Final Report

Newmarket, NH

Preliminary Design Report
MacIntosh Well Treatment
Alternatives Analysis

September 2010

Weston&Sampson

Weston & Sampson Engineers, Inc.
100 International Drive, Suite 152
Portsmouth, NH 03801
www.westonandsampson.com
Tel: 603-431-3937 Fax: 603-433-4358

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF APPENDICES

1.0 INTRODUCTION

1.1 General

The Town of Newmarket is located approximately 15 miles west of the City of Portsmouth, along the banks of the Lamprey River and Great Bay. Newmarket has a population of 8,027 according to the 2000 Federal population census and a population of 9,436 as estimated by the U.S. Census Bureau for 2008.

The Town of Newmarket groundwater withdrawal points are in the Piscassic River Watershed and the Lamprey Watershed. The town is currently registered through the Department of Environmental Services (DES) to withdraw water from two groundwater wells and one surface water withdrawal point that is currently not in service.

The Town of Newmarket's water distribution system includes one service area without any sub-systems of varying pressure. Groundwater is pumped into the system through two well pumping stations and is distributed through a network of water mains approximately 24.1 miles long. The entire water distribution system serves approximately 1,922 accounts.

1.2 Objectives

The Town of Newmarket has been fortunate to be able to meet its municipal water supply needs with high quality groundwater from the Newmarket Plains stratified-drift aquifer utilizing the Sewell and Bennett groundwater wells. Due to recent population growth and expected future growth, the Town will need to augment its water supply using other sources to meet an increase in water demand. It is with this objective that the Town is pursuing permitting and construction of the MacIntosh bedrock well located between Hersey Lane and Ash Swamp Road.

This Preliminary Design Report provides an assessment of the project and details several options available to provide adequate pumping capacity and treatment for the proposed bedrock well.

2.0 EXISTING FACILITIES

2.1 Existing facilities and treatment

The Town of Newmarket relies upon two gravel packed, shallow groundwater production wells to supply the approximate average daily demand of 0.50 million gallons per day.

The Bennett Well is located on Wadleigh Falls Road and has a current dependable yield of 220 gallons per minute (gpm). The well receives chemical treatment via chlorine and sodium hydroxide injection. The chlorine residual leaving the pumphouse is between 0.2 and 0.7 mg/L and caustic is added to increase the pH to a range between 7.5 and 8.5. The Bennett Well consists of a 48-inch by 24-inch gravel packed well that is 48-feet deep with a 10-foot 100 slot stainless steel screen. The well was installed in 1974.

The Sewell Well is located on Wadleigh Falls Road and has a current dependable yield of 270 gallons per minute (gpm). The well receives chemical treatment via chlorine and sodium hydroxide injection. Similar to the Bennett Well, the Sewell Well chlorine residual leaving the pumphouse is between 0.2 and 0.7 mg/L and caustic is added to increase the pH to a range between 7.5 and 8.5. Sewell Well consists of a 24-inch by 18-inch gravel packed well that is 83-feet deep with a 10 foot screen. The well was installed in 1983.

Both the Bennett Well and Sewell Well can pump at a slightly higher rate, but lowering the groundwater levels generally results in an increase in undesirable minerals. Due to this, the wells are not run at an aggressive flowrate. Refer to the 2008 Preliminary Hydrogeologic Report by EGGI for further details.

2.2 Historical water quality

Historical water quality information was primarily used for determination of blending alternatives while evaluating the proposed bedrock well. The tables presented below represent data provided on NHDES' One-Stop website as well as information collected by the Town. Please note that a dash indicates the constituent was not a parameter of the test whereas "ND" indicates the constituent was a parameter of the test and not detected in the water.

Table 2.1 – Bennett Well Raw Water Quality

Date	Iron (mg/L) [MCL=0.3]	Manganese (mg/L) [MCL=0.05]	Arsenic (mg/L) [MCL=0.01]	Sodium (mg/L) [MCL=100]	Chloride mg/L) [MCL=250]	TDS (mg/L)	Alkalinity (mg/L)
10/28/2003	--	--	0.001	28.8	52	--	34.2
12/7/2004	--	--	--	29.7	56	--	38.1
11/8/2006	--	--	0.0011	36.9	73	--	43.2
10/13/2009	ND	ND	ND	35.3	54	--	--
6/23/2010	ND	ND	ND	26	52	150	29
7/21/2010	ND	ND	ND	26	51	150	29

In addition to the constituents listed above, the Bennett Well has been found to contain levels of radon at 1,700 pCi/L.

Table 2.2 – Sewell Well Raw Water Quality

Date	Iron (mg/L) [MCL=0.3]	Manganese (mg/L) [MCL=0.05]	Arsenic (mg/L) [MCL=0.01]	Sodium (mg/L) [MCL=100]	Chloride mg/L) [MCL=250]	TDS (mg/L)	Alkalinity (mg/L)
10/28/2003	--	--	0.002	45.2	77	--	42.8
12/7/2004	--	--	0.002	46.4	79	--	40.8
11/8/2006	--	--	0.0017	46.4	78	--	41.3
10/13/2009	ND	ND	0.0015	41.8	66	--	--
6/23/2010	ND	ND	ND	33	59	170	27
7/21/2010	ND	ND	ND	34	62	190	27

In addition to the constituents listed above, the Sewell Well has been found to contain levels of radon at 1,000 pCi/L.

3.0 PROPOSED BEDROCK PRODUCTION WELL

3.1 General

The proposed well has been identified as Well NGE-2B; the MacIntosh Well. The well is also being permitted with NHDES as the NGE-PW3 Production Well. For the purposes of this report, the well will be referred to as the MacIntosh Well. The well is located off Ash Swamp Road and Hersey Lane. The wellsite is situated in the Piscassic River watershed on land to be transferred via a conservation easement (see Figure 3-1).

3.2 Reported Water Quality and Recommendations

The MacIntosh Well water quality is presented in Table I as prepared by Emery & Garrett Groundwater, Inc. (EGGI) in the report titled: *Summary Letter, 56-Day Water Quality Assessment and Pumping Program of Newmarket Production Well No. 3 (NGE-2B)* dated May 24, 2010.

EGGI conducted a pumping test and evaluation of the bedrock well which revealed the presence of elevated levels of chloride, arsenic, and manganese. The levels of chloride and arsenic are of particular concern in identifying appropriate levels of treatment to reduce concentrations to below their respective Maximum Contaminant Levels (MCL).

The EGGI report indicated that the elevated chloride concentration levels are likely a reflection of a relatively stagnant recharge condition under non-pumping conditions and, with extended pumping, the chloride levels are expected to reduce. It is with this assumption that Weston & Sampson has prepared our evaluation and recommendations.

3.3 Well Safe Yield and Proposed Treatment Plant Capacity

EGGI's 56-day pump test results confirmed that the MacIntosh Well safe yield is 300 gallons per minute (gpm). It is at this yield that the well will be permitted with DES.

It should also be noted that the Town is beginning discussions regarding the development of Well NGE-1A (Sharon Tucker Well) which is located south of Ash Swamp Road and may, in the future, utilize the infrastructure described in this report to treat the Sharon Tucker Well. According to EGGI's pump test, the Sharon Tucker Well was found to have a safe yield of 275 gpm.

Preliminary design should consider the initial need to treat the MacIntosh Well (300 gpm) and have the necessary capacity and infrastructure to allow for the inclusion of the Sharon Tucker Well (275 gpm) in the future. Please refer to Weston & Sampson's memorandum titled "Sharon Tucker Well Water Quality, Blending Potential and Pipe Routing Options" dated August 6, 2010 for a more detailed account of the Sharon Tucker Well's water quality and blending suitability.

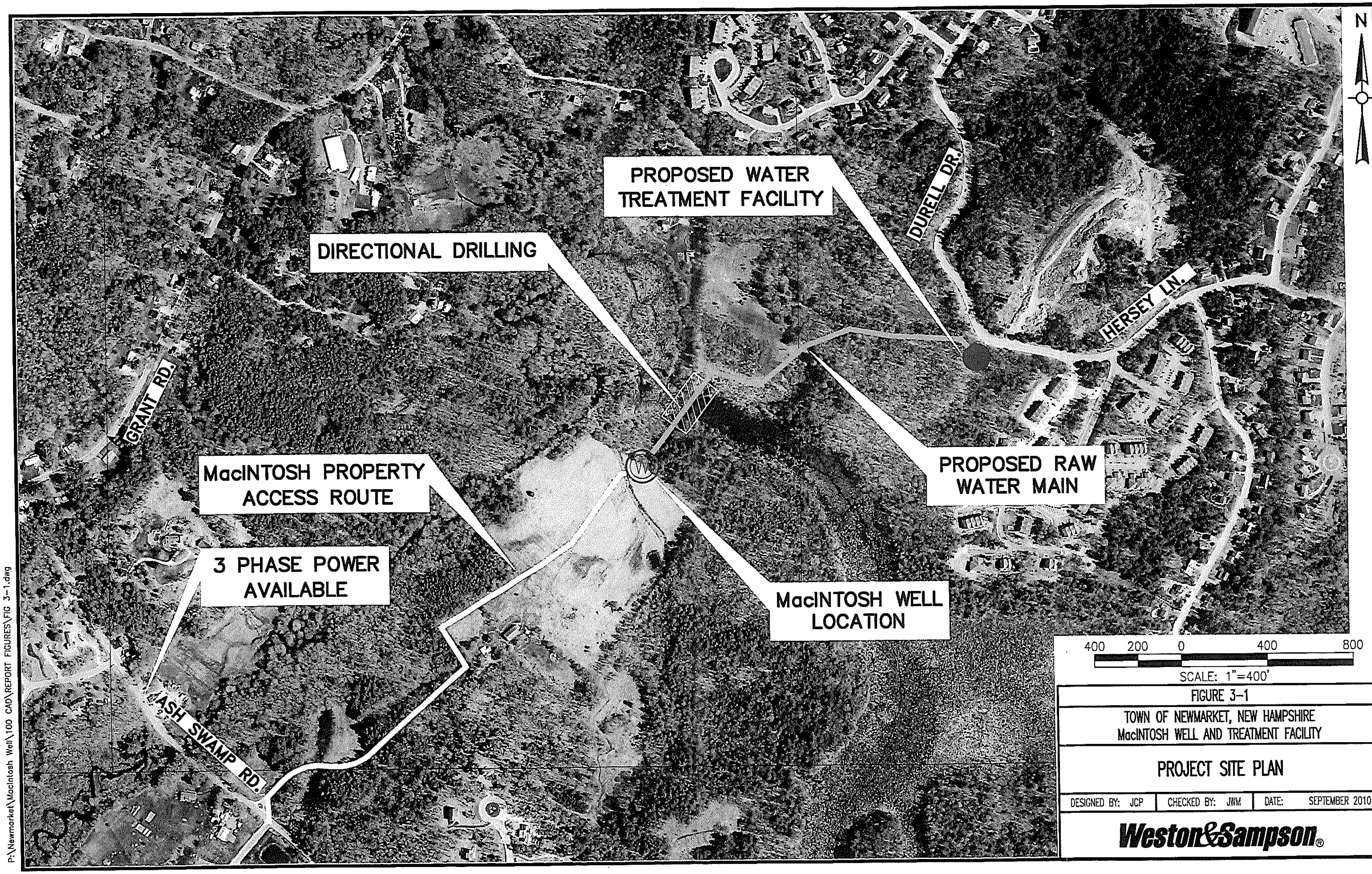


FIGURE 3-1

TOWN OF NEWMARKET, NEW HAMPSHIRE
MacINTOSH WELL AND TREATMENT FACILITY

PROJECT SITE PLAN

DESIGNED BY: JCP	CHECKED BY: JWM	DATE: SEPTEMBER 2010

Weston&Sampson®

P:\Newmarket\MacIntosh Well\100 CAD\REPORT FIGURES\FIG 3-1.dwg

4.0 PROJECT LAYOUT

4.1 Access Road Easement and Raw Water Main Layout

The project site is located on the MacIntosh farm off Ash Swamp Road. The property is currently being transferred to The Nature Conservancy and an easement is being prepared for the well area, access road to the well, and raw water line to Hersey Lane. Figure 3-1 identifies the location of these elements. Currently, the Nature Conservancy proposed easement width is 20 feet. However, upon further investigation with Public Service of New Hampshire (PSNH) with respect to supplying overhead three phase power to the well site, a wider easement may be warranted. PSNH also has vegetation trimming requirements for utility poles and minimum offsets between utility poles and the traveled way. The anticipated width to meet both of these PSNH requirements is 15 feet. Should the Town of Newmarket desire a 20-foot wide access road and overhead power, then a total easement width of 35 feet should be sought.

