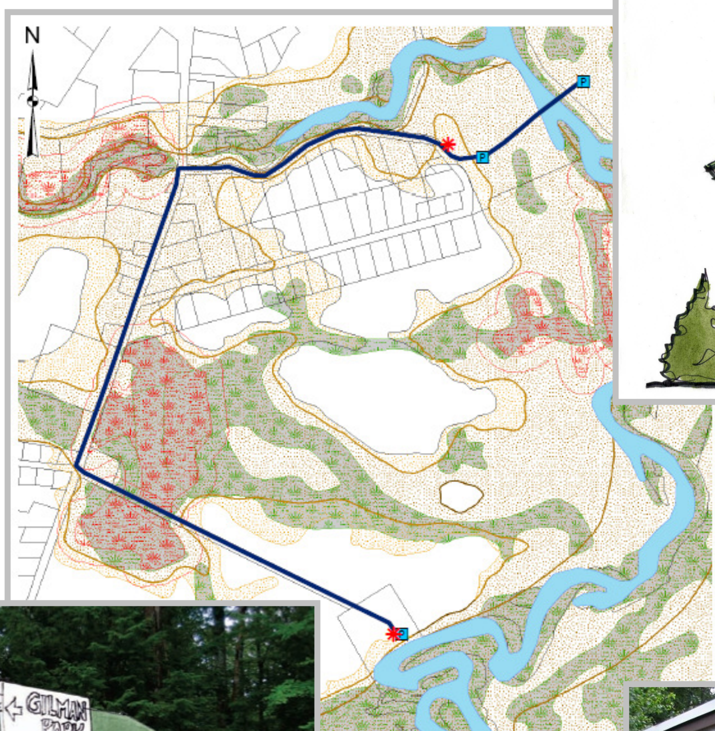
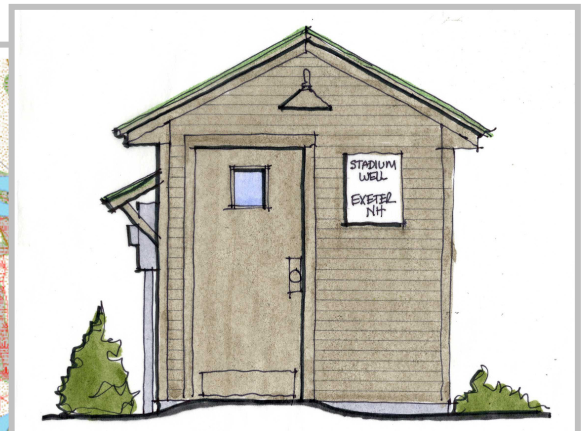
Town of Exeter, New Hampshire

Groundwater System Preliminary Design Report

May 2011 (DRAFT)





Weston&Sampson®

Town of Exeter, New Hampshire

Groundwater System Factsheet

As Presented During the Town of Exeter's Deliberative Session

February 5, 2011

Article 12
Groundwater System & Treatment Facility

DESCRIPTION

Article 12 on the 2011 warrant would raise and appropriate $6,350,000 for the design and construction of new groundwater system and treatment facility.

After a multi-year, thorough review of water deficiency issues facing the Town and investigating water supply alternative solutions, the Board of Selectmen, Town Manager, and Public Works Department have reorganized the Town's Water Capital Improvement Plan to include a recommendation for implementing the Groundwater System project this year. Warrant Article 12 is another step in implementing solutions to the Town's long-range water supply improvement plan.

These improvements will allow the Town to further diversify its water supply sources and improve overall water quality in the system. These improvements include upgrades to the Lary Lane Well, equipping the Gilman Park and Stadium Wells and constructing a new Groundwater Treatment Facility to be located either at Gilman Park or adjacent to the Lary Lane Well.

BACKGROUND

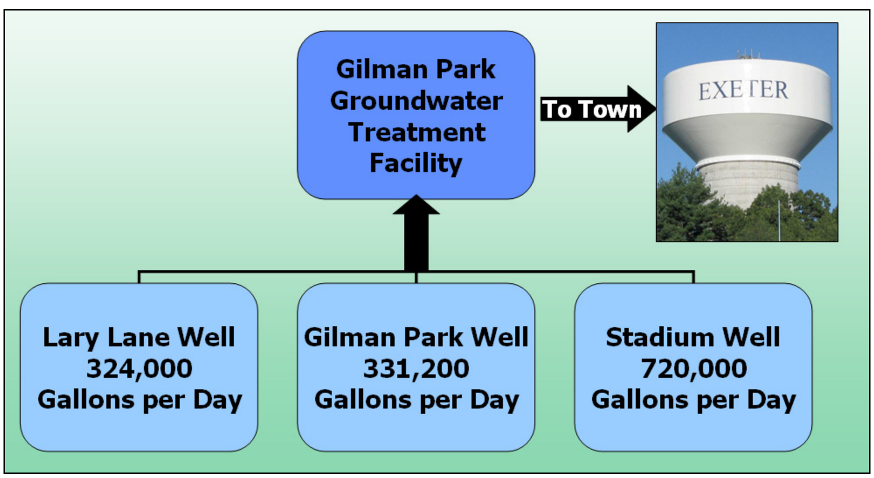
The Town conducted a detailed study in 2007 to further explore the groundwater options. Following extensive research of available sites it was recommended to the Board of Selectmen in 2008 that two inactive sites, the Stadium and Gilman Park wells, be rehabilitated and reactivated. That work was approved and the following items have been accomplished to date:

- **Spring 2009** – Rehabilitation of the Stadium and Gilman Park wells
- **July 2009** – Pumping test and water treatment piloting of Stadium, Gilman Park and Lary Lane wells
- **2009/2010** – Data analysis and submittal of final pumping test and piloting reports to the New Hampshire Department of Environmental Services (DES)
- **2009/2010** – Water Supply Options Study which included an analysis of potential operational cost savings of approximately $50,000 to $100,000 a year by using more groundwater supply versus surface water to meet the Town's water demands
- **2010** – Preliminary design of well and groundwater treatment facility construction
- **November 1, 2010** – Town receives letter from DES approving reactivation of the Stadium and Gilman Park wells

THE WELLS

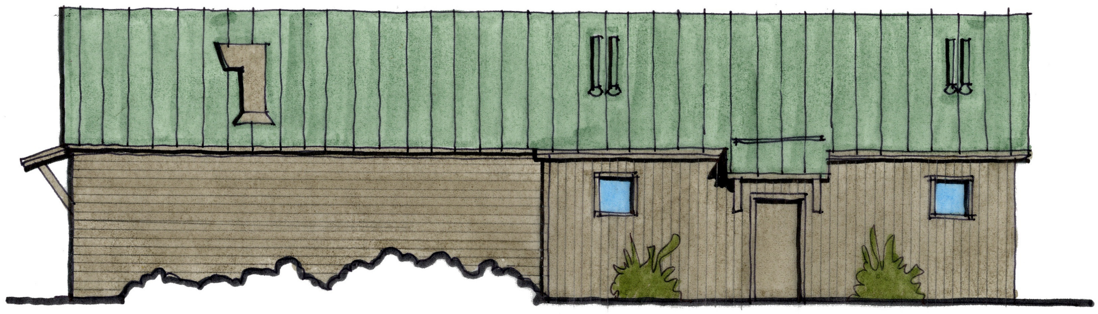
Existing Lary Lane and Gilman Park Wells – Stadium Well Prior to Rehabilitation

The Town's water system relied on groundwater from the Gilman Park, Lary Lane and Stadium wells from the 1950's until 1974 when they shifted over to the surface water system. The Lary Lane well remained in service and has been used as a supplemental water source since that time. Changes to regulations lowered the acceptable level of arsenic in this well from 50 parts-per-billion (ppb) to 10 ppb. This change has warranted the need for treatment of the well. The Stadium and Gilman Park wells require treatment for high levels of iron and manganese, therefore, the intent is to construct pipelines to combine all three wells and treat them at a new groundwater treatment facility. The following schematic shows the wells, their rated maximum daily flow and the proposed groundwater treatment facility:



As the schematic shows, the total system capability for the three combined wells is 1.375 million gallons a day. This volume is more than the Town's average day of water demand, which is approximately 1.1 million gallons, but less than the peak demand of approximately 1.8 million gallons. Therefore, the intent of the new groundwater system is to blend the groundwater with the surface water and manage all of the sources in a sustainable and integrated manner in order to optimize all the sources.

PROPOSED GROUNDWATER TREATMENT SYSTEM



Architects Rendering of Proposed Groundwater Treatment – TMS Architects

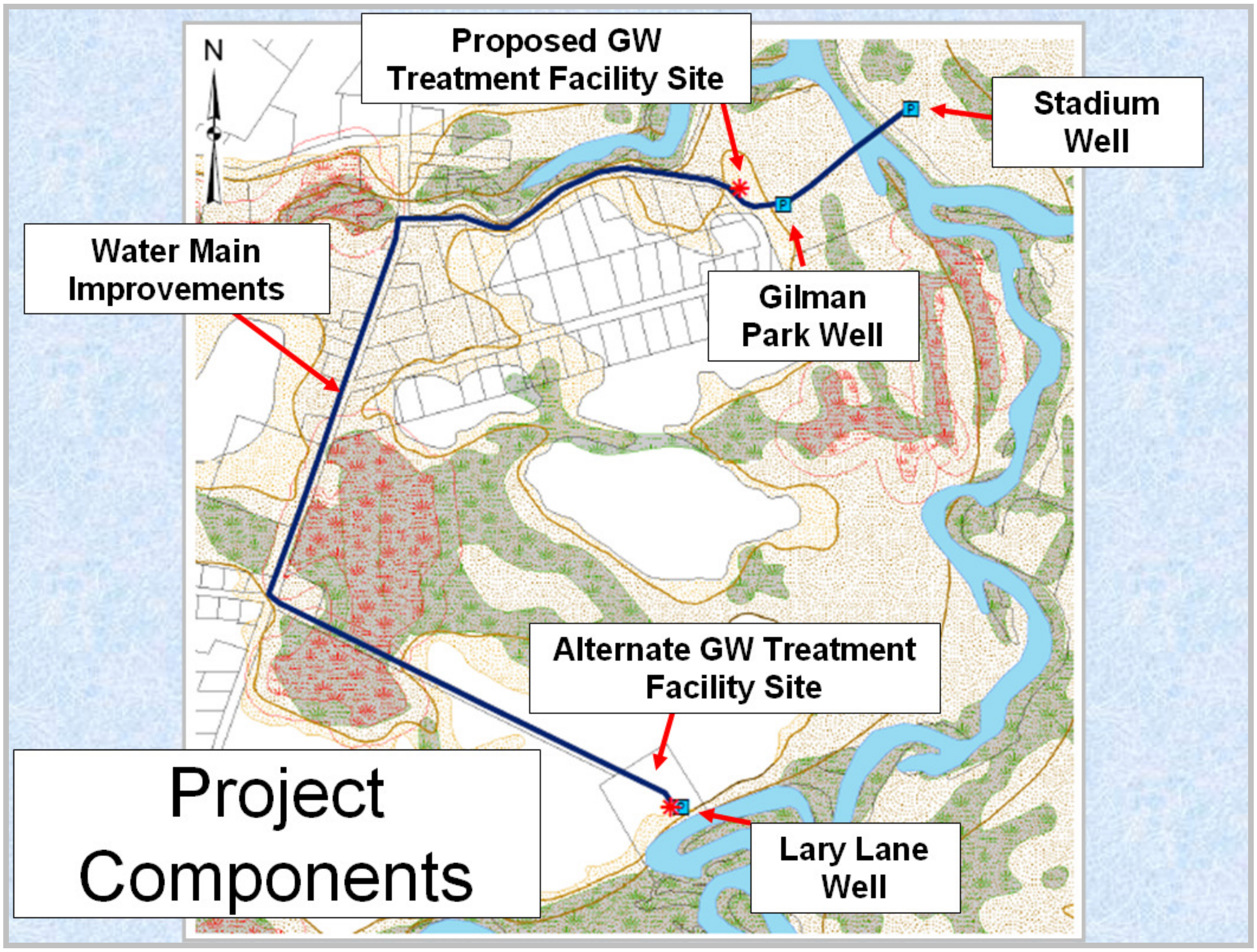
After reviewing the groundwater treatment pilot findings and considering site and operational issues Weston & Sampson has developed a preliminary design of groundwater treatment (GWT) to include:

- Construction of a new GWT facility located at the former volleyball court at Gilman Park. The facility would be designed to be:
    - An approximate footprint of 30 feet x 80 feet (2400 square feet of interior space).
    - Built to be as energy and environmentally efficient as possible, including porous pavement, rain gardens and other "green" building components.
    - Incorporated into the park-like atmosphere of the Gilman Park, and possibly have the construction coincide with the construction of a pavilion to serve users of the park. This concept has already been brought forth through the preliminary architectural design of the facilities
- Four (4) new 8-foot diameter pressure filters and filter media with Greensand Plus.
    - These filters would be capable of producing 250 gallons per minute (GPM) of flow at a loading rate of 5.0 gpm/square foot (higher rates may be possible). With all four filters this provides a design flow rate of 1.44 million gallons a day.
    - Provisions to add two additional filters at a later date to increase capacity of system to 2.16 million gallons a day.
- An alternate site, located adjacent to the Lary Lane Well, is available for the purpose of locating this facility if the Gilman Park site is not feasible.

THE PROPOSED PROJECT

- Renovate and equip Gilman Park Well pump house, build and equip Stadium Well pump house. Both sources were recently approved by NHDES for reactivation as water supply sources at a combined flow of approximately 1 million gallons per day.
- Renovate the Lary Lane Well to allow treatment for arsenic removal. This well has a daily flow of approximately 324,000 gallons per day.
- Install new transmission water mains to manifold the three wells and upgrade distribution water mains in the area of Gilman Park.

- Construct a new groundwater treatment facility to remove iron, manganese and arsenic from the source water. This facility will be design to handle the maximum flow from the three combined wells, plus have room for expansion if necessary in future years.



PROBLEMS ADDRESSED:

- Intermittent violations of the arsenic standard (lowered from 50 to 10 parts per billion) for the Lary Lane Well will be solved by blending with the other two wells and treating at the groundwater treatment facility.
- Current violations of the Total Trihalomethane (TTHM) standard will be partly addressed by the blending of treated groundwater into the water system.
- Two sources of supply which have been idle for over 35 years will be reactivated and utilized by the Town in an integrated manner.

BENEFITS:

- This is a less expensive capital project than building an entirely new surface water treatment facility.
- The operations and maintenance of the groundwater system is also less expensive than surface water, about half the electrical and chemical costs of what surface water costs to produce.
- The source water is of higher and more consistent quality than surface water.
- The state Department of Environmental Services is likely to limit the amount of withdrawals the town can make from the Exeter River, the town's main water source, in the future.
- Having multiple sources will provide more security in the event of a drought or if something goes wrong with the surface water supplies.
- If approved, the Town is currently in line for receiving some bonding forgiveness from the state's Revolving Loan fund, which is estimated to amount anywhere from $1.0 to $1.5 million.

FINANCING:

The project costs include Well Improvements $775,000; Water Transmission and Distribution Main Improvements $1,093,000; and Groundwater Treatment Facility $4,482,000. Total project cost $6,350,000 will be financed through the following sources:

- Ratepayer fees of 4.88 to 5.33 million, depending on SRF principal forgiveness.
- The Town will issue a bond anticipation note to complete the design phase of the project and bond the remainder of the project when bids are received on the construction phase. The length of the bond will be 20 years.
- The Town will seek a State Revolving Fund (SRF) loan at more favorable interest rates than the general bond market. SRF loans are granted through the New Hampshire DES for qualifying water and wastewater projects. This project is ranked high on the list of DES's SRF water projects and will qualify to receive 30% forgiveness on a percentage of the loan.

