommended to supplement the usable storage of 0.18 MG from the Brake Hill Tank. Further discussion of this recommendation is provided in Section 4.

<u>Adequacy of Existing Water Supply Sources</u>

Typically in the past, supplies were evaluated based on their ability to meet the projected maximum day demands with the largest supply out of service. However, a more realistic evaluation of a Town's water supply is provided in the guidelines outlined in the Recommended Ten State Standards for Water Works and proposed by the Massachusetts DEP to be adopted as part of their 2001 Guidelines. These guidelines outline two components to be considered when evaluating the adequacy of existing water supplies and are as follows:

- The yield of the active sources must be greater than or equal to the summer average day demand with the largest source off-line.
- The yield of the active sources must be greater than or equal the maximum day demand.

It should be noted that both criteria should be satisfied for the supply to be considered adequate. These two components are reviewed herein.

Summer Average Day Demands

As stated in Section 2, the current and projected summer average day demands (ADD) are 0.29 mgd and 0.42 mgd, respectively. Since the Town has one existing supply source, with this source out of service, the total available yield is 0 mgd.

Maximum Day Demands

This evaluation criterion requires that the yield of the sources be greater than or equal to the existing and projected maximum day demand (MDD). As stated in Section 2, the current MDD and projected 2020 MDD are 0.44 mgd and 0.73 mgd, respectively. During high demand periods when the water table is low, the Town's total available supply from the Wellfield is approximately 0.06 mgd. Therefore, an existing and projected MDD deficit of 0.38 and 0.66 mgd exist, respectively. Under this scenario, the Town would need to rely on another source, such as their interconnection with the City of Newburyport or a new water supply source to help supplement these demands.

Table No. 3-4 presents the system's capacity with the largest source out of service compared to both current and projected summer average day demands, as well as the capacity of the existing supply source compared to current and projected maximum day demands.

Potential Sources of Additional Supply

According to the water supply evaluation completed herein, the Town currently has a water supply deficit relative to existing and projected demands. The Town has been actively pursuing the development of a supplemental water source and three potential bedrock well sites have been identified which are discussed later in this report. Although a comprehensive water supply investigation is beyond the scope of this study, a brief review of the Town's efforts and potential supply sources is presented in this section. The potential sources of additional supply include:

- Maximizing the yield from the existing water supply sources,
- New well(s) in unconsolidated aquifers,
- Bedrock wells, and
- Regional water supply sources.

Maximize Existing Water Supply Sources

Based on discussions with the Water Department, the available capacity of the wellfield varies depending on the time of year. During the winter months when the groundwater table is high, the Town is able to withdraw approximately 0.20 mgd from the wellfield, which is the reported safe yield. However, during high demand periods that typically occur during the summer when the groundwater table is low, it is our understanding that the Town is only able to withdraw approximately 0.06 mgd from the wellfield due to excessive drawdown in Well No. 4. The safe yield (0.20 mgd) is the volume of water the wellfield should be capable of pumping all year round since it was determined using the most severe pumping and recharge conditions. Since there is a discrepancy in what is being pumped versus what should be pumped, we recommend reevaluating the safe yield of the wellfield. If the reevaluated safe yield is equal to or greater than 0.20 mgd, an evaluation of the existing wellfield configuration is recommended in an effort to maximize the yield from this supply year round. Further discussion of this recommendation is presented in Section 4.

Unconsolidated Groundwater Wells

In an effort to mitigate the water supply deficit, another alternative would be the development of a new groundwater supply in a sand and gravel (unconsolidated) aquifer. Due to the extensive new source approval process through DEP, costs of land ownership and legal fees, wellhead protection and zoning obstacles, the development of a new water supply source can take several years from the time a source is identified until it is actually activated.

In 1983, the Town initiated a groundwater exploration program in an effort to increase their water supply capacity. A potential groundwater supply source was identified near the Artichoke River but was never evaluated. Therefore, we recommend investigating this site in more detail.

Since the Town is primarily overlain by glacial materials of till or marine clays, only four additional sites were previously identified by D.L. Maher as potential groundwater supply sources. The potential sites are located at the southeast corner of Mill Pond, east of Coffin Street, east of the Indian River and east of Bridge Street. Due to refusal, shallow overlying thickness, soils and ownership issues, it was concluded during the D.L. Maher investigation that none of the above mentioned sites would be viable water supply sources. As such, it was determined that the Town should investigate the development of a bedrock supply source.

Bedrock Wells

Another alternative groundwater supply source that the Town is considering is development of bedrock wells. A bedrock well differs from a gravel-packed well or other surficial well in that the well is drilled through bedrock and taps into the groundwater that flows through the fractured bedrock. Drawbacks of bedrock wells include the following: contamination may travel from long distances to the well, the source of contamination may be difficult to identify, bedrock wells are generally not capable of yielding the same quantities of water as surficial wells and the costs to locate and develop a bedrock source are high.

Despite these drawbacks many Towns in Massachusetts have successfully developed bedrock wells for public water supply systems. It is advantageous to pursue the development of bedrock wells when the presence of materials that are conducive to the development of high yielding groundwater source such as sand and gravel deposits are limited throughout an area. Due to the limited surficial sites available, the Town has identified three potential bedrock supply sources through a fracture trace analysis.

The three potential bedrock sources currently being investigated are located off Indian Hill Street (Andreas Well Site), off Middle Street (Knowles Well Site) and off Middle Street (Dunn Well).

The proposed Andreas Well site is located in the northeastern section of Town off Indian Hill Street between Garden Street and Middle Street. The Andreas Well is an 8-inch diameter well with and estimated safe yield of approximately 101 gpm or 0.145 mgd.

The Andreas Well is currently undergoing the New Source Approval process through the DEP. To date, the Request for Site Exam, Pump Test Proposal, extended pump test and extended pump test report has been completed. The United States Geological Survey (USGS) currently delineating the Zone II for the Andreas and Knowles wells. Tata & Howard has submitted the extended pump test report and the Water Management Act (WMA) permit, which is required for all new sources, to DEP for review approval.

The proposed Knowles Well Site is located in the northeastern portion of the Town off Middle Street near the intersection of Chase Street. This well was pump tested at the same time as the Andreas Well and the results of the pump test were included in the same report as the Andreas Well that was submitted to the DEP for review. Its estimated safe yield was approximately 150 gpm or 0.216 mgd. However, it should be noted that this site in no longer under investigation by the Town and was recently purchased for development.

The proposed Dunn Well is located off Middle Street in the northeastern portion of the Town adjacent to the Knowles well site. Based on preliminary pumping data, the potential yield of the well is estimated to be approximately 130 gpm or 0.187 mgd. The Dunn Well is in the preliminary stages of the new source approval process. A Request for Site Exam with the required Water Management Act (WMA) components including the Site Screening and a Pump Test Proposal must be completed and submitted to DEP for approval. Upon receipt of approval, an extended pump test can be conducted, followed by the preparation of the extended pump test report, Zone II delineation and WMA permit. Typically, development of a new source can take approximately 3 to 5 years to be approved by the DEP.

The Andreas and the Dunn wells may require treatment for radon removal if the State adopts the proposed MCL of 300 pCi/L. It is anticipated that Massachusetts will chose to adopt an alternative maximum contaminant level (AMCL) of 4,000 pCi/L and implement a multimedia mitigation (MMM) program. This would mean no treatment for radon removal would be required.

Additionally, the arsenic concentration detected in the Andreas and Dunn Wells exceeds the arsenic MCL of 0.01 mg/l. Therefore, treatment will be required for arsenic removal. Based on water quality data, pH adjustment and manganese sequestering will be required.

Regional Water Supply Sources

West Newbury has one existing interconnection with the neighboring City of Newburyport, which is currently used to supplement their wellfield. In addition, it is our understanding that the Town previously had an interconnection with the Town of Groveland, which was abandoned due to water quality issues. The Town should investigate reactivating the abandoned interconnection with Groveland. This interconnection could provide supply to the high service area, which is currently supplied by the booster pump station that has no emergency back-up power. The Town should carefully review the potential to purchase water from each of the neighboring communities and consider the benefits of a regional water supply source.

Table No. 3-1
Tata & Howard Fire Flow Tests
October 23 and 24, 2001
West Newbury, Massachusetts

Test No.	Location of Flowing Hydrant	Flowing Hydrant Static Pressure (psi)	Residual Hydrant Static Pressure (psi)	Residual Hydrant Residual Pressure (psi)	Flow (gpm)	Estimated Flow @20 psi (gpm)
1	End of Mirra Way	71	86	48	889	1200
2	Main Street Near Garden Street	95-96	82	44	1114	1450
3	Garden Street Last Hyd.	86	82	58	823	1370
4	Parson's Road	96	98	42	1009	1200
5	Main Street near Wellfield	112	110	48	1151	1400
6	End of Woodcrest Drive	82	86-87	38	936	1110
7	End of Crane Neck Street	40	54-56	50	186	530
8	Meeting House Hill Road	74	40-42	18	1114	1060
9	End of Stewart Street	66	64-66	62	264	1260
10	Last Hydrant Behind H.S.	112	112	74-76	256	470
11	Main Street Near H.S. Entrance	106	98	76	841	1660
12	End of Rivercrest Drive	113	112-114	62	1257	1740
13	End of Waterside Lane	106	106	70	1188	1900
14	End of River Meadow Court	106	106-108	50	1089	1360
15	Main Street @ Merrill Street	84	86	72	1247	2880
16	Main Street Near Maple Street and Church Street	75	84	52	1037	1500
17	Main Street Near Prospect Street	72	78	46	921	1270
18	End of River Road	112	128	72	1188	1690
19	Albion Lane	68	76-78	40	752	950
20	Main Street @ Bailey Lane	72	76-78	44	841	1130
21	End of Courtland Lane	84	92-94	52	921	1250
22	Main Street @ Page School	56	92	66	711	1230

Notes:
1. The wellfield and BPS were off-line during fire flow testing.
2. The difference in static pressure between the flowing hydrants and residual hydrants is due the elevation of each hydrant (i.e. higher or lower). For example, if one hydrant is lower in elevation than the other, the lower hydrant will have a higher static pressure.

Table No. 3-2
1988 ISO Hydrant Flow Data Summary
West Newbury, Massachusetts

Test No.	Location	Static Pressure (psi)	Residual Pressure (psi)	Available Flow gpm @ 20 psi	Hydraulic Study** gpm @ 20 psi	Needed Fire Flow gpm @ 20 psi
1*	Main Street @ Brake Hill Terrace	108	38	800	-	2250
2 *	Main Street @ Prospect Street	83	54	1700	1270	2000/4000
3*	Main Street West of Chase Street	57	37	1400	-	1750
4*	Main Street East of Training Field Road	90	55	1400	-	1750
5*	Main Street @ Maple Street	83	54	1700	1500	2500
6*	Main Street East of Garden Street	100	51	1400	1450	1750
7	Middle Street @ Crane Neck Street	80	52	1100	-	750
8	Church Street @ Bridge Street	123	58	1400	-	750
9	End of Hilltop Circle	83	47	900	-	500
10	Parsons Road @ Main Street	110	38	1100	-	500
11	Meadow Sweet @ Summer Sweet Lane	108	47	1300	-	750

*Tests noted with an asterisk did not meet the ISO fire flow requirement.
**Estimated available flow is based on fire flow tests conducted under this Hydraulic Study. The location of the fire flow tests were conducted in the approximate location as 1988 ISO test.

Table No. 3-3
Year 2020 Water Storage Evaluation
West Newbury, Massachusetts

Storage Components (MG)	Low Service Area	High Service Area
Equalization (Peak Hourly Flows)*	0.04	0.15
Basic Fire Flow**	0.24	0.24
Emergency Storage***	-	-
Total Storage Requirement (MG)	0.28	0.39
Available Storage (MG)	0.43	0.18
Surplus/(Deficit)	0.15	(0.21)

* Equalization is 25% of the 2020 MDD.