It is anticipated that the Town will construct the access road to the well during Fall 2010 after the easement has been finalized and approved. This will allow the Town to access the wellsite throughout the winter as final design of the wellhead is completed. The raw water main will emanate from the well casing, continue on an historical road (which is currently the site of beaver activity), past the test well, and through the field to the unimproved portion of Hersey Lane. We anticipate that the raw water main will be installed using horizontal directional drilling under the wetland at the beaver dam and via open cut methods in all other areas.

4.2 Wellhead Options

Although not shown to be in the 100-year floodplain (Zone A) on the most recent FEMA mapping, the lower lying areas surrounding the MacIntosh Well (within 25 feet of the casing pipe) were submerged by flood water during the March 2010 rain events. EGGI estimates the elevation of the water during flood conditions extended to approximately elevation 67 feet. A recent survey of the site revealed a ground elevation of 68.3 feet at the MacIntosh Well casing pipe.

One option for construction of the wellhead is outfitting the well with a submersible pump and pitless adapter and furnishing the electrical controls in an all-weather electrical box. This minimizes the infrastructure at the site but exposes the electrical equipment to the weather when the panel is opened for inspection or service. Motor drives also have a temperature limit below which they will not operate. As a result, a sufficient heating unit will need to be included within the electrical cabinet.

A second option is to include a small building to house the well and electrical controls. This option provides better protection of the well and electrical components. The well building can be precast concrete, metal sandwich panel, brick and block, or wood framed. Depending on the size and weather rating of the electrical cabinet required in the option above, providing a low cost building can be a cost competitive alternative to an electrical cabinet while providing superior access and protection of the well pump components.

4.3 Raw Water Main Size

WATER TREATMENT PLANT

The MacIntosh Well has a rated flow rate of 300 gpm. The approximate distance from the well head to the proposed WTP location is 2,200 feet. We evaluated using HDPE SDR 11 (160 psi) 8-inch diameter (DI pipe size) pipe, 8-inch diameter DI or PVC pipe for this installation. The following table compares hydraulics for the different pipe materials and interior diameters.

Table 4.1 – Raw Water Main Comparison for 300 gpm Flow Rate

Pipe Type	Nominal ID (in.)	C value*	Length (ft)	hL (ft/1,000 ft)	hL (ft)
8" HDPE	7.34	130	2,200	2.91	6.40
8" DI or PVC	8.00	135	2,200	1.83	4.03

* C value is an assumed Hazen Williams roughness coefficient

The Sharon Tucker Well has a rated flow rate of 275 gpm. Therefore, the combined flowrate of the MacIntosh and Sharon Tucker wells could be 575 gpm. The sources would manifold at the MacIntosh well site allowing for one transmission main between the MacIntosh Well and the water treatment facility located on Hersey Lane. We evaluated HDPE SDR 11 10-inch and 12-inch diameter pipe for this application. We also evaluated 10-inch and 12-inch diameter DI and PVC pipe. The following table compares hydraulics for the different pipe materials and interior diameters. The table also demonstrates the hydraulics that are expected should an 8-inch diameter pipe be installed and need to transmit up to 575 gpm in the future.

Table 4.2 – Raw Water Main Comparison for 575 gpm Flow Rate

Pipe Type	Nominal ID (in.)	C value*	Length (ft)	hL (ft/1,000 ft)	hL (ft)
8" HDPE	7.34	130	2,200	9.71	21.36
8" DI or PVC	8.0	135	2,200	6.11	13.44
10" HDPE	9.0	140	2,200	3.22	7.08
10" DI or PVC	10.0	140	2,200	1.93	4.25
12" HDPE	10.7	140	2,200	1.39	3.06
12" DI or PVC	12.0	140	2,200	0.79	1.74

* C value is an assumed Hazen Williams roughness coefficient

If the Town were to manifold the Sharon Tucker well in the future with the MacIntosh well, installing a larger diameter raw water main between the MacIntosh well and the site of the proposed WTP is warranted.

4.4 WTP Location

Idealy, the proposed water treatment plant would be located at the intersection of Hersey Lane and Durell Drive. At this location, there are three parcels of land that the Town should consider for the WTP. The first property is listed as parcel 2 on the Tax Map sheet R-4 of Newmarket. This parcel is described as a 0.86 acre property owned by A.W.L. Power, Inc of Newmarket, NH. It is also listed as open space on the Sewall Farm subdivision plan No. A-1704. It should be noted that A.W.L. Power, Inc. has been dissolved and the title to the property has been named to Durell Woods Association, Inc.

The second property is listed as parcel 134 on the Tax Map sheet R-4 of Newmarket. This parcel is described as a 2.6 acre property previously owned by A.W.L. Power, Inc. of Newmarket, NH and currently owned by Durell Woods Association, Inc. The parcel was approved by the Newmarket Planning Board to house two 4-unit condominium buildings as demonstrated on the Durell Woods subdivision plan No. A-1708.

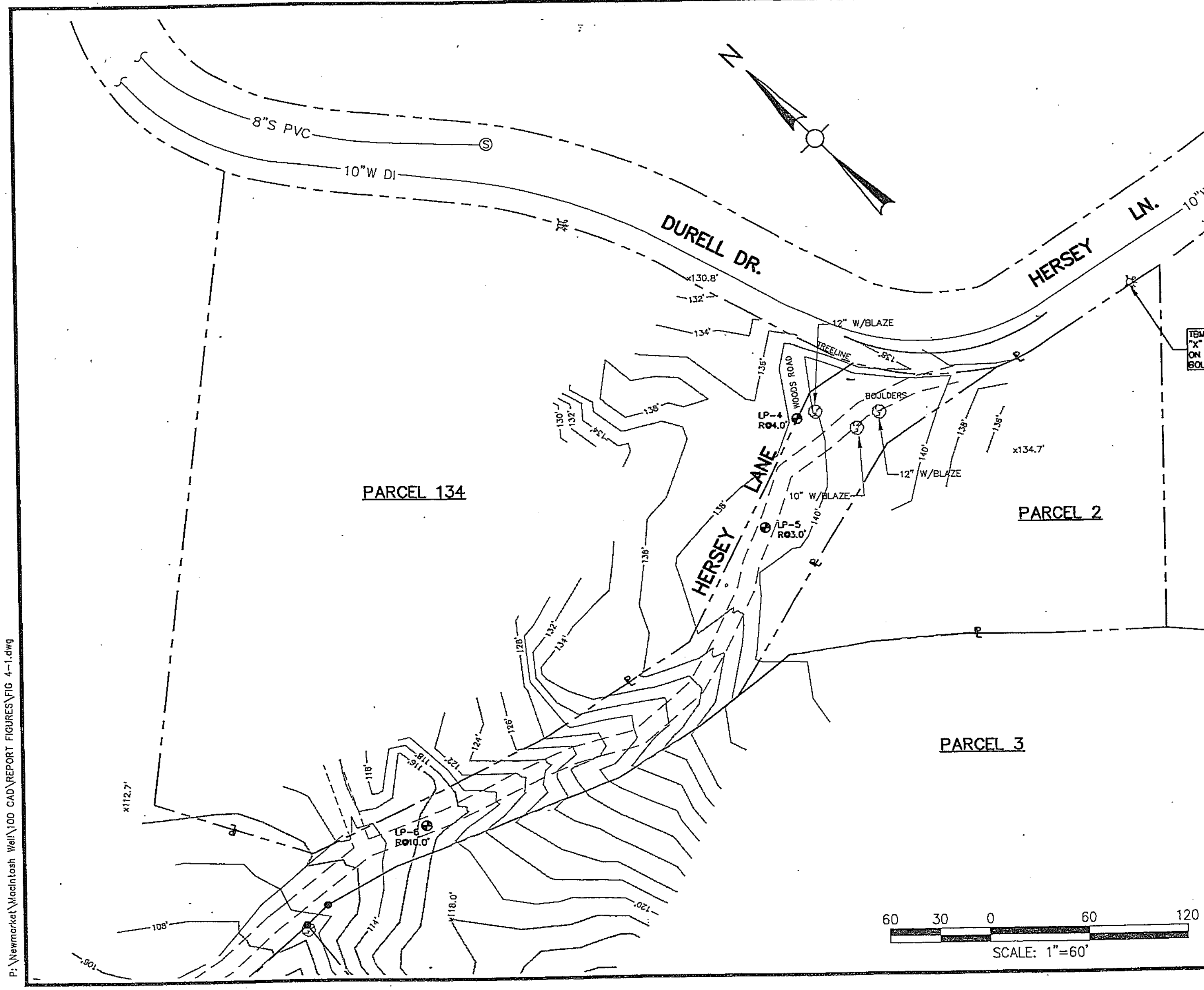


FIGURE 4-1

TOWN OF NEWMARKET, NEW HAMPSHIRE
MacINTOSH WELL AND TREATMENT FACILITY

PROPOSED WTP PARCEL LOCATIONS

DESIGNED BY: JCP	CHECKED BY: JWM	DATE: SEPTEMBER 20

Weston&Sampson®

P:\Newmarket\MacIntosh Well\100 CAD\REPORT FIGURES\FIG 4-1.dwg

The third property is listed as parcel 3 on the Tax Map sheet R-4 of Newmarket. This parcel is described as a 12 acre property now or formerly owned by the Hanna Webb estate. Refer to Figure 4-1 for the location of each parcel described herein.