WATER RATE IMPACT:

This project will be funded entirely by water users. The total obligation is estimated to be $4.88 to $5.33 Million. The impact for the average residential ratepayer portion of the project is estimated to be $4.36 to $4.77 per month. This translates to $52 to $57 per year.

PROPOSED TIMELINE:

2011 - 2012 – Final design
2012 - 2013 – Bidding, Construction and Startup

Town of Exeter, New Hampshire
Groundwater Supply System
Preliminary Design Report
May 2011 (draft)

T A B L E O F C O N T E N T S

A P P E N D I X E S

A P P E N D I X A - F I G U R E S

1. Introduction and Background

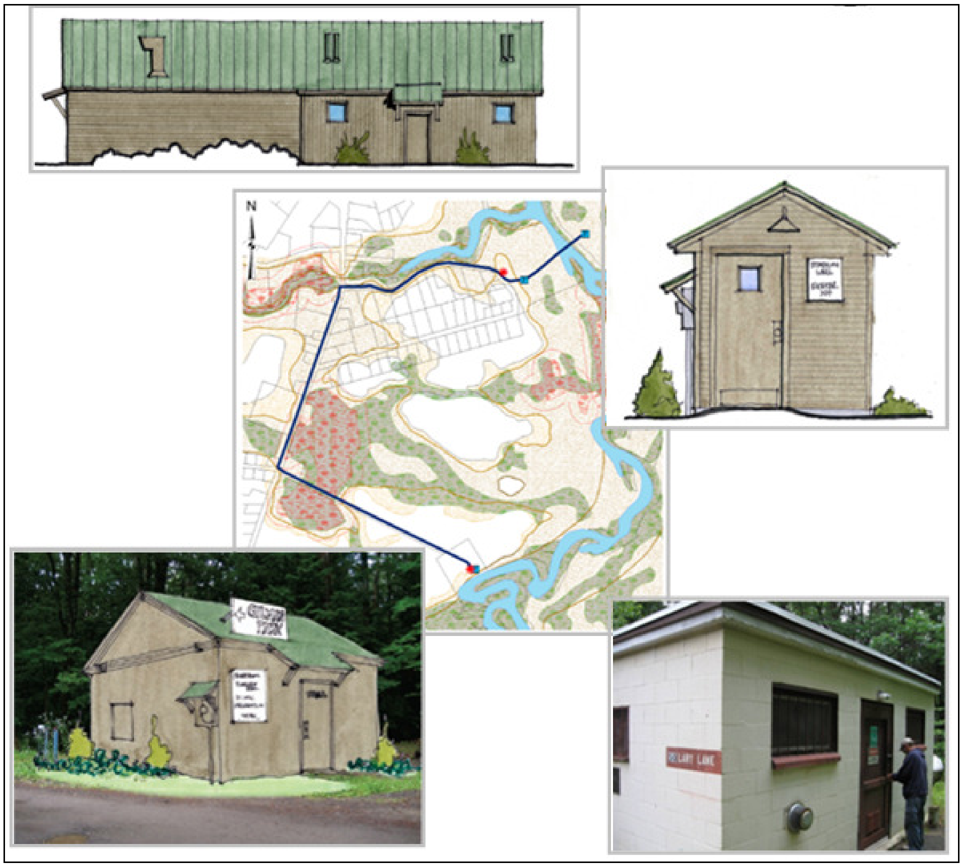


Figure 1.1 – Map of Project Area and Sketches of Project Components

Weston & Sampson has been working with the Town of Exeter, New Hampshire since 2007 toward developing long-range water system upgrades which include the potential for the reactivation and treatment of the Stadium and Gilman Wells. Rehabilitation of the wells along with a pumping test and piloting for treatment was performed during the summer of 2009.

Piloting of groundwater treatment for the Gilman and Stadium occurred during pumping tests of each well in July 2009 and the Lary Lane well thereafter. Gilman and Stadium had similar water quality. Both had iron concentrations typically above 1 mg/L, and a pH near 7. Lary Lane had iron concentrations of approximately 0.1 mg/L, and a pH near 8. These characteristics required different approaches to arsenic treatment at Gilman/Stadium versus Lary Lane. The pilot study tested three different granular media: GreensandPlus™, Pureflow© PM-200, and LayneOx™. All were found to be effective for iron, manganese and arsenic removal. Effluent water quality was comparable for all three filtration media. The high-iron sources, Gilman and Stadium, were effectively treated at Filter Surface Loading Rates from 2.5 to 5 gpm/sf. Higher rates were likely possible, but were not piloted.

The piloting results showed that all of the three medias tested perform satisfactorily with regards to iron, manganese and arsenic removal for all three source waters. The Gilman and Stadium wells required chlorination to oxidize the iron and arsenic with absorption of manganese also occurring during the filtering process. The Lary Lane Well required chlorination and the addition of Ferric chloride since its source water was low in natural iron. It is assumed that combining the higher iron Gilman/Stadium sources with the Lary Lane well and blending them prior to treatment would eliminate the need for the addition of ferric.

Based on this work, Weston & Sampson made the following recommendations to the Town:

- Pursue the reactivation of the Gilman and Stadium wells (with a combined yield of approximately 1.0 MGD)
- Manifold these wells together with the Lary Lane well (yield of approx. 0.25 MGD)
- Design and construct an iron-manganese pressure filtration system capable of treating 1.5 MGD with the ability to expand if other groundwater sources are added at a later date.

Separate reports have been prepared and submitted to the Town and the New Hampshire Department of Environmental Services (DES) with the results of the 2009 pumping test for the reactivation of the Stadium and Gilman wells and groundwater treatment piloting results. Based on the reports submitted to the NHDES they subsequently issued an approval for the reactivation of these two wells on November 1, 2010. This report summarizes the preliminary design parameters for the following items related to the construction of a groundwater treatment system for the Town:

1. Develop conceptual design plans for reactivating the Gilman and Stadium wells:
   - Site improvements, including floodplain issues.
   - Preliminary sizing of pumping equipment, electrical loads and process control.
   - Determine pipe size and routes in order to manifold the Gilman, Stadium and Lary Lane wells together.
2. Develop basis of groundwater treatment design plans, sizing and functional operation descriptions for submittal to DES for the groundwater treatment system:
   - Sizing and selection of chemical feed systems
   - Sizing and selection of process piping and valves
   - Loading rates and preliminary filtration design
   - Conceptual building layout and materials
3. Concurrently with item 4, determine final site layout, footprint and location of groundwater treatment. The current sites under consideration include:
   - Former volleyball court in Gilman Park.
   - Basketball court in Gilman Park (This site might be coordinated with the Town's potential construction of a pavilion for the Gilman Park).
   - Lary Lane well site. (This site would require discussions with PEA)
4. Finalize cost estimates to present for budgets in the fall of 2010 so that warrant articles can be voted on in March 2011 for final design, bidding and construction of facilities.
5. Work with Town officials to develop a public outreach effort in order to gain support of the project.
   - Artist rendering of potential buildings and site improvements. The current thought is to incorporate park-like facility structures for the Gilman Park site and to minimize facilities at the Stadium site, possibly integrating them into the existing river pumping station.

2. *Conceptual Design Parameters*

2.1 Source Water Quality

The groundwater treatment piloting which was performed in conjunction with the Gilman and Stadium Well pumping tests produced quality water with minimal treatment. The water quality of the wells was also tested throughout the 5-day pumping tests, which provided a detailed picture of the raw water quality of the sources. The following table summarizes the data collected in the field during this testing:

Raw Water Quality Testing – (excerpted from Table 6 of Pilot Study Report)

Parameter (units)	**Median** (minimum – maximum) [sample count]				
	Gilman Well	Stadium Well	Blend (Gilman + Stadium)	Lary Lane Well	River (Pump Station)
pH	**7.11** (6.28-7.58) [20]	**7.00** (6.39-7.39) [14]	**7.00** (6.52-7.71) [6]	**7.88** (7.43-8.32) [10]	**6.46** (5.50-6.93) [10]
Temperature (C)	**13.8** (11.8-15.3) [10]	**11.4** (11.1-13.4) [7]	**12.5** (12.3-13.1) [3]	**10.5** (10.4-10.6) [5]	**18.0** (12.0-19.1) [5]
Total Iron (mg/L)	**1.21** (1.09-1.63) [10]	**1.32** (1.16-1.67) [6]	**1.23** (1.19-1.27) [2]	**0.13** (0.12-0.14) [5]	**0.71** (0.66-0.79) [6]
Dissolved Iron (mg/L)	**1.16** (1.16-1.34) [3]	**1.39** (1.14-1.63) [2]			**0.50** (0.47-0.52) [2]
Total Manganese (mg/L)	**0.378** (0.321-0.403) [10]	**0.563** (0.513-0.582) [6]	**0.478** (0.47-0.486) [2]	**0.149** (0.139-0.155) [5]	**0.071** (0.051-0.081) [6]
Dissolved Manganese (mg/L)	**0.377** (0.369-0.4) [3]	**0.580** (0.579-0.580) [2]			**0.091** (0.086-0.096) [2]
Total Arsenic (µg/L)	**8** (2-8) [7]	**2** (1.5-3) [6]		**5** (5-10) [5]	**BDL (<0.5)** (<0.5-0.5) [2]
Turbidity (NTU)	**0.160** (0.146-0.304) [626]	**0.232** (0.158-0.721) [550]	**0.232** (0.149-0.360) [785]	**0.163** (0.150-0.242) [1088]	**3.85** [(2)] (3.55-4.14) [2]
Specific Conductance (µs/cm)	**315** (313-439) [3]	**310** (304-319) [3]	**327** [1]		**127** (115-145) [3]

[(1)] Turbidity monitored continuously (5 minute intervals) by Hach 1720E turbidimeter.
[(2)] Turbidity reading using Hach 2100P field turbidimeter.

Laboratory analysis for each of these three wells was also performed. As the Pilot Report points out, iron and manganese results were comparable to the field methods; however, the laboratory measured pH for Gilman was 7.6 and it was 7.3 for the Stadium well. Arsenic concentrations were also measured by the laboratory. The following table summarizes the averages for each of the three wells.

Well	pH	AS (ug/L)	Fe (mg/L)	Mn (mg/L)	Alkalinity (mg/L)	Specific Cond. (us/cm)
Gilman	7.6	12.5	1.26	0.40	160	490
Stadium	7.3	3.5	1.25	0.62	130	350
Lary Ln.		11.0	<0.05	0.16		

According to the Pilot Report the Gilman and Stadium wells had iron to arsenic rations of 100:1 or greater. It also noted that with field readings with a pH near neutral that this suggests that both Gilman and Stadium have sufficient iron mass available for co-precipitation of arsenic. The piloting filtering runs did produce non-detectable arsenic levels after treatment, correlating with this theory.

The Lary Lane well has a higher pH (approximately 7.8) than either the Stadium or Gilman wells and also does not have much iron to provide the co-precipitation ratio. Therefore, the piloting for this well used the addition of ferric chloride to provide the ability for co-precipitation of arsenic. Jar tests were also performed to see if a blend of the Gilman and/or Stadium well water would provide the ability to co-precipitate arsenic, however this was inconclusive. Therefore; for the preliminary design of treatment a provision for injecting ferric chloride at the Lary Lane well is recommended.

The piloting did not perform any post-treatment with chlorine, pH adjustment or the addition of zinc orthophosphate. Provisions for the addition of these treatment chemicals are included in this preliminary design as it is hypothesized that some addition of each will be necessary post-treatment to match the Town’s surface water supply quality.

2.2 Groundwater Treatment Design

After reviewing the pilot recommendations and considering site and operational issues Weston & Sampson has developed a preliminary design of groundwater treatment (GWT) to include:

- Construction of a new GWT facility located at the former volleyball court at Gilman Park. The facility would be designed to be:
  - An approximate footprint of 30 feet x 80 feet (2400 square feet of interior space).
  - Built to be as energy and environmentally efficient as possible, including porous pavement, drainage that goes to rain gardens and other “green” building components.
  - Incorporated into the park-like atmosphere of the Gilman Park, and possibly have the construction coincide with the construction of a pavilion to serve users of the park. This concept has already been brought forth through the preliminary architectural design of the facilities
- Four (4) new 8-foot diameter pressure filters and filter media with Greensand Plus.
  - These filters would be capable of producing 250 gallons-per-minute (GPM) of flow at a loading rate of 5.0 gpm/square foot. Data from the pilot and water quality of the wells support the concept that higher flow rates may be possible; however, for design purposes we are assuming the flow rate of 5.0 gpm/square foot. With all four filters on line this provides a design flow rate of 1.44 million gallons a day.
  - Additional room to expand the system by adding two more filters will be incorporated into final design. With six filters on line this would provide a design flow rate of 2.16 million gallons a day.
- Chemical feed bulk and day tank storage for the following:
  - Sodium Hypochlorite (CL2) in 15% solution
    - 100 gallon day tank
    - 1,500 gallon bulk tank
  - Ferric Chloride (Lary Lane Well Only)

- 55 gallon drums
- Sodium Hydroxide (25%)
  - 40 gallon day tank
  - 500 gallon bulk tank
- Zinc Orthophosphate
  - 55 gallon drums

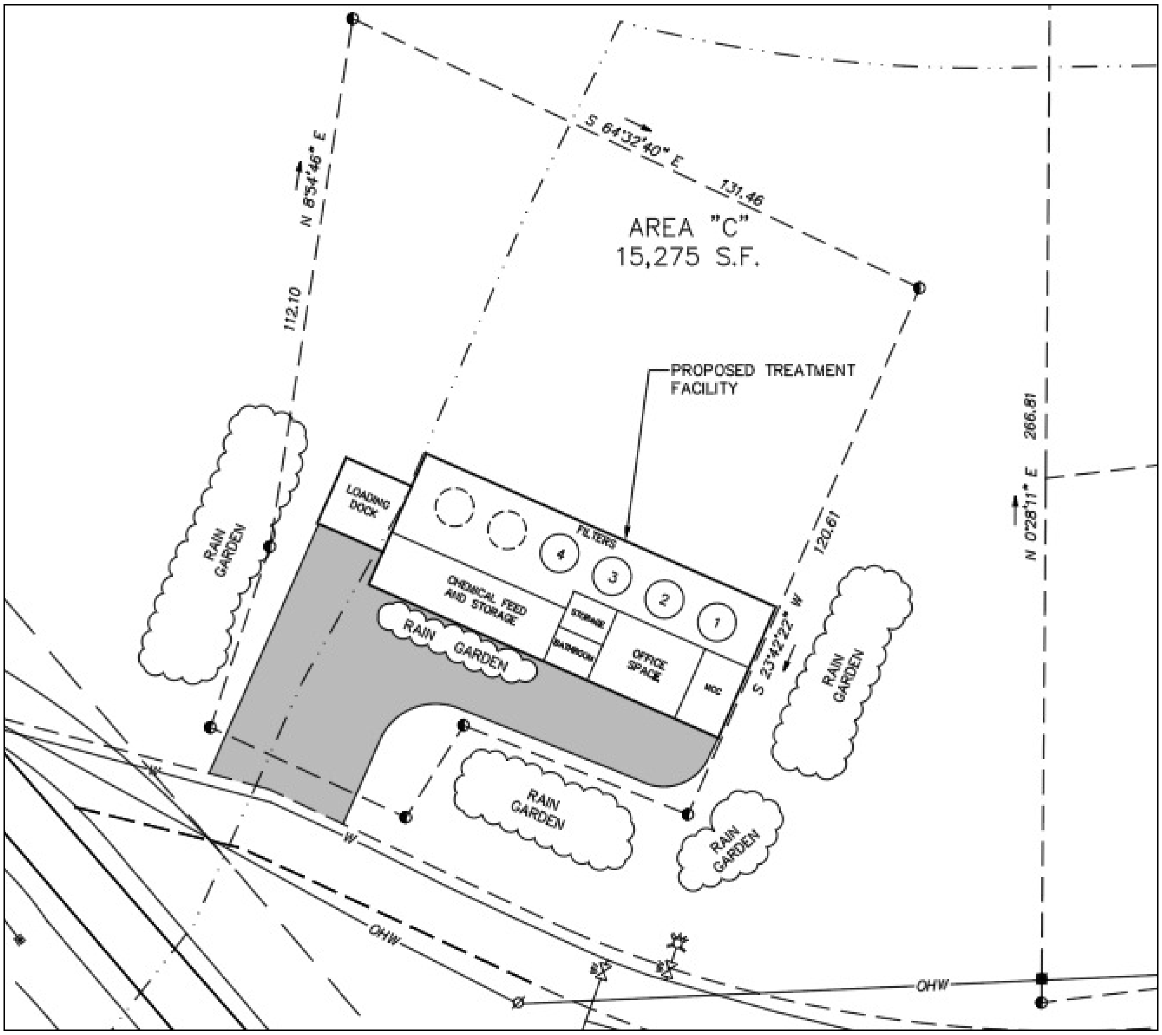


Figure 2.1 – Preliminary Site Layout of Groundwater Treatment Facility at Gilman Park

