

Bourne, MA

**Preliminary Hydrogeologic Study for
Wastewater Disposal**

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1.0 INTRODUCTION

This hydrogeologic evaluation documents the investigation conducted on the subject properties owned by the Town of Bourne, MA (Plate 1). The two sites investigated include the Community Center property located at 239 Main Street, Buzzards Bay, MA and the Queen Sewell Park property located at 51 Cranberry Road, Bourne, MA. The proposed treated effluent subsurface disposal sites are currently being used as playing fields, a park, and undeveloped land.

In 1989 Bourne entered into a 20-year agreement with the Town of Wareham which allows Bourne to send up to 200,000 gallons per day (gpd) to the Wareham wastewater treatment plant. This agreement was extended for an additional 20 years in 2009. The capacity was loosely based on an allocation of 140,000 gpd from Buzzards Bay Village and 60,000 gpd from Hideaway Village based on Title V flow estimates. The Town completed construction of the necessary sewers, pumping stations and force mains in 1992. According to June 2012 *Wastewater Management Planning for Bourne's Downtown* Report, the most recent three year average suggests that 87,000 gpd of Bourne wastewater is being conveyed to the Wareham wastewater treatment plant. Since the agreement requires that Wareham approve any future connections when Bourne reaches 90% percent of their allotted capacity or 180,000 gpd, the Town has about 66,000 gpd of capacity available for future connections to the sewer system before reaching this threshold. Some of this capacity, however, is reserved for land-owners who have paid betterment fees over the past years, and the agreement with Wareham requires that 10% or 20,000 gpd of capacity be reserved for daily fluctuations in flow. In any event, Wareham has informed Bourne that there will be no additional capacity available for Bourne in the future.

As a result of the limitations discussed above, as well as a buildout analysis completed by the Cape Cod Commission (2011), the Town of Bourne, in its effort to revitalize economic development in its downtown area, authorized the Cape Cod Commission to conduct an evaluation of potential wastewater treatment and discharge sites within the town. Using decision criteria established by the Bourne Wastewater Advisory Committee (BWAC), forty-five initial parcels were screened for suitability, resulting in five wastewater treatment plant sites and preferred subsurface wastewater discharge sites selected for further analysis.

This study represents the continuation of the BWAC's effort to identify sites within the downtown area capable of handling the 335,000 gallons per day (GPD) of wastewater flow projected for the practical buildout over 15 to 25 years determined by the Commission.

This report summarizes the viability of the two sites investigated for the proposed subsurface discharge of treated wastewater. It provides a description of the work completed and the results of the hydrogeologic analyses and engineering considerations. A hydrogeologic work plan was also completed and is appended to this document as Appendix A. The work plan can be used as the first submittal to the Department of Environmental Protection to initiate the hydrogeologic portion of the Groundwater Discharge Permit (GWDP) process if the Town wishes to proceed with the recommendations in this report.

2.0 SITE SELECTION

As mentioned in Section 1 of this report, the two sites investigated include the Bourne Community Center property and the Queen Sewell Park property (Plate 1). The area of interest for the Community Center is a 4.2 Acre area located behind (northwest) of the existing Bourne Community Center Building. This area is currently being used as a sports field and skateboard park. The entire 8 Acres associated with the Queen Sewell Park property was investigated as part of the study. This parcel is currently being used as a park and recreational field.

The two sites investigated were recommended by a wastewater management study prepared for the town by the Cape Cod Commission. The study, completed in June 2012, is available online and can be viewed at the following web address.

http://www.capecodcommission.org/resources/initiatives/WastewaterMgmtPlan_BourneDowntown_FinalRept_062012.pdf

3.0 SITE DESCRIPTION

3.1 Community Center

The Community Center site is located at 239 Main Street, Buzzards Bay, MA (Figure 1). The site is improved with the Bourne Community Center building that spans across two parcels (0.97 Acres + 5.85 Acres) totaling 6.83 Acres. As the southern portion of the parcels (frontage along Main St) is built on, the area being investigated for this study is the northwestern portion of the lot that is approximately 4.2 Acres. Very little topographic relief is observed in the area of interest as the current uses of the property consist of two baseball/softball fields, a tennis court, and a skateboard park. The northern property boundary, however, does have a significant slope and approximately 27 feet of relief. A thorough review of property deeds revealed that no land use restrictions would prevent the Town from pursuing a subsurface discharge permit on the property.

Figure 1: Community Center Site Map



3.2 Queen Sewell Park

The Queen Sewell Park site is located at 51 Cranberry Road and also consists of two parcels (Figure 2). The two parcels are 2.1 Ac and 5.96 Acres in size for a total of 8 Acres. This property is located north of the Buzzards Bay Bypass (Rt. 28) from the Community Center site. This site is currently being used as a small park for children as well as a softball field, an abandoned basketball court / tennis court and wooded walking trails. This property exhibits more local variation in topographic relief than the Community Center site but less of a total range in elevation from 15 to 38 feet above mean sea level (23 ft of relief). Anecdotal information from local residents indicate that the abundance of boulders observed on the site is the result of development of the neighborhood as the site was used as a local dumping ground for large boulders excavated during road construction. Many boulders were observed both on the surface and subsurface (borings and geophysical investigations) but are not viewed as an impediment to development of a subsurface wastewater discharge facility. A thorough review of property deeds revealed that no land use restrictions would prevent the Town from pursuing a subsurface discharge permit on the property.

Figure 2: Queen Sewell Park Site Map



4.0 REGIONAL SURVEY

This section describes the location of potential receptors proximate to the two sites, including public water supplies and associated wellhead protection areas, and environmental receptors including wetlands and surface water bodies. Potential impacts to receptors were determined using the groundwater flow model as described in Section 5.0 of this report.

4.1 Public Water Supplies

A review of MassGIS data shows there are 3 public water supply wells located within one mile of the proposed treated effluent discharge site (Plate 1). The three wells consist of the following:

1. PWS ID: 4036001-02G - Community Water Well (Pump Station 2) owned and operated by the Buzzards Bay Water District that is located 3,600 feet northeast of the Queen Sewell Park site.
2. PWS ID: 4036007-01G - Transient Non-Community Well owned and operated by Sandy's Restaurant that is located 2,800 feet east northeast of the Community Center.
3. PWS ID: 4036007-02G - Transient Non-Community Well owned and operated by Sandy's Restaurant that is located 2,800 feet east northeast of the Community Center.

All three of these wells draw groundwater from the northeast and are hydraulically upgradient of the proposed discharge location(s).

Similarly, recharge zones depicted by Zone II areas (Plate 1) do not intersect the proposed disposal areas. No public surface water supply sources are known within this general region and specifically not within a one-half mile radius of the site. The Zone II areas are further discussed below while predicted groundwater movement from the proposed discharge is characterized in Section 5.0 of this report.

