ATTACHMENT 9

NPDES Permit Compliance Alternatives

Technical Memoranda

Town of Marion

April 12, 2016

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Section 1

Aucoot Cove Total Nitrogen Watershed Load Estimate

Memorandum

To: *Robert Zora, DPW Director*

From: *Zachary T. Eichenwald*
Bernadette Kolb

Date: *March 9, 2016*

Subject: *Aucoot Cove Total Nitrogen Watershed Load Estimate*

Background and Overview

In November 2014, the Town of Marion (Town) was issued a Draft National Pollution Discharge and Elimination System (NPDES) permit for its wastewater treatment plant (WWTP) that included new limits for total nitrogen of 3 mg/l and 14.7 lb/d. In its comment letter on the draft NPDES permit (February 2015), the Town disputed the bases, supporting analysis, and need for these limits.

An additional provision of the Draft NPDES Permit allows that the Town can request a permit modification [to the total nitrogen limits] if it can demonstrate that reductions in nonpoint source and stormwater nitrogen are sufficient to achieve water quality standards in Aucoot Cove (Section F – Compliance Schedule, last paragraph). To address this possibility, it is important to understand the magnitude of each nitrogen source to Aucoot Cove so that the Town can properly evaluate alternatives for cost effectively reducing nitrogen load to the Cove.

This memorandum documents the findings of a land-use based watershed nitrogen loading analysis and point source estimates, as follows:

- The Massachusetts Estuaries Project (MEP) approach for estimating land-use based nonpoint source loads was followed, using parameters developed as part of the MEP for watersheds tributary to Buzzards Bay and Cape Cod,
- Potential leakage from three lagoons at the WWTP was estimated using the results of a lagoon water budget based on data from a 5-month deployment of high-resolution pressure transducers and a water quality sample of lagoon water analyzed for total nitrogen, and
- The load associated with effluent discharge from the WWTP was estimated using data from four years of monthly operating reports.

In the final section of this memorandum, the results of this analysis are compared to the estimates by EPA as described in the Fact Sheet to the Draft NPDES permit.

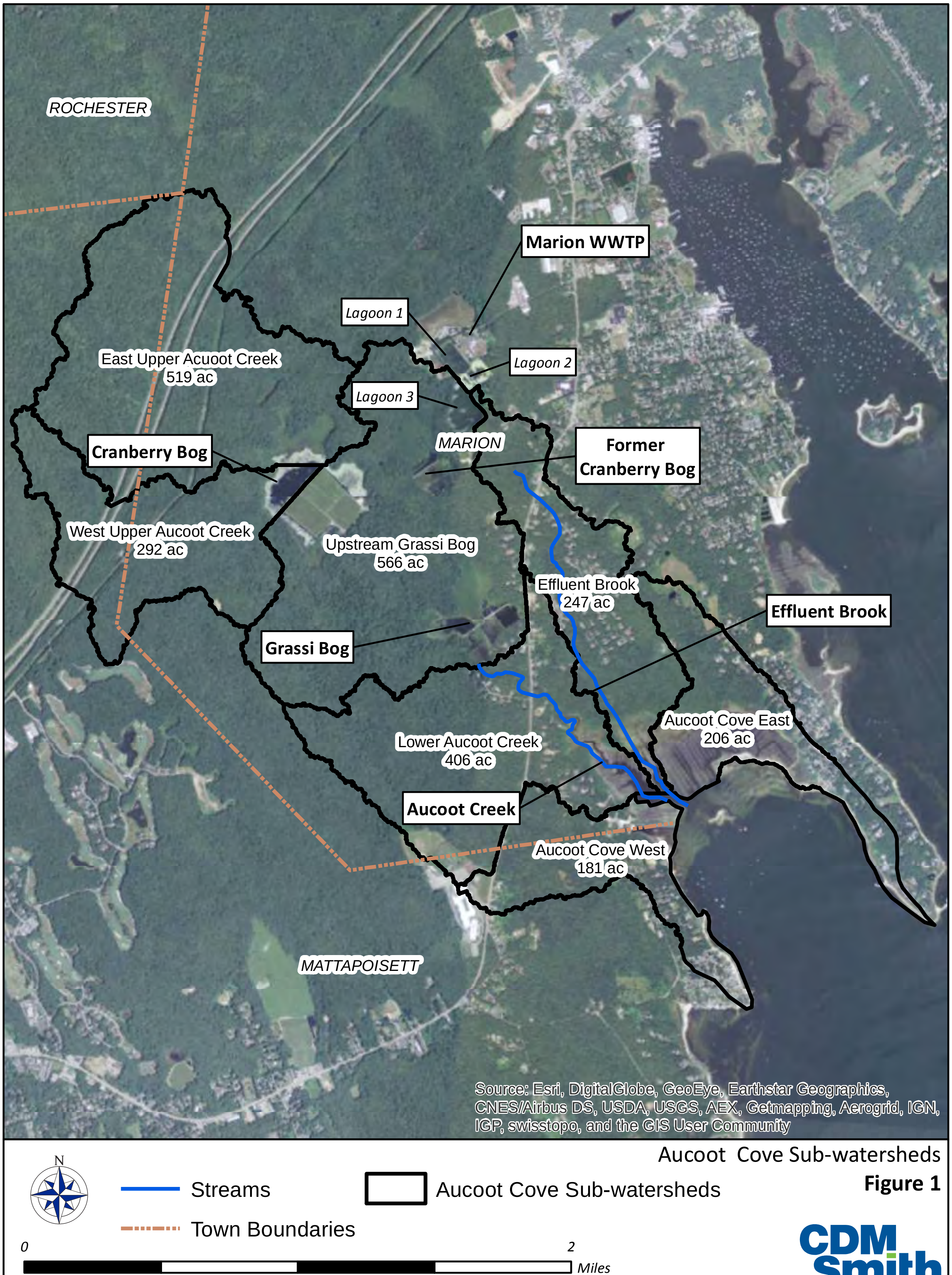
Development of a Nitrogen Loading Model for Aucoot Cove

An abbreviated version of the MEP Linked Watershed-Embayment Model approach, focused on developing site-specific, land-use based watershed load estimates, was implemented to estimate nitrogen loads to Aucoot Cove. The loads are based on parcel data from the Towns of Marion and Mattapoisett and, in most cases where the parcel has a septic system, parcel-based water use data obtained from Marion. An expanded MEP approach would then have used receiving water quality data, and a hydrodynamic or water quality modeling in the cove to calibrate the watershed loads; these were not done for this study. As a result, the watershed model has not been calibrated in the same way that the MEP Linked Watershed-Embayment Model approach is calibrated, but instead relies on the well-calibrated and well-studied loading factors developed by MEP for other watersheds in Buzzards Bay and Cape Cod.

The MEP land-use based nitrogen loading analysis model was applied to seven Aucoot Cove sub-watersheds to estimate the annual average total nitrogen load from nonpoint sources such as agricultural runoff, septic systems, and stormwater runoff. The following data were collected and analyzed.

- Land use area by parcel and land use code for both Marion and Mattapoisett. Marion parcel data also specify whether each individual parcel is served by the municipal sewer system. Mattapoisett properties tributary to Aucoot Cove are served by septic systems.
- Water use data from 2008 to present from the Marion Department of Public Works.
- Total area of active cranberry bog production as digitized from satellite imagery collected in 2014.
- Building footprint area based on the MassGIS Building Structures database, which was derived from ortho imagery collected between 2011 and 2015.
- Total road surface area, obtained from the MassDOT Roads official street transportation dataset.
- Area of Aucoot Cove whose outer boundary is a line from the end of the Aucoot Cove East and Aucoot Cove West sub-watersheds (see **Figure 1**), which will receive additional nitrogen from atmospheric deposition.

Other required parameters and constants were taken from the MEP loading methodology guidance. These include a factor to calculate the nitrogen load from septic systems, average lawn size for each parcel, residential lawn fertilizer application and leaching rate, woodlot equivalent nitrogen load and leaching rate, cranberry bog fertilizer application rate, average driveway size for each parcel, average nitrogen load for roads and driveways, the average nitrogen load for roofs, the average atmospheric deposition load, and the average nitrogen load from natural areas. Assumptions for parameters and constants are described in **Table 1**.



Aucoot Cove Sub-watersheds

Figure 1

CDM Smith

EICHENWALDZT C:\Projects\MarionNPDES\Watershed_Load\GIS\Figure 1 - Watershed Map.mxd 2/23/2016

Table 1. MEP Parameters and Constants

Parameter	Value	Source
Septic Nitrogen Concentration	26.25 mg/l	CCC Technical Bulletin 91-001, "Nitrogen Loading"
Average Lawn Size	5,000 sf/parcel	MEP Constant
Residential Lawn Fertilizer Application Rate	1.08 lb/1,000 sf	Howes Lawn survey in Poppy, 3 Bay watersheds
Residential Lawn Fertilizer Leaching Rate	20%	CCC Technical Bulletin 91-001, "Nitrogen Loading"
Percent of Residences Fertilized	50%	Howes Lawn survey in Poppy, 3Bay watersheds
Woodlot Equivalent Nitrogen Load Rate	0.7 lb/1,000 sf	Application rate (Howes & Costa N loading); Howes Nantucket 1987
Woodlot Equivalent Nitrogen Leaching Rate	30%	Application rate (Howes & Costa N loading); Howes Nantucket 1987
Cranberry Bog Fertilizer TN Application Rate	31 lb/d	Teal/Howes
Average Driveway Size	1,500 sf/parcel	MEP Constant
Nitrogen Concentration for Road/Driveway Runoff	1.5 mg/l	CCC Technical Bulletin 91-001, "Nitrogen Loading"
Nitrogen Concentration for Roof Runoff	0.75 mg/l	CCC Technical Bulletin 91-001, "Nitrogen Loading"
Recharge Rate from Precipitation on Impervious Surfaces	40 in/y	CCC Technical Bulletin 91-001, "Nitrogen Loading"
Nitrogen Load from Natural Areas	0.072 mg/l	MEP Constant
Recharge Rate from Precipitation on Natural Surfaces	27.25 in/y	CCC Technical Bulletin 91-001, "Nitrogen Loading"

Watershed Delineation

The Aucoot Cove watershed is approximately 3.8 square miles and drains portions of the Towns of Marion and Mattapoisett. The Aucoot Cove watershed was delineated using ArcHydro tools applied to LiDAR data collected by USGS during 2013 and 2014 and available from MassGIS (the LiDAR-based watershed delineation). While an older delineation (the BBNEP delineation) of Buzzards Bay area watersheds is available from the Buzzards Bay National Estuaries Program (BBNEP), we wanted to validate the BBNEP delineation with the more recently collected (2013-2014) LiDAR data. The LiDAR data was also used to delineate seven sub-watersheds based on major hydrologic features. **Figure 1** attached shows the watershed and sub-watershed delineations. While the MEP methodology often uses a watershed delineation based on groundwater flow patterns for its nitrogen analysis in Cape Cod embayments, a review of surficial geology and known aquifers within the study area found that a watershed delineation based on surface hydrology is adequate for this study with the exception on the area of Marion's wastewater lagoons.

The LiDAR-based watershed delineation compares favorably to the BBNEP delineation, which has a watershed area of 4.1 square miles. The principal difference is an area located in the vicinity of The Bay Club golf course in Mattapoisett, which is included in the BBNEP delineation but not in the updated delineation. The USGS topographic maps indicate that a stream exits a wetland at The Bay Club and travels east to join Aucoot Creek, which then flows into Grassi Bog. The LiDAR data as well as elevation data from the National Elevation Dataset indicate that there is a ridge between The Bay Club and Aucoot Creek, directing the drainage from The Bay Club's wetland south towards Mattapoisett. Therefore, the updated, LiDAR-based watershed delineation does not include this drainage in the Aucoot Cove watershed.

One consideration in the development of the revised watershed delineation was the location of the watershed boundary near the Marion WWTP since the WWTP and its three lagoons are located on a local high point (watershed boundary). Marion's lagoon 3 is located within the LiDAR-based watershed delineation. The placement of this lagoon within the watershed boundary was further informed by historic groundwater levels collected by Horsley Witten in the vicinity of the lagoons, EPA and Horsley Witten's assumed distribution of groundwater flow between Aucoot Cove, Sippican Harbor, and Benson Brook, the BBNEP watershed delineation, and the elevation data used for the watershed delineation. Historic groundwater levels collected by Horsley Witten show a groundwater divide roughly in the vicinity of lagoon 3, with lagoons 1 and 2 generally flowing towards Sippican Harbor and Benson Brook. This is supported by the relative fraction of groundwater flow between Aucoot Cove and other nearby water bodies, where 50% of the groundwater flow is assumed to enter Aucoot Cove; lagoon 3 is approximately 50% of the total lagoon surface area. Furthermore, the BBNEP watershed delineation indicates that at least a portion of lagoon 3 is within the Aucoot Cove watershed, with the remaining lagoons lying in the Sippican Harbor and Benson Brook watersheds. These supporting details, in conjunction with the LiDAR elevation data, provide a weight of evidence, in absence of detailed groundwater measurements, to suggest that lagoon 3 is within the Aucoot Cove watershed, with lagoons 1 and 2 located outside of the Aucoot Cove watershed.

Land Use

While the MEP methodology relies on a detailed, parcel-level accounting of land use, a high-level general accounting of land use patterns was performed using the most recent (2005) MassGIS land use survey to understand the distribution between developed and undeveloped land in the watershed. **Figure 2** shows the areas of the major land use categories within the Aucoot Cove watershed based on the MassGIS land use survey. The majority of the land (86%) within the Aucoot Cove watershed is undeveloped. Of this undeveloped land, nearly 60% is forest with the remaining undeveloped land comprised of forested and non-forested wetlands. Residential development is the next largest land use in the Aucoot Cove watershed. Low- to medium-density residential development is approximately 6% of the total land use in the watershed, with the remaining residential development spread between multi-family and high density residential properties.

Septic Loads from Water Use and Parcel Data

Parcel data for Marion derived from the Massachusetts Level 3 Parcel Mapping dataset were obtained from the Marion Assessor's office, which included information indicating which parcels are connected to the municipal sewer system. The number and location of properties served by septic systems was calculated using data from the assessor's database and supplemented by information from the Town's municipal water and sewer billing database. Parcel data for the

Mattapoisett portion of the Aucoot Cove tributary area was obtained directly from MassGIS; there is no municipal sewer system serving the area of Mattapoisett in the Aucoot Cove watershed.

Water use data, when available, is used in the MEP nitrogen loading methodology to estimate the nitrogen load from septic systems. This provides a more accurate estimate of each individual parcel's likely nitrogen load to Aucoot Cove. Water use data obtained from Marion town billing records is scaled by 90% to account for consumptive use based on factors from other MEP studies. Marion properties missing water use data or on private wells were assigned an average water use based on their land use code. Average annual water use by land use code is shown in **Table 2**.

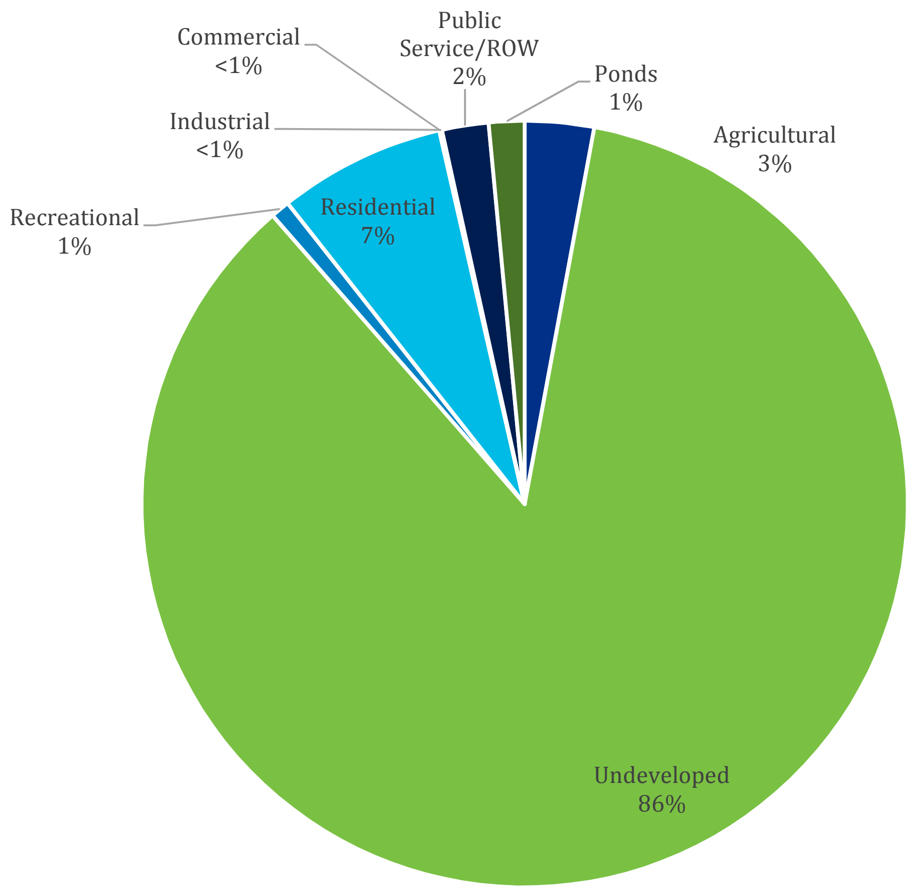


Figure 2. Relative Percentage of Land Use Types in the Aucoot Cove Watershed

Table 2. Average Water Use by Land Use Type in the Aucoot Cove Watershed in Marion

Land Use Type	Land Use Codes	Average Annual Water Use (CF/year/parcel)
Single Family Residential	1010, 1012, 1015	7,685
Two-family Residential	1040	7,963
Multiple Houses on One Parcel	1090	18,521
All land designated under Chapter 61 (not classified as Open Space)	6010	3,136

Water use data were not obtained for Mattapoisett properties within the Aucoot Cove watershed. The average water use for each land use code calculated from the Marion water use data was used with the Mattapoisett parcel data to estimate the septic load from the parcels on the western edge of Aucoot Cove.

The total nitrogen load from septic systems was calculated using the estimated or actual water use data for all properties served by septic systems within the Aucoot Cove watershed. Loads were calculated using the MEP loading factors, which assumes a total nitrogen concentration of 26.25 mg/l applied to the water use data after subtracting consumptive use. Properties served by the municipal sewer system are not included in this estimate as the wastewater from these properties is treated by the Marion WWTP.

Fertilizer Application

Fertilizers are applied to residential lawns and active agricultural properties and can have a significant impact on downstream water quality. Fertilizer loads are expressed in terms of an application rate (lb/1,000 sf) and a leaching rate (percentage of the fertilizer that goes into groundwater and ultimately to Aucoot Cove); the effective fertilizer load rate is the application rate multiplied by the leaching rate. There are no golf courses within the Aucoot Cove watershed. As stated above, The Bay Club was determined to be outside of the limits of the Aucoot Cove watershed.

Residential lawn areas were not explicitly measured for this analysis. Instead, lawn area was assumed to be 5,000 square feet based on the Cape Cod Commission Technical Bulletin 91-001. A survey performed by Dr. Brian Howes found that the fertilizer application rate is 1.08 pounds per 1,000 square feet and 50% of residents fertilize their lawns. MEP assumes a leaching rate of 20%.

Active agricultural areas were explicitly measured for this analysis, and consist of crops, productive woodlands, and cranberry bogs. While a few parcels in Marion are identified in the parcel database as crops, a review of satellite imagery indicates that there are no active farming operations in the Aucoot Cove watershed. Therefore, it was assumed that there is no fertilizer application on these areas. Productive woodlots are assumed to have an equivalent nitrogen application rate of 0.7 pounds nitrogen per 1,000 square feet with a leaching rate of 30%. There is one active, 27-acre cranberry bog within the Aucoot Cove watershed. The cranberry bog was assumed to be fertilized at a rate of 31 pounds of nitrogen per acre with an attenuation rate of 34% based on factors used in MEP nitrogen loading analysis.

Impervious Areas

Impervious areas considered in the MEP nitrogen loading analysis methodology include roads, runways, driveways, parking lots, and roof areas. Each type of impervious area is assigned an area and a nitrogen load. The Aucoot Cove watershed does not have any runways nor does it have any significant parking lot areas. Assumed areas, discussed below, are taken from assumptions used in other MEP studies. Assumed nitrogen concentrations for impervious runoff were derived from Cape Cod Commission Technical Bulletin 91-001, "Nitrogen Loading," and match those used in the MEP studies.

Road area was obtained from the MassDOT Roads official street transportation datalayer available from MassGIS. Road surface area was computed for each sub-watershed for MassDOT. A nitrogen concentration of 1.5 mg/l and an average annual precipitation depth of 40 inches was assumed for all roads based on constants used in prior MEP studies.

Driveway surface area was not explicitly measured. Each parcel was assumed to have a driveway area of 1,500 square feet and a nitrogen runoff concentration of 1.5 mg/l.

Roof areas were estimated from the MassGIS Building Structures database, which was derived from ortho imagery collected between 2011 and 2015. The total building footprint was calculated for each of the 7 sub-watersheds. Building runoff was assumed to have a nitrogen concentration of 0.75 mg/l.

Ponds

The MEP nitrogen analysis explicitly considers atmospheric deposition to large ponds and other waterbodies within the watershed. There are no large ponds in the Aucoot Cove watershed. While there are two large former cranberry bogs, these were assumed to have the lower natural area nitrogen concentration (see discussion below) as these are more similar to large, forested wetland areas and were repurposed to reduce non-point source loads to Aucoot Cove.

Natural Areas

Natural areas were assumed to represent all areas not explicitly accounted for in the impervious or agricultural surfaces noted above. The constants assumed for the nitrogen concentration from natural areas was assumed to be 0.072 mg/l, with a recharge rate of 27.25 inches per year. The nitrogen concentration and recharge rate are both derived from the MEP analysis.

Nonpoint Nitrogen Load Estimate

The MEP methodology outlined above was used to estimate the nonpoint source contribution to Aucoot Cove from septic systems, agricultural areas, impervious surfaces, and natural areas throughout the watershed. The sum of the load contribution from each of these sources was computed for each of the seven sub-watersheds.

Nitrogen attenuation in ponds is included for major ponds and salt marshes in the Aucoot Cove watershed area and was set to the midpoint of the range of attenuation values used in MEP nitrogen load assessments. The nitrogen attenuation rate was assumed to be 35% for all loads entering the active cranberry bog, Grassi Bog, and the salt marsh at the northern boundary of Aucoot Cove and 15% for streams and rivers, applied to the Lower Aucoot Creek and West Upper Aucoot Creek sub-watersheds. The Aucoot Cove East and Aucoot Cove West sub-watersheds were assumed to drain directly to Aucoot Cove, so no attenuation was assumed. The surface water nitrogen attenuation rate is approximate, and is used as a calibration parameter in MEP studies; since this analysis does not include the hydrodynamic water quality model used in the MEP analysis, the attenuation rate is considered to be constant based on the results of prior MEP studies.

Total Nitrogen Load – Nonpoint Sources

The total nitrogen load from nonpoint sources is 22.5 lb/d without attenuation and 17.3 lb/d with attenuation in the sources noted above. This is significantly larger than the nonpoint source loading estimate of 9.4 lb/d cited in the Draft NPDES Permit Fact Sheet. **Table 3** presents the distribution of unattenuated loads (in lb/yr) and **Table 4** presents the distribution of attenuated loads (in lb/yr) for septic systems, fertilizers, impervious surfaces, and natural surfaces.

Table 3. Distribution of Unattenuated Non-point Source Loads (lb/yr) in the Aucoot Cove Watershed

Watershed	Septic	Residential Fertilizers	Agriculture (woodlot, cranberry bog)	Impervious Surfaces	Undeveloped Land
Aucoot Cove East	120	117	0	149	169
Aucoot Cove West	1,030	139	0	131	101
East Upper Aucoot Creek	0	0	228	113	430
Effluent Brook	426	130	0	157	288
Lower Aucoot Creek	535	214	0	136	251
Upstream Grassi Bog	141	25	2,562	48	315
West Upper Aucoot Creek	0	0	0	41	223
Total	**2,251**	**626**	**2,790**	**775**	**1,775**
Total Unattenuated Nonpoint Source Loads to Aucoot Cove = 8,217 lb/yr or 22.5 lb/d					

The unattenuated results in **Table 3** are presented for comparison with the attenuated loads in **Table 4,** but the attenuated loads represent the portion of the nonpoint source nitrogen load that actually reach Aucoot Cove and therefore form the basis for the rest of this discussion and analysis.

Table 4. Distribution of Attenuated Non-point Source Loads (lb/yr) in the Aucoot Cove Watershed

Watershed	Attenuation Factor (%)	Septic	Residential Fertilizers	Agriculture (woodlot, cranberry bog)	Impervious Surfaces	Undeveloped Land
Aucoot Cove East	0[1]	120	117	0	149	169
Aucoot Cove West	0	1,030	139	0	131	101
East Upper Aucoot Creek	35[2]	0	0	148	74	279
Effluent Brook	35	277	85	0	102	187
Lower Aucoot Creek	15[3]	455	182	0	116	213
Upstream Grassi Bog	35	92	16	1,665	31	205
West Upper Aucoot Creek	15	0	0	0	35	190
Total		**1,973**	**539**	**1,813**	**637**	**1,343**
Total Attenuated Nonpoint Source Loads to Aucoot Cove = 6,305 lb/yr or 17.3 lb/d						

Notes:

1. Aucoot Cove East and Aucoot Cove West are assumed to drain directly to Aucoot Cove without attenuation.
2. East Upper Aucoot Creek, Effluent Brook, and Upstream Grassi Bog drain into large water bodies or into the salt marsh at the head of Aucoot Cove, and are assumed to have a 35% nitrogen attenuation rate based on values used in MEP studies.
3. Lower Aucoot Creek and West Upper Aucoot Creek are streams tributary to the active cranberry bog. A 15% attenuation rate within the stream is assumed based on values used in MEP studies.

The attenuated results in **Table 4** show that the dominant source of nitrogen varies depending on the land use in each sub-watershed. Dense development dominates the Aucoot Cove West sub-watershed in both Mattapoisett and Marion, and septic systems are the main source of nitrogen. A similar pattern is evident for Lower Aucoot Creek, which has houses along Route 6 immediately upstream of the salt marsh and Aucoot Cove. Fertilizer, largely from the active cranberry bog, is the main source of nitrogen from the sub-watershed 'upstream Grassi Bog.' The northern areas of the Aucoot Cove watershed – West Upper Aucoot Creek and East Upper Aucoot Creek – are primarily undeveloped, so the minimal total nitrogen contribution is from the forested areas.

In decreasing magnitude, the largest loads from nonpoint sources to Aucoot Cove come from 'agricultural' fertilizers and septic systems, followed by undeveloped land and residential fertilizers, and finally impervious surfaces. Roadways under the jurisdiction of MassDOT (Interstate Route 195 and Route 6) contribute approximately 229 lb/yr out of the 637 lb/yr estimated for impervious surfaces. However, the values in **Table 4** indicate that the majority of the fertilizer load comes from areas upstream of Grassi Bog, which is almost entirely agricultural contributions from the active cranberry bog and an active woodlot. **Figure 3** shows the relative contribution of each of these sources to nonpoint source nitrogen load to Aucoot Cove.

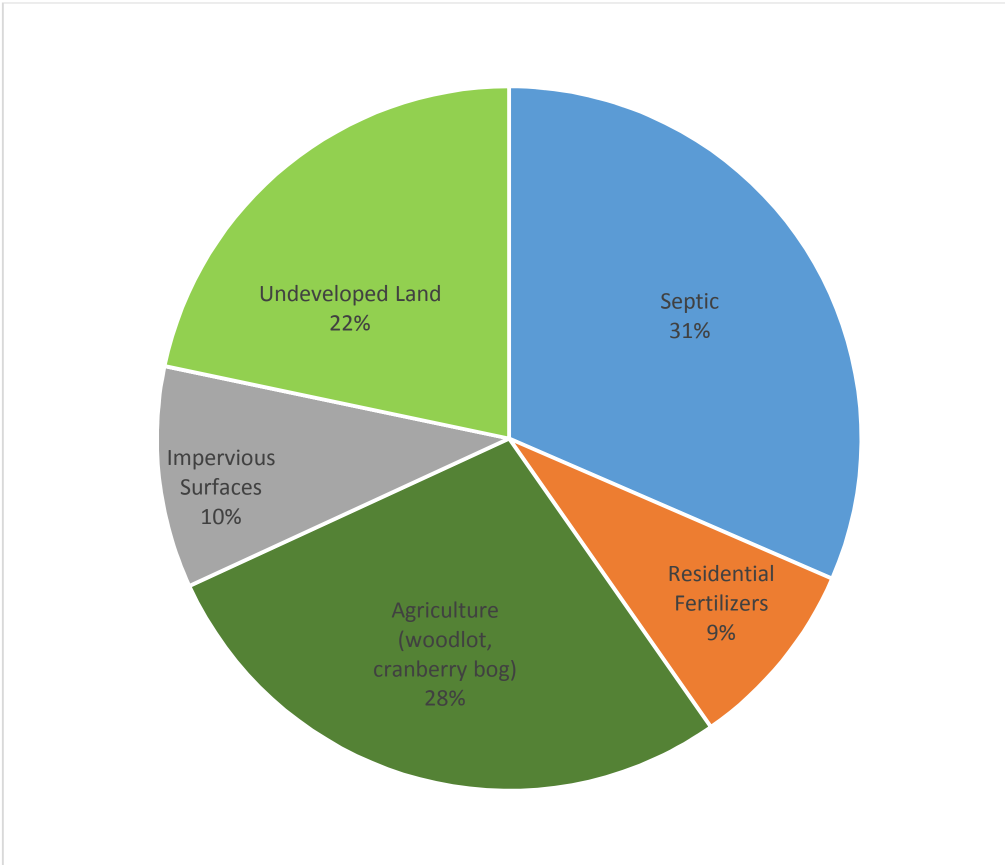


Figure 3. Relative Contribution of Attenuated Total Nitrogen Load from Nonpoint Sources to Aucoot Cove

Other Nitrogen Sources

The two other sources of nitrogen to Aucoot Cove are the effluent discharge from the Marion WWTP, which reaches the Aucoot Cove via Effluent Brook, and potential leakage from the unlined wastewater lagoons at the WWTP. The Marion WWTP daily load is 13.75 lb/d based on flow and total nitrogen concentration data from monthly operating reports (MORs) submitted to EPA and DEP between 2011 and 2013 as part of the Town's reporting requirements. This is the same value used by EPA in the Town's Draft NPDES permit.