2.3 Recommendations for Filter Media, Flow Rates and Chemical Feed Dosages

The following sections describe the design parameters for the Groundwater Treatment System (GWTS) regarding the filtering, flow rates and chemical feed dosages. As previously mentioned, a water treatment pilot of the Gilman, Stadium and Lary Lane Wells was performed in July 2009 as part of the pumping test of the Gilman and Stadium Wells. A summary of the study results is as follows (excerpted from "Pilot Study Report for Iron, Manganese and Arsenic Treatment – Exeter, NH, July 2009" by Blueleaf, Inc.):

> The pilot study tested three different granular media: GreensandPlus™, Pureflow© PM-200, and LayneOx™. All were found to be effective for iron,

manganese and arsenic removal. Effluent water quality was comparable for all three filtration media. The high-iron sources, Gilman and Stadium, were effectively treated at Filter Surface Loading Rates from 2.5 to 5 gpm/sf. The Engineer anticipated the design loading rate would be no higher than 5 gpm. Higher rates were likely possible, but were not piloted. The raw water was pretreated sodium hypochlorite for oxidation, at a dose sufficient to also provide a disinfecting chlorine residual.

The low-iron source, Lary Lane, was effectively treated at Filter Surface Loading Rates from 5 to 10 gpm/sf. Pureflow© Filtration Division requested that their PM-200 media be piloted at higher loading rates in order to better determine the limits of the media, and allow more flexibility in design. All three filtration media were therefore piloted at rates up to 10 gpm/sf to provide an equal basis for comparison. Chemical pretreatment included sodium hypochlorite for oxidation, and ferric chloride to provide supplementary iron for arsenic co-precipitation. Supplemental iron was necessary since Lary Lane had a low native iron concentration. Using ferric chloride, the pilot filters performed similarly with Lary Lane as they did with the high-iron sources in terms of water quality, filter run times, and pressure losses.

Filter run times were limited by iron breakthrough. Typical filter run times were on the order of 24 hours or more at 5 gpm/sf, and 18 hours or more at 10 gpm/sf. At a loading rate of 2.5 gpm/sf the filter run time exceeded 54 hours without any indication of breakthrough, while piloting the Gilman Well. Iron breakthrough preceded manganese and arsenic breakthrough. Filter effluent arsenic and manganese concentrations remained below their respective MCLs beyond the point of iron breakthrough, defined as an effluent iron concentration greater than the EPA Secondary MCL of 0.3 mg/L total iron.

Filter performance was effectively maintained by backwashing with finished water according to manufacturer recommendations. Recommended backwash procedures differed for the various filtration media. Nearly all of the iron and arsenic mass accumulated during filter service was recovered in the backwash wastes. Backwash solids settled quickly, and sludge volumes were reduced to 4% or less of total backwash volume within 24 hours.

Findings of this study concluded that:

- For the Gilman Well, the Stadium Well, and a Blend of Gilman and Stadium raw water, it was found that oxidation with sodium hypochlorite was the only chemical pretreatment required for effective contaminant removal. Effective operating conditions for Gilman, Stadium, and the Gilman/Stadium Blend were:
    - Filter Surface Loading Rates: Up to 5 gpm/sf (higher FSLRs were not piloted).
    - Pre-Oxidation with NaOCl at 4 – 8 mg/L (2 – 4 mg/L as Cl2).

- For the Lary Lane Well, it was found that ferric chloride was needed to provide supplemental iron for co-precipitation, in addition to oxidation with sodium hypochlorite. It was not necessary to adjust the pH. Effective operating conditions for Lary Lane were:
  - Filter Surface Loading Rates: Up to 10 gpm/sf.
  - Pre-Oxidation with NaOCl at 3.2 mg/L (1.6 mg/L as Cl2).
  - Ferric Chloride at a dose of 2.9 mg/L (higher doses were not piloted).

- Filter run times were limited by iron breakthrough. Iron breakthrough preceded manganese and arsenic breakthrough. When operating using Gilman, Stadium, or the Blend, the observed filter run times to iron breakthrough were:
  - GreensandPlus: >54 hours at 2.5 gpm/sf, 29 hours at 5 gpm/sf.
  - Pureflow PM-200: 26 hours at 5 gm/sf.
  - LayneOx 20x40: 26 hours at 5 gpm/sf.

- When operating using Lary Lane, filter run times were dependant upon the ferric chloride feed. Using a ferric chloride dose of 2.9 mg/L, the observed filter run times to breakthrough were:
  - GreensandPlus: 25 hours at 10 gpm/sf
  - Pureflow PM-200: 18 hours at 10 gm/sf.
  - LayneOx 20x40: 18 hours at 10 gpm/sf.

- Differential pressures increased mainly as a function of the iron mass loading rate. The PM-200 filter developed slightly less differential pressure than the GreensandPlus filters, at equal loading rates. The LayneOx filter developed more differential pressure than either the GreensandPlus or PM-200 filters.

- Effluent turbidity proved to be a reliable indicator of iron breakthrough. For the high iron sources, Gilman and Stadium, a turbidity of 0.150 NTU generally coincided with iron breakthrough. For Lary Lane, a turbidity of 0.120 NTU generally coincided with iron breakthrough while using a ferric chloride dose of 2.9 mg/L.

- The filters were backwashed according to manufacturer recommendations, using filter effluent. Backwashing effectively maintained filter performance. Contaminant mass recovery calculations indicated that nearly all accumulated iron and arsenic mass was removed by backwashing.

- Backwash settling was evaluated during piloting of the Gilman and Stadium Wells. Backwash solids settled quickly, and sludge volumes were reduced to 4% or less of total backwash volume within 24 hours of settling. Backwash supernatant typically had iron and manganese concentrations lower than raw water iron and manganese.

2.4 Filter Configuration and Recommended Design Parameters:

Based upon the results of the filtration pilot testing, any of the three brands of filter media would adequately remove contaminants to meet drinking water standards. The finished water quality after treatment with each media is similar at similar filter loading rates and chemical pre-treatment dosages. The following parameters are recommended for final design:

- Adsorptive Media of either GreensandPlus™ (GSP), Pureflow© PM-200, and LayneOx™ (LNX), (note: other medias may be acceptable but were not piloted)
- Four (4) Filters installed, two (2) filters for future well tie-in
- Surface Loading Rate (FSLR): 5.0 gpm/sf each
- Filter Vessel Diameter: 8 feet
- Filter Flow Rate: approximately 250 gpm per filter
- Depth of Adsorptive Media: 24 in. (GSP) or 32 in. (PM-200 or LNX)
- Depth of Anthracite: 12 in. (GSP) or 0 in. (PM-200 or LNX)
- Total Filter Media Depth: 36 in. (GSP) or 32 in. (PM-200 or LNX)
- Freeboard Height: 24 in. minimum

The filter vessels will be designed for a 36 inch media depth with at least 24 inches of freeboard and 24 inches for underdrain and media support system. The straight shell height will be no less than 7 feet.

Depending upon the media chosen for installation, the backwash rate and duration will vary. This will present a challenge for designing the backwash piping, valves, instruments, holding tank and backwash controls. During final design, the approach to backwash design will be finalized.

While it is assumed that the initial construction will involve installation of four filters, the water treatment plant will be designed for installation of an additional two filters to treat future groundwater supply.

Provisions for future potential treatment via aeration or other means to remove radon, if those regulations to into effect and require treatment, will be made in final design.

2.5 Chemical Feed Systems

Sodium hypochlorite was piloted as the oxidation chemical for the raw water. While piloting treatment of the Lary Lane Well, Ferric chloride was added to the low iron content raw water to aid in arsenic removal by co-precipitation. It is anticipated that blending Lary Lane with the Gilman and Stadium Wells will sufficiently increase iron content to render Ferric chloride addition unnecessary. In addition, the pilot indicated that pH adjustment is not needed. The conceptual site layout will identify space for future chemical storage and feed areas, in the event pH adjustment (to match with the blended surface water quality), ferric chloride addition (for Lary Lane well), zinc orthophosphate (to match treated surface water quality) or other chemical treatment is required in the future.

Based on the pilot and Town of Exeter typical chemical treatment, we assume that only Sodium hypochlorite will be added for oxidation pre-treatment ahead of the pressure filters. Preliminary sizing of the sodium hypochlorite chemical feed and storage equipment is listed below with basis of design noted.

Table 2.1 – Chemical Storage Design Parameters

Design Parameter	Value	Basis of Design
Sodium hypochlorite liquid strength	15%	Currently used at surface water WTP
Chemical Dosage	4.0 to8.0 mg/L as NaOCL	Results of Pilot Test – Gilman and Stadium blend
Bulk Storage Volume Required	1,450 gal	30 day supply (ave)
Bulk Storage Tank Size (volume)	2,000 gal	tank manufacturer standard size selection – for use in layout
Bulk Storage Tank Diameter/Height	7'-0" 8'-6"	
Day Tank Volume Required	95 gal	30 hour max day volume
Bulk Storage Tank Design Size (Volume/Diameter/Height	155 gal 2'-8" 4'-6"	tank manufacturer standard size selection – for use in layout
Chemical Pumps Required	2	Lead/standby
Chemical Pump Size (flow range/pressure)	0.5 to 3.5 gph 200 psi	Based on Pilot dosage range and min/max plant design flow
Minimum Chemical Pipe Size	½ inch	Standard design practice

During final design, chemical treatment will be revisited to ensure optimization of the layout, system performance and reliability.

2.6 Treatment Plant Operational Concept

The controls and instrumentation at the wells and water treatment plant will allow for fully automatic operation of the system via a PLC or PAC based SCADA system and radio telemetry. In addition, full manual local control will be designed at each component to allow for routine maintenance and backup to the automatic control.

The water treatment plant operating flow rate will depend upon which wells are operating, as selected by the operator or sequenced by SCADA. The automatic control strategy will be developed during final design and may incorporate:

- tank level and system pressure control
- well pump VFD speed control
- well level drawdown control setpoints
- seasonal, calendar day, or daily time based controls
- other online monitoring or operator entered control parameters

In addition to automatic programmed control and local manual control, the operator will be able to manually adjust the flow rate of each well VFD at the SCADA screens to suit the demand, season or well condition.

The maximum design flow rate through each filter is 250 gpm and the FSLR of 5.0 gpm/sf maximum. The design flow and loading rate will allow for treatment of the combined maximum well flow rates, as proposed to NHDES.

During operation, a backwash will be triggered automatically based on setpoints compared to various online monitoring parameters. The following parameters and recommended setpoints would automatically initiate backwash of an individual filter:

Turbidity:	0.1 NTU
Differential Pressure:	10 psi
Run Time/Filtered Volume:	determine in field

Turbidity will be used as an indicator of iron breakthrough. The pilot indicated that turbidity of 0-12 to 0.15 NTU correlated to iron breakthrough. W&S recommends the turbidity not exceed 0.1 NTU in the filtered water. The operator should sample and analyze finished water periodically to ensure iron, manganese and arsenic are below the contaminant levels required or recommended by state and federal agencies.

A differential pressure of 10 psi across the filter media is a typical trigger for pressure filtration backwash. Note that the manufacturer's recommended allowable pressure differential across the filter media may be higher than 10 psi.

Once the groundwater treatment plant has been started up and running, the operators may recognize patterns in deterioration of filter effluent quality with respect to filter run time or gallons treated. Adjustable setpoints for these parameters will be included in the SCADA system graphic screens.

In addition, the operator will be able to manually initiate a backwash at the SCADA control screen or filter control panel.

3. Filters and Process Piping

After discussions with Town staff it was determined that the final design of the project would benefit from developing a Request for Proposal (RFP) process for the following internal water treatment components:

- Filter vessels and media
- Internal filter process piping, air wash system and valves
- Electrical controls and monitoring

The rationale for taking this approach is due to the fact that all of the media piloted during the pumping test performed satisfactorily. Utilizing this approach a bid specification would be developed and vendors would have access to the full pilot report data and anticipated operating parameters. They would then submit their intended system design and components to meet these specifications. Vendors would also be required to work with the selected general contractor on the project during construction, startup and commissioning of the system. They would also be required to warrant the system for a period of time beyond its final acceptance.

The remainder of the groundwater system; building design, chemical storage, site and facilities design, well equipment and station upgrades, piping, backwash residuals handling, and miscellaneous items would be handled with a traditional design, bid and construction process.

4. *Instrumentation and Control Systems*

The design will include instrumentation and new instruments to provide operators with improved filter and well monitoring and control. The following instruments will be included in the design:

- Wells and finished water discharge pressure transmitters (4)
- Well pump flow meter – magnetic flow meters (3)
- Well level – submersible pressure (3)
- Combined Raw Water filter influent flow meter – magnetic flow meter (1)
- Finished Water flow meter – magnetic flow meter (1)
- Filter flow control devices – magnetic flow meter or differential pressure (4)
- Filter head loss monitors – differential pressure transmitters (4)
- Backwash Influent flow meter – magnetic flow meter (1)
- Backwash Effluent flow meter - magnetic flow meter (1)
- Sewer flow monitoring – magnetic flow meter (1)
- Recycle flow monitoring – magnetic flow meter (1)
- Backwash tank level – ultrasonic or submersible pressure (1)
- Turbidity monitors – raw, finished, recycle, filters (7)
- pH monitor –(1)
- Suspended solids monitor – backwash tank influent, sewer discharge (2)
- Chemical feed flow switches (2)
- Chemical containment spill switches (1)
- Chemical day tank scales or level instrument (2)
- Chemical bulk tank level (2)
- Transfer pump discharge pressure (1)

In addition to the process instruments indicated above, the buildings will be equipped with sensors, alarms, and monitoring devices for security, fire protection, temperature, and power. Building systems instrumentation will be incorporated in the building system design.