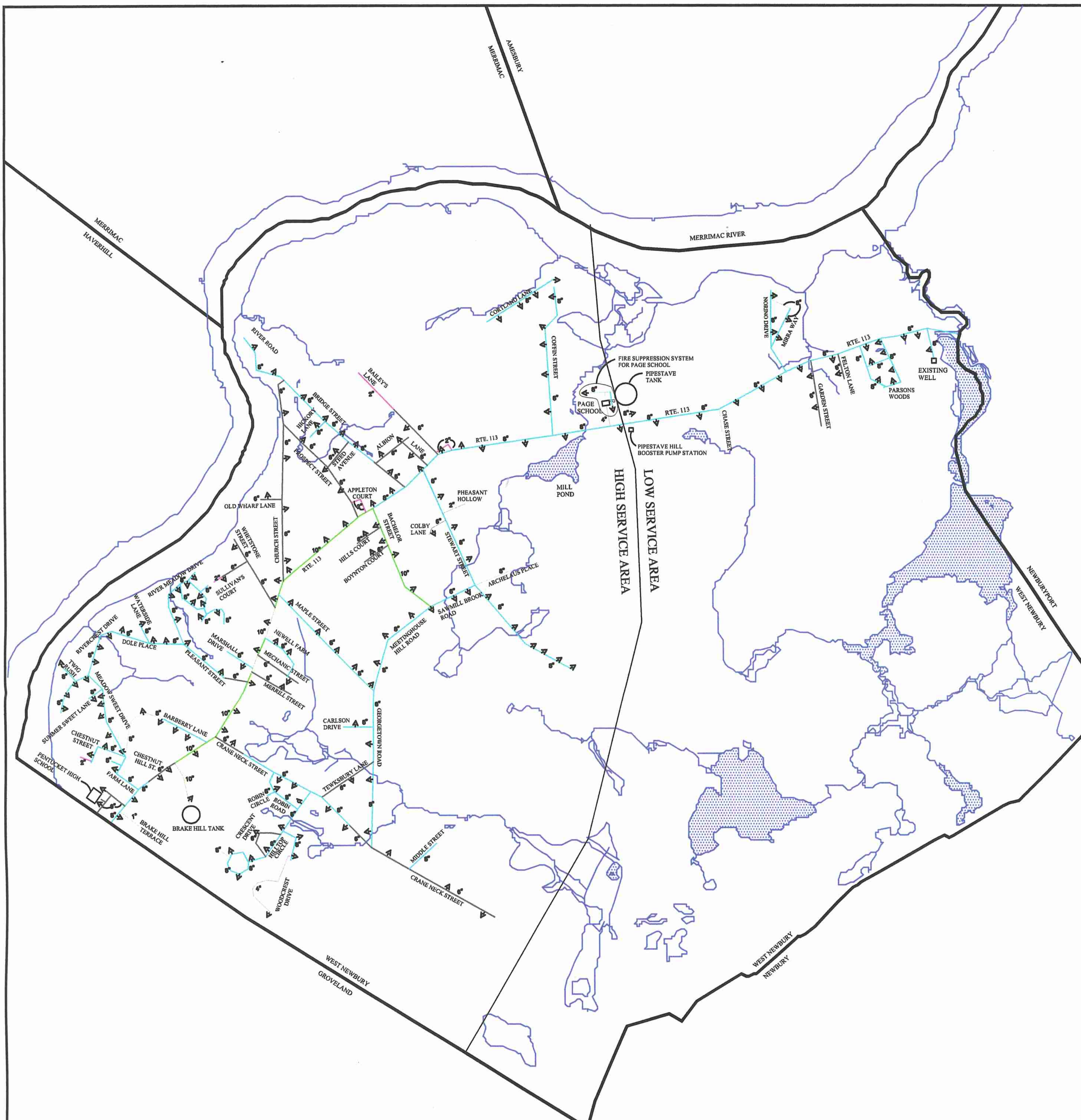
** Basic fire flow requirement is 2,000 gpm for 2 hours or 0.24 million gallons.

*** Emergency storage component waived - In the low service area, there is an emergency generator and an interconnection with Newburyport. In the high service area, an emergency generator is recommended in Section 4.

Table No. 3-4
Comparison of Supply versus Demand
West Newbury, Massachusetts

	2001	**2020**
Wellfield Out of Service (mgd)	0.00	0.00
Summer ADD (mgd)	-0.29	-0.42
Summer ADD Deficit with	(0.29)	(0.42)

	2001	**2020**
Total Sustainable Yield (Summer)	0.06	0.06
Maximum Day Demand (mgd)	-0.44	-0.72
Maximum Day Demand Deficit	(0.38)	(0.66)



DRAWING NO. 3-1

LEGEND	
	1" DIAMETER WATER SERVICE
	2" DIAMETER WATER MAIN
	4" DIAMETER WATER MAIN
	6" DIAMETER WATER MAIN
	8" DIAMETER WATER MAIN
	10" DIAMETER WATER MAIN
	HYDRANT
	WATER STORAGE TANK

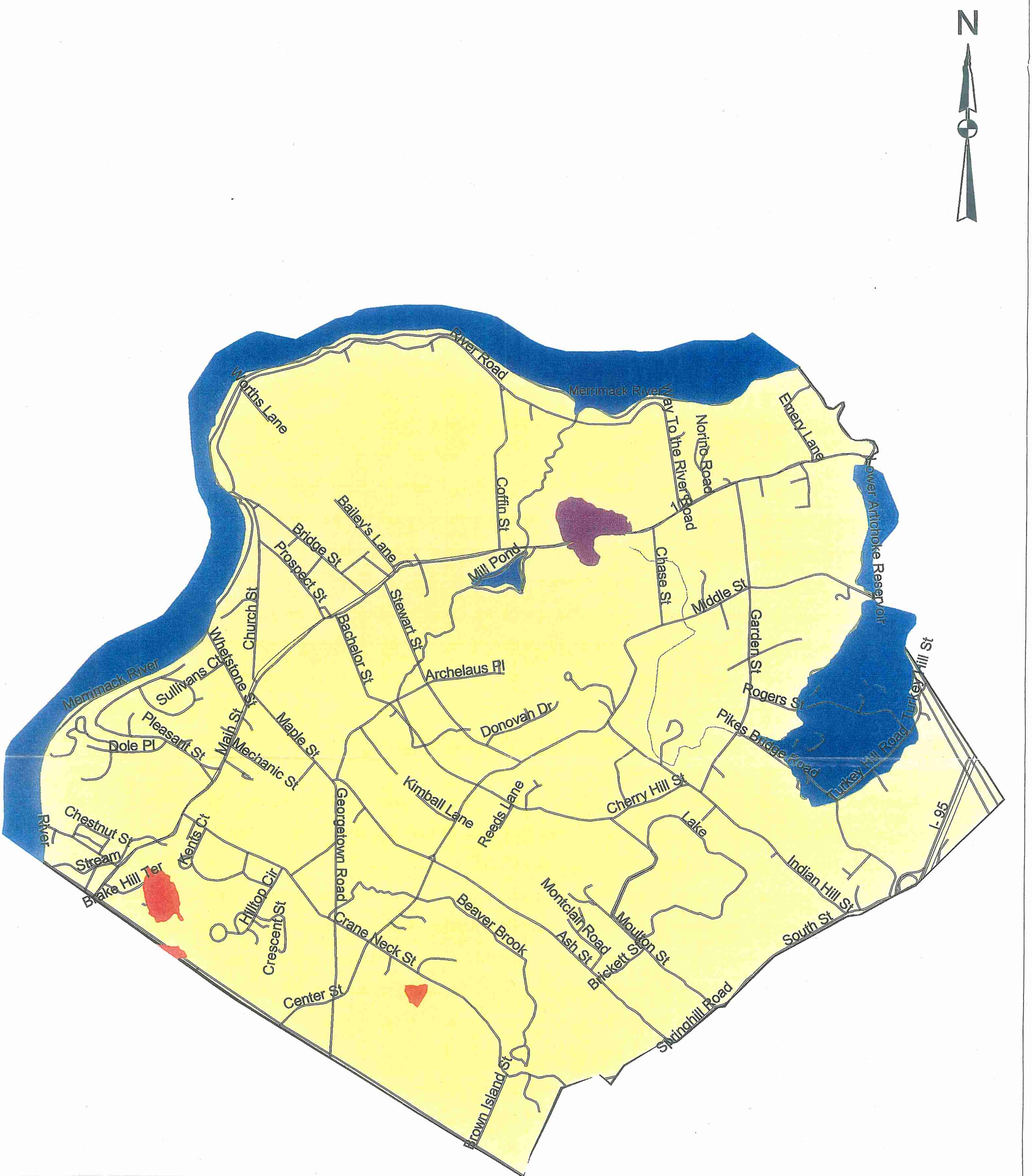
WATER DISTRIBUTION SYSTEM MAP

TOWN OF WEST NEWBURY

TATA & HOWARD, INC.

OCTOBER 2003

0 1000 2000 3000
SCALE FEET



LEGEND

(purple)	Elevations Above Low Service Area Limits (Existing 2001 Conditions)
(red)	Elevations Above High Service Area Limits (Existing 2001 Conditions)

Tata & Howard, Inc. Westborough, MA Date: Oct. 2003 Scale: 1 : 33,000	Limits of Existing Service Areas Town of West Newbury, MA	Figure No. 3-1

Section 4

Conclusions and Recommendations

Section 4

Conclusions and Recommendations

General

This section summarizes the findings of the study and presents a phased plan for recommended improvements and associated costs. The phasing of improvements allow for constructing the necessary distribution system improvements such as the development of new water supply sources, the construction of a new water storage facility and the construction of new water mains over an extended period of time as funds allow. This is a common requirement for small towns such as West Newbury with a limited customer base.

Conclusions

Water Distribution & Storage

Due to the age and size of the mains in the transmission grid, distribution reinforcements are required to strengthen and improve flow throughout the system. In addition, several infrastructure improvements are required to replace or rehabilitate 6-inch diameter and small mains and older unlined water mains in the system. The availability of the updated hydraulic computer model provides the Town with an important tool in evaluating system expansion or changes in the future, particularly when evaluating the impacts of proposed new developments or water main replacement.

Since the distribution system is comprised of two service areas, storage was evaluated separately for each service area. Based on the water storage evaluation presented in this study, no additional storage is required in the low service area. However, additional storage is required in the high service area to meet demand fluctuations within the system and furnish an adequate reserve for fire fighting.

The total required storage for the high service area for the design year 2020 is approximately 0.39 MG. The usable storage of the Brake Hill water storage tank is equivalent to 58 percent of its total volume of 0.312 MG tank or 0.18 MG. Therefore, additional storage is required for the high service area.

Water Supply

As discussed, in Section 3, the sources must be able to meet the projected 2020 summer ADD with the largest source off-line and the projected 2020 MDD with all sources on-line in order to be considered adequate. Since the Town has one existing supply source, the Town is unable to meet the projected 2020 summer ADD. As previously stated, during high demand periods when the water table is low, the Town's total available supply from the Wellfield is approximately 0.06 mgd. Therefore, a projected MDD deficit of 0.66 mgd exists.

Recommendations

Typically, in water distribution system studies, improvements are divided into three components. The first component is general recommendations, followed by Priority I Improvements, which generally address water supply and storage and ISO fire flow deficiencies. Lastly, Priority II

Improvements are intended to mitigate additional deficiencies at extremities and assist in meeting general fire flow requirements. However, in an effort to provide the Town with a plan, in which each year monies can be appropriated to address necessary improvements, the improvements are divided into six phases, which are prioritized in ranking order. The first four phases address water supply, storage and ISO fire flow deficiencies. The last two phases address deficiencies at extremities and general fire flow requirements. A listing of the recommended improvements are summarized below:

1. Water Supply - Existing and projected deficit exists.
2. Storage and Associated Transmission Mains
3. High Service Area Transmission Mains
4. Low Service Area Transmission Mains
5. Secondary Transmission Mains and ISO Fire Flow Requirements
6. Restrictions

Each phase, if the Town chooses, can be carried out over a period of years, depending on the availability of funds. This approach may provide the Town with a more realistic recommendation plan so deficiencies can be addressed relative to the costs associated with these improvements. Table No. 4-1 presents the estimated costs for the Phased Recommended Improvements. Costs are based on the June 2002 Engineering News Record (ENR) index and include a 25 percent allowance for engineering and contingencies. It should be noted that the estimated construction costs are based on the current ENR index, which represents current market rates. However, the actual costs may be higher depending on the time construction takes place and the current construction trend (i.e. slow, busy). These improvements are also present on the recommended improvements map provided in Appendix D.

It should be noted that there are several loan and grant programs available to small systems such as West Newbury that can assist in funding these important recommended improvements. The Massachusetts Department of Environmental Protection (DEP) administers the Drinking Water State Revolving Fund (DWSRF) program. This program offers communities low interest loans for water related projects such as the construction of water treatment facilities and chemical feed systems, booster pump stations, water storage facilities and water main replacements. The Town should consider applying for DWSRF funding for some of the recommended improvements.