4.2 Private Water Supplies

Potential private water supplies downgradient of the two sites were also evaluated. First, some preliminary groundwater modeling of a 335,000 gpd subsurface discharge at each site was completed to define the area of influence downgradient of the proposed discharge for both sites. Second, the Buzzards Bay Water District was contacted to obtain the billing records of their clients in Bourne. This database was then compared to the parcel database to eliminate all parcels that were on the public water system. This information was then reviewed for accuracy by the Bourne Board of Health. Plate 1, appended displays the results of our findings both for the 'Area of Interest (downgradient)' and for the suspected private water supplies.

If the Town wishes to pursue further permitting of either site for the discharge of wastewater, these parcels would need to be confirmed. Of particular interest is National Guard Armory, the abutting parcel (downgradient) of the Queen Sewell Park site. If this property owns and operates a private water supply, it may be impacted by the discharge of wastewater at the Queen Sewell Park site.

4.3 Zone II Wellhead Protection Areas

One Zone II area falls within 1-mile of the proposed discharge location as illustrated in Plate 1. This Zone II is owned by the Buzzards Bay Water District and is associated with Pump Station 2. This Zone II is also located upgradient of the proposed discharge and will therefore not be impacted as a result of a subsurface discharge at either the Community Center or the Queen Sewell Park sites.

As illustrated in Plate 1, no Zone II wellhead protection areas overlie the area of the proposed discharge sites. Public water supply sources will not be impacted by the proposed subsurface disposal of treated effluent.

4.4 Endangered Species Habitat

The Natural Heritage and Endangered Species Program (NHESP) Atlas, 2008 edition (with updates of the vernal pools in January 2013), was reviewed for priority habitats of rare species, estimated habitats of rare wildlife, and certified vernal pools within the proposed treated effluent discharge areas. Although a total of two potential vernal pools are located within 1-mile of the proposed discharge locations (Plate 2), they are both located across the Cape Cod Canal and therefore are removed from any risk of impact from the proposed subsurface disposal of treated effluent at either site.

Downgradient of the sites investigated, both Buttermilk Bay and the Cape Cod Canal represent areas that fall within Estimated and Priority Habitats designated by the Massachusetts Natural Heritage and Endangered Species Program. If the Town wishes to pursue further permitting of either site for the discharge of wastewater, the Natural Heritage and Endangered Species Program would need to be contacted to determine what species are located in this area.

4.5 Wetland Areas

Using 1:12,000 scale mapping provided by MassGIS and the Massachusetts Department of Environmental Protection (DEP) (Plate 3), no wetland resource areas are located within 1,200 feet of either site. The closest mapped wetland is a small (0.6 Acre) salt marsh located east of Summer Street by the Cape Cod Canal. This wetland is not expected to be impacted by the proposed discharge. Further evidence and discussion of this is provided in Section 7 of this report.

4.6 Septic Systems

Elevation data of ground surface, the water table, and septic systems was collected from a variety of sources in an effort to evaluate the potential impact the proposed discharge of treated effluent would have on downgradient septic systems. The water table elevations were derived from the numerical groundwater model. The topographic data was derived from publically available LiDAR (Light Detection and Ranging) remote sensing elevation data (accurate to less than 1 ft

vertically). A GIS raster calculation was performed to provide an ‘unsaturated thickness’ map of the downgradient region of Bourne from which to assess whether sufficient unsaturated thickness was available to dissipate a groundwater mound produced by a 335,000 gpd discharge. In addition, select field data provided in Board of Health permits for septic systems was plotted spatially to ensure that a groundwater mound produced by a groundwater discharge at either of the sites would not impact the effectiveness of leaching fields downgradient of the two sites. Plate 4 is provided in an effort to show the location of septic systems evaluated. These areas are select parcels identified where the model showed less than a four foot depth to water, i.e. areas downgradient of the proposed discharge site(s) susceptible to impact from a groundwater discharge. Further discussion of this analysis is provided in Section 7.5.

4.7 Contaminant Site Review

Weston & Sampson conducted a contaminant site review of the Environmental Data Report (EDR) to evaluate the potential impact of existing nearby contamination sites on the Town’s ability to permit and construct treated effluent discharge facilities at both the Community Center and Queen Sewall Park sites. The EDR Reports for both sites are attached as Appendix B of this document. Contaminant sites posing a likely or potentially significant impact were reviewed further through communication with MA DEP personnel and others. In brief, the impact of contaminant sites located near the Community Center (RTN# 4-0016075) pose a significant impediment to the permitting requirements and cost of developing the site for subsurface discharge. In contrast, there are no significant impacts posed to the Queen Sewall Park site. A document review of contaminant sites near the Community Center revealed 11 contaminant sites within ¼ mile of the Community Center. Those contaminant sites are summarized in Table 1.

Table 1: Contaminant Sites near the Community Center

EDR Report ID	Name	Address	Relevant Reports
H20	BOURNE COMMUNITY BLDG	229-239 MAIN ST.	LUST, RELEASE
E14	BUZZARDS BAY MOBIL	246 MAIN ST.	SHWS, LUST, RELEASE, SPILLS
H21	PHILS AUTO FMR	229 MAIN ST.	LUST, RELEASE
B11	BUZZARDS BAY XTRA MART	261 MAIN ST.	UST, Financial Assurance
B7	RED TOP SPORTING GOODS	265 MAIN ST.	HW GEN
A6	TIMBERLINE EQUIPMENT INC.	270 MAIN ST.	RCRA NonGen/NLR FINDS
A1	G W DOUGLAS CO. INC.	273 MAIN ST.	HW GEN
A3		275 MAIN ST.	EDR US Hist Auto Stat
A4		279 MAIN ST.	EDR US Hist Auto Stat
C8	BOURNE SUNOCO	282 MAIN ST.	UST, Financial Assurance
D12	SUNOCO SERVICE STATION	298 MAIN ST.	LUST, RELEASE

Note: See EDR Reports in Appendix B for map of sites and explanation of acronyms.

Of particular concern is the presence of a contaminant site on the property of the Community Center itself. According to documents on file with the State, the site has been noted for a

petroleum contaminant. Weston & Sampson first contacted the Licensed Site Professional of record for the site, William Baird of Web Engineering, to determine the status of the contaminant site H20. According to Mr. Baird, he was not aware of any remediation being conducted on the site over the last five years. Weston & Sampson then contacted the MA DEP to identify the contaminant site's potential impacts to permitting and development of the Community Center for treated effluent disposal. Weston & Sampson spoke with Gerard Martin and Brian Dudley of the MA DEP Southeast Regional Office, both of whom confirmed that any contamination at the site would need to be remediated prior to any groundwater discharge permitting.

In contrast, a document review of the Queen Sewall Park site revealed no contaminant sites within ¼ mile. Based on that document review, there appear to be no contamination issues that would hinder the development of the Park site for subsurface discharge.

4.8 100-Year Flood Zone

Weston & Sampson has evaluated the likely impacts of flood zoning and corresponding regulations on the potential development of either the Community Center or Queen Sewall Park site for groundwater discharge. Based on that review, it appears that the Community Center site would not be able to support the additional construction of any above ground structures or the discharge of treated effluent below the ground surface, as those actions would impact the FEMA 100-year flood level. In contrast, the Queen Sewall Park site appears to have no flood-related impacts.