4.1 Controls, Telemetry and SCADA

A complete instrumentation and control system will be designed to oversee equipment and instrumentation operation at the Groundwater Treatment Plant and each well station. The design will include a SCADA system designed based upon the latest hardware and software components. The new SCADA system will be designed to ensure interconnectivity with the Town of Exeter’s existing system.

Communications between the well stations, the Groundwater Treatment Plant, water tanks, and existing main SCADA station will be via radio unless radio paths are unacceptable. The strength and reliability of the paths between the sites will be determined by a radio path field survey during final design. In the event that radio is not feasible, alternate communications will be investigated such as phone, cable, fiber optic media, cellular and satellite.

The preliminary equipment selection for the control system upgrade is listed below:

- SCADA control station at the GWTP (robust workstation or small server)
- SCADA control station laptop for remote access and backup node
- SCADA software compatible with existing Exeter SCADA system
- Remote Access Software (or integrate into existing system)
- Alarm Handling Software or Hardware (or integrate with existing system)
- Radios, antennas, and accessories
- Control Panels:
    - each well station (3)
    - filters (1)
    - backwash tank recycle and waste (1)
    - chemical (1)
    - main control and telemetry (1)
- PLC (in each panel) – compatible with existing system PLCs
- Local operator interfaces (OIT/HMI)
- Network switches and accessories
- Printers

The Town of Exeter's existing instrumentation and control system will be reviewed with the Town personnel to determine which components should become standard and specified without an equal in the Groundwater Treatment Plant final design.

Functional descriptions of each process will be developed for final design including:

- Well pump VFD control
- Well pump flow and pressure monitoring
- Combined raw water instrumentation monitoring
- Filter valve operation and flow control
- Filter instrumentation monitoring
- Backwash sequencing
- Air wash system control and monitoring
- Finished water instrumentation monitoring
- Chemical storage monitoring and transfer pump operation
- Chemical feed equipment control and monitoring
- Containment area spill detection
- Equipment maintenance runtimes
- Recycle system operation and water quality monitoring
- Backwash tank level and solids monitoring,
- Water tank storage tank monitoring and level control
- Line power and standby generator monitoring

In addition to process-related monitoring and control, each well pump station will be equipped with low building temperature sensor, security and intrusion sensors, fire alarm, and power monitoring. These signals will be input to the local control panel and monitored via telemetry by the SCADA system.

5. *Residuals and Recycle System*

5.1 Residuals Handling Overview

Filtration of groundwater will produce waste as the material filtered out must be backwashed from the media to maintain acceptable filter effluent quality. The backwash waste water from filtration will consist primarily of the iron, manganese and arsenic removed from the raw water. The three medias will generate similar quality backwash waste. The pilot assessed these values and estimated the volumes based on grab samples taken from their tanks after backwash.

The intent of the design is to collect and store the backwash waste water to minimize wasted water, maximum settling and removal of solids, and return a high quality supernatant recycle volume to the head of the filters.

The site of the new treatment plant is not conducive to the construction of large open storage beds or lagoons to naturally dry residuals from filter backwash. The Town of Exeter owns and operates a wastewater treatment plant which may accept the settled solids from backwash and existing lagoons at the Town's surface water treatment plant could also take settled solids to dewater prior to land disposal. The preliminary design includes a residuals holding and settling tank to allow for concentration of solids at the bottom of the tank, recycle of supernatant, and pumping of settled solids. Prior to final design of the pumps, determination of the discharge location for the settled solids pumps is required. The pumps may be designed to allow for pumping to either an onsite sewer connection or a tank truck loading station. Alternatively, a secondary settled solids holding tank may be incorporated into the water treatment plant foundation to allow for longer periods between hauling or higher concentration of solids wasted to the sewer. An overflow will be designed in the tank to allow discharge by gravity to the sewer in the event that pumps are not working or cannot keep up with the influent volume.

The following sections detail the preliminary design of the backwash tank volume, consider the backwash quality as it relates to design, describe the recycle system design and considerations for operation of the residuals system.

5.2 Backwash Volume

The average daily backwash volume generated during treatment of the wells will depend upon the filter media installed. For all medias, the pilot indicated that runtimes exceeded 26 hours at a filter surface loading rate of 5.0 gpm /sf. For tank design, we assume a 24 hour runtime which will require one backwash per filter per day. The backwash rates for each media and volumes generated during a current design day and future design day are listed below.

Table 5.1 – Backwash Rate Comparisons

	Greensand Plus	Pureflow PM-200	Layne-Ox
Backwash Rate (gpm/sf)	12	20	25
Duration, per manufacturer (minute)	10	4	5
Flow Rate for 8' dia vessel (gpm)	603	1,005	1,257
Volume per backwash (gal)	6,030	4,020	6,285
Wasted Volume per Treated MG (gal/MG)	16,750	11,170	17,460
Daily Volume, current day (gal)	24,120	16,080	25,140
Daily Volume, future day (gal)	36,180	24,120	37,710
Two day storage volume (gal)	72,360	48,240	75,420

The current design day flow through the plant was taken as 975 gpm, the combined sustained well flow rates of Gilman, Stadium, and Lary Lane wells. The future design day flow through the plant was taken as the maximum future capacity of plant with 6 filters online at 250 gpm per filter. It is recommended that the residuals holding tank be designed to store the anticipated future backwash volume for two days. NOTE: The two day volume could be split into two tanks; one for daily operations and one typically empty for overflow with drain to the sewer. In the event of equipment failure, this will allow for a conservative day of storage until recycle or sludge pumping equipment is fixed. If the overflow tank fills, then the tank would drain to the sewer (need flap check valve, Tideflex or other low pressure backflow prevention valve).

The backwash volumes in Table 5.5 do not incorporate rinse or filter-to-waste volumes generated by recommended cycles. Manufacturers have various drain and rinse cycles. Standard engineering practice is to design for a filter-to-waste cycle to allow for wasting until turbidity of the filter effluent drops below an acceptable turbidity level, typically 0.1 NTU. These cycles contribute additional waste volume to the residuals tank. For preliminary design, the residual tank minimum required two day storage volume is calculated assuming the media with largest backwash volume (Layne Ox) and a 15% addition to account for drain, rinse, and filter-to-waste cycles. With this contingency for rinse and filter-to-waste, the wasted volume per million gallon of treated water becomes 20,100 gal/MG, or 2%. This compares with the current waste percentage at the existing surface water treatment facility that ranges between 10 to 20%, depending on seasonal treatment efficiencies.

Based on the above, the design volume for the backwash tank is a minimum of 86,700 gallons or 11,600 cf. With a minimum working height of 8 feet, the required tank area is 1,450 sf. This is approximately half the square footage of the proposed water treatment facility. The tank will be designed to equalize daily flow and will provide a buffer volume for recycle intake operation.

The design current day volume will be used to design the sludge management equipment and recycle equipment. If future filters are installed and operating, pumps would need to be evaluated at that time and replaced if necessary.

5.3 Backwash Water Quality

Typically, iron and manganese residuals generated from filter backwash can be settled over a 2 hour period and decanted. After settling, the remaining iron and manganese sludge typically contains approximately 10-30% solids. ("Water Treatment Plant Design" AWWA/ASCE McGraw Hill 3rd Edition)

During piloting, the backwash residuals were collected and analyzed for solids content at the Town's waste water facility laboratory by Town personnel. The results of the sampling and analysis indicate total suspended solids in the backwash water ranges from 150 mg/L to 280 mg/L. Visual observation of the backwash waste indicated that the majority of the solids settle within one hour. Within four hours, the settled solids layer is less than 6% of total volume.

Field analysis and laboratory analysis of the backwash volume indicate that nearly 100% of iron was filtered and recovered, 90% of manganese was recovered and arsenic recovery ranged from 81% to 95%.

Table 5.2 – Backwash Water Quality

Constituent	Gilman (mg/L)	Stadium (mg/L)	Gilman/Stadium Blend 70/30 (mg/L)	Lary Lane with FECL3 added (mg/L)*
Iron (Fe)	105.0	57.8	61.0	101.1
Manganese (Mn)	8.0	17.8	15.9	11.7
TSS (mg/L)	220	207	169	176

The values are the weighted average of results from sampling backwash volumes generated through each media piloted. Weighted average was used because the medias have different backwash rates and resultant volumes and concentrations of iron, manganese and suspended solids.

The backwash water quality analysis listed above represents a completely mixed sample concentration. During operations the settled volume will represent 10% to 30% of the backwash volume and the majority of solids will be concentrated in that volume. The supernatant was collected and sampled after 22 to 24 hours of settling. The quality of the supernatant was similar to raw water quality with respect to iron and manganese.

The supernatant was sampled and analyzed to compare with raw water quality with respect to iron and manganese. The results are below. Values shown are the average for piloted medias.

Table 5.3 – Supernatant Water Quality

Constituent	Gilman Only (mg/L)	Stadium Only (mg/L)	Gilman and Stadium Blend (mg/L)	Lary Lane with FeCl3 added (mg/L)*
Iron (Fe)	0.58	0.27	0.36	0.43
Manganese (Mn)	0.10	0.13	0.17	0.13

* - After 22 to 24 hours of settling

Note that the actual operating conditions in the backwash tank will not allow for 24 hours of settling.

5.4 Residuals System Design

The conservative estimate of maximum daily settled residuals volume is 8,700 gallons as discussed above. The residuals can be discharged to the Town's sewer system, discharged to an onsite holding tank for further settling or on-site treatment, or hauled to an off-site treatment locations.

Preliminary design of the residuals handling system assumes that settled residuals will be pumped to the sewer system daily. Preliminary design of the residuals pumps is based upon a conservative estimate of the solids content of the settled backwash residuals as noted above. The residuals pump will discharge a volume of 8,700 gallons over a 3 hour period. The total dynamic head is estimated based upon an assumption of 10 feet static head to the discharge point, piping and valve losses of 10 feet. Final design configurations will be used to calculate more accurate estimate of TDH during final design.

Preliminary design of the residuals pumps results in the following pump curve design point:

Flow rate:	50 gpm
Estimated TDH:	20 ft

Residuals pumps will be submersible solids-handling pumps equipped with constant speed motors. Pumps will operate as lead and standby with only one pump operating at a time.

The pumps will be located in a sump in the residuals tank easily accessible from above without entrance to the tank. The design will incorporate a GWTP floor hatch, stainless steel rail system and discharge slide-off disconnect coupling.

5.5 Residuals Tank:

It is recommended that the residuals tank be designed as a cast-in-place concrete tank structure also serving as the foundation for all or part of the water treatment facility structure. Hatches will be provided in the water treatment facility floor to access the residuals tank, residual pumps and recycle equipment. The floor of the tank will be sloped to a sump area for collection and pumping of residuals.

Design Guidelines:
Cast-in-place concrete tanks are to be designed and constructed in accordance with ACI 350. The tank shall also be tested for tightness in accordance with ACI 350.1.

Design loads:
The tank structure will be designed to support the weight of the water treatment facility structure and fully loaded equipment above the tank plus an additional live load. An additional surcharge will be necessary to take into account snow storage, additional equipment over the structure, and any potential future renovations within the building.

In addition to supporting the water treatment facility, the tank walls will be have to be designed for fluid loads with a unit weight and for the soil pressure as required from a future geotechnical report (anticipated as part of final design).

The base slab of the structure will be designed to resist buoyant effects due to groundwater conditions.

A comprehensive geotechnical assessment of the site would be necessary to determine the best location and final design of this structure. Site design would also have to account for flood elevation at the site so that potential flooding in the area would not effect the water quality in the tank.

5.6 Residuals and Recycle Systems Operation

The residuals handling preliminary design is based upon a wasted volume of 20,100 gallons wasted volume per 1 million gallons of treated water. Filter runs are assumed to be 24 hours for design (pilot indicated 26 hour run times). With a loading rate of 250 gpm per filter and a maximum current day flow rate of approximately 1,000 gpm, there would be four filters operating with one backwash per day each for a total of 4 filter backwashes. The recommended residuals and recycle operations at maximum current day flow of 1,000 gpm (1.44 MG) with 4 backwashes per day are:

Backwash	4 hours	+ 29,000 gal IN
Settle	10 hours	---
Recycle	7 hours (5%)	-20,300 gal OUT
Drawoff Settled solids	3 hours	- 8,700 gal OUT

The above scenario assumes that backwashes are performed consecutively once per hour over a four hour period, rather than at four different times of day. If backwashes are initiated automatically, controls will be in place to stop recycle pumps until a pre-set minimum settling time has passed after backwash completion. Also, recycle pumps will stop at a pre-set minimum water level one foot above the settled solids range. The settled solids drawn off are conservative at 30% of the total backwash volume. If actual residuals conditions allow for greater concentration of solids, then more volume will be available for recycle and less residuals volume will be discharged to the sewer.

Recycle pumps will be selected to provide approximately 5% of the raw water maximum combined flow rate. The recycle pumps will be specified with variable speed driven motors to allow for raw water flow proportional operation. Designing for more than 5% of the raw water is not recommended since the wasted volume generated is not sufficient to warrant the larger pumps. Even at 5% recycle rate, the pumps will return the supernatant in 7 hours on a maximum current day.

5.7 Recycle System Design

The recycle system will consist of an intake in the residuals tank, three recycle pumps and motors with variable frequency drives, instrumentation and controls, and piping. Recycle pump design will incorporate high efficiency pumps and motors. The operating points on the curve will enable recycle to be returned at a percentage of influent raw water flow. The design target percentage will be 5%.

Based on minimum flow to maximum current day design flows, the recycle rates will be:

Minimum current day design rate:	200 gpm
Minimum recycle rate (5% of min flow):	10 gpm
Maximum current day design rate:	975 gpm
Maximum recycle rate (5% of max flow):	49 gpm

Preliminary design of the recycle pumps results in the following pump curve design points:

Flow rate:	10 gpm to 50 gpm
Estimated TDH:	300 ft

The total dynamic head (TDH) is estimated based upon water system model information and preliminary allowances for piping and filter headloss. This value will be calculated based upon final design of filter and pipe configuration, pump elevations and field data from distribution system pressure testing.

Two pumps will be required to allow for an efficient pumping system over this range. A third pump will be installed as a standby for use during maintenance or failure of the operating pumps.

The recycle pumps shall be located between the tank inlet velocity reduction area and the overflow assembly and are designed to draw clarified water from the top of the holding tank volume for recycle to the head of the treatment facility. Recycle volume will be drawn from the top of the holding tank volume via a floating suction flexible inlet hose attached to an end suction or submersible pump located in a dry chamber adjacent to the holding tank. In addition to housing the pumps, the adjacent chamber would house the pump motors, discharge piping and valves, instrumentation and sample taps to monitor water quality of the recycled water.

Operating the recycle system with a floating suction inlet requires the inlet to be located approximately six inches below the water surface at all times to prevent unwanted suction of air into the system. The inlet would remain afloat and set at six inches below the water surface via an attached buoy. As water is drawn down in the tank, and as the inlet approaches the bottom of the tank, there will be a minimum tank elevation that the inlet should not drop below. The minimum tank elevation will be dictated by the settled solids/supernatant interface, the physical configuration of the flexible inlet hosing and floating buoy or both. We assume that the minimum elevation the tank will be drawn down is three feet above the tank floor, on average.

The recycle system will be designed to meet the Filter Backwash Rule if necessary as well as provide operational flexibility with monitoring and control. It is recommended that the NHDES be consulted regarding the applicability of this rule for a groundwater treatment system. Appurtenances will include flow monitoring, turbidity monitoring and sample line to the laboratory. Based on the pilot, recycle water is anticipated to be comparable to raw water quality.