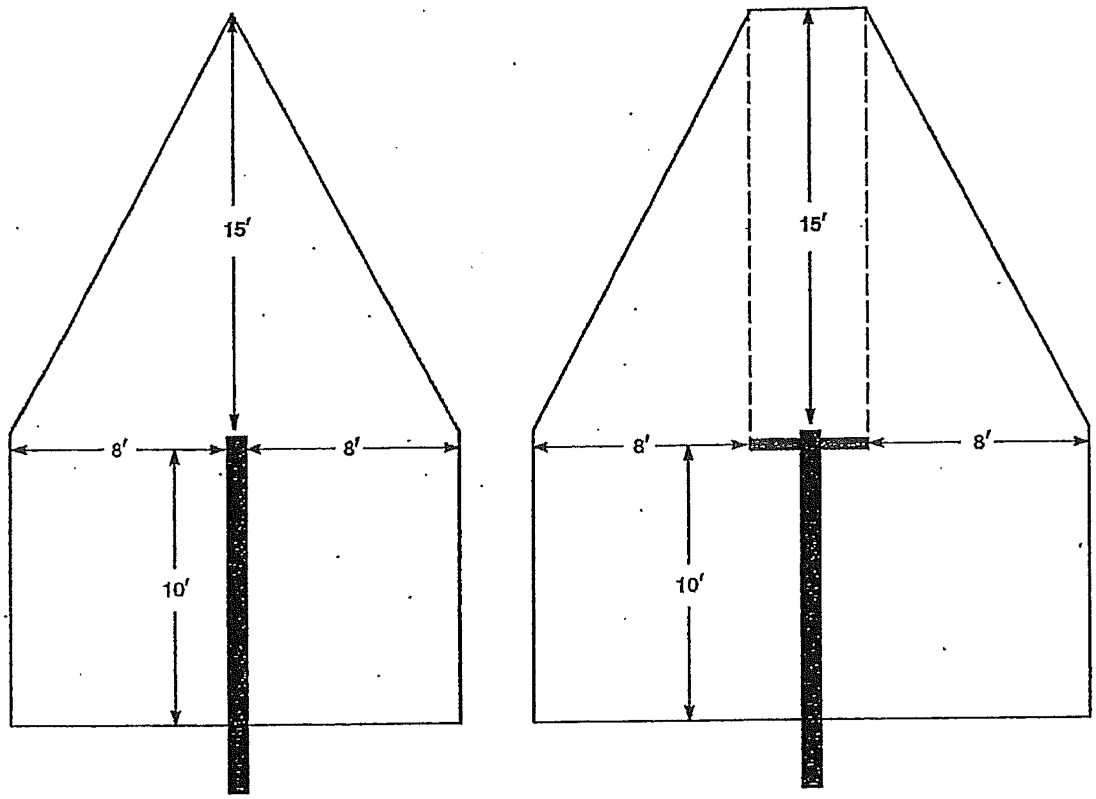
Parcel 2 or Parcel 134 should be considered prior to Parcel 3 due to their frontage on Durell Drive and/or Hersey Lane. Parcel 134 is closer to the existing sewer manhole in Durell Drive but the parcel is part of a subdivision plan for the Durell Woods condominiums. Parcel 2 should be looked at more closely to evaluate its status as open space for the Sewall Farm subdivision and its suitability to build a treatment plant.

4.5 Three Phase Power Investigation

The well site will require three phase power to power the well pump. One option to provide three phase power to the MacIntosh wellsite involves the installation of overhead wire and utility poles from Ash Swamp Road, up the driveway at 190 Ash Swamp Road and continuing along the proposed Nature Conservancy easement to the well (approximately 2,600 feet). Although there are existing utility poles between the MacIntosh house and Ash Swamp Road, they are owned by Verizon and are not available for use by Public Service of New Hampshire (PSNH).

To install overhead wires and utility poles for a new service, PSNH has multiple specifications that need to be met. One specification pertains to vegetation clearing distances around the utility pole and overhead primary conductors. Refer to Figure 4-2 for a diagram of clear distances. In recent years, PSNH has assumed the responsibility of trimming vegetation to acceptable standards during the installation of the new service.

In addition to the conservation easement, Newmarket is required to submit a utility easement application to PSNH. The utility easement is required under NH Tariff NHPUC No. 34 – Electricity. Based on the clearing distances described in Figure 4-2 and the minimal offsets required between a utility pole and the travel way, a utility easement of up to 20 feet wide may be required for an overhead electrical service installation.



CUSTOMER RESPONSIBILITY

Overhead Primary (2.4 – 34.5 kV) Conductors

See single-phase and three-phase figures. Minimum 10 feet clearance to the nearest primary conductor. Species recognized as fast growing and/or structurally weak are to be removed; examples include red maple, ash, white pine, cherry, silver maple, poplar, birch and willow. All other trees and limbs are to be trimmed back to suitable laterals consistent with approved arboricultural practices.

Hazardous Trees

Trees and/or limbs up to 16 inches diameter at breast height outside or inside the specified trim zone shall be removed when deemed structurally weak and likely to be a risk to the electrical system.

Secondary And Service Wire Conductors

Vegetation shall be trimmed if necessary to prevent hard rubbing and chafing which could lead to wear and failure of the conductors.

Inspections

An inspection of proper trimming clearances will be made by a PSNH representative. New services will not be installed or energized unless properly cleared of vegetation.

FIGURE 4-2
TOWN OF NEWMARKET, NEW HAMPSHIRE
MacINTOSH WELL AND TREATMENT FACILITY

VEGETATION CLEARING FOR NEW ELECTRICAL SERVICE

SCALE: 1"= N.T.S

P:\Newmarket\MacIntosh Well\100 CAD\REPORT FIGURES\FIG 4-2.dwg

The second option to install three phase power to the MacIntosh well site involves the installation of underground wire from Ash Swamp Road, up the driveway at 190 Ash Swamp Road and above ground continuing along the proposed conservation easement to the well. Installing electric service underground reduces the total utility easement area needed as compared to overhead service requirements but could cost up to two and a half times more to install.

Installing underground in the driveway also minimizes any vegetation trimming that is required along the entire length of the driveway.

The third option for installing three phase power to the MacIntosh well site would involve installation of underground wire from Ash Swamp Road to the well. As stated above, installing underground electric could be performed within the 30-foot easement width that is currently being sought. Underground electric installation along the entire easement route would also greatly minimize any vegetation trimming requirements. However, the cost to install underground electric along the entire easement route may cost up to two and a half times more than installing overhead electric.

The last option we considered, furnishing three phase power to the MacIntosh well site from Hersey Lane, was determined to be a non-viable option by PSNH. PSNH reports that the primary reason why this route was not an option is due to the complexities of installing power over or under the beaver dam. Three phase power, however, can be extended from the intersection of Hersey Lane and Pear Tree Lane to the site of the proposed WTP at the intersection of Hersey Lane and Durell Drive. Depending on which treatment alternative is chosen, either three phase or single phase power will need to be extended to the WTP. For budgetary purposes, we estimated the cost to extend three phase power to the site. Refer to Table 4.3 for a comparison of the total cost of the three options.

The above table incorporates the assumption that the Town will perform all necessary tree trimming costs, as required by PSNH, during the installation of the access road. If any trimming is left for PSNH to perform, the estimated cost will increase. The cost to furnish and install 5-inch electrical conduit includes the material cost of the conduit and pull string plus the cost to excavate a trench, install the conduit and backfill the trench. This cost could be reduced should the Town handle any of the labor associated with the installation of the conduit. For both options, the pad mount service transformer would be located within the 400-foot well protection zone. At this point in the preliminary design, PSNH has assumed that the transformer will be located approximately 200 feet away from the well head. Also, because the transformer is located in this protection zone, a special vegetable oil must be used with the transformer.

Table 4.3 – Three Phase Power Options Comparison

	Option 1	Option 2	Option 3
Overhead wires along Ash Swamp to driveway entrance	$43,000	$43,000	$43,000
Overhead wires from Ash Swamp to the service transformer	$65,000	N/A	N/A
PSNH pull underground electric in driveway Overhead wires in remainder of easement to service transformer	N/A	$70,000	N/A
PSNH pull underground electric between Ash Swamp to service transformer	N/A	N/A	$80,000
Underground 5-inch electrical conduit in driveway	N/A	$45,000	N/A
Underground 5-inch electrical conduit between Ash Swamp and service transformer	N/A	N/A	$110,000
Pad mount transformer for service drop	$8,500	$8,500	$8,500
5-inch electrical conduit between transformer and well head (approximately 200') and PSNH install charge	$10,000	$10,000	$10,000
Three phase overhead wires along Hersey Lane between Pear Tree Lane and Durell Drive (site of proposed WTP)	$40,000	$40,000	$40,000
Total	**$166,500**	**$216,500**	**$291,500**

As expressed in Table 4.3, each alternative has common costs associated with overhead wires along Ash Swamp Road and Hersey Lane. Since the location of these wires would be located within a public right of way, the Town should consider negotiating the final cost with PSNH.

4.6 Subsurface Investigation

On August 6th 2010, geotechnical borings were performed along the historical road on either side of the current beaver activity. The primary purpose of the borings was to identify existing subsurface conditions adjacent to the prime wetland for use in a potential horizontal directional drill. The borings were located as seen in Figure 4-3 and the logs are attached in Appendix A.

Boring B-1 is located on the north side of the wetland area adjacent to the existing test well. The boring was extended 44 feet without refusal and materials generally consisted of sand, silt, and clay.

Boring B-2 is located on the south side of the wetland area closer to the MacIntosh Well site. The boring was extended 9 feet 2-inches prior to meeting refusal and the materials encountered consisted of fine sand, silt, and clay. To confirm the presence of ledge, the boring (B-2B) was moved 5 feet away from the wetland area with the results finding refusal at 8 feet. The boring (B-2C) was moved an additional 10 feet away from B-2B and the wetland area to find refusal at 7.5 feet. The boring (B-2D) was moved an additional 10 feet away from B-2C and the wetland area to find refusal at 6 feet. It would appear there is a rising ledge profile between the wetland and the MacIntosh Well. The ledge profile dips down as the MacIntosh Well construction log shows bedrock at 28 feet below grade.

A series of ledge probes were conducted throughout the project area to assess the potential rock removal required during utility construction. Table 4.4 identifies the location and result of the ledge probes.

Table 4.4 – Subsurface Testing Results

Ledge Probe No.	Location	Depth to Ledge
LP-1	Access Road – Ditch Crossing	8'
LP-2	Access Road – Plowed Area	2.5'
LP-3	Access Road – Corner of Plowed Area	9'
LP-4	Hersey Lane – 80' from Durell Drive	4'
LP-5	Hersey Lane – 70' from LP-4	3'
LP-6	Hersey Lane – 43' uphill from x/c easement	> 10.5'
LP-7	Hersey Lane – 20' uphill from DSI 14 Hub	3.5'

LP-2, in Table 4.4, indicates some rock excavation will be necessary should the three phase power be provided from Ash Swamp Road via underground construction. Other than LP-6, the Hersey Lane ledge probes shown in Table 4.4 indicate significant rock excavation will be required to construct the water main in Hersey Lane at a 5 foot depth of cover on the pipe.

