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HYDROGEOLOGIC EVALUATION
PROPOSED EFFLUENT DISPOSAL FACILITIES
223 BEACH ROAD
TOWN OF ORLEANS, MASSACHUSETTS

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1.0 INTRODUCTION AND BACKGROUND

1.1 Introduction and Purpose

The Hydrogeologic Evaluation Technical Memorandum documents the process used to evaluate 223 Beach Road (Figure 1) for a groundwater discharge from the proposed Meetinghouse Pond area wastewater treatment facility (WWTF). This Memorandum includes the following:

- Description of the initial process that was employed to develop the Orleans Consensus Plan and associated potential groundwater discharge sites;
- Description of the initial steps taken in the process of evaluating groundwater discharge sites;
- Summary of existing data that is available to help evaluate the 223 Beach Road site;
- Summary of field investigations conducted at 223 Beach Road site;
- Evaluation of field investigations and other available data;
- Results of groundwater flow modeling and groundwater mounding analysis; and
- Recommended maximum discharge capacity for the 223 Beach Road Site.

The purpose of this document is to provide a transparent and objective assessment of the 223 Beach Road Site for the discharge of WWTF effluent. The Hydrogeologic Evaluation will be submitted to the Massachusetts Department of Environmental Protection (MassDEP) as part of the Groundwater Discharge Permit (GWDP) application process.

1.2 Consensus Plan Description

The Orleans Water Quality Advisory Panel (OWQAP) was convened to achieve consensus and build widespread community support for a customized, affordable water quality management plan for the Town of Orleans. The panel consisted of stakeholder representatives (Orleans Selectmen and representatives of engaged citizen constituencies), and liaisons from key town boards and commissions, organizations, neighboring towns, and regional, state, and federal partners. The OWQAP met for twelve half-day meetings starting in July 2014, all of which were open to public attendance and comment.

Potential alternative planning scenarios to meet water quality standards were developed for the OWQAP and presented at meetings and workshops. Initially, a Hybrid Plan was designed that included specific sites for aquaculture and coastal habitat restoration (CHR), as well as permeable reactive barriers (PRB) and floating constructed wetlands (FCW). The number of acres of shellfish growing area, as well as linear feet of PRBs and square footage for FCW, were quantified to achieve specific nitrogen-removal targets. This exercise was undertaken to ensure that the quantities of shellfish (and all other non-traditional technologies) proposed in the Consensus Plan were feasible to install. These specific locations became the basis for potential demonstration site locations for aquaculture and CHR, as well as FCWs and PRBs.

Figure 1 – Locus

The Hybrid Plan was vetted through the OWQAP during three meetings, including a day-long workshop. This iterative process resulted in a draft Consensus Plan that included a combination of non-traditional and traditional technologies. Once the feasibility of using shellfish and other non-traditional technologies as part of the Town's nutrient management strategy was established, the OWQAP decided that the final Consensus Plan would not specify exact growing locations, but instead focus on overall area of shellfish and other alternative technologies needed to remove the appropriate mass of nitrogen at the watershed level.

The resulting map (Figure 2), entitled Conceptual Approach to Meet Orleans Water Quality Goals (March 2015) shows the agreed upon water quality management plan and includes 5.5 acres of shellfish in the Nauset Harbor watershed and 9 acres of shellfish in Pleasant Bay. Neither coastal habitat restoration nor aquaculture is part of the plan for the Rock Harbor watershed. This map also specifies acreages for FCW and linear feet of PRBs.

1.3 Initial Process of Site Identification

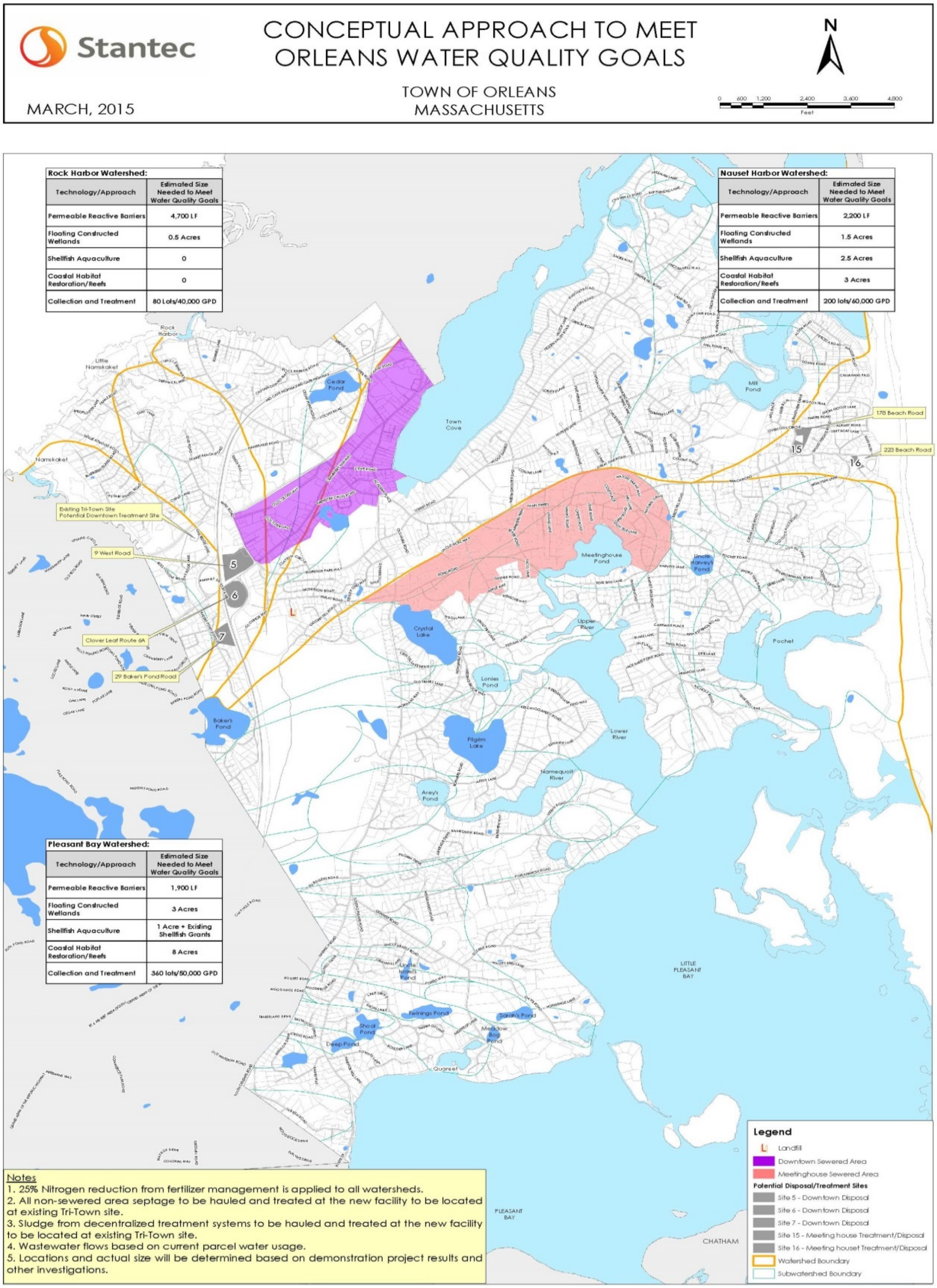
As part of updating the 208 Plan, The Cape Cod Commission (CCC) created Traditional and Non-Traditional Scenarios that would meet the regulatory requirements for nitrogen, formalized as Total Maximum Daily Loads (TMDLs) for Orleans' impaired water bodies. The Traditional Scenario for Orleans used centralized sewers exclusively. The Non-Traditional Scenario met nitrogen-removal goals through a subset of the many alternatives that are described in the 208 Plan's Technology Matrix. The subset of technologies in the Commission's Non-Traditional Scenario included PRBs, FCWs, CHR, shellfish aquaculture, fertigation, composting and urine-diverting toilets and innovative/alternative septic systems. In order to ensure consistency with this established regulatory framework, the Non-Traditional Scenario developed by the Commission became the starting point for customizing a non-traditional bookend for the OWQAP and consensus-building process.