6. *Well Stations*

6.1 Overview of Proposed Well Station Upgrades

Upgrades to all three well stations will be necessary for this project. The following is a general summary of those components:

Lary Lane Well:

- Refurbish building
- Install new high efficiency pump and motor
- Upgrade SCADA (if necessary)
- Install ferric chloride chemical feed equipment

Gilman Park Well:

- Refurbish building
- Install new roof
- Upgrade heating and electric components
- Level floor
- Install high efficiency pump and motor

Stadium Well:

- Install new piping to separate from surface water transmission main
- Line or upgrade river crossing pipe
- Construct meter and mechanical building two feet above high water mark
- Install backup generator

6.2 Well Pumps

The following table relays the well pump flow, head and motor values used in preliminary design. The flow rates are based upon the approved application to the NHDES for permitted withdrawal rates from the wells.

Table 3.1 – Well Pump Design Points (maximum)

Well	Flow (gpm)	Head (ft)	Efficiency	Motor HP
Gilman	500	300	60%	40
Stadium	250	300	60%	75
Lary Lane	225	300	60%	40

W&S estimated head requirements based on verbal data. We are currently reviewing Town's hydraulic model to determine head requirements. A combined pump and motor efficiency of 60% is a conservative assumption for preliminary design. Actual pump efficiency should be 75% or greater. Motors will be specified as premium efficiency which would require at least 90% efficiency in this horsepower range.

7. *Water Mains*

7.1 General Overview of Water Main Options

As part of this Preliminary Design Report we analyzed what transmission and distribution system water main upgrades would be necessary as they relate to the groundwater treatment system envisioned for the Gilman Park site. A detailed memorandum on our draft findings was presented to the Town staff in the fall of 2010. That information has been updated and presented in this report.

We utilized the hydraulic model data provided to us on disk from the Town. We did not do any field verification of the data as presented in that model. We would recommend some field work be performed prior to any final design of any new water main.

We looked at manifolding the three existing wells (Lary Lane, Gilman and Stadium) so that they all get treated at the Gilman Park site. We assessed two scenarios for the transmission line from Lary Lane to Gilman Park and have presented them herein. Finally, we performed an analysis of transmission and distribution water main needs should the Town chose to locate the new groundwater treatment facility adjacent to the Lary Lane Well.

For the distribution system we utilized the normal operating parameters, provided by the Town water system operators, for a majority of the analysis. An additional fire flow scenario was run in the model during a low pressure simulation. Although the low pressure conditions were not used for most of the analysis, we believe that our assessment is representative of what upgrades would be necessary to allow adequate and efficient flow from the Gilman Park site into the greater distribution system if the groundwater treatment system were producing 1.5 million gallons per day.

Finally, we took into account the Town's capital improvement plan needs as discussed with Town Engineer, Paul Vlasich. All of this is presented herein and in the accompanying summary graphic.

7.2 Raw Water Main Options – Transmission Main

The Town identified two potential routes for transmitting raw water from the Lary Lane well to a new ground water treatment plant (GWTP) located at Gilman Park. The first route utilizes existing water main in Lary Lane in combination with the installation of new 8-inch transmission main in Court Street and Bell Avenue. The second route is a cross country route through Phillips Exeter Academy property. The following is a brief discussion of our findings. It should be noted that the costs presented are budgetary project costs representing construction, engineering and contingency costs.

Option 1 - Lary Lane, Court Street, Bell Avenue Route

This option first involves converting approximately 1,900 linear feet (lf) of 10-inch diameter finished water main into raw water main in Lary Lane. The existing pipe appears to be, at a minimum, cast iron with a cement lining (CICL) and a roughness factor of C = 110 based on the hydraulic modeling database. For the purpose of this preliminary evaluation, the water main appears to be in good condition for use as a raw water main to transmit Lary Lane well water to the proposed GWTP. Since the existing finished water main would be converted into a raw water main, furnishing finished drinking water to the one customer that is serviced from the Lary Lane water main is needed. One option is to install a residential well with residential-level treatment capabilities on the property of the customer. The second option is to install a 2-inch diameter water service from the finished water main in Court Street to the customer (1,900 lf). For this preliminary evaluation, we are providing a cost estimate for the residential well option. The water service option is approximately four times more expensive.

Construction Cost Estimate = $15,000
Engineering and Contingency = $5,250
Project Cost Estimate = $20,250

New 8-inch raw water main would be installed in Court Street between Lary Lane and Bell Avenue. This section of Court Street, approximately 1,850 lf, is classified as New Hampshire state highway and as such, will be subjected to higher installation costs due to additional permitting, additional traffic management measures and more stringent construction standards.

Construction Cost Estimate = $175,000
Engineering and Contingency = $61,250
Project Cost Estimate = $236,250

Approximately 1,800 lf of new 8-inch raw water main would also be installed in Bell Avenue between Court Street and the site of the proposed GWTP at Gilman Park. Since installing a new finished water main in Bell Avenue is a high priority per the capital improvement plan (CIP), the proposed raw water main could be installed in the same trench as new finished water main in Bell Avenue. Installing both the raw water and finished water mains in the same trench reduces the overall installation cost as compared to installing these pipes in separate trenches.

We assumed the cost to install 8-inch ductile iron pipe to serve as the raw water main would be approximately 1/3 of the total cost to install both water mains in the same trench. As a result, we estimate the construction cost for this section of raw water main to be approximately $65,000.

Construction Cost Estimate = $65,000
Engineering and Contingency = $22,750
Project Cost Estimate = $87,750

Option 1 Total Project Cost Estimate = $344,250

Option 2 – Academy Cross Country Route

This option pertains to installing 8-inch diameter HDPE pipe cross country through Phillips Exeter Academy (PEA) property between the Lary Lane well and the site of the proposed GWTP at Gillman Park.

This route through PEA property is a heavily wooded route with the potential for multiple wetland crossings. For the purpose of this preliminary evaluation, we assume that the entire pipe installation, approximately 3,000 lf, will be installed via horizontal directional drill methods through both upland and wetland areas. Since geologic conditions are unknown for this preliminary evaluation, the cost to directionally drill the pipeline will be heavily contingent to account for these unknowns. This factor has therefore been included in the cost estimate.

An in depth evaluation of pipe installation alternatives along this route, however, may result in a combination of open cut and horizontal directional drill (HDD) pipe installation methods as the best alternative. Due to the heavily wooded condition, significant clearing and grubbing would most likely be required along any portion of the route where open cut installation could occur. In locations where HDD installation could occur, additional investigation into geologic conditions is warranted to assess any complexities that could be encountered during HDD operations. If a combination of open cut and HDD installation methods were employed, the conservative cost assigned to this route should still be valid for budgetary purposes to account for the costs of clearing and grubbing and other site conditions that could contribute to higher overall installation costs.

Construction Cost Estimate	**= $450,000**
Engineering and Contingency	**= $157,500**
Project Cost Estimate	**= $607,500**

Option 2 Total Project Cost Estimate = $607,500

Option 3– Lary Lane GWTP Alternate Location

If Gilman Park cannot be utilized to locate the proposed GWTP, the Lary Lane site has been identified as a potential alternate location. Locating the GWTP at the Lary Lane site, as opposed to the Gilman Park site, will result in the following:

- A design flow of 800 gpm will be transmitted from Gilman Park to the Lary Lane site as opposed to a design flow of 225 gpm transmitted from Lary Lane to the Gilman Park site. The design flow of 800 gpm accounts for the potential contribution of raw water from the Stadium and Gilman wells plus any future contribution from the Drinkwater Road well, should that well require treatment.
- This increase in raw water transmission flow rate would warrant upsizing the raw water main diameters from 8-inch to 12-inch in Court Street and Bell Avenue. Converting the existing 10-inch potable water main in Lary Lane to a raw water main is still feasible even with the increased design flow rate of 800 gpm.

- Approximately 1,900 feet of 12-inch ductile iron water main would be installed in Lary Lane to transmit the finished water from the Lary Lane GWTP site into the distribution system. The 1,800 feet of 12-inch ductile iron water main in Bell Avenue would not be required, as discussed in Option 1 above.
- Under this scenario, three separate trenches would need to be excavated to install the water mains in Bell Avenue, Court Street and Lary Lane. If the GWTP were located at Gilman Park, the proposed raw and finished water main could be installed in the same trench in Bell Avenue, as stated in Option 1 above.
- The Gilman and Stadium well pumps would need to be sized to accommodate additional headloss encountered in the raw water transmission main from the Gilman Park area to the Lary Lane site. If the GWTP were located at the Gilman site, only the Lary Lane well pump would need to be sized to accommodate headlosses encountered in the raw water transmission main route.
- For this option, we only assumed the street routes mentioned in Option 1 above as a viable raw water transmission main route. We did not evaluate the cross country raw water main route described in Option 2 above when comparing the difference between the GWTP located at Lary Lane versus Gilman Park.

Construction Cost Estimate (raw water only)	**= $340,000**
Engineering and Contingency	**= $119,000**
Project Cost Estimate	**= $459,000**

Option 3 Total Project Cost Estimate ***= $459,000***

The following figure shows the preferred water transmission main route, which follows the existing roadways rather than going through the PEA property:

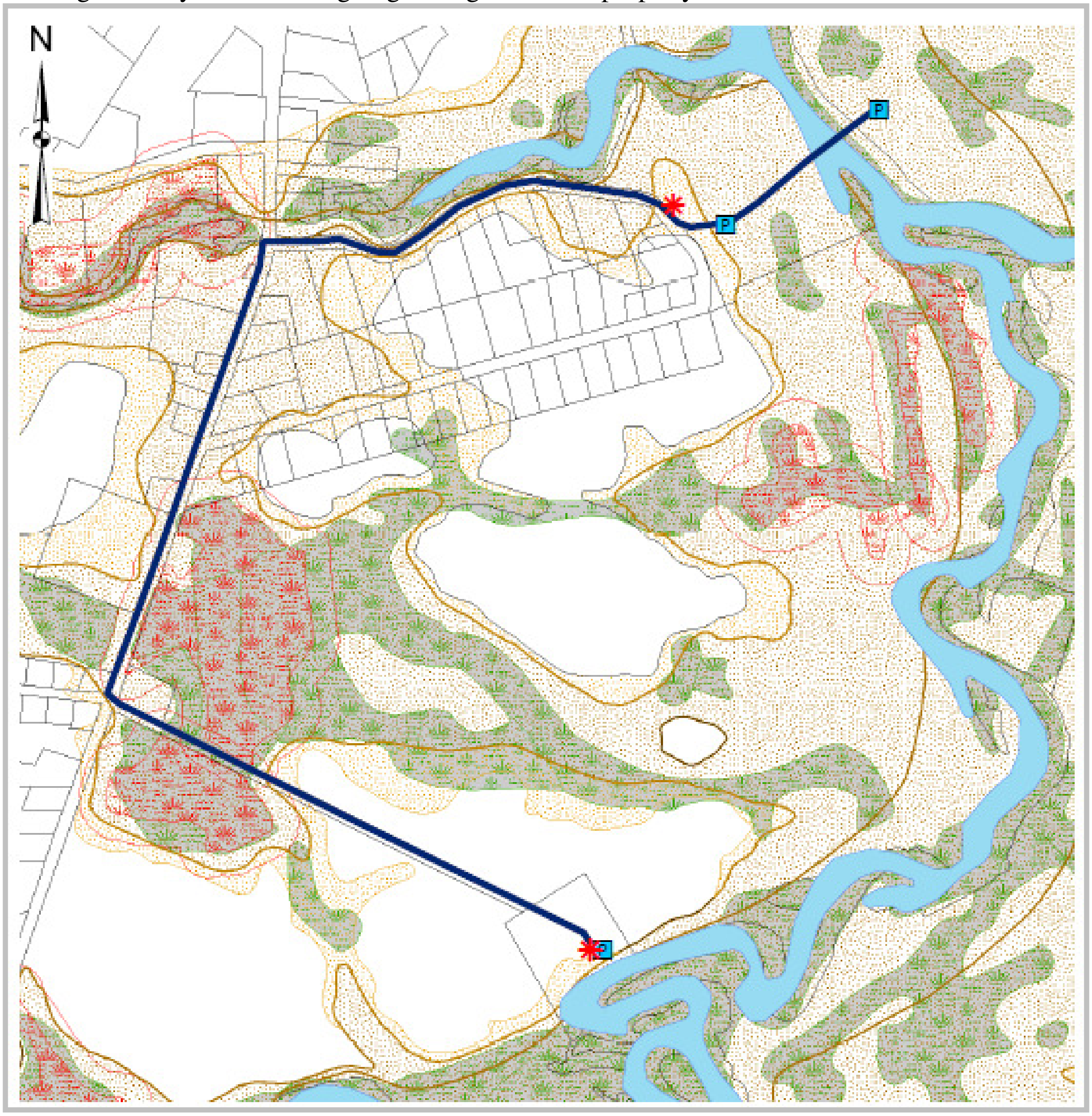


Figure 7.1 – Preferred Transmission Main Route

7.3 Finished Water Main Options – Distribution System

At the Town's request, we evaluated four sections of the water distribution system using the existing hydraulic model and identified probable water main upgrades and costs for each section. The following is a brief discussion of our findings. Similar to the raw water main evaluation, the costs presented are budgetary project costs representing construction, engineering and contingency costs.
The following table is a listing of key parameters used during modeling:

Table 7.1 – Hydraulic Model Normal Operating Parameters

Parameter	Value
Epping Road Tank	Tank Level = 234'
Hampton Road Tank	Tank Level = 203'
Cross Road Tank	Tank Level = 224'
Average Day Demand	1.09 MGD
Surface WTP	1.09 MGD
Lary Lane Well	0.32 MGD
Stadium Well	0.72 MGD
Gilman Well	0.43 MGD

Section 1 – Upgrade Bell Avenue water main

Per the Capital Improvement Plan (CIP) Exeter has identified Bell Avenue as a top priority for replacement of the existing 10-inch diameter finished water main. We recommend that 12-inch diameter cement lined ductile iron (DI) pipe be installed to replace the existing 10-inch diameter pipe.

The existing finished water main in Bell Avenue is 10-inch diameter pipe assumed to be unlined cast iron with a roughness factor of C = 55 based on the hydraulic modeling database. A maximum flow rate of 1,025 gpm from the GWTP was used in the model to determine the hydraulic conditions that could be expected within the existing main if upgrades were not performed.

The existing water main would most likely be upgraded with 12-inch diameter DI pipe. A proposed 12-inch diameter DI pipe with a C value of 140 was modeled using the same GWTP flow rate conditions (1,025 gpm) as stated above. The results of the modeling analysis are included in the following table:

Table 7.2 - Bell Avenue Finished Water Main Upgrade Hydraulic Analysis

Diameter	C value	Flow (gpm)	Headloss (ft/kft)
10 inch	55	750	17.5
12 inch	140	940	2.0

By upgrading the finished water main in Bell Avenue from 10-inch to 12-inch, the anticipated headlosses would be reduced by over 15 ft/kft and the carrying capacity of the water main would be increased by almost 200 gpm. If no upgrades are made, approximately 275 gpm from the GWTP would flow through the 6-inch diameter water main in Crawford Avenue creating sustained headlosses of 12.5 ft/kft and resulting in significant additional power consumption by the GWTP pumps.