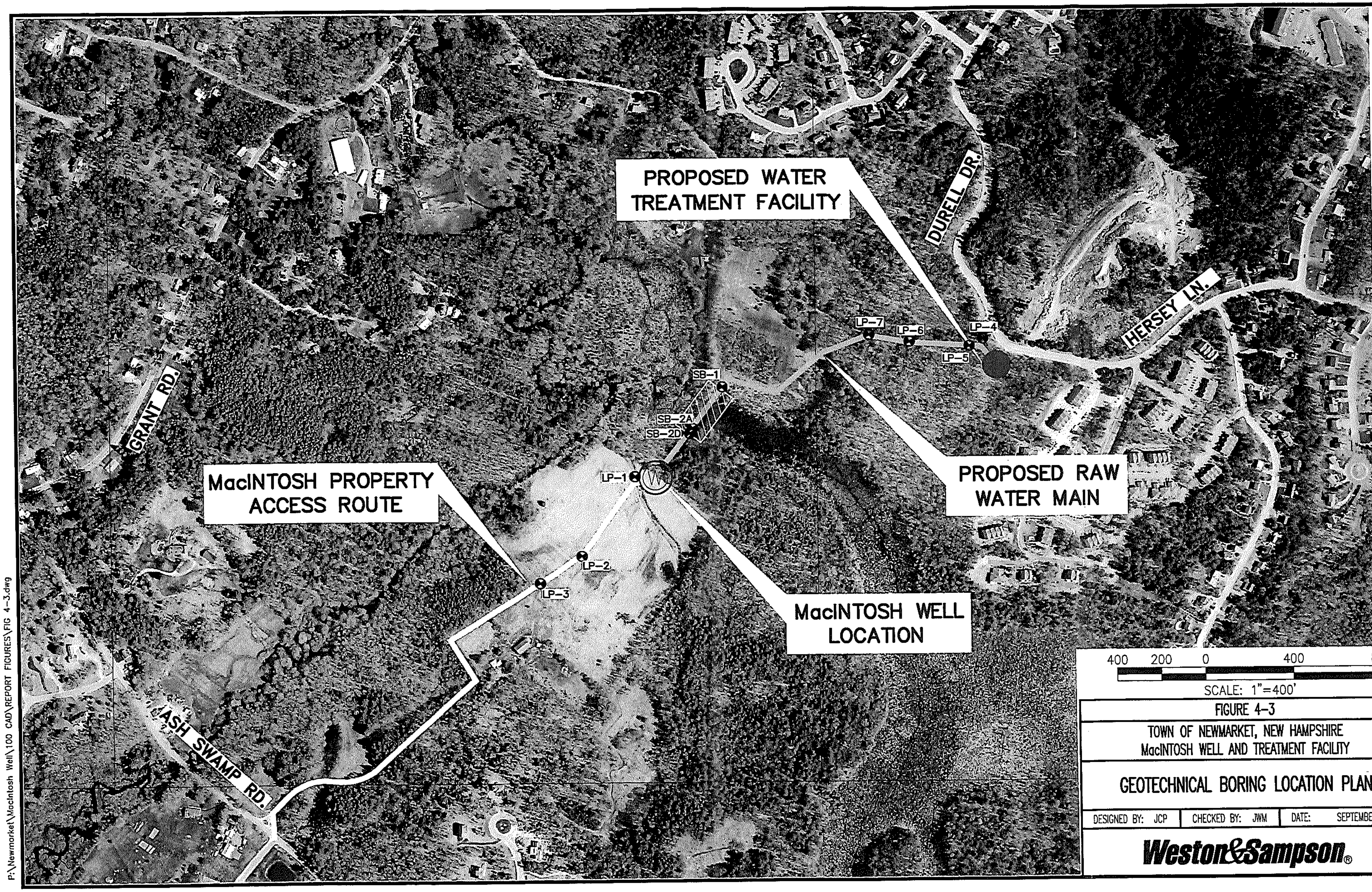


FIGURE 4-3

TOWN OF NEWMARKET, NEW HAMPSHIRE
MacINTOSH WELL AND TREATMENT FACILITY

GEOTECHNICAL BORING LOCATION PLAN

DESIGNED BY: JCP	CHECKED BY: JWM	DATE: SEPTEMBER 2010

Weston&Sampson®

P:\Newmarket\MacIntosh Well\100 CAD\REPORT FIGURES\FIG 4-3.dwg

5.0 ALTERNATIVES ANALYSIS

5.1 General

The following section presents our evaluation of the different treatment alternatives for treating MacIntosh well water.

5.2 Disinfection and pH Adjustment Only

If water from the proposed MacIntosh well is pumped into the municipal distribution system with only disinfection and pH adjustment applied, the following can be expected:

- On occasion, based on limited raw water quality data, it appears as though water from the MacIntosh well will violate the MCL for arsenic. NHDES will require the MacIntosh well be taken off line when the pumped levels of arsenic are 0.01 mg/l or higher and the Town will have to implement the pubic notification requirements for arsenic limit violations.
- On occasion, the Town may receive black color or staining water complaints from users in the immediate vicinity of the proposed WTP at Hersey Lane and Durell Drive due to the high manganese levels in the MacIntosh well water.
- During the MacIntosh well pump test performed by EGGI, a white precipitate formed at the well head. Similar mineral deposits can be expected in hot water heaters, dish washers, distribution pipes and bath fixtures throughout the distribution system due to the high TDS levels in the MacIntosh well water. In addition, high TDS levels can act as a stool softener and may impact more sensitive water customers.
- Sodium and chloride taste complaints will occur, particularly where the water from the MacIntosh Well interfaces with water from the two existing gravel pack wells in the distribution system due to improper mixing.

Although the raw water quality of the MacIntosh well could improve over time due to continual pumping of the well, the initial raw water quality suggests that pumping MacIntosh well water directly into the distribution system with only disinfection and pH adjustment is not an option.

5.3 Sequestering Agent Addition

Adding a sequestering agent allows certain constituents contained in raw water to remain in solution. One common type of sequestering agent is polyphosphate, which provides many functions including the sequestering of iron, manganese, calcium and magnesium. Different types of polyphosphates exhibit a different reactive sequestering rate for each of these metals/minerals. Two common types of polyphosphates are hexametaphosphates and pyrophosphates.

Hexametaphosphates (polyphosphate) inhibit scale formation caused by calcium and magnesium through sequestration and crystal growth modification at a rate 20 times more effective than pyrophosphates. However, pyrophosphates can sequester iron and manganese at a rate 16 times more effective than hexametaphosphates. For the MacIntosh Well both calcium and manganese would be sequestered. However, since calcium and scale formation from high TDS levels are the predominant concern, polyphosphate addition would be the preferred sequestrant.

In order to sequester calcium and manganese, a 1:1 mole ratio with polyphosphate is required. This is generally achievable when dealing with manganese concentrations that are present at levels less than 5 mg/L. However, polyphosphates act as crystal modifiers that need only a fraction of that ratio to effectively modify the crystalline structure of calcium. A hexametaphosphate dosage of 0.7 mg/L is required to modify the crystal growth of calcium carbonate, reduce the occurrence of precipitation into hard scale formation and inhibit manganese from coming out of solution.

Sequestering of calcium and manganese would limit scale formation, stool softening impacts from elevated TDS levels and black staining complaints. However, the arsenic, sodium and chloride levels in the raw water would remain elevated. As a result, if only a sequestrant were added to treat the target raw water constituents listed above, the MacIntosh Well water will violate the MCL for arsenic on occasion and produce a salty taste in the finished water where the MacIntosh Well water interfaces with the gravel pack well water in the distribution system.

5.4 Blending MacIntosh Well Water

5.4.1 Water Blending Methodologies

One option for treating the MacIntosh Well water is to blend it with distribution system water prior to pumping into the distribution system. Currently, the Town receives its source water from the Bennett and Sewell gravel packed wells. Both wells have very low to non-detect levels of iron, manganese and arsenic. In addition, the levels of sodium, chloride and TDS are approximately four to six times lower in the Bennett and Sewell wells than those levels present in the MacIntosh well.

Blending would be achieved by redirecting distribution system water into the proposed WTP, combining it with MacIntosh well water and thoroughly mixing the two sources. We evaluated a 50/50 blend of MacIntosh Well water and distribution system water primarily due to the level of treatment required to reduce the elevated levels of TDS in the MacIntosh water. If a 50% distribution system and 50% MacIntosh Well blend is used, the net product after blending would be a water quality that meets the MCL's and/or secondary standards for arsenic, chloride and TDS. The manganese levels would be equal to or slightly above the secondary standard. The sodium levels would be below the World Health Organization (WHO) guidelines but above levels at which taste is detected (60 mg/L) by consumers. To achieve a 50/50 blend, both the Sewell and Bennett wells would need to be running since neither well matches the 300 gpm rating of the MacIntosh well.

Currently, the existing wells are operated based on the elevated water storage tank water levels. At elevation 56 feet in the existing water storage tank, the Sewell well turns on and after a few minutes delay, the Bennett well turns on. If a proposed blending WTP were constructed, the MacIntosh Well would be interlocked with the existing SCADA system that controls the operation of the existing wells. When the MacIntosh well is called to turn on, the existing wells would turn on and remain on throughout blending operations.

If the larger of the two existing sources, the Sewell well, were out of service, and the MacIntosh well needed to be run at its maximum rating, 300 gpm, the blend ratio of the Bennett and MacIntosh wells would be 40% Bennett/60% MacIntosh. It should be noted that distribution system hydraulic concerns would be present under this scenario, as described below.

With a 40/60 ratio, the net product after blending would be a water quality that meets the MCL's and/or secondary standards for arsenic, chloride and TDS. The secondary standard for manganese would be slightly exceeded. The sodium levels would be below the WHO guidelines but above the level that a salty taste is detected by consumers. By comparison, the finished water quality declines with the 40/60 blend versus the 50/50 blend as shown in Table 5-1.

From the EGGI report, continued use of the MacIntosh Well should lower the raw water chloride to acceptable levels. If the MacIntosh Well is out of service for a given length of time, purging of the well is recommended prior to blending. A blow-off mechanism at the well head should be utilized in order to waste the initial raw water produced by the MacIntosh well when the well pump turns on.

5.4.2 Hydraulic Modeling Results

Blending the MacIntosh well water with distribution system water was simulated in the water distribution system hydraulic model. A blending tank, booster pump, flow control valve, automated in-line valve and pressure reducing valve were inserted into the model at the proposed location of the water treatment plant located at Hersey Lane and Durell Drive. The following describes preliminary concepts for achieving distribution system operation during blending operations.

To effectively distribute water away from the proposed treatment plant, we envision an automated in-line valve installed on the existing 10-inch DI water main in Hersey Lane. This valve would automatically close when the MacIntosh well is in operation to create a unidirectional flow of water on Durell Drive, through the treatment plant blending tank and out to Hersey Lane. When the MacIntosh well is off, the automated valve would open allowing water to flow in both directions on Durell Drive.

To regulate the flow of distribution system water into the blending tank, we envision a flow control valve would be located on the upstream side of the blending tank. The flow control valve would be set to the flow rate required to meet the blending ratio desired or needed based on the gravel packed well(s) in operation.

Operating one or both existing wells is imperative to blending the MacIntosh well water from both a water quality and hydraulic standpoint. In section 5.3.1, we describe the significance of operating the existing wells for the purpose of water quality operations. For hydraulic considerations, we used the model to assess the available flow at the proposed WTP. Under a maximum day water demand, 300 gpm from the distribution system is available if both the Bennett and Sewell wells are in operation. However, during a maximum day water demand, flows higher than 200 gpm with one existing well in operation will not maintain a minimum distribution system pressure of 35 psi with the proposed in-line valve closed. No distribution system flow is available (for blending operations) with both the Bennett and Sewell wells offline and with the proposed in-line valve closed. It should be noted that a hydraulic restriction in the distribution system contributes to these results. The hydraulic restriction is an unlined cast iron 6-inch diameter water main that exists in South Main Street between the 10-inch diameter DI water main at the entrance into Newmarket high school and the 8-inch diameter DI water main at the entrance into Newmarket elementary school.

As stated above, during blending operations a uni-directional flow condition would be created in the Hersey and Durell area while the MacIntosh well is operating and the automated in-line valve is closed. We assessed fire flow capacity of the system just upstream of the WTP to determine what impacts the blending infrastructure would have on the distribution system during a fire event.