The Community Center is currently located within a FEMA flood zone of type XF, indicating the 500-year floodplain. FEMA flood mapping is continuously under revision, however, and the Bourne area has recently been remapped. While those remapped FEMA flood zones are not in effect yet, they are expected to be in the near future (2014). Based on a preliminary Flood Insurance Rate Map (FIRM), dated May 3rd 2013, the Community Center site will soon be located within a FEMA AE flood zone, **indicative of the 100-year floodplain**. Construction or development within the FEMA 100-year floodplain that would serve to increase the 100-year flood level is prohibited. In addition, according to the "Study of Flood Hazard Mitigation and Design for the Main Street Business District" final report, commissioned by the Village of Buzzards Bay in 2007, the Community Center site falls within a SLOSH zone consistent with a Category 2 hurricane. According to that 2007 Flood Study, SLOSH (Sea, Lake, and Overland Surge from Hurricanes) is a computer model developed by the National Weather Service. The SLOSH model identifies areas that would be expected to see impacts from hurricane-induced storm surges. Further, the 2007 Flood Study indicates that several political entities are considering instituting additional regulation regarding parcels subject to such storm surges. Such regulation may have additional impacts on the Community Center site, further limiting consideration for effluent disposal.

In contrast, the Queen Sewall Park site is not affected by any existing or proposed flood regulations as shown in the attached preliminary FIRM maps (Appendix C).

4.9 Conclusions

A thorough review of potential fatal flaws with respect to ecological, infrastructure, and human sensitive receptors from the perspective of permitting a groundwater discharge at both the Community Center site and the Queen Sewell Park site was conducted. This review identified potential complications with the Community Center site for the two reasons detailed above. This review did not identify any complications with respect to the Queen Sewell Park site.

5.0 WORK PERFORMED

5.1 Subsurface Investigations

One test boring was advanced at the Queen Sewell site and another at the Community Center site on October 18th and 21st, 2013 respectively, to assess the subsurface conditions. The borings were advanced using a GeoProbe® Direct Push drilling rig to obtain continuous samples in 5-foot acetate sleeves to depths of 50 feet below ground surface (bgs). Monitoring wells were then constructed in each boring with a 10 feet screen from 40-50 feet below ground surface to monitor water level at that location. No boring was advanced to bedrock refusal.

5.1.1 Queen Sewell Park

The top 23 feet (unsaturated) of Queen Sewell Park is described as 2 to 10 foot layers of various grades of stratified sands. All sands observed in this interval were poorly graded (well sorted) with some intervals observed to be of eolian (wind-blown) origin. The sands become somewhat finer at the 5 to 8 foot interval. This interval was specifically chosen for grain size analysis and sent to the laboratory for testing. Below 23 feet, there was a 1 foot layer of silty sand. This is also an interval sent for testing. Below 24 feet, the material can be characterized as very transmissive sand and gravel. A sample of this material was also chosen for grain size analysis. The piezometric surface was measured at 22.41 feet below top of casing on October 18, 2013.

The samples from the 8-10 ft, 24-25 ft interval as well as the 30-35 ft interval were collected and submitted to GeoTesting Express of Boxborough, MA for particle size distribution analysis. Boring locations and monitoring wells were surveyed using a GPS unit, and post-processed to NAD 1983 Horizontal Datum and NAVD 1988 Vertical Datum. The location of each test boring/monitoring well is provided on Plate 5. Soil boring logs are attached as Appendix D. In addition to the soil testing, water level data was observed with pressure transducers from October 30th to November 15th in an effort to understand the variability in groundwater elevations and how that might affect a groundwater discharge.

5.1.2 Community Center

Similar to the Queen Sewell Park boring, the unsaturated portion (0 – 8.5 ft) of the B2 boring advanced at the Community Center is characterized by very fine to fine poorly graded (well sorted) sand consistent with eolian deposition. Below this was a 3 ft layer of silty sand from 8.5 to 11.5 feet below ground surface, similar to 8-10 ft horizon at Queen Sewell Park. Various layers of stratified sands typical of moraine deposits were observed to a depth of 22 feet, which is underlain by coarse sand and gravel to a depth of 42.5 feet below ground surface. The bottom of the boring was characterized by silty sands from 42.5 to 50 feet below ground surface. Select samples were chosen from the B2 boring for further particle size analysis. The samples selected for analysis were from the 5-7 ft, 8.5 – 9.5 ft, and the 25-31 ft intervals. The piezometric surface was measured at 8.95 feet below top of casing on October 21, 2013.

Similar to samples from B1, the samples from the B2 boring were collected and submitted to GeoTesting Express of Boxborough, MA for particle size distribution analysis. Boring locations and monitoring wells were surveyed using a GPS unit, and post processed to NAD 1983 Horizontal Datum and NAVD 1988 Vertical Datum. The location of each test boring/monitoring well is shown on Plate 5. Soil boring logs are attached as Appendix D. In addition to the soil testing, water level data was observed with pressure transducers from October 30th to November 15th in an effort to understand the variability in groundwater elevations and how that might affect a groundwater discharge. This data is presented in Section 5.3. Monitoring well construction details are summarized in Table 2 below.

Table 2: Monitoring Well Construction Details

<i>Well ID</i>	<i>Wellhead Elevation¹</i>	<i>Boring Depth</i>	<i>Screen Depth Interval</i>	<i>Screen Elevation Interval</i>			<i>Static WL Elevation²</i>
	<i>(Top of PVC)</i>	<i>(ft)</i>	<i>(ft)</i>				<i>(ft)</i>
B1	24.68	50	40-50	-15.32	-	-25.32	2.27
B2	10.65	50	40-50	-29.35	-	-39.35	1.7

Notes:

1: Elevation measured at Top of PVC. (Based upon LiDAR remote sensing data).

2: Water Level measured from top of PVC on 10/18/13 (B1) and 10/21/13 (B2)

5.2 Geophysical Investigations

Weston & Sampson contracted Geophysical Applications Inc., to conduct three non-intrusive subsurface investigations at each of the two sites. Of particular interest in this study was a) depth to bedrock, b) presence of any confining (impermeable/semi-impermeable) layers, and c) depth to groundwater across the site. The following three sections give a brief explanation of each technique employed at the sites.

5.2.1 Ground Penetrating Radar

GPR profiling is based on the principle that materials with contrasting electrical properties reflect different radar signals back to the ground surface. Metal or concrete objects (pipes, etc.), or electrically conductive clayey strata, generally produce high-amplitude GPR reflections. Further details regarding the methodology are available in Appendix E, which includes the report from the geophysical subcontractor (Geophysical Applications Inc.).

5.2.2 Seismic Refraction Profiling

Seismic refraction profiling was performed primarily to estimate bedrock depths at each of the two survey areas. The refraction method exploits the contrasting seismic velocities between dry overburden, water-saturated overburden and bedrock, to allow calculating depths to interfaces between those materials. Further details regarding the methodology are available in Appendix E, which includes the report from the geophysical subcontractor (Geophysical Applications Inc.).