The estimated costs for Section 1 are as follows:

Construction Cost Estimate = $135,000
Engineering and Contingency = $47,250
Project Cost Estimate = $182,250

Section 2 – Upgrade Court Street water main between Bell Avenue and Pine Street

The existing finished water main in Court Street between Bell Avenue and Pine Street is 10-inch diameter pipe assumed to be CICL with a roughness factor of C = 95 based on the hydraulic modeling database. A maximum flow rate of 1,025 gpm from the GWTP was used in the model to determine the hydraulic conditions that could be expected within the existing main if upgrades were not performed.

The existing water main would most likely be upgraded with 12-inch diameter DI pipe. A proposed 12-inch diameter DI pipe with a C value of 140 was modeled using the same GWTP flow rate conditions (1,025 gpm) as stated above. The results of the modeling analysis are included in the following table:

Table 7.3 – Court Street Finished Water

Main Upgrade Hydraulic Analysis

Diameter	C value	Flow (gpm)	Headloss (ft/kft)
10 inch	95	800	2.0
12 inch	140	830	1.5

By upgrading the finished water main in Court Street from 10-inch CICL to 12-inch DI, the anticipated headlosses would only be reduced by a negligible amount and the carrying capacity of the water main would increase slightly. This section of Court Street, approximately 750 lf, is classified as New Hampshire state highway and as such, will be subjected to higher installation costs due to additional permitting, additional traffic management measures and more stringent construction standards.

The estimated costs for Section 2 are as follows:

Construction Cost Estimate = $100,000
Engineering and Contingency = $35,000
Project Cost Estimate = $135,000

Section 3 – Upgrade Pine Street water main between Front Street and Court Street

The existing finished water main in Pine Street between Front Street and Court Street is 12-inch diameter pipe assumed to be CICL with a roughness factor of C = 100 based on the hydraulic modeling database. A maximum flow rate of 1,025 gpm from the GWTP was used in the model to determine the hydraulic conditions that could be expected within the existing main if upgrades were not performed.

The existing water main would most likely be upgraded with 12-inch diameter DI pipe. A proposed 12-inch diameter DI pipe with a C value of 140 was modeled using the same GWTP flow rate conditions (1,025 gpm) as stated above. The results of the modeling analysis are included in the following table:

Table 7.4 – Pine Street Finished Water Main Upgrade Hydraulic Analysis

Diameter	C value	Flow (gpm)	Headloss (ft/kft)
12 inch	100	500	1.2
12 inch	140	590	0.8

By upgrading the finished water main in Pine Street from 12-inch CICL to 12-inch DI, the anticipated headlosses would only be reduced by a negligible amount and the carrying capacity of the water main would marginally increase.

Installing water main in Pine Street, approximately 1,600 lf, would involve tie-ins in State Highway roads at both ends; Front Street and Court Street respectively. Pine Street itself is not a State Highway but rather a residential suburban road with less than 20 water services and only one side street connection.

The estimated costs for Section 3 are as follows:

Construction Cost Estimate = $160,000
Engineering and Contingency = $56,000
Project Cost Estimate = $216,000

Section 4 – Upgrade Lincoln Street water mains between Maine Street (Route 27) and Front Street (Route 111)

The existing finished water main in Lincoln Street between Main Street and Front Street is mostly 6-inch diameter pipe assumed to be CI with a roughness factor of C = 40 based on the hydraulic modeling database. The water main between Tremont Street and Main Street is 8-inch diameter CI pipe with a roughness factor of C = 45. A maximum flow rate of 1,025 gpm from the GWTP was used in the model to determine the hydraulic conditions that could be expected within the existing mains if upgrades were not performed.

At the request of the Town, we modeled 12-inch diameter DI pipe, with a C value of 140, between Route 27 and Route 111. We ran the model under the same GWTP flow rate conditions (1,025 gpm) as stated above. The results of the modeling analysis are included in the following table:

Table 7.5 – Lincoln Street Finished Water Main Upgrade Hydraulic Analysis

Diameter	C value	Flow (gpm)	Headloss (ft/kft)
6 inch/8-inch	40/45	60	4.3
12 inch	140	225	0.2

By upgrading the finished water main in Lincoln Street from 6-inch/8-inch CI to 12-inch DI, the anticipated headlosses would be significantly reduced and the carrying capacity of the water mains would significantly improve. In addition to the headloss and domestic water demand comparisons listed in Table 5, we evaluated the impact a 12-inch water main would have on fire flows in the Lincoln Street area. In particular, we simulated a 2,000 gpm fire flow at the Lincoln Street Elementary School. The results of the model indicate that a fireflow of 1,250 gpm with a residual pressure of 20 psi is available at the school under existing normal conditions. Upon upgrading to a 12-inch diameter DI water main, a 2,000 gpm fire flow can be achieved at the school. The residual pressure drops to approximately 64 psi during a 2,000 gpm fire flow. It should be noted that the existing 8-inch CICL water main in Front Street, between Pine Street and Lincoln Street, experiences high headlosses and velocities during a 2,000 gpm fire flow on Lincoln Street. Upgrading this 300 foot section of the distribution system with 12-inch diameter DI water main would minimize hydraulic constraints in the area and would provide better transmission of water from the proposed GWTP to Route 27.

Upon receiving parameters used in the model to simulate a low pressure scenario, we re-ran the model to assess the residual pressures that could be experienced at the Lincoln Street School during a 2,000 gpm fire flow event with low system pressures. The following table compares the normal system conditions and low system pressure modeling runs.

Table 7.6 - Fireflow comparison
Average vs. Max Day Demand Scenario

Lincoln Street School

Scenario	Static Pressure (psi)	Fire Flow (gpm)	Residual Pressure (psi)
Avg. Day	84	2,000	64
Max Day (low pressure)	66	2,000	38

In both scenarios, the residual pressure remains above 20 psi during a 2,000 gpm fire flow event at steady state conditions in the model. During the low pressure scenario, however, the availability of 2,000 gpm at residual pressures above 20 psi for an extended duration may be limited if all sources remain off during a fire event. Further evaluation and/or an extended period modeling simulation may be warranted to confirm fire flow availability during a low pressure scenario.

Installing 12-inch diameter DI water mains in Lincoln Street, **approximately 2,100 lf**, would involve tie-ins in State Highway roads at both ends; Main Street and Front Street respectively. Lincoln Street itself is not a State Highway but rather a residential/commercial suburban road with less than 25 water services and five side street connections. Since the Lincoln Street Elementary School and the Exeter Amtrak Downeaster Train Station reside on these roads, increased traffic is present that will most likely require additional traffic management measures.

The estimated costs for Section 4 are as follows:

Construction Cost Estimate = $225,000
Engineering and Contingency = $78,750
Project Cost Estimate = $303,750

Section 5 – Front Street water main replacement additional discussion

The existing water main in Front Street between Railroad Avenue and Center Street is listed as 8-inch diameter cast iron water main in the hydraulic model database. Furthermore, the 8-inch water main in Pine Street is unlined cast iron pipe between Pine Street and Center Street. Due to the age of the pipe and presumably the poor interior condition of the water main, water main upgrades are a likely recommendation along this entire route.

We describe a 300 foot section of Front Street water main between Pine Street and Lincoln Street in Item 4 of this memo. We identify this 300 foot section of water main as a hydraulic constraint for transmitting water from the proposed GWTP at Gillman Park to Route 27. However, 12-inch ductile iron water main would end at the intersection of Lincoln Street and Main Street.

Performing water main upgrades between Pine Street and Lincoln Street may be warranted should the Town continue upgrades in the Route 27 area to connect proposed 12-inch water main in Lincoln Street with large diameter main present in Epping Road. This would create a large diameter transmission route between the proposed GWTP and the Epping Road storage tank. Subsequently, the Town should consider replacing the existing water main in Front Street between Railroad Avenue and Center Street as part of any overall water main replacement program. At this time, however, replacement of the 300 foot section of pipe did not significantly impact the modeling results we evaluated for this report as run under steady state conditions.

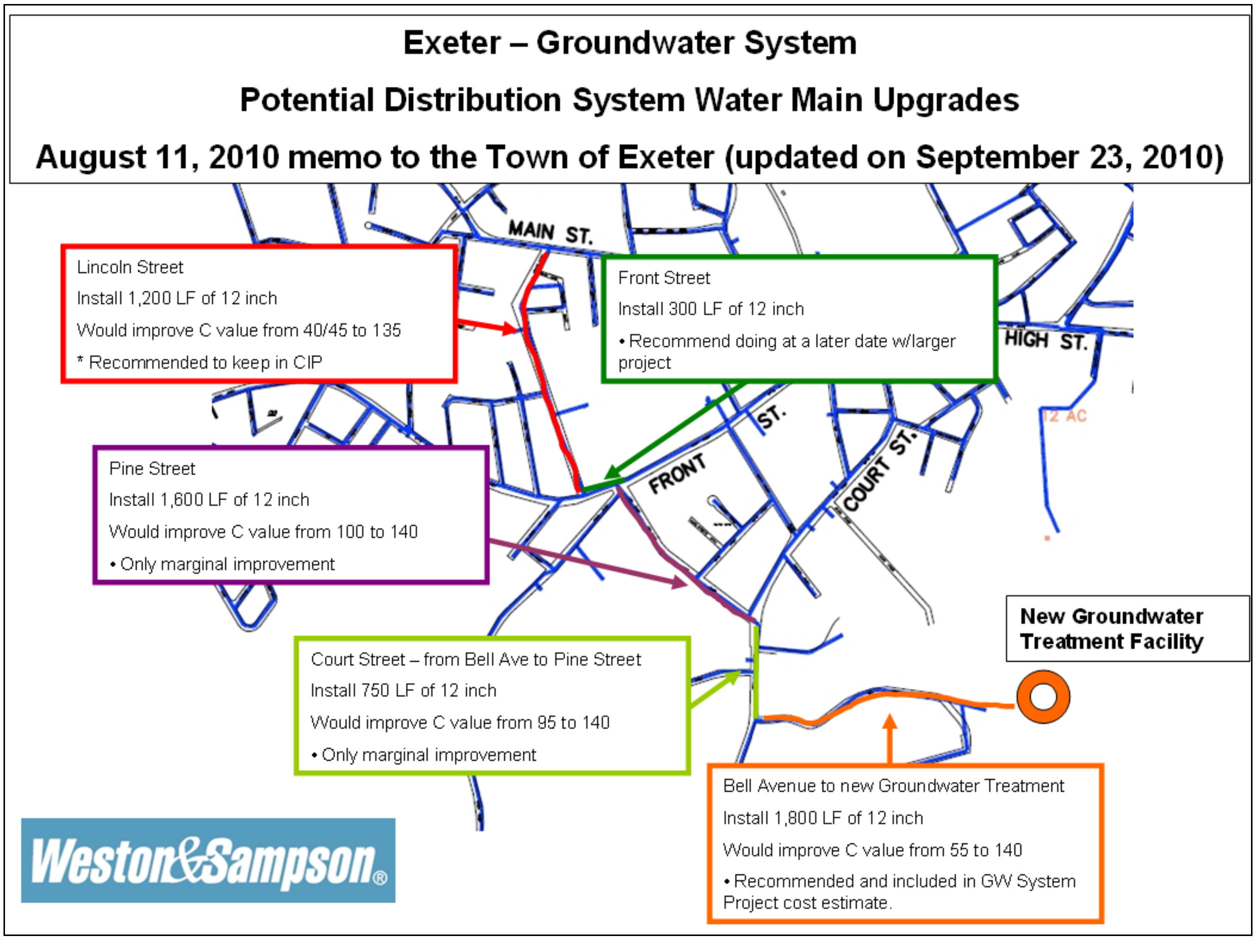


Figure 7.2 – Potential Distribution System Water Main Upgrades

8. Site Infrastructure

8.1 Overview of Site Improvements

The Gilman Park area is the preferred location for the new groundwater treatment facility. Working with the Town and our architectural design partner, TMS Architects, we have developed a preliminary conceptual plan that would integrate the site infrastructure into that park theme. The buildings would be constructed so that they blend into the site surroundings rather than become stand-a-lone fixtures. "Green" components such as rain gardens for roof drains and permeable pavement to reduce stormwater runoff would be incorporated into the project. The following graphic shows the proposed layout of the site's preliminary design. Note that there is also a pavilion located at the existing basketball courts. This is just a conceptual idea at this point and is included only for planning purposes. If the Town were to chose to move forward with a pavilion too, then we would suggest that it be designed and constructed of similar materials to match the other two buildings.

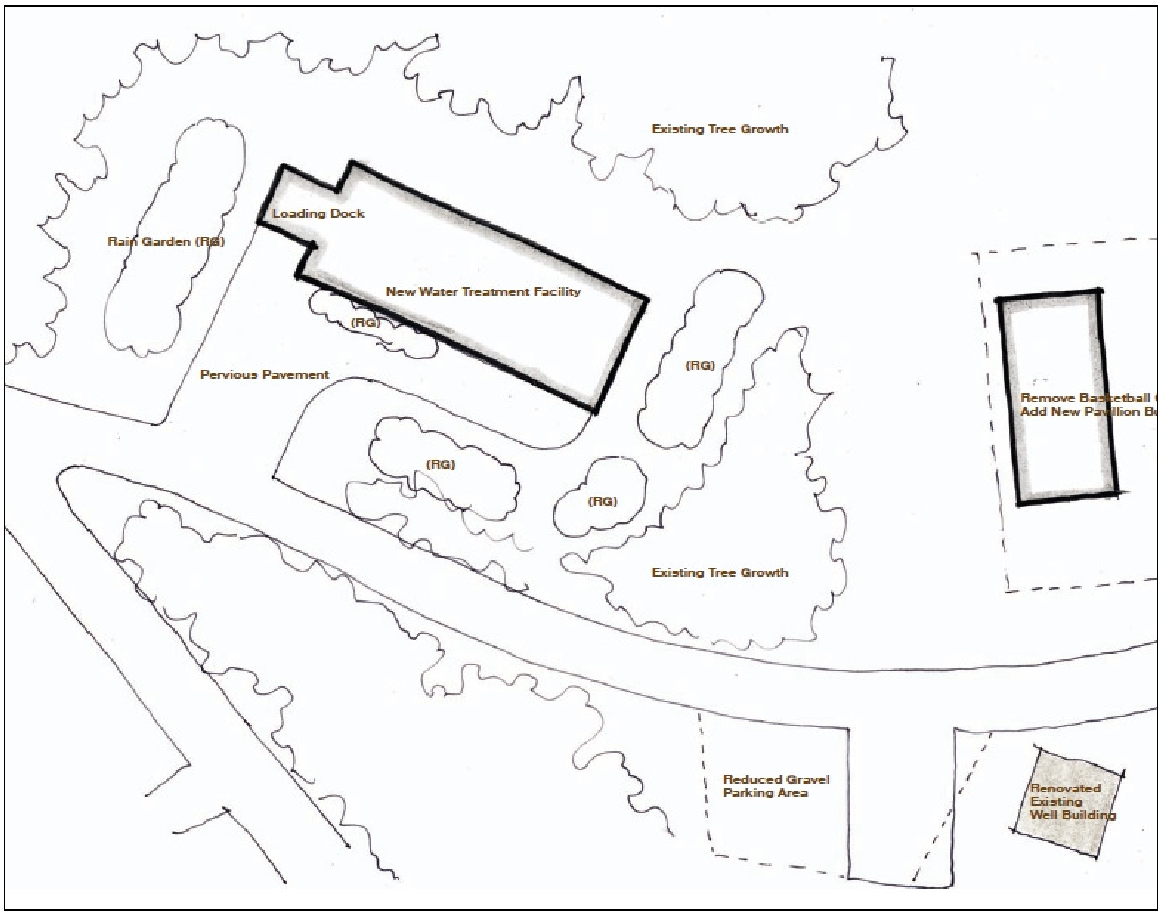


**Figure 8. 1 - Gilman Park Groundwater Treatment Project –
Preliminary Design of Proposed Site Layout (TMS Architects)**

The next graphic shows the conceptual site upgrades for the Gilman Park Well. Currently, the well site has a small parking area adjacent to the existing building. To improve site security and wellhead protection we have incorporated the site improvements shown in the following figure. These include reducing the gravel parking area size by revegetating it and adding a rain garden. A parking area that would accommodate one maintenance vehicle for water operations staff is all that would remain. This area would be gated in order to limit access.