Our findings indicate that with both the Bennett and Sewell wells running at 220 gpm and 270 gpm, respectively, and the proposed valve at the Hersey and Durell intersection closed, the system can achieve a 550 gpm fireflow during a maximum day demand event with 20 psi residual pressure. However, the proposed WTP blending alternative requires 300 gpm from the distribution system, which leaves an effective 250 gpm fireflow on Durell Drive with the proposed in-line valve closed. Fireflow is severely reduced due to the hydraulic capacity of the 6-inch unlined main on South Main Street which restricts the water in the storage tank from contributing to the fire flows when the valve at the Hersey and Durell intersection is closed.

To increase fire flows on Durell Drive with the proposed in-line valve closed as required during the blending treatment alternative, we used the hydraulic model to analyze the effect that a new pressure reducing valve would have on allowing water to flow north on Hersey and Durell. The PRV would be designed to open if system pressures on the down stream side of the valve approach 35 psi during blending operations. Installation of a PRV at this location would enable a fire flow during a maximum day demand event of up to 1700 gpm, at 20 psi residual pressure, at the proposed WTP during blending operations with one existing gravel pack well out of service. This flow is consistent with the fire flow achieved if the proposed in-line valve were open and is the current flow available at this location in the distribution system.

5.5 Temporary Treatment Systems

Since EGGI indicates the MacIntosh Well water quality will improve as the well is pumped in the future, one option is to treat the MacIntosh Well water with a temporary rental of an ion exchange treatment system. If the raw water quality changes over time, the treatment requirements can be reassessed at a later date. It should be noted that temporary pressure treatment filtration units and temporary electrodialysis reversal (EDR) units are not readily available for potable water treatment.

5.5.1 Ion Exchange

The ion exchange or deionization process works by removing all ions from a solution. Cation exchangers remove all cations (positively charged ions such as sodium, calcium, and magnesium) and anion exchangers remove all anions (negatively charged ions such as sulfate, bicarbonate, and chloride). In a treatment system, the cation exchanger is operated in front of the anion exchanger and converts the salts to acids.

Treating the maximum flow from the MacIntosh Well (300 gpm) with ion exchange technologies is considered cost prohibitive. Treating approximately 150 to 175 gpm of the 300 gpm MacIntosh well water is recommended to reduce costs. The 150 to 175 gpm of treated water would be demineralized through the ion exchange process resulting in virtually no dissolved ions remaining in the finished product. The treated water would then be blended with the approximately 125 to 150 gpm of untreated MacIntosh well water so that the blend would result in an approximate 60% reduction in the dissolved ion concentrations (sodium, chloride, TDS, calcium, and magnesium). There would likely only be a 10 to 30% reduction in the manganese and arsenic concentrations.

Typically, with a temporary ion exchange rental system, trailers having cation-anion-mixed beds would treat the supply until the ion exchange resin becomes exhausted. Due to the MacIntosh Well's water quality, the ion exchange resin would be exhausted every day due to the high ionic loading. At that point, a new trailer with freshly regenerated ion exchange resin would be delivered to the site and would replace the trailer with the exhausted resin. In a typical application, the water system operators would perform the plumbing and power connections each time the resin beds are changed out. The Town would also be responsible for monitoring water quality to determine the frequency of replacement.

Temporary ion exchange treatment systems are often used in the power generating industry and could be furnished by any number of firms that offer this type of service in the New England area. As such, the equipment and services described above are readily available to the Town.

5.6 Permanent Treatment Systems

5.6.1 Pressure Filtration

Pressure filtration is one option for a manganese and arsenic removal treatment plant. In pressure filtration, vertical or horizontal pressure vessels are installed in parallel and utilize a filter media designed to operate at predetermined loading rates and differential pressures. Pretreatment with chemicals is typically performed to achieve oxidation of the arsenic and manganese and to perform pH adjustment, if necessary. Post treatment with chemicals can be performed for pH adjustment and disinfection. It should be noted that pressure filtration will not remove radon, chlorides, sodium or TDS; other than iron and manganese. It should also be noted that ferric chloride may need to be added to the raw water to assist in arsenic removal if the naturally-occurring iron to arsenic ratio is not optimal for arsenic reduction.

The following is a brief description of three filter media types that would be considered for manganese and arsenic removal at the proposed WTP.

GreensandPlus

GreensandPlus media, manufactured by Inversand of Clayton, NJ, is a filter media that consists of an adsorptive coating fused to a durable silica sand particle core. The manufacturer reports that the media can withstand differential pressures as high as 30 psi with no breakdown of the media or coating and can, therefore, operate at filter loading rates of 6 to 8 gpm/sf (or greater, depending on raw water quality). The GreensandPlus media has a coating of manganese oxide that assists in the removal of arsenic and manganese from the water. This type of filtration typically requires chemical pretreatment with sodium hypochlorite or potassium permanganate to oxidize dissolved arsenic and manganese. The coating on the filter media is maintained through either continuous or intermittent feed of potassium permanganate or sodium hypochlorite. Depending on raw water characteristics, pH adjustment and/or addition of ferric chloride may also be necessary as a pretreatment step in order to optimize precipitation of arsenic and manganese. A typical manufacturer-recommended filter comprises 18 inches of GreensandPlus media with 18 inches of anthracite on top, and is backwashed at 12 gpm/sf with or without air scour.

Pureflow PM-300

The PM-300 media, manufactured by Pureflow Filtration of Whittier, CA, is a high-rate adsorptive media that can effectively operate at filter loading rates up to 15 gpm/sf, but may be limited to 10 gpm/sf as approved by certain regulatory agencies. The arsenic and manganese are first oxidized typically with sodium hypochlorite, and the precipitant of arsenic and manganese are adsorbed onto the filter media. Depending on raw water characteristics, pH adjustment and/or addition of ferric chloride may also be necessary as a pretreatment step in order to optimize precipitation of arsenic and manganese. At the completion of a filter run, full-scale systems are backwashed at 20 gpm/sf for four minutes.

LayneOx

LayneOx is a granular filter media manufactured by Layne Christensen of Bridgewater, NJ. The media operates both as a classical filter with an oxidant (typically sodium hypochlorite, potassium permanganate, or occasionally potassium hydroxide) and as a catalytic media, which operates at filter loading rates between 8 and 15 gpm/sf but may be limited to 10 gpm/sf as approved by certain regulatory agencies. Depending on raw water characteristics, pH adjustment and/or addition of ferric chloride may also be necessary as a pretreatment step in order to optimize precipitation of arsenic and manganese.

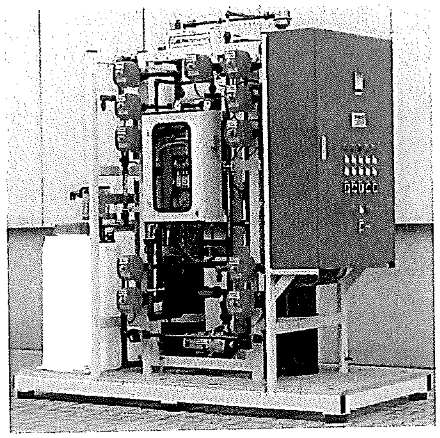
A typical filter comprises 24- to 48-inch depths of LayneOx media. Since the media has a high unit weight (120 pounds per cubic foot), full-scale systems are generally backwashed at 30 gpm/sf for three minutes without air scour, or at 12 to 15 gpm/sf for 5 to 10 minutes with air scour.

Residuals

For the three filter media types listed, the residuals would be collected in a holding basin and discharged to the sewer system. Depending on the settling characteristics of the solids in the residuals volume, recycling of supernatant contained in the top half of the collected residuals may be possible. Residuals volumes will vary depending on the type of filter media used but could range from 10,000 to 20,000 gallons per day.

5.6.2 Ion Exchange

Similar to the temporary ion exchange treatment alternative, a permanent ion exchange system could treat a side stream flow of 150 to 175 gpm of the 300 gpm MacIntosh Well water, resulting in virtually no dissolved ions in the treated product. This treated side stream would then be blended with approximately 125 to 150 gpm of untreated MacIntosh well water so that the blend would result in an approximately 60% reduction in dissolved ion concentrations (sodium, chloride, TDS, calcium, and magnesium) and 10-30% reduction in the manganese and arsenic concentrations. The ion exchange system would consist of (2) 100% capacity trains of cation-anion-mixed bed ion exchangers. The expected service life of each train would be approximately six to eight hours before the train is taken off-line for regeneration; a process that typically takes approximately four hours. The cation resin is regenerated with sulfuric acid and the anion resin is regenerated with sodium hydroxide. The expected water recovery is approximately 90-percent. Regeneration wastes would be neutralized and discharged to the sewer system.

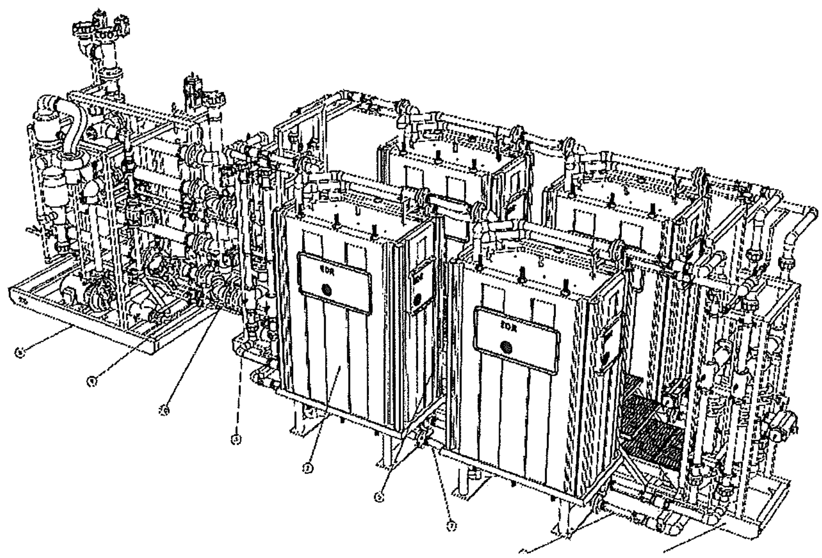


Typically, ion exchange treatment systems are not a cost-effective means of dematerializing water that has a total dissolved solids concentration greater than 250 mg/L expressed as CaCO3 (calcium carbonate). The MacIntosh Well water is approximately 1,100 mg/L as CaCO3.

5.6.3 *Electrodialysis Reversal*

Electrodialysis reversal (EDR) is a process that applies an electric potential that moves dissolved salt ions through an electrolysis stack consisting of alternating layers of cationic and anionic ion exchange flat sheet membranes. This creates alternate channels of desalted product water and concentrated reject water. By automatically reversing the polarity of the applied electric potential on the stack every 15 to 30 minutes, the effects of inorganic scaling and fouling are minimized; converting product channels into concentrate channels and vice versa. On a daily basis, the electrodes are typically cleaned for 45 minutes with hydrochloric acid to control carbonate scale formation. Periodically, the membrane stacks require a chemical clean in place (CIP) to remove foulants by circulating a chemical cleaning solution through the EDR stacks. This would occur every 4-6 weeks and take four to five hours to complete, thereby taking the WTP offline for four to five hours.

A single EDR stage can typically remove 50-60 percent of the dissolved solids. For greater salt reduction, membrane and electrode stacks are staged in series one after another. EDR does not remove silica and micro-organisms from the feed water and removes very little total organic carbon (TOC). Feed water quality requirements to the EDR system require that turbidly be less than 2 NTU, iron less than 0.3 mg/L, and manganese less than 0.1 mg/L or pretreatment will be required to reduce the contaminants. EDR membranes can tolerate ~ 0.5 mg/L free chlorine continuously. Typical water recovery of an EDR system is 85 percent although recoveries of greater than 90 percent are possible treating certain types of water.