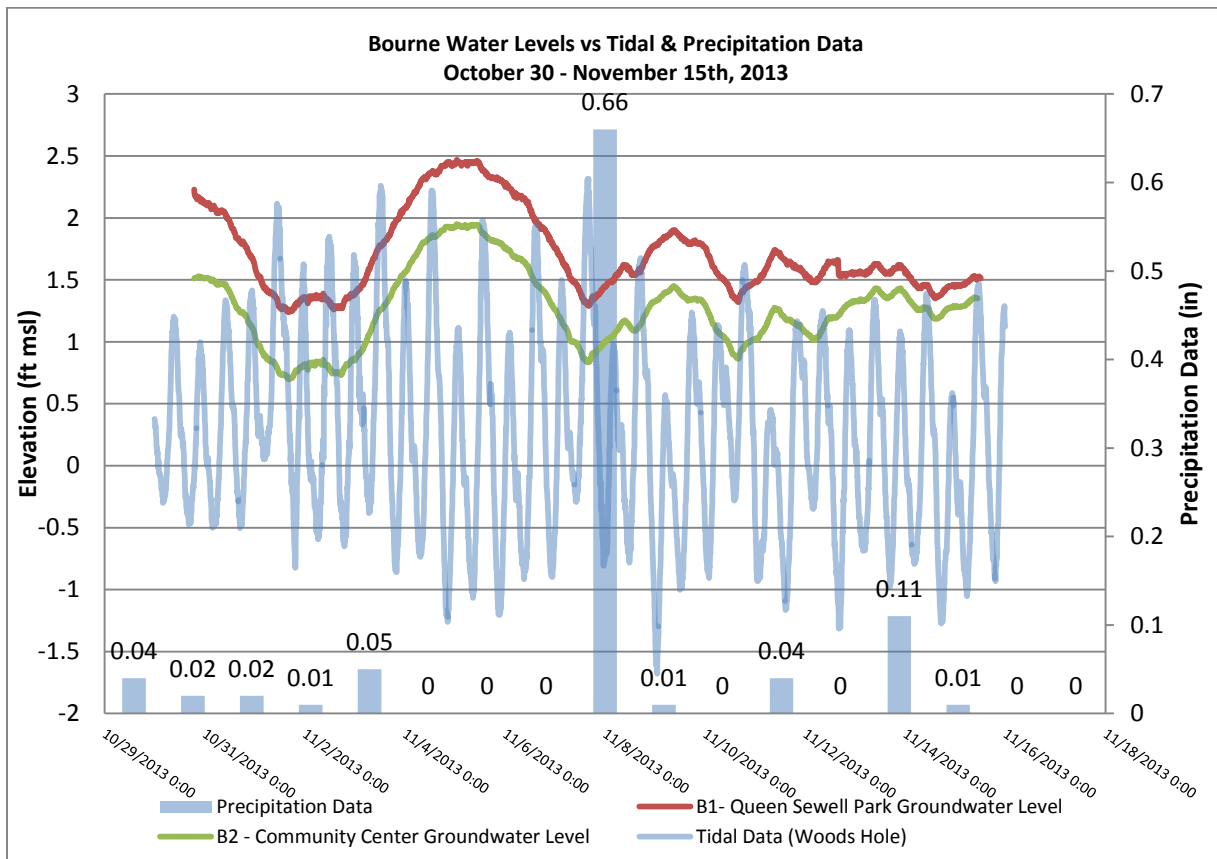
5.2.3 Resistivity Survey

Geophysical Applications Inc. also performed two-dimensional resistivity imaging to measure lateral and vertical variations in the so-called “apparent” resistivity of subsurface materials. These variations can indicate changes in soil stratigraphy, depending upon the layer depths and resistivity contrasts. Further details regarding the methodology are available in Appendix E, which includes the report from the geophysical subcontractor ().

5.3 Groundwater Monitoring

In an effort to understand the range of variability of the groundwater table, a pressure transducer was installed in each of the aforementioned groundwater monitoring wells (B1 and B2). The transducers were installed in the wells on October 30th, 2013 and retrieved on November 15th, 2013. Approximately two weeks of data was logged. Upon completion of the data collection activities, the data was corrected to groundwater elevation using the LiDAR data as an estimate for top of casing of the monitoring well. The data was then plotted on the chart below (Figure 3) to determine if fluctuations in water level were associated with tidal activity and/or precipitation events. Tidal data was obtained from NOAA Station CO-OPS 8447930 in Woods Hole, Massachusetts. Precipitation data was obtained from weather station KMABOURN12 in Bournedale, MA.

Figure 3: Water Level Data vs Tidal & Precipitation Data



As shown in the plot above, tidal effects from the nearby Cape Cod Canal and Buttermilk Bay have no impact on groundwater levels at either the Community Center or the Queen Sewell Park sites. Groundwater fluctuations are primarily attributed to precipitation events.

6.0 SITE CONDITIONS

The region surrounding and including both the Queen Sewell Park and the Community Center sites lies over what is known as the Plymouth-Carver aquifer within the Coastal Lowlands physiographic province of New England. Land surface is only 10-40 feet above sea level across both sites. The land surface consists of flat, gently sloping sand plains pitted with kettles (depressions) and moraines. Surface drainage is poorly developed due to the highly transmissive soils and shallow sloping topography that allow for quick infiltration of precipitation events. The unconsolidated deposits that comprise the Plymouth Carver aquifer were deposited in recent geologic time by a series of glacial advances and retreats. The predominant glacial features are moraines and outwash plains. The subsurface geology encountered in our field investigations was found to be consistent with moraine deposits consisting of stratified sand and gravel.

As the sites are located on a peninsula with major surface water discharge areas located on both sides (Buttermilk Bay and Cape Cod Canal), there is somewhat of a divide in the groundwater contours separating the sites. As it is not possible to determine actual groundwater gradient with one well (on each site), groundwater gradient and flow direction are derived from the regional numerical groundwater model constructed by the United States Geological Survey (USGS). In addition to the groundwater elevation and gradient, additional aquifer parameters important to evaluating the ability of a site to accept and dissipate a subsurface groundwater discharge include hydraulic conductivity and the depth to bedrock or impervious unconsolidated deposits. These parameters are discussed in further detail for each site below.

6.1 Queen Sewell Park

Estimates of hydraulic conductivity were calculated based on sieve analyses. Sediment samples collected from B1 (8-10 ft below ground surface [bgs]), (24-25' bgs), and (30-35' bgs) were submitted for sieve analysis. These samples were selected to gather information about the finer fraction of materials observed as well as the predominant soil type and to facilitate estimation of hydraulic conductivity of the sediments at depths near the water table.