An estimate of leakage from the three wastewater lagoons was developed using a water budget analysis of the lagoons based on water level data collected for five months using three high-resolution pressure transducers. The transducers were installed – one in each lagoon – to assess the change in depth over time. This study found that, at most, the leakage rate is on the order of 0.05 mgd from all three lagoons.

Water from each of the three lagoons was sampled on October 28, 2015, and tested for total nitrogen. The configuration of the three lagoons is shown on **Figure 1**. The total nitrogen concentrations in lagoons 1, 2, and 3 were 24.2 mg/l, 2.74 mg/l, and 1.3 mg/l, respectively. Lagoon 1 is the lagoon immediately adjacent to the WWTP and receives daily inputs of sludge, which may account for its higher total nitrogen concentration. Lagoon 2 receives occasional diversions of influent wastewater. The concentration in lagoons 2 and 3 are lower because their inflows are primarily precipitation. Water level data shows that lagoons 2 and 3 are at the same level and move together, and thus are assumed to be hydraulically connected; lagoon 1 also moves together with the other lagoons but averaged about a half inch lower during the transducer deployment.

As discussed in this memorandum, the updated LiDAR-based watershed delineation only includes lagoon 3 in the Aucoot Cove watershed. Therefore, combining the estimated potential leakage rate and measured nitrogen concentration in lagoon 3 yields a potential load from lagoon 3 to Aucoot Cove of approximately 0.3 lb/d.

The total nitrogen load to Aucoot Cove is the sum of attenuated nonpoint sources, the Marion WWTP, and the lagoon leakage load, which is 31.3 lb/d. **Figure 4** shows the relative percentage of nitrogen load between these three sources.

This assessment shows that the principal loads to Aucoot Cove are nonpoint sources and the WWTP, with the lagoon load a very small fraction of the total load to Aucoot Cove.

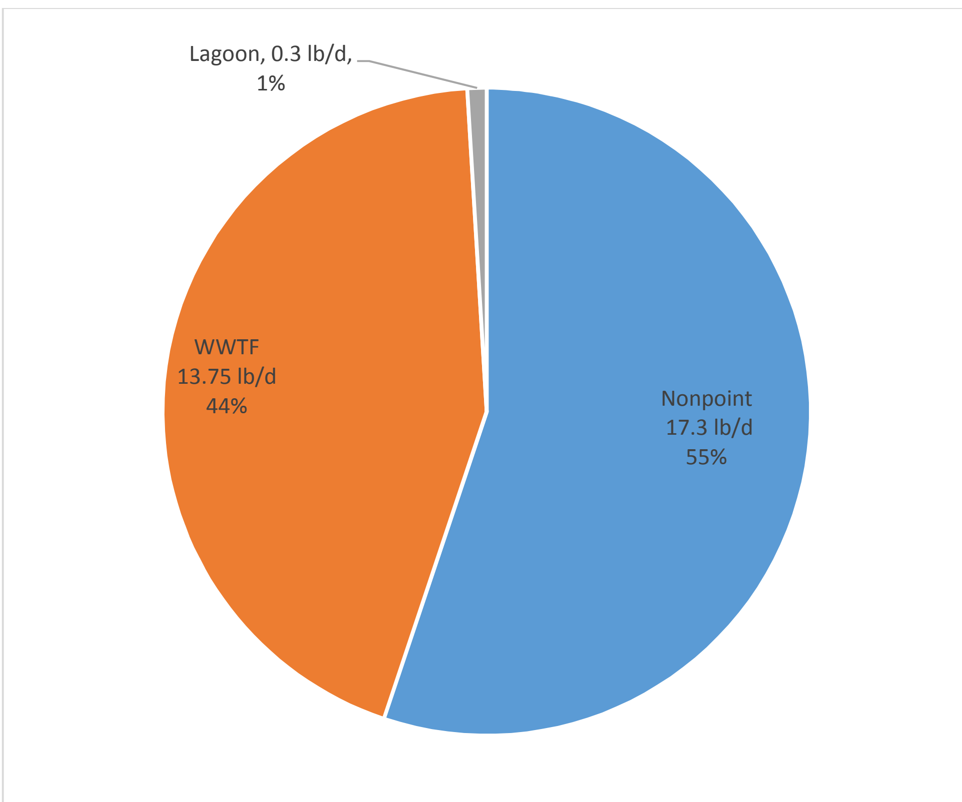


Figure 4. Relative Contributions of Total Nitrogen Load to Aucoot Cove

Comparison with the EPA Nitrogen Analysis

The load analysis presented in this memorandum differs significantly from the analysis and nitrogen loads presented in the Draft NPDES Permit Fact Sheet. EPA estimated that the total daily nitrogen load to Aucoot Cove was comprised of three principal components.

- A watershed nonpoint source load of 9.4 lb/d based on a superimposed areal loading rate for the Segregansett River. This load was not developed using site-specific data from the Aucoot Cove watershed.
- A WWTP point source load of 13.75 lb/d based on the total nitrogen concentration and flow rate reported in the WWTP monthly operating reports from 2011 – 2013.
- An additional groundwater nonpoint source load from the WWTP's three unlined lagoons of 45.75 lb/d based on a study of nitrogen loading by Horsley & Witten Group completed in 2011 in a report titled *Environmental Assessment of the Marion Wastewater Treatment Plan Sewage Lagoons*.

As documented in this memorandum, and in the response to comments on the Draft NPDES Permit submitted to the EPA in February 2015, there are serious and significant technical flaws in EPA's estimates of the watershed and lagoon nonpoint source load to Aucoot Cove. While EPA estimated a watershed load in the Fact Sheet, it noted that a detailed loading analysis of nonpoint source and stormwater point sources had not been performed and as a result used a superimposed areal load rate computed from a loading assessment performed by EPA for the Segregansett River watershed. The areal load based EPA analysis ignores the effects of woodlots, cranberry bogs, and septic systems on the nitrogen load to Aucoot Cove, and does not take into account differences in land use between the Segregansett River watershed and the Aucoot Cove watershed. The EPA estimated nonpoint source load using this methodology was 9.4 lb/d, which is a factor of two less than the land use-based analysis presented in this memorandum.

The lagoon load assumed from lagoon infiltration is overstated in the EPA loading analysis. The Draft NPDES Permit states that the total nitrogen concentration is 45.8 lb/d, based on an assumed lagoon leakage rate of 1 inch per day and a total nitrogen concentration in the lagoons of 20 mg/l. Key points of concern with the EPA estimate in the draft NPDES permit include:

- The leakage rate is equivalent to 0.5 mgd, which is greater than the average flow into the plant. First, if this much flow was indeed leaking, the lagoons would be dry most of the time (not accounting for precipitation). More importantly, the town only intermittently diverts some of the influent to the lagoons, and thus the estimate of leakage of 0.5 mgd is a gross overestimate. Our revised estimate shows that the leakage rate is at most 0.05 mgd.
- The total nitrogen concentration assumed to be present in the lagoons (20 mg/l) is an order of magnitude larger than concentration in the lagoon 3 – the lagoon expected to comprise the majority of flow to Aucoot Cove.

- Together, these factors mean that the potential contribution of the lagoons to Aucoot Cove nitrogen load is tremendously lower than the EPA estimate – on the order of 0.3 lb/d, over 100 times less than the EPA estimated 45.8 lb/d.

This updated load analysis presents a significantly different distribution of nitrogen sources to Aucoot Cove than is suggested in the Draft NPDES Permit Fact Sheet. While the analysis used to develop the Draft NPDES Permit conditions suggests that the largest and most significant load to Aucoot Cove is the leakage from the lagoons, this analysis suggests that the lagoon load is a small fraction of – less than a pound per day, and nearly 60 times less than the nonpoint source load. The analysis suggests that the largest loads are nonpoint sources and the WWTP and that removal nitrogen load contributed from the lagoons would have an insignificant impact on the overall nitrogen load to Aucoot Cove.

cc: Paul F. Dawson, Town Administrator
Board of Water and Sewer Commissioners, Town of Marion
Frank Cooper, Wastewater Treatment Plant Superintendent
Shawn Syde and Mike Guidice, CDM Smith

Section 2

Sediment Cores to Assess Historical Eelgrass in Inner Aucoot Cove

Smast

University of Massachusetts Dartmouth
The School for Marine Science and Technology

Technical Memorandum

To: Bernadette Kolb, CDM Smith

From: David Schlezinger, Research Associate, Coastal Systems Program
Brian Howes, Director, Coastal Systems Program

Re: Sediment cores to assess historical eelgrass in inner Aucoot Cove.

Date: March 19, 2016

Background: The Town of Marion operates a WWTF within the watershed to Aucoot Cove. Some of the treated effluent is carried by the freshwater stream known as Effluent Brook discharging to the salt marsh at the head of Aucoot Cove and ultimately into the receiving waters of Aucoot Cove. Given the nitrogen in the treated effluent there is a question of how eel grass habitat maybe influenced in the open basin of Aucoot Cove and particularly in the innermost region where the open basin waters intersect with the outer margin of salt marsh. The present lack of eelgrass in this innermost region has recently been questioned as to its cause: a natural phenomenon due to suitability or caused by high nutrient waters ebbing from the salt marsh. The work described below is aimed at addressing this question.

At present, there is significant eelgrass in the open water basin of Aucoot Cove, but coverage does not extend to the marsh border. In fact, there is no evidence of historic or recent eelgrass coverage in the region of the basin bordering the salt marsh in the upper region and specifically, adjacent Haskell Island and in the shallow area to the east. Recent analysis by CDM Smith indicates that this region predominantly consists of sandy substrate with stable (year to year) sand waves. Sand waves are clearly apparent in multi-year aerial photos and are generally in the same places. Since sand waves present an unstable substrate for eelgrass, it has been suggested that there may have been eelgrass in this region before the development of the sand waves and that the sand waves are "obscuring" detection of the historic eelgrass habitat. The concept is that while it is clear that there has not been eelgrass in recent times, that there was once eelgrass and possibly once again could be eelgrass should the substrate stabilize. To test for this, an examination of the historic surface was undertaken by penetrating the sand overburden and analyzing the prior sediment surface where eelgrass rhizomes/roots and potentially eelgrass seeds should be preserved. Preservation of the historic surficial sediments is aided by the present overburden of sand.

Approach: Scientists from the Coastal Systems Program (CSP), University of Massachusetts-Dartmouth, School for Marine Science and Technology (SMAST) designed an approach to determine if historic eelgrass habitat was present in this upper region of Aucoot Cove. As an initial survey 6 cores were collected using a vibrating head until refusal from a transect beginning near the main tidal outlet from the salt marsh extending east at intervals of approximately 50 meters (Figure 1). Cores were collected about mid-way between the marsh edge and Haskell Island offshore. Two additional cores from active eelgrass beds were collected for comparison. Cores penetrated the overlying sand layer at all sites and collected a section of the fine-grained sediments. Cores were collected during a half moon tide in less than 0.5m water depth. Although small regions on the eastern edge of the transect and the channel on the western edge of the cove were deeper, most of the region behind Haskell Island appeared to be intertidal during at least some of the lunar cycle.

Upon return to the laboratory cores were split lengthwise into equal halves. One half was archived for future use and the other half was photographed and described visually for gross morphology and stratigraphy and then destructively analyzed. The core halves were sectioned at morphological boundaries, weighed and subsequently wet sieved through a 300μm screen. Material trapped by the sieve was examined with 6X -12X dissecting scope. All seeds, roots, rhizomes, and other potentially identifiable materials were logged and photographed for later identification and interpretation.

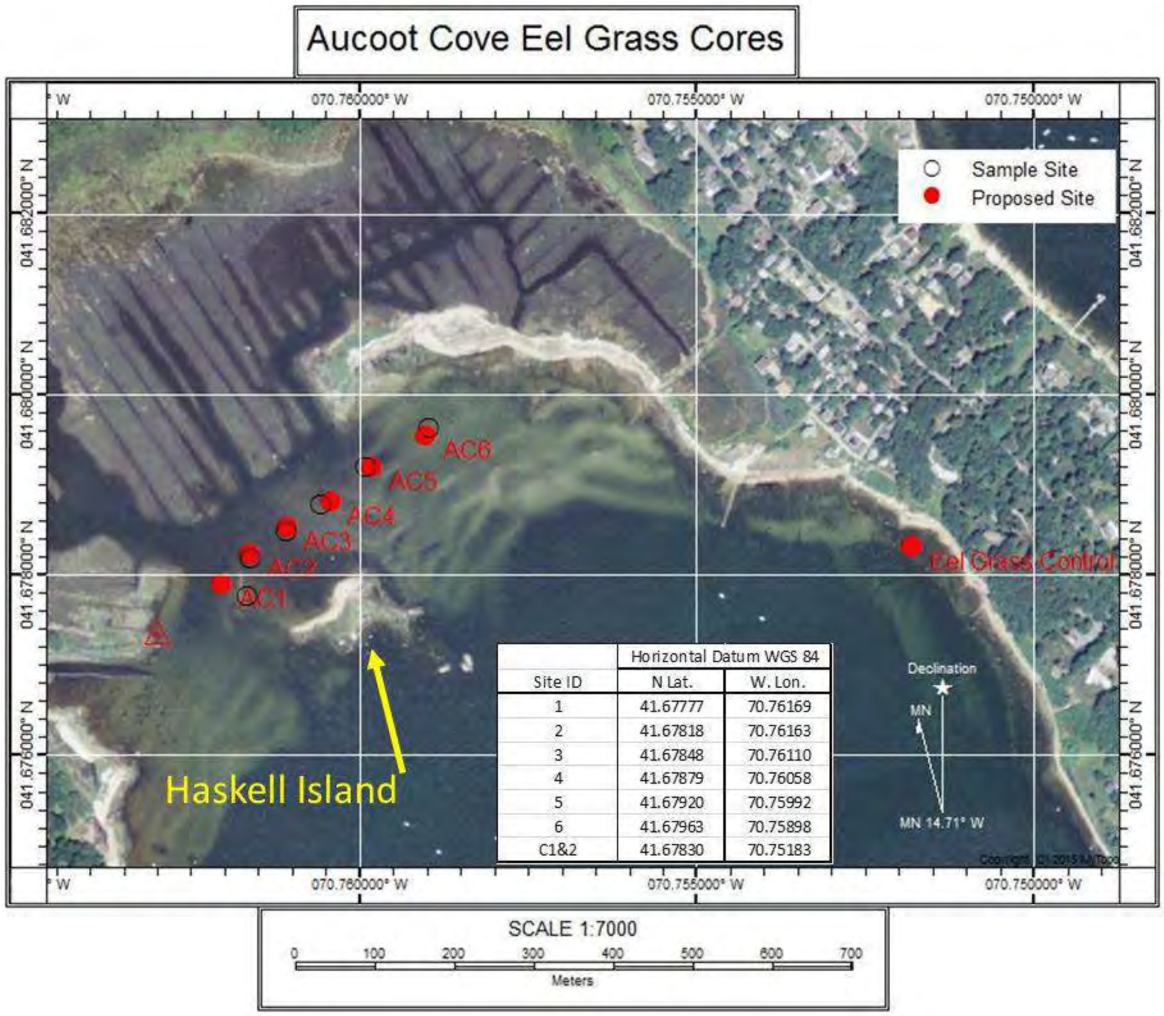
Discussion: Core lengths varied from a maximum length of 55cm to a minimum of 15cm (Figures 2-9). In every location core penetration was limited by a layer of coarse sand and gravel. The bottom layer of Core 2 and Core 4 consisted of decomposed granite with no secondary weathering suggesting sub-aerial deposition prior to the formation of the sand waves and estuarine sediment transport. Control cores taken from an eelgrass area along the east side of the cove (Figure 1) contained abundant roots and robust rhizomes throughout the 15cm depth collected (Cores C1-C2, Figures 8-9), however, no seeds were found. Unarticulated rhizomes were found and showed good preservation. Poor preservation might explain the absence of eel grass seeds north of Haskell Island, however, seed coats from eel grass were found in Cores 2, 3, and 4 at similar depths ranging from 14.5-27.5 cm below the sediment surface suggesting that preservation was possible.. Only six seed coats were found in the entire study. Unfortunately, seed coats can be deposited with other organic detritus and are not indicative of a deposition within an existing eelgrass bed, i.e. they can be transported to a depositional area and may only indicate the presence of nearby eel grass beds. This transport concept is clearly seen in the results from the transect cores from inside of Haskell Island, all of which contained numerous seeds derived from terrestrial plants, suggesting that dominant seed transport occurred from the land margin outward. It should be noted that all the core sections contained abundant organic detritus; only the items listed in Table 1 were identifiable as far as species or provenance. On core logs organic detritus was only logged when discrete dense bands were observed.

The absence of historic eelgrass deposits or traces nearshore to the marsh, shoreward of Haskell Island is consistent with the work of Costello (2011) and Costa (1988) who did not record the presence of eel grass beds in the region of Aucoot Cove north of Haskell Island in any of their eel grass inventories of Buzzards Bay dating back to 1988, though both indicate the presence of extensive eel grass beds south of Haskell Island. Further, Costello (2011) indicates an expansion of eel grass beds in the outer portions of Aucoot Cove beginning in 2001 and extending to the near present, but no colonization inshore. These observations are consistent with the observations in this report and with anecdotal evidence presented by land owners extending back to the 1960's. Much of the area inshore of Haskell Island is intertidal, and thus not conducive to eel grass survival; historic aerial photographs available on Google Earth suggest that the sand waves comprising this area have been a stable part of this environment since at least 1995. It is likely that the sand waves also create an unsuitable habitat for eelgrass colonization as does the intertidal nature of the study area during at least some portions of the lunar cycle.

Based upon the results of this study and previous work it appears unlikely that eel grass was present in the study area over the past half century. Neither preserved seeds, nor rhizomes were observed in the transect cores, though the data indicates that preservation was possible. Photographic evidence (Costa and Costello) did not indicate eel grass presence after 1988. Lacking evidence of eel grass in the sedimentary record prior to the presence of sand waves and no photographic evidence after the presence of sand waves we believe that eel grass would be a poor metric for interpreting the environmental health of Aucoot Cove between Haskell Island and the salt marsh. Outside of Haskell Island eel grass has been present since at least 1988 and although these eel grass beds declined during the 1990s they have been expanding since then and now occupy nearly the same area as they did in 1988, indicating an improving estuarine environment. There was no evidence for the prior existence of eel grass between Haskell Island and the marsh at the head of Aucoot Cove.

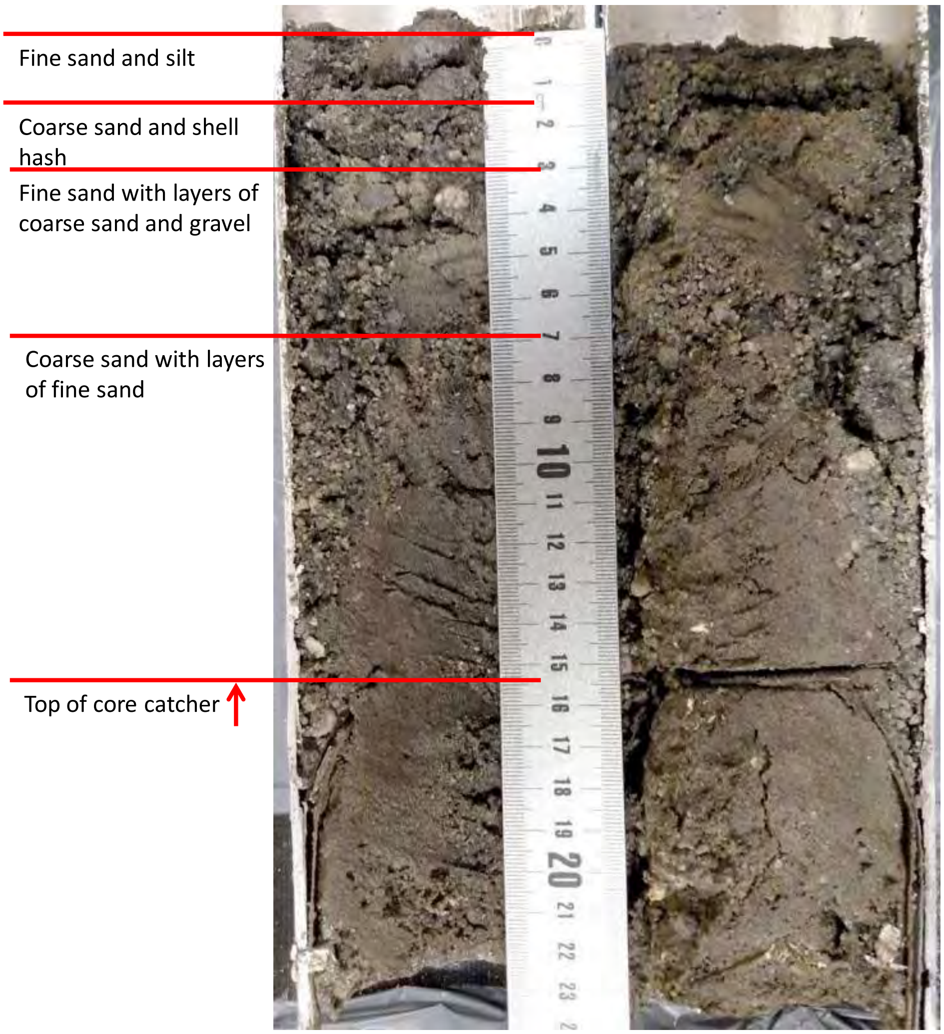
Costa, J.E. 1988. Eelgrass in Buzzards Bay: Distribution, production, and historical changes in abundance. EPA 503/4/88-002. 204pp.

Costello, C. T., Kenworthy, W. J., 2011. Twelve-year mapping and change analysis of eelgrass (*Zostera marina*) areal abundance in Massachusetts (USA) identifies statewide declines. Estuaries and Coasts 34, 232-242.



Site ID	N Lat.	W. Lon.
	Horizontal Datum WGS 84	
1	41.67777	70.76169
2	41.67818	70.76163
3	41.67848	70.76110
4	41.67879	70.76058
5	41.67920	70.75992
6	41.67963	70.75898
C1&2	41.67830	70.75183

Figure 1. Aucoot Cove coring locations, both proposed and actual, in relation in Haskell Island and Aucoot Creek. Inset provides core names and GPS coordinates.



Core 1
N 41.677 W 70.762
Total Length 15.5cm

Figure 2. Photograpgh and observation log of transect Core 1.

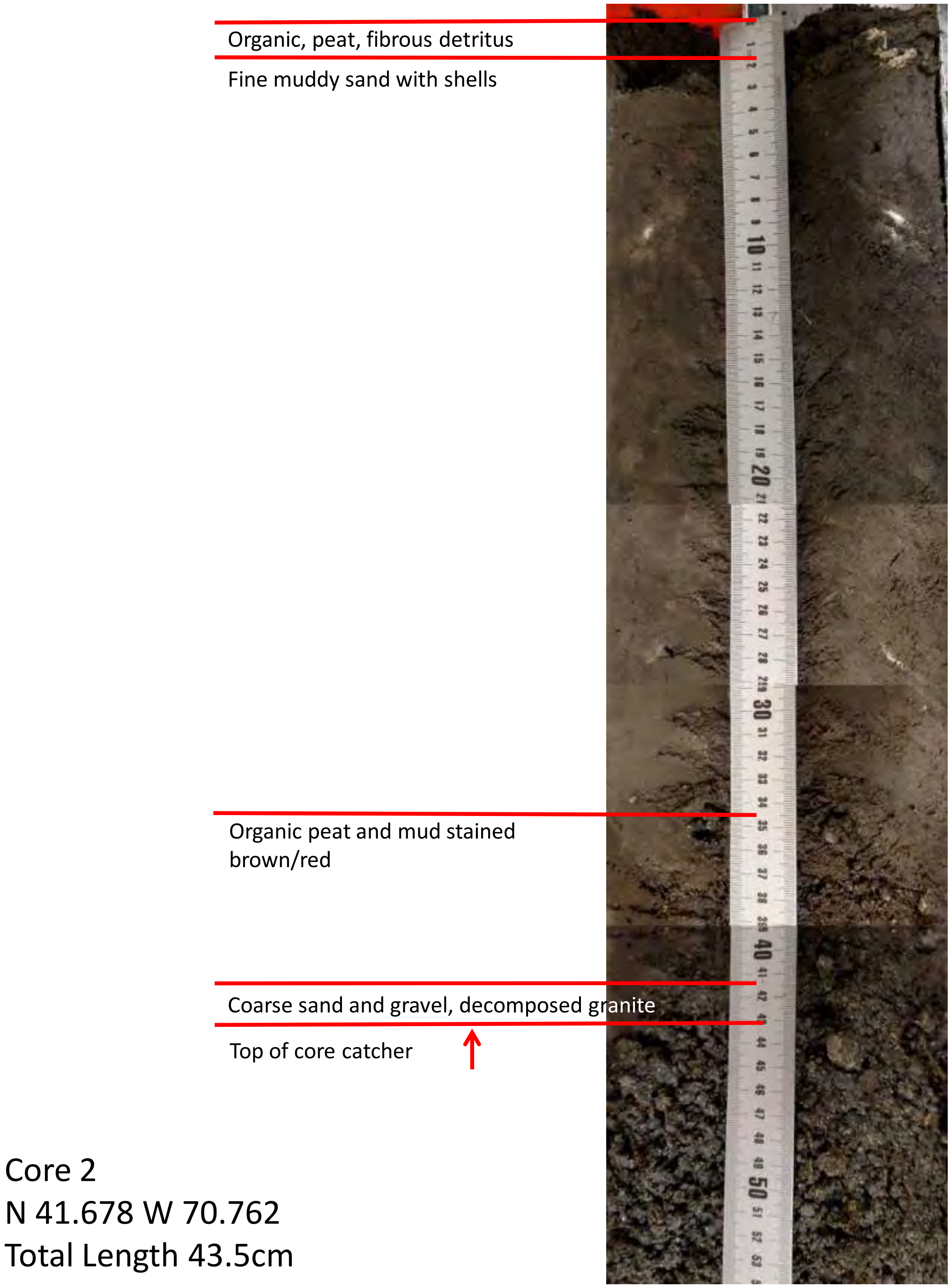
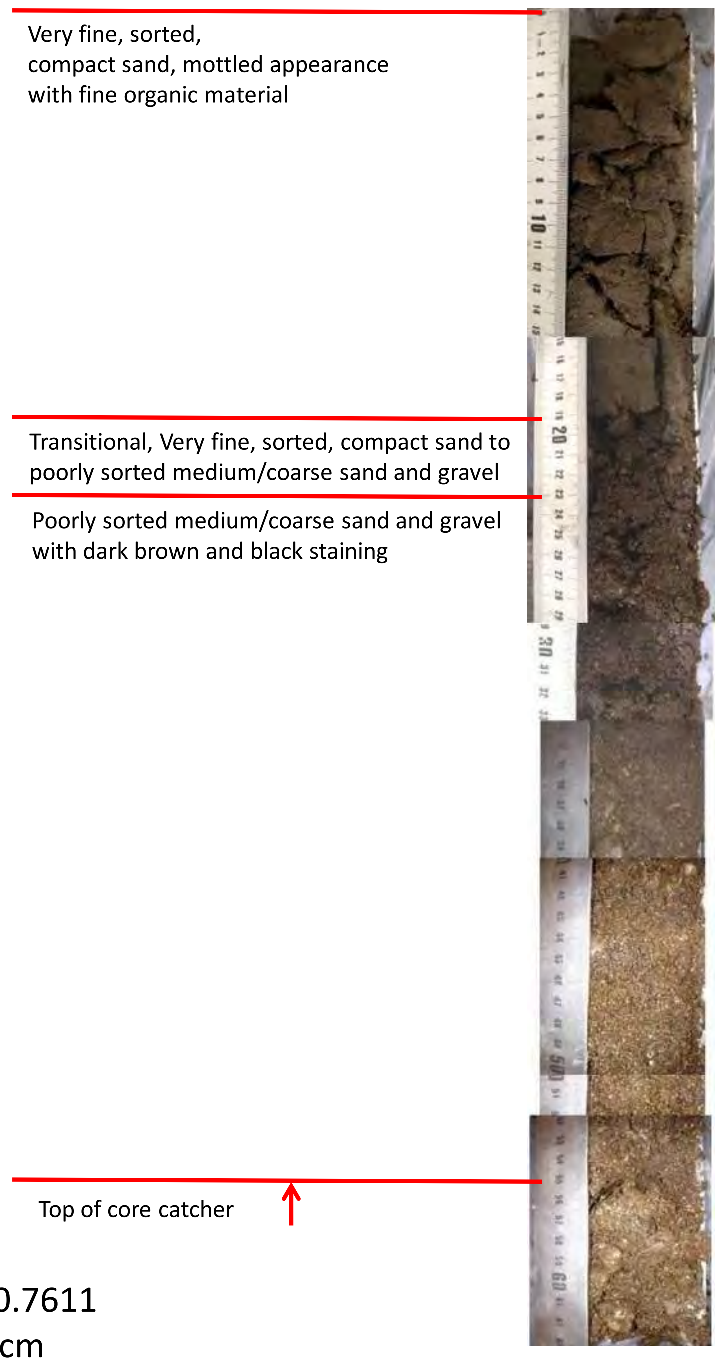


Figure 3. Photograpgh and observation log of transect Core 2.