<u>Phase I – Water Supply</u>

1. If the safe yield of the wellfield is 0.20 mgd, as previously reported by Cammett & Kutensky Engineering, the Town should be able to pump this volume year round and not only during the winter/spring season. Therefore, several factors relative to maximizing this source need to be evaluated. Since the wellfield has not been professionally cleaned and rated recently, we recommend that the wellfield be cleaned during low demand periods to estimate the capacity of each well. Upon completion of the wellfield cleaning, we recommend that an evaluation of the wellfield's configuration be conducted to determine if it is inefficient and requires redesigning. The costs include additional test drilling to locate potential well sites on the property.

The estimated cost of the wellfield cleaning and the wellfield configuration evaluation is $10,000 and $30,000, respectively.

1. Although maximizing the existing site and purchasing water from neighboring communities may assist in mitigating the water supply deficit, the Town should investigate developing an additional water supply sources in addition to the two bedrock wells. It is our understanding that the Artichoke River site has been identified as a potentially viable surficial supply source. Since it is anticipated that a wellfield would be developed on this site, in accordance with DEP wellhead protection guidelines, the Zone I radius for a wellfield is 250 feet, while the Zone I of the Andreas and Dunn Wells are 400 feet. Therefore, we recommend that the Town conduct further testing on the Artichoke River site to evaluate the potential yield from the site. If the results of the test well work are favorable, we recommend the development of this site.

   The estimated cost to conduct a test well investigation at this site is $30,000. The estimated cost to develop this site is approximately $750,000, which include wellfield construction and a remote pump control station, but excludes treatment costs.

3. In order to meet existing demands, the Town currently relies on the City of Newburyport to supplement their water supply. It is recommended that the Town investigate the possibility of an interconnection with the Town of Groveland. There are several advantages to pursuing this interconnection. Constructing an interconnection with Groveland will reduce the Town's dependency on the City of Newburyport. Since there is no formal agreement with the City, the Town of West Newbury is subject to unexpected rate increases, which may negatively impact the Water Department's operating budget. Based on the 2001 Tighe & Bond Water Rate Study, the Town of Groveland rates are lower than the City of Newburyport rates. As such, cost savings may result. Lastly, since the existing booster pump station, which supplies water to the high service area, has no emergency power source, an interconnection with the Town of Groveland could potentially supply water to the high service area during an emergency condition. It is recommended that the Town pursue discussions with the Town of Groveland in an effort to evaluate if Groveland is interested in pursuing this interconnection and has adequate supply to meet West Newbury's demands. If both parties agree to pursue the interconnection, the Towns should enter into a formal agreement, which will outline the unit costs for water and any conditions that may apply.

   The estimated probable construction cost of this improvement is $ 25,000, which includes costs associated with upgrading the piping at the interconnection.

4. Although maximizing the capacity of the existing wellfield will assist the Town in meeting existing demands, it will not mitigate the projected water supply deficit. In an effort to assist in mitigating the water supply deficit, it is recommended that the Town pursue the development of the Andreas Well and the Dunn Well.

The estimated costs associated with developing the Andreas Well include the completion of the production well, a remote pump control station, and approximately 3,500 linear feet of transmission mains from this well site to the Dunn Well site, since there are no existing water mains in the vicinity of the Andreas site. Please note that the costs associated with this improvement do not include costs associated with the proposed Dunn Well site. The estimated probable construction costs for the pump control station with production well and the transmission mains are approximately $775,000 and $440,000, respectively. We have added to the estimated pump station cost a budgetary estimate of $150,000 to extend three phase power to the Andreas site. If treatment for the removal of arsenic and radon, chemical feed for corrosion control and manganese sequestering, and SCADA controls are added the estimated probable construction costs for this improvement is approximately $1,650,000, which does not include the estimated cost of $440,000 for the transmission main.

The estimated capital costs associated with the Dunn Well include the construction of the production well, pump station and approximately 2,000 linear feet of transmission mains since there are no water mains in the vicinity of the Dunn property and they would be required to interconnect this site and the Andreas site into the Town's existing distribution system. The transmission main would be located on Chase Street and would tie into the Town's existing distribution system on Main Street. The estimated probable construction costs for the remote pump control station with production well and the transmission mains are approximately $775,000 and $250,000, respectively. We have added to the estimated pump station cost a budgetary estimate of $150,000 to extend three phase power to the Dunn site. If treatment for the removal of arsenic and radon, chemical feed for corrosion control and manganese sequestering, and SCADA controls are included the estimated probable construction costs for this improvement is approximately $1,500,000, which does not include costs associated with the Andreas property or the recommended transmission main for this improvement.

If both sites are developed, it is recommended to pump the water from the Andreas site to the Dunn site and construct one treatment plant to treat water from both the Dunn and Andreas sites. This would reduce the estimated probable construction cost of this improvement associated with treatment since only one plant would require construction. However, this would not reduce the piping required to interconnect these wells into the Town's existing distribution system. Under this scenario, the estimated costs associated with transmission mains are approximately $690,000. A pump control station would be required at the Andreas site at an estimated cost of approximately $775,000 an the estimated costs associated with a single treatment facility located at the Dunn site is approximately $1,650,000.

Although the initial capital costs associated with developing new water supply sources are costly, development of these potential supply sources will reduce the amount of water the Town purchases from the City of Newburyport, as well as decrease the dependency on Newburyport to meet demands.

Phase 2 – Water Storage & Associated Transmission Mains

5. As stated in Section 3, emergency power is required at the Main Street booster pump station in order to provide the ADD to the high service area and thus reduce the estimated needed storage. It is recommended that an exterior generator be installed at the booster pump station. However, if an interconnection to the Town of Groveland is implemented than this improvement is not required.

   The estimated probable construction cost of this improvement is $ 85,000.

6. In order to provide adequate storage in the high service area, it is recommended that a new water storage facility with a usable capacity of at least 0.39 million gallons (MG) be constructed at the existing Brake Hill Tank site if additional space at the Town's existing tank site is available. In addition, approximately 1,100 linear feet of 12-inch diameter water main will be required to connect the proposed storage facility to the existing distribution system. It is not uncommon for water storage tanks to have a service life in excess of 120 years, as such we recommend that the Town maintain the existing Brake Hill Tank, through routine maintenance, so it can also be used for additional storage.

   The estimated probable construction cost of this improvement is $800,000.

7. Currently, the transmission main from the Brake Hill Tank to the existing distribution system (Main Street) is an 10-inch diameter water main. In order to reduce the headloss through the existing main and to increase the inherent carrying capacity of this main, it is recommended that a new 12-inch diameter water main from the Brake Hill Tank to the intersection of Main Street be constructed. This improvement can be completed once the new water storage tank is constructed, which will allow for the Brake Hill Tank being taken out of service while the new 12-inch diameter water main is being constructed. This improvement will assist in mitigating estimated needed ISO fire flows on Main Street.

   The estimated probable construction cost of this improvement is $ 154,000.

Phase 3 - High Service Area Transmission Mains

8. In an effort to improve the east to west transmission grid in the high service system, a new 16-inch diameter main on Main Street from the new 12-inch main (Improvement No. 7) to Bachelor Street is recommended. This improvement will help to assist in improving several ISO fire flow deficiencies in the distribution system. In addition, this improvement will replace old, undersized water mains in this area and benefit the entire high service area by strengthening the main transmission grid of this service area.

   The estimated probable construction cost of this improvement is $ 881,000.

9. In an effort to further strengthen the transmission grid in the high service area system, cleaning and lining the existing 8-inch water main on Main Street from the existing 6-inch

diameter water main to the Groveland town boundary is recommended. Prior to cleaning and lining the existing 8-inch diameter main on Main Street, it is recommended that the Town take coupons (small side samples) of the main to determine the condition of the main. Additionally, it is recommended to construct a new 8-inch diameter water main on Main Street to replace the portion of 6-inch diameter water main that creates a restriction in the distribution system. This improvement, along with the above recommendations, will significantly improve transmission in the high service system while improving available fire flows in the area. This improvement will also provide the inherent capacity to meet furture maximum day demands through water purchased from the Town of Groveland.

The estimated probable construction cost for this improvement is $263,000.

10. An ISO estimated needed fire flow of 1,750 gpm is required on Main Street east of Training Field Road. Although the above recommended transmission reinforcements (Improvement No. 8) improve flow to this area, additional improvements are needed to meet the estimated ISO fire flow requirement. Additional improvements recommended include a new 12-inch diameter water main on Main Street from the intersection of Bachelor Street to Training Field Road. This improvement, along with the proposed 16-inch diameter water main on Main Street, will assist in meeting the ISO fire flow requirement and further strengthen the west to east transmission grid. In addition, the new water mains will replace older, undersized tuberculated mains in the central portion of the high service area grid.

    The estimate of probable construction cost for this improvement is $274,000.

<u>Phase 4 – Low Service Area Transmission Mains</u>

11. Several ISO fire flows ranging from 500 to 1,750 gpm are needed along Main Street west of Chase Street and east of Garden Street. It is recommended that a new 12-inch diameter water main be constructed on Main Street from the existing tank to Garden Street. This improvement will strengthen the transmission grid of the low service system and replace older, tuberculated water mains, while mitigating these ISO fire flow requirements.

    The estimate of probable construction cost for this improvement is $ 677,000.

<u>Phase 5 – Secondary Transmission Mains and ISO Fire Flow Requirements</u>

12. An estimated needed ISO fire flow of 2,250 gpm is required at the intersection of Main Street and Brake Hill Terrace. In addition to the transmission main improvements recommended in Phase 3, Improvement No. 8 and 9, the following infrastructure improvements are recommended:

- a new 8-inch diameter main on Crane Neck Street from the intersection of Main Street to the existing 8-inch diameter water main on Crane Neck Street,
- clean and line the existing 8-inch water main on Crane Neck Street from the existing 6-inch diameter water main to the intersection of Hilltop Circle,

These improvements will assist in providing the inherent capacity to meet the 2250 gpm ISO fire flow requirement, as well as two additional inadequate fire flows along Meadow Street and Hilltop Circle. In addition, the construction of the new 8-inch diameter water mains on Main Street and Crane Neck Street will eliminate two bottlenecks in the system by replacing portions of inadequately sized 6-inch diameter water main.

The estimate of probable construction cost for this improvement is $ 233,000.

13. An ISO estimated needed fire flow of 2,500 gpm is required at the intersection of Main Street and Maple Street. In addition to the previously mentioned Phase 3 and Phase 5, improvements, it is recommended that a new 8-inch diameter water main on Tewksbury Lane from Crane Neck Street to Meetinghouse Hill Road be constructed. Additionally, this improvement will eliminate the bottleneck created by the existing 6-inch diameter water main on Tewksbury Lane. Eliminating this restriction will reduce headloss in this portion of the system and improve transmission and flows.

    This improvement will assist in providing the inherent capacity to meet the estimated ISO fire flow requirement, as well as smaller estimated fire flow requirements in this portion of the system, while improving west to east transmission in the high service area.

    The estimate of probable construction cost for this improvement is $144,000.

14. An ISO estimated needed fire flow of 750 gpm at the intersection of Crane Neck Street and Middle Street is required. In addition to the new 16-inch diameter water main on Main Street recommended in Phase 3, it is recommended that the existing 6-inch diameter water main on Crane Neck Street be replaced with a new 8-inch diameter water main. This improvement, along with the transmission improvements on Main Street, will assist in mitigating the ISO fire flow requirement, while improving transmission to this extremity of the high service area.

    The estimate of probable construction cost for this improvement is $ 400,000.

<u>Phase 6 – Restrictions and Estimated Fire Flow Requirements</u>

15. An estimated needed fire flow of 750 gpm is required at the end of Cortland Lane. Although the previously recommended improvements on Main Street improve transmission in this portion of the system, it is recommended that a new 12-inch diameter main be constructed on Main Street from the intersection of Bailey's Lane to the intersection of Coffin Street. This improvement will assist in meeting the estimated fire flow requirement. In addition, this improvement will replace a long length of older, tuberculated 8-inch diameter water main.

    The estimate of probable construction cost of this improvement is $ 367,200.

16. An estimated needed fire flow of 750 gpm is required at the end of Farm Lane. Currently, Farm Lane is serviced by a 6-inch diameter water main. It is recommended that a new 8-inch diameter water main be constructed on Farm Lane from the intersection of Main Street to the end of this street. This improvement will increasing the inherent carrying capacity, thereby assist in mitigating the estimated needed fire flow.

    The estimated probable construction cost of this improvement is $ 89,000.