Both the Fair-Hatch (Todd, 1959) and Shepard (1989) empirical estimation equations were used to calculate the hydraulic conductivity based on the sieve results. The Fair-Hatch equation estimates intrinsic soil permeability, k , (see calculations, Appendix D) which can be used to calculate hydraulic conductivity using the following equation:

$$K = \frac{k \rho g}{\mu}$$

where: K = hydraulic conductivity (m/s)
 k = intrinsic permeability (m^2)
 ρ = density of water kg/m^3
 g = acceleration due to gravity m/s^2
 μ = dynamic viscosity of water $kg/(m-s)$

Additional validation of these values was obtained by assuming the deposits are generally anisotropic and relatively well sorted, hydraulic conductivity can also be estimated using the following formula (Shepherd, 1989):

$$K \text{ (ft/day)} = (3,500d^{1.65})/7.48$$

The results are presented below in Table 3.

Table 3: Hydraulic Conductivity Estimates of Queen Sewell Park Boring (B1)

Interval (ft bgs)	Material Description		d ₅₀ (mm)	Hydraulic Conductivity (ft/d)	
	(ASTM)	AASHTO		Shepard	Fair-Hatch
5-8	Poorly graded sand (SP)	Fine Sand	0.2	33	18
8-10	NA	Gravel and Sand	0.6936	256	135
24-25	NA	Gravel and Sand	0.5407	170	171
30-35	Poorly graded sand w/ gravel (SP)	Gravel and Sand	1.9806	1,445	811

Based on the results of the geophysical techniques employed at the Queen Sewell Park site, the depth to bedrock was found to range from 130 feet to 110 feet (-94 to -113 ft msl). With a measured water table elevation of 2.27 ft msl (10/30/13), the saturated thickness at the site varies from 96 to 113 feet, depending on the location. No confining layers were evident to a depth of 35 feet below ground surface.

6.2 Community Center

Estimates of hydraulic conductivity were calculated based on sieve analyses. Sediment samples collected from B2 (5-7 ft bgs), (8.5-9.5 ft bgs), and (25-31 ft bgs) were submitted for sieve analysis. These samples were selected to gather information about the upper and lower deposits observed in the borings. The samples were sieved to facilitate estimation of hydraulic conductivity of the sediments at depths near the water table. The results are presented below in Table 4.

Table 4: Hydraulic Conductivity Estimates of Community Center Boring (B2)

Interval (ft bgs)	Material Description		D ₅₀ (mm)	Hydraulic Conductivity (ft/d)	
	(ASTM)	AASHTO		Shepard	Fair-Hatch
5-7	NA	Gravel and Sand	0.4795	139	94
8.5-9.5	NA	Silty Soils	0.0824	8	25
25-31	Poorly Graded Sand (SP)	Gravel and Sand	0.7757	308	302

Based on the results of the geophysical techniques employed at the Community Center site, the depth to bedrock was found to range from 90 feet to 105 feet (-84 to -93 ft msl). With a measured water table elevation of 1.70 ft msl (10/30/13), the saturated thickness at the site varies from 85 to 95 feet, depending on the location. No confining layers were evident to a depth of 35 feet below ground surface.

7.0 ANALYSIS

7.1 Field Size

The first step for calculation of a mound height is to determine the field size. Various site and regulatory constraints ultimately guide the field sizing effort, but this can become an iterative process with the mound height calculations and the size of the field, depending on site conditions (depth to water, impervious layers, etc). Because this is a preliminary analysis, no test pits were excavated, and therefore no percolation tests were conducted. Due to the nature of the sediments observed, however, it is reasonable to assume that the percolation rate would be on the order of < 2 min/in, which corresponds to a design loading rate of 3 gpd/ft² for subsurface leaching trenches, if infiltration testing is conducted in the permitting process.

Based upon the MADEP 2013 guideline, “Guidelines for the Design, Construction, Operation, and Maintenance of Small Wastewater Treatment Facilities with Land Disposal”, the maximum loading rate (Percolation Rate < 2 min/in) is 3 gpd/ft² (for hydrogeologic investigation based on infiltration testing). Using a leaching trench design with a 2 ft width, a 2 ft depth, and additional 1 ft of crushed stone (total 5.5 ft bury for frost protection), there is a total of 6 square feet per linear foot of leaching trench. For a proposed discharge of 335,000 gpd, there is a required leaching area (with no reserve) of 111,667 square feet using the design loading rate of 3 gpd/ft². This can be accommodated with four 210 ft x 100 ft fields containing fifty (50) one hundred (100) foot long trenches. Field size calculations are provided in Appendix D.

Pursuant to the 2013 guideline cited above, the reserve area can be accommodated with one additional offline 210 ft x 100 ft field if a combination of 1) a proven treatment process and 2) permeable soils are satisfied.

7.2 Seasonal High Groundwater Table

In an effort to estimate the seasonal high water table elevation the Frimpter method (1981) was used, as discussed below. The index well chosen (MA-BHW 198 BOURNE, MA) is the closest USGS groundwater observation point set in glacial outwash deposits.

From Frimpter :

$$S_h = S_c - \frac{S_r}{OW_r} (OW_c - OW_{\max})$$

S_h = estimated depth to probable high water level at the site (ft);

S_c = measured depth to water at site (ft);

S_r = range of water level where the site is located (ft).

OW_c = measured depth to water in observation well which is used to correlate with the water levels at the site (ft);

OW_{\max} = depth to recorded maximum water level at the observation well which is used to correlate with the water levels at the site (ft);

OW_r = recorded upper limit of annual range of water level at the observation well that is used to correlate with the water levels at the site (ft).

USGS Well	October 30, 2013
Bourne (BHW 198)	$OW_c = 33.43$ $OW_{max} = 29.57$ $OW_r = 6.60$ $S_r = 4.20$

Given this data, the seasonal high water table elevation at the Queen Sewell Park site becomes:

Queen Sewell Park
4.73 ft

7.3 Mound Height

An evaluation of mound height was conducted for the Queen Sewell Park site using both an analytical modeling approach as well as a numerical modeling approach. In general, the analytical approach is more simplistic, relying on assumptions that the aquifer is unconfined, of infinite extent, and with no slope to the water table. The numerical groundwater model is a sophisticated three-dimensional finite difference numerical groundwater model consisting of a grid of eight vertical layers and a horizontal discretization of 400 feet by 400 feet. Each grid cell has aquifer properties consistent with regional geologic mapping. The results of the two methods are described below and provide a range of solutions to guide the Town in this feasibility study. As mentioned previously, the Community Center site is not evaluated in this chapter due to the regulatory constraints determined in the regional survey portion of the study.

7.3.1 Colorado School of Mines Model

Initially calculated hydraulic parameters were input into a program developed by the Colorado School of Mines. This program uses the modified Hantush (1967) equation to estimate radial flow and mound height from a recharge source. Appendix D includes results from the mounding analysis of the Queen Sewell Park site. Table 5 summarizes the results of the mound height calculations directly under the center of the four fields and at downgradient distances from the field to understand the attenuation of the mound downgradient and its potential impact on downgradient Title V septic systems. Utilizing the field sizing information presented in Section 7.3 above, a conservative horizontal hydraulic conductivity (calculated from sieve analysis of fine sands), an initial saturated thickness of 96 feet (from seismic refraction), and discharge rates of 335,000 gpd, mound heights were predicted. Model simulations were conducted for a period of 10 years in an effort to determine the steady state mound height. As such, a specific yield (S_y) of 0.001 was selected as an aquifer parameter for the modeling run.