Core 3
N 41.6785 W 70.7611
Total Length 55cm

Figure 4. Photograpgh and observation log of transect Core 3.

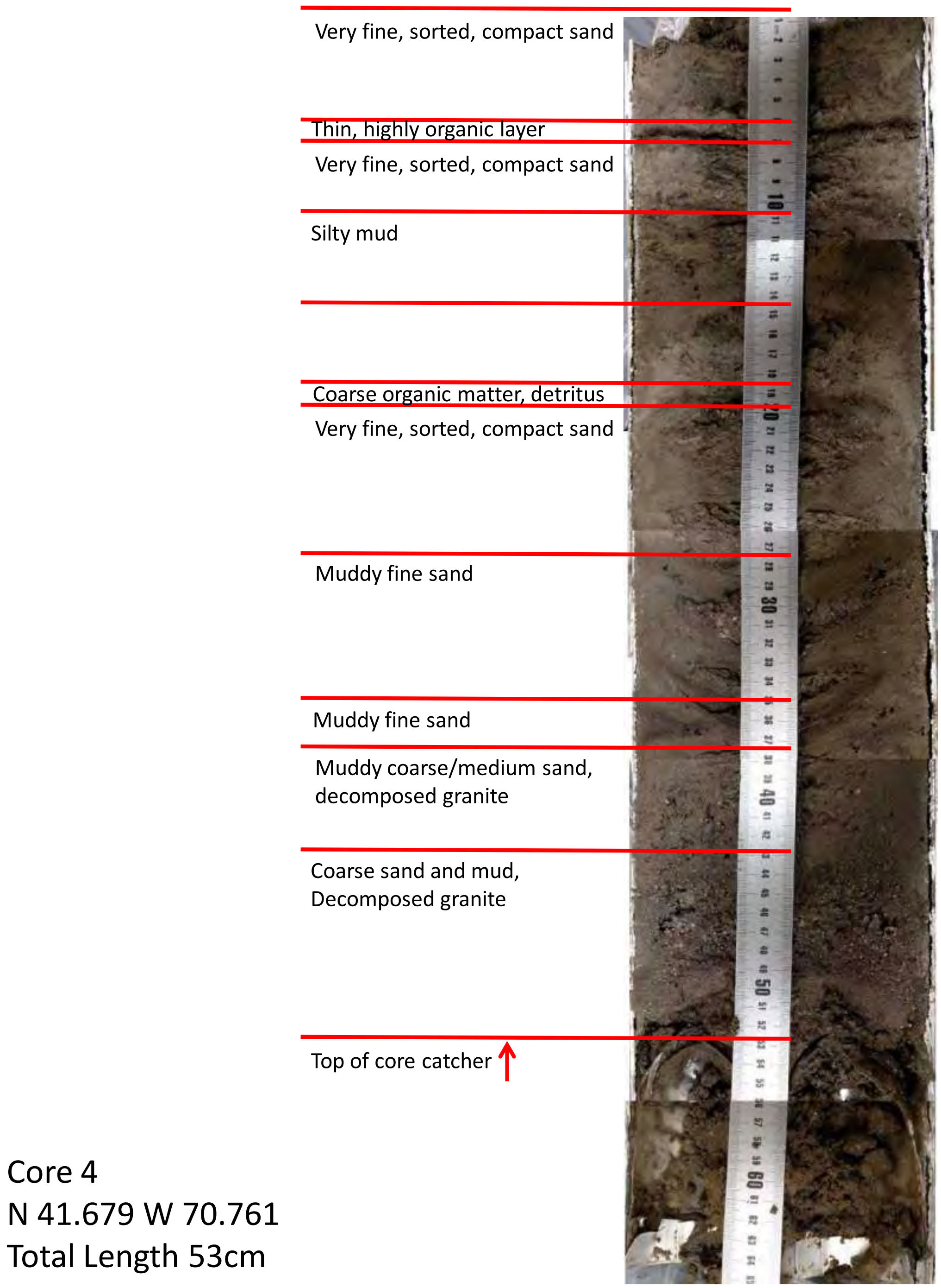
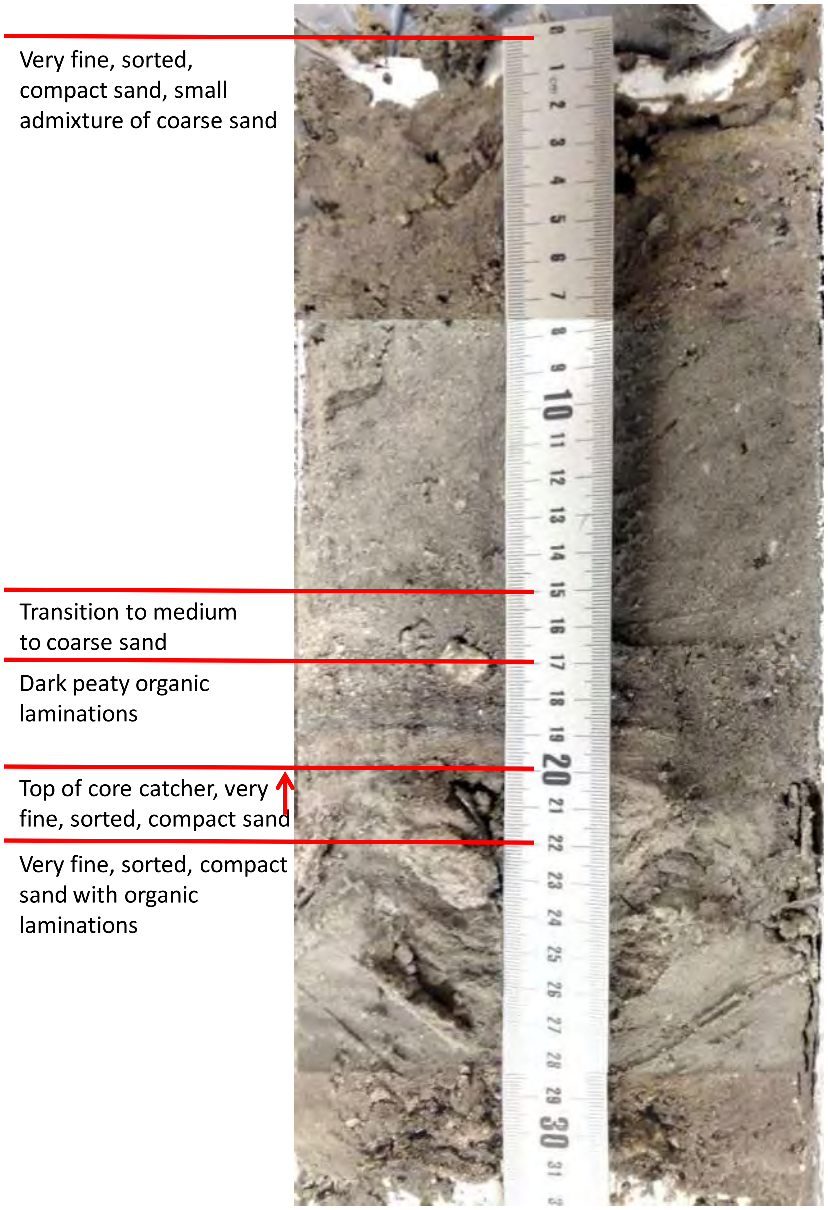
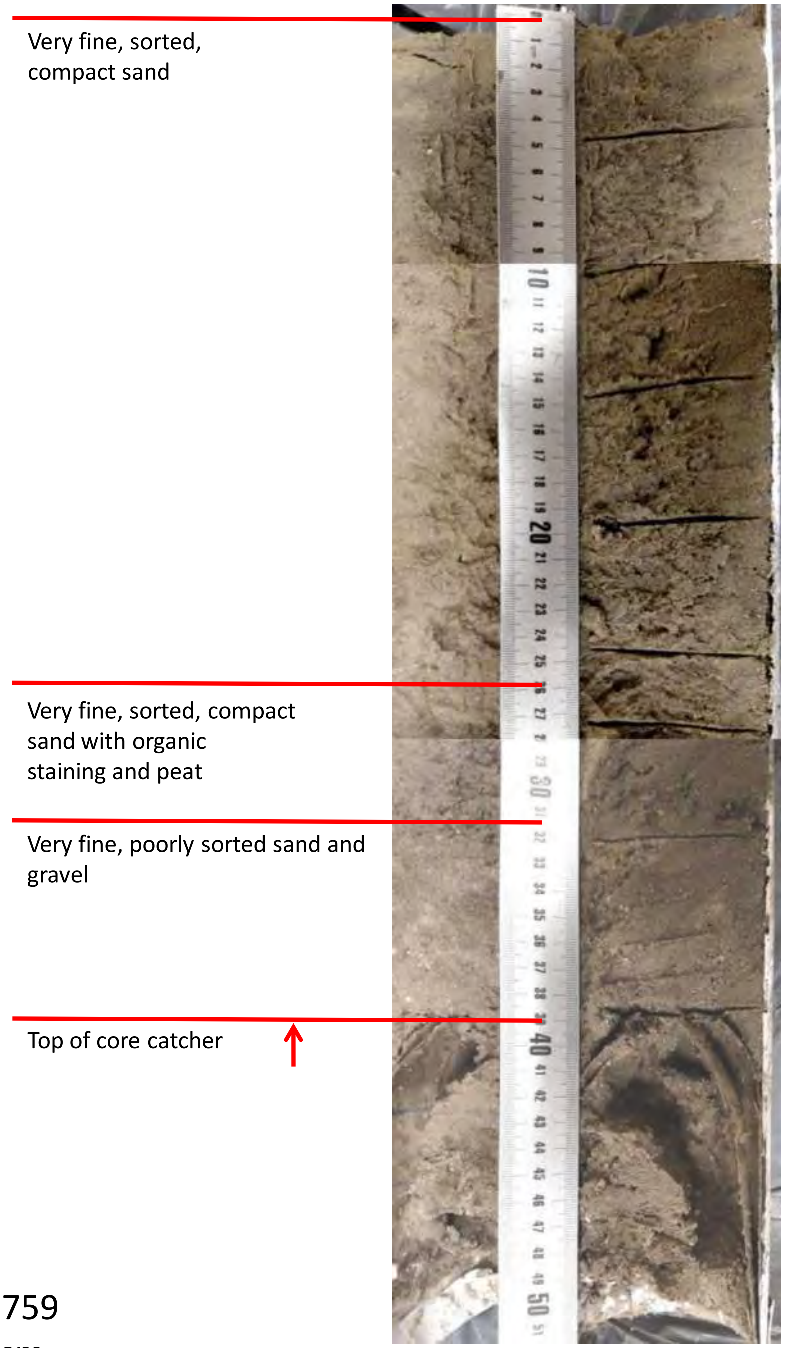


Figure 5. Photograpgh and observation log of transect Core 4.



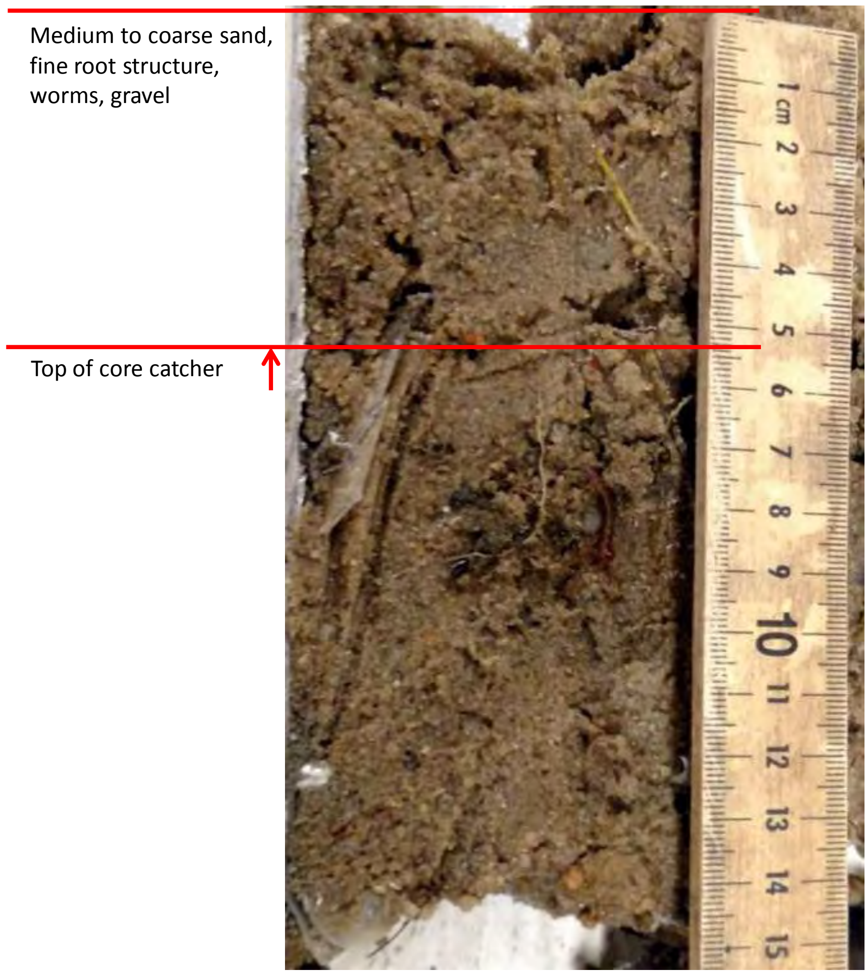
Core 5
N 41.679 W 70.760
Total Length 20cm

Figure 6. Photograph and observation log of transect Core 5.



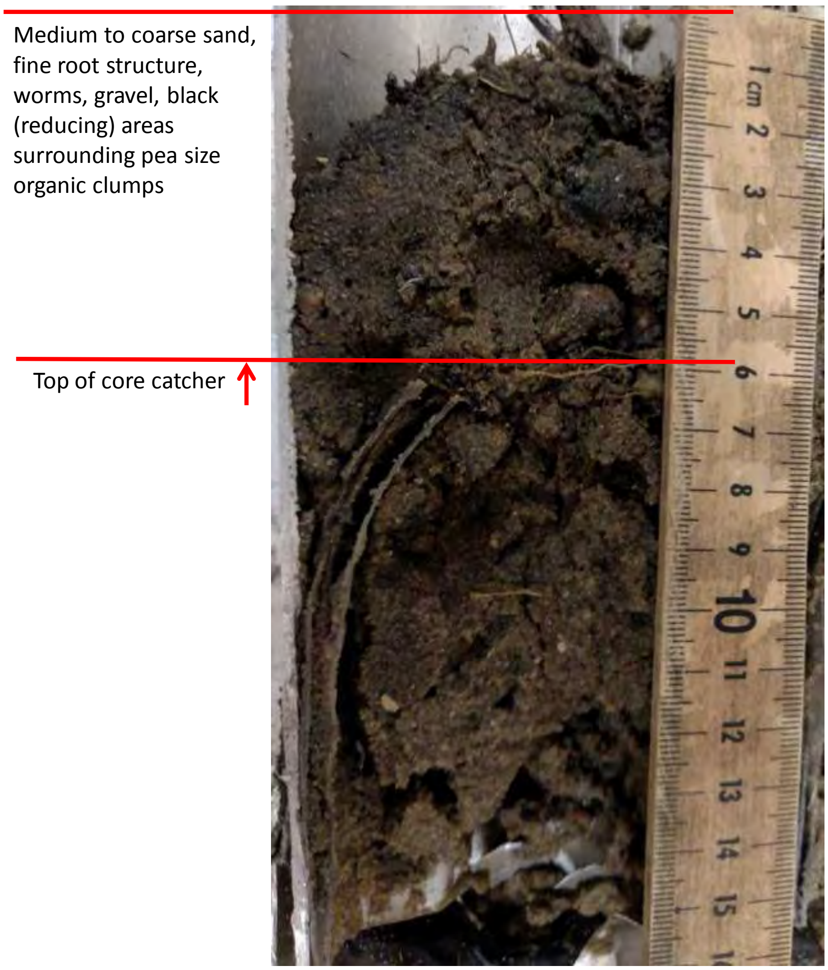
Core 6
N 41.680 W 70.759
Total Length 39cm

Figure 7. Photograph and observation log of transect Core 6.



Core C1 Eel Grass Control
N 41.678 W 70.752
Total Length 15cm

Figure 8. Photograph and observation log of eel grass Control Core C1.



Core C2 Eel Grass Control
N 41.678 W 70.752
Total Length 15cm

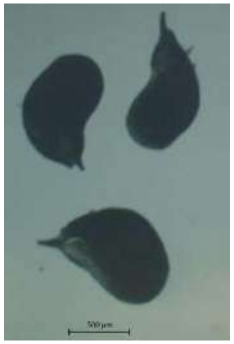
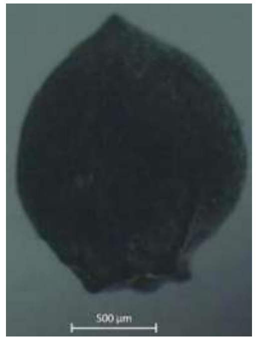
Figure 9. Photograph and observation log of eel grass Control Core C2.

Table 1. Identifiable organic contents (wet sieved >300μm) of core sections viewed under 6X-12X magnification.

Table 2. Log of both type and quantity of organic content observed in core sections.

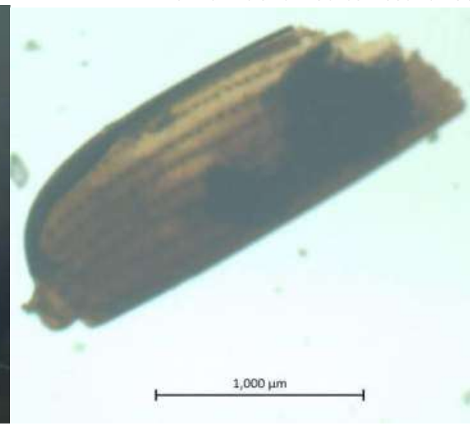
Core 1		Core 2		Core 3		Core 4		Core 5		Core 6	
Depth (cm)	Type-ID	Depth (cm)	Type-ID	Depth (cm)	Type-ID	Depth (cm)	Type-ID	Depth (cm)	Type-ID	Depth (cm)	Type-ID
0-1.5	NA	0-1.5	NA	0-5	TS-2	0-6	NA	0-5	TS-1	0-5	NA
1.5-3	NA	1.5-6.5	NA	5-10	TS-2	6-7	TS-11	5-10	TS-1	5-10	NA
3-7	NA	6.5-11.5	TS-1; U-1	10-15	TS-45	7-10.5	TS-5	10-15	TS-1	10-15	TS-1
7-15.5	NA	11.5-16.5	TS-14 RH-1*	15-19	TS-27	10.5-14.5	TS-6	15-17	TS-5; U-1	15-20	NA
		16.5-21.5	TS-27	19-23	TS-24; EG-2	14.5-18.5	TS-41; U-2; EG-1	17-20	TS-2	20-25	TS-2
		21.5-26.5	TS-24; EG-1	23-30	TS-9; U-1	18.5-19.5	TS-15	20-22	TS-6	25-28	NA
		26.5-31.5	TS-7	30-35	NA	19.5-27.5	TS-130; EG-2	22-32	TS-2	28-32.5	TS-6
		31.5-34.5	TS-2	35-40	NA	27.5-35	DG; TS-140			32.5-39	TS-3; RH-1
		34.5-41.5	DG; TS-14	40-45	NA	35-37.5	DG; TS-47				
		41.5-54.5	DG; TS-1	40-50	NA	37.5-43	DG; TS-39				
				50-55	NA	43-53	DG; TS-5				
				55-	NA	53-	DG; NA				

Core Control 1		Core Control 2	
Depth (cm)	Type-ID	Depth (cm)	Type-ID
0-5		0-5.5	
5-15		5.5-16	

TS=**T**errestrial **S**eed  

U=**U**nidentified terrestrial seed 

EG=**E**el **G**rass Seed Coat 



RH=**R**oot **H**airs  **RH***= 

Figure 10. Key to abbreviations used in Table 1 with representative photographs.

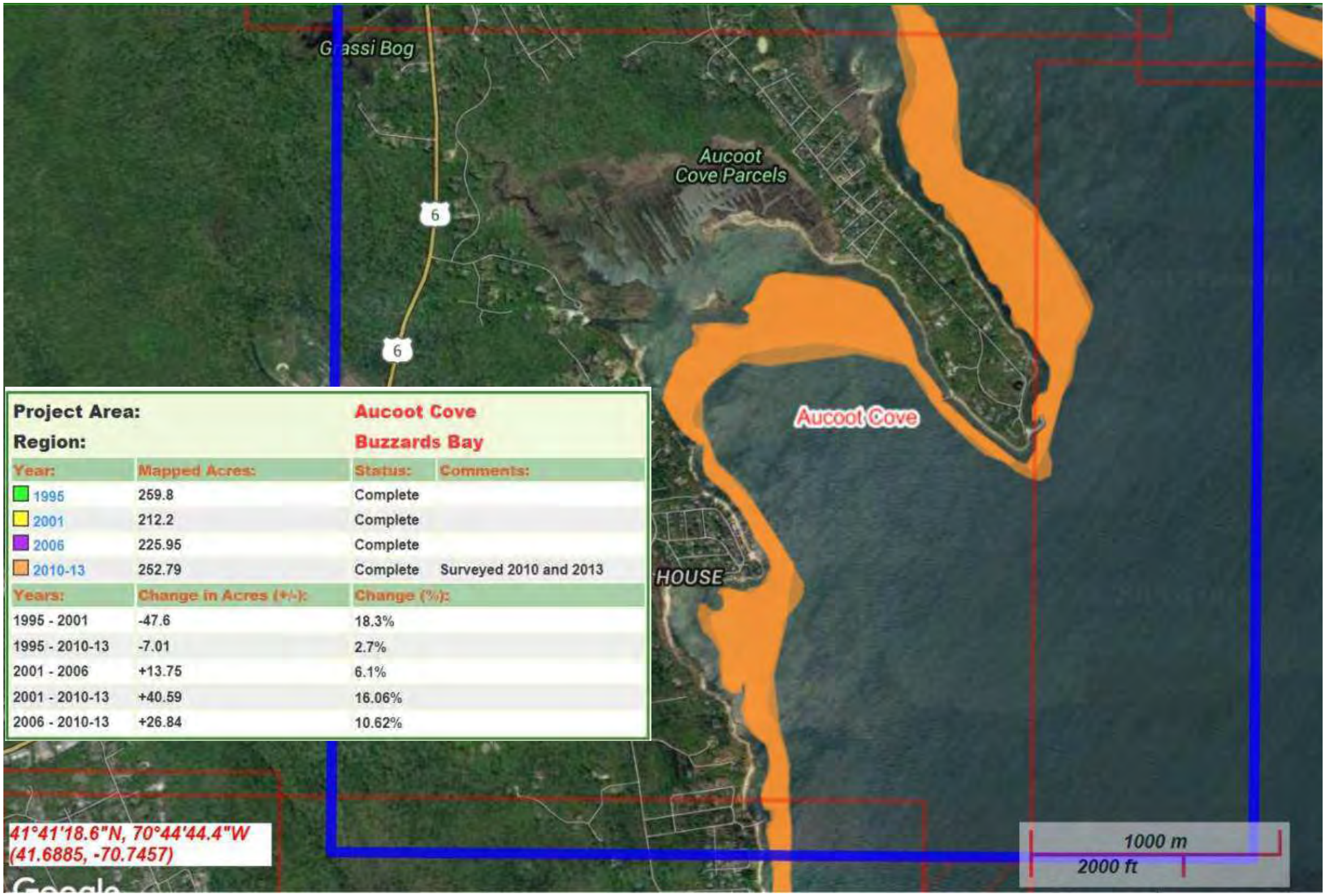


Figure 11. Eel grass coverage map (2010-2013) from MassDEP Eelgrass Mapping Program, C. Costello.

http://maps.massgis.state.ma.us/images/dep/eelgrass/eelgrass_map.htm

Section 3

WWTP Influent Equalization Lagoon Improvements, Analysis of Alternatives

CDM Smith

Memorandum

To: *Mr. Robert Zora, Superintendent of Public Works*

From: *William McConnell, P.E.*
Matthew Pitta, P.E.

Date: *April 4, 2016*

Subject: *Wastewater Treatment Plant Influent Equalization Lagoon Improvements Analysis of Alternatives*

Background

The On November 28, 2014 the United States Environmental Protection Agency (USEPA) issued a Draft National Pollution Discharge Elimination System (NPDES) Permit for the Town of Marion's (the Town) Wastewater Treatment Plant (WWTP) located off of Benson Brook Road. Three of the conditions outlined in the Draft NPDES Permit include reduction of total phosphorous (TP), and further reduction of total nitrogen (TN) and copper concentrations in the WWTP effluent. The draft permit also requires that the existing lagoons at the WWTP be lined as a condition of future use. The Town submitted comments on the draft permit and is awaiting response from the USEPA.

This memorandum describes improvements that would be required at the WWTP to meet the draft permit limits and lagoon requirements as written. As an alternative compliance strategy, CDM Smith also evaluated several options to discharge the treated effluent by extending the outfall to the saltmarsh or the ocean; these options would potentially eliminate permit requirements for TP, TN, and/or copper. These alternatives are presented in a technical memorandum "Town of Marion Wastewater Treatment Plant Outfall Alternatives" dated March 10, 2016. The various improvements described in this memorandum were developed independently of the outfall alternatives.

Existing Operations

The Marion WWTP, originally constructed in 1969 and upgraded in 2005, is a sequencing batch reactor (SBR)-based plant that provides advanced treatment; facultative lagoons from the plant's original design were converted for use as influent equalization basins with the 2005 upgrade. The design average daily flow to the WWTP is 0.588 million gallons per day (mgd). The WWTP has a peak design capacity of 1.18 mgd. The flow streams described below can be seen on the Process Flow Diagram included as **Figure 1.**

Raw influent is pumped to the headworks of WWTP from the Front Street pump station, which is capable of pumping at a peak rate of up to approximately 3 mgd. Flow entering the headworks discharges to one of two aerated channels, then passes through a mechanical screen. There are means of bypassing the mechanical screen and sending flow through a manual screen, should the mechanical screen need maintenance. A grit classifier, located downstream of the screens is a grit chamber, and is the final apparatus in the headworks.

Flow from the headworks enters a splitter box where it is routinely directed to the pipe feeding the SBRs; however, during high flow periods, some of the flow is diverted to the lagoons for storage and future treatment. The splitter box has a motorized weir to control diversion of flow that WWTP staff typically operate manually. Raw influent can also be sent directly from the Front Street pump station to the lagoons, which is done on rare occasions for maintenance of the aerated influent channels or other major maintenance projects at the plant.

Influent is treated in the SBRs and is then discharged to an equalization tank. From the equalization tank, it is pumped through a flow meter to the disk filter building. A meter on the pump discharge monitors effluent plant flow. Flow from the equalization tank can also be pumped (recycled) to the lagoons; this happens rarely to allow for maintenance of the downstream tertiary treatment facilities.

Flow entering the disk filter building is sent to one of two filter bays. After passing through the disk filters effluent flows through an ultraviolet (UV) disinfection system in the UV building. The UV building has two channels, though only one of the channels has UV equipment. Typically, both channels would have UV equipment to provide redundancy, but storage available in the lagoons allows for the discontinuation of flow should the UV equipment require maintenance. WWTP effluent leaving the UV building enters the outfall pipe that currently discharges to Effluent Brook south of Mill Street (Route 6).

The WWTP has several ancillary systems. Odor is mitigated by a biofilter, located next to the headworks building. Plant water is drawn from the equalization tank for use as spray water, soda ash solution, and other non-potable use throughout the WWTP. Soda ash, stored in a silo onsite, is added to flow prior to entering the SBRs to increase alkalinity. Waste activated sludge (WAS) and scum from the SBRs, as well as sidestream flows including disk filter backwash, floor drains, biofilter drainage, and sanitary wastewater from the facilities at the WWTP are all sent to the Lagoon 2 for treatment and recycle back through the plant at an average combined rate of 70,000 gallons per day (gpd).

The SBRs, and the remainder of the plant, were designed to have a 1.18 mgd maximum treatment rate, though the typical maximum throughput is limited to approximately 0.95 mgd. The difference between design and operational maximum flow rates is a function of programming, instrumentation, and staff professional judgement as to the best way to utilize the SBR/lagoon operational flexibility. At present, when influent flow nears 0.95 mgd, staff begins diverting to the lagoons, as a way to utilize the benefits to process stability the lagoons provide.

Flow is metered at both the Front Street pump station and the pipe feeding the SBRs. There is no flow meter on the line from the splitter box to the lagoons. The flow to the lagoons can be estimated by level over the diversion weir but is more accurately monitored by calculating the difference between the meter at the Front Street pump station and the meter upstream of the SBRs.