This planning and design process for tailoring a non-traditional bookend for Orleans included studying the information prepared by the CCC, and collecting and analyzing a significant amount of additional local data that was not reviewed as part of the regional planning process undertaken by the Commission. Local data from satellite images, geographic information system (GIS) maps, groundwater maps, and coastal pond bathymetry data was reviewed. Paper records on the history of local aquaculture, and Town shellfish propagation were aggregated into a database for trending and other analyses. Site visits both by land and by water were conducted to validate locations for shellfish aquaculture and CHR. Interviews with the Orleans Shellfish Constable, the former Shellfish Constable, and local shell fishers were also conducted to verify initial findings.

This local data was used to:

- Evaluate depth to groundwater and aquifer thickness for PRB installations;
- Assess roads and neighborhoods for PRB installations;
- Classify water bodies in terms of suitability for aquaculture and/or CHR based on water quality data contained in Massachusetts Estuaries Project (MEP) Reports and data synthesis from the Pleasant Bay Alliance;
- Inventory potential and existing use conflicts (boating, moorings, aesthetic preferences);
- Identify specific areas for shellfish growing within waterbody; and
- Recommend different species for specific areas, including quahogs, oysters, and mussels.

Figure 2 – Consensus Plan



This local data collection and evaluation allowed the Non-Traditional Bookend for Orleans to be based on key validated site parameters, ensuring that the non-traditional technologies were feasible in their planned locations. In addition, a Technical Memorandum on Non-Traditional Technologies (Appendix A) was prepared and submitted to the OWQAP. This Technical Memorandum detailed initial performance expectations, as well as key site and permitting considerations that should be used to verify the usefulness of these technologies for specific subwatersheds in Orleans.

The results of this detailed analysis and resulting initial locations for non-traditional technologies were presented and thoroughly discussed during the October 8, 2014 OWQAP Stakeholder meeting. Based on this technical review, as well as direction from the OWQAP, specific non-traditional technologies were then selected to be used to create a “Hybrid Plan” that included both non-traditional as well as traditional technologies for Orleans. The Hybrid Plan showed both technologies in specific locations in order to verify that appropriate nutrient loads could be removed.

1.4 Hybrid Plan Site Identification Criteria Used During OWQAP Process

During a day-long OWQAP public workshop on December 17, 2014, the Hybrid Plan was presented, screened, and evaluated. This plan described a combination of traditional and non-traditional technologies that meet the MEP load-reduction targets for nitrogen in each impaired waterbody. The OWQAP then formed three subgroups to discuss, evaluate and revise the Hybrid Plan. To assist in this process, the OWQAP received a Technology Evaluation Decision Support Tool that allowed risks and benefits of each technology to be evaluated by subwatershed. Preliminary comparative costs were also presented on a relative dollars/kilogram of nitrogen removed basis. Spreadsheets with ranking for each subwatershed are included in Appendix B.

Ranked categories include:

- Nutrient removal certainty: nitrogen (saltwater), phosphorus (freshwater);
- Implementation certainty;
- Other benefits: ecosystems, economic, social;
- Adaptability to uncertainty in nutrient-reduction goals and build-out; and
- Contaminants of emerging concern (CEC) removal.
- Overall cost

Based on these criteria, two areas were identified for wastewater collection, treatment and discharge. These areas were identified as the Downtown and Meetinghouse Sewered Areas (Figure 2). The following WWTF effluent discharge sites were identified as part of the process of defining the Consensus Plan:

- 9 West Road;
- Cloverleaf Route 6A;
- 29 Baker Pond Road;

- 178 Beach Road; and
- 223 Beach Road.

1.5 Site Review and Shortlisted Sites

The initial groundwater discharge sites, taken from the Hybrid Plan developed during the OWQAP process were further evaluated by the Town. Two locations were initially shortlisted for hydrogeologic evaluation; Cloverleaf Route 6A and 223 Beach Road. These sites met the goals and objectives of the OWQAP's Consensus Plan.

The Town of Orleans did not own the parcel within the southeast cloverleaf of the Route 6 Interchange (Exit 12) with Route 6A. In the fall of 2015, the Town approached MassDOT for permission to access the Cloverleaf site to perform a hydrogeologic investigation. When a site access agreement could not be obtained, the Cloverleaf Site was dropped from consideration in mid-January, 2016. Alternative groundwater discharge sites are being considered.

The 223 Beach Road site is located in east Orleans near Nauset Beach. The parcel is owned by the Town. On November 30, 2015, a proposed scope of work to conduct a Hydrogeologic Site Evaluation was submitted to MassDEP for review and comment. The notification of the proposed scope of work was published in the December 23, 2015 Environmental Monitor. The proposal was open to public comment. The Proposed Hydrogeologic Site Evaluation was approved by MassDEP on January 19, 2016 after receiving no public comments. Copies of the proposed Hydrogeologic Evaluation scope of work, Environmental Monitor Notification and MassDEP approval letter are provided in Appendix A. The following Technical Memorandum reports on the scope of work, methodology, findings, conclusions, and recommendations of the 223 Beach Road Hydrogeologic Evaluation.

2.0 PROPOSED COLLECTION, TREATMENT AND DISCHARGE FACILITIES

(Additional detail will be added to this section in the final submittal)

2.1 Collection Area and WWTF

MEP estimates that 100 percent of the nitrate load from existing and future septic systems will need to be removed from the Meetinghouse Pond Watershed to meet the nutrient load reduction target for Meetinghouse Pond. The OWQAP evaluated traditional and non-traditional nutrient reduction technologies for nitrate removal within the watershed. Due to the high percentage of nitrate removal and the limited potential for use of non-traditional technologies within the Meetinghouse Pond Watershed, the OWQAP proposed to manage a majority of the nitrate load through the construction of wastewater collection, treatment, and disposal facilities in the Consensus Plan. Flow from the collection area will be primarily residential and commercial. Flows are estimated at 50,000 to 60,000 gallons per day (gpd). The proposed collection area is shown on the Consensus Plan map (Figure 2). The location of the proposed WWTF was being evaluated at of the time of this technical memorandum. The location of the proposed discharge site, 223 Beach Road, is shown on Figure 3.

2.2 Effluent Disposal – Primary and Secondary Discharge Area

The Beach Road site was considered for two primary reasons. First, the site is owned by the Town; second, the site is located outside the Meetinghouse Pond Watershed and discharges to the Atlantic Ocean and not to an estuary where the nitrate in the effluent could be a potential issue (Figure 3). The Town also plans to use the site for Nauset Beach parking. Installation of a subsurface discharge at the site would not interfere with these plans.