17. There are portions of the distribution system where smaller water mains are the sole means of transporting water between larger mains. Specifically, there are portions of 6-inch diameter water mains on Harrison Avenue that create a bottleneck in the system. In an effort to eliminate these restrictions, it is recommended that a new 8-inch diameter water main be constructed on Harrison Avenue from the intersection of Main Street to the existing 8-inch diameter water main on Harrison Avenue. Eliminating this restriction will increase the inherent carrying capacity of this main.

    The estimate of probable construction costs for this improvement is $56,000.

18. Currently, Appleton Court and Middle Street are serviced by 2-inch diameter service lines that do not have the inherent carrying capacity to meet the estimated fire flows of 750 gpm. It is recommended that the service lines on Appleton Court and Middle Street be replaced with 6-inch diameter mains. New 6-inch diameter mains will increase the capacity along these roads and eliminate older, smaller diameter mains in the system. The following improvements and associated estimate of probable construction costs are recommended:

A.	Appleton Court	$ 49,000
B.	Middle Street	$ 77,000

19. Currently, Bailey's Lane is serviced by a 6-inch diameter water main and a 2-inch diameter service line. It is recommended that an 8-inch diameter water main on Bailey's Lane from the intersection with Main Street to its end be constructed. This improvement will provide the inherent carrying capacity to meet the estimated fire flow requirement of 750 gpm.

    The estimated probable construction cost of this improvement is approximately $ 264,000.

20. Currently, Garden Street is serviced by a heavily tuberculated 6-inch diameter water main. It is recommended that a new 8-inch diameter water main be constructed on Garden Street from the intersection of Main Street to the end of the existing water main. A new 8-inch diameter water main is recommended rather than rehabilitate the existing 6-inch diameter water main due to the longer length of the water main. In addition to increasing the carrying capacity, this improvement will assist in mitigating the estimated needed fire flow requirement of 750 gpm.

    The estimated probable construction cost of this improvement is approximately $153,000.

21. In various portions of the system, heavily tuberculated 6-inch diameter water mains are used to service dead end streets. It is recommended that these mains be cleaned and lined. Specifically, the mains on Albion Lane, Merrill Street, and Sullivan Court do not have the transmission capacity to meet the estimated fire flows of 750 gpm. Rehabilitating the existing 6-inch diameter mains will increase the inherent carrying capacity of these mains and assist in mitigating the estimated fire flow requirement. The following improvements and associated estimate of probable construction costs are recommended:

A.	Albion Lane	$ 132,800
B.	Merrill Street	$ 78,800
C.	Sullivan Court	$ 69,800

<u>General Recommendations</u>

Distribution System Expansion

Whenever improvements or expansion of a water distribution system occur, factors such as size and location of the main should be considered in order to provide adequate flows and pressures. Water mains, which extend for 600 feet or more without an interconnection, should have a minimum diameter of 8 inches. If a section of proposed main is less than 600 feet, the minimum water main diameter could be reduced to 6 inches if the main is used to complete a water main loop and the fire flow requirement is minimal. Wherever possible, dead end mains should be eliminated by looping or interconnecting. Parallel water mains should be interconnected at reasonable intervals. All older and smaller water mains that are not able to meet fire flow requirements in an area should be replaced with larger diameter mains. In addition, "bottlenecks" such as a smaller water main being the sole means of transporting water between larger mains should be eliminated. Currently, no specific looping projects are recommended. However, as the Town extends their distribution system looping of dead ends should be considered in long-term planning schedules.

Water Audit and Leak Detection

In accordance with the Town's Water Management Act Permit, system wide biannual leak detection is required. The testing is conducted biannually to remain compliant with the WMA permit. The Town recently completed a leak detection survey in the Fall/Winter of 2001. The last previously completed leak detection survey was completed in 1999. We recommend that the Town continue with this biannual testing to minimize unaccounted-for water loss and to remain compliant with their WMA permit.

General Maintenance

The Town should continue with its regularly scheduled maintenance program which includes hydrant flushing twice a year, routine inspection and maintenance at the pump stations, and meter testing. The general maintenance program should include monitoring of the capacity of the wellfield so that well cleaning and redevelopment, when required, can be scheduled during low demand periods. Although the Water Department has cleaned the wellfield twice over the past three years, it is recommended that a professional well cleaning be conducted every 3 to 5 years. Therefore, we recommend that each well be cleaned and tested to estimate its pumping capacity.

The Town should implement a meter installation and replacement program. AWWA recommends meter replacement every seven to ten years, depending on the water quality of the system and use of the meter. In addition, a totalizing flow meter is recommended for the Town's existing booster pump station. The flow meter will allow the Town to record the volume of water pumped from the low service area to the high service area for comparison to overall system wide water usage.

The Town conducts an annual water distribution system replacement program, during which they identify valves and hydrants which do not function as intended. By eliminating components that are defective, proper system operation is facilitated. In addition, by replacing hydrants that are old or broken, fire protection capabilities in the system will be improved. The Town should continue its efforts on this program.

Table No. 4-1
Phase 1 - 6 Improvements
West Newbury, Massachusetts

Phase	Improvement	Description/Location	From	To	Proposed Dia.	Length	Estimated Cost*
1	1	Wellfield Cleaning					$10,000
		Wellfield Evaluation					$30,000
	Subtotal:						$40,000
	2	Artichoke River Site Investigation					$30,000
		Site Development/Control Station					$750,000
	Subtotal:						$780,000
	3	Groveland Interconnection					$25,000
	4	Andreas Well Treatment Facility					$2,090,000
		Dunn Well Treatment Facility					$1,750,000
	Subtotal:						$3,840,000
Total:							$4,685,000
2	5	Emergency Generator @ BPS					$85,000
	6	Water Storage Tank					$800,000
	7	Brake Hill Tank Main	Tank	Main Street	12	1120	$154,000
Total:							$1,039,000
3	8	Main Street	New 12" (Impr. 7)	Bachelor Street	16	5870	$881,000
	9	Main Street	Break Hill Tank Road	Existing 8" Main	8	500	$63,000
		Main Street	Existing 8" Main	Groveland Townline	C&L (8")	2280	$200,000
	Subtotal:						$263,000
	10	Main Street	Bachelor Street	Training Field Road	12	1990	$274,000
Total:							$1,418,000

Table No. 4-1
Phase 1 - 6 Improvements
West Newbury, Massachusetts

Phase	Improvement	Description/Location	From	To	Proposed Dia.	Length	Estimated Cost*
4	11	Pipe Stave Tank Main	Tank	Main Street	12	420	$58,000
		Main Street	Pipe Stave Tank	Garden Street	12	4495	$619,000
Total:							$677,000
5	12	Crane Neck Street	Main Street	Existing 8" Main	8	760	$76,000
		Crane Neck Street	Existing 6" Main	Hilltop Circle	C&L (8")	1790	$157,000
	Subtotal:						$233,000
	13	Tewksbury Lane	Crane Neck Street	Meetinghouse Hill Road	8	1440	$144,000
	14	Crane Neck Street	Existing 6-inch Main	End	8	4000	$400,000
Total:							$777,000
6	15	Main Street	Bailey's Lane	Coffin Street	12	2670	$367,200
	16	Farm Lane	Main Street	End of Farm Lane	8	890	$89,000
	17	Harrison Avenue	Main Street	Existing 8" Main	8	560	$56,000
	18	Appleton Court	Main Street	Prospect Street	6	560	$49,000
		Middle Street	Crane Neck Street	End	6	880	$77,000
	Subtotal:						$126,000
	19	Bailey's Lane	Main Street	End	8	2640	$264,000
	20	Garden Street	Main Street	End of existing main	8	1530	$153,000
	21	Albion Lane	Main Street	Bridge Street	C&L (6")	1770	$132,800
		Merrill Street	Main Street	End	C&L (6")	1050	$78,800
		Sullivan Court	Whetstone Street	End	C&L (6")	930	$69,800
	Subtotal:						$281,400
Total:							$1,336,600

Note: All costs include 25 percent for Engineering and contingencies.
Estimated costs are based on December 2001 ENR Engineering News Record Index

Appendix A

Water Distribution Map

Appendix B

Fire Flow Test Data

Tata & Howard, Inc.

Fire Flow Test

Community:	West Newbury		
Client:	West Newbury		
Date:	10/23/01	**Time:**	10:45 pm/11:00 pm
Weather:	Clear	**Inspector:**	Mike & Bob – W.N.
			Shira & Jenna – T&H

Hydrant Location	**Test No. 1**	**Test No. 2**
Flowing Hydrant:	End of Mirra	Main near Garden Street
Residual Hydrant:	End of Norino	#841 Main St. (Rte. 113)

	Test No. 1	**Test No. 2**
Flowing Hydrant:		
Flow Opening (Inches):	2-1/2	2-1/2
No. Butts Flowing:	1	1
Static Pressure (psi):	71	95-96
Pitot Reading (psi):	28	44
Hydrant Coefficient:	0.9	0.9
Flow (GPM) (Qf):	987 * 0.9 = 889 ✓	1238 * 0.9 = 1114 ✓
Residual Hydrant:		
Static Pressure (Hs):	86	82
Residual Pressure (Hf):	48	44

Test No. 1 $Q = \underline{889}\ (Qf) \times \left\{ \frac{\underline{86}\ Hs - \underline{20}}{\underline{86}\ Hs - \underline{48}\ Hf} \right\}^{.54} = \underline{1198}\ GPM\ @\ \underline{20}\ PSI$ ✓

Test No. 2 $Q = \underline{1114}\ (Qf) \times \left\{ \frac{\underline{82}\ Hs - \underline{20}}{\underline{82}\ Hs - \underline{44}\ Hf} \right\}^{.54} = \underline{1451}\ GPM\ @\ \underline{20}\ PSI$ ✓

Checked By: JWK

ff_1&2.doc

Tata & Howard, Inc.

Fire Flow Test

Community:	West Newbury		
Client:	West Newbury		
Date:	10/23/01	**Time:**	11:08 pm/11:20 pm
Weather:	Clear	**Inspector:**	Mike & Bob – W.N. Shira & Jenna – T&H

Hydrant Location	**Test No. 3**	**Test No. 4**
Flowing Hydrant:	Last Hydrant on Garden	Parsons Road
Residual Hydrant:	#841 Main St. (Rt. 113)	#20 Parsons Road

	Test No. 3	**Test No. 4**
Flowing Hydrant:		
Flow Opening (Inches):	2-1/2	2-1/2
No. Butts Flowing:	1	1
Static Pressure (psi):	86	96
Pitot Reading (psi):	24	36
Hydrant Coefficient:	0.9	0.9
Flow (GPM) (Qf):	914 * 0.9 = 823	1121 * 0.9 = 1009
Residual Hydrant:		
Static Pressure (Hs):	82	98
Residual Pressure (Hf):	58	42

Test No. 3 $Q = \underline{823}\,(Qf) \times \left\{ \dfrac{\underline{82}\ Hs - \underline{20}}{\underline{82}\ Hs - \underline{58}\ Hf} \right\}^{.54} = \underline{1374}$ GPM @ $\underline{20}$ PSI

Test No. 4 $Q = \underline{1009}\,(Qf) \times \left\{ \dfrac{\underline{98}\ Hs - \underline{20}}{\underline{98}\ Hs - \underline{42}\ Hf} \right\}^{.54} = \underline{1207}$ GPM @ $\underline{20}$ PSI

Checked By: JWR

ff_3&4.doc

Tata & Howard, Inc.

Fire Flow Test

Community:	West Newbury		
Client:	West Newbury		
Date:	10/23/01	**Time:**	11:30 pm/11:45 pm
Weather:	Clear	**Inspector:**	Mike & Bob – W.N.
			Shira & Jenna – T&H

Hydrant Location	**Test No. 5**	**Test No. 6**
Flowing Hydrant:	Rte. 113 near wellfield	Last Hyd. on Woodcrest
Residual Hydrant:	Main and Parsons Road	#4 Woodcrest Street

	Test No. 5	**Test No. 6**
Flowing Hydrant:		
Flow Opening (Inches):	2-1/2	2-1/2
No. Butts Flowing:	1	1
Static Pressure (psi):	112	82
Pitot Reading (psi):	47	31
Hydrant Coefficient:	0.9	0.9
Flow (GPM) (Qf):	1279 * 0.9 = 1151 ✓	1040 * 0.9 = 936 ✓
Residual Hydrant:		
Static Pressure (Hs):	110	86-87
Residual Pressure (Hf):	48	38

Test No. 5 $Q = \underline{1151}\ (Qf) \times \left\{ \dfrac{\underline{110}\ Hs - \underline{20}}{\underline{110}\ Hs - \underline{48}\ Hf} \right\}^{.54} = \underline{1408}\ GPM\ @\ \underline{20}\ PSI$ ✓

Test No. 6 $Q = \underline{936}\ (Qf) \times \left\{ \dfrac{\underline{86}\ Hs - \underline{20}}{\underline{86}\ Hs - \underline{38}\ Hf} \right\}^{.54} = \underline{1112}\ GPM\ @\ \underline{20}\ PSI$ ✓

Checked By: ____________________

Tata & Howard, Inc.