There are three 8-foot-deep (maximum) lagoons, totaling about 20 acres and approximately 52 million gallons. These lagoons pre-date the commissioning of the SBR upgrades in 2005, and were originally the plant's primary treatment process. Lagoon 1 and 2 are five acres each and Lagoon 3 is ten acres. The lagoons are all connected hydraulically by a series of pipes with valves intended to allow for control of flow between each lagoon; at present. The lagoons are open to the atmosphere and are aerated primarily to help control odor, although the transferred oxygen does reduce biological oxygen demand (BOD).

Diverted flow from the splitter box, or from upstream of the headworks, enters the lagoon inlet box, which was initially designed to distribute flow to either Lagoon1 or 2, however, the valve on the feed to Lagoon 1 is stuck in the closed position, so all diverted flow currently enters Lagoon. 2. Sidestream flows are also directed to Lagoon 2. All WAS and scum from the SBRs is discharged to Lagoon1, though the scum pumps for the SBRs are rarely used. The WAS and scum pumped to the lagoons is aerobically and anaerobically treated where it naturally degrades.

Flow from the lagoons can be pumped to either upstream of the headworks or the SBR influent pipe, though the latter is infrequent. The lagoon recycle pump discharge line is metered, with an average pumping rate of approximately 100 gallons per minute (gpm) and up to 120 gpm when water levels in the lagoons are particularly high.

Lagoon Sizing and Wastewater Treatment Plant Peak Hydraulic Capacity

The existing lagoons afford a significant advantage to the WWTP operation by providing substantial storage capability during high flows and periods of WWTP maintenance. The large storage capacity (52 million gallons total volume) is proportionally much higher than would normally be anticipated for a 0.588-mgd treatment plant, and therefore it is prudent to determine whether the full existing storage volume is needed, or whether it may be possible to reduce this volume while still meeting the plant's needs. CDM Smith performed an analysis to determine the lagoon volume needed for influent peak flow equalization to ensure the continued successful operation of the WWTP, while seeking to reduce the costs associated with lining of the lagoons as required per the draft permit. The analysis was completed utilizing a number of different data sources including:

- Historic flow data as measured at the Front Street pump station from October 2005 to August 2015,
- Lagoon depth measurements -- ultrasonic depth sensors that were installed in stilling wells at each of the three lagoons and collected data from July to December 2015.

In addition to the current flow condition, an increased flow condition was also considered to include anticipated future expansion of the collection system. To determine the increased condition flows, the 2005-2015 daily average flow of 0.54 mgd was increased to account for future connections to the wastewater collection system from the Indian Cove/Harbor Beach area (151 additional connections) and a general 10% increase in connections (180 additional connections), bringing the evaluated total number of connections from 1,648 connections to 1,979. This increased the long-term average flow to 0.65 mgd.

Sidestream flows, precipitation, WAS, scum, and evaporation were unchanged; they are small, and thus, do not influence the water budget significantly.

Another consideration in the analysis was a modification of the lagoon aeration system to allow for a greater range of depths in the lagoons, leading to more available active storage volume per acre (i.e., the volume of lagoons actually used for equalization of flows and not the required for operation of the aeration system). At present the original design minimum depth in the lagoons is 5 feet to accommodate the aeration system, with the diffusers suspended 4 feet below the floating aeration laterals and 1 foot above the lagoon bottom. WWTP staff indicate that the depths in the lagoons are occasionally brought down to 30 to 36 inches, and the aeration system remains functional.

Discussions with the aeration system manufacturer, Environmental Dynamics Incorporated, and oxygen transfer efficiency calculations confirmed a minimum depth of only 2.5 feet is needed for the system to operate. The concept for a revised aeration diffuser arrangement is shown on **Figure 2**. Reducing the depth between the diffusers and floating aeration lateral will nearly double the effective volume per acre available for flow equalization. Currently, approximately 60 percent of the total lagoon volume cannot be used for equalization (when considering a 5-foot minimum depth). The proposed modifications to the aeration system will reduce the unusable storage volume to approximately 30 percent of the total volume.

The two key variables in the lagoon sizing analysis are the return rates from the lagoons and the WWTP peak hydraulic capacity. The current average return pumping rate from the lagoons is 100 gpm, and an increased value of 200 gpm was considered in this analysis. The current operating maximum plant throughput is 0.95 mgd and the design hydraulic capacity of 1.18 mgd was evaluated. Return flow from the lagoons to the WWTP was assumed to begin when adequate hydraulic capacity is available and after a 3-day precipitation depth of 0.5 inches or less. Generally the return flow begins when the influent flow plus the return flow (100 or 200 gpm) is approximately 0.2 mgd less than the assumed plant capacity, or when the influent flow falls below 0.6 mgd, whichever is greater.

Seven scenarios were modeled for both current and buildout conditions. Each of these scenarios determined the minimum storage volume needed for capturing the 100th percentile of influent peak flow (i.e., worst-case) while accounting for minimum and maximum lagoon depths of 2.5 and 8 feet, respectively. The first four scenarios varied the lagoon storage and return flow threshold to find the minimum lagoon storage requirement given the current 0.95 mgd peak flow and the design capacity of 1.18 mgd. Return flow rates were varied between 100 gpm and 200 gpm to understand the relationship between return flow rate and the minimum lagoon storage requirements. The four minimum lagoon storage requirement scenarios are listed below.

Scenario 1. Peak plant capacity set to 0.95 mgd with return flow of 100 gpm

Scenario 2. Peak plant capacity set to 0.95 mgd with return flow of 200 gpm

Scenario 3. Peak plant capacity set to 1.18 mgd with return flow of 100 gpm

Scenario 4. Peak plant capacity set to 1.18 mgd with return flow of 200 gpm

Table 1 summarizes the minimum lagoon storage footprint and volume for Scenarios 1 through 4 for both the current and increased flow conditions. For these scenarios the WWTP peak capacity and lagoon return rate are inputs, and the lagoon sizes and volumes are calculated based on the historic flow observed at Front Street.

Table 1 – Summary of Lagoon Size Needed given WWTP Capacity and Lagoon Return Rate

Scenario	Current Conditions				Buildout Conditions			
	Inputs		Outputs		Inputs		Outputs	
	Peak WWTP Capacity (mgd)	Lagoon Return Rate (gpm)	Lagoon Size (acres)	Lagoon Volume (MG)	Peak WWTP Capacity (mgd)	Lagoon Return Rate (gpm)	Lagoon Size (acres)	Lagoon Volume (MG)
1	0.95	100	13.7	35.5	0.95	100	n/a	n/a
2	0.95	200	9.1	23.7	0.95	200	n/a	n/a
3	1.18	100	4.6	11.9	1.18	100	13.8	35.9
4	1.18	200	4.1	10.6	1.18	200	9.6	24.9

The results show that the required storage volume decreases fairly significantly with increased plant capacity. Increased plant capacity means that diversion of influent to the lagoons would occur less frequently. A more modest decrease in required lagoon volume occurs when the lagoon return rate is increased. Increasing the return flow rate means that additional storage capacity is made available earlier to accommodate additional inflow.

The increased flow scenario results indicate that Scenarios 1 and 2, which represent the plant's current peak operating capacity of 0.95 mgd, would require more equalization volume than can reasonably be supplied within the WWTP's footprint. In order to reliably accommodate the projected increase in flows, the WWTP would need to increase the current maximum throughput of approximately 0.95 mgd.

Three additional scenarios were run to evaluate the plant capacity required for lagoon footprints of 5 acres, 10 acres, and 20 acres, assuming that the lagoons remain 8 feet deep with a 2.5-foot-minimum depth for the aeration system. These scenarios indicate the optimum plant conditions needed to reliably use each lagoon footprint for flow equalization. Each scenario assumes that the return flow pumps will be upgraded to achieve 200 gpm to minimize the lagoon volume required for any given plant capacity. Scenarios 5, 6, and 7 are described below.

Scenario 5. Peak plant capacity required for lagoon size of 5 acres, 200 gpm return flow

Scenario 6. Peak plant capacity required for lagoon size of 10 acres, 200 gpm return flow

Scenario 7. Peak plant capacity required for lagoon size of 20 acres, 200 gpm return flow

Table 2 summarizes the key inputs and modeled results for each of these scenarios. For these scenarios the lagoon volume and capacity are treated as targets, and are set to be as close as possible to the 5-acre, 10-acre, and 20-acre target areas. The output from these three scenarios is the WWTP capacity.

Table 2 – Summary WWTP Capacity Modeling given Assumed Lagoon Size and Return Rate

Scenario	Current Conditions				Buildout Conditions			
	Output	Inputs			Output	Inputs		
	Peak WWTP Capacity (mgd)	Lagoon Return Rate (gpm)	Lagoon Size (acres)	Lagoon Volume (MG)	Peak WWTP Capacity (mgd)	Lagoon Return Rate (gpm)	Lagoon Size (acres)	Lagoon Volume (MG)
5	1.12	200	5.0	13.1	1.3	200	5.0	13.11
6	0.96	200	10.0	25.8	1.15	200	10.0	25.8
7	0.95	200	20.0	51.4	1.15	200	20.0	51.4

The results of Scenarios 5-7 indicate the relationship between WWTP peak capacity and lagoon size. Between 5 acres and 10 acres, the required WWTP capacity decreases substantially for both the current and buildout conditions. The minimal impact of increasing from 10 to 20 acres seen in Scenarios 6 and 7 is due to the increased effects of precipitation over the larger lagoon areas. The 20-acre lagoon scenario shows virtually no change in the required WWTP capacity relative to the 10-acre lagoon scenario. This suggests that there is a diminishing rate of returns with the lagoon size requirements, where at a certain point the contributions of precipitation during wet weather negates the benefits of additional storage.

It should be noted that the majority of the scenarios and conditions for return from the lagoons are generally more aggressive than the present operating approach. The flexibility afforded by the existing storage volume has allowed WWTP staff to operate in a fairly conservative manner.

Most of the modeled scenarios consider returning flow from the lagoons at a higher rate and all consider returning flow earlier than is presently done. It is also important to note that based on input from the Town, the approach taken was to maintain complete treatment for all modeled demands (i.e., the 100th percentile of “worst case” conditions modeled), and thereby meet permit limits for all events. There may be an opportunity during design to potentially reduce the total area of the lagoons to be lined, and thereby decrease cost, by increasing the throughput of the WWTP during very rare storm conditions, resulting in reduced effluent quality until flow subsides. This approach requires an evaluation and decision balancing the degree of cost savings vs. the increased risk of permit excursions.

The increase in WWTP capacity from 0.95 mgd to 1.18 mgd, or points in between, presents some significant savings in terms of lagoon storage required. The reason that the SBRs have been historically capped at 0.95 mgd is that there is minimal tolerance in the existing SBR control system as the flow begins to approach about 1 mgd. The SBR tanks have a high water level (HWL) setting, and if the water surface elevation exceeds the HWL, the system effectively shuts down. There is no reason that this control sensitivity cannot be addressed and modified to allow the SBRs to increase the treatment rate up to 1.18 mgd. An increase to a capacity of 1.3 mgd, as required under Scenario 5 with buildout, is also feasible during high flows by reducing SBR cycle time, discussed below.

It is possible – and very simple – to modify SBR operation for short-term periods during high-flow periods to process more flow. The SBRs currently run on 4.8-hour cycles. Automatic programming can be initiated that accelerates the cycle time as needed to pass a higher flow rate. This is a common feature of SBRs, and a standard approach by the SBR supplier is to initiate a 3.6-hour cycle when needed. This represents a cycle time that is 75% of the normal cycle time, that would increase the treatment rate by 4.8/3.6, or = 1.33 times the current peak rating, or 1.18 mgd x 1.33, 1.57 mgd. Improvements at other locations throughout the plant, discussed later in this memorandum, would also need to be made to allow for increased flow rates through the WWTP.

A different approach to increasing the volume available for flow equalization and decreasing the acres of lagoons needed is to raise the berms. This would increase maximum depth of the lagoons, reducing the total area required to be lined. Raising the maximum water surface in the lagoons would affect hydraulics in the splitter box, lagoon feed, and lagoon interconnections (should they remain), though the changes would be manageable. The cost tradeoff between earthwork required to raise the berms and reduced lining is negligible. Raising the maximum depths in the lagoons was not evaluated in detail as part of this evaluation, but warrants further investigation during the preliminary design phase of the project.

The balance of increasing WWTP capacity, treatment level of high flows, lagoon size, and lagoon return rates is a function of costs and comfort with operating the system as laid out in the various scenarios. Lagoon lining costs for the various scenarios, as well as various WWTP capital improvements and operations & maintenance (O&M) costs were developed to help with the evaluation and are presented later in this memorandum.

Nutrient Removal

Total Nitrogen

The draft permit includes an average monthly TN concentration limit of 3 mg/L, seasonally from April through October. This is a very tight limit; 3 mg/L TN is often referred to as the “limit of technology” with regards to the highest level of treatment that can be accomplished at publically-owned treatment works employing commercially available and feasible technology. The primary reason that this level of performance is so challenging is that in order to achieve this level of treatment on a consistent, permit-compliant (i.e., average monthly) basis, it is necessary to:

- Completely nitrify the influent ammonia-nitrogen (NH3-N) to less than 1 mg/L;
- Very efficiently denitrify the resulting nitrate-nitrogen (NO3-N) to nitrogen gas (again targeting less than 1 mg/L NO3-N); and
- Not have more than about 1 mg/L of refractory dissolved organic nitrogen (rDON) in the effluent.

Compliance with an average monthly limit of 3 mg/L TN is also made difficult because absolute optimum treatment on any given day may approach 2 mg/L – so if at any time during the month the facility has a “bad day” (if an effluent of [say] 5 mg/L TN can be considered bad), it is virtually impossible to reduce the average monthly level to achieve compliance. Note that the last parameter listed – rDON – is not within the plant’s control, but is instead a characteristic of the wastewater, and is “untreatable”. rDON will pass through biological treatment and filtration, regardless of how robust the design is.

Two alternatives were evaluated to enable the Marion WWTP to comply with the proposed TN limit: 1) modifications/optimizations of the existing process; and 2) addition of a new, tertiary nitrogen removal process. A third alternative – the possible use of wetlands treatment – is also discussed below.

Alternative 1 – Modifications/Optimization of Existing SBR Process

The Marion WWTP uses two SBRs to provide biological treatment. The SBR process is a variant of the activated sludge process, where various environmental conditions (e.g., anoxic, aerobic, or quiescent) are provided in a timed sequence within one tank, rather than in a physical sequence (e.g., anoxic tanks, aerobic tanks, and clarifiers). The existing SBRs were originally designed to include sufficient aerobic time (and therefore capacity) to completely nitrify the influent wastewater year round, and to provide a moderate level of denitrification as well. The cycle design includes anoxic time, and was originally targeted to achieve an effluent nitrate-nitrogen concentration in the 5-8 mg/L range, such that the plant’s effluent TN concentration would be in the 7–10 mg/L range.

The plant has historically out-performed this original design target for TN; in particular, the plant has performed remarkably well recently, averaging less than 4 mg/L in effluent TN on a long-term basis. The reasons that the plant effluent TN has typically been less than the original design range are:

- The influent TN concentration and load have been less than anticipated in the design; while the performance of SBRs, like other biological processes, is not constrained by removal percentage, it is also true that if influent concentrations are lower, it is easier to achieve lower effluent concentrations.

- The beneficial impact that the lagoons have provided; the lagoons, among other benefits, enable the plant operators to adjust flow and load to the plant to reduce swings in influent conditions, and provide more stable conditions than otherwise would be experienced, and the more consistent influent quality is, the better performance that can be expected from biological processes.

- The skill and attention of the plant operating staff. Marion's operators have a demonstrated dedication and knowledge level that serves to optimize the treatment that has been provided.

Because SBRs offer a tremendous amount of process flexibility, it is reasonable to evaluate whether the existing process could be modified to achieve compliance with the proposed average monthly TN limit of 3 mg/L. If it is possible to adjust SBR cycle times (for example, provide more anoxic time, or add intermittent periods of anoxic/aerobic time) to achieve this limit, this would be by far the least costly, lowest impact solution to achieving compliance with the proposed draft permit limit. Two other CDM Smith SBR designs (Northbridge, MA and Southern Regional Tertiary Treatment Plant, Camp Pendleton, CA) have both successfully adjusted cycle times to improve effluent TN in response to new permit limits (though, it is critical to note, the permit limits of these plants are higher than the 3 mg/L average monthly limit that Marion may be required to meet).

Though this option of adjusting cycle times is very appealing and seemingly "close" (the plant's effluent TN has not been much higher than 3 mg/L on a long-term average), as a technology SBRs have not been shown capable of consistently achieving less than 3 mg/L *on an average monthly basis*. To be clear, SBRs (including the SBRs at Marion) have shown the ability to achieve 3 mg/L at times, and in fact fairly regularly; however, reliable, consistent compliance with an average monthly TN permit limit has not been demonstrated for this technology.

The above does not mean that the SBRs cannot meet this level of treatment; just that permit compliance cannot be assured, and would always be very challenging. (Note – the process selected must be able to consistently and reliably meet permit requirements or the Town could be subjected to fines and penalties from the regulatory agencies) To provide the best chance of compliance, the following improvements would be recommended:

- Modification of the cycle phases in the SBR. The primary tool used to estimate optimal cycle times would be dynamic plant-simulation modeling, which can be used to predict impacts of modified cycles while considering the wastewater temperature, dissolved oxygen concentration, mixed liquor suspended solids levels and other factors; however, the basis of these cycle modifications would eventually consist of full-scale, real-time adjustment trials, achieved by modifying process controls in the field and monitoring results.

- Installation of on-line process analyzers for ammonia-nitrogen and nitrate-nitrogen in each SBR. These instruments provide real-time feedback regarding the impacts of process modifications and would allow operators to adjust cycles in response to the data. It is also possible to automatically program cycle time adjustments based on feedback from these analyzers.

- Construction of a supplemental carbon storage and feed system to provide sufficient carbon to drive denitrification to a very low level of effluent nitrate-N. Carbon addition would be timed to coincide with secondary anoxic periods in the SBRs.

With the above improvements, the Marion WWTP could theoretically comply with an average monthly TN limit of 3 mg/L; however, it would be very difficult due to the innate variability of biological process, the practical limits of the technology, and the mathematical reality that one relatively poor sample would make compliance extremely challenging. If the final NPDES permit holds firm to this limit, with no forgiveness for occasional exceedances, then relying on the SBRs alone to meet this permit condition would be very risky.

Alternative 2 - New Tertiary Nitrogen Removal Process

The average monthly 3 mg/L TN limit could be reliably achieved by adding a tertiary nitrogen-removal process to the Marion WWTP's process train. These tertiary processes are referred to as Biologically Active Filters (BAFs) and can be used to provide a very high level of nutrient and solids removal to polish treated effluent prior to discharge or reuse. As treatment requirements become more and more stringent, including TN of 3 mg/L, TP down to 0.1 mg/L, effluent total suspended solids (TSS) to 1 to 2 mg/L or less, and very low turbidity, BAFs provide a reliable and small-footprint way of achieving compliance. Many plants, including the nearby Wareham, MA and Scituate, MA facilities, employ BAFs for denitrification.

BAFs can be used to accomplish multiple treatment goals, including removal of BOD, nitrification, and/or denitrification. At Marion, BAFs (in this case, also known as denitrification filters) would be used to provide denitrification polishing, removing any nitrate still remaining in the SBR effluent. The BAFs would be installed downstream of the SBRs, and upstream of the effluent cloth disk filters. BAFs achieve denitrification by establishing a biofilm on the filter's granular media. A source of external carbon (e.g., methanol) is added to maintain the proper carbon-nitrogen ratio for healthy biofilm growth in an anoxic environment.

Denitrification BAFs are sized based on flow and nitrate load. **Table 3** summarizes preliminary design criteria for a BAF installation at the Marion WWTP.

Table 3 – Design Criteria, BAF

Item	Value
Flow Average annual Maximum month Peak day	 0.588 mgd 0.77 mgd 1.2 mgd
Nitrate load (assuming maximum 8 mg/L NO3-N) Average annual Maximum month	 39 lbs/day 51 lbs/day
Filter Bays No. Surface area (each; total)	 4 (3 active, 1 standby) 100 sf; 300 sf
Hydraulic loading rate Average annual Maximum month	 1.4 gpm/sf 1.8 gpm/sf
Nitrate loading rate Average annual Maximum month	 0.13 lb/d/sf 0.18 lb/d/sf
Estimated backwash rate	Approx. 2 percent of forward flow
Estimated methanol usage (based on 8 mg/L influent NO_3-N)[1] - Average annual - Maximum month	 19 gpd 25 gpd

[1]8 mg/L is a conservative value; SBR effluent is typically less than 8 mg/L nitrate.

At the Marion WWTP, the discharge from the post-SBR equalization tank would be redirected to the new BAF facility. The BAF would be located adjacent to and south of the existing filter building as shown on **Figure 3**. The BAF facility would consist of four filter bays (including one standby) and an adjacent small building to house blowers and controls. Discharge from the BAFs would flow by gravity through the existing cloth disk filters then on to UV disinfection prior to discharge. Backwash would be re-treated through the WWTP by sending it to either the lagoons or the headworks. Operating a BAF to achieve low levels of effluent TN requires addition of a supplemental carbon source. To date, methanol is by-far the most commonly used carbon source for denitrification BAFs; however, alternatively, less hazardous carbon sources are available and should be considered during the preliminary design stage of this process should it be selected.

Alternative Approach – Wetlands Treatment
The Marion WWTP facilities are conducive to consideration of wetlands treatment as a low-impact nitrogen removal alternative to the advanced process-mechanical technologies described in Alternatives 1 and 2. Specifically, the presence of large lagoons of the appropriate depth and resulting detention time to support treatment by constructed wetlands makes the option potentially feasible.

This memorandum previously discussed feasible and appropriate options for reducing the amount of lagoon storage volume currently used for influent equalization. Scenario 6 in Table 2 indicates that using half of the existing lagoon volume for influent storage (equivalent to the two smaller lagoons) would be feasible with only moderate modifications to the main treatment plant. This would make the large lagoon available for treatment with constructed wetlands.

The wetlands would function as a nitrogen-removal process, taking SBR effluent of similar quality to the plant's historical performance. Experience has shown that constructed wetlands are typically very effective at removal of nitrate-nitrogen, down to a level that could comply with an effluent TN concentration limit of 3 mg/L. Wetlands treatment effluent would flow back through the plant's effluent filters and UV disinfection. As described, the constructed wetlands would serve the same process function as the BAFs described in Alternative 2, above – but would be a low-technology, relatively passive, and more sustainable alternative.

Despite these potential advantages, this option has not been developed in more detail in this memorandum because 1) it has not been demonstrated that the wetlands treatment can effectively meet the potential TN limit at during the colder 'shoulder' months of the April-October permit limit due to the cold water temperatures that can be expected in spring (existing facilities are predominantly in warmer climates), and 2) it is not known whether this option would be considered as an approvable alternative by the regulating community (e.g., a significant question affecting cost viability is whether the constructed wetland would be required to be lined). In order to advance this potential option, the concept should be discussed with the regulators to assess its feasibility and a pilot system would need to be operated to confirm treatment performance.

Total Phosphorus

The draft permit includes an average monthly TP concentration limit of 0.2 mg/L, seasonally, from April through October and a less stringent average monthly limit of 1.0 mg/L from November through March. Both the 0.2 mg/L TP mg/L limit, effective through the warmer months, and the less stringent colder-month 1.0 mg/L will require modifications to the plant processes to achieve compliance.

Unlike nitrogen, there is no gaseous phase of P that can be achieved as a way of reducing effluent P. Influent wastewater contains both particulate and soluble phosphorus. Removal of phosphorus requires that much of the soluble influent P (how much depends on the P limit) be converted to particulate form and then that the particulate P be removed from the plant with the waste sludge. There are two general means for accomplishing this conversion: enhanced biological phosphorus removal (EBPR) and chemical phosphorus removal (CPR).

EBPR is a proven, though sometimes challenging, biological process that can be incorporated into activated sludge processes (such as SBRs). In general, the biological process must include an anaerobic zone or time phase. The combination of anaerobic conditions, coupled with subsequent aerobic conditions, is conducive to growth of a group of microorganisms (phosphorus-accumulating organisms, or PAOs) that can store excess P in their cells, more than is strictly needed for cell growth. When the PAOs are wasted as part of the WAS, EBPR is achieved.

Implementing EBPR in a SBR is possible, though it may prove to be very challenging. It would require setting cycle times to achieve aerobic, anoxic, and anaerobic phases, all balanced appropriately to meet all process goals. In addition and importantly, EBPR alone cannot reliably achieve effluent TP as low as 0.2 mg/L except in rare circumstances.

CPR is much simpler to operate and is very reliable. With the addition of metal salts (e.g., ferric chloride or alum), P in the soluble form of orthophosphate can be converted into particulate form and then these chemical solids can be removed with the WAS. The dose of coagulants can be set to target a certain P removal. While CPR is easy and reliable, it does have certain disadvantages, including the need to purchase, store, and handle the chemical, and the generation of chemical solids. These chemical solids are not biodegradable and would require modification of the Town's current operations of using the lagoon system to naturally degrade the WAS.

To achieve effluent TP in the 0.2 mg/L or less range, it is also necessary to have very low effluent TSS. Fortunately, the Marion WWTP is equipped with cloth disk filters. This type of filter has demonstrated successful compliance at other plants in Massachusetts with 0.2 mg/L TP limits, when preceded upstream by sufficient chemical addition. It is important to note that a jar testing protocol conducted at the Marion WWTP in 2007 did NOT provide confirmation that CPR followed by cloth filters could result in less than 0.2 mg/L TP. It is likely, however, that this result was caused by erroneous or insufficient testing or sampling; and prior to preliminary design of improvements to meet the draft permit's proposed P limit, another jar testing program should be conducted to 1) verify that there is nothing peculiar to Marion's wastewater that somehow interferes with the chemical precipitation of orthophosphorus; and 2) to determine optimal dosage needed.

Given the above, it is recommended that the Marion WWTP employ CPR if/when the draft TP limits are in effect. The process reliability and simplicity are significant advantages to EBPR process at the Marion WWTP, given the size of the plant and the process in place (SBRs). The main building of the Town's WWTP was originally designed with the anticipation that CPR could be needed at some point during the plant's service life. The effluent filters are an appropriate technology, although with this permit limit, it is recommended that the filters be upgraded (see subsequent discussion below).

It is important to note that the proposed TP limit will require that the Marion WWTP alter the plant's current approach with regard to solids handling. Currently, the plant removes biological WAS from the SBRs and sends these solids to the lagoons for further biological breakdown. Chemical solids will not break down – so with the implementation of CPR, solids would accumulate in the lagoons (if their use is continued as part of the solids train) or they would have to be handled and removed from the site by other means. This is described in more detail later in this memorandum.

Copper

The draft permit includes a very stringent copper concentration limits of 3.73 µg/L average monthly and 5.78 µg/L maximum daily. Publically owned treatment works' best approach to control of effluent copper levels is to focus on source control, as the capital and O&M costs of implementing a unit process to comply with such a tight copper limit at a POTW is ordinarily not considered feasible. Typically copper enters the collection system by either leaching from the water distribution piping system or industrial contributors. Copper can also be introduced to the plant by the inadvertent addition of impurities in chemicals often used for treatment. It can also be found in source water – copper is present in stormwater runoff and use of algicide could result in a temporary increase.

If required, one way to accomplish copper removal by way of upgrading the treatment plant is to add a tertiary chemical treatment and solids removal process downstream of the existing unit processes. Lime or another alkaline chemical would be added to raise pH to very high levels (above 8.5), which would result in precipitation of soluble copper (though compliance with the draft copper limits would still be in doubt) and generate chemical sludge. Removal of the precipitate, typically in a clarifier or filter, would be required. Further chemical addition might then be required to lower the pH prior to discharge to comply with the permitted range. This level of treatment is highly irregular at wastewater facilities and as such is not recommended.

A relatively new approach -- using designer metal sulfide coagulants as manufactured by Jenfitch, LLC or equal to precipitate soluble copper out of wastewater -- has shown success at a handful of facilities, with effluent copper concentrations under 5 µg/L. At Marion the metal sulfides would be added to the SBRs during aeration, with copper-laden sludge being removed from the SBRs. Though this is feasible, from a cost and operational complexity perspective, this process is not sufficiently established at this time to ensure compliance with the proposed monthly average limit of 3.73 µg/L. It would be necessary to pilot this approach to determine how effective the process would be in precipitating soluble copper at the Marion WWTP and whether that would result in compliance with the copper limit. As with CPR, this process would generate non-degradable sludge that would require an alternative means of solids handling than by lagoon treatment.

Copper is best addressed by continued source control, solids management, and continued monitoring and reporting as required by the current Administrative Order and CDM Smith recommends continuation of those practices.

Lagoon Improvements

Regardless of the final size of the lagoons implemented in the future, the existing sludge that has collected in the lagoons to date would need to be removed to comply with the draft permit. CDM Smith has performed a volume and condition assessment of the sludge and prepared a cost estimate for the removal. This information is documented in a Technical Memorandum "Marion Wastewater Treatment Plant Lagoon Sludge Disposal Alternatives Evaluation" dated March 1, 2016, attached as **Appendix A**.