The potential discharge area at 223 Beach Road is shown on Figure 4. The potential discharge area is approximately 140,000 square feet (sqft). At this time the projected flow and location of the primary and reserve discharge areas within the highlighted area has not been determined. However, at a flow of 60,000 gpd, approximately 30,000 sqft would be required for the primary discharge with the reserve located between the laterals of the primary discharge.

Based on the soils investigations performed by AECOM, the primary and reserve discharge leaching facilities will be designed using a percolation rate less than 2 minutes per inch. This percolation rate has been verified by percolation tests conducted on January __, 2016. As required in the MassDEP’s “Guidelines for the Design, Construction, Operation, and Maintenance of Small Sewage Treatment Facilities with Land Disposal”, this percolation rate combined with leaching chamber trenches, provides an allowable loading rate of 3 gallons per day per square foot (gpd/ft^2) of effective leaching surface area. Therefore, a total of __ square feet of leaching area (__ linear feet of Infiltrator Chamber) is required.

Figure 3 – 223 Beach Road Site – Site Map

Figure 4 – Primary and Reserve Discharge Areas

The potential discharge area, including 25-foot buffer around each area, is shown on Figure 4. The actual configuration and layout of the area will be based on the topography, results of the hydrogeologic investigations and the results of the groundwater mounding analysis. The trenches will consist of 1.33’ x 2.83’ x 100’ High Capacity Infiltrator (or approved equal) leaching chamber trenches as shown on Figure 5. It is anticipated that the leaching chambers will be arranged in a trench format with approximately __ feet of separation between the centerline of the primary discharge trenches. Should two discharge areas be proposed for the site, it is recommended that a minimum of 25 feet be provided between each area for the necessary piping and appurtenances.

The effluent disposal chamber trenches will be installed to provide a minimum of 4 feet of vertical separation to the groundwater mound that is predicted to develop beneath the system. The groundwater mound at each of the proposed effluent disposal sites is discussed in Section 5.7.

2.3 Effluent Disposal - Buffer Area

A 25-foot buffer has been reserved around each proposed effluent disposal area. The buffer area is for future maintenance of the discharge infrastructures. The width of the buffer area has been approved by the MassDEP. The buffer areas are shown on Figure 4.

Figure 5 – Leaching Chamber Trenches

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3.0 HYDROGEOLOGIC INVESTIGATION

(Additional detail will be added to this section in the final submittal)

Results of the hydrogeologic investigation are included in the following sections. Investigations included the excavation of test pits, performing Title 5 percolation tests, the installation of soil borings and monitoring wells, grain-size analysis of soil samples, and the performance of slug tests. The data obtained were used to evaluate subsurface conditions, estimate the groundwater flow direction and calculate aquifer characteristics. Results of the field investigations and data analysis were incorporated into a numerical groundwater flow model to simulate groundwater flow across the site and estimate groundwater mounding under various discharge scenarios. A summary of these investigations follows.

3.1 Previous Subsurface Investigations

In April 2000, the Cape Cod Commission contracted with Hardin-Knight Associates, Inc. to investigate soils at the proposed discharge site. A copy of the report including soil boring logs, well construction diagrams, geologic cross sections, and ___ is provided in Appendix __.

3.2 Test Pit Excavation and Percolation Tests

On February 18, 2016 a total of __ test pits and __ percolation tests were conducted at the proposed discharge site. The test pits were performed to evaluate the overall suitability of the subsurface soils for the proposed discharge. The test pits were excavated to depths of between __ and __ feet.

Test pits TP-1 through TP-6, were witnessed by MassDEP and the Town of Orleans Health Department. All test pits and percolation test were excavated under the direction of a Massachusetts licensed soil evaluator (____ of Coastal Engineering, License # ___). Copies of the Soil Evaluator Forms, including the percolation test results, are contained in Appendix D. A summary of the test pit characteristics is provided in Appendix E.

3.3 Soil Boring and Monitoring Well Installation

A total of four soil borings and four monitoring wells were installed December 28th and 29th, 2015. The soil borings were advanced with a truck-mounted direct-push drilling rig at locations MW-1, MW-2, MW-3, and MW-4. Continuous sampling was conducted from the ground surface to depths of between 60 and 80 feet at locations MW-1, MW-3, and MW-4.

Locations where continuous soil samples were collected were drilled using 2-inch diameter drill casing. Samples were collected using 60-inch long, one-inch diameter clear liners (diameter of core barrel?). Once soil collection was completed, 3-inch diameter drill casing was advanced in the same borehole to allow for the installation of a 2-inch diameter monitoring well. Once the borehole drilling was completed, a single 2-inch diameter monitoring well was installed at each location.

Each monitoring well consisted of ten-foot sections of schedule 40 PVC riser pipe attached to 10-foot sections of 10-slot well screen. An artificial sand pack was installed between the well and the formation from the base of the well to approximately 2 feet above the well screen. The sand pack was then capped by approximately 2 feet of bentonite pellets and allowed to hydrate. Grout was then installed to a depth of 5 to 10 feet below the ground surface.

The monitoring wells were used to determine the water table elevation and to perform slug tests. All wells were surveyed for location and elevation relative to National Geodetic Vertical Datum. Water-table elevations were used to estimate groundwater flow patterns (Section 4.0). Copies of the boring logs are provided in Appendix E. A summary of the well construction details is provided in Table 1.

3.4 Grain-Size Analysis

Soil samples were collected during the installation of the soil borings. Select samples were submitted to a laboratory for grain size analysis. Copies of the grain-size analysis reports are provided in Appendix __. Results of the soils analysis are discussed in Section 4.2.

3.5 Slug Testing

Slug tests consist of measuring the recovery of water levels in a monitoring well after a near-instantaneous change in head. The slug tests were performed by using air pressure to depress the water table "slug" in the well. A submersible pressure transducer and data logger was used to record the water level response over time.

The rising head slug tests were performed by rapidly releasing the air pressure in the well allowing the water level in the well to rise, simulating removing a slug of water out of the well. In wells where the static water level was within the screened interval (MW-1), the pneumatic slug test could not be performed. The pneumatic slug tests were conducted at monitoring wells MW-2, MW-3, and MW-4. The analysis methodology and results are discussed in Section 4.2.

Insert Table 1 - Summary of Well Construction Details

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4.0 HYDROGEOLOGIC CONDITIONS AND DATA ANALYSIS

(Additional detail will be added to this section in the final submittal)

4.1 Geology, Groundwater Flow and Boundary Conditions

Orleans is underlain by glacially derived sediments deposited 15,000 or so years ago during the waning stages of continental glaciation. The sediments consist of outwash deposited from the melting of the Cape Cod Bay and South Channel Lobes of the glacier (Wordsworth and Wigglesworth, 1934). The Harwich Outwash Plain deposits were derived from the Cape Cod Bay Lobe, while the Nauset Heights and Eastham plain deposits were derived from the South Channel Lobe (Oldale et.al., 1971).