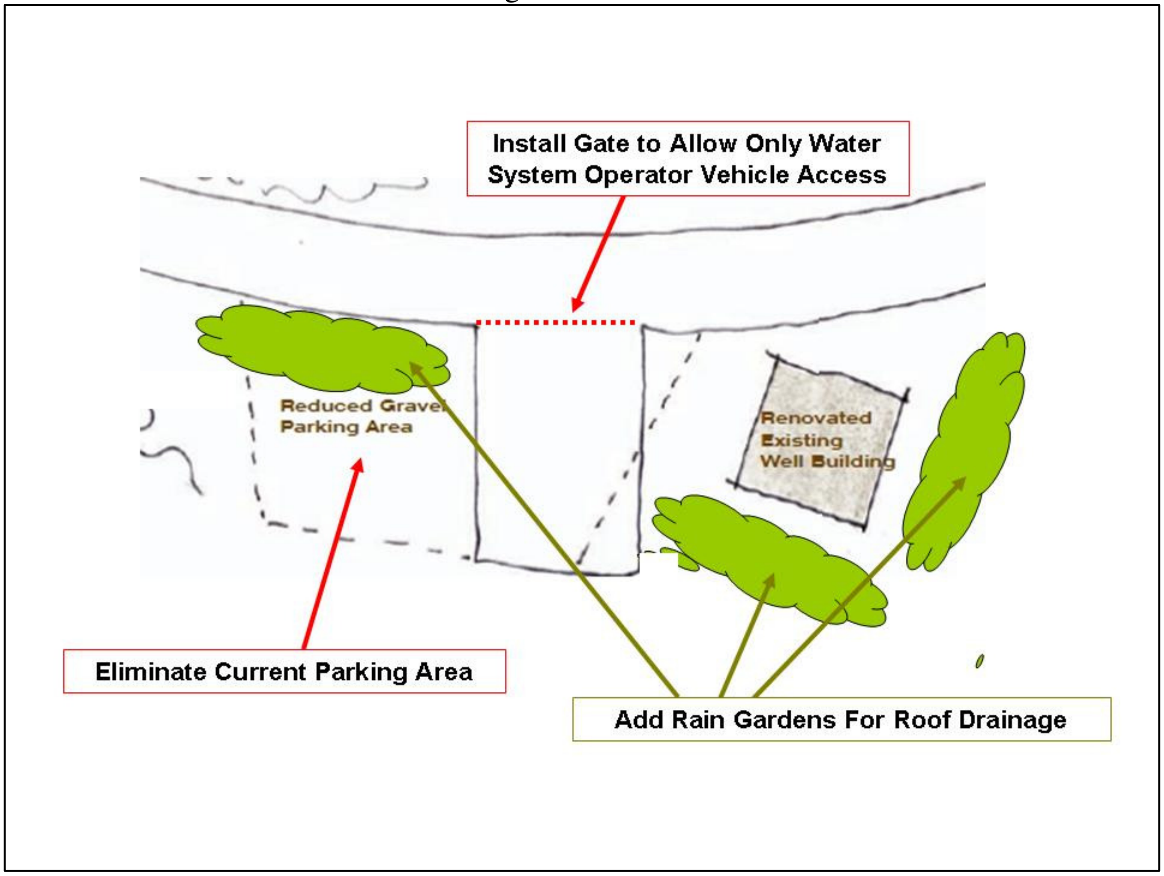


Figure 8.2 - Proposed Site Improvements for Gilman Park Well

8.2 Proposed Structures:

TMS Architects prepared the following summary for the proposed Gilman Park and Groundwater Treatment Facility building components:

Gilman Park Well Building Upgrade Components:

1. New exterior roof, entry door, canopy elements and well head access hatch
2. Materials: (highlights)
    a. Roof; metal standing seam
    b. Trim; Azek (or other synthetic material)
    c. Concrete Paint (interior and exterior) environmentally friendly paints
    d. Wet type lighting (interior and exterior)

The following graphic shows TMS's architectural rendering of the Gilman Park Well improvements:

Figure 8.3 - Architectural Rendering of Gilman Park Well Site Improvements (TMS Architects)

8.3 Proposed Gilman Park Groundwater Treatment Building:

1. New 30'x75' building 2,250 sf
2. Materials: (highlights)
   - a. Metal roof and metal siding (vertical, horizontal and standing seam mix)
   - b. Trim, metal to coordinate with roof and siding
   - c. Slab on grade foundation with thermal barriers
   - d. Environmentally friendly paints
   - e. Ventilation and heating to be reviewed and coordinated with wet location issues and required air changes
   - f. Windows and doors to be energy efficient and to be installed to help with day-lighting and security. Orient the windows higher to allow for greater light infusion into the building as well as deter vandals.
   - g. Building insulation to be foam insulation or similar (no batt insulation) to help reduce the yearly heating and cooling needs
   - h. Coordinate office space for occasional use with electric heat source or possible geothermal if used in the building
   - i. Renewable energy sources should be looked at and could be solar (photovoltaic and thermal tubes) as well as geothermal. Coordinate to achieve as efficient a building envelope as possible to realize the lowest operating costs, and then add in renewable energy options. Lighting fixtures and pumps in the building could be energy efficient components to help reduce electrical consumption on a daily basis.

The following graphic shows TMS's architectural rendering of the proposed Gilman Park Groundwater Treatment Facility:

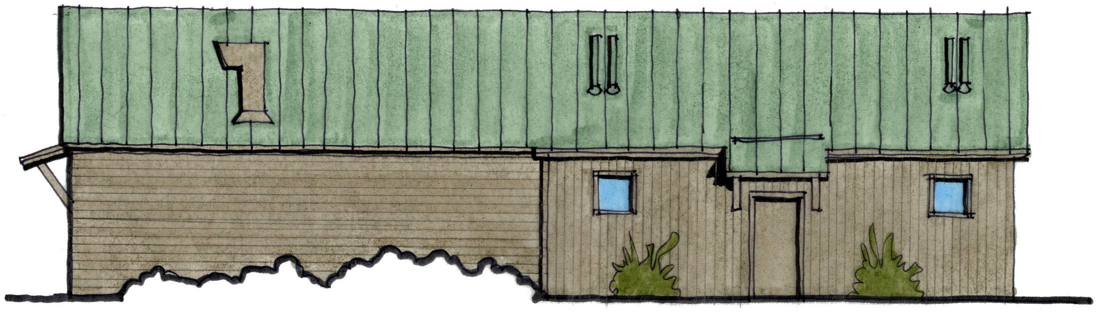


Figure 8.4 - Architectural Rendering of Groundwater Treatment Facility (TMS Architects)

8.4 New Stadium Well Building Components:

Due to the flood elevation at the existing Stadium Well site we are proposing that the area be mounded to raise the floor elevation of a new well house to approximately two feet above the highest flood elevation. The proposed well equipment would be a submersible pump installed in the well a pitless adapter going into a meter pit located under a new well building. The top of the casing of the well would also be two feet above the maximum flood elevation.

The following figures depict the preliminary site plan. Final design would require a detailed site survey to determine final location and height of structures.

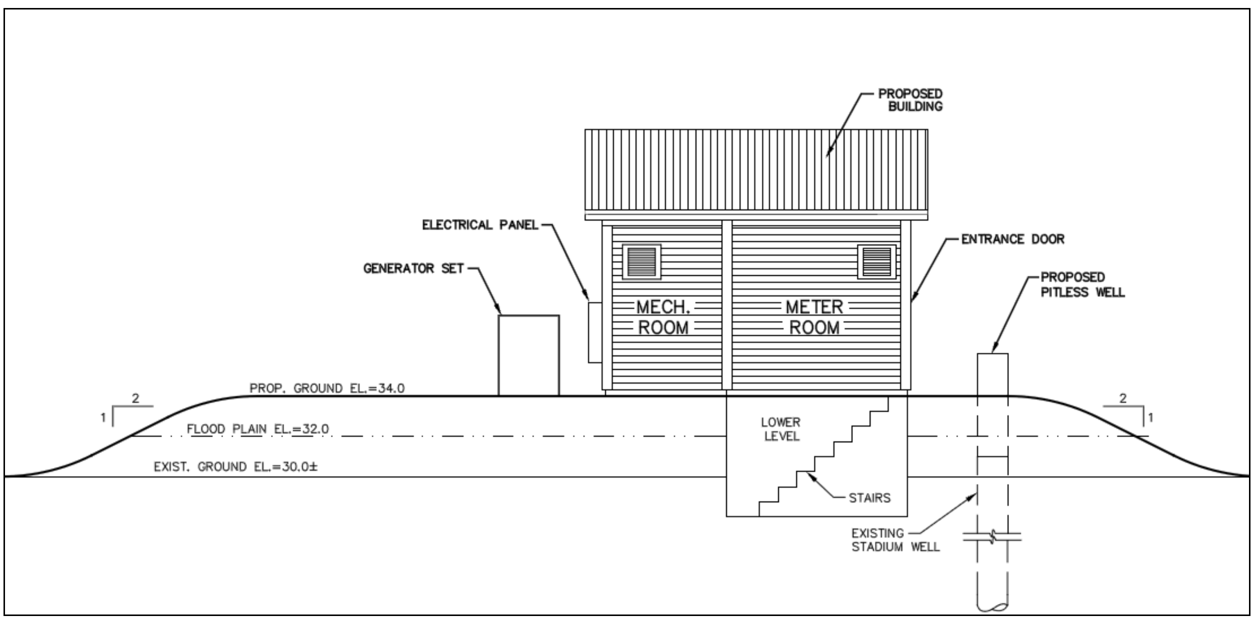


Figure 8.5 - Stadium Well - Proposed Building Layout

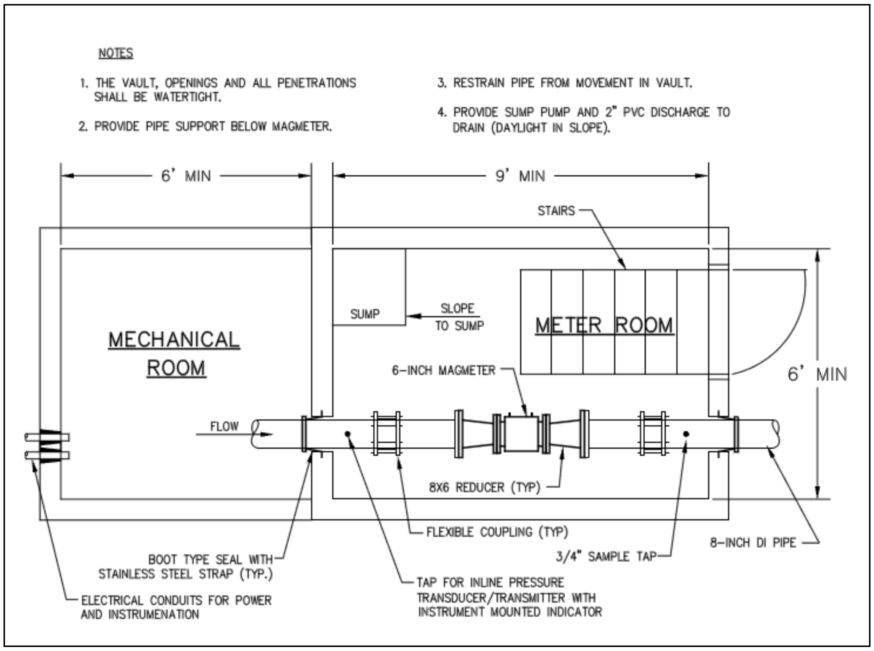


Figure 8.6 - Stadium Well – Proposed Meter Pit Layout

1. New building constructed over a meter pit.
2. Materials: (highlights)
    a. Roof; metal standing seam
    b. Trim; Azek (or other synthetic material)
    c. Concrete Paint (interior and exterior) environmentally friendly paints
    d. Wet type lighting (interior and exterior)
3. Standby Power Generator with Propane Tank utilized for both heating and standby power. (This facility could also be sized to accommodate the river pumping station for standby power. This should be addressed in final design of the system components.)

The following graphics show TMS's architectural rendering of the Stadium Well building:

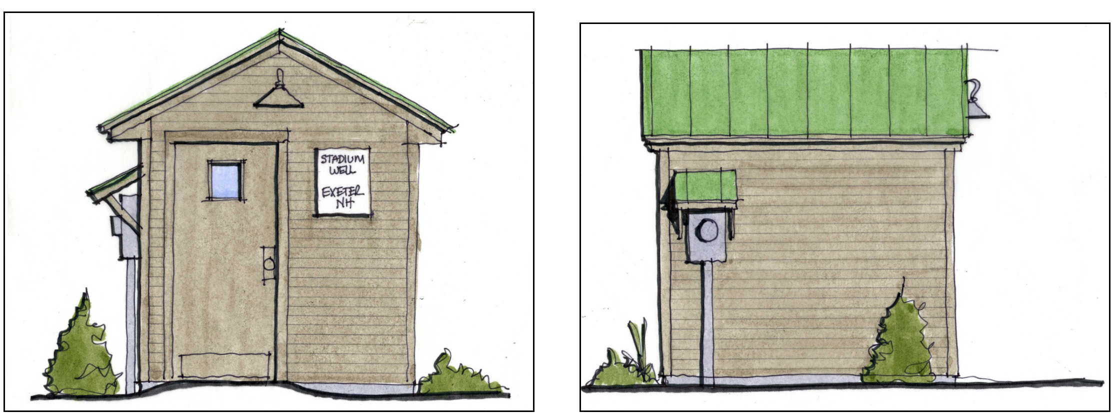


Figure 8.7 - Stadium Well – Proposed Front and Side View of New Building (TMS Architects)

8.5 Alternative Site at the Lary Lane Well Property

In the event that the Gilman Park site cannot be utilized for the groundwater treatment, an alternative site has been selected adjacent to the Lary Lane Well. An overlay of the estimated property boundaries (based on Town GIS data) on the following figure reveals that a groundwater treatment facility could fit within the currently town-owned property. If this site were selected to the preferred alternative to the treatment facility then a detailed survey would be necessary to assess the exact property boundaries, 300-foot shoreland protection and potential building footprint.