After the accumulated sludge is removed from the lagoons, the desired lagoon area can be lined to allow for their continued use in accordance with the draft permit requirements. A cross section of possible liner configuration is shown on Figure 2. An 18-inch-thick sand layer would be placed on top of a 40-mil high density polyethylene (HDPE) liner to protect the liner from heavy machinery during future sludge removal efforts. The purpose of the sand layer is to allow for future lagoon cleaning operations without damaging the liner. The liner would sit on a 6-inch-thick dense sand bedding, prepared after sludge removal. The liner would be welded at its seams. The top of the sand elevation would match the original design elevation of the lagoon floor such that there would be no loss in volume. The liner would be installed along the lagoon berms to an elevation above the anticipated maximum water surface elevation. Alternatively, the liner could be installed at the bottom of the lagoons without a protective layer of land. This approach would require increased manual labor and the use of water cannons during future sludge removal operations, as heavy machinery cannot be used on the exposed liner. The optimal liner section for the lagoons would be further evaluated during design.

A number of the existing valves that control distribution and isolation to and between the lagoons are no longer operational. As part of the lagoon improvements all valves still necessary for the modified lagoon system should be replaced. As their depth is modified, the aeration diffusers should be inspected, relocated and replaced if necessary, as they are in the latter stages of their anticipated service life.

Depending on the chosen lagoon size and improvements, the lagoon recycle pumps may need to be replaced to increase the peak rates. Further investigation is needed to determine if the suction and discharge lines associated with these pumps would need replacement as well.

There are several potential uses for the existing lagoons that are not lined for flow equalization and possibly sludge treatment, and eventually decommissioned, including green space, conversion to constructed wetlands and alternative energy harvesting (e.g., photovoltaic files or wind turbines). The best long-term use of these areas requires further consideration. For the purposes of this evaluation, it was assumed that the berms of decommissioned lagoons (where they do not abut lagoons remaining in service) would be eliminated, with materials removed from the berms added to the decommissioned lagoon floor to create a level surface. Note that the constructed wetlands option could include no or partial flattening of the berms. The cost for removing the berms of decommissioned lagoons is included in the costs presented for the various lagoon scenarios.

Additional Improvements

The Marion WWTP upgrade was completed in 2005, and it is anticipated that any improvements recommended herein will likely be under construction no sooner than the 2018–2020 time frame.

There are several items that should be addressed as capital improvements or repair/replacement as part of the pending capital project to extend the design life of the facility. These are described below.

Disk Filter Upgrades

The effluent disk filters were placed in service in 1999, prior to the SBR upgrade, and should be improved and their service life extended, as part of the pending improvements. The two filter basins are each equipped with two disk filters. The basins and installed equipment were originally laid out to allow for the easy addition of two new disks per filter basin, or four total. These disks should be installed to improve redundancy of the equipment. In addition, it is suggested that new 5-micron pile cloth filter media be installed on the disks, and that the filter control systems and auxiliary equipment be updated to suit. These improvements are low-cost, and will improve the process' flexibility and reliability in providing compliance with the pending TP limits.

Along with additional disks, improvements in the disk filter building should include stairs and a platform around the disk filter bays to allow for better maintenance access, and an alarm connection to the SCADA system should be installed. At present, disk filter system alarms are only local to the building.

Ultraviolet Disinfection System Upgrades

The plant is equipped with two UV channels, with UV equipment installed in one. The typical redundancy included with UV systems is not in place at the Marion WWTP because the lagoons have provided a means to stop discharge of effluent as needed should the UV system be out of service. With the operation and/or sizing of the lagoons changing, and the adjusted SBR cycle time flexibility to be added, it is recommended that the second UV channel be equipped with new equipment similar to the UV system installed. These improvements will improve the process' flexibility and reliability in providing compliance with the plant's disinfection limits. In addition to new controls for the new UV equipment, existing UV controls are outdated, becoming increasingly unreliable and require replacement. The new controls should have an alarm connection to the SCADA system. At present, UV system alarms are only local to the building.

Gravity Thickener

The potential implementation of CPR to help meet TP limits would produce non-biodegradable chemical sludge. This chemical sludge would not be amenable to further treatment and breakdown in the lagoons and would accumulate relatively quickly. Even without chemical sludge, a means of sludge of thickening and storage at the WWTP should be considered for future operation.

The simplest approach for sludge handling and storage at the WWTP would be to construct and operate a gravity thickener. Other thickening equipment, such as a rotary drum thickeners or gravity belt thickeners, require more intensive O&M attention, possibly unthickened sludge equalization storage, and separate thickened sludge storage facilities. A properly-sized gravity thickener can provide both thickening and storage.

Preliminary sizing of a gravity thickener for the Marion WWTP indicates that a 36-foot-diameter thickener with a 10-foot side water depth and a 4:1 horizontal to vertical sloped floor would suffice. Sizing considered average annual day sludge production, including chemical sludge, under anticipated increased flow and load conditions. The thickener would provide storage for up to 15 days of sludge (at average annual daily production).

Thickened sludge would require regular hauling by a third party (e.g., Synegro Technologies). Thickened sludge transfer pumps would be required to fill the trucks. For planning purposes it was assumed that the pumps would be placed in a vault adjacent to the gravity thickener, with the pumps below the elevation of the bottom of thickener to avoid needing a suction lift. The vault would include a small building to allow for sheltered access. The anticipated gravity thickener and pump vault location are shown on **Figure 3**.

The gravity thickener would be located next to the existing biofilter. Odor from the thickener will need to be addressed and the biofilter modified to suit. Supernatant from the gravity thickener could be sent to the headworks or the lagoons for later treatment.

Third Sequencing Batch Reactor

One of the operational issues faced at the WWTP is that taking one of the two SBRs offline for maintenance presents a significant disruption to plant operations and capacity. The plant has the ability to run one SBR unit during operating hours, but cannot run this automatically, so the majority of the time in this circumstance, all influent flow is typically diverted to the lagoons. In the past this condition has persisted for 3 to 4 weeks during unscheduled maintenance.

Considering the potential for reduced volume available in the lagoons, a third SBR would provide an important benefit in terms of process redundancy, flexibility and reliability. The intent would not be to run three SBRs concurrently, but to have a two duty, one standby arrangement. The third SBR would be the same size as the existing SBRs and would be located adjacent to SBR No.2, as shown on **Figure 3.**

Additional Metering

The addition of magnetic flow meters to the influent lines to the lagoons from the lagoon splitter box would allow for improved monitoring of diverted flow. Relocating the existing flow meter on the WAS pump discharge line to a point downstream of the confluence with the scum pump discharge (or addition of a new meter) would allow for separate monitoring of scum being pumped from the SBRs. The existing effluent meter is upstream of the disk filters and does not account for filter backwashing, likely resulting in flow readings 3 to 5% higher than the actual effluent from the WWTP. The meter should be relocated downstream of the filter building to provide more accurate readings of plant effluent flow.

In addition to new and relocated magnetic flow meters, pressure transducers should be installed in the lagoons to provide accurate, remote reading of the depths. At present, staff read static gauges.

Miscellaneous Improvements

There are a number of smaller improvements that would increase the reliability and efficiency of operations at the WWTP. Such improvements include upgrades to the overflow weir motor controls, soda ash piping, floor drains in the garage, and the plant-wide SCADA system. Items of this nature are not typically broken out individually at this level of study, but should be evaluated in greater detail during preliminary design. An allowance is included in the costs as part of this evaluation.

Classification of Wastewater Treatment Facility

The Marion WWTP is presently classified as a Class 5 (C) facility. Implementation of the improvements discussed in this memorandum would most likely result in the WWTP becoming a Class 6 (C) facility. If the plant classification is increased, certain plant staff (i.e., chief operator and assistant chief operator) would need to increase their operator's licenses.

Costs

Opinions of probable cost were developed for each of the improvements identified in this memorandum. These costs include escalation to the anticipated midpoint of construction, assumed to be in 2020 in accordance with the anticipated permitting period, at a 3% annual inflation rate. Capital costs also include the following allowances: 25% for engineering, permitting, and implementation; 20% for construction contingencies for lagoon lining; 30% construction contingencies for other WWTP improvements; and 10% for project contingencies as is appropriated at this level of project development. These contingency percentages are appropriate to use at this early stage of cost estimating, for the purpose of developing planning-level costs. O&M costs (increases) were developed for those improvements requiring modifications from existing operation. An electrical rate of $0.17 per kilowatt-hour was used. Due to the number of improvements, two additional WWTP staff members at a total cost of $100,000 per year each have been included in the O&M costs. The O&M costs were escalated at a 3% annual inflation rate to the anticipated completion of construction in 2020.

The costs developed for each of the lagoon lining scenarios includes removal and hauling of existing accumulated sludge in all three lagoons, maintenance of plant operations during the lining process, lining of the specified acreage, replacement of existing piping, valves and catwalks, and modifications to the existing aeration systems. The acreage to be lined under the various scenarios was taken from the increased flow conditions evaluation, which is why Scenarios 1 and 2 were not included. Scenario 7 was also not included as it presented no advantage to Scenario 6. Although Scenarios 3 and 4 require lining 13.8 and 9.6 acres, respectively, costs were developed considering lining 15 and 10 acres to match existing footprints as it is more cost effective than constructing new berms to meet the modeled sizes. The increase in sizes of the lagoons associated with these Scenarios reduces the required increase to plant capacity. The costs do not reflect any reduced storage demands resulting from reducing SBR cycle times during high flow events, except for Scenario 5, which requires a WWTP peak hydraulic capacity of 1.3 mgd.

The capital and O&M costs are presented in **Table 4**. The net present value (NPV) of each alternative is presented. A detailed development of costs can be found in **Appendix B.**

Table 4 – Summary of Project Costs

Alternative		Capital Costs (2019 - midpoint of construction)	O&M Costs (2020)	Total NPV (2016)
Lagoon Lining				
Scenario 3	Line 15.0 acres Keep existing lagoon return pumps Increase WWTP capacity to 1.15 mgd	$4,600,000	-	$4,200,000
Scenario 4	Line 10.0 acres Replace existing lagoon return pumps Increase WWTP capacity to 1.15 mgd	$3,700,000	$800	$3,400,000
Scenario 5	Line 5.0 acres Replace existing lagoon return pumps Increase WWTP capacity to 1.3 mgd	$2,600,000	$800	$2,400,000
Scenario 6	Line 10.0 acres Replace existing lagoon return pumps Increase WWTP capacity to 1.15 mgd	$3,700,000	$800	$3,400,000
Other Improvements				
SBR Modifications (controls, carbon addition)		$ 300,000	$ 9,000	$400,000
Biologically Active Filters		$ 4,100,000	$ 13,200	$4,000,000
Chemical Phosphorus Reduction		$ 300,000	$ 16,000	$500,000
Disk Filter Upgrades		$ 200,000	$ 3,400	$200,000
Ultraviolet Disinfection System Upgrades		$ 500,000	-	$400,000
Gravity Thickener		$ 1,300,000	$ 126,000	$3,400,000
Third Sequencing Batch Reactor		$ 3,600,000	-	$3,300,000
Additional Metering		$ 200,000	-	$180,000
Miscellaneous Improvements		$ 980,000	-	$890,000
Additional WWTP Staff		-	$ 225,000	$3,900,000
Other Improvements Subtotal				*$ 17,200,000*
Total – Line 10 Acres + Other Improvements				**$20,600,000**

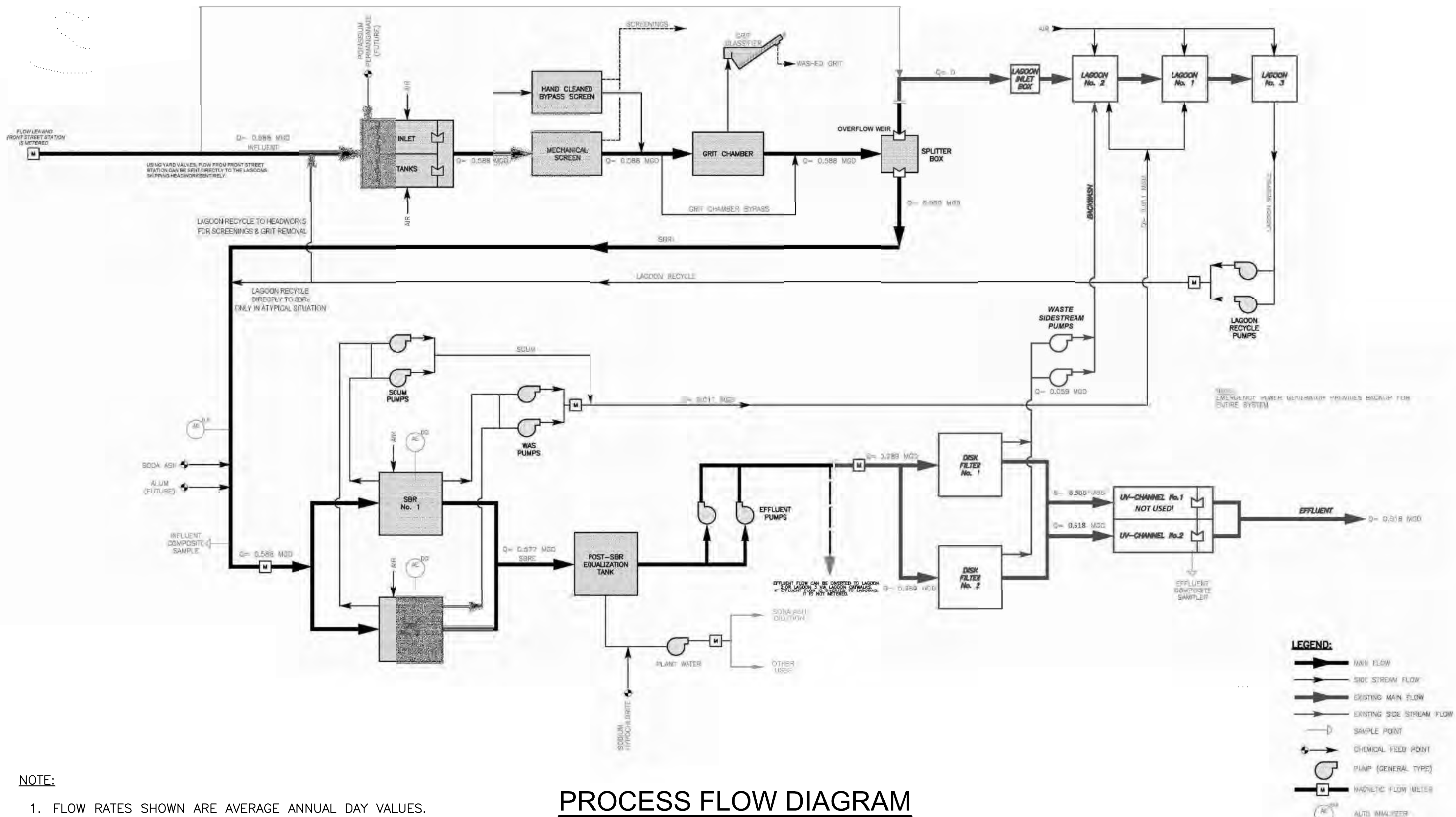
As shown, the lagoon lining ranges from $2.4 million to $4.2 million. The BAF, gravity thickner, and additional SBR represent the highest capital cost items. The highest increases for O&M include sludge hauling, carried under the gravity thickener costs, and additional personnel.

Considering lagoon lining of 10 acres (Scenarios 4 and 6, $3.4 million), the total net present value for improvements at the plant is approximately $20.6 million.

Next Steps

Several of the improvements presented in this memorandum, those aimed at meeting Draft NPDES Permit limits need to be considered against alternative compliance strategy of extending the WWTP's outfall and the USEPA's feedback on draft permit comments including those related to the compliance schedule associated with the improvements. The extent of the lining of the lagoons would need to be coordinated with potential improvements to both the WWTP and the outfall, as a number of combinations of options are possible.

As indicated earlier, if constructed wetlands treatment is an option that the regulators endorse as a feasible alternative, then the Town should consider further, more detailed evaluation and subsequent development of a piloting program to determine whether the option can achieve reliable compliance with the anticipated 3 mg/L TN limit. Such piloting would by necessity be long-term, at least two growing seasons.



NOTE:

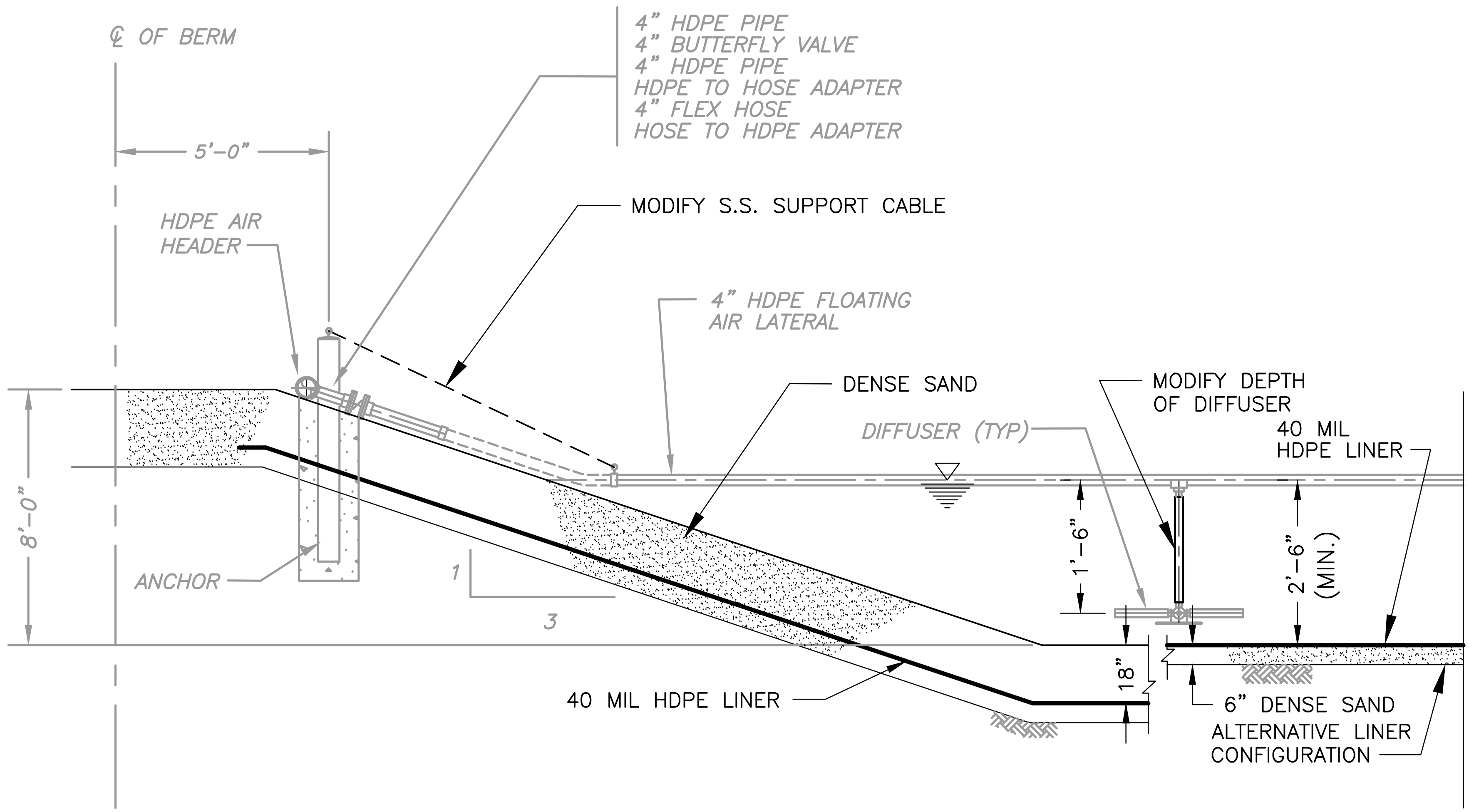
1. FLOW RATES SHOWN ARE AVERAGE ANNUAL DAY VALUES.

PROCESS FLOW DIAGRAM

NOT TO SCALE

CDM Smith

Figure No. 1
TOWN OF MARION, MASSACHUSETTS
WASTEWATER TREATMENT PLANT UPGRADE
APRIL 2016

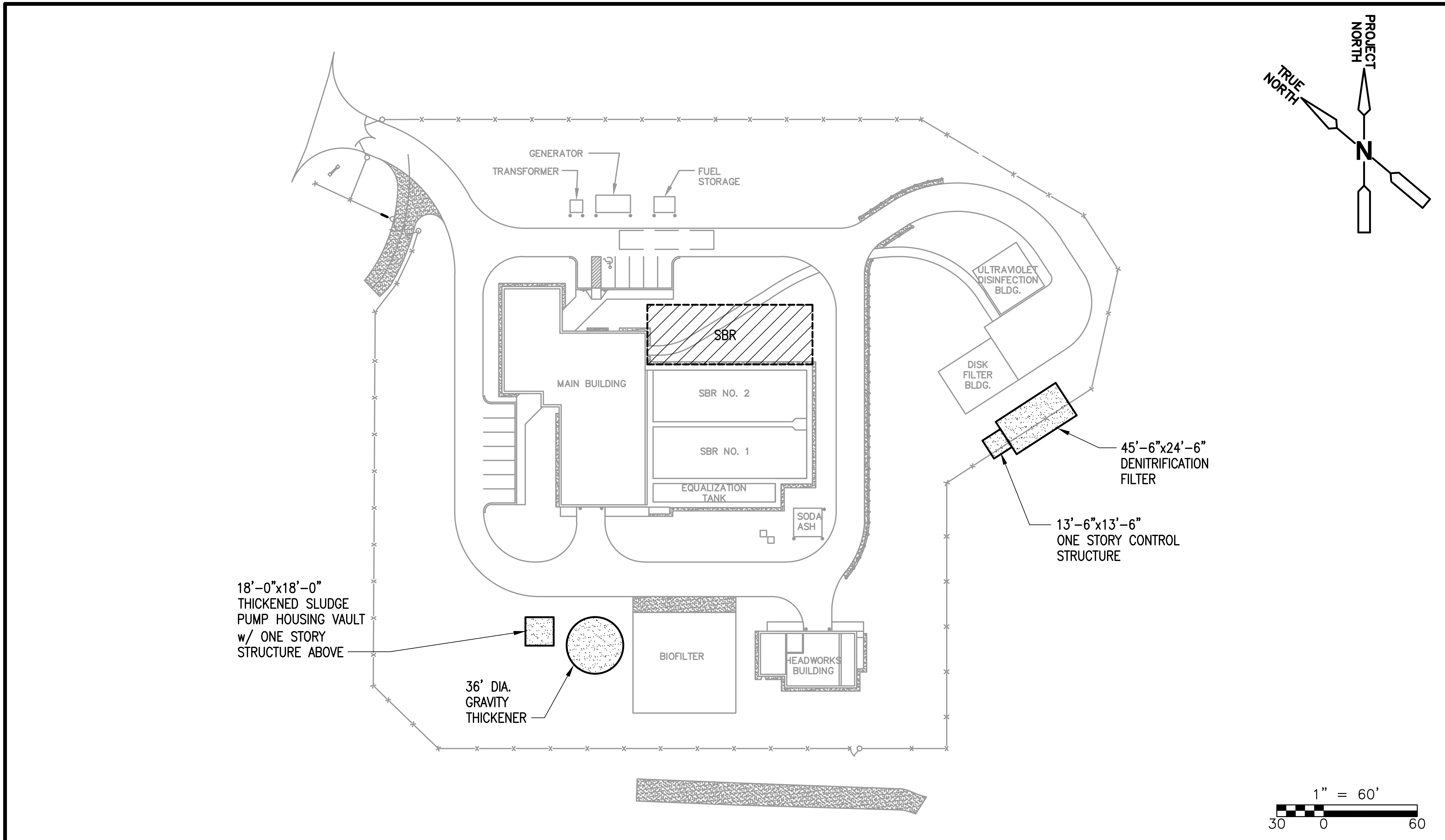


TYPICAL LAGOON SECTION AT LATERAL

NOT TO SCALE

CDM Smith

Figure No. 2
TOWN OF MARION, MASSACHUSETTS
WASTEWATER TREATMENT PLANT UPGRADE
APRIL 2016



CDM Smith

Figure No. 3
TOWN OF MARION, MASSACHUSETTS
WASTEWATER TREATMENT PLANT UPGRADE
APRIL 2016

Appendix A

Marion Wastewater Treatment Plant Lagoon Sludge Disposal Alternatives Evaluation

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CDM Smith

Memorandum

To: *Shawn Syde*

From: *Jack Hoar*

Date: *March 1, 2016*

Subject: *Marion WWTP Lagoon Sludge Disposal Alternatives Evaluation*

Introduction

The Marion Wastewater Treatment Plant is located at 50 Benson Brook Road in Marion, Massachusetts and provides wastewater treatment for an average daily flow of 1.18 million gallons per day (mgd) of municipal wastewater from the Town of Marion. The wastewater treatment plant includes three wastewater lagoons that have been in operation since the early 1970s. The lagoons are aerated, unlined and cover approximately 20 acres. The lagoons receive inflow from the following:

- Untreated wastewater influent in excess of the plant's average daily capacity of 1.18 mgd;
- Waste activated sludge from the plant sequencing batch reactors (SBRs);
- Backwash water from the plant disk filters

Sludge is not removed from the lagoons and degrades naturally.

As a result of concerns regarding leakage of untreated wastewater from the unlined lagoons and potential impacts to groundwater - removal and disposal of the sludge in the lagoons is being evaluated to allow for upgrades, including lining of the lagoons with an impermeable liner.

The volume of sludge in the lagoons was measured by collection of approximately 180 sludge depth samples (50 in Lagoon 1, 50 in Lagoon 2, and 80 in Lagoon 3) and two sludge samples from each lagoon sludge samples were collected for disposal characterization analyses at Alpha Laboratories in Westborough, MA. The sludge samples were collected by EST Associates, Inc. from Needham, MA in October 2015 and the depths are summarized below:

- Lagoon 1 sludge depth ranged from 6 inches to 42 inches
- Lagoon 2 sludge depth ranged from 2 inches to 19 inches
- Lagoon 3 sludge depth ranged from 2 inches to 11 inches

Each of the two sludge samples from each lagoon were analyzed for the following parameters:

- Volatile Organic Compounds (VOCs) by MCP 5035/8260C
- Semivolatile Organic Compounds (SVOCs) by MCP 8270D
- Total Solids by SM 2540
- Polychlorinated Biphenyls (PCBs) by MCP 8082A
- Total MCP 14 Metals by MCP 6010C/7470A
- Hazardous Waste Characterization

Toxicity Characteristic Leaching Procedure (TCLP) where total metals results indicated a potential for TCLP based on the 20X rule.

- Synthetic Precipitation Leaching Procedure (SPLP)
- TCLP and SPLP extracts were analyzed for mercury and selenium (Lagoon 1);
- Mercury and lead (Lagoon 2)

Lagoon Sludge Analyses Summary Discussion:

Metals

In general the metals concentrations results were similar in all three lagoons with results somewhat lower for several metals (e.g., mercury) in Lagoon 3.

All 14 of the metals tested for each of the two sludge samples collected from each lagoon were generally low and should not limit sludge disposal options.

The concentrations for the eight metals for which standards exist were all below the Ceiling Concentration Limits for All Biosolids Applied to Land.

TCLP analyses of sludge sample from Lagoon 1 for mercury and selenium indicated concentrations well below the threshold for classification as a characteristic hazardous waste.

TCLP analyses of sludge sample from Lagoon 2 for mercury and lead indicated concentrations well below the threshold for classification as a characteristic hazardous waste.

Volatile Organic Compounds

VOCs were generally reported as undetected at method detection limits for all six sludge samples from the three lagoons, with a few low concentrations of VOCs reported for samples from Lagoons 1 and 2. The few VOCs detected (e.g. acetone, methyl ethyl ketone, and benzene compounds) are typical solvent or petroleum related compounds.

The VOCs detected should not restrict sludge disposal alternatives.

Semivolatile Organic Compounds

No SVOCs were detected in any sludge sample at concentrations above method detection limits.

Polychlorinated Biphenyls

No PCBs were detected in any sludge sample at concentrations above method detection limits.

Total Solids

- The total solids concentration in the two samples collected from Lagoon 1 ranged from 4.05% to 4.12%.

- The total solids concentration in the two samples collected from Lagoon 2 ranged from 5.81% to 7.15%.

- The total solids concentration in the two samples collected from Lagoon 3 ranged from 8.68% to 9.17%.

Hazardous Waste Characterization

The pH of the sludge samples collected from all three lagoons was generally neutral, ranging from 7.0 to 7.4 indicating it is not a hazardous waste based on the characteristic of corrosivity.

The United States Environmental Protection Agency currently does not have an approved test method to determine the characteristic of reactivity. Determination of the characteristic of reactivity must be based on generator knowledge. A reactive hazardous waste is identified in 40 CFR 261.23 as a solid waste that exhibits any of eight characteristics including:

1. Normally unstable;

2. Reacts violently with water;

3. Forms potential explosive mixtures with water;

4. Generates toxic gases when mixed with water;

5. Is a cyanide or sulfide bearing waste that can generate toxic gases;

6. Is capable of detonation or explosion if subjected to a strong initiating source;

7. Is readily capable of detonation or explosive decomposition at standard temperature and pressure; or

8. Is a forbidden explosive as defined by 49 CFR 173.54.

The lagoon sludge does not meet any of these characteristics and is therefore not a hazardous waste based on the characteristic of **reactivity**.

The analyses of the six sludge samples indicated they were all not ignitable indicating that the sludge is not a hazardous waste based on the characteristic of **ignitability**.