Previous geologic investigations in Orleans indicate the bedrock surface occurs at about 400 feet below sea level. A layer of basal till, perhaps 100 feet thick, lies on top of bedrock. Above the till, lie fine and coarse-grained soils, consisting of outwash, lake and deltaic sediments. The lowermost stratified drift is a mixture of fine-grained soils (fine sand, silt and clay), presumed to be glacial-lake in origin, coarse-grained soils (sand and gravel), presumed to be glacial-stream in origin (need to update and site reference).

The Beach Road parcel is underlain by Nauset Heights deposits, as mapped by Oldale et.al (1971). These deposits consist primarily of sand and gravel. Till and boulders can occur interbedded with or overlying portions of the stratified drift deposits. Due east of the site, Dune (windblown sands from Beach deposits) and Beach Deposits (wave deposited sands and coarser-grained deposits from stratified drift) are mapped (Oldale et.al., 1971). In general, these deposits are fairly well sorted and very permeable.

The geologic conditions found at the Beach Road site are consistent with the deposits described above. Monitoring Well MW-1, installed in December 2015, was drilled to a depth of 80 feet below the ground surface. At MW-1, we encountered tan to light brown fine-to-coarse sand from ground surface to a depth of 80 feet. At MW-2, we encountered light-brown fine-to-coarse sand to a depth of 60 feet. Finally, at MW-3 we found tan to light-brown fine-to-coarse sand to a depth of 60 feet below ground. The soils at each location were very similar with trace amounts of gravel and silt. There was no refusal at any of the locations due to cobbles or boulders.

The lines of geologic cross-section are shown on Figure 6. The geologic cross-sections A-A’ and B-B’ (Figures 7 and 8, respectively) graphically depict geologic conditions locally and regionally. Well logs used to construct the geologic cross-section are in Appendix E.

The test wells are screened in a water-table aquifer that is at least 80 feet thick at the Beach Road site. Laterally, the water-table aquifer extends from Nauset Harbor north to the upper reaches of Pochet Neck south. The drainage divide to Pochet Neck, part of the Pleasant Bay Watershed, is located just south of the site. Groundwater modeling studies completed by the USGS indicate that groundwater in the water-table aquifer in the Beach Road site area flows easterly to the Atlantic Ocean (USGS, 2005).

Percolation rates in selected test pits ranged from less than 1-inch per minute to __ minutes per inch throughout the study area. Groundwater was not encountered in any of the test pits. Copies of the soil evaluator forms and soil boring logs are provided in Appendix D. Specific subsurface characteristics observed at each site follow.

Figure 6 – Lines of Geologic Cross Section

Figure 7 – Geologic Cross Section A-A'

Figure 8 – Geologic Cross Section B-B’

4.2 Calculated Aquifer Values

Aquifer characteristic were estimated through analysis of laboratory grains size tests and analysis of slug test data. A description of each method follows.

4.2.1 Soils Testing and Data Analysis

Samples collected from selected soil boring and test pit excavations were submitted to an engineering laboratory for grain-size analysis. Hydraulic conductivity values for each of the grain-size analyses were estimated using several methods described in the literature and summarized by Vukovic and Soro (1992).

Average calculated hydraulic conductivity values from the grain-size analysis ranged from __ feet per day (ft/day) to __ ft/day for sand and gravel, __ ft/day to __ ft/day for sand, __ ft/day to __ ft/day for fine the finer sands. The calculated hydraulic conductivity values estimated from the grain-size analysis are summarized in Table 2. The hydraulic conductivity approximation reports are provided in Appendix F. Laboratory reports for the soils analysis are provided in Appendix G.

4.2.2 Slug Testing and Data Analysis

Analysis of the slug test data were performed with the aid of Aquifer Test software (Waterloo Hydrogeologic, 2001) via the Bower and Rice straight line method. The results of the slug test analysis indicate hydraulic conductivity values ranging from __ ft/day to __ ft/day for sand and gravel, __ ft/day to __ ft/day for medium to coarse sand, and __ ft/day to __ ft/day for finer sands.

In general, calculated hydraulic conductivity values were greater than ___ ft/day. The data collected from the monitoring wells could not be accurately analyzed due the inability to create a significant change in the water level due to an extremely fast recovery of the water table. A summary of the estimated aquifer characteristics obtained from the slug test data is provided in Table 3. Copies of the slug test data analysis reports are contained in Appendix H.

Insert Table 2 - Summary of Aquifer Characteristics – Grain Size Analysis

Insert Table 3 - Summary of Aquifer Characteristics – Slug Test Analysis

4.3 Groundwater

The study area is located in a watershed that discharges directly to the Atlantic Ocean. Locally, groundwater flows eastward towards the Atlantic Ocean. Water levels measured on January __, 2016, from __ observation wells were used to estimate the groundwater flow direction across the study area. Each well was surveyed relative to ___. Using the survey data, the water levels obtained at each location were converted to elevation in feet ___. A summary of groundwater elevation data is provided in Table 1.

The static groundwater elevations were plotted on a map and approximated contours of the potentiometric surface were drawn. The resulting contours are shown on Figure 9. Contours were inferred between monitoring wells. Based on the contours, groundwater flow across the site is to the east, approximately __ degree ____ of east (E__N). The hydraulic gradient between the __ foot and __ foot contour on Figure 9 was calculated at __ ft/ft.

High groundwater conditions were calculated for the site. This was done in order to verify that the groundwater mound resulting from the discharge would not cause the water table to rise within four feet of the bottom of the discharge beds or ground surface. High groundwater levels were estimated using a method described by Frimpter (1980). This method involves comparing water levels at a specific site with long-term water level data at a reference site. The long-term reference site used in this analysis was a USGS well located north of the site in Orleans (OSW-__).

When the observed levels were obtained at the Beach Road site, the water levels from that date were relatively close to long-term average conditions at the USGS well (within __ feet). Therefore, it is assumed that the water levels at the Beach Road site were ___.

The Frimpter report provides a formula for determining how much higher the potential range of water levels at any particular site. The formula for the additional height of water is:

Sr/Owr (Owc – Owmax)

where:

Sr = the potential range of water levels at the site

Owr = the maximum annual range of water levels at the reference site

Owc = the level at the reference well when the levels were obtained

Owmax = the maximum water level at the reference well

The potential range in groundwater levels at the site was estimated on the basis of tables provided by Frimpter (1980). Much of the site would be categorized as _____. According to Frimpter, the range of water level fluctuations for a __ would be less than __ feet (in 95% of the cases). Therefore, we will assume that a conservatively high value for the range in water levels (Sr) at the site is __ feet (the range over much of the site is probably significantly less??).

Figure 9 - Water Table Contour Map

The maximum range in water levels at the reference well (Owr) was determined to be __ feet based on data collected at the site from __ to __. The depth to water under the maximum water table condition at the reference well (Owmax) was __ feet. The depth to water at the reference well on January __, __ (Owc) was __ feet. Based on these values, the estimated additional height of water above the levels observed at the site in late January would be:

__ ft/__ ft (__ ft – __ ft) = __ ft

In later analyses of the potential mound height at the site, __ feet will be added to the water levels to account for the maximum groundwater levels.

4.4 Surface Water

The nearest surface water bodies are the Atlantic Ocean and the upper reaches of Pochet Neck, a subwatershed of the Pleasant Bay Watershed. The Atlantic Ocean is located between 900 and 1,200 feet east of the proposed discharge area. The marshes and tidal area of Pochet Neck are located approximately 400 foot to the south and southwest of the closest area of the discharge (Figure 4). Despite the proximity of Pochet Neck to the discharge area, the aquifer drains to the Atlantic Ocean. Changes in the groundwater flow direction resulting from the proposed discharge are discussed in Section 5.7.