The EDR system proposed by General Electric (GE) for Newmarket would produce approximately 300 gpm of product water using a single two-stage treatment process train and remove approximately 68-80 percent of the dissolved inorganic ions (sodium, chloride, calcium, magnesium, and TDS) as well as approximately 55-60 percent of the dissolved metals (manganese and arsenic).

Major components of the EDR system would include the following:

- Cartridge filters
- Feed pump
- EDR stacks
- Piping and valves
- Wiring
- Concentrate recycle pump
- Electrode CIP system
- Stack CIP system
- Electrical rectifier

- Instruments and controls
- Control panel

GE has indicated that the water recovery of the EDR would be approximately 92 percent. The final water quality from the EDR system would be very similar to that of the other two existing production wells. The average waste flow rate from the EDR treatment system would be approximately 26.1 gpm and could be directed to the Town's sewer system. Assuming that the system runs for twelve hours a day, this equates to a residuals volume of approximately 19,000 gallons.

There are no EDR system installations in New England at the time of this report. New England groundwaters that require treatment typical contain elevated levels of iron, manganese and/or arsenic. New England groundwater wells are not typically installed in areas subjected to brackish groundwater environments with high TDS levels in combination with elevated levels of arsenic and manganese; which is the case with the MacIntosh Well. As a result, EDR treatment may not be the best alternative treatment system for typical New England groundwaters but appears to be a suitable treatment alternative for the MacIntosh Well water in particular.

There are hundreds of EDR systems installed worldwide including several within the United States. The following is a partial list of continental United States locations:

- Mason City, Iowa
- Magna Water, Utah
- City of Suffolk, Virginia
- Sarasota County, Florida
- City of Sherman, Texas

5.7 Impacts of Treatment Alternatives on Finished Water Quality

In order to better compare the advantages and disadvantages of each treatment alternative listed in Chapter 5, Table 5.1 was developed to summarize some of the advantages and disadvantages of each process. Table 5.2 presents the anticipated finished water quality that can be expected for each stand-alone treatment alternatives described in Chapter 5. Maximum reported MacIntosh Well raw water concentrations were used to compute estimated finished water quality in the table. The following is a discussion of the different water quality parameters we evaluated.

Manganese

The blended treatment, pressure filtration and EDR alternatives will achieve generally acceptable levels of manganese in the finished water. The following is offered as a guide to the customer impacts of the varying levels of manganese associated with the treatment alternatives:

If the pressure filtration alternative is implemented, finished water manganese levels should be below 0.025 mg/L. Manganese at this low level will have no impact on customers.

If the blended water or EDR alternative is implemented, anticipated finished water manganese levels are computed to be between 0.06 mg/L and 0.04 mg/L. We recommend a sequestrant be added to the MacIntosh well water prior to blending and chlorine addition. Manganese at this level will have no impact on typical residential customers. Industrial customers with high temperature boilers or chillers would notice the change in water quality and would need to adjust their existing water treatment practices to account for the higher manganese levels.

If the ion exchange alternative is implemented, anticipated finished water manganese levels are computed to be 0.08 mg/L. Manganese at this level will oxidize and precipitate over time, even in the presence of a sequestrant, likely causing black staining complaints. Flash hot water heaters in particular will cause the sequestrant to break down faster. High water age in the distribution system will result in an increase in black staining complaints from customers.

Table 5.1 – Advantages and Disadvantages Treatment Alternatives Comparison

Treatment Alternative	ADVANTAGE	DISADVANTAGE
Disinfection and pH Adjustment Only	• Low cost • Minimal disruption to existing operations • No residuals produced	• Severe decline in overall water product served to customers • Risk of DES shut down due to arsenic levels • Noticeable sodium taste • Black staining and scale concerns • High TDS levels which could lead to customers experiencing loose stools
Sequestering Agent Addition	• Low cost • Minimal O&M • Should effectively reduce black staining and scale formation potential • No residuals produced	• Water heaters and boilers will have scale formation • Sequestering does not reduce arsenic or sodium levels
Blending MacIntosh Well Water	• Moderate cost • Reduces all five constituents evaluated in this report. Mn at or slightly above secondary standard • Can be constructed to blend Sharon Tucker Well water • No residuals produced	• Relies on existing wells. Not an independent treatment alternative. • Requires distribution system flow control components • Requires double pumping
Pressure Filtration	• Reduces Mn and As effectively • Common type of treatment for New England waters • Reasonable cost for level of treatment provided • Easily expandable for Sharon Tucker Well • Does not require double pumping	• Does not remove sodium, chloride, calcium and several other total dissolved solids • May require ferric chloride addition for arsenic removal
Ion Exchange	• Effectively reduces sodium, calcium and TDS. • Easily expandable for Sharon Tucker Well	• High capital and O&M costs • Provides only a slight reduction in Mn and As levels
EDR	• Provides a reduction in the five primary constituents we evaluated in this report • Sodium and chloride are reduced the most with this treatment • Reasonable cost for level of treatment • Easily expandable for Sharon Tucker Well	• Moderately high maintenance costs • No New England installations • Requires a greater level of operator experience • Requires double pumping

Table 5.2 – Finished Water Quality Comparison of Treatment Alternatives

Constituent	Current MCLG[1] (mg/L)	Current MCL[2] (mg/L)	Current Secondary Standard (mg/L)	WHO Advisory Limit (mg/L)	50/50 Blend - MacIntosh/ Sewell + Bennett Well	60/40 Blend - MacIntosh/ Bennett Well	Pressure Filtration - MacIntosh Well	Ion Exchange - MacIntosh Well	EDR - MacIntosh Well
Manganese			0.05		0.053	0.058	0.025	0.080	0.043
Arsenic	0	0.01			0.0067	0.0068	0.0010	0.0088	0.0047
Sodium				200*	152	158	250.0	100.0	77.5
Chloride			250		203	209	264	128	48
TDS			500		456	469	700	280	350

1 – Maximum Contaminant Level Goal (MCLG)
2 – Maximum Contaminant Level (MCL)
* WHO advisory 200 mg/L based on taste. Taste is detected at 60 mg/L.

Arsenic

Treating MacIntosh well water via pressure filtration, EDR or blending of water from the existing gravel pack wells will all lower MacIntosh well water arsenic levels to acceptable finished water levels below the MCL.

If the pressure filtration alternative is implemented, arsenic levels could be reduced to levels that are at or below 0.001 mg/L. This level of treatment is considered the Best Alternative Treatment (BAT) for arsenic treatment by the EPA.

If the EDR or blended water treatment alternatives are implemented, arsenic should be lowered to levels that are approximately half to two thirds of the MCL limit. These levels are generally considered acceptable, but some customers may object to the introduction of an increased concentration of arsenic into the water supply.

If the ion exchange alternative is implemented, finished water arsenic levels are anticipated to be as high as 0.009 mg/L using ion exchange treatment. At this level, the finished water arsenic concentration is only 0.001 mg/L below the MCL limit, which raises concern about the ability of ion exchange to effectively treat the MacIntosh Well.

Total Dissolved Solids, Chloride and Sodium

EDR, Ion Exchange and blending of water from the existing gravel pack wells will all lower dissolved solids of the MacIntosh well water to levels below the secondary standards as seen in Table 5-1. EDR treatment reduces the sodium and chloride levels to a higher degree than the other treatment alternatives.

Pressure filtration will only reduce the TDS concentrations by the level of manganese that is removed. The overall TDS levels would stay above the secondary standards.

If treatment via the blended water alternative is implemented, TDS levels over 400 mg/L but below the secondary standard of 500 mg/L are expected. Customers will notice the change in water quality and standard hot water heaters and automatic dish washers will be negatively impacted. Industrial customers and users that require ultra pure water will have increased costs to treat for the removal of the higher TDS levels. To control carbonate deposition, we recommend a sequestrant be added to the MacIntosh well water prior to blending.

If EDR or Ion Exchange treatment is implemented, the TDS levels will be below 400 mg/L. The average residential water customer should not be significantly impacted. Industrial customers and users that require ultra pure water will have increased costs to treat for the removal of the higher TDS levels. Although TDS levels are lower with either EDR or Ion Exchange treatment, to control carbonate deposition, we recommend a sequestrant be added to the MacIntosh well water after treatment by EDR or Ion Exchange.

5.8 Piloting Requirements

Piloting one or more of the treatment alternatives described above is warranted and, per NHDES regulations, will need to be conducted prior to the start of the final design phase. Due to the remote location of the MacIntosh well, access and power capability will need to be established prior to the start of pilot testing. Generally, companies that specialize in on-site pilot testing services will house their pilot equipment inside a trailer.

A cart path through the existing field leads to the well head providing suitable access to the well. Power, is not currently available at the well head and a portable electric generator (or generators) would be required. The field portion of the pilot program will most likely be conducted in one week's time.

6.0 PRELIMINARY TREATMENT COST ESTIMATE

The following provides a preliminary cost estimate of the different treatment alternatives described in Chapter 5. We are not providing a cost for the disinfection and pH adjustment only alternative nor the sequestration only alternative as we do not believe these are viable stand-alone treatment options.

6.1 Blended Water

The blended water alternative is estimated to cost $100,000 to reroute water main into the proposed WTP and install the PRV, automated in line valve, flow control valve, mixing equipment and booster pump. For this option, we have assumed a building size of 40 feet by 25 feet to contain chemical feed equipment, flow control equipment, pump control equipment and a below grade mixing tank. The cost of the building, chemical feed equipment, pump equipment and installation of a mixing tank and rock excavation needed to install the below grade mixing tank is estimated to be $365,000. The estimated construction cost as it relates specifically to this alternative is $465,000. The estimated annual O&M cost is approximately $30,000 per year. Based on amortizing the installed capital costs over 30 years at 4-percent interest rate and including the annual O&M costs (with an average 2.5% inflation rate) , the total cost of treated water is approximately $0.89/1,000 gallons for the blended water system.

The cost to upgrade the 6-inch water main in Main Street is included in the overall capital cost of this alternative since this section of the distribution system was found to be a hindrance to blending operations. The estimated project cost to replace approximately 1,300 linear feet of 6-inch unlined cast iron water main with 12-inch ductile iron water main in state highway (Route 152) is $227,500 (including engineering and contingency). We did not include this cost with the total cost of treated water calculation described above.

6.2 Pressure Filtration

The pressure filtration system would cost approximately $350,000 for the equipment, $87,500 to install and approximately $700,000 to furnish a 40 ft. by 60 ft. building, chemical feed equipment, process piping, below grade residuals holding tank (plus the cost of rock excavation, residuals pumps and piping to the sewer system). Constructing a 40 ft. by 60 ft. building would allow for the installation of an additional filter(s) in the future to treat the Sharon Tucker well water if the town pursues this water source. The estimated construction cost as it relates specifically to this alternative is $1,137,500. The estimated annual O&M cost is approximately $37,000/year. These costs include chemical, labor, heat, energy/power, media replacement, general maintenance costs and residuals handling. Based on amortizing the installed capital costs over 30 years at 4-percent interest rate and including the annual O&M costs (with an average 2.5% inflation rate), the total cost of treated water is approximately $1.51/1,000 gallons for the pressure filtration system alone.