As noted above, based on the TCLP testing performed the sludge is not a hazardous waste based on the characteristic of **toxicity**.

The results of the lagoon sludge analyses are summarized in the attached tables.

Sludge Disposal Alternatives Discussion

Sludge Volume

Based on the sludge depth evaluation completed by EST Associates and the area of each lagoon the following sludge quantities were calculated by CDM Smith for each Lagoon (see attached Cut/Fill Report 11/18/15):

Sludge Quantities		
Lagoon 1:	8527.1	CY
Lagoon 2:	5857.04	CY
Lagoon 3:	3600.3	CY
Total:	17,984.44	CY

The sludge % solids concentration ranged from 4.05% to 9.17% with an average concentration of 6.5% and a weighted average concentration of 5.84% based on % solids by lagoon.

One option for sludge disposal includes pumping the sludge from the lagoons and transport by tanker truck to a disposal facility such a wastewater treatment plant that could process and dispose of the sludge along with its own sludge. The estimated volume of approximately 18,000 CY would amount to:

18,000 CY x 202 gallons/CY = 3,636,000 gallons

Discussion with Jim Beard from J. P. Noonan Transportation Inc. from West Bridgewater, MA, whose services include sludge hauling, indicated that a vacuum truck with a 9,000 gallon capacity could be used to remove the sludge from the lagoons, provided the solids content remained below 6%. Under this scenario it is likely that the liquid in each lagoon would be decanted/removed and discharged back to the plant. The sludge would then be vacuumed into the tanker truck. This process would typically include a jetter to liquefy the sludge sufficiently to facilitate the vacuuming process. The jetter could be expected to add approximately 15 gallons per minute of water which over the course of the removal process would amount to an additional 720,000 gallons of liquid to be removed from the lagoons (assuming sludge is removed over 10 hours per day over a 3 month period). Adding the water volume from the jetter of 720,000 gallons to the estimated sludge volume of 3,636,000 gallons results in a total volume of sludge of 4,356,000 gallons. Assuming a 9,000 gallon tanker truck amounts to 484 truck trips. Assuming 6 truck trips per day results in a schedule of approximately 80 working days. The sludge hauling contractor indicated that transportation costs for a distance of approximately 80 miles would amount to approximately $500 per 9,000 gallon truckload. Assuming approximately 500 truckloads would be required results in a transportation cost of $250,000.

Sludge removal, transport & disposal costs based on J. P. Noonan Inc. information:

Review of a sludge disposal marketing study prepared by CDM Smith in 2012 indicates that transportation and disposal (T&D) costs per dry ton of sludge ranged from $309/dry ton to $769/dry ton with an average T&D cost of $430/dry ton. (The marketing study summary is attached for reference.)

Dry tons calculation:
4,356,000 gallons x 5% solids x 0.0000417 = 908 dry tons

Assume approximately 1000 dry tons

Sludge Removal, Transport and Disposal Costs - J.P. Noonan Inc.		
Transportation Costs at $500/truckload	$250,000	
Disposal Costs at off-site WWTP	$300,000	Allowance
Operator and equipment for vacuum/jetter	$100,000	Allowance
Water for jetter	$15,000	Allowance
Sludge removal, transport and disposal		
Subtotal	$665,000	

Sludge removal, transport & disposal costs based on CDM Smith 2012 Transport and Disposal Survey:

Sludge Removal, Transport and Disposal Costs - CDM Smith		
Total Transport and Disposal costs based on 1000 dry tons and $430/dry ton	$430,000	
Operator and equipment for vacuum/jetter	$100,000	Allowance
Water for jetter (720,000 gallons)	$15,000	Allowance
Sludge removal, transport and disposal	$15,000	Allowance
Subtotal	$545,000	

A second option for sludge disposal would include hiring a sludge dewatering contractor to reduce the volume of sludge by reducing the water content. Assuming the approximately 6% solids content could be increased to 20% by dewatering equipment such as a filter press or centrifuge would result in a volume reduction of 18,000 CY to 5,400 CY.

V1 = 100-Pm2 = 100-80 = 20

V2 = 100-Pm1 = 100-94 = 6

V1 (Initial Volume) = 18,000 CY

V2 (Final Volume) = ?

Pm1 (Initial Moisture Content) = 6%

Pm2 (Final Moisture Content) = 20%

6 x 18,000 CY = 20V2

V2 = 5,400 CY

Assuming a unit weight of 1.6 tons/CY results in a tonnage of sludge cake requiring disposal of 1.6 tons/CY x 5,400 CY = 8,640 tons. Disposal costs for this material could amount to $950,400- based on a T&D unit cost estimate of $110/tons received from Waste Management Inc.

A call was made to Senesac Inc. from Milton, VT., which provides water and wastewater services, including dewatering equipment and operators for sludge dewatering projects. The Mr. Justin Senesac reported a rough unit price of approximately $700/dry ton for providing on-site centrifuge dewatering equipment and operating personnel along with transportation and disposal of the sludge cake. Mr. Senesac estimated the dewatering equipment would produce a sludge cake with a % solids concentration between 20% and 25%. Mr. Senesac reported that their company had two sets of equipment – one that could process sludge at 100 gallons/minute (or 1,000 pounds/hour) and a larger unit capable of a processing rate of 500 gallons/minute (or 4,000 pounds per hour). Applying the sludge processing rate of 100 gallons/minute to the estimated sludge volume of 3,636,000 gallons results in an estimated time period for processing the sludge of approximately 60 to 80 days, depending on the hours of operation of either 10 or 8 hours per day.

Sludge removal, transport & disposal costs based on Senesac Inc. information:

Assuming that we have approximately 1000 dry tons of sludge to remove at $700/dry ton results in a sludge removal, transport and disposal subtotal cost of **$700,000.**

These cost are rough estimates for the sludge removal, transportation and disposal component of the lagoon cleanout work. They do not include additional costs including mobilization, demobilization, permits, fees, health and safety requirements, submittals, site preparation, utilities, insurance, contractor's overhead and profit, other costs and contingencies. In addition, following removal of the sludge that can be effectively vacuumed from the lagoons, the task will remain to clean up the residual sludge and impacted soil that remains over the 20 acre lagoon area as required to prepare for lining for the new lagoons or restoration for the future use of this area.

Assuming that the entire 20 acre area was excavated to a depth of 8 inches (assuming a four thickness of sludge remained after vacuuming and a four inch thickness of underlying soil) would result in a volume of soil/sludge mixture of:

20 acres x 43,560 square feet (SF)/acre x 0.667 ft. = 580,800 cubic feet (CF)/27 CF/cubic yard (CY) = 21,511 x 1.6 tons/CY = **34,418 tons**

Transportation and disposal of this sludge/soil mixture to an out-of-state landfill such as the Waste Management Turnkey disposal facility in Rochester, NH at an estimated unit cost of $110/ton would add an additional cost of approximately $3,786,000, not including the costs of excavation and loading this material for transport.

Sludge Disposal Vendor Survey

Telephone calls were made to the following waste disposal companies to inquire about costs for transportation and disposal of the lagoon sludge:

Company	Contact	Telephone/email
Waste Management	Jason Barroso	jbarroso@wm.com
Veolia	Bill Johnson	(860) 230 3801
Casella	Pat Owens	(603) 661 3820

Jason Barroso reported that the sludge cake could be disposed of at the Waste Management Turnkey Landfill in Rochester, NH - provided that dewatering resulted in no free liquids in the sludge. The estimated cost for transportation from Marion, MA to Rochester, NH was $40/ton and the disposal cost was $70/ton for a combined Transportation and Disposal (T&D) Cost of $110/ton.

Discussion with the waste disposal company representatives indicated that there are options for removal and disposal of the lagoon waste sludge and the representatives discussed disposal options available to their companies based on the specific details of any request for proposal for removal of the sludge. One important factor discussed was the schedule requirements for removal of the sludge. The vendors indicated that if the schedule was flexible and included an extended period of time (e.g., over a period of X years) to remove the sludge it opened up more options for better pricing at different disposal locations. The option of beneficial reuse was discussed. This option would require more time and more work with state regulators and more testing to confirm the material met applicable regulations for recycling.

Landfilling on site in possible portions of lagoons to be closed is not permissible.

Conclusions:

Based upon the information available from the previous CDM Smith marketing survey and discussion with local vendors – removal and off-site disposal of the Marion WWTP lagoon sludge can be accomplished under two general scenarios. One option would include setup of dewatering equipment on-site to increase the solids content of the sludge sufficient for acceptance at an off-site disposal or recycling facility. Disposal would likely involve landfilling at an out-of-state landfill like the Turnkey Facility in Rochester, NH. Beneficial reuse of the dewatered sludge cake may also be an option, although it appears that this would require more time and effort working with regulators, completing required sampling and analyses and obtaining necessary permits. A second option involves removing, transporting and disposing of the sludge, without dewatering, at a wastewater treatment plant. The sludge would be processed along with the wastewater treatment plant's own sludge and disposed of in a similar manner (e.g. incineration or landfill).

The sludge sampling and analyses completed to-date indicates the sludge would not require management as a hazardous waste. It appears that different sludge disposal contractors have options for management and disposal of the sludge and their means and methods would be decided based on the requirements for disposal included in the bid documents and other factors (e.g., schedule for sludge removal; space limitations for storage on-site; WWTP availability and capacity for sludge processing; etc.).

The estimated costs for removal, transport and disposal of the estimated 18,000 CY of sludge would be in the range of $550,000 to $950,000, however there would be significant additional costs for general conditions and removal of residual sludge and restoration/preparation of the 20 acres of lagoons for any proposed future uses. If the residual sludge and underlying soil required removal and off-site disposal at an out-of-state landfill – the T&D costs alone could be close to $4,000,000.

Marion WWTP

Lagoon Sludge Sampling Results-Table 1

Lagoon No.	Samples Collected		Sample Date	Analyses	Results
1	P1-01	L1526333	10/15/2015	Full Suite	
		L1529164	11/9/2015	TCLP with NO2/NO3 and TKN	
			11/9/2015	SPLP with NO2/NO3 and TKN	
	P1-02	L1526333	10/15/2015	Full Suite	
		L1529164		TCLP with NO2/NO3 and TKN	
				SPLP with NO2/NO3 and TKN	
		L1526607-01		TCLP for mercury and selenium	
2	P2-01	L1526757	10/22/2015	Full Suite	
	P2-02		10/22/2015	Full Suite	
3	P3-01	L1527685	10/28/2015	Full Suite	
	P3-02	L1528095	10/30/2015	Full Suite	
		L1529162	11/6/2015	TCLP with NO2/NO3 and TKN	
				SPLP with NO2/NO3 and TKN	

Notes:

1. Two sludge samples were collected from each of the three lagoons for a total of six sludge samples.
2. The percent solids for each of the sludge samples ranged from 4% to 9%
3. Each sludge sample was analyzed for the following parameters:

Volatile organic compounds (VOCs) by EPA Method 5035/8260C
Semivolatile organic compounds (SVOCs) by EPA Method 8270D
Polychlorinated biphenyls (PCBs) by EPA Method 8082A
Total MCP 14 Metals by MCP 6010C/7470A
Total Solids by SM 2540
Hazardous Waste Characterization
Toxicity Characteristic Leaching Procedure (TCLP) if the total metals concentration indicated a potential TCLP failure based on the 20X rule. Based on the 20X rule -TCLP analyses was performed for mercury and selenium on sample P1-02; for lead and mercury on sample P2-01; and for mercury on sample P2-02 - No TCLP failures were reported for any of these samples.

4.The estimated volume of sludge in each lagoon is:

Lagoon #1 8527 CY
Lagoon #2 5857 CY
Lagoon #3 3600 CY
Total: 17,984 CY

Cut/Fill Report

Generated: 2015-11-18 10:40:29
By user: coroajj
Drawing: E:\Documents\My Projects\MA, Marion\Sludge\E:\Documents\My Projects\MA, Marion\Sludge\GIS Export.DWG

Volume Summary

Name	Type	Cut Factor	Fill Factor	2d Area (Sq. Ft.)	Cut (Cu. Yd.)	Fill (Cu. Yd.)	Net (Cu. Yd.)
Pond 1 - Volume	full	1.000	1.000	213569.45	0.00	8527.10	8527.10<Fill>
Pond 2 - Volume	full	1.000	1.000	218338.62	0.00	5857.04	5857.04<Fill>
Pond 3 - Volume	full	1.000	1.000	433086.90	0.00	3600.30	3600.30<Fill>

Totals

	2d Area (Sq. Ft.)	Cut (Cu. Yd.)	Fill (Cu. Yd.)	Net (Cu. Yd.)
Total	864994.97	0.00	17984.44	17984.44<Fill>

* Value adjusted by cut or fill factor other than 1.0

Table 3: Market Disposal Price Evaluation

	Type of Sludge	Disposal ($/dry ton)	Transportation ($/dry ton)	Distance to Disposal vs. Hartford (mi)	Notes
Amherst, MA (Casella price option to Fitchburg, MA)	Thickened (4 to 8%)	$291	$124	50 vs. 55	Transport is based on 6% at $0.0311/gal. July 2009 - July 2011
Amherst, MA (Casella price option to Upper Blackstone)	Thickened (4 to 8%)	$296	$124	56 vs. 55	Transport is based on 6% at $0.0311/gal. July 2009 - July 2011
Amherst, MA (Casella price option to Naugatuck)	Thickened (4 to 8%)	$260	$243	90 vs. 55	Transport is based on 6% at $0.0609/gal. July 2009 - July 2011
Ansonia (hauled to Waterbury operated by Synagro)	Thickened (3.1% average; 2.1 to 4% range)	$769/dry ton for transport and disposal		16 vs. 43	Based on 3.1% ($0.0994/gal). Plus fuel surcharge of $270/month.
Branford (hauled to New Haven operated by Synagro)	Thickened (5 to 6%)	$383/dry ton for transport and disposal		10 vs. 43	Based on 5.5% ($0.0879/gal). CPI annual adjustment
Bridgeport (hauled to New Haven)	Thickened (4%)	~$400/dry ton for transport and disposal		21 vs. 55	Per NEWEA presentation, ~$2.3M/yr and 5800 dry tons/yr
Bristol (hauled to Naugatuck)	Cake (average 24%)	$285/dry ton for transport and disposal		21 vs. 18	Based on 24% solids. ($68.33/wet ton)
Cheshire (hauled to Waterbury operated by Synagro)	Cake (16%)	$372	self- hauls	11 vs. 26	Separate communication quoted $338.45 plus transport

Table 3: Market Disposal Price Evaluation (Continued)

	Type of Sludge	Disposal ($/dry ton)	Transportation ($/dry ton)	Distance to Disposal vs. Hartford (mi)	Notes
Groton (hauled to Naugatuck or Cranston, RI)	Thickened (6 to 7%)	$309/dry ton for transport and disposal		76 vs. 55	Based on 6.5% ($0.0838/gal).
Killingly (hauled to Woonsocket operated by Synagro)	Cake (23%)	$283	unknown	28 vs. 48	2011 conversation, not verified
Lowell, MA (hauled to Woonsocket operated by Synagro):	Cake (22 to 24%)	$349/ dry ton		60 vs. 110	2009 or 2010
Mansfield, MA (hauled to Woonsocket operated by Synagro)	Thickened	$280	$96	22 vs. 105	$0.02/gal (or $140.40 per 9000-gallon tankerload). Say 5% for transport calcs. 2009 or 2010
Mattabasset charges to receive outside sludge	Combined rate given, based on dry tons	$266	NA	NA	Conversation with Brian Armet, Nov 2010. Previous years' cost $338. Includes tip fee ($71, previous year $93)
Meriden (receiving location unknown)	unknown	$287.50	unknown	unknown	2011 conversation, not verified
Middletown (hauled to Mattabasset WPCF)	Thickened (3.5%)	$240	$107	3 vs. 14	Based on their 2010 total cost of $399k for transport and disposal of ~1150 tons
New Haven, CT (incinerates onsite)	Cake (26%) External ~5%	$425 / dry ton for disposal contract ops onsite		0 vs. 40	Pays Synagro per dry ton (no transport), gets money back for cost-sharing from receiving

Table 3: Market Disposal Price Evaluation (Continued)

	Type of Sludge	Disposal ($/dry ton)	Transportation ($/dry ton)	Distance to Disposal vs. Hartford (mi)	Notes
Northbridge, MA (hauled to Woonsocket)	Thickened (3%)	$321	$160	16 vs. 70	$0.02/gal (or $140.40 per 9000-gal tanker). Signed 2008 Synagro, annual CPI increment
Norwich (hauled to Waterbury)	Cake (average 19%)	$408/dry ton for transport and disposal		61 vs. 41	$77.47/wet ton. Alternate locations Woonsocket/New Haven
Plainville (hauled to Mattabasset WPCF)	Thickened (4 to 7%)	$385/dry ton for transport and disposal		15 vs. 15	Based on 5.5% ($0.0882/gal). Additional typical fuel surcharge of 12%, recently as high as 33%.
Springfield, MA (hauled to Rochester, NH landfill)	Cake (26%)	$320/dry ton for transport and disposal		160 vs. 27	Pays Waste Management $82.50 per wet ton
Stratford (hauled to Naugatuck WPCF)	Thickened (average 5%)	$498/dry ton for transport and disposal		24 vs. 52	Based on 5% ($0.1038/gal).
Torrington (hauled to Naugatuck WPCF)	Thickened (6 to 7%)	$314/dry ton for transport and disposal		25 vs. 38	Based on 6.5% ($0.0850/gal).
Wallingford (receiving location unknown)	Unknown	$288	unknown		2011 conversation, not verified
Waterbury (onsite incinerator)	Cake	$280	NA		Annual CPI. 1993 contract ops with Synagro (currently being renegotiated)
West Haven (onsite incinerator)	Cake (26%)	Included in operations budget		0 vs. 43	As of June 2011, municipal operations (for 15 yrs prior, contract ops)

Notes: Distances based on town to town, not specific addresses.

Appendix B

Development of Opinion of Probable Costs

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Marion, Mass - Lagoon/WWTP Improvements - O&M Costs

Alternative	Input shaft HP	kW input to motor	Operational Period (hr/yr)	kW-hr/yr	cost/yr	Annual Chemical Costs	Total
Scenario 3							
Scenario 4	10	8.182	480	3,927	$ 668		$ 668
Scenario 5	10	8.182	480	3,927	$ 668		$ 668
Scenario 6	10	8.182	480	3,927	$ 668		$ 668
SBR Modification							
Biologically Active Filters	from manufacturer:			21,783	$ 3,703	$ 8,000	$ 11,703
Chemical Phosphorus Reduction	1	0.818	8760	7,167	$ 1,218	$ 13,000	$ 14,218
Disk Filter Upgrades	3	2.454	7271	17,846	$ 3,034		$ 3,034
UV Disinfection							
Gravity Thickner	3	2.454	8760	40,646	$ 6,910		$ 6,910
Third SBR							
Additional Metering							
Miscellaneous Improvements							

pump efficiency	75%	
motor efficiency	94%	
VFD efficiency	97%	
electrical cost	0.17	$/kW-Hr

Marion, Mass - Lagoon/WWTP Improvements - Net Present Value

Alternative	present unloaded cap costs	escalated cap costs w/ contingencies (2019)	present annual O*M costs	escalated annual O&M costs (2020)	2016 NPV total	Notes
Scenario 3	$ 2,542,400	$ 4,583,946	$ -	$ -	$ 4,194,960	sludge removal, lining, leveling unused berms
Scenario 4	$ 2,031,600	$ 3,662,974	$ 668	$ 751	$ 3,365,226	sludge removal, lining, leveling unused berms, new pumps, add'l HP
Scenario 5	$ 1,450,800	$ 2,615,792	$ 668	$ 751	$ 2,406,906	sludge removal, lining, leveling unused berms, new pumps, add'l HP
Scenario 6	$ 2,031,600	$ 3,662,974	$ 668	$ 751	$ 3,365,226	sludge removal, lining, leveling unused berms, new pumps, add'l HP
SBR Modification/Carbon	$ 150,000	$ 292,987	$ 8,000	$ 9,004	$ 424,929	tank/totes, pumps, piping, chemical
Biologically Active Filters	$ 2,110,000	$ 4,121,356	$ 11,703	$ 13,172	$ 4,001,011	filter structure/media/underdrains, blowers, tanks, piping, chemical
Chemical Phosphorus Reduction	$ 150,000	$ 292,987	$ 14,218	$ 16,003	$ 546,812	tank, pumps, piping, chemical
Disk Filter Upgrades	$ 86,000	$ 167,979	$ 3,034	$ 3,415	$ 213,189	additional filters, stairs/platform, minor increase in operating HP
UV Disinfection	$ 250,000	$ 488,312	$ -	$ -	$ 446,875	redundant modules in existing channel, no increase in operations
Gravity Thickner	$ 690,000	$ 1,347,742	$ 111,910	$ 125,956	$ 3,426,858	structure, drive/mechanism, pumps, housing (vault), hauling costs
Third SBR	$ 1,850,000	$ 3,613,512	$ -	$ -	$ 3,306,875	structure, decanter, pumps, instrumentation, diffusers, piping
Additional Metering	$ 100,000	$ 195,325	$ -	$ -	$ 178,750	allowance
Miscellaneous Improvements	$ 500,000	$ 976,625	$ -	$ -	$ 893,750	allowance
Personnel			$ 200,000	$ 225,102	$ 3,920,088	allowance

Construction Contingency - Vertical	30%	
Construction Contingency - Lagoons	20%	
Engineering, Permitting, and Implementation	25%	
Project Contingency	10%	
Inflation Rate	3%	
Mid-point of Construction (2019)	3	years
Start O&M (2020)	4	years
O&M Evaluation Period Duration	30	years

Section 4

WWTP Outfall Alternatives, Analysis of Alternatives

Technical Memorandum

To: *Robert Zora, Superintendent of Public Works*

From: *Shawn Syde, P.E.*
Matthew Pitta, P.E.

Date: *April 7, 2016*

Project: *Town of Marion Wastewater Treatment Plant Outfall Alternatives*
Analysis of Alternatives

Background

On November 28, 2014 the United States Environmental Protection Agency (USEPA) issued a Draft National Pollution Discharge Elimination System (NPDES) Permit for the Town of Marion's (the Town) Wastewater Treatment Plant (WWTP) located off of Benson Brook Road. Three of the conditions outlined in the Draft NPDES Permit include reduction of total phosphorous (TP), and further reduction of total nitrogen (TN) and copper concentrations in the WWTP effluent. The Town submitted comments on the draft permit and is awaiting response from the USEPA.

As outlined in the Town's comments to USEPA on the Draft NPDES Permit (a letter report dated February 6, 2015 et al.), there may be other potentially cost-effective solutions to meet the requirements of the permit that are not included in the Draft NPDES Permit. In accordance with its responsibility to provide the sewer ratepayers and citizens of Marion with cost-effective wastewater services, the Town is exploring alternative pathways for complying with the Draft NPDES permit that involve changes to the current discharge point of the treated WWTP effluent.

CDM Smith Inc. (CDM Smith) has developed and evaluated several potential alternatives to the current location of the WWTP outfall, which currently discharges to the freshwater Effluent Brook off of Abel's Way downstream of Route 6. The two primary options for relocation of the outfall include:

- Extending the existing outfall pipe to discharge at the head of the saltmarsh that fronts Aucoot Cove. This option would potentially eliminate TP permit limits by bypassing the fresh waters of Effluent Brook.
- Extending the existing outfall pipe into Outer Aucoot Cove. This option would potentially eliminate TP and TN permit limits and reduce or eliminate copper limits by discharging to deep waters in Aucoot Cove. An amendment to the Ocean Sanctuaries Act passed in August 2014 makes the introduction of a new ocean discharge a viable option.

This memorandum provides the Town with additional information related to these potential alternative compliance pathways to meet the requirements of the draft permit by working with its existing treatment system and constructing modifications to the existing outfall. This memorandum does not provide a final recommended plan or selection of a particular route, as selecting the final location of the outfall is dependent on a number of factors including subsurface geotechnical conditions, cost, and regulatory considerations including the pre-design studies that would be required under the Ocean Sanctuaries Act, other permit requirements and the agreement on a compliance schedule. This memorandum is, however, the first step in determining if construction of such a project is feasible and whether it should be considered by the Town as a cost-effective approach to continue discussions with the regulatory agencies about how it could be included in the draft NPDES permit.

Existing Conditions

As shown on **Figure 1**, Effluent Brook begins south of the WWTP, collecting and conveying runoff from adjacent areas. The brook runs south, crossing under Mill Street (Route 6), Abel's Way and Olde Meadow Road, eventually discharging to Aucoot Cove. Additional discussion related to the tributary area to Effluent Brook can be found in the technical memorandum prepared by CDM Smith "Aucoot Cove Total Nitrogen Watershed Load Estimate" dated March 9, 2016.

Treated effluent from the WWTP is currently conveyed to the southeast by gravity via an 18-inch diameter high density polyethylene (HDPE) pipe that traverses approximately 3,300 feet to the former chlorination facility on Route 6. This portion of the outfall pipe was upsized to 18-inch in 2006 by pipe bursting the original 10- and 12-inch vitrified clay pipe, constructed circa 1920). From the former chlorination facility, the outfall continues another 1,400 feet further south as an 18-inch-diameter reinforced concrete (RC) pipe (constructed in the early 1970s); the pipe ends at the discharge to Effluent Brook which is located approximately 320 feet west of 39 Abel's Way.

Between the outfall discharge location and Aucoot Cove the brook passes through undeveloped strip of land in the Indian Cove section of Marion. The strip is bordered by wooded marsh along most of its length. The entire length of Effluent Brook to the head of the saltmarsh is thickly vegetated. The region between Route 6 and the head of the Aucoot Cove includes a number of environmental resource areas. These areas are bordering vegetated wetlands (BVW), including wooded marsh; salt marsh; and three areas regulated by the Natural Heritage and Endangered Species Program (NHESP): State listed estimated habitats of rare wildlife, priority habitat of rare species, and designated natural communities. Aucoot Cove itself also contains eelgrass beds. A complete list of environmental resource areas can be found in the Technical Memorandum "List of Potential Environmental Permitting Requirements – Effluent Discharge Pipe and Ocean Outfall, Marion, Massachusetts" by CDM Smith, dated April 7, 2016 ("the Permitting Memorandum"), which is attached to this memorandum as **Appendix A**. The wooded marsh, saltmarsh, NHESP Natural Communities, and eelgrass beds are shown on **Figure 1**. Additional environmental resource areas (i.e., the entire BVW, NHESP priority habitat of rare species and wildlife) can be seen on the figures in the Permitting Memorandum.

CDM Smith conducted a site visit along the route of the existing outfall and Effluent Brook between Route 6 and the saltmarsh to the south on November 19, 2014. Effluent Brook between Route 6 and the

BVW varies in size, averaging approximately 5 feet wide from bank to bank and 1-foot-deep from top of bank to channel invert, with the width and depth of flow varying greatly. The brook passes through a mixture of residential and lightly wooded areas, becoming less defined as it passes through the BVW and salt marsh. Photos from CDM Smith's site visit are included in **Appendix B**.

Development of Alternatives

Five different alternatives were developed for the relocation of the existing WWTP outfall. Each alternative is shown on **Figure 2**. One of the alternatives extends the existing outfall pipe to the head of the saltmarsh; while the four other alternatives extend the existing outfall pipe to Outer Aucoot Cove (i.e., a deep water outfall). The development of the alternatives and considerations for various implementation options, construction approaches and engineering challenges are discussed below.

Outfall Locations

Salt Marsh Discharge Outfall Location

An outfall located at the head of the saltmarsh in Aucoot Cove would likely eliminate the TP limit in the permit since the treated effluent would no longer discharge to Effluent Brook, a freshwater stream. An 18-inch-diameter, HDPE outfall terminus at the head of the salt marsh would consist of a new concrete headwall, riprap apron and an elastomeric duckbill check valve to prevent saline water and debris from entering the pipe; while the terminus would be placed inland of the high tide line, storm surge or extreme tides could reach its location

Outer Aucoot Cove Outfall Location

Several different locations were evaluated for the terminus a new deep water outfall, including Sippican Harbor and Aucoot Cove. At this early stage of analysis, criteria for siting the terminus were to be distant from the seaward edge of eelgrass, in deep water to improve initial dilution and closer to Buzzards Bay to improve subsequent mixing. Two deep water outfall locations in Outer Aucoot Cove were selected by studying bathymetric mapping and locating the deepest points. The locations in Outer Aucoot Cove shown in **Figure 2** correspond to a depth of approximately 17 feet below mean water surface elevation. Note, however, that an initial mixing analysis has not been performed and further study could indicate that a terminus at another, shallower location in Aucoot Cove would be acceptable. Sippican Harbor was not chosen as a discharge location since similar depths in the harbor are nearly twice as far from shore, and as an embayment is more enclosed than the open waters of Aucoot Cove.

As mentioned previously, the deep water ocean outfall option only became permissible in August 2014 when the Massachusetts Legislature passed an amendment to the Ocean Sanctuaries Act. Prior to this amendment, (non-vested) municipal wastewater discharges were prohibited in some ocean sanctuaries, while in others the applicant was required to demonstrate that there was no feasible alternative to ocean discharge. The 2014 amended Act allows new or modified discharges from municipal wastewater treatment plants to an ocean sanctuary, provided a series of conditions and requirements are met. These conditions and requirements specify receiving water studies be conducted; the Town has requested that the regulatory agencies provide further information on the specific of the studies that would be needed. A more detailed discussion of the amendment to the Ocean Sanctuaries Act can be found in the attached Permitting Memorandum.