5.0 GROUNDWATER MODELING EVALUATION

(Additional detail will be added to this section in the final submittal)

Groundwater modeling was proposed as a part of the evaluation of disposal options as modeling can provide feedback on potential designs; this feedback can be used to help make design decisions. Specifically for this project, numerical groundwater modeling is a tool that was used to predict changes in groundwater elevations and flow directions based on locations and volume of wastewater disposal.

For this project, the Beach Road Site is considered. This site is shown in Figure 3. The objective of the groundwater modeling was to evaluate changes to groundwater elevations and flow directions under various subsurface discharge scenarios.

More specifically, the purpose of the groundwater modeling was to: 1) evaluate the potential impacts of the wastewater discharge on ambient groundwater flow and water levels and 2) to evaluate the ultimate discharge of groundwater originating at the disposal sites.

5.1 Modeling Method

A three-dimensional groundwater flow model of the study area was developed for the analysis of potential impacts to groundwater. The groundwater model used for this analysis was the U.S. Geological Survey's three-dimensional groundwater flow model, MODFLOW (MacDonald and Harbaugh, 1988

The three-dimensional groundwater flow model was coupled with a particle tracking model called PATH3D (Zheng, 1991) in order to illustrate the potential movement of groundwater over time.

5.2 Conceptual Model of the Aquifer System

A conceptual model of an aquifer is a representation of how that aquifer functions based on available data. Our understanding of how the aquifer functions is based on the geological and hydrological data presented in Sections 3.0 and 4.0.

The conceptual site model (CSM) for hydrogeology and groundwater flow in the Monomoy Lens is well documented in the USGS, 2004 report. Overall, the CSM documented in the USGS (2004) report is the same as is used for this modeling effort. However, there was a sub-surface hydrogeologic investigation (field effort) completed in December 2015/January 2016 at the Beach Road site. This work included new wells and hydraulic testing of the aquifer (to estimate K). These results of the investigation and data analysis are discussed in Sections 3.0 and 4.0 of this document.

This preliminary modeling effort does not incorporate the results of the field effort. Once those data are fully analyzed, the model may be updated to include those data and the scenarios may be re-run to consider potential changes.

In general, the aquifer at this site is a relatively simple water table aquifer composed of relatively homogeneous deposits of sand with trace amounts of gravel and silt. The bottom of the aquifer is estimated at ___ based on ____.

Water enters the aquifer system primarily in the form of rainfall recharge. A single recharge value is used for the entire model area. Groundwater leaves the aquifer system in lowland areas drained the ocean. These marshes and brooks are simulated primarily by drain nodes and general head boundaries and constant head nodes. This is discussed in further detail below.

5.3 Approach and Model Design

The numerical groundwater flow modeling was completed using the USGS numerical groundwater flow model of the Monomoy Lens as a basis. This model is documented in "Simulated Water Sources and Effects of Pumping on Surface and Ground Water, Sagamore and Monomoy Flow Lenses, Cape Cod, Massachusetts" (USGS, 2005). Modeling files were received from the USGS and were imported into the GMS 10.0.6 platform. GMS is a pre and post processor for MODFLOW-2000 that facilitates data input and interpretation of output.

A number of changes were made to the model to meet model objectives. They are as follows:

- AECOM converted the solver package to the PCG2 package from the LMG package based on a recommendation from the USGS in the model documentation that accompanied the model files ("…due to licensing restrictions, the USGS is no longer able to publicly distribute the Algebraic Multi-Grid (AMG) solver, on which the Link-AMG (LMG) Package relies. There are two possible solutions: 1) use a standard solver publicly available from USGS, such as SIP or PCG2 or 2) obtain the AMG/LMG solver from Fraunhofer-Institute for Algorithms and Scientific Computing (SCAI).).

- The model domain was made smaller to reduce computational time. A natural hydrogeologic boundary defined by the groundwater high and associated divide in the eastern part of the Monomoy Lens (in Brewster, MA) was selected as a new domain limit.

- AECOM refined the grid around the Beach Road site. The USGS model used a grid size of 400 feet by 400 feet over the entire model domain. AECOM adjusted the grid to range from 50 feet by 50 feet to 400 feet by 400 feet, with the most refined portions of the model grid located in the areas of interest. This was completed to provide better resolution on the model inputs (i.e., discharge areas) and outputs.

- In refining the grid, some model features were updated:

  - Drain cell conductances were adjusted to reflect the geometry of the grid cells.

  - General head boundary conductances were adjusted to reflect the geometry of the grid cells.

o Horizontal flow barrier segments were added as needed to encompass the ponds/lakes in the model domain.

Figure 10 shows the new model grid, general head boundaries and drain nodes. The hydraulic conductivity values used by the USGS and AECOM are shown in Figure 11.

After these changes above were made, the model was verified to be an adequate representation of the original USGS model in the following ways:

- A comparison of predicted groundwater elevations were made in select cells/areas to demonstrate that the new version of the model predicted groundwater elevations similar to that predicted by the USGS model. Generally, the differences were less than 0.5 feet and lower in many instances. There are a handful of places where the model predictions are greater than 1 foot, but these are far from the areas of interest and so should not impact model predictions.

- A comparison of groundwater flow directions as demonstrated with particle tracking. Particles were seeded in select areas to verify that groundwater flow paths and divides are similar as those mapped by the USGS (specific reference). Generally speaking the new version of the model was the same or similar to the particle tracking under ambient conditions from the USGS version of the model.

- A comparison of mass balance generated by the model was completed. Specifically, recharge, well, general head boundary, drain boundary volumes were compared to verify that the water balances were the same or similar to the USGS model. The following table (Table 4) summarizes the comparison. (Additional detail to be added in final version).

Overall, the differences between the new (re-gridded) version of the model and the original version of the model are attributed to a combination of the following three things:

- The grid refinement, which doesn't allow a direct comparison of groundwater elevations in places where the grid has been changed significantly,

- The changes in conductance (in the drain and general head boundary packages) where the grid is refined and groundwater flows into and out of the model differently between the models and therefore results in slightly different elevations, and

- The change in model solver (from LMG to PCG2), which results in a slightly different solution.

In summary, a number of changes to the original USGS model domain and structure were completed to better meet the project objectives. Despite the changes, the model replicated the USGS output adequately; differences can be explained and are not expected to impact model predictions. Overall, AECOM considers this model a good tool to complete a preliminary evaluation of effluent disposal scenarios. Predictions will be updated once the result of the recent hydrogeologic investigation are summarized and incorporated, as needed, into this new version of the model.

5.4 Changes to Model Input Parameters

Model input parameters were changed only in the vicinity of the proposed discharge site. Changes to input were based on values obtained from the current study. Input parameters changed by AECOM included: horizontal and vertical hydraulic conductivity, elevation of the aquifer surface, and storage coefficients (for transient simulations).