Table 5: Mound Height (Analytical Model)

Distance from Field Center (ft)	Site Reference ¹	Predicted Mound Height (ft)
0	Center of Field	2.36
500	I	1.92
1000	L	1.73
2200	R	1.52

1. Site Reference refers to sites located on Plate 4.

7.3.2 USGS Plymouth-Carver-Kingston-Duxbury Aquifer Model

A modified version of the published USGS Plymouth-Carver-Kingston-Duxbury Aquifer Model (SIR 2009-5063) was used to evaluate the study area. This model was developed on the industry-standard MODFLOW computer software developed by the U.S. Geological Survey by McDonald and Harbaugh (1984). The model simulates three-dimensional flow using a block-centered finite difference grid.

The MODFLOW program is utilized to simulate groundwater flow. A pre- and post-processor known as Groundwater Vistas® (GV) was used to develop input files and for plotting model results. GV is a graphical interface for input and analysis of MODFLOW and MODPATH data. Full model documentation is provided in the USGS Scientific Investigation Report (SIR 2009-5063).

Weston & Sampson modified the model in an effort to simulate the proposed discharge of 335,000 gpd of treated effluent at the Queen Sewell Park site to determine mound height and potential impacts to environmental receptors. The only modification consisted of altering the recharge value of one 400 x 400 ft cell that represents the Queen Sewell Park site to a value of 0.28 ft/day, calculated as follows:

$$335,000 \frac{\text{gal}}{\text{day}} \times \frac{1 \text{ ft}^3}{7.48 \text{ gal}} \times \frac{1}{(400 \text{ ft} \times 400 \text{ ft})} = 0.28 \text{ ft/day}$$

Of particular interest in modeling the discharge was the elevation of the static water table in the model under a no-discharge condition. The static water table at the location of the B1 boring in the model was found to be 4.2 ft above mean sea level (msl). This elevation is similar to the calculated seasonal high groundwater table of 4.73 ft msl presented in Section 7.2, therefore simulating a worst case condition with respect to mound height in the model.

Prior to running the model, a comparison of aquifer parameters (specifically hydraulic conductivity) was conducted to verify that the model was representative of the site-specific information collected on the Queen Sewell Park property. As such, the following table was prepared to compare the top two layers in the model to the average hydraulic conductivity estimates calculated based upon the particle size distribution data from the boring samples.

Table 6: Comparison of Field Data to USGS Numerical Groundwater Model

Model Layer	Top Elevation (ft)	Model Data			K _{xy}	K _z	Field Data (Queen Sewell Park)		
		Bottom Elevation (ft)	Layer Thickness (ft)	Depth (ft)			Boring Depth (ft)	K Estimate (ft/day)	Layer Average (ft/day)
Layer 1	2	-8	10	10	227	46	5-8	25	110
							8-10	195	
Layer 2	-8	-28	20	30	227	46	24-25	170	649
							30-35	1,128	

Table 6 presents model information on the left and field data on the right. A “layer average” hydraulic conductivity was calculated based upon an average of particle size distribution samples that were located within the physical horizon that corresponds to a simulated model layer. The table shows that Layer 1 over-estimates the hydraulic conductivity in the model and Layer 2 significantly underestimates the hydraulic conductivity in the model. The recommendation is to strip the top 10 feet from the elevation of B1 down to the elevation of the softball field at the park. This would be commensurate with the Layer 2 in the model, which is simulated as a lower hydraulic conductivity than calculated from site specific information. The resulting interpretation is that the model simulates a conservative estimate of mound height.

When the 5-year transient model run was conducted, a maximum mound height of 1.25 feet was simulated at the center of the four fields.

Table 7 presented below summarizes the results of the mound height calculations directly under the center of the four fields and at downgradient distances from the field to understand the attenuation of the mound downgradient and its potential impact on downgradient receptors.

Table 7: Mound Height (Analytical Model)

Distance from Field Center (ft)	Location Description ¹	Predicted Mound Height (ft)
0	Center of Field	1.25
500	I	1.0
1000	L	0.75

1. Site Reference refers to sites located on Plate 4.

7.4 Groundwater Mounding Results

Groundwater mounding calculations were performed to verify the engineering design of the subsurface effluent disposal based upon the design flow of 335,000 gpd. The effect of the disposal of treated effluent was based on a design loading rate of 3 gpd/ft² in four 210 x 100 foot leaching fields (Table 3 of Guidelines for the Design, Construction, Operations, and Maintenance of Small Wastewater Treatment Facilities with Land Disposal, 2004). The models were run to simulate the effect of the proposed discharge to the water table. The groundwater flow models predicted a maximum mound height of 1.25 – 2.36 feet directly below the center point of the four fields.

Using the information presented above, the following conceptual diagram (Figure 4) was prepared to illustrate the design parameters for a leaching trench at Queen Sewell Park if one

considers the softball field elevation of 16.68 ft above mean sea level to be the surface elevation across the site.

Figure 4: Conceptual Drawing of Maximum Allowable Mound Height at Queen Sewell Park.