6.3 Ion Exchange Rental (Temporary)

If trailers with ion exchange vessels are used, the cost would be approximately $62,500 per month ($750,000/year) to treat 150 to 175 gpm of MacIntosh well water for 12 hours per day. The resulting short-term rental cost to produce potable water via the ion exchange process alone is estimated to be $9.51/1,000 gallons for one year. We assumed that one of the proposed WTP sites described in section 4.4 would be used to locate the trailers. As a result, all capital costs presented in table 7.1 would be applicable to this alternative.

6.4 Ion Exchange Permanent

The permanent ion exchange treatment system including chemical storage, chemical feed and a neutralization system would cost approximately $1,255,000 for the equipment, approximately $615,000 to install and approximately $545,000 for a 40 ft. by 60 ft. building, process piping, residuals holding tank (plus the cost of residuals pumps and piping to the sewer system). Constructing a 40 ft. by 60 ft. building would allow for the expansion of the ion exchange system in the future to treat the Sharon Tucker well water. The estimated construction cost as it relates specifically to this alternative is $2,415,000. The estimated annual O&M cost is approximately $300,000/year for chemicals and $50,000/year for power, labor, and consumables (12 hours per day). Based on amortizing the installed capital costs over 30 years at 4-percent interest rate and including the annual O&M costs (with an average 2.5% inflation rate), the total cost of treated water is approximately $8.25/1000 gallons for the ion exchange treatment system alone.

6.5 Electrodialysis Reversal

A single EDR treatment train is estimated to cost approximately $375,000 while the installation of the EDR system would be approximately 25 percent of the equipment cost (or $93,750 for the single train system). The cost to furnish a 40 ft. by 60 ft. building, chemical feed equipment, process piping, below grade residuals holding tank (plus the cost of blasting, residuals pumps and piping to the sewer system) is estimated to be $675,000. Constructing a building of this size would allow for the installation of a second EDR system in the future to treat the Sharon Tucker well water. The estimated construction cost as it relates specifically to this alternative is $1,143,750. The estimated annual operation and maintenance (O&M) cost for the EDR system is $66,300/year including labor, chemicals, consumables (membrane and electrode replacements), and electric power and operation of 12 hours per day average. Based on amortizing the installed capital costs over 30 years at 4-percent interest rate and including the annual EDR O&M costs (with an average 2.5% inflation rate), the total cost of treated water is approximately $2.06/1,000 gallons for the single process train EDR system alone.

7.0 PRELIMINARY COST SUMMARY

7.1 Common Capital Costs Summary

In section 4.5, we outlined the costs to provide power to the MacIntosh Well and to a new WTP at the intersection of Hersey Lane and Durell Drive. For budgetary purposes, we will include the Option 2 cost, as presented in Table 4.3, as a common capital cost. In addition to the electrical costs, the following table presents costs that are common to all alternatives evaluated as discussed in Chapter 4.

Table 7.1 – Common Capital Costs

Electrical Service – Option 2, Table 4.3	$216,500
MacIntosh well development, site work (including stream crossing) and instrumentation to tie into existing system	$200,000
Raw water main between MacIntosh Well and WTP (assumed to be 10" HDPE). Includes horizontal directional drilling across wetland and rock excavation in water main trench.	$210,000
Proposed WTP site work including clearing and grubbing, yard piping and tie-in to distribution system	$125,000
Total	**$751,500**

7.2 Project Cost Comparison

Table 7.2 presents a cost summary of the different alternatives presented in Chapter 6. The last column of Table 7.2 is an estimate of the total cost of the treated water for each alternative listed in the table. The total cost is based on amortizing the water treatment specific capital costs over 30 years at a 4-percent interest rate and including the annual O&M costs with a 2.5-percent inflation rate on average. The cost per 1,000 gallons of water produced assumes each alternative operates for 12 hours a day on average for 30 years.

Table 7.2 – Project Cost Summary

Treatment Alternative	Common Capital Cost	Treatment Specific Capital Cost	Engineering (20%)	Contingency (10%)	Total Project Capital Cost	O&M	Total Cost of Treated Water
Blending Water	$751,500	$640,000*	$280,000	$140,000	$1,811,500*	$30,000/yr	$0.89/1,000 gal.
Pressure Filtration	$751,500	$1,137,500	$380,000	$190,000	$2,459,000	$37,000/yr	$1.51/1,000 gal.
Ion Exchange Rental (Temporary)	$751,500	---	$150,000	$75,000	$976,500	$750,000/yr	$9.51/1,000 gal.**
Ion Exchange Permanent	$751,500	$2,415,000	$635,000	$317,500	$4,119,000	$350,000/yr	$8.25/1,000 gal.
Electrodialysis Reversal (EDR)	$751,500	$1,143,750	$380,000	$190,000	$2,465,250	$66,300/yr	$2.06/1,000 gal.

* Includes the cost to replace 6-inch water main on Main Street
** Treatment cost is for a one year rental

P:\Newmarket\MacIntosh Well\Preliminary Design Report\Draft Report.doc

Weston & Sampson	PROJECT Newmarket, NH	REPORT OF BORING No. B-1 SHEET 1 OF 2 Project No. CHKD BY

BORING Co. New Hampshire Boring
FOREMAN Steve/Jack
WSE GEOLOGIST: Emily Faivre

BORING LOCATION See attached plan
GROUND SURFACE ELEV. DATUM
DATE START 8/6/10 DATE END 8/6/10

SAMPLER: Emily Faivre. No samples collected for laboratory analysis
Samples collected in jars.
CASING:
CASING SIZE: Method Drive-and-wash

GROUNDWATER READINGS

DATE	TIME	WATER AT	CASING AT	STABILIZATION TIME
8/6/2010	725			

DEPTH (feet)	CASING (lb/ft)	SAMPLE No.	PEN/REC (in)	DEPTH (ft)	BLOWS/6"	PID (ppm)	SAMPLE DESCRIPTION Burmister Classification	NOTES	STRATUM DESCRIPTION
			24/21	0-2	3-4-3-4	-	0-4" Dark brown sandy LOAM 4-16" Brown fine to medium SAND and SILT 16-18" Dark brown fine to medium SAND and SILT 18-21" Gray brown fine to medium SAND and SILT		
5									
			24/14	4-6	10-9-10-13	-	0-8" Gray brown fine to medium SAND 8-9" Gray SILT trace CLAY 9-14" Gray fine to medium SAND		entire spoon - wet
10									
			24/12	9-11	4-4-5-4	-	0-11" Gray fine SAND little medium SAND 11-12" Gray medium to coarse SAND		
			24/13	14-16	3-3-6-8	-	0-6" Gray fine to medium SAND trace SILT 6-7" Gray fine SAND and SILT 7-11" Gray fine to medium SAND and SILT 11-13" Gray fine to medium SAND		
15									
20			24/21	19-21	2-1-3-3	-	0-4" Gray fine to medium SAND 4-6" Gray fine SAND and SILT 6-10" Gray fine SAND and SILT trace CLAY 10-12" Gray SILT and CLAY 12-21" Gray SILT and CLAY trace fine SAND		
25									
			24/21	24-26	3-5-5-3	-	0-4" Gray fine to coarse SAND 4-8" Gray SILT and fine SAND 8-9" Gray fine SAND trace SILT 9-11" Gray clay 11-16" Gray fine to medium SAND 16-17" Gray SILT and CLAY trace fine SAND 17-21" Gray fine SAND and SILT		
30									
35									

GRANULAR SOILS		COHESIVE SOILS	
BLOWS/FT	DENSITY	BLOWS/FT	DENSITY
0-4	V. LOOSE	0-2	V. SOFT
4-10	LOOSE	2-4	SOFT
10-30	M. DENSE	4-8	M. STIFF
30-50	DENSE	8-15	STIFF
> 50	V. DENSE	15-30	V. STIFF
		> 30	HARD

REMARKS:

NOTES:
1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THIS BORING LOG. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS ARE MADE.

BORING No. B-1

Weston & Sampson	PROJECT Newmarket, NH	REPORT OF BORING No. B-1 SHEET 2 OF 2 Project No. CHKD BY

BORING Co. New Hampshire Boring
FOREMAN Steve/Jack
WSE GEOLOGIST: Emily Faivre

BORING LOCATION See attached plan
GROUND SURFACE ELEV. DATUM
DATE START 8/6/10 DATE END 8/6/10

SAMPLER: Emily Faivre. No samples collected for laboratory analysis
Samples collected in jars.
CASING:
CASING SIZE: Method Drive-and-wash

GROUNDWATER READINGS

DATE	TIME	WATER AT	CASING AT	STABILIZATION TIME
8/6/2010	725			

DEPTH (feet)	CASING (lb/ft)	SAMPLE No.	PEN/REC (in)	DEPTH (ft)	BLOWS/6"	PID (ppm)	SAMPLE DESCRIPTION Burmister Classification	NOTES	STRATUM DESCRIPTION
			24/19	29-31	5-4-6-7	-	0-9" Gray medium to coarse SAND 9-15" Gray fine SAND 15-19" Gray SILT and CLAY trace fine SAND		
5			24/22	34-36	2-1-1-4	-	0-4" Gray SILT and CLAY 4-17" Gray CLAY trace SILT 17-18" Gray fine SAND 18-19" Gray CLAY 19-22" Gray fine SAND and SILT		
10			24/24	39-41	2-2-WR/12*	-	0-2" Gray coarse SAND 2-24" Gray CLAY		
15									
20									
25									
30									
35									

GRANULAR SOILS BLOWS/FT	DENSITY	COHESIVE SOILS BLOWS/FT	DENSITY
0-4	V. LOOSE	0-2	V. SOFT
4-10	LOOSE	2-4	SOFT
10-30	M. DENSE	4-8	M. STIFF
30-50	DENSE	8-15	STIFF
> 50	V. DENSE	15-30	V. STIFF
		> 30	HARD

REMARKS:
* Weight of rod for 12"
Tie-offs: Approximately 50' to confluence of wetland and stream
Approximately 59' to monitoring well

NOTES:
1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THIS BORING LOG. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS ARE MADE.

BORING No. B-1

Weston & Sampson	PROJECT Newmarket, NH	REPORT OF BORING No. B-2 SHEET 1 OF 1 Project No. CHKD BY

BORING Co. New Hampshire Boring
FOREMAN Pete/Jack
WSE GEOLOGIST: Emily Faivre

BORING LOCATION See attached plan
GROUND SURFACE ELEV. DATUM
DATE START 8/6/10 DATE END 8/6/10

SAMPLER: Emily Faivre. No samples collected for laboratory analysis
Samples collected in jars.
CASING:
CASING SIZE: Method Drive-and-wash

GROUNDWATER READINGS

DATE	TIME	WATER AT	CASING AT	STABILIZATION TIME
8/6/2010	1015			

DEPTH (feet)	CASING (lb/ft)	SAMPLE No.	PEN/REC (in)	DEPTH (ft)	BLOWS/6"	PID (ppm)	SAMPLE DESCRIPTION Burmister Classification	NOTES	STRATUM DESCRIPTION
			24/18	0-2	3-1-6-11	-	0-1" Black topsoil 1-3" Light brown fine SAND 3-6" Brown fine SAND trace SILT 6-11" Gray brown fine SAND and SILT 11-15" Gray fine SAND and SILT 15-18" Gray fine SAND		
5			24/22	4-6	5-7-8-11	-	0-22" Gray-brown CLAY trace SILT		
			24/--	9-11	Refusal	-			
10									
15									
20									
25									
30									
35									

GRANULAR SOILS BLOWS/FT	DENSITY	COHESIVE SOILS BLOWS/FT	DENSITY
0-4	V. LOOSE	0-2	V. SOFT
4-10	LOOSE	2-4	SOFT
10-30	M. DENSE	4-8	M. STIFF
30-50	DENSE	8-15	STIFF
> 50	V. DENSE	15-30	V. STIFF
		> 30	HARD

REMARKS:
Refusal - 9'2"

B-2b: Moved 5' up road (toward farm, away from wetland), refusal at 8'
B-2c: Moved 10' up road, refusal at 7.5'
B-2d: Moved 10' up road, refusal at 6'

See figure for tie-offs.