Figure 10 - Model Area and Location of General Head Boundaries and Drain Nodes

Figure 11 - Hydraulic Conductivity Values

Insert Table 4 – Comparison of Mass Balance

5.4.1 Hydraulic Conductivity

Data on the potential hydraulic conductivity of the aquifer materials on the site were obtained by a number of means including: 1) slug tests at observation wells, and 2) estimates based on grain-size analyses. Descriptions of these methods and the resulting hydraulic conductivity values are presented in Section 4.2.

In general, fine- to coarse-grained sediments, resulting in ___ hydraulic conductivity values within the model, were found as a result of the site investigations. Hydraulic conductivity values used in the groundwater model ranged from __ ft/day for sandy silts to __ ft. /day for sand and gravel deposits. The distributions of hydraulic conductivity values for layer 8 (the layer representing the water table aquifer) are shown in Figure 11. The hydraulic conductivity values for layers 7 and 8 are identical in the area of the proposed discharge.

5.4.2 Storage Coefficient

For transient simulations, storage coefficients were assigned to all model layers. Model Layer 1 was assigned a value of __, a typical value for the specific yield of water table aquifers. The lower layers were given an elastic storativity value of __.

5.4.3 Rainfall Recharge

Rainfall recharge was changed across the model grid to simulate a wet year to obtain high water table conditions. In the original model, Model Layer 1 was assigned a value of ___, a typical value for recharge to a Cape Cod aquifer. To increase the water table elevation to simulate high water table conditions, rainfall recharge was increased to ___ in order to raise the water table __ feet at the site.

5.5 Model Calibration

The calibration process consists of comparing observed and simulated groundwater elevations and then adjusting the input parameters to obtain results that more closely match the observed conditions. Parameters adjusted during calibration included horizontal and vertical hydraulic conductivities and river and drain characteristics and placement.

The initial condition used to create the first model data sets were based on the USGS's initial conceptual model of the aquifer. No changes were made to the modeled aquifer characteristics used by the USGS.

The model was then calibrated to groundwater conditions that approximated long-term average conditions, as much as was practical. Water levels obtained on the site in January 2016 were compared to water levels at a nearby long-term USGS monitoring well ("reference well") as described in Section 4.0. The water levels at the USGS long-term monitoring wells were found to be within __ feet of long-term average conditions. Therefore, it is assumed that the water levels at the site were reasonably close to long-term average conditions and it is possible to calibrate the model in steady state mode. "Steady state" means that all the stresses on the model (such as rainfall, and flow into and out of the model from neighboring areas) occur continuously at average annual rates and therefore the model simulates average long-term conditions.

The results of the calibrated steady-state model output (water table levels) are shown in Figure 12. These are the simulated water levels in Layer 8 of the model. The difference between simulated and observed water-level elevations ranges from __ to __ feet. The differences between observed and simulated water levels are referred to as a residual. A summary of model residuals at the observation well sites is summarized in Table 5.

Several statistical techniques are used to evaluate how well a model has been calibrated to observed conditions (Anderson & Woessner, 1992). One is to calculate the mean error, which is the mean difference between observed and simulated groundwater elevations (residual). Table 5 lists the residuals at five monitoring wells. These are referred to as “calibration targets” on Figure 10. The mean residual was calculated to be __ foot.

One drawback to using the mean residual to evaluate calibration is that positive and negative residuals tend to cancel each other out, making the calibration appear better than it might be. A better overall indication of error is the mean absolute residual. For this, the mean is taken of the absolute values of all the residuals. The mean absolute residual for the final steady-state calibration was __ foot.

Probably the best measure of the calibration is to consider the standard deviation of the residuals over the observed range in head from the calibration targets. This value is __. By all of these measures, the model would be considered well calibrated.

5.6 Sensitivity Analysis

A sensitivity analysis is applied to numerical groundwater flow models in order to evaluate which parameters have the greatest impact on the model output. The sensitivity analysis was undertaken by applying a module of the USGS inverse modeling program called UCODE (Poeter and Hill, 1998). UCODE automatically subjects each model run to an extensive sensitivity analysis. Each parameter was adjusted by a specified amount (10% higher and lower than the calibrated values) and the differences in water levels are compiled from each target location. In this case, the target locations are the observation wells monitored during the loading test.

UCODE calculates the sensitivity of each parameter with respect to each observation point and then takes the mean of all of the sensitivities. The sensitivity is expressed as a dimensionless scaled sensitivity. The term “dimensionless” refers to the fact that the units of the parameter (e.g. feet or feet/day) are factored out so that the sensitivity of each parameter can be compared on an equal basis. They are “scaled” in that the sensitivity value takes into account a weighting factor for each observation (water levels). The weighting factor is a measure of the reliability of each observation. In this case, the reliability of each observation is assumed to be the same, so there is no need for scaling the sensitivities. Refer to Hill (1998) for a full description of dimensionless-scaled sensitivities.

The parameter sensitivities for the model are summarized in Table 6. The sensitivity analysis was conducted on horizontal and vertical hydraulic conductivity, recharge, and river bottom and drain hydraulic conductivities. Since the sensitivities are dimensionless, it is possible to directly compare the sensitivities, regardless of the parameter type. Based on the data in Table 6 it is easy to see that the most sensitive parameter for the model calibration is the hydraulic conductivity.

Figure 12 - Model Results - Steady State Contours & Calibration Targets

Insert Table 5 – Summery of Model Residuals

Insert Table 6 – Summary of Sensitivity Analysis

5.7 Predictive Simulations

The calibrated groundwater flow model was used to simulate several proposed discharge scenarios in order to predict groundwater mounding and the flow of mounded groundwater from the discharge sites. The Beach Road Site is proposed to be the primary discharge area for the effluent discharge. The entire discharge from the WWTF is proposed to be discharged at this area. The reserve area would be used only if necessary.

Five separate discharge scenarios were simulated using the groundwater flow model. All five scenarios assume a different discharge rate at the site under the average ambient water table conditions described above and illustrated in Figures 13 through 17.

Scenarios A through E are five scenarios simulating groundwater discharges between 25,000 and 200,000 gpd through subsurface leaching trenches within the primary discharge area at the Beach Road Site. Table 7 summarizes the model scenarios.

- Simulation A – 25,000 gpd - Figure 13 shows the simulated paths of groundwater flow as determined by the particle tracking module. The ultimate discharge points of groundwater are shown by the particle tracks along with the estimated groundwater travel times.

  The modeling indicates that all of the groundwater will discharge east to the Atlantic Ocean. The model predicts that groundwater discharge will reach the Atlantic in approximately __ days at the discharge rate of 25,000 gpd. Compared to the baseline conditions, the flow path spread is slightly greater, however there are no particles that cross the watershed divide (mapped by the USGS) to the south (Figure 13). The baseline groundwater elevation near the center of the discharge area is 4.65 feet; with discharge, the groundwater elevation at the same location is 4.83 feet, indicating 0.18 feet of mound.

- Simulation B – 50,000 gpd - Figure 14 shows the results of Scenario 2, treated effluent discharge at a rate of 50,000 gpd. The maximum mound elevation is predicted to be approximately 5.01 feet indicating 0.36 feet of mounding above the ambient groundwater levels. This is approximately 39 feet below ground surface.