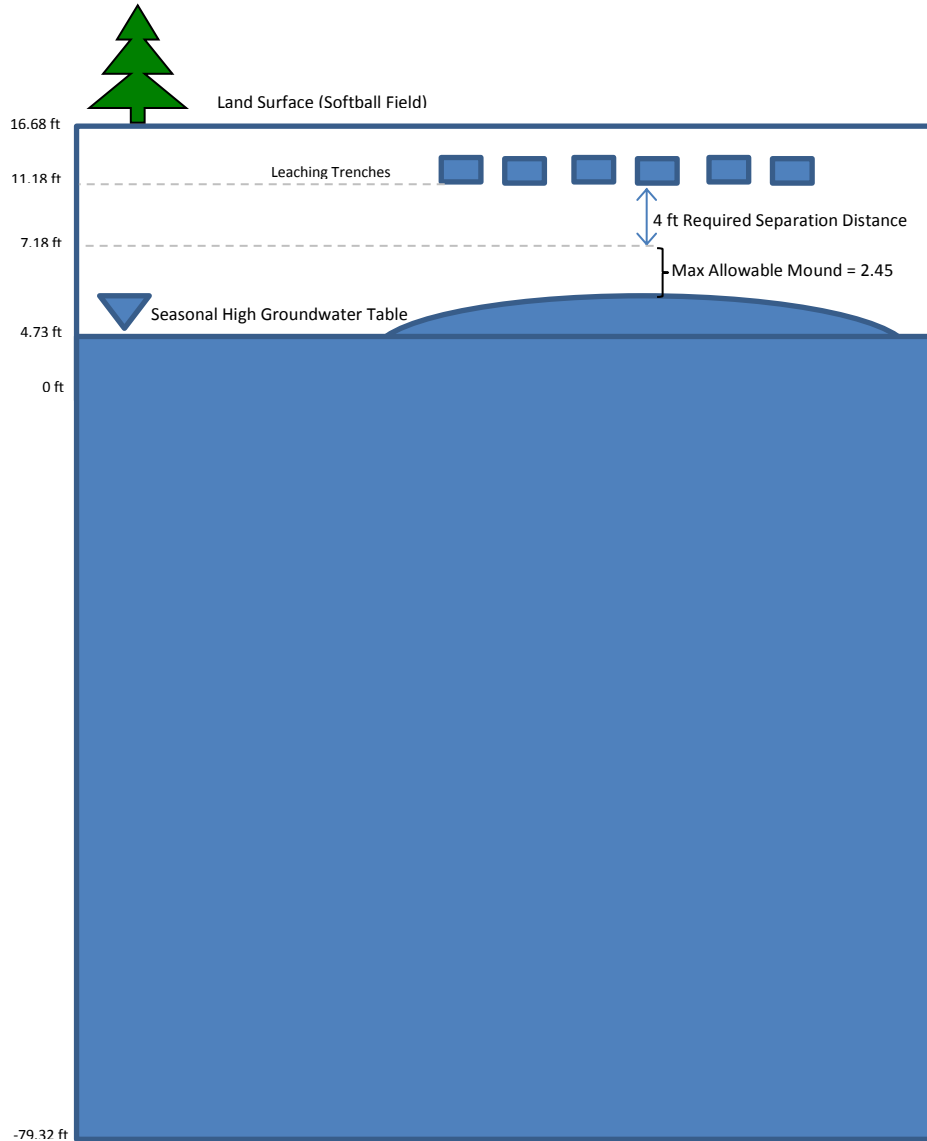


Figure 4 illustrates the maximum allowable mound height at the Queen Sewell Park site if the elevation of the existing softball field is taken as the starting land surface elevation.

It is important to note that if more extensive site investigation is conducted in support of any groundwater discharge permitting efforts at Queen Sewell Park; the site will be re-modelled with the newly acquired information. If that information and modeling predicts a higher mound than predicted herein, the trench system can be raised if more Title V sands can be brought in to raise the elevation of the property. Onsite sands may also be viable due to the extent of the proposed regrading.

7.5 Impacts from Groundwater Discharge

7.5.1 Water Quality

Once the numerical groundwater model was run, an additional post-processing package (MODPATH) was run using the results of the flow model in an attempt to quantitatively determine the path and ultimate discharge location(s) of the proposed discharge (flow splits). MODPATH is a particle-tracking post-processing package that was developed to compute three-dimensional flow paths using output from steady-state or transient ground-water flow simulations by MODFLOW. Essentially, the module uses a semi-analytical particle tracking scheme that allows an analytical expression of the particle's flow path to be obtained within each finite-difference grid cell. Particle paths are computed by tracking particles from one cell to the next until the particle reaches a boundary, an internal sink/source, or satisfies some other termination criterion. A pathline is the projected two-dimensional representation of the mean trajectory an individual fluid particle follows through the flow domain. The pathline describes the expected planar location and time of travel of the fluid from any initial location, along its trajectory, to its terminus (ex: Well or flow domain boundary).

The MODPATH model run was started by 'seeding' the location of the discharge beds in Queen Sewell Park with 60 evenly spaced particles in Layer 1 of the model. This simulates the distribution or discharge of treated effluent on the leaching trenches. The MODPATH model was then run for the 5 year transient model run to determine the fate of the particles that simulate the discharged treated effluent.

The results of the MODPATH model run are presented on Plate 5. The aforementioned 60 particles flowed downgradient perpendicular to the modeled piezometric surface, ultimately discharging into both Buttermilk Bay and the Cape Cod Canal. The calculated flow splits resulted in 32% of the treated effluent discharging into Buttermilk Bay and 68% into the Cape Cod Canal.

In May 2009, the US EPA Region 1 approved pathogen Total Maximum Daily Loads (TMDLs) for 52 areas in the Buzzards Bay watershed. These TMDLs set water quality standards in 45 estuaries and seven river areas and will have broad implications for the issuance of discharge permits. The Buzzards Bay Pathogen TMDL is the result of years of effort by DEP, which "is responsible for monitoring the waters of the Commonwealth, identifying those waters that are impaired, and developing a plan to bring them back into compliance with the Massachusetts Water Quality Standards (WQS). The list of impaired waters, better known as the '303d list', and now part of the Integrated List of Waters, identifies problem lakes, coastal waters and specific segments of rivers and streams and the reason for impairment. "

Both of the receiving water bodies, the Cape Cod Canal and Buttermilk Bay, are on the 303d list as impaired waterways. Buttermilk Bay has been labeled as a Category 5, which means it has been labeled as "impaired" and a TMDL is said to be "required" for this water body. The Cape

Cod Canal is labeled as a Category 3 as a result of pathogens and a TMDL has already been set. (Plate 6).

Given the constraints of the receiving water bodies, various mitigation strategies may be required by the DEP to permit this discharge. There are many possible mitigation strategies to assess that are beyond the scope of this project. A few strategies to consider include:

- Lower effluent discharge concentration,
- Removal of downgradient Title V systems by sewerage,
- Reduction of lawn fertilization in the drainage basin, and
- Stormwater treatment systems.

Given an effluent discharge Nitrogen concentration of 5 mg/L, it is predicted that there will be an increased Nitrogen load of approximately 3,400 lbs/year to the Cape Cod Canal and approximately 1,600 lbs/year to Buttermilk Bay from a 335,000 gpd system.

7.5.2 Wetlands

The numerical groundwater model was used to assess the assimilation of treated effluent into the proposed leaching trench system. The model is a computer code that solves the groundwater flow equation, a mathematical relationship that is used to describe the flow of groundwater through an aquifer. Each cell in the model has properties and boundary conditions to compute the flux of water into and out of cells. The model does not, however, calculate surface water flow, which makes interpretation of the interaction with surface water bodies, such as rivers, streams, and especially wetlands, complicated. The only wetland identified in Section 4.5 is located cross-gradient from the discharge location and our modeling efforts do not show any impact on this wetland by discharge at the Queen Sewell Park site.

7.5.3 Title V Systems

Weston & Sampson also evaluated the degree to which the water table might rise as a result of these discharge scenarios in downgradient residential neighborhoods. Table 8, along with Plate 4, provides a guide to potential impacts to downgradient Title V septic systems.

Impacts to the water table beneath the downgradient residential neighborhoods were evaluated at sixteen locations: eight locations along the near side of the neighborhood, four locations near the middle, and four locations on the far side of the neighborhood. The table below (Table 8) provides a summary of the magnitude and distribution of impacts to the water table in that area as a result the discharge.

Weston & Sampson reviewed 25 representative septic system plans from the Bourne Health Department to further investigate potential impacts to septic systems in the downgradient neighborhoods. Health Department data for several locations distributed through the neighborhood (shown on Plate 4) was reviewed to evaluate potential impacts to septic systems from the proposed groundwater discharge. A summary of the data reviewed is included in Table 8 below.