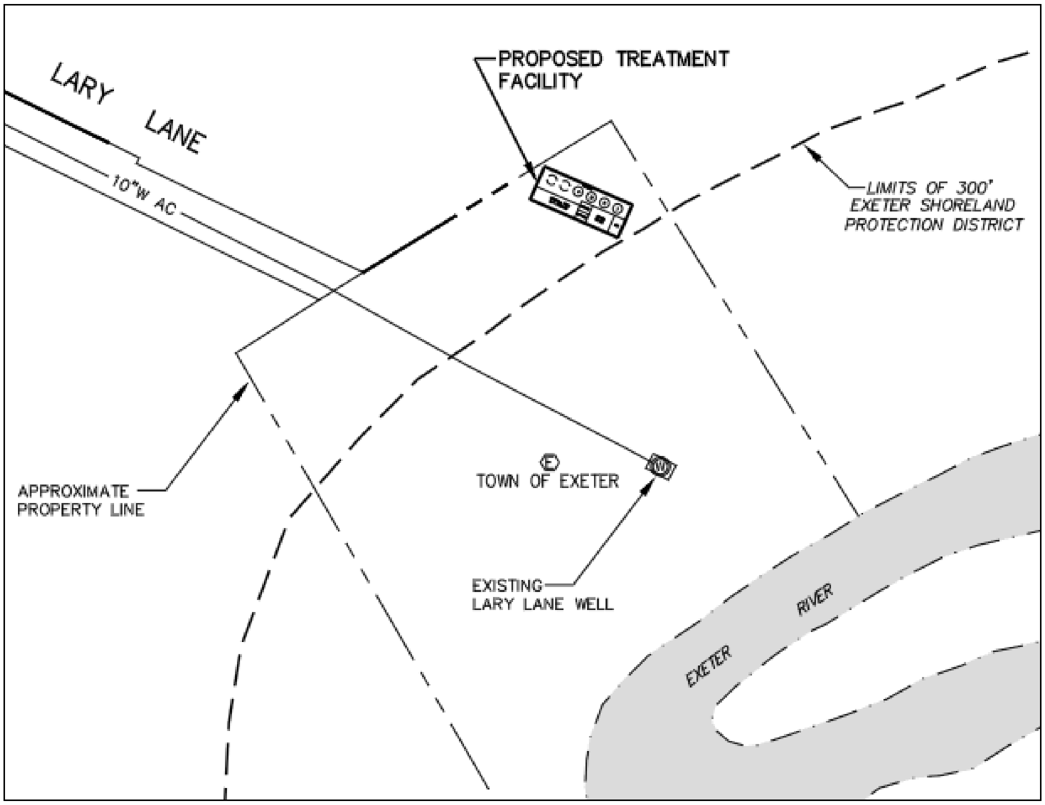


Figure 8.8 - Alternative Groundwater Treatment Facility Location at Lary Lane Well

Note: Following the Town's warrant article vote in March 2011 for the Groundwater System project it was determined by Town staff that further development of the conceptual site plans for building a facility at the Lary Lane site would be beneficial. Though the site is further away from the center of the distribution system and would require additional piping and energy to pump from the wells, especially if a fourth well were brought on line, it may have some site issues that would benefit the system, such as:

- Location is more remote and out of site than the Gilman Park location.
- The site may have the ability to handle backwash residuals with lagoons, holding tanks or other processing equipment.
- Blending with the Lary Lane Well may be accomplished more efficiently than at the Gilman Park site.
- The treatment building orientation and site constraints may be more flexible than the Gilman Park site.

9. *Operational and Capital Cost Estimates*

Weston & Sampson performed an extensive cost estimate of the Exeter water system's cost per million gallons as part of our 2010 Water Supply Alternatives Report. Operating costs related to chemical usage and electrical consumption were investigated for the surface water treatment system using actual data from January 2008 to May 2009. Groundwater costs were initially developed based on estimated electrical costs and utilizing only chlorine and ferric chloride as treatment chemicals. For this report we have updated these chemicals to include the application of zinc orthophosphate at the same dosage as the surface water treatment facility in order to match the waters when blended. Again, as with the pilot review and initial analysis, we do not anticipate pH adjustment at this time. However, provisions for future chemical feed of this treatment chemical will be made during final design.

9.1 Surface Water Treatment Costs

The cost of treating surface water at the Town's Portsmouth Avenue water treatment facility varies seasonally, mostly due to the source being utilized at the time. As the following table shows, when available, the reservoir water costs less to treat. This is primarily due to the fact that the water gravity feeds into the surface water treatment facility from the reservoir. When water from the river is utilized it must be pumped to the facility at a considerable cost for the electricity.

Table 9.1 – Surface Water Treatment Cost Estimates

Exeter Water System
Surface Water Treatment Cost

	Source	Million Gallons	Chem Cost	Electric KWH per MG	Electric Cost Per MG	Total Cost per MG	Total Monthly Cost
Jan	Reservoir	37.6	$214.57	871	$91.64	$306.22	$11,504
Feb	Reservoir	32.1	$214.36	871	$91.64	$306.01	$9,823
Mar	Reservoir	29.4	$222.00	871	$91.64	$313.64	$9,235
Apr	Reservoir	36.2	$222.22	871	$91.64	$313.87	$11,358
May	River	44.6	$281.88	2118	$232.14	$514.02	$22,915
Jun	River	42.5	$263.01	2118	$232.14	$495.15	$21,050
Jul	River	42.4	$347.03	2118	$232.14	$579.18	$24,576
Aug	River	43.4	$381.42	2118	$232.14	$613.56	$26,600
Sep	River	43.2	$384.97	2118	$232.14	$617.12	$26,639
Oct	River	43.1	$356.39	2118	$232.14	$588.54	$25,388
Nov	River	34.7	$280.94	2118	$232.14	$513.08	$17,808
Dec	Reservoir	36.8	$234.82	871	$91.64	$326.46	$12,015
Total		**466.0**					**$218,910**
Average/Day		**1.3**					**$600**
Month Ave	Reservoir	34.4	$221.60	871.0	$91.64	$313.24	$10,786.83
Month Ave	River	42.0	$327.95	2118.0	$232.14	$560.09	$23,567.98

9.2 Groundwater Treatment Costs

The following assumptions were made regarding demand, operating rates, chemical dosages and electrical use for the groundwater system:

- Flow rates are assumed from pump test results and the NHDES reactivation letter.
- Total dynamic head for the pumps assume a well pumping depth of 30 feet, system pressure of 85 psi, head loss of up to 15 psi through the filter plant.
- Chemical feed rates were taken from the pilot study and reflect the high end of the estimated range. For Stadium and Gilman Wells a sodium hypochlorite dosage of 8 mg/l was used. For the Lary Lane Well a sodium hypochlorite dosage of 4 mg/l was used and dosage of 2.9 mg/l of ferric chloride was assumed. Zinc Orthophosphate was calculated to be fed at the same dosage per million gallons as the existing surface water treatment system in order to match water quality of the blended sources.

Table 9.2 – Groundwater Treatment Cost Estimate

Water Pumping and Treatment Electrical and Chemical Costs
Gilman, Stadium and Lary Lane Wells

	Million Gallons	Chem Cost per MG	Electric Cost Per MG	Total Cost per MG	Total Monthly Cost
Jan	37.6	$76.74	$172.45	$249.20	$9,362
Feb	32.1	$76.74	$172.45	$249.20	$7,999
Mar	29.4	$76.74	$172.45	$249.20	$7,337
Apr	36.2	$76.74	$172.45	$249.20	$9,018
May	44.6	$76.74	$172.45	$249.20	$11,109
Jun	42.5	$76.74	$172.45	$249.20	$10,594
Jul	42.4	$76.74	$172.45	$249.20	$10,574
Aug	43.4	$76.74	$172.45	$249.20	$10,803
Sep	43.2	$76.74	$172.45	$249.20	$10,757
Oct	43.1	$76.74	$172.45	$249.20	$10,750
Nov	34.7	$76.74	$172.45	$249.20	$8,649
Dec	36.8	$76.74	$172.45	$249.20	$9,171
Total	**466.0**				**$116,124**
Average/Day	**1.3**				**$318**
Month Ave	34.4	$76.74	$172.45	$249.20	$9,676.99

Anticipated chemicals - Chlorine and Zinc Orthophosphate for Gilman and Stadium Wells
Ferric Chloride for Lary Lane Well if blending with two other wells does not achieve removal of arsenic through co-precipitation

As revealed in these two tables, the groundwater treatment operating cost estimate is almost half the current annual operating costs to the Exeter water system. The electrical usage for groundwater treatment is higher than reservoir treatment but lower than the river pumping treatment. To determine brake horsepower, an assumed combined motor and pump efficiency of 0.6 was assumed. This is very conservative for vertical turbine pumps if premium efficiency motors are used. The real cost savings would be in the significant reduction in chemicals required for treatment with a groundwater system.

9.3 Cost Comparisons

The following table summarizes the cost comparison scenarios on a per million gallon basis utilizing information gathered from this report. It also presents a hypothetical cost if each source were the sole source for the Town's municipal water supply for a year. These costs have been updated to factor in our recent analysis performed as part of this preliminary design.

The gallons treated vs. gallons produced is a factor of the treatment system inefficiencies attributed to the amount of backwash waste water necessary to produce treated water to the system (this factor is not included in Tables 9.1 and 9.2). Recent efforts of the Town's water treatment staff have reduced the surface water system's waste to approximately 10 to 15%. For the purpose of this report we have used the mid-point of 12.5%. According to the groundwater treatment pilot report, if water is recycled from the groundwater backwash waste there will likely be only 1 to 2% of waste produced by that system. To be conservative, we utilized 2% for this report.

Table 9.3 – Cost Comparison by Source

Source of Supply	Cost per Million Gallons	System Demand (million gallons per year)	Gallons treated vs. gallons produced	Total Gallons treated per year	Hypothetical Cost per Year (365 million gallons)
Reservoir	$313	365	1.125	410.63	$128,624
River	$560	365	1.125	410.63	$229,988
Groundwater	$249	365	1.020	372.30	$92,776

For planning purposes, using the midpoint average cost of surface water, the total annual cost to produce 1 million gallons per day of treated water to the system is as follows:

- **Surface Water System - $179,306 per year**
- **Groundwater System - $92,776 per year**

9.4 Construction and Capital Cost Estimated

The anticipated Groundwater System Cost Estimate is as follows:

- Well Improvements - $775,000
- Water Transmission and Distribution Main Improvements - $1,093,000
- Groundwater Treatment Facility Design and Construction - $4,482,000

Total Estimated Project Cost - $6,350,000

10. Other Items and Construction Considerations

With the proposed project's close proximity to wetlands and the Exeter River, several permits may be triggered. The information below summarizes potential permitting requirements for the construction of a pipeline as shown on the attached figure. Permitting requirements may change based on the nature of the final project to be constructed. A New Hampshire Certified Wetland Scientist should be contracted to delineate wetlands and vernal pools within the final project area and establish boundaries of prime wetlands based on Town information.

Potential permitting requirements may include:

- NH Wetlands Bureau Dredge and Fill permit.
    - Construction through or within 100 feet of prime wetlands is considered a "major" project.
    - Preliminary review by the Natural Heritage Bureau regarding rare species and exemplary natural community indicated multiple exemplary wetlands
- Building Permit and Conditional Use Permit (depending on area of impervious) - Town
- If more than 100,000 square feet are disturbed (50,000 within protected shoreland) an Alteration of Terrain permit may be required; if more than 1 acre is disturbed, a NPDES general construction permit is required.
- Federal water quality certification - for any activity that may discharge pollutants during construction or operation of the activity
- New stream crossing rules may apply to pipeline length across stream.

Although not a permit, Env-Dw 302 – Large Production Wells for Community Water Systems requirements will also apply to the Site.

Exemptions:

- Comprehensive Shoreland Protection Act (RSA 483-B:9, III)

A. Town

Category	Notes
Exeter Shoreland Protection District	Area of land within 300 feet horizontal distance to the season high water level of the Exeter River. The Gilman Well is NOT within this District; the Stadium Well and Lary Lane IS within this District. Proposed construction at basketball or volleyball courts would not be within this District. >10% impervious in a lot requires Conditional Use Permit
Floodplain Development Ordinance	All areas designated as special flood hazard areas by FEMA/FIRMs Includes both wells and the sport courts. Lary Lane is outside of 100-year floodplain All new construction/substantial improvement of non-residential structures must have the lowest floor elevated to or above the 100-year flood level or shall be flood-proofed and engineered to resist buoyancy **All proposed development in any special flood hazard area shall require a building permit.**
Zoning R-2 Flood Hazard	R-2: Single family residential, public schools, open space development. Flood Hazard: Structures shall not result in any increase in flood levels during the 100-year flood
Aquifer Protection District Code 2	Applies to both wells and potential construction areas No information found online; area is "Code 2"
Conservation Area	GIS layer, no additional information

B. **State -** Regulatory sections of interest:

<u>Env-Dw 302 – Large Production Wells for Community Water Systems</u>

Env-Dw 302.04 – Requirements for New Large Production Wells

1. The production well may not be
   a. Within 50' of normal high water line of any surface water
   b. Located less than 50' from wetlands
   c. Subject to 100-year flooding. The applicant may fill to elevate the wellhead and pumping station for flood protection purposes, provided that all required permits for placing of fill within wetland and flood plains have been obtained.

100-year floodplain elevation: 32 feet

Elevation information from CADD/Survey (updated based on 09-15-2010 field survey by Millennium Engineering)

- Stadium Well
  - Top of Well Casing: 30.22 ft
- River Pump Station
  - NE Corner: 28.66 ft
  - SE Corner: 28.64 ft
  - SW Corner: 28.60 ft
  - NW Corner: 28.60 ft
- Gilman Well
  - 3 ft from NE Corner: 27.92 ft
  - SE Corner: 30.07 ft
  - SW Corner: 30.71 ft
- Basketball Court
  - NE Corner: 32.1 ft
  - SE Corner: 32.1 ft
  - SW Corner: 32.3 ft
  - NW Corner: 32.2 ft
- Old Volleyball Court
  - Approx Center: 34.0 ft

- Door Sill: 30.70 ft

Env-Wq 1400 – Shoreland Protection

"Protected shoreland" means, for natural, fresh water bodies without artificial impoundments, for artificially impounded fresh water bodies, and for coastal waters and rivers, all land located within 250 feet of the reference line of public waters (for rivers: high water mark).

Env-Wq 1406.02 – Statutory Exemptions: Public water supply facilities, including water supply intakes, pipes, water treatment facilities, pump stations, and disinfection stations shall be permitted by the commissioner as necessary, consistent with the purposes of this chapter and other state law. Private water supply facilities shall not require a permit. http://www.gencourt.state.nh.us/rsa/html/L/483-B/483-B-9.htm

The Exeter River is protected under RSA 483:15 – *Rivers Designated for Protection* "as a ""rural river" from its headwaters at the Route 102 bridge in Chester 29.7 miles to its confluence with Great Brook in Exeter." Does not include the pumping test area.

Env-Wt 100-800: Wetlands Rules

Applies to: all surface waters of the state as defined in RSA 485-A:2 which contain fresh water, including the portion of any bank or shore which borders such surface waters, and to any swamp or bog subject to periodical flooding by fresh water including the surrounding shore.

No person shall excavate, remove, fill, dredge or construct any structures in or on any bank, marsh, or swamp in and adjacent to any waters of the state without a permit from the department.

Three permit levels – major, minor, minimum impact. Level depends on area impacted.

Env-Wt 304.13 – Utility Crossings: The potential for crossing of utility lines through wetlands along Lary Lane and the Exeter River does exist. **A NH Dredge and Fill permit would need to be filed in the event that these crossings are necessary.**

Prime Wetlands

Town regulation - all projects that are in or within 100 feet of a prime wetland are classified as **major projects**.

Others - FYI

Env-Wq 900 – Official List of Public Waters

The Exeter River, in the area of the Site, is considered a "public water": year-round flowing waters of fourth order or higher, as shown on USGS maps. Also listed on http://des.nh.gov/organization/commissioner/pip/publications/wd/documents/wd-08-9.pdf

Planning Board Submittal

It is assumed that once this project moves forward an application will be submitted to the Planning Board for review.

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