Fire Flow Test

Community:	West Newbury		
Client:	West Newbury		
Date:	10/23/01	**Time:**	12:00 am/12:15 am
Weather:	Clear	**Inspector:**	Mike & Bob – W.N. Shira & Jenna – T&H

Hydrant Location	**Test No. 7**	**Test No. 8**
Flowing Hydrant:	Crane Neck Last Hyd.	Meeting House Road
Residual Hydrant:	#151 Crane Neck Road	#34 Meeting House Road

	Test No. 7	**Test No. 8**
Flowing Hydrant:		
Flow Opening (Inches):	1-1/8	2-1/2
No. Butts Flowing:	1	1
Static Pressure (psi):	40	74
Pitot Reading (psi):	30	44
Hydrant Coefficient:	0.9	0.9
Flow (GPM) (Qf):	207 * 0.9 = 186	1238 * 0.9 = 1114
Residual Hydrant:		
Static Pressure (Hs):	54-56	40-42
Residual Pressure (Hf):	50	18

Test No. 7 $Q = \underline{186}\,(Qf) \times \left\{ \dfrac{\underline{55}\ Hs - \underline{20}}{\underline{55}\ Hs - \underline{50}\ Hf} \right\}^{.54} = \underline{532}\ GPM\ @\ \underline{20}\ PSI$

Test No. 8 $Q = \underline{1114}\,(Qf) \times \left\{ \dfrac{\underline{41}\ Hs - \underline{20}}{\underline{41}\ Hs - \underline{18}\ Hf} \right\}^{.54} = \underline{1061}\ GPM\ @\ \underline{20}\ PSI$

Checked By: JWR

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Tata & Howard, Inc.

Fire Flow Test

Community:	West Newbury
Client:	West Newbury

Date:	10/23/01	Time:	12:30 am/12:55 am
Weather:	Clear	Inspector:	Mike & Bob – W.N.
			Shira & Jenna – T&H

Hydrant Location	Test No. 9	Test No. 10
Flowing Hydrant:	Last Hyd. Steward Street	Last Hyd. Back of H.S.
Residual Hydrant:	#126 Stewart Street	Farm Street @ H.S.

	Test No. 9	Test No. 10
Flowing Hydrant:		
Flow Opening (Inches):	1-1/8	1-1/8
No. Butts Flowing:	1	1
Static Pressure (psi):	66	112
Pitot Reading (psi):	60	72
Hydrant Coefficient:	0.9	0.9
Flow (GPM) (Qf):	293 * 0.9 = 264	321 * 0.9 = 289
Residual Hydrant:		
Static Pressure (Hs):	64-66	112
Residual Pressure (Hf):	62	74-76

Test No. 9 $Q = \underline{264}\ (Qf) \times \left\{ \dfrac{\underline{65}\ Hs - \underline{20}}{\underline{65}\ Hs - \underline{62}\ Hf} \right\}^{.54} = \underline{1265}$ GPM @ $\underline{20}$ PSI

Test No. 10 $Q = \underline{289}\ (Qf) \times \left\{ \dfrac{\underline{112}\ Hs - \underline{20}}{\underline{112}\ Hs - \underline{75}\ Hf} \right\}^{.54} = \underline{473}$ GPM @ $\underline{20}$ PSI

Checked By: [signature]

Tata & Howard, Inc.

Fire Flow Test

Community: West Newbury
Client: West Newbury

Date: 10/23/01 **Time:** 1:10 am/1:45 am
Weather: Clear **Inspector:** Mike & Bob – W.N.
Shira & Jenna – T&H

Hydrant Location	**Test No. 11**	**Test No. 12**
Flowing Hydrant:	Main St. Near H.S. Entr.	End of Rivercrest Drive
Residual Hydrant:	Rte. 113 across from #64	#24 Rivercrest Drive

	Test No. 11	**Test No. 12**
Flowing Hydrant:		
Flow Opening (Inches):	2-1/2	2-1/2
No. Butts Flowing:	1	1
Static Pressure (psi):	106	113
Pitot Reading (psi):	25	56
Hydrant Coefficient:	0.9	0.9
Flow (GPM) (Qf):	934 * 0.9 = 841 ✓	1397 * 0.9 = 1257 ✓
Residual Hydrant:		
Static Pressure (Hs):	98	112-114
Residual Pressure (Hf):	76	62

Test No. 11 $Q = 841\ (Qf) \times \left\{ \frac{98\ Hs - 20}{98\ Hs - 76\ Hf} \right\}^{.54} = 1666$ GPM @ 20 PSI ✓

Test No. 12 $Q = 1257\ (Qf) \times \left\{ \frac{113\ Hs - 20}{113\ Hs - 62\ Hf} \right\}^{.54} = 1739$ GPM @ 20 PSI ✓

Checked By: JWR

ff_11&12.doc

Tata & Howard, Inc.

Fire Flow Test

Community: West Newbury
Client: West Newbury

Date: 10/23/01
Time: 1:55 am/2:20 am
Weather: Clear
Inspector: Mike & Bob – W.N.
Shira & Jenna – T&H

Hydrant Location	Test No. 13	Test No. 14
Flowing Hydrant:	End of Waterslide Lane	River Meadow Road
Residual Hydrant:	Dole Pl. & Waterslide Ln.	River Meadow Rd. & Ct.

	Test No. 13	Test No. 14
Flowing Hydrant:		
Flow Opening (Inches):	2-1/2	2-1/2
No. Butts Flowing:	1	1
Static Pressure (psi):	106	106
Pitot Reading (psi):	50	42
Hydrant Coefficient:	0.9	0.9
Flow (GPM) (Qf):	1320 * 0.9 = 1188 ✓	1210 * 0.9 = 1089 ✓
Residual Hydrant:		
Static Pressure (Hs):	106	106-108
Residual Pressure (Hf):	70	50

Test No. 13 $Q = \underline{1188}\ (Qf) \times \left\{ \frac{\underline{106}\ Hs - \underline{20}}{\underline{106}\ Hs - \underline{70}\ Hf} \right\}^{.54} = \underline{1901}\ GPM\ @\ \underline{20}\ PSI$ ✓

Test No. 14 $Q = \underline{1089}\ (Qf) \times \left\{ \frac{\underline{107}\ Hs - \underline{20}}{\underline{107}\ Hs - \underline{50}\ Hf} \right\}^{.54} = \underline{1368}\ GPM\ @\ \underline{20}\ PSI$ ✓

Checked By: JWR

Tata & Howard, Inc.

Fire Flow Test

Community:	West Newbury
Client:	West Newbury

Date:	10/23/01	**Time:**	2:35 am/2:45 am
Weather:	Clear	**Inspector:**	Mike & Bob – W.N.
			Shira & Jenna – T&H

Hydrant Location	**Test No. 15**	**Test No. 16**
Flowing Hydrant:	Main and Merrill Street	Main, Maple & Church St.
Residual Hydrant:	#169 Main Street	#323 Main Street

	Test No. 15	**Test No. 16**
Flowing Hydrant:		
Flow Opening (Inches):	2-1/2	2-1/2
No. Butts Flowing:	1	1
Static Pressure (psi):	84	75
Pitot Reading (psi):	54-55	38
Hydrant Coefficient:	0.9	0.9
Flow (GPM) (Qf):	1385 * 0.9 = 1247 ✓	1152 * 0.9 = 1037 ✓
Residual Hydrant:		
Static Pressure (Hs):	86	84
Residual Pressure (Hf):	72	52

Test No. 15 $Q = 1247\,(Qf) \times \left\{ \frac{86\ Hs - 20}{86\ Hs - 72\ Hf} \right\}^{.54} = 2881$ GPM @ 20 PSI ✓

Test No. 16 $Q = 1037\,(Qf) \times \left\{ \frac{84\ Hs - 20}{84\ Hs - 52\ Hf} \right\}^{.54} = 1508$ GPM @ 20 PSI ✓

Checked By: JWR

Tata & Howard, Inc.

Fire Flow Test

Community: West Newbury
Client: West Newbury

Date: 10/23/01
Weather: Clear

Time: 2:55 am/3:05 am
Inspector: Mike & Bob – W.N.
Shira & Jenna – T&H

Hydrant Location	**Test No. 17**	**Test No. 18**
Flowing Hydrant:	Main St. near Prospect	Last Hyd. On River Road
Residual Hydrant:	#350 Main Street	#87 Bridge Street

	Test No. 17	**Test No. 18**
Flowing Hydrant:		
Flow Opening (Inches):	2-1/2	2-1/2
No. Butts Flowing:	1	1
Static Pressure (psi):	72	112
Pitot Reading (psi):	30	50
Hydrant Coefficient:	0.9	0.9
Flow (GPM) (Qf):	1023 * 0.9 = 921 ✓	1320 * 0.9 = 1188 ✓
Residual Hydrant:		
Static Pressure (Hs):	78	128
Residual Pressure (Hf):	46	72

Test No. 17 $Q = \underline{921}\,(Qf) \times \left\{ \frac{\underline{78}\ Hs - \underline{20}}{\underline{78}\ Hs - \underline{46}\ Hf} \right\}^{.54} = \underline{1270}\ GPM\ @\ \underline{20}\ PSI$ ✓

Test No. 18 $Q = \underline{1188}\,(Qf) \times \left\{ \frac{\underline{128}\ Hs - \underline{20}}{\underline{128}\ Hs - \underline{72}\ Hf} \right\}^{.54} = \underline{1694}\ GPM\ @\ \underline{20}\ PSI$ ✓

Checked By: JWR

Tata & Howard, Inc.

Fire Flow Test

Community:	West Newbury		
Client:	West Newbury		
Date:	10/23/01	**Time:**	3:15 am/3:25 am
Weather:	Clear	**Inspector:**	Mike & Bob – W.N. Shira & Jenna – T&H

Hydrant Location	**Test No. 19**	**Test No. 20**
Flowing Hydrant:	Albion Lane	Main @ Bailey Lane
Residual Hydrant:	#25 Albion Lane	Rte. 113 and Stewart

	Test No. 19	**Test No. 20**
Flowing Hydrant:		
Flow Opening (Inches):	2-1/2	2-1/2
No. Butts Flowing:	1	1
Static Pressure (psi):	68	72
Pitot Reading (psi):	20	25
Hydrant Coefficient:	0.9	0.9
Flow (GPM) (Qf):	835 * 0.9 = 752 ✓	934 * 0.9 = 841 ✓
Residual Hydrant:		
Static Pressure (Hs):	76-78	76-78
Residual Pressure (Hf):	40	44

Test No. 19 $Q = \underline{752}\ (Qf) \times \left\{ \frac{\underline{77}\ Hs - \underline{20}}{\underline{77}\ Hs - \underline{40}\ Hf} \right\}^{.54} = \underline{950}$ GPM @ $\underline{20}$ PSI ✓

Test No. 20 $Q = \underline{841}\ (Qf) \times \left\{ \frac{\underline{77}\ Hs - \underline{20}}{\underline{77}\ Hs - \underline{44}\ Hf} \right\}^{.54} = \underline{1130}$ GPM @ $\underline{20}$ PSI ✓

Checked By: JWR

Tata & Howard, Inc.