A deep water outfall would consist of a pipe to the location of discharge and a multiport diffuser. The configuration of the diffuser would be determined by subsequent study but would either be a manifold pipe laid directly on the seabed (with appropriate anchoring and/or riprap for cover) or a buried manifold with riser extending above the seabed (with appropriate caps to protect the riser ports). Because Marion's discharge is intermittent due to the batch processing of wastewater, the ports would be fitted with Tideflex 'duckbill' check valves or equivalent to, prevent seawater from entering the pipe.

Peak Flow and Hydraulics

CDM Smith created hydraulic models for each of the alternatives to ensure flow could be conveyed in the various systems without surcharging resulting in unpermitted discharges. Manning's equation and the Hazen-Williams equation were used to calculate the headloss in gravity and force main (or gravity pipe under head) scenarios, respectively. A tailwater elevation of 15.0 (NAVD88), equivalent to the 100-year Federal Emergency Management Agency flood elevation and a peak flow of 1.18 million gallons per day (mgd), matching the WWTP's design peak operating capacity, were used in these calculations.

A minimum velocity of 1 foot per second (fps) at peak flows was used to determine pipe sizes. This relatively low velocity is allowable due to the high quality the WWTP effluent. In general, gravity lines were sized at 18 inches diameter, resulting in a full flow velocity of just above 1 fps. At peak flow conditions with lower tailwater elevation, the outfall pipe would experience open channel flow, with lower flow depths and velocities of approximately 2 fps in minimally sloped (0.0012 vertical to horizontal) reaches. Force mains and the final reaches of gravity lines discharging to deep water outfalls that would be constantly submerged were sized at 16 inches, resulting in a velocity of approximately 1.3 fps at peak flows. Pipes with 18-inch-diameters were considered for these reaches, but the various configurations were able to accommodate the headloss associated with the slightly higher velocity. Hydraulic calculations for each alternative are attached in **Appendix C**. Ground surface profiles used to aid the hydraulic calculations are also included in **Appendix C**.

Increased peak flows and tailwater conditions were also considered, although not directly used for the development of alternatives. In some cases, HDPE was chosen instead of RC to allow for future implementation of pump stations to counter higher tailwater and headloss. As discussed in the technical memorandum prepared by CDM Smith "Wastewater Treatment Plant Influent Equalization Lagoon Improvements Analysis of Alternatives" and dated April 4, 2016 (the "WWTP Improvements memorandum"), the WWTP could potentially increase peak capacity to 1.3 mgd. Additionally, to account for the effects of climate change, the highest National Oceanic and Atmospheric Administration prediction for relative sea level change at Woods Hole, 6.9 feet, were considered. Since the increased peak flow of 1.3 mgd represents a relatively minor increase, the increased tailwater of 21.9 feet and increased peak flow were considered in combination to determine the "worst case scenario" rather than examined individually. The impacts to each design considering these criteria are discussed below.

Installation Methods

Various methods of pipe installation, including open cut, trenchless and on-surface-conduit (direct lay) installation were considered for appropriate portions of each alternative route shown in **Figure 2**. Each method of installation has pros and cons related to cost, permitting and environmental impacts, constructability, and access, as discussed below.

Open Cut Installation

Open cut construction is the most typical pipe installation method. Trenches are excavated with machinery and excavation support systems and the pipe is installed on the bottom of the trench, typically on top of granular material bedding and the trench is then backfilled to ground surface. Open cut installation can be cost effective, but becomes more costly and increasingly difficult in wet soil, at significant depths, through bedrock, and through private property. Open cut construction is more invasive than trenchless methods as it requires direct access to the pipe location with large machinery. Open cut installation within water bodies is also possible, using specialized equipment (e.g., dredgers, barges) and/or divers.

Direct Lay Installation

Direct lay of conduit involves laying pipe on the ground or sea floor. The pipe is either weighed down, typically with concrete anchors, or heavy pipe is used, to prevent floatation and/or movement. On-the-surface conduit leaves the pipe exposed to a number of potential hazards. Direct lay on the sea floor was used in a number of alternatives. In these cases, the pipe would be covered with riprap to help prevent floatation, movement, and damage. Direct lay on the ground was not used in any of the alternatives.

Trenchless Construction Methods

Several methods of trenchless pipe installation were considered in this evaluation. It is important to note that the feasibility and relative advantages of each methodology are highly dependent on existing conditions. Since the potential routes being evaluated vary considerably, the installation methods were assessed as a single approach as well as in conjunction with other methods. Below is a description of the trenchless methods that could potentially be used for the installation of a new WWTP outfall pipe.

- **Horizontal Directional Drilling** - Horizontal directional drilling (HDD) is a multi-stage installation method producing an arched path that begins and ends at the ground surface. The stages of this trenchless installation method consist of drilling a pilot hole, enlarging the pilot hole to the required diameter (reaming), and finally, pulling the pipe back through the hole. This trenchless construction technology minimizes the impact of excavation and avoids potential conflicts through its ability to span long distances far below the ground surface.

  The pipe size, pipe material, and environmental constraints are factors in determining the geometry of the arched path. An assumed spring line depth of 40 to 50 feet below ground surface was used for this specific evaluation. Factors influencing the feasibility of HDD installation include surface and subsurface conditions, pipe size and length and the construction area that is available for the installation equipment. For this project it was assumed that a minimum of 0.25-acre laydown area would be needed for HDD installation. A single span limit of 2,500 linear feet was used for the evaluation of this installation option when considering strictly above ground applications (although longer lengths have been constructed).

HDD is commonly used to install pipes under roadways, private properties and environmentally sensitive areas. HDD is significantly more challenging and costly when the downstream end of the pipe is submerged (as would be the case for final reaches of alternatives with deep water outfalls); requiring barges and careful attention to prevent materials used in the process (i.e., bentonite) from entering the water. Alternatively, large cofferdams could be constructed and dewatered at the downstream end to create an "in-the-dry" environment. It is important to note that while HDD may allow for the bypassing of environmentally sensitive areas, access to both ends of the installation is necessary; meaning equipment may still have to be conveyed through these sensitive areas. HDD is a constantly evolving technology. More and more applications with submerged ends are being completed at longer lengths.

- **Microtunneling** - Microtunneling is a pipe jacking technology that uses a remote-controlled boring machine. This installation method was considered as a potential trenchless construction approach for the segment of outfall pipe extending out into Aucoot Cove and also for portions of the pipeline to be installed beneath roadways and environmentally sensitive areas. For this application, the most appropriate pipe diameter for microtunneling is 48 inches. This pipe diameter allows sufficient space for the entrance of a construction worker to access the microtunnel boring machine for routine inspection and maintenance. Since this diameter is much larger than required for the outfall, a smaller carrier pipe would be installed inside a 48-inch-diameter steel casing, which is a very common scenario when using this technology. This operation works much more efficiently when the annular void between the two pipes has adequate space for the slurry pipes to fill this void, which would be the case in this application.

  Microtunneling requires approximately 21-foot-diameter access shafts at either end of the installation as well as a staging area for construction equipment and materials. This area must be stable to bear the load of the heavy equipment. For a project of Marion's size, a minimum of one acre is usually required, of which at least 0.5 acre should be at the launch shaft site. Since this installation method would be used for the final segment of pipe extending into Aucoot Cove (i.e., underwater location), the site could potentially be very soft, requiring fill stone to be brought onto the site for construction traffic and significant restoration/landscaping required at the completion of the work. The maximum length of microtunneling is highly variable, dependent on subsurface rock and soil conditions. For this project it was assumed that a maximum installation length of 3,000 feet is feasible considering a 48-inch-diameter tunnel. This length would be achieved by using below ground intermediate jacking stations (IJS). An IJS basically allows the jacked pipe to advance in short increments, thus reducing the required jacking thrust to advance the pipe. At lengths greater than 3,000 feet, this system tends to become inefficient compared to using another launch shaft. Microtunneling becomes exponentially more expensive as the application lengths increase.

- **Other Trenchless Installation Methods** - Other trenchless installation methods considered include tunnel boring and jack and bore. Similar to microtunneling, tunnel boring is restricted to large diameters that would be oversized and significantly more costly for this particular application. Jack and bore is not feasible for applications below the groundwater table and therefore would not be a viable option.

HDD was used as the trenchless installation method for the purposes of the development of alternatives and associated costs, including for the final reaches of deep water outfalls. HDD was chosen as it is generally more cost effective than microtunneling. As noted, additional subsurface investigations are needed to confirm whether or not HDD (or microtunneling) are truly viable options for the various alternatives.

Alternatives Evaluation

Five primary alternatives were developed for further evaluation under this study:

- Alternative 1 – Extension of the existing 18-inch outfall pipe to Olde Meadow Road to reach the head of the salt marsh.
- Alternative 2 –Extension of the existing 18-inch outfall pipe to Olde Meadow Road to reach deep water in Outer Aucoot Cove.
- Alternative 3 – Relocating the existing outfall pipe starting at Route 6 and then along Route 6 and Converse Road to reach deep water in Outer Aucoot Cove.
- Alternative 4 – Extension of the existing 18-inch outfall pipe to Olde Logging Road, Olde Knoll Road, and Converse Road to reach deep water in Outer Aucoot Cove.
- Alternative 5 – Relocating the existing outfall pipe to Mattapoisett via Route 6, Indian Cove and Harbor Beach areas to reach deep water in Outer Aucoot Cove.

Under each primary alternative, various combinations of trenchless and open cut installation methods were considered, resulting in a series of sub-alternatives. These sub-alternatives are described below in greater detail. Other alternatives were also considered, but did not provide optimal solutions based on the needs, site constraints and construction methods available. **Figure 2** presents an overview of each of the five primary alternative routes described below.

Alternative 1 – Extension of Existing Pipe via Olde Meadow Road to Head of Salt Marsh

Alternative 1 consists of an extension of the existing discharge pipe to the head of the salt marsh, requiring approximately 5,200 linear feet of new pipe. The proposed path of the pipeline would begin at the current discharge location and continue south through a wooded area, adjacent to Effluent Brook, then along Olde Meadow Road approximately 2,300 feet. Beyond the southern end of Olde Meadow Road the pipe would continue another 1,400 linear feet to a discharge point at the head of the salt marsh at the northern end of Aucoot Cove. Olde Meadow Road is a private way and work there would require coordination with owners.

The pipe discharge would need to be fitted with an elastomeric flexible duckbill check valve to prevent tidal water and debris from entering the outfall. The valve would require routine inspection/maintenance to clear away debris built up on the exterior of the valve and to ensure proper operation.

Extending the outfall to the head of the salt marsh would eliminate TP limits, but would require improvements at the WWTP for TN removal. This location would not alter the proposed copper limits. As addressed in the WWTP Improvements Memorandum, copper removal is extremely difficult, and having a discharge location that does not reduce or eliminate the proposed copper limit is a notable disadvantage to this option.

Two sub-alternatives (Alternatives 1A and 1B) were developed along this route with varying lengths of RC and HDPE pipe and open cut vs. HDD installation. The primary difference between the sub-alternatives is the final reach, with Alternative 1A using open cut and Alternative 1B using HDD installation to mitigate impacts to the NHESP Natural Communities Area and wetlands upstream of the salt marsh. While HDD installation allows for limited disturbance to existing resource areas, direct access to the proposed outfall location would be necessary for completion of the HDD installation and construction of the headwall with riprap. Alternatives 1A and 1B are shown on **Figures 3 and 4**, respectively.

Under both alternatives, an 18-inch pipe would be installed for the entire length of the alignment. . During high flows and high tailwater associated with coastal storm flooding, a significant portion of the outfall extension would be under pressure. To prevent overflows from occurring under these conditions, solid HDPE pipe with fittings would be installed through the manholes in the affected reaches. These pipe reaches will not be open to the atmosphere at manholes; however, access to the pipe will be accommodated via tees with blind flanges located inside the manholes.

The following summarizes the materials/components associated with each of these alternatives.

Alternative 1A

- Open cut installation of approximately 2,300 linear feet of 18-inch RC pipe
- Open cut installation of approximately 2,600 linear feet of 18-inch HDPE pipe
- 9 precast concrete manholes (4-foot-diameter)
- Outlet Structure – Headwall with riprap and duckbill check valve

Alternative 1B

- Open cut installation of approximately 2,300 linear feet of 18-inch RC pipe
- Open cut installation of approximately 1,500 linear feet of 18-inch HDPE pipe
- 8 precast concrete manholes (4-foot-diameter)
- HDD installation of approximately 1,400 linear feet of 18-inch HDPE pipe

- Outlet Structure – Headwall with riprap and duckbill check valve.

Increased Hydraulic Demands
At a flow rate of 1.3 mgd and tailwater elevation of 21.9 feet, Alternatives 1A and 1B would still function as a gravity outfall. As a result of the increased hydraulic demands, the length of RC pipe for both alternatives was reduced from 2,900 linear feet of 2,300 linear feet. Changing this length to HDPE accounts for the increased hydraulic grade line resulting from increased hydraulic demands.

Alternative 2 – Extension of Existing Pipe via Olde Meadow road to Deep Water Outfall

Alternative 2 consists of an extension of the existing discharge pipe to a deep water outfall in Outer Aucoot Cove. The extensions would be a gravity line, with a similar route to Alternative 1, except that the lower portion of the new outfall would move east downstream of Olde Meadow Road, before heading southeast to a deep water outfall in Aucoot Cove. The total length of this alignment is approximately 12,800 feet of extension; approximately 7,600 feet longer than Alternative 1. Two sub-alternatives were developed for this alternative (Alternatives 2A and 2B) as shown in **Figures 5 and 6** respectively.

Both alternatives would have approximately 5,900 linear feet of 18-inch-diameter HDPE pipe. The difference between the two alternatives would be the method of installation of the final 6,900-foot-long, 16-inch-diameter HDPE reach. Under Alternative 2A the final reach would be installed using open cut. Under Alternative 2B, the final reach would be installed via HDD to mitigate impacts to various resource areas. In both cases the diffuser would either be partly buried with an exposed pipe crown with ports or fully buried with risers extending above the seabed. As with Alternative 1, work in the private Olde Meadow Road would require coordination with owners.

During high flows and high tailwater associated with storm flooding, a significant portion of the outfall extension would be under pressure. To prevent overflows from occurring under these conditions, solid HDPE pipe with fittings will be installed through the manholes in the affected reaches. These pipe reaches will not be open to the atmosphere at manholes; however, access to the pipe will be accommodated via tees with blind flanges located inside the manholes.

A flushing station would be provided at the southern end of Olde Meadow Road to allow for flushing of the downstream end of the outfall to remove any sediment accumulated in the pipe. The flushing station would include a tee and with a valve just upstream. The valve would be closed and water would be pumped into the line from tanker trucks at a rate that would produce velocities of at least 2 feet per second in the pipe (approximately 1,600 gallons per minute). The high-quality effluent from the WWTP makes this approach feasible because flushing would rarely, if ever, expect to be needed.

The following summarizes the materials/components associated with each alternative.

Alternative 2A
- Open cut installation of approximately 5,900 linear feet of 18-inch HDPE pipe

- 9 precast concrete manholes (4-foot-diameter)
- 1 flushing station
- Underwater, open cut installation of approximately 6,900 linear feet of 16-inch HDPE pipe
- Multiport diffuser

Alternative 2B

- Open cut installation of approximately 5,900 linear feet of 18-inch HDPE pipe
- 9 precast concrete manholes (4-foot-diameter)
- 1 flushing station
- HDD installation of approximately 6,900 linear feet of 16-inch HDPE pipe
- Multiport diffuser

Increased Hydraulic Demands

At a flow rate of 1.3 mgd and tailwater elevation of 21.9 feet, Alternatives 2A and 2B would not be able to function as a gravity outfall. To meet these demands, the existing RC pipe between Route 6 and the existing outfall terminus could be slip lined with 16-inch HDPE, creating a continuous HDPE outfall between the WWTP and the deep water outfall. Hydraulic modeling indicates that with the liner there would be sufficient freeboard at the WWTP to operate the outfall by gravity such that plant hydraulics would not be affected. To account for this possibility, the entire 5,900-linear-foot reach described above was included as HDPE pipe. Under present hydraulic conditions, the upstream-most 2,300 linear feet could be RC pipe.

Alternative 3 –Relocation of Outfall via Route 6 and Converse Road to Deep Water

Alternative 3 consists of re-routing of the current WWTP discharge pipe to a deep water outfall in Outer Aucoot Cove. This alternative requires a new pumping station, located at the former chlorination facility, and approximately 17,900 feet of new force main pipe. From the pumping station the force main would run northeast on Route 6 to its intersection with Converse Road. The proposed route then continues southerly on Converse Road to the intersection of Converse Road and Wianno Road. At this intersection, the force main would be directed southwest to a deep water terminus in Aucoot Cove. Three sub-alternatives were developed for this alternative (Alternatives 3A, 3B, and 3C) as shown in **Figures 7, 8, and 9,** respectively.

Each of the sub-alternatives require a pump station. An all-gravity approach for the route was evaluated from the outfall back to the WWTP, but the topography along the route, combined with the headloss associated with the length, would result in system backups and unpermitted discharges during coastal flooding conditions.

Alternatives 3A and 3B would have 14,400 linear feet of 16-inch diameter HDPE force main installed by open cut methods. The difference between the two alternatives would be the method of installation of the final 3,500-foot long HDPE reach. Under Alternative 3A the final reach would be installed using of cut. Under Alternative 3B, the final reach would be installed via HDD to mitigate impacts to eel grass beds in Aucoot Cove. Alternative 3C is similar to Alternative 3B, except that approximately 5,000 feet of 16-inch-diameter HDPE between Puckerbush Lane and the upstream end of the final reach would be installed via HDD. In all cases the diffuser would either be partly buried with an exposed pipe crown with ports or fully buried with risers extending above the seabed.

Air release valve structures would be located at highpoints along the force main to prevent air pockets from developing as part of each sub-alternative. The structures will include tees with blind flanges to allow access for cleaning of the force main.

Alternative 3 would require construction in both Route 6 and Converse Road, which was recently repaved at a high cost to the Town. While approaches implementing HDD were developed to reduce impacts, HDD still entails significant construction activity at certain points along the route.

The following summarizes the materials/components associated with each alternative.

Alternative 3A

- 1.18 mgd pumping station at the location of the former chlorination facility, with approximately 32 feet of head required from pump system
- Open cut installation of 14,400 linear feet of 16-inch HDPE force main
- 7 air release valve structures
- Underwater, open cut installation of approximately 3,500 linear feet of 16-inch HDPE pipe
- Multiport diffuser

Alternative 3B

- 1.18 mgd pumping station at the location of the former chlorination facility, with approximately 32 feet of head required from pump system
- Open cut installation of 14,400 linear feet of 16-inch HDPE force main
- 6 air release valve structures
- HDD installation of approximately 3,500 linear feet of 16-inch HDPE force main
- Multiport diffuser

Alternative 3C

- 1.18 mgd pumping station at the location of the former chlorination facility, with approximately 32 feet of head required from pump system
- Open cut installation of 9,400 linear feet of 16-inch HDPE force main
- HDD installation of approximated 5,000 linear feet of 16-inch HDPE force main beneath roadways

- 7 air release valve structures
- HDD installation of approximately 3,500 linear feet of 16-inch HDPE force main
- Multiport diffuser deep water outfall

Increased Hydraulic Demands
At a flow rate of 1.3 mgd and tailwater elevation of 21.9 feet, the pumps associated with Alternatives 3A, 3B, and 3C would require a minor increase in head capacity of approximately 3 feet. This is because the discharge head is governed by highpoints along the force main route and not the tailwater.

Alternative 4 - Extension of Existing Pipe via Olde Logging Road, Olde Knoll Road, and Converse Road to Deep Water

Alternative 4 includes an extension of the existing outfall with a gravity line down Olde Logging Road, continuing south to its intersection with Olde Knoll Road. The route then continues northeast on Olde Knoll Road to the intersection at Converse Road. From this point the pipe would turn southeast and continue along the same route as Alternative 3 to a deep water outfall in Aucoot Cove. The total extension length would be approximately 16,400 linear feet. Two sub-alternatives were developed for the alternative (Alternatives 4A and 4B) as shown in **Figures 10 and 11,** respectively.

Both alternatives would have approximately 12,900 linear feet of 18-inch-diameter HDPE pipe. The difference between the two alternatives would be the method of installation of the final 3,500-foot-long, 16-inch-diameter HDPE reach. Under Alternative 4A the final reach would be installed using of open cut. Under Alternative 4B, the final reach would be installed via HDD to mitigate impacts to eel grass beds in Aucoot Cove. In both cases the diffuser would either be partly buried with an exposed pipe crown with ports or fully buried with risers extending above the seabed.

A flushing station would be provided at the upstream end of the final reach to allow for periodic flushing of the reach to remove any sediment accumulated in the pipe. The flushing station would include a tee and with a valve just upstream. The valve would be closed and water would be pumped into the line from tanker trucks at a rate that would produce velocities of at least 2 feet per second in the pipe (approximately 1,600 gallons per minute). The high-quality effluent from the WWTP makes this approach feasible because flushing would rarely, if ever, expect to be needed.

Alternative 4 would require construction in Converse Road, which was recently repaved at a high cost to the Town. Though no alternative with trenchless method was developed to limit impacts to the roadway (as was done with Alternative 3C), it is an option. Additionally, Alternative 4 includes work in Olde Knoll Road and Olde Logging Road, which are private roads, requiring coordination with owners.

The following summarizes the materials/components associated with each alternative.

Alternative 4A

- Open cut installation of approximately 12,900 linear feet of 18-inch HDPE pipe
- 16 precast concrete manholes (4-foot-diameter)

- 1 flushing station
- Underwater, open cut installation of approximately 3,500 linear feet of 16-inch HDPE pipe
- Multiport diffuser

Alternative 4B

- Open cut installation of approximately 12,900 linear feet of 18-inch HDPE pipe
- 16 precast concrete manholes (4-foot-diameter)
- 1 flushing station
- HDD installation of approximately 3,500 linear feet of 16-inch HDPE pipe
- Multiport diffuser

Increased Hydraulic Demands

At a flow rate of 1.3 mgd and tailwater elevation of 21.9 feet, Alternatives 4A and 4B would not be able to function as a gravity outfall. To meet these demands, the existing RC pipe between Route 6 and the existing outfall terminus could be slip lined with 16-inch HDPE, creating a continuous HDPE outfall between the WWTP and the deep water outfall. Hydraulic modeling indicates that with the liner there would be sufficient freeboard at the WWTP to operate the outfall by gravity such that plant hydraulics would not be affected. To account for this possibility, the entire 12,900-linear-foot reach described above was designed with HDPE pipe. Under present hydraulic conditions, all but 130 linear feet of the extension could be constructed with RC pipe.

Alternative 5 - Relocation of Outfall via Route 6 and Mattapoisett to Deep Water

Alternative 5 consists of a re-routing of the current WWTP discharge pipe that would require approximately 18,300 linear feet of new pipe. The route of Alternative 5 begins at the location of the former chlorination facility and continues southerly on Route 6 to its intersection with Indian Cove Road. The pipe is then directed east on Indian Cove Road and then south into the Town of Mattapoisett, on Aucoot Road, to the intersection of Aucoot Road and North Road. The final segment of pipe would be constructed east on North Road and continue to a deep water outfall in Aucoot Cove. As Alternative 5 passes through the Town of Mattapoisett, coordination and agreements between the Towns of Marion and Mattapoisett would be required. Additionally, Indian Cove Road is a private and coordination with owners would be required.

Alternative 5 has two sub-alternatives: Alternatives 5A and 5B, both requiring a pump station. Alternatives 5A and 5B are shown on **Figures 12 and 13**, respectively. An all-gravity approach for the route was evaluated, but the topography but the number of hills along the route would necessitate several extremely deep cut installations to avoid having a significant number of high and low points along the outfall.

Alternatives 5A and 5B would have 16,200 linear feet of 16-inch diameter HDPE force main installed by open cut methods. The difference between the two alternatives would be the method of installation of the final 2,100-foot-long, 16-inch-diameter HDPE reach. Under Alternative 5A the final reach would be installed using open cut. Under Alternative 5B, the final reach would be installed via HDD to mitigate impacts to eel grass beds in Aucoot Cove. In both cases the diffuser would either be partly buried with an exposed pipe crown with ports or fully buried with risers extending above the seabed.

Air release valve structures would be located at highpoints along the force main to prevent air pockets from developing as part of both sub-alternative. The structures will include tees with blind flanges to allow access for cleaning of the force main.

The following summarizes the materials/components associated with each alternative.

Alternative 5A

- 1.18 mgd pumping station at the location of the former chlorination facility, with approximately 8 feet of head required from pump system
- Open cut installation of 16,200 linear feet of 16-inch HDPE force main
- 7 air release valve structures
- Underwater, open cut installation of approximately 2,100 linear feet of 16-inch HDPE pipe
- Multiport diffuser

Alternative 5B

- 1.18 mgd pumping station at the location of the former chlorination facility, with approximately 8 feet of head required from pump system
- Open cut installation of 16,200 linear feet of 16-inch HDPE force main
- 7 air release valve structures
- HDD installation of approximately 2,100 linear feet of 16-inch HDPE force main
- Multiport diffuser

Increased Hydraulic Demands

At a flow rate of 1.3 mgd and tailwater elevation of 21.9 feet, the pumps associated with Alternatives 5A and 5B would require a minor increase in head capacity of approximately 4 feet. This is because under current conditions the discharge head is governed by a highpoint along the force main route and not the tailwater. Under the increased hydraulic conditions the discharge head would be governed by tailwater slightly higher than the highpoint.

Permitting Overview

Implementation of any of the alternatives would require a number of permits and approvals. Depending on the selected alternative, permits and approvals may be required from USACE, USEPA, the United States Fish and Wildlife Service (USFWS), DEP, the Massachusetts Department of Fish and Game, the

Massachusetts Office of Coastal Zone Management, the Department of Conservation and Recreation, the Massachusetts Historical Commission (MHC), Tribal Historic Preservation Officers (THPO), the Massachusetts Department of Transportation (MassDOT), the Marion Conservation Commission and/or the Mattapoisett Conservation Commission.

Permitting an open ocean outfall is complex, and at present has several unknowns, particularly related to requirements under the Ocean Sanctuaries Act. Time will needed to conduct field investigation efforts, required environmental analyses, pre-application coordination with applicable agencies, permit preparation and the permit approval process. Additional consideration needs to be given to access issues for each of the alternatives. Depending on the field studies required for the Ocean Sanctuaries Act, an ocean outfall (Alternatives 2-5) could take 5 years to receive all required permissions. It is estimated that permitting for Alternative 1 could take 3 years.

In general, Alternatives 1A & B, 2A & B, 3A, 4A, and 5A will require more permitting effort than Alternatives 3B, 4B, and 5B, as they will require a Variance from the WPA for impacts to resource areas exceeding the 5,000 sf threshold, as well as preparation of an Environmental Impact Report (EIR), which is necessary for any project that proposes a new deep water outfall.

Alternatives 3B, 3C, 4B, and 5B are likely to require less permitting effort than the other alternatives. These alternatives will minimize impacts to environmental resource areas by utilizing HDD for installation of the final reach to the deep water outfall.

Table 1 presents the anticipated approvals required for each alternatives. Sub-alternatives with the same impacts were kept in groups. The table includes approximate timelines for receiving agency approvals after submittal of the required application/document. It is important to note that the time required to prepare these permits is significantly longer than the approval period. The Permitting Memorandum, attached to this memorandum, includes greater detail on the regulatory agencies and approvals.

Table 1 – Summary of Permitting Requirements

Environmental Permitting Needs		Alts. 1A and 1B	Alts. 2A and 2B	Alt. 3A	Alts. 3B and 3C	Alt. 4A	Alt. 4B	Alt. 5A	Alt. 5B
Federal Approvals	**USACE Pre-Construction Notification** ***(Approximately 3-4 months)****	X			X		X		X
	USACE Individual Permit ***(Approximately 9 months to 1 year)****		X	X		X		X	

Environmental Permitting Needs		Alts. 1A and 1B	Alts. 2A and 2B	Alt. 3A	Alts. 3B and 3C	Alt. 4A	Alt. 4B	Alt. 5A	Alt. 5B
	USFWS Agency Coordination ***(Approximately 1-2 months)****	X	X	X	X	X	X	X	X
	NPDES Construction General Permit (GP) ***(Contractor Obtains)***	X	X	X	X	X	X	X	X
State Approvals	**MEPA Certificate (ENF)** ***(Approximately 1 month)****	X	X	X	X	X	X	X	X
	MEPA Certificate (EIR) ***(Approximately 1.5 months)****	X	X	X	X	X	X	X	X
	Mass DMF ***(Approximately 1-2 months)****	X	X	X	X	X	X	X	X
	Natural Heritage and Endangered Species Program ***(Approximately 1 month)****	X	X	X	X	X	X	X	X
	MassDEP Variance from the WPA ***(Approximately 1-2 years)****	X	X	X		X		X	
	MassDEP 401 WQC ***(Approximately 6 months)****	X	X	X	X	X	X	X	X
	MassDEP Chapter 91 Waterways License ***(Approximately 6 months)****	X	X	X	X	X	X	X	X
	Massachusetts Historical Commission ***(Approximately 1 month)****	X	X	X	X	X	X	X	X
	Tribal Head Preservation Office's ***(Approximately 1-2 months)****	X	X	X	X	X	X	X	X

Environmental Permitting Needs		Alts. 1A and 1B	Alts. 2A and 2B	Alt. 3A	Alts. 3B and 3C	Alt. 4A	Alt. 4B	Alt. 5A	Alt. 5B
	MassDOT Highway Access Permit ***(Approximately 1-3 months)****			X	X			X	X
Local Approvals	**Marion and/or Mattapoisett Conservation Commission Order of Conditions** ***(Approximately 2-3 months)****	X	X	X	X	X	X	X	X

Costs

Capital costs were developed for each of the alternatives. These costs include escalation to the midpoint of construction, considered to be in 2020 for Alternative 1 and 2022 for Alternatives 2 through 5 in accordance with the anticipated permitting period previously discussed, at a 3% annual inflation rate. Capital costs also include 20% for engineering, permitting and implementation, 20% for construction contingencies and 10% for project contingencies. Costs for easements and land acquisition are not included.