- Simulation C – 100,000 gpd - Simulation C (Figure 15) also shows the simulated paths of groundwater flow as estimated by the groundwater model. At 100,000 gpd, the model indicates that all of the groundwater will discharge along the coast east of the site. The model predicts the groundwater elevation at 5.36 feet or 0.71 feet of mounding. The estimated high groundwater condition would bring the mound less than a foot closer to the ground surface. The overall spread of the discharge continues to widen when compared to the no discharge simulation as well as at lower discharge rates (Figure 15). However, the discharge continues to flow to Atlantic.

- Simulation D – 150,000 gpd - Figure 16 also shows the simulated paths of groundwater flow as determined by the particle tracking module. The model results indicate that most of the groundwater will discharge to the brooks located to the east and west. Smaller amounts will discharge to the springs located to the north. The model predicts that groundwater discharged at the eastern portion of the beds will reach the western brook in approximately six months at the discharge rate of 102,000 gpd.

  The overall spread continues to widen (Figure 16). The groundwater elevation is 5.71 feet indicating 1.06 feet of mounding. By comparison to the 25,000 gpd discharge, the overall spread widens, but does not intersect with the divide to the south (Figure 16).

- Simulation E – 200,000 gpd - Model Scenario E, simulates a groundwater discharge of 200,000 gpd. Figure 17 shows the model results of Scenario E. The maximum mound height, directly below the discharge, is predicted to be approximately 1.41 feet above the ambient groundwater levels. This is approximately 38 feet below ground surface. MassDEP requires a four-foot separation between the bottom of the infiltration beds and the high water table. Using subsurface discharge, we estimate that there would be over 30 feet separation between the groundwater and the base of the discharge.

  A higher rate of discharge was not simulated. At a rate of 200,000 gpd, we start to see particle tracks crossing into the Pochet River watershed, part of the Pleasant Bay Watershed. Although the model indicates the discharge would travel under the tidal marsh and discharge to the ocean, there remains the possibility that a small portion of the discharge could end up in the nitrate sensitive estuary triggering a high level of treatment at the WWTF.

  The overall spread continues to widen and some particles cross over the groundwater divide to the south, suggesting that the divide to the south would shift and some discharged effluent would have a travel path through that watershed (Figure 17). However, the ultimate discharge area is still the Atlantic Ocean, not the wetland to the south. The groundwater elevation is 6.06 feet indicating 1.41 feet of mounding. The estimated high groundwater condition would add approximately one foot to the mound height.

Figure 13 – Simulation A – 25,000 gpd

Figure 14 – Model Scenario B – 50,000 gpd

Figure 15 – Model Scenario C – 100,000 gpd

Figure 16 – Model Scenario D – 150,000 gpd

Figure 17 – Model Scenario E – 200,000 gpd

Insert Table 7 – Summary of Model Scenarios

5.8 Model Limitations

A numerical groundwater flow model is a simplified representation of a complex system and therefore carries with it inherent uncertainties. These should be considered in the interpretation of the predictions.

All groundwater flow models are mathematical approximations of complex natural systems. The ultimate success of a modeling effort must be evaluated with respect to the purpose of the model. In this case, the model was intended to predict groundwater mounding and groundwater discharge flow paths. The model was also meant to estimate the area and quantities of discharge as a result of discharging treated effluent at the site and to determine the ultimate discharge of groundwater affected by treated effluent. Based on the calibration statistics, sensitivity analysis and verification step, the model appears to function reasonably well for this purpose.

The primary sources of uncertainty in the groundwater model are difficulties in accurately reproducing the heterogeneous nature of the aquifer and the potential for unsaturated flow in the aquifer. Since the model is relatively sensitive to changes in hydraulic conductivity, the heterogeneous nature of the aquifer adds some uncertainty in the model results. This has been addressed by using relatively conservative hydraulic conductivity values (that is, values that result in a larger groundwater mound).

6.0 WATER QUALITY

(Additional detail will be added to this section in the final submittal)

On January __, 2016, baseline water quality sampling was performed in the area of the proposed primary discharge area. Groundwater samples were collected from one up gradient (MW-4) and three (MW-1, MW-2, and MW-3) downgradient monitoring wells. The samples were submitted to a State of Massachusetts certified analytical laboratory for inorganic and organic analyses. In addition, in-situ water quality sampling was performed in the field using a YSI water quality meter. Results of the field and laboratory testing are summarized in Table 8 (to be completed at 100%). Copies of the laboratory reports are provided in Appendix I.

Insert Table 8 - Summary of Baseline Water Quality Results

7.0 POTENTIAL IMPACTS

(Additional detail will be added to this section in the final submittal)

The potential impacts resulting from the proposed wastewater discharge fall into two general categories: 1) potential water quality impacts and 2) potential impacts from the resulting groundwater mound. Although the wastewater will be treated to high levels, there will be higher levels of nutrients and other contaminants in the wastewater discharge than in the ambient groundwater. Potential mounding impacts include: the discharge of groundwater in areas where groundwater does not presently discharge and the rise of water levels on surrounding properties. The proposed discharge will be examined for these potential impacts.

7.1 Potential Groundwater Mounding Impacts

The primary site of proposed discharge is at 223 Beach Road in Orleans. There are no nearby properties that would be affected by the rise in water table resulting from the discharge. In addition, the groundwater model indicates that there will be no discharge of groundwater in areas other than where groundwater already discharges. The mounded groundwater will discharge to the Atlantic Ocean approximately 1,000 feet to the east and will not impact any public drinking water resources. According to Town records, the property at 9 Hubler Lane (Map 38, Parcel 16-6) has a private well (Figure 18). If the well is a drinking water or irrigation supply, the Town should consider connecting the property to the public water supply as the private well may be located with the influence of the proposed discharge.

7.2 Potential Water Quality Impacts

Need to address potential nitrate impacts, as necessary.

Figure 18 – Location of Private Wells

8.0 GROUNDWATER MONITORING PLAN

A groundwater monitoring plan will be implemented to assess both baseline and compliance groundwater quality in the vicinity of the proposed primary discharge. The location of the proposed monitoring wells, baseline water quality parameters and compliance water quality parameters are discussed in the following sections.

8.1 Proposed Monitoring Locations

To date, four monitoring wells (MW-1, MW-2, MW-3, and MW-4) have been sampled as part of the hydrogeologic investigation. The location of the proposed discharge and monitoring wells are shown on Figure 19. These wells were located in close proximity to the discharge to obtain soil samples and to estimate the groundwater flow direction below the site. However, these wells are located in close proximity to the discharge. Three additional monitoring wells are being proposed for compliance monitoring. The wells will be installed and sampled for baseline water quality before the WWTF becomes operational. The wells are shown on Figure 19 and labeled as MW-5, MW-6, and MW-7. If any of these locations is destroyed during construction of the discharge, a new monitoring well will be installed. Details of the monitoring well installation, groundwater sampling and laboratory analysis are discussed in the following sections. The replacement monitoring well will be installed as outlined in Section 8.4.

8.2 Baseline Water Quality

An initial round of water level data and groundwater samples was collected from the proposed monitoring wells in January of 2016. A summary of the field and laboratory results is provided in Section 4.0. Prior to discharging effluent (Figure 19) two rounds of groundwater samples will be collected at monitoring well locations MW-1, MW-2, MW-3, and MW-4 and sent to a laboratory for analysis. Water samples will be analyzed in the field for temperature, pH and specific conductance. Groundwater samples collected from each of the monitoring wells will also be sent to a MassDEP approved laboratory for analysis. At a minimum, laboratory analysis will include nitrate-nitrogen, total nitrogen, total phosphorus, sodium, and volatile organic compounds (VOCs). Groundwater sampling will be conducted in accordance with MassDEP's "Standard References for Monitoring Wells". A round of water levels will also be collected at the time of sampling. The water level data and water quality results will be summarized and submitted to the MassDEP for review.