Fire Flow Test

Community: West Newbury
Client: West Newbury

Date: 10/23/01 **Time:** 3:40 am/3:50 am
Weather: Clear **Inspector:** Mike & Bob – W.N.
Shira & Jenna – T&H

Hydrant Location	**Test No. 21**	**Test No. 22**
Flowing Hydrant:	Last Hyd. Courtland Ln.	Main Street Paige School
Residual Hydrant:	#9 Courtland Lane	#613 Main Street

	Test No. 21	**Test No. 22**
Flowing Hydrant:		
Flow Opening (Inches):	2-1/2	2-1/2
No. Butts Flowing:	1	1
Static Pressure (psi):	84	56
Pitot Reading (psi):	30	18
Hydrant Coefficient:	0.9	0.9
Flow (GPM) (Qf):	1023 * 0.9 = 921 ✓	790 * 0.9 = 711 ✓
Residual Hydrant:		
Static Pressure (Hs):	92-94	92
Residual Pressure (Hf):	52	66

Test No. 21 $Q = \underline{921}\,(Qf) \times \left\{ \frac{\underline{93}\ Hs - \underline{20}}{\underline{93}\ Hs - \underline{52}\ Hf} \right\}^{.54} = \underline{1258}$ GPM @ $\underline{20}$ PSI ✓

Test No. 22 $Q = \underline{711}\,(Qf) \times \left\{ \frac{\underline{92}\ Hs - \underline{20}}{\underline{92}\ Hs - \underline{66}\ Hf} \right\}^{.54} = \underline{1232}$ GPM @ $\underline{20}$ PSI ✓

Checked By: JWR

Appendix C

Hydraulic Input Data

Hydraulic Input Pipe Data
West Newbury, Massachusetts

Pipe	Length (ft)	Diameter (in)	Hazen- Williams C
P-2	712	10	120
P-4	381	8	110
P-8	515	10	120
P-10	532	10	120
P-12	649	10	120
P-14	634	8	100
P-16	201	8	100
P-18	2048	8	110
P-20	1605	8	110
P-26	296	8	110
P-28	2593	8	110
P-30	97	8	110
P-32	350	8	110
P-34	158	8	110
P-36	578	8	110
P-38	900	8	110
P-40	184	10	90
P-42	203	10	90
P-44	915	10	90
P-46	1351	10	90
P-48	335	10	70
P-50	311	10	70
P-54	431	10	70
P-56	333	10	70
P-58	88	10	70
P-60	246	10	70
P-64	225	10	90
P-66	883	10	90
P-68	483	6	45
P-70	564	8	50
P-72	258	8	50
P-74	232	8	50
P-76	466	8	50
P-78	492	8	110
P-80	606	8	110
P-82	577	8	110
P-84	485	8	110
P-86	560	8	110
P-88	504	8	110
P-90	594	6	95
P-92	1092	6	70
P-94	439	6	100
P-96	651	8	110
P-98	774	8	110
P-100	271	8	110
P-102	627	8	110
P-104	1350	8	110
P-108	418	10	110
P-112	951	8	110
P-114	872	8	110

Hydraulic Input Pipe Data
West Newbury, Massachusetts

Label	Length (ft)	Diameter (in)	Hazen- Williams C
P-116	1489	6	50
P-118	1150	2	50
P-120	794	6	60
P-122	576	6	60
P-124	218	6	60
P-126	857	8	60
P-128	179	6	70
P-130	819	8	100
P-132	719	8	100
P-134	1332	8	100
P-136	313	8	100
P-138	1217	8	100
P-140	1087	6	90
P-142	656	6	55
P-144	782	6	55
P-146	984	6	55
P-148	363	6	55
P-150	224	2	50
P-152	331	2	50
P-154	757	6	55
P-156	732	8	110
P-158	1739	6	60
P-160	1890	6	60
P-162	504	6	90
P-164	1306	8	110
P-166	1634	8	110
P-168	1787	8	110
P-170	1270	8	110
P-172	632	6	100
P-174	1143	8	110
P-176	1257	8	110
P-178	574	8	110
P-180	652	8	110
P-182	862	8	100
P-184	813	8	100
P-186	1969	8	85
P-188	1788	10	95
P-190	475	10	95
P-192	471	10	95
P-194	388	6	100
P-196	264	6	100
P-198	616	6	100
P-200	403	6	100
P-202	624	6	100
P-204	263	6	100
P-206	1279	8	70
P-208	1850	8	70
P-210	1245	6	90
P-212	591	6	90
P-214	467	6	55

Hydraulic Input Pipe Data
West Newbury, Massachusetts

Label	Length (ft)	Diameter (in)	Hazen- Williams C
P-216	462	6	55
P-218	922	8	110
P-220	305	8	110
P-222	293	6	55
P-224	849	6	55
P-226	1044	6	55
P-228	712	10	60
P-230	557	6	110
P-232	1212	8	110
P-234	1212	8	110
P-236	377	8	110
P-238	755	8	110
P-240	169	8	110
P-242	150	8	110
P-244	202	8	110
P-246	471	8	110
P-248	377	8	110
P-250	435	8	110
P-252	203	8	110
P-254	269	8	110
P-256	393	8	110
P-258	787	8	110
P-260	711	8	110
P-262	1190	8	110
P-264	978	8	110
P-266	311	8	110
P-268	854	8	110
P-270	437	8	110
P-272	453	8	110
P-274	1714	8	110
P-276	1178	6	100
P-278	1586	8	110
P-280	448	8	90
P-282	663	8	90
P-284	372	8	90
P-286	684	8	90
P-288	286	8	90
P-290	603	8	50
P-292	402	8	50
P-294	144	8	50
P-296	493	8	50
P-298	760	6	100
P-300	816	8	100
P-302	966	8	100
P-304	294	8	100
P-306	914	8	100
P-308	736	8	100
P-310	993	8	100
P-312	979	8	100
P-314	1286	8	100

Hydraulic Input Pipe Data
West Newbury, Massachusetts

Label	Length (ft)	Diameter (in)	Hazen- Williams C
P-316	878	2	50
P-318	1431	6	90
P-320	506	8	100
P-322	615	8	100
P-324	106	8	110
P-326	604	8	110
P-328	414	8	110
P-330	516	8	110
P-332	1041	8	110
P-334	856	8	110
P-336	487	6	100
P-338	626	6	100
P-340	2508	8	110
P-342	1231	8	110
P-344	679	8	100
P-346	225	8	100
P-348	493	8	100
P-350	1113	10	110
P-352	322	8	110
P-354	573	8	110
P-358	156	8	110
P-360	481	8	110
P-362	266	8	110
P-364	218	8	110
P-366	284	10	120
P-368	288	10	120
P-370	541	10	120
P-372	741	10	90
P-374	771	10	90
P-376	376	10	70
P-378	365	10	70

Hydraulic Input Node Data
West Newbury, Massachusetts

Label	Elevation (ft)	Zone	Type	Demand (gpm)
J-2	10	Zone 2	Inflow	390
J-4	13	Zone 2	Demand	2.62
J-8	49	Zone 2	Demand	2.62
J-10	59	Zone 2	Demand	2.62
J-12	56	Zone 2	Demand	2.62
J-14	57	Zone 2	Demand	2.62
J-16	75	Zone 2	Demand	2.62
J-18	75	Zone 2	Demand	2.62
J-20	157	Zone 2	Demand	2.62
J-22	148	Zone 2	Demand	325
J-24	82	Zone-1	Demand	2.62
J-26	75	Zone-1	Demand	2.62
J-28	89	Zone-1	Demand	2.62
J-30	121	Zone-1	Demand	2.62
J-32	121	Zone-1	Demand	2.62
J-34	121	Zone-1	Demand	2.62
J-36	113	Zone-1	Demand	2.62
J-38	115	Zone-1	Demand	2.62
J-40	131	Zone-1	Demand	2.62
J-42	131	Zone-1	Demand	2.62
J-44	125	Zone-1	Demand	2.62
J-46	110	Zone-1	Demand	2.62
J-48	118	Zone-1	Demand	2.62
J-50	112	Zone-1	Demand	2.62
J-52	118	Zone-1	Demand	2.62
J-54	115	Zone-1	Demand	2.62
J-56	105	Zone-1	Demand	2.62
J-58	108	Zone-1	Demand	2.62
J-60	108	Zone-1	Demand	2.62
J-62	95	Zone-1	Demand	2.62
J-64	102	Zone-1	Demand	2.62
J-66	118	Zone-1	Demand	2.62
J-68	89	Zone-1	Demand	2.62
J-70	66	Zone-1	Demand	2.62
J-72	46	Zone-1	Demand	2.62
J-74	43	Zone-1	Demand	2.62
J-76	39	Zone-1	Demand	2.62
J-78	30	Zone-1	Demand	2.62
J-80	59	Zone 2	Demand	2.62
J-82	60	Zone 2	Demand	2.62

Hydraulic Input Node Data
West Newbury, Massachusetts

Label	Elevation (ft)	Zone	Type	Demand (gpm)
J-84	62	Zone 2	Demand	2.62
J-86	69	Zone 2	Demand	2.62
J-88	62	Zone 2	Demand	2.62
J-90	72	Zone 2	Demand	2.62
J-92	79	Zone 2	Demand	2.62
J-94	89	Zone 2	Demand	2.62
J-96	66	Zone 2	Demand	2.62
J-98	66	Zone 2	Demand	2.62
J-100	95	Zone 2	Demand	2.62
J-104	39	Zone-1	Demand	2.62
J-106	79	Zone-1	Demand	2.62
J-108	95	Zone-1	Demand	2.62
J-110	138	Zone-1	Demand	2.62
J-112	108	Zone-1	Demand	2.62
J-114	134	Zone-1	Demand	2.62
J-116	108	Zone-1	Demand	2.62
J-118	98	Zone-1	Demand	2.62
J-120	148	Zone-1	Demand	2.62
J-122	85	Zone-1	Demand	2.62
J-124	69	Zone-1	Demand	2.62
J-126	13	Zone-1	Demand	2.62
J-128	7	Zone-1	Demand	2.62
J-130	35	Zone-1	Demand	2.62
J-132	20	Zone-1	Demand	2.62
J-134	39	Zone-1	Demand	2.62
J-136	85	Zone-1	Demand	2.62
J-138	128	Zone-1	Demand	2.62
J-140	128	Zone-1	Demand	2.62
J-142	23	Zone-1	Demand	2.62
J-144	23	Zone-1	Demand	2.62
J-146	72	Zone-1	Demand	2.62
J-148	108	Zone-1	Demand	2.62
J-150	138	Zone-1	Demand	2.62
J-152	138	Zone-1	Demand	2.62
J-154	89	Zone-1	Demand	2.62
J-156	131	Zone-1	Demand	2.62
J-158	197	Zone-1	Demand	2.62
J-160	128	Zone-1	Demand	2.62
J-162	121	Zone-1	Demand	2.62
J-164	108	Zone-1	Demand	2.62

Hydraulic Input Node Data
West Newbury, Massachusetts

Label	Elevation (ft)	Zone	Type	Demand (gpm)
J-166	108	Zone-1	Demand	2.62
J-168	82	Zone-1	Demand	2.62
J-170	128	Zone-1	Demand	2.62
J-172	128	Zone-1	Demand	2.62
J-174	128	Zone-1	Demand	2.62
J-176	128	Zone-1	Demand	2.62
J-178	128	Zone-1	Demand	2.62
J-180	128	Zone-1	Demand	2.62
J-182	108	Zone-1	Demand	2.62
J-184	43	Zone-1	Demand	2.62
J-186	30	Zone-1	Demand	2.62
J-188	39	Zone-1	Demand	2.62
J-190	43	Zone-1	Demand	2.62
J-192	102	Zone-1	Demand	2.62
J-194	102	Zone-1	Demand	2.62
J-196	95	Zone-1	Demand	2.62
J-198	108	Zone-1	Demand	2.62
J-200	98	Zone-1	Demand	2.62
J-202	89	Zone-1	Demand	2.62
J-204	59	Zone-1	Demand	2.62
J-206	26	Zone-1	Demand	2.62
J-208	26	Zone-1	Demand	2.62
J-210	43	Zone-1	Demand	2.62
J-212	43	Zone-1	Demand	2.62
J-214	43	Zone-1	Demand	2.62
J-216	39	Zone-1	Demand	2.62
J-218	59	Zone-1	Demand	2.62
J-220	43	Zone-1	Demand	2.62
J-222	43	Zone-1	Demand	2.62
J-224	30	Zone-1	Demand	2.62
J-226	30	Zone-1	Demand	2.62
J-228	43	Zone-1	Demand	2.62
J-230	41	Zone-1	Demand	2.62
J-232	30	Zone-1	Demand	2.62
J-234	49	Zone-1	Demand	2.62
J-236	30	Zone-1	Demand	2.62
J-238	26	Zone-1	Demand	2.62
J-240	23	Zone-1	Demand	2.62
J-242	39	Zone-1	Demand	2.62
J-244	66	Zone-1	Demand	2.62