For Alternatives 3 and 5, pumping station costs were considered uniform despite the differences in required head associated with different alternatives since the difference in pump costs would be small relative to the entire capital cost associated with a pumping station. The pumping station costs also include demolition of the existing former chlorination facility, where applicable. An allowance of $2 million was include in the escalated 2022 capital costs for Alternatives 2 through 5 to account the extensive studies required for the permitting of a deep water outfall under the Ocean Sanctuaries Act.

O&M costs were developed for Alternatives 3 and 5 to account for electrical costs associated with pump operation. A flow rate of 0.588 mgd (average daily flow at the WWTP), tailwater of 2 (representing water levels in Aucoot Cove) and the associated head values for each of the systems were considered when developing the costs. It was assumed that the pumps would be on variable frequency drives (VFDs) to maximize efficiency. A pump efficiency of 75%, motor efficiency of 94% and VFD efficiency of 97% were applied. An electrical rate of $0.17 per kilowatt-hour was used, without a discount. O&M costs associated with pump maintenance and maintenance of gravity or force main pipelines were not considered. An annual operating allowance of $1,000 was included as an O&M cost for flushing activities associated with Alternatives 2 and 4, representing flushing every five years at a cost of $5,000. The O&M costs were escalated to the anticipated completion of construction in 2023 considering a 3% inflation rate, then evaluated over a 30-year period.

The estimated capital and O&M costs for each alternative are presented in **Table 2**. (Note – Additional improvements at the WWTP are required as summarized below which are not included in the costs summarized in Table 2.) The net present value (NPV) of each alternative is presented. Alternatives using open cut installation for underwater construction (i.e., Alternatives 1A, 2A, 3A, 4A, and 5A) do not include costs for mitigation of eelgrass beds that would be required. A detailed development of costs can be found in **Appendix D**.

Table 2 – Summary of Alternative Costs

Alternative	Midpoint of Construction	Capital Cost (at midpoint of construction)	Annual O&M Costs (2023)	Total NPV (2016)
1A*	2020	$2,100,000	-	$1,900,000
1B*		$2,600,000	-	$2,300,000
2A	2022	$22,700,000	$1,200	$19,000,000
2B		$24,600,000	$1,300	$20,600,000
3A		$20,300,000	$4,600	$17,100,000
3B		$21,300,000	$4,600	$17,900,000
3C		$23,300,000	$4,600	$19,600,000
4A		$17,300,000	$1,200	$14,500,000
4B		$18,300,000	$1,200	$15,400,000
5A		$17,500,000	$1,000	$14,600,000
5B		$18,100,000	$1,000	$15,100,000

* Alternative would require further treatment to reduce nitrogen to be comparable to Outer Aucoot Cove alternatives.

As shown, the costs for Alternative 1A and 1B, are significantly less than the other alternatives. This is due primarily to the shorter length, absence of underwater construction, and permitting allowance for a deep water outfall, as well as the reduced length of installation. The NPVs of the remaining alternatives range from $15.5 million to $20.6 million, though a number of the costs are tightly grouped, averaging approximately $17.1 million. Alternative 2B carries the highest costs, due largely to the length of underwater HDD.

Costs associated with lining the lagoons and associated improvements at the WWTP ($10.8 million) were not included for any of the alternatives. The required WWTP upgrades are necessary for a number of reasons which are discussed in detail in the WWTP Improvements memorandum.

Summary

Each of the alternatives presented herein are feasible options. The following summarizes the costs and permitting issues associated with each alternative.

- For Alternatives 1A and 1B an additional $6.4 million associated with improvements is required at the WWTP to meet TN limits (i.e., biological active filter, carbon addition to the sequencing batch reactors, one additional plant operator), in addition to the NPV values presented in **Table 2**.
- Alternatives 1A and 1B are gravity options. They are significantly less costly than other alternatives. This alternative requires implementation of a biological active filter and additional personnel to meet draft NPDES permit TN limits and source control or further treatment to address copper limits, as discussed in the WWTP Improvements Memorandum.
- Alternatives 2A and 2B are gravity options. These alternatives are expensive relative to the other alternatives, with Alternative 2B being the most costly option. The increase in cost between the sub-alternatives is a result of increased costs associated with underwater HDD over a significant length. These alternatives include work along Olde Meadow Road, which is a private way.
- Alternatives 3A, 3B, and 3C require pumping stations. They represent average costs (excluding Alternatives 1A and 1B). The increase in cost between Alternatives 3B and 3C ($1.7 million) captures the cost associated with HDD construction along Converse Road. These alternatives require work in Route 6 and Converse Road. Cost savings for this alternative could be realized if the route were to continue south along Moorings Road, which is private, before heading out to deep water. The overall length of the outfall would increase, but the savings associated with the reduced length of underwater construction would represent a net reduction in costs
- Alternatives 4A and 4B are gravity options. They represent relatively low costs compared to the other alternatives (excluding Alternatives 1A and 1B). These alternatives require work within Olde Logging Road and Olde Knoll Road, which are private ways. As with Alternatives 3A, 3B, and 3C, costs could be reduced by extending the outfall south along Moorings Road before going to the deep water terminus.
- Alternatives 5A and 5B require pumping stations. They are similar in cost to Alternatives 4A and 4B. These alternatives include construction along Route 6, along Indian Cove Road (a private way), and in the Town of Mattapoisett.

Next Steps

The alternative improvements presented in this memorandum need to be considered against improvements to the WWTP required to meet permit limits discussed in the WWTP Improvements Memorandum.

Should the Town want to pursue either the extension of the outfall to the head of the salt marsh or an ocean outfall, the next step would be to prepare a concept design. The study for a concept design would require collection of field data on potential alignments and resource areas, including topography (which might be obtained from existing sources), flagging of terrestrial resource areas that could be intersected by alignments, bathymetry, and preliminary geotechnical investigations; and for an ocean outfall for Alternatives 2 through 5, the data required by the Ocean Sanctuaries Act, which remains to be defined by the regulatory agencies.

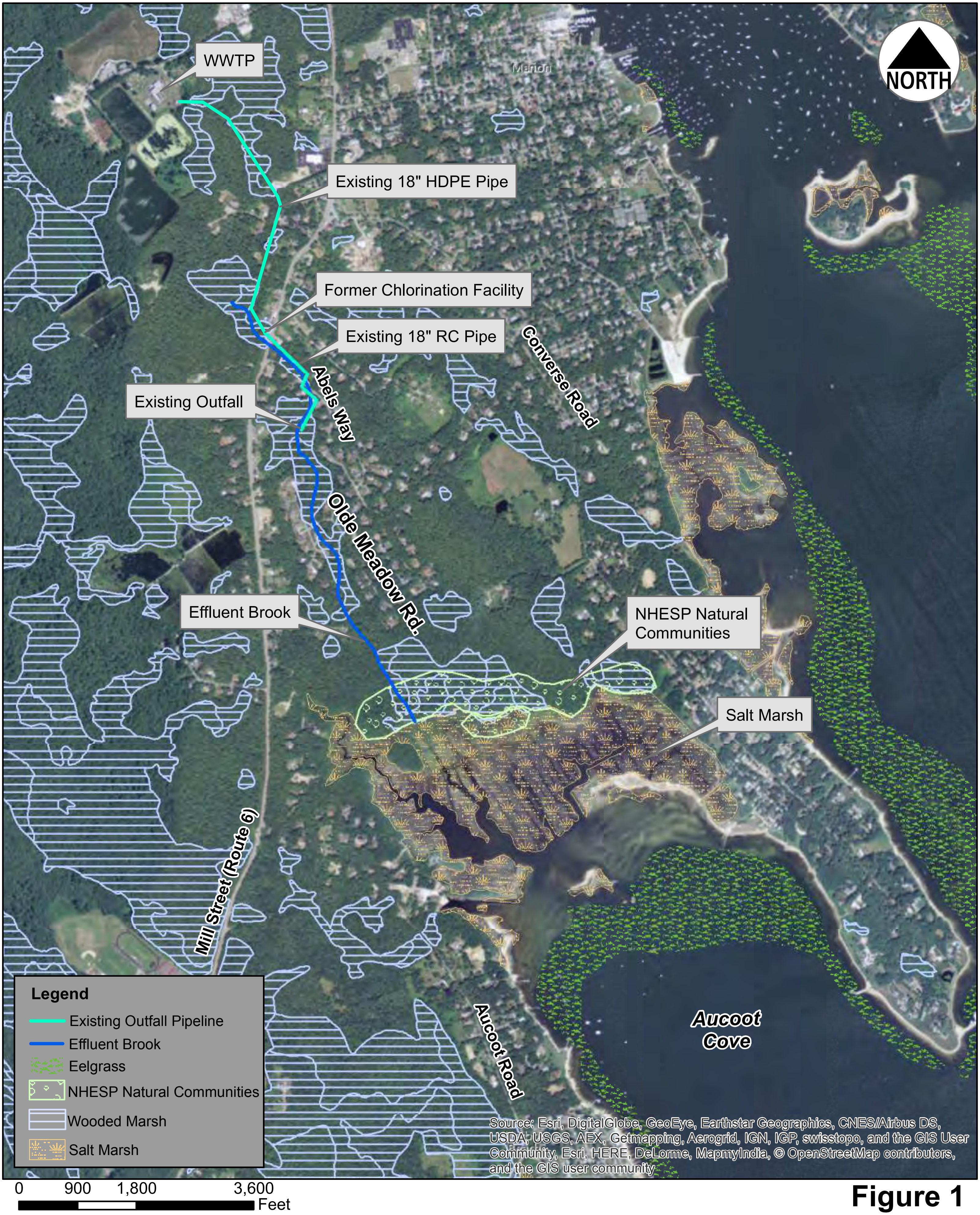
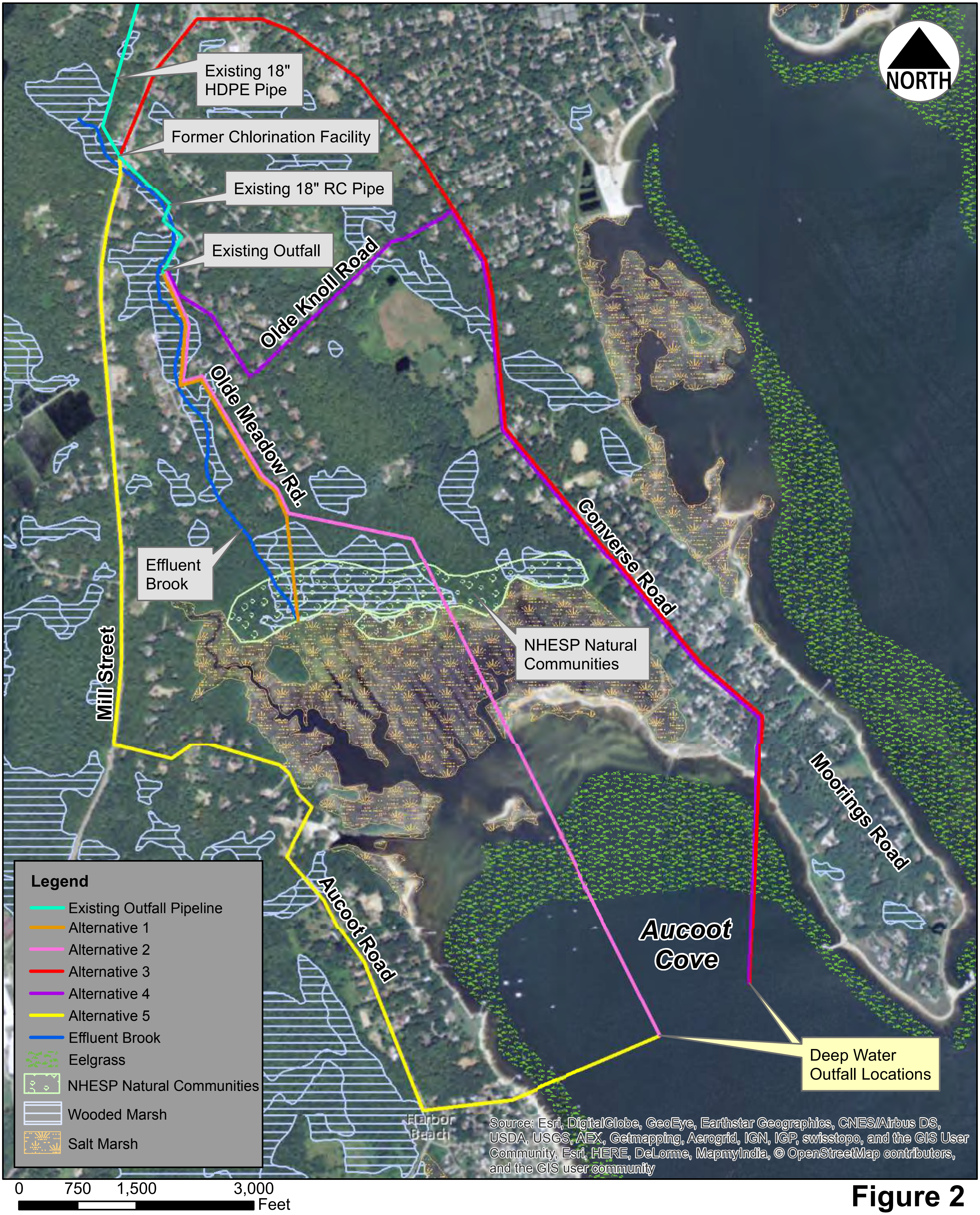


Figure 1

Town of Marion, MA
Existing WWTP Outfall

CDM Smith



CDM Smith

Figure 2

Town of Marion, MA
Outfall Alternatives

Figure 3

Town of Marion, MA
Outfall Alternative 1A

CDM Smith

Figure 4

Town of Marion, MA
Outfall Alternative 1B

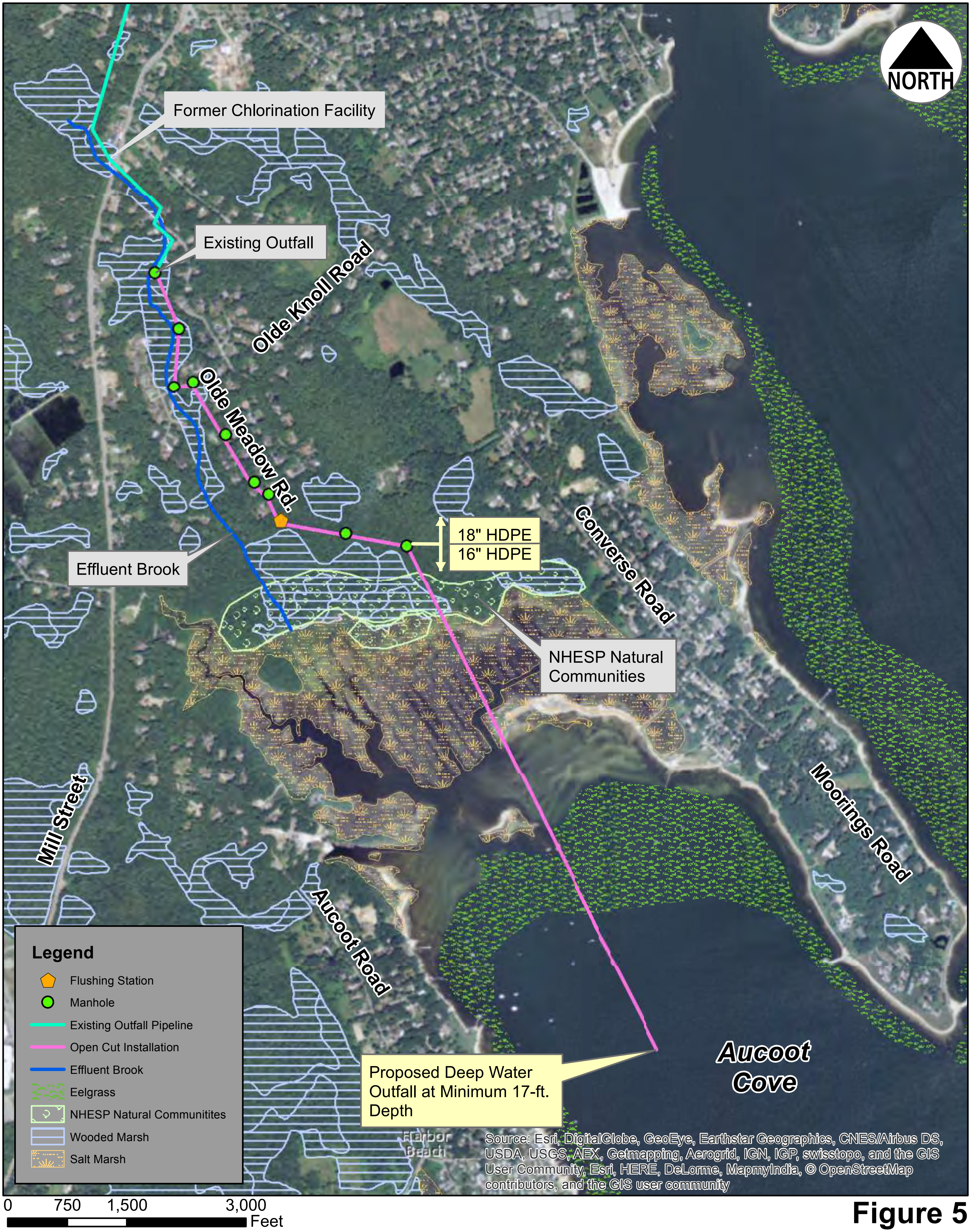


Figure 5

Town of Marion, MA
Outfall Alternative 2A

CDM Smith

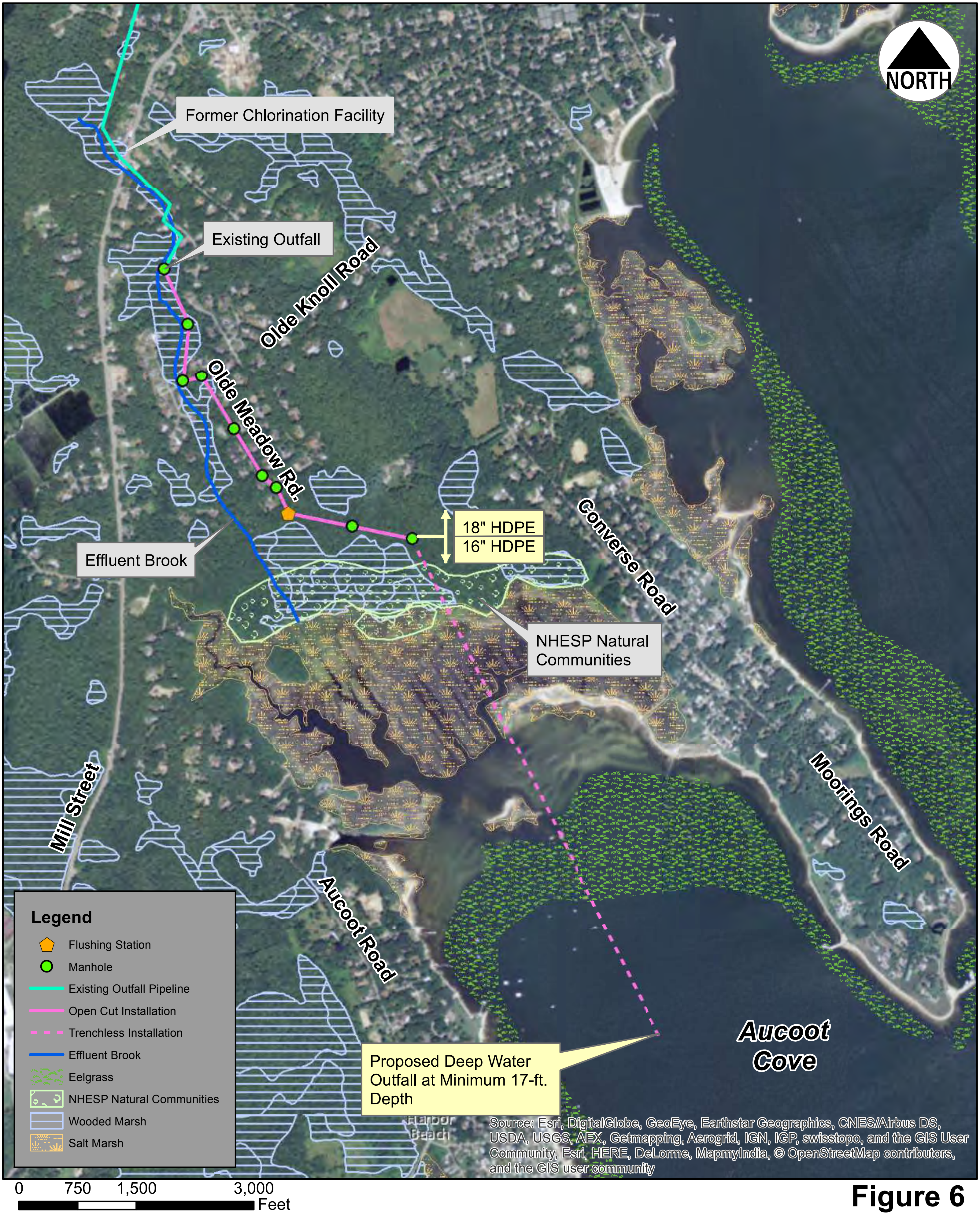


Figure 6

Town of Marion, MA
Outfall Alternative 2B

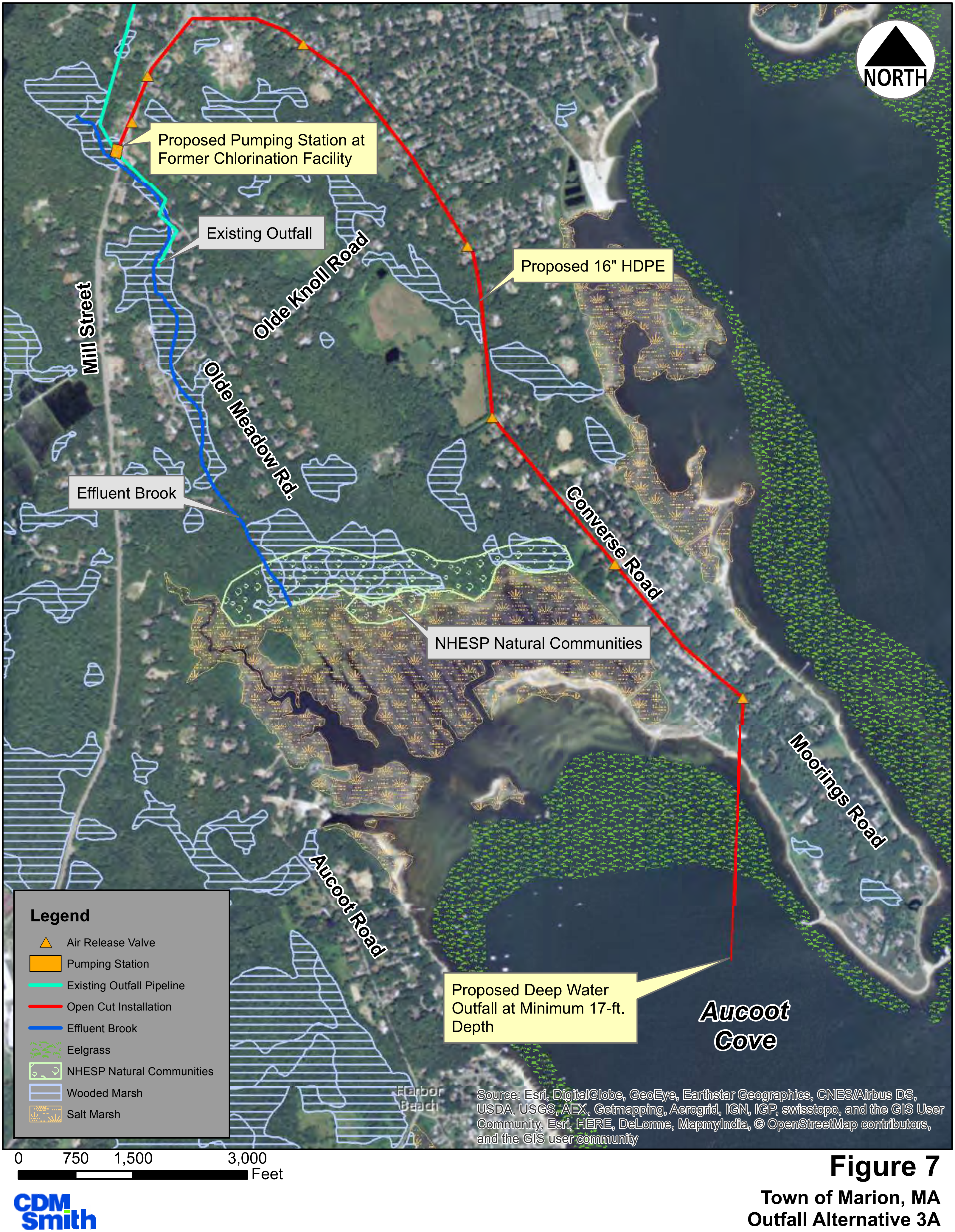
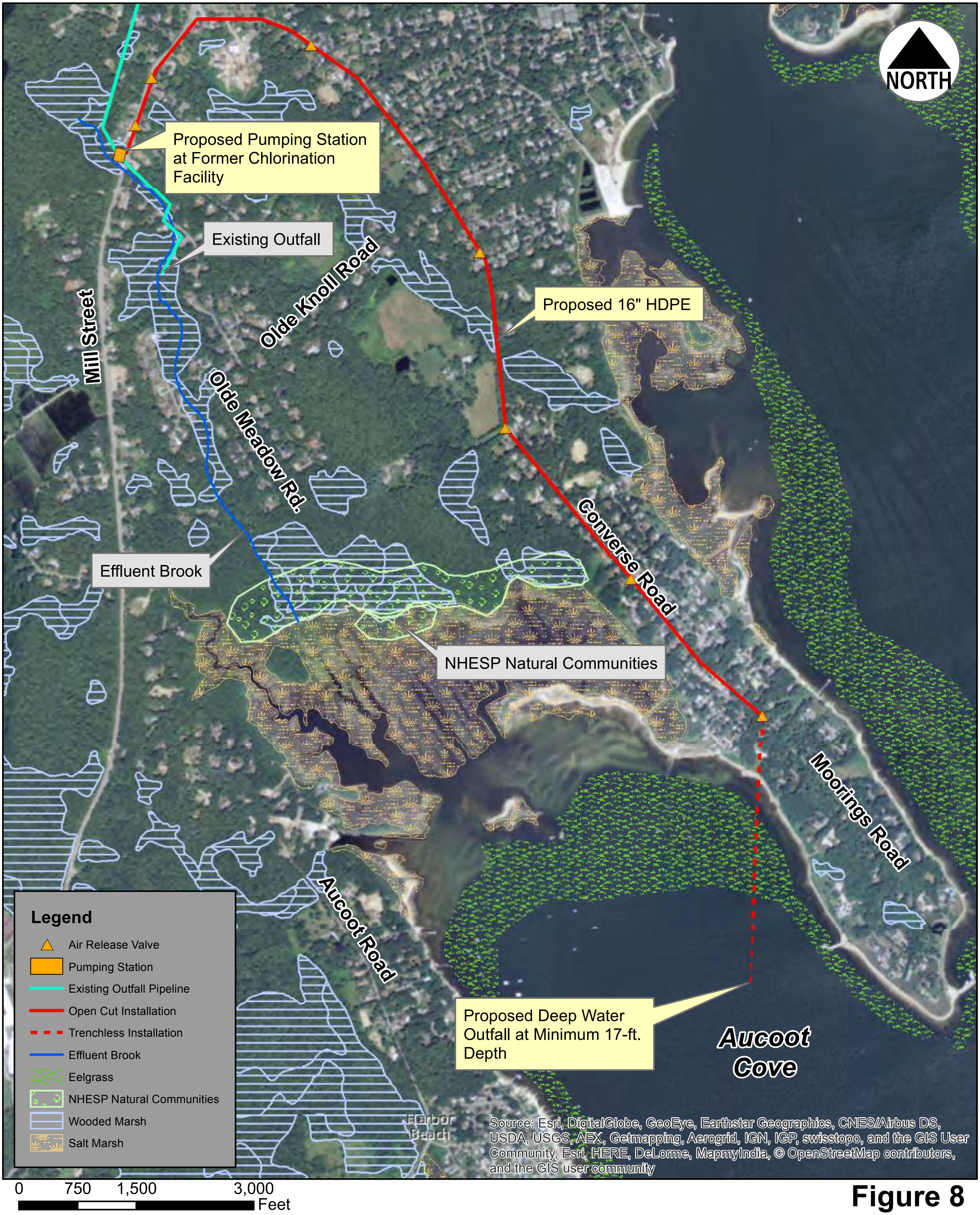


Figure 7

Town of Marion, MA
Outfall Alternative 3A



CDM Smith

Figure 8

Town of Marion, MA
Outfall Alternative 3B