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THIS BORING LOG. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS ARE MADE.

BORING No. B-2

Weston & Sampson	PROJECT Newmarket, NH	REPORT OF BORING No. Ledge Probes SHEET 1 OF 1 Project No. CHKD BY

BORING Co. New Hampshire Boring
FOREMAN Pete/Jack
WSE GEOLOGIST: Emily Faivre

BORING LOCATION See attached plan
GROUND SURFACE ELEV. DATUM
DATE START 8/6/10 DATE END 8/6/10

SAMPLER: Emily Faivre. No samples collected for laboratory analysis
Samples collected in jars.
CASING:
CASING SIZE: Method Drive-and-wash

GROUNDWATER READINGS

DATE	TIME	WATER AT	CASING AT	STABILIZATION TIME

DEPTH (feet)	CASING (lb/ft)	SAMPLE No.	SAMPLE PEN/REC (in)	SAMPLE DEPTH (ft)	SAMPLE BLOWS/6"	PID (ppm)	SAMPLE DESCRIPTION Burmister Classification	NOTES	STRATUM DESCRIPTION
							LP-1 (to 10'): Refusal at 8' Location: 10' uphill (barn side) from lowest point in road along ditch. In middle of road		
5									
							LP-2 (to 10'): Refusal at 2.5', 2.5', 8.5' 2x attempts at one location (~1' apart) Move uphill 5' (toward barn), third refusal Location: In road, at stake w/ yellow flag, where existing road meets stakes at 45°		Bent rod
10									
							LP-3 (to 10'): Refusal at 9'. Location: At corner of plowed field near edge of woods (see figure)		
15									
							LP-4 (to 10'): Refusal at 4'. Location: Approximately 80' from edge of Durell Dr. 2.5' uphill from tree stump in middle of road (almost at grade).		
20									
							LP-5 (to 10'): Refusal at 3'. Location: At top of hill - Hersey Road Durell Road visible. Location in road at curve approximately 70' uphill from LP-4.		Bent rod
25									
							LP-6 (to 10'): No refusal. Location: Below large bedrock outcrop and intersection with old road loop off Hersey. 43' uphill from lower intersection.		
30									
							LP-7 (to 10'): Refusal at 3.5'. Location: 20' uphill from "DSI 14 HUBS" yellow marker. 150 yards uphill from corner/clearing.		
35									

GRANULAR SOILS BLOWS/FT	GRANULAR SOILS DENSITY	COHESIVE SOILS BLOWS/FT	COHESIVE SOILS DENSITY
0-4	V. LOOSE	0-2	V. SOFT
4-10	LOOSE	2-4	SOFT
10-30	M. DENSE	4-8	M. STIFF
30-50	DENSE	8-15	STIFF
> 50	V. DENSE	15-30	V. STIFF
		> 30	HARD

REMARKS:

NOTES:
1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THIS BORING LOG. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS ARE MADE.

BORING No. Ledge Probes

Table 6-2 Weighted Criteria Matrix

	Criterion	Safety		Ability to Supply Water		Potential Critical Failure and/or Redundancy		Critical Customer Impact		Fire Flow		Cost		Water Quality Impacts		Hydraulic Priority		Other Project Coordination		Enviornmental Impact and Permitting			
	Weight	7		6		6		6		5		4		3		3		2		1		Total	
Project ID	Description	Cost (Rounded)	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Score	Rank
St1	New Water Storage Tank	$ 1,370,000	1	7	5	30	4	24	1	6	5	25	2	8	3	9	5	15	1	2	3	3	129	1
F1	Bennett Pump Station Upgrade	$ 110,000	5	35	3	18	4	24	1	6	1	5	5	20	1	3	2	6	1	2	5	5	124	2
F2	Sewall Pump Station Upgrade	$ 110,000	5	35	3	18	4	24	1	6	1	5	5	20	1	3	2	6	1	2	5	5	124	2
S1	New Well Development (Tucker)	$ 1,400,000	1	7	5	30	5	30	1	6	3	15	2	8	1	3	5	15	1	2	3	3	119	4
F3	Water Storage Tank Vault Improvements	$ 30,000	5	35	3	18	3	18	1	6	1	5	5	20	1	3	2	6	1	2	5	5	118	5
P2	South Main St (to Gerry Ave) & Creighton St	$ 180,000	1	7	1	6	1	6	5	30	4	20	5	20	1	3	5	15	1	2	5	5	114	6
P1	South Main St (Between Wadleigh Falls Rd & Railroad St)	$ 990,000	1	7	1	6	1	6	5	30	4	20	3	12	1	3	5	15	1	2	5	5	106	7
P6	Cross-country main from Great Hill Tank to Rt. 108	$ 210,000	1	7	4	24	1	6	1	6	4	20	5	20	1	3	5	15	1	2	3	3	106	7
P5	Exeter Rd (Great Hill Tank to car wash)	$ 625,000	1	7	4	24	1	6	1	6	4	20	4	16	1	3	5	15	1	2	5	5	104	9
P8	Exeter Rd (Great Hill Tank to Forbes Rd)	$ 791,000	1	7	1	6	1	6	5	30	4	20	4	16	1	3	3	9	1	2	3	3	102	10
Wq1	Uni-directional flushing program	$ 20,000	1	7	2	12	1	6	1	6	1	5	5	20	5	15	4	12	1	2	5	5	90	11
P10	Beech St Extension (to Pine St)	$ 306,000	1	7	1	6	1	6	1	6	4	20	5	20	3	9	3	9	1	2	5	5	90	11
P11	Hersey Lane & McIntosh	$ 1,776,000	1	7	1	6	1	6	5	30	4	20	1	4	1	3	3	9	1	2	3	3	90	11

		Criterion	Safety		Ability to Supply Water		Potential Critical Failure and/or Redundancy		Critical Customer Impact		Fire Flow		Cost		Water Quality Impacts		Hydraulic Priority		Other Project Coordination		Environmental Impact and Permitting			
		Weight	7		6		6		6		5		4		3		3		2		1		Total	
Project ID	Description	Cost (Rounded)	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Score	Rank
	Well Connector																							
P3	Packer Falls Rd (South Main St to Elm St)	$ 641,000	1	7	1	6	1	6	1	6	4	20	4	16	1	3	5	15	1	2	5	5	86	14
P4	Elm St	$ 663,000	1	7	1	6	1	6	1	6	4	20	4	16	1	3	5	15	1	2	5	5	86	14
P7	Grant Rd (Wadleigh Falls Rd to Brialla Circle)	$ 465,000	1	7	1	6	1	6	1	6	4	20	4	16	1	3	5	15	1	2	5	5	86	14
P12	North Main St (Sanborn to Dame and Bay Rd)	$ 281,000	1	7	1	6	1	6	1	6	4	20	5	20	1	3	3	9	1	2	5	5	84	17
P14	Pine St	$ 128,000	1	7	1	6	1	6	1	6	4	20	5	20	1	3	3	9	1	2	5	5	84	17
P15	New Rd	$ 281,000	1	7	1	6	1	6	1	6	4	20	5	20	1	3	3	9	1	2	5	5	84	17
P16	Bay Rd and Dame Rd	$ 293,000	1	7	1	6	1	6	1	6	4	20	5	20	1	3	3	9	1	2	5	5	84	17
P17	Creighton St	$ 143,000	1	7	1	6	1	6	1	6	4	20	5	20	1	3	3	9	1	2	5	5	84	17
P18	Route 108 (Bay Rd to Simons Lane)	$ 236,000	1	7	1	6	1	6	1	6	4	20	5	20	1	3	3	9	1	2	5	5	84	17
P26	Route 108 (Bay Rd to Simons Lane)	$ 408,000	1	7	1	6	1	6	1	6	4	20	4	16	1	3	1	3	4	8	5	5	80	23
P13	Dame Rd, New Rd, and Lamprey St	$ 689,000	1	7	1	6	1	6	1	6	4	20	4	16	1	3	3	9	1	2	5	5	80	23
P22	New subaqueous crossing downstream of Rt 108 Bridge	$ 135,000	1	7	1	6	1	6	1	6	4	20	5	20	1	3	3	9	1	2	1	1	80	23
P28	Second river crossing to Durham	$ 294,000	1	7	1	6	1	6	5	30	1	5	4	16	1	3	1	3	1	2	1	1	79	26
P23	Ham St	$ 102,000	1	7	1	6	1	6	1	6	4	20	5	20	1	3	1	3	1	2	5	5	78	27
P24	Church St (between Rock and Granite	$ 89,000	1	7	1	6	1	6	1	6	4	20	5	20	1	3	1	3	1	2	5	5	78	27

		Criterion	Safety		Ability to Supply Water		Potential Critical Failure and/or Redundancy		Critical Customer Impact		Fire Flow		Cost		Water Quality Impacts		Hydraulic Priority		Other Project Coordination		Enviornmental Impact and Permitting			
		Weight	7		6		6		6		5		4		3		3		2		1		Total	
Project ID	Description	Cost (Rounded)	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Raw Score	Wtd Score	Score	Rank
	Streets)																							
P25	New Road	$ 207,000	1	7	1	6	1	6	1	6	4	20	5	20	1	3	1	3	1	2	5	5	78	27
P21	Lang's Lane	$ 1,632,000	1	7	1	6	1	6	1	6	4	20	1	4	4	12	3	9	1	2	5	5	77	30
P9	Spring St Area (Spring, Central, Chapel, Rock, and Church Streets)	$ 956,000	1	7	1	6	1	6	1	6	4	20	3	12	1	3	3	9	1	2	5	5	76	31
P20	Exeter Rd (south of Forbes Rd)	$ 982,000	1	7	1	6	1	6	1	6	4	20	3	12	1	3	3	9	1	2	5	5	76	31
M1	Leak detection program	$ 5,000	1	7	3	18	1	6	1	6	1	5	5	20	1	3	1	3	1	2	5	5	75	33
St2	Paint Existing Water Storage Tank & Install mixer	$ 416,000	1	7	1	6	1	6	1	6	1	5	4	16	5	15	1	3	1	2	5	5	71	34
P27	Dame Rd	$ 102,000	1	7	1	6	1	6	1	6	1	5	5	20	1	3	1	3	1	2	5	5	63	35
Ma1	Water GIS Mapping	$ 5,000	1	7	1	6	1	6	1	6	1	5	5	20	1	3	1	3	1	2	5	5	63	35
P29	Forbes Rd Extension	$ 2,206,000	1	7	1	6	1	6	1	6	1	5	1	4	4	12	1	3	1	2	5	5	56	37

Scales:

Criteria Weight: 1 Low to 4 high

Ranking: 1 Low to 3 med to 5 high

P	Piping	S	Supply	M	Maintenance	Wq	Water Quality
F	Facilities	St	Storage	Ma	Management		

Water q Quality