Hydraulic Input Node Data
West Newbury, Massachusetts

Label	Elevation (ft)	Zone	Type	Demand (gpm)
J-246	184	Zone-1	Demand	2.62
J-248	49	Zone-1	Demand	2.62
J-250	39	Zone-1	Demand	2.62
J-252	66	Zone-1	Demand	2.62
J-254	30	Zone-1	Demand	2.62
J-256	30	Zone-1	Demand	2.62
J-258	30	Zone-1	Demand	2.62
J-260	30	Zone-1	Demand	2.62
J-262	98	Zone-1	Demand	2.62
J-264	92	Zone-1	Demand	2.62
J-266	89	Zone-1	Demand	2.62
J-268	92	Zone-1	Demand	2.62
J-270	82	Zone-1	Demand	2.62
J-272	121	Zone-1	Demand	2.62
J-274	161	Zone-1	Demand	2.62
J-276	197	Zone-1	Demand	2.62
J-278	112	Zone-1	Demand	2.62
J-280	102	Zone-1	Demand	2.62
J-282	88	Zone-1	Demand	2.62
J-284	98	Zone-1	Demand	2.62
J-286	108	Zone-1	Demand	2.62
J-288	148	Zone-1	Demand	2.62
J-290	157	Zone-1	Demand	2.62
J-292	98	Zone-1	Demand	2.62
J-294	118	Zone-1	Demand	2.62
J-296	16	Zone-1	Demand	2.62
J-298	79	Zone-1	Demand	2.62
J-300	71	Zone-1	Demand	2.62
J-302	82	Zone-1	Demand	2.62
J-304	157	Zone-1	Inflow	0
J-306	26	Zone-1	Demand	2.62
J-308	33	Zone-1	Demand	2.62
J-310	90	Zone-1	Demand	2.62
J-312	98	Zone-1	Demand	2.62
J-314	39	Zone-1	Demand	0
Wellfield	20	Zone 2	Demand	0

Appendix D

Phased Improvements Map

Appendix E

Capital Plan Update

TATA & HOWARD

INCORPORATED

June 27, 2008

Mr. Michael Gootee, Superintendent
West Newbury Water Department
381 Main Street
West Newbury, MA 01985

Subject: Capital Plan Update
T&H Project No. 1980

Dear Mike:

In accordance with our contract we have completed the Capital Plan Update relative to improvements completed since the completion of the 2003 Water Distribution System Study (WDS) Report completed by Tata & Howard, Inc. As part of this project, we reviewed the existing WDS report and the completion of recommended improvements from that study. We reevaluated the prioritized improvements and have recommended new improvements in a prioritized and phased manner similar to that presented in the original WDS report. Estimates of probable construction costs have also been updated to reflect recent construction trends and include 25% for engineering and contingencies.

Improvement Prioritization

Improvements are typically recommended based on their priority. Priority I projects generally consists of water supply, storage and higher ISO fire flow requirements. The Town has proactively completed or initiated several of the Priority I - Phase 1 and 2 projects recommended in the 2003 WDS report, specifically those relating to water supply and storage. Priority II improvements include the smaller ISO fire flow requirements while Priority III improvements can be considered as part of the Town's general maintenance of the distribution system.

Improvements Completed Since 2003

Since the completion of the 2003 WDS report, the Town has completed several priority I, phase 1 improvements relative to water supply. Currently the Town of West Newbury is supplied water from their wellfield and water purchased from the City of Newburyport. Since the Town has only one supply, recommendations were made to maximize their existing supply and conduct test well work to develop one or more new supplies. The existing wellfield has been cleaned to improve capacity and the pumps that furnish water from the wellfield and from Newburyport were replaced with new pumps. The former pumps had incompatible heads that did not allow the Town to pump the wellfield concurrently with the Newburyport interconnection. The new pumps have compatible

MAIN OFFICE
67 Forest Street, Marlborough, MA 01752
508-303-9400 Fax: 508-303-9500

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heads which allows the Town to maximize the yield from this source and minimize the volume of water purchased from Newburyport. New variable frequency drives (VFD's) were also installed at this site which have aided in the optimization of the station's operation thereby improving savings in electrical costs.

In addition to the wellfield cleaning and new pumps, a new gravel packed well was constructed and tied into the existing wellfield. The new well was constructed to maximize the safe yield (0.2 million gallons per day) of this site and to replace the horizontal well. The horizontal well was constructed to a depth of approximately 17 feet. At times when the regional groundwater levels are near or exceed 17 feet this well could not be used. Generally, this occurs during the higher demand periods of the summer and fall when water use was at its highest.

Another priority I, phase 1 improvement recommended was the potential development of the Andreas and Dunn well sites. Both of these sites consisted of bedrock wells located remotely from the existing water distribution infrastructure. An extended pump test was completed at the Andreas well site and its estimated yield was approximately 100 gpm. Unfortunately, the costs associated with developing and treating this site eliminated developing at this time.

Preliminary testing was completed at the Dunn site and its estimated yield was approximately 130 gpm. Further testing was not completed due to associated costs for development and treatment.

Another potential well site recommended for investigation was located adjacent to the Artichoke River. However, the property owner is not interested in the sale of this property at this time. A fourth site in the high service area near at the High School is currently under consideration. The Town is currently negotiating permission to conduct test well work on this property.

A new generator was installed at the booster pump station that feeds the high service area. This generator is able to provide average day demands during emergencies, such as a power outage and reduces the total volume of storage needed for the high service area. Recently the Town completed renovations and painting to the Pipe Stave Tank. The improvements include a new exterior tank ladder and replacement of some of the exterior gunnite, where the existing gunnite is failing.

As a condition for approval of the new Ocean Meadow subdivision, approximately 1,600 linear feet of new 12-inch diameter water main have been constructed in the low service area along Main Street from the existing valve pit next to the Pipestave Tank to Chase Street. This water main had already been identified as deficient in meeting necessary fire flows in the Town's 2003 Water Distribution Study. The new water main replaces an older, undersized 8-inch main. The Ocean Meadow developer was required to make the improvement because the hydraulic computer model created as part of the 2003 Study demonstrated that the new subdivision would cause unacceptable pressure drops in parts of the existing system. Consequently, this improvement was made a requirement for approval by the Planning Board, at the behest of Water Department.

Recommended Capital Plan Update

Although several improvements have been completed since the 2003 WDS report, additional improvements are still needed relative to supply, storage and distribution. Phase 1 improvements have been revised to include supply and storage which are typically considered Priority I improvements. Phases 2 and 3 improvements are associated with infrastructure improvements needed in the high and low service areas to mitigate higher ISO fire flow requirements, which are typically also considered Priority I improvements. Phase 4 and 5 improvements are associated with deficiencies at extremities and general fire flow requirements, which are considered Priority II improvements. The improvements listed in Phases 2 through 5 have not changed from the 2003 WDS report, however, costs have been adjusted accordingly. A listing of the recommended improvements is summarized below and their costs are present in Table No. 1:

1. Water Supply and Water Storage
2. High Service Area Transmission Mains
3. Low Service Area Transmission Mains
4. Secondary Transmission Mains and ISO Fire Flow Requirements
5. Restrictions

Phase 1 – Water Supply and Storage

1. As stated in the 2003 WDS report, the Town of West Newbury has inadequate supply to meet existing and future demands. One or more new water supply sources are needed to mitigate this deficiency. The Town has proactively identified numerous sites that have potential as water supply sources. The sites include the Pentucket School, Crane Neck Street, 902 Main Street, and the Mullen property. Additionally, the Town plans to construct new gravel packed wells at the existing wellfield site and to seek a bedrock well adjacent to that site. The costs associated with the preliminary testing (test well work), permitting and construction of these various sites are included herein. The cost associated with construction does not include water treatment.

The Town is currently negotiating permission to conduct test well work at the Pentucket High School located in the High Service Area. We recommend the continuation of this negotiation for the testing and, if viable, potential development of this site. The existing High School property is ideally located in the high service area where the majority of system demand occurs. It is also located near the existing distribution system which would minimize costs of interconnection. We recommend installing test wells to determine the viability of a potential source.

Additional testing sites have been identified in the Crane Neck Street area. This area is in a sub-basin of the Parker River which is a stressed basin and would require extensive permitting. The upper area additionally has a small drainage basin that would limit the yield of a potential well. If property becomes available in the lower area it may be worthwhile to install test wells to determine the viability of a potential source.

The property at 902 Main Street is currently available. This site abuts the Merrimack River and has potential for water supply development. The cost of the property may be prohibitive, but we recommend installing test wells to determine the viability of a potential source.

A large, mixed-use housing development has been proposed for the Town-owned Mullen property located on Main Street, and development of a well on this property would conflict with the plans for subsurface disposal of the sewage. However, this site abuts the Merrimack River and has marked potential for water supply development. Since the available areas for new source development in Town are limited, we recommend installing test wells to determine the viability of this potential source. If the site proves to be sufficiently viable, the Water Department can make the case that a well presents the highest and best use of the property for the Town.

There are two options at the existing wellfield that could provide additional water supply. The first option is installation of two additional gravel packed wells to supplement the single well recently installed so as to replace the tubular wells in the wellfield. The wellfield yield is limited by the loss in the 2-1/2 inch diameter tubular wells and the suction lift capabilities of the pumps. Installation of larger diameter wells would reduce the suction lift and maximize the yield of the site. We recommend installing two gravel packed wells. If necessary, submersible pumps can be added to the three wells at a later date.

The second option for the existing wellfield site is installation of a bedrock well at the site. A new well on this site would be far less costly to develop than the Andreas or Dunn sites because of its proximity to electrical power and the existing distribution system. A fracture trace was performed and a location for the well was selected. A new bedrock well would require meeting the New Source Approval process.

As a general rule, we recommend that any property that abuts the Merrimack River be given consideration as a potential source if a Zone I (250 or 400 foot protective radius for a wellfield or gravel packed/bedrock well, respectively) can be obtained.

The estimated cost for water supply development in the surficial aquifer is broken down into the several components listed below:

Preliminary Test Well Investigations (various sites) - $ 50,000

Note that the $50,000 provide for multiple preliminary tests that typically vary from $10,000 to $15,000 per site.

If a viable site is located during the test well investigations the costs for development are listed below:

Pump Test & Zone II Delineation (any one site)	- $110,000
Well Pump Station (No Treatment) (any one site)	- $800,000

We also recommend the development of a bedrock well adjacent to the existing wellfield to maximize yield from this site. Since the new well would not be part of the wellfield it would require development under the new source approval process. The costs associated with the construction and permitting of this site are included herein.

Pump Test & Zone II Delineation	- $130,000
Well Pump Station (No Treatment)	- $600,000

Finally, we recommend supplementing the existing gravel packed well by installing two additional gravel packed wells at the existing wellfield. The three wells would be connected to the existing vacuum suction line. The Zone II for the wellfield would not change.

Install two Gravel Packed Wells / Existing Wellfield - $ 60,000

2. As recommended in the original WDS, additional water storage is needed in the high service area. The Town is current investigating the area required for a second water storage tank. We recommend the construction of a new steel water storage facility with a usable capacity of at least 0.39 million gallons (MG) at the existing Brake Hill Tank site. It is not uncommon for water storage tanks to have a service life in excess of 120 years, as such we recommend that the Town maintain the existing Brake Hill Tank, through routine maintenance, so it can also be used for additional storage.

   The estimated probable construction cost of this improvement is $1,000,000.

3. Currently, the transmission main from the existing Brake Hill Tank to the existing distribution system (Main Street) is a 10-inch diameter water main. In order to reduce the head loss through the existing main and to increase the inherent carrying capacity of this main, it is recommended that a new 16-inch diameter water main from the Brake Hill Tank to the intersection of Main Street be constructed. In addition, a second 16-inch diameter main is recommended from this tank site to Hilltop Circle. These improvements can be completed once the new water storage tank is constructed, which will allow for the Brake Hill Tank being taken out of service while the new 16-inch diameter water main is being constructed. These improvements will assist in mitigating estimated needed ISO fire flows on Main Street and improve fire flow capacity along Hilltop Circle and Crane Neck Street.

   The estimated probable construction cost of this improvement is $ 438,000.

Phase 2 - High Service Area Transmission Mains

4. In an effort to improve the east to west transmission grid in the high service system, a new 16-inch diameter main on Main Street from the new 16-inch main (Improvement No. 3) to Bachelor Street is recommended. This improvement will help to assist in improving several ISO fire flow deficiencies in the distribution system. In addition, this improvement will replace old, undersized water mains in this area and benefit the entire high service area by strengthening the main transmission grid of this service area.

   The estimated probable construction cost of this improvement is $ 1,028,000.

5. In an effort to further strengthen the transmission grid in the high service area system, cleaning and lining the existing 8-inch water main on Main Street from

the Groveland town boundary to the existing 6-inch diameter water main located at the intersection of Farm Lane and Main Street is recommended. Prior to cleaning and lining the existing 8-inch diameter water main on Main Street, it is recommended that the Town take coupons (small side samples) of the water main to determine the condition of the main. Additionally, it is recommended to construct a new 8-inch diameter water main on Main Street to replace the portion of 6-inch diameter water main that creates a restriction in the distribution system. From the west end of town, the 6-inch water main begins at the intersection of Farm Lane and Main Street and ends at 117 Main Street where it intersects the distribution line that feeds the Brake Hill Standpipe. This improvement, along with the above recommendations, will significantly improve transmission in the high service system while improving available fire flows in the area. This improvement will also provide the inherent capacity to meet future maximum day demands through water purchased from the Town of Groveland.