8.3 Compliance Monitoring

Once the WWTF is brought online, groundwater samples will be collected and analyzed to demonstrate that the groundwater quality meets the standards set by MassDEP. Groundwater monitoring will be performed at the six monitoring well locations (MW-1, MW-5, 1-02, 3-02, 10-02 and 4-OB) outlined above. The sampling frequency and parameters are as follows:

- Monthly Sampling – Water quality analysis for pH and conductivity will be analyzed on a monthly basis. In addition, a round of water levels will be collected and recorded.

Figure 19 - Location of Proposed Monitoring Wells

- Quarterly Sampling – In addition to the monthly sampling, total nitrogen, total phosphorus, nitrate-nitrogen, sodium and fecal coliform will be sampled and analyzed quarterly.

- Twice Annual Sampling. In addition to the monthly and quarterly sampling, VOCs will be sampled and analyzed twice annually.

- Groundwater samples collected during each round will be sent to a MassDEP approved laboratory for analysis. Groundwater sampling will be conducted in accordance with MassDEP's "Standard References for Monitoring Wells". After each round of sampling, the water level data and water quality results will be summarized and submitted to the MassDEP for review.

8.4 Replacement Monitoring Well Installation

Should any of the monitoring wells become damaged or need to be replaced; the MassDEP will be notified prior to replacing the well. The installation of the replacement well(s) will be as follows.

For each replacement well, one soil boring will be drilled using augers to a depth of approximately 10 to 15 feet below the water table. At a minimum, split spoon samples will be collected every five feet to the total depth of the boring. The soil borings will be drilled in accordance with MassDEP's "Standard References for Monitoring Wells".

Once the soil boring is completed, a single monitoring well with ten feet of 10-slot well screen will be installed. The bottom of the well screen will be installed approximately 15 feet below the water table. The two-inch diameter PVC monitoring wells will be installed and developed in accordance with MassDEP's "Standard References for Monitoring Wells".

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9.0 ESTIMATED COSTS OF GROUNDWATER DISPOSAL FACILITIES

(Additional detail will be added to this section in the final submittal)

Estimated costs for groundwater discharge facilities are provided in the following section. Two discharge methods were considered. Both subsurface and wick discharges were evaluated. Open bed discharge was not considered as this method of discharge would not be in line with the Town's plans to develop the site as parking for Nauset Beach area. A summary of the discharge methods and estimated costs to permit, install and operate is discussed in the following sections.

9.1 Subsurface Groundwater Discharge – Estimated Costs

Estimated costs for a 60,000 gpd subsurface discharge as described in Section __, is summarized in Table 9. The square footage, discharge rate per square foot, depth of discharge, and estimated range of costs per item are provided.

9.2 Wick Discharge – Estimated Costs

A wick is a vertical subsurface structure constructed for the purpose of transporting highly treated effluent to groundwater. A wick is basically a large diameter (normally – to – feet) borehole filled with pea stone or gravel. The highly treated effluent is discharged into the wick just below the ground surface, allowing the wastewater to flow over the stone to the underlying groundwater. Typically, a minimum depth of 20 to 30 feet to the water table is required.

The concept of a wick is similar to conventional effluent disposal systems where the wick applies the discharge vertically instead of horizontally. The component that is common to both is the receiving groundwater system, which must be capable of receiving and transporting the effluent that is applied to it. The wick is merely the means of transporting the highly treated effluent from the surface and dispersing it into the groundwater.

A wick is installed in much the same way as a well. A borehole is excavated and casing is installed to the entire depth of the proposed wick. Once the total depth of the wick has been excavated, the borehole is backfilled with pea stone. The casing is then pulled back to expose the stone to the surrounding formation. When completed, effluent is piped to the top of the wick for gravity discharge into the wick. The entire top of the wick is to be enclosed in a precast concrete structure to protect it from the elements. At this time, it is anticipated that a total of three to five wicks would be installed within the primary discharge area at the site.

Wicks similar to the ones that would be proposed for the 223 Beach Road site have been installed, tested, and used at Linden Ponds Retirement Community in Hingham, Massachusetts. These wicks have been in use for approximately 12 years. These wicks have performed well and remain in use today. In general, as with conventional subsurface wastewater disposal systems, wick applications require certain favorable hydrogeologic conditions relative to subsurface conditions and depths to water tables; as described herein, the Beach Road site provides such favorable conditions.

Insert Table 9 - Summary of Estimated Costs – Subsurface Groundwater Discharge

To demonstrate that a wick is a viable discharge option, a test wick will need to be installed and tested on the Beach Road Site within the proposed primary discharge area. Wick loading test results, analysis and conclusions would need to be submitted as part of the Hydrogeologic Evaluation submitted to MassDEP. The wick installation and loading test is required for all proposed wick installations.

The Beach Road site provides sufficient area for the reserve wastewater discharge for the wicks. The reserve area would be located adjacent to the wicks and within the discharge area identified in Figure 4. The proposed method of discharge in the reserve areas would be through conventional through subsurface groundwater discharge leaching trenches. The areas identified on Figure 4 are sufficient to allow for the discharge of the ultimate design flow of __ gpd.

Estimated costs for a 60,000 gpd subsurface discharge as described in Section __, is summarized in Table 9. The square footage, discharge rate per square foot, depth of discharge, and estimated range of costs per item are provided.

Insert Table 10 - Summary of Estimated Costs – Wick Discharge

10.0 REFERENCES

Anderson, M.P. and W.W. Woessner, 1992, Applied Groundwater Modeling, Academic Press, Inc., San Diego, California.

Frimpter, M.H., 1980, “Probable High Ground-Water Levels in Massachusetts,” U.S. Geological Survey Water Resource Investigations 80-1205.

McDonald, M.G. and A.W. Harbaugh, 1988, "A Modular Three-Dimensional Finite Difference Groundwater Flow Model", Techniques of Water Resources Investigations 06-Al, USGS.

Poeter, E.P and M.C. Hill, 1998, “Documentation of UCODE, A Computer Code for Universal Inverse Modeling,” U.S. Geological Survey Water Resources Investigations Report 98-4080.

Randall, A.D., 1996, “Mean Annual Runoff, Precipitation and Evapotranspiration in the Glaciated Northeastern United States, 1951-1980,” U.S. Geological Survey, Open File Report, OFR 96-395.

Oldale, R. N., Carl Koteff, and J. H. Hartshorn, 1971, Geologic Map of the Orleans Quadrangle, Barnstable County, Cape Cod, Massachusetts”, GQ-931, U. S. Geological Survey.

Woodworth, J. B. and Wigglesworth, Edward, 1934, Geography and Geology of the Region including Cape Cod, the Elizabeth Islands, Nantucket, Martha’s Vineyard, No Mans Land, and Block Island: Harvard Coll. Mus. Comp. Zoology Mem. v.52, 322 p.

Zheng, C., 1991, "PATH3D. A Groundwater Path and Travel-Time Simulator".

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LIST OF APPENDICES