Table 8: Bourne Health Department Data Review Summary

Site	Elevation of Bottom of Trench (ft) ¹	Predicted Mound Height (ft)	Modeled Water Table (ft)	Distance between Bottom of Trench and Water Table (no mound) (ft)	Distance between Bottom of Trench and Water Table (with mound) (ft)
J	18.5	0.75	2.2	16.30	15.55
Y	-8.5	0.5	2.2	-10.70	-11.20
P	10.2	0.88	2.8	7.40	6.52
S	9.6	0.88	2.9	6.70	5.82
F	18.2	0.88	3.8	14.40	13.52
W	6.8	0.75	2.8	4.00	3.25
T	5.5	1	3	2.50	1.50
B	26.4	0.75	3.7	22.70	21.95
G	17.1	0.88	3.9	13.20	12.32
C	18.8	0.88	3.9	14.90	14.02
M	13	0.88	4	9.00	8.12
L	15.2	0.75	3.7	11.50	10.75
V	11.2	0.88	4	7.20	6.32
D	16.3	0.75	2.7	13.60	12.85
I	15.3	1	4	11.30	10.30
O	13.2	1.13	3.2	10.00	8.87
H	19.3	0.88	2.9	16.40	15.52
Q	9.55	0.88	3.1	6.45	5.57
E	21.1	0.63	2.7	18.40	17.77
K	14.8	1.13	3.2	11.60	10.47
A	27.7	0.75	2.8	24.90	24.15
R	6.2	1.13	3.2	3.00	1.87
N	12.8	1.25	3.3	9.50	8.25
X	2.9	0.5	2.4	0.50	0.00
U	7.8	0.5	2.4	5.40	4.90

¹ Elevation calculated based on information from Bourne Health Department records.

As shown in Table 8 above, four of the twenty-five queried properties were calculated to be out of compliance without an additional groundwater discharge. Five properties were determined to be out of compliance with the addition of a groundwater discharge. The properties were specifically chosen in areas a) downgradient of the proposed discharge, and b) demonstrating a depth to water of less than 4 feet according to a comparison of LiDAR elevation data and water table data derived from the numerical groundwater model. From this analysis, it does appear that additional site specific work may be necessary to a) identify additional Title V septic systems impacted by a groundwater discharge and b) verify those properties that appear to be impacted if future permitting is pursued.

8.0 CONCLUSIONS & RECOMMENDATIONS

8.1 Study Conclusions

1. The Queen Sewell Park site appears to be a favorable location for the subsurface discharge of 335,000 gpd or less of treated wastewater effluent.
2. The Community Center property has existing contamination on-site, is located in the newly proposed 100-year flood zone, and has limited unsaturated thickness available to attenuate a groundwater mound. This site was therefore eliminated from further analysis.
3. Regional survey results indicate:
 - a. No public water supplies or Zone II Wellhead Protection Areas are located downgradient of a proposed subsurface discharge at Queen Sewell Park.
 - b. Both receiving waters (Buttermilk Bay and the Cape Cod Canal) represent endangered species habitat as well as impaired water bodies on the 303(d) list of impaired waters.
 - c. No wetland areas are located downgradient of a proposed subsurface discharge at Queen Sewell Park.
 - d. One Title V septic system (reviewed) was identified to have less than the required four feet of separation distance between the bottom of the soil absorption system and the seasonal high groundwater table with the resultant groundwater mound from the subsurface discharge of 335,000 gpd.
 - e. The Community Center site has an active release tracking number from an historical release of gasoline on the property. This contaminated site would need to be remediated prior to a permit for groundwater discharge would be approved.
 - f. The Community Center site is located within the soon to be adopted FEMA 100-year floodplain. Discharge of treated effluent within a 100-year floodplain would not be permitted by the DEP.
4. Hydrogeologic analysis results for Queen Sewell Park indicate:
 - a. Aquifer parameters (hydraulic conductivity, saturated thickness) are favorable for subsurface discharge of treated effluent.
 - b. Four 210 ft x 100 ft fields would be sufficient to discharge 335,000 gpd of treated effluent. One additional 210 ft x 100 ft field would be required for a reserve area. All fields can be accommodated by the Queen Sewell Park site construction constraints.
 - c. The calculated seasonal high groundwater table is 4.73 ft above mean sea level.
 - d. Maximum predicted mound height is 2.36 ft at the center of the four fields.
5. Potential impacts from a discharge at the Queen Sewell Park Site include:
 - a. Potential water table impacts to Title V systems. Although only a few systems were identified as being either non-conforming or close to the

required four-foot separation distance, some system modifications may need to be incorporated into an overall mitigation strategy.

- b. Mitigation strategies for nitrogen loading to already impaired receiving water bodies (Buttermilk Bay and Cape Cod Canal) will likely need to be addressed. This mitigation can be phased as wastewater flows increase over time. Stakeholder involvement through both the MEPA process and the Groundwater Discharge Permit will be the key to eventual acceptance and use of the Queen Sewell Park Location.

8.2 Recommended Effluent Disposal Configuration

The proposed effluent disposal system would be designed as subsurface leaching fields located on the southeast area of the Queen Sewell Park site. Based on the modeling completed, the proposed configuration of the effluent discharge system would optimize the flow toward the Cape Cod Canal rather than the Category 5 (303(d)) list) water body (Buttermilk Bay).

All modeling undertaken in this study assumes the existing property will be excavated to an elevation of greater than 16.68 and clean filter sand material will be stockpiled on site to be evaluated for possible reuse.

8.3 Recommended Effluent Disposal Reserve Area

The initial designs shown in this report will need refinement pursuant to the 2013 “Guidelines for the Design, Construction, Operation, and Maintenance of Small Wastewater Treatment Facilities with Land Disposal”. The reserve area can be accommodated with one additional offline 210 ft x 100 ft field if a combination of 1) a proven treatment process, and 2) permeable soils are satisfied.

Glossary of Terms

Finite Difference – In mathematics, finite difference methods are numerical methods for approximating the solutions to differential equations using finite difference equations to approximate derivatives.

Internal Sink – To calculate the mass balance of a transient groundwater system using Darcy's Law, an internal sink accounts for mass lost from the system. In a groundwater system, this may include a pumping well or a discharge to a surface water body.

Internal Source - To calculate the mass balance of a transient groundwater system using Darcy's Law, an internal source accounts for mass gained in the system. In a groundwater system, this may include precipitation or a wastewater discharge.

Transient Modelling - Groundwater flow models describe their capabilities as either steady state and/or transient. Steady state flow occurs when the magnitude and direction of flow is constant with time throughout the entire domain. Conversely, transient flow occurs when the magnitude and direction of the flow changes with time. In other words, the hydraulic head doesn't change with time in a steady state flow system, but does change during transient flow. This does not mean that in a steady state system there is no movement of groundwater, it simply means that the amount of water within the domain remains the same, and that the amount of water that flows into the system, is the same amount as flows out.

Anisotropic - is the property of being directionally dependent, as opposed to isotropy, which implies identical properties in all directions. It can be defined as a difference, when measured along different axes, in a material's physical or mechanical properties (absorbance, refractive index, conductivity, tensile strength, etc.)

APPENDIX A

APPENDIX B

APPENDIX C

APPENDIX D

APPENDIX E