The estimated probable construction cost for this improvement is $314,000.

6. An ISO estimated needed fire flow of 1,750 gpm is required on Main Street west of Training Field Road. Although the above recommended transmission reinforcements (Improvement No. 5) improve flow to this area, additional improvements are needed to meet the estimated ISO fire flow requirement. Additional improvements recommended include a new 12-inch diameter water main on Main Street from the intersection of Bachelor Street to Training Field Road. This improvement, along with the proposed 16-inch diameter water main on Main Street, will assist in meeting the ISO fire flow requirement and further strengthen the west to east transmission grid. In addition, the new water mains will replace older, undersized tuberculated mains in the central portion of the high service area grid.

   The estimate of probable construction cost for this improvement is $299,000.

Phase 3 – Low Service Area Transmission Mains

7. Several ISO fire flows ranging from 500 to 1,750 gpm are needed along Main Street east of Chase Street and west of Garden Street. As part of the Open Space Preservation Development approximately 1,600 linear feet of 12-inch diameter water main has recently been constructed. This improvement assists in mitigating some of the smaller fire flow requirements and improves fire protection in this area. However, additional improvements are needed to meet higher fire flow requirements and to help strengthen the overall transmission grid in this area.

Therefore, we recommend the construction of a new 12-inch diameter water main on Main Street from the Chase Street to Garden Street.

The estimate of probable construction cost for this improvement is $ 497,000.

<u>Phase 4 – Secondary Transmission Mains and ISO Fire Flow Requirements</u>

8. An estimated needed ISO fire flow of 2,250 gpm is required at the intersection of Main Street and Brake Hill Terrace. In addition to the transmission main improvements recommended in Phase 3, Improvement No. 5 and 6, the following infrastructure improvements are recommended:

- Replace existing 6-inch water main on Crane Neck Street, which runs from the intersection with Main Street to the existing 8-inch diameter water main further down Crane Neck Street, with a new 8-inch water main,
- clean and line the existing 8-inch water main on Crane Neck Street from the existing 6-inch diameter water main to the intersection of Hilltop Circle,

These improvements will assist in providing the inherent capacity to meet the 2250 gpm ISO fire flow requirement, as well as two additional inadequate fire flows along Meadow Street and Hilltop Circle. In addition, the construction of the new 8-inch diameter water mains on Main Street and Crane Neck Street will eliminate two bottlenecks in the system by replacing portions of inadequately sized 6-inch diameter water main.

The estimate of probable construction cost for this improvement is $ 292,000.

9. An ISO estimated needed fire flow of 2,500 gpm is required at the intersection of Main Street and Maple Street. In addition to the previously mentioned Phase 2 and Phase 4, improvements, it is recommended that a new 8-inch diameter water main on Tewksbury Lane from Crane Neck Street to Meetinghouse Hill Road be constructed. Additionally, this improvement will eliminate the bottleneck created by the existing 6-inch diameter water main on Tewksbury Lane. Eliminating this restriction will reduce head loss in this portion of the system and improve transmission and flows.

This improvement will assist in providing the inherent capacity to meet the estimated ISO fire flow requirement, as well as smaller estimated fire flow requirements in this portion of the system, while improving west to east transmission in the high service area.

The estimate of probable construction cost for this improvement is $180,000.

10. An ISO estimated needed fire flow of 750 gpm at the intersection of Crane Neck Street and Middle Street is required. In addition to the new 16-inch diameter water main on Main Street recommended in Phase 2, it is recommended that the existing 6-inch diameter water main on Crane Neck Street be replaced with a new 8-inch diameter water main. This improvement, along with the transmission improvements on Main Street, will assist in mitigating the ISO fire flow requirement, while improving transmission to this extremity of the high service area.

    The estimate of probable construction cost for this improvement is $ 500,000.

Phase 5 – Restrictions and Estimated Fire Flow Requirements

11. An estimated needed fire flow of 750 gpm is required at the end of Cortland Lane. Although the previously recommended improvements on Main Street improve transmission in this portion of the system, it is recommended that a new 12-inch diameter main be constructed on Main Street from the intersection of Bailey's Lane to the intersection of Coffin Street. This improvement will assist in meeting the estimated fire flow requirement. In addition, this improvement will replace a long length of older, tuberculated 8-inch diameter water main.

    The estimate of probable construction cost of this improvement is $ 401,000.

12. An estimated needed fire flow of 750 gpm is required at the end of Farm Lane. Currently, Farm Lane is serviced by a 6-inch diameter water main. It is recommended that a new 8-inch diameter water main be constructed on Farm Lane from the intersection of Main Street to the end of this street. This improvement will increasing the inherent carrying capacity, thereby assist in mitigating the estimated needed fire flow.

    The estimated probable construction cost of this improvement is $ 112,000.

13. There are portions of the distribution system where smaller water mains are the sole means of transporting water between larger mains. Specifically, there are portions of 6-inch diameter water mains on Harrison Avenue that create a bottleneck in the system. In an effort to eliminate these restrictions, it is recommended that a new 8-inch diameter water main be constructed on Harrison Avenue from the intersection of Main Street to the existing 8-inch diameter water

main on Harrison Avenue. Eliminating this restriction will increase the inherent carrying capacity of this main.

The estimate of probable construction costs for this improvement is $70,000.

14. Currently, Appleton Court located off Main Street and Middle Street located off Crane Neck Street are serviced by 2-inch diameter service lines that do not have the inherent carrying capacity to meet the estimated fire flows of 750 gpm. It is recommended that the service lines on Appleton Court and Middle Street be replaced with 6-inch diameter mains. New 6-inch diameter mains will increase the capacity along these roads and eliminate older, smaller diameter mains in the system. The following improvements and associated estimate of probable construction costs are recommended:

A	Appleton Court	$ 56,000
B	Middle Street	$ 88,000

15. Currently, Bailey's Lane is serviced by a 6-inch diameter water main and a 2-inch diameter service line. It is recommended that an 8-inch diameter water main on Bailey's Lane from the intersection with Main Street to its end be constructed. This improvement will provide the inherent carrying capacity to meet the estimated fire flow requirement of 750 gpm.

The estimated probable construction cost of this improvement is approximately $ 330,000.

16. Currently, Garden Street is serviced by a heavily tuberculated 6-inch diameter water main. It is recommended that a new 8-inch diameter water main be constructed on Garden Street from the intersection of Main Street to the end of the existing water main. A new 8-inch diameter water main is recommended rather than rehabilitate the existing 6-inch diameter water main due to the longer length of the water main. In addition to increasing the carrying capacity, this improvement will assist in mitigating the estimated needed fire flow requirement of 750 gpm.

The estimated probable construction cost of this improvement is approximately $192,000.

17. In various portions of the system, heavily tuberculated 6-inch diameter water mains are used to service dead end streets. It is recommended that these mains be cleaned and lined. Specifically, the mains on Albion Lane, Merrill Street, and Sullivan Court do not have the transmission capacity to meet the estimated fire flows of 750 gpm. Rehabilitating the existing 6-inch diameter mains will increase the inherent carrying capacity of these mains and assist in mitigating the estimated fire flow requirement. The following improvements and associated estimate of probable construction costs are recommended:

A	Albion Lane	$ 160,000
B	Merrill Street	$ 95,000
C	Sullivan Court	$ 84,000

General Recommendations

Groveland Interconnection

In the 2003 WDS report we recommended the Town pursue an agreement with the Town of Groveland, which is still recommended. Prior negotiations with Groveland resulted in a draft water contract, however, in 2004 supply concerns in Groveland temporarily suspended these negotiations. An emergency water supply agreement with Groveland would be beneficial during the investigation of a new water supply source and would provide added security during emergencies. If both parties agree to pursue the interconnection, the Towns should enter into a formal agreement, which will outline the unit costs for water and any conditions that may apply.

Distribution System Expansion

Whenever improvements or expansion of a water distribution system occur, factors such as size and location of the main should be considered in order to provide adequate flows and pressures. Water mains, which extend for 600 feet or more without an interconnection, should have a minimum diameter of 8 inches. If a section of proposed main is less than 600 feet, the minimum water main diameter could be reduced to 6 inches if the main is used to complete a water main loop and the fire flow requirement is minimal. Wherever possible, dead end mains should be eliminated by looping or interconnecting. Parallel water mains should be interconnected at reasonable intervals. All older and smaller water mains that are not able to meet fire flow requirements in an area should be replaced with larger diameter mains. In addition, "bottlenecks" such as a smaller water main being the sole means of transporting water between larger mains should be eliminated. Currently, no specific looping projects are recommended.

However, as the Town extends their distribution system looping of dead ends should be considered in long-term planning schedules.

Water Audit and Leak Detection

In accordance with the Town's Water Management Act Permit, system wide biannual leak detection is required. The testing is conducted biannually to remain compliant with the WMA permit. We recommend that the Town continue with this biannual testing to minimize unaccounted-for water loss and to remain compliant with their WMA permit.

General Maintenance

The Town should continue with its regularly scheduled maintenance program which includes hydrant flushing twice a year, routine inspection and maintenance at the pump stations, and meter testing. The general maintenance program should include monitoring of the capacity of the wellfield so that well cleaning and redevelopment, when required, can be scheduled during low demand periods.

The Town is proactively implementing a meter installation and replacement program. Currently Badger Orion meters with radio-head capabilities are being installed. AWWA recommends meter replacement every seven to ten years, depending on the water quality of the system and use of the meter.

The Town conducts an annual water distribution system replacement program, during which they identify valves and hydrants which do not function as intended. By eliminating components that are defective, proper system operation is facilitated. In addition, by replacing hydrants that are old or broken, fire protection capabilities in the system will be improved. The Town should continue its efforts on this program.

The Town should also work with the Merrimack Valley Planning Commission in incorporating tabular data associated with its meter population into the existing GIS database showing their location. It is also recommended that a new GIS layer be created for use in storing locations and tabular information for the different distribution system components such as valves, hydrants, pumps, etc.

In addition, the West Newbury Water Department should consider approaching the Town Government Officials regarding the implementation of a general tax levy to aid in the payment of distribution system improvements that impact large numbers of townspeople.

We appreciate this opportunity to work with you on this important project. The recent updates discussed in this report, such as the new pump curves and 1,600 linear feet of 12-inch diameter water main, have been incorporated into the existing hydraulic model.

Should you have any questions or require additional information, please do not hesitate to contact us.

Sincerely

TATA AND HOWARD, INC.

Paul B. Howard

Paul B. Howard, P.E.
Senior Vice President

Table No. 1
Phase 1 - 5 Improvements
West Newbury, Massachusetts

Phase	Improvement	Description/Location	From	To	Proposed Dia.	Length	Estimated Cost*
1	1	Test Well Investigation - HS Site					$25,000
		Pump Test & Zone II Delineation					$110,000
		Pump Station (No Treatment)					$750,000
	Subtotal:						$885,000
	2	Bedrock Well Testing					$125,000
		Bedrock Well Development					$100,000
	Subtotal:						$225,000
	3	Water Storage Tank					$1,000,000
	4	Brake Hill Tank Main	Tank	Main Street	16	1120	$168,000
			Tank	Hilltop Circle	16	1800	$270,000
	Subtotal:						$438,000
Total:							$2,548,000
2	5	Main Street	New 12"	Bachelor Street	16	5870	$1,028,000
	6	Main Street	Break Hill Tank Road	Existing 8" Main	8	500	$63,000
		Main Street	Existing 8" Main	Groveland Townline	C&L (8")	2280	$251,000
	Subtotal:						$314,000
	7	Main Street	Bachelor Street	Training Field Road	12	1990	$299,000
Total:							$1,641,000
3	8	Pipe Stave Tank Main	Tank	Main Street	12	420	$63,000
		Main Street	Pipe Stave Tank	Garden Street	12	2890	$434,000
Total:							$497,000
4	9	Crane Neck Street	Main Street	Existing 8" Main	8	760	$95,000
		Crane Neck Street	Existing 6" Main	Hilltop Circle	C&L (8")	1790	$197,000
	Subtotal:						$292,000
	10	Tewksbury Lane	Crane Neck Street	Meetinghouse Hill Road	8	1440	$180,000
	11	Crane Neck Street	Existing 6-inch Main	End	8	4000	$500,000
Total:							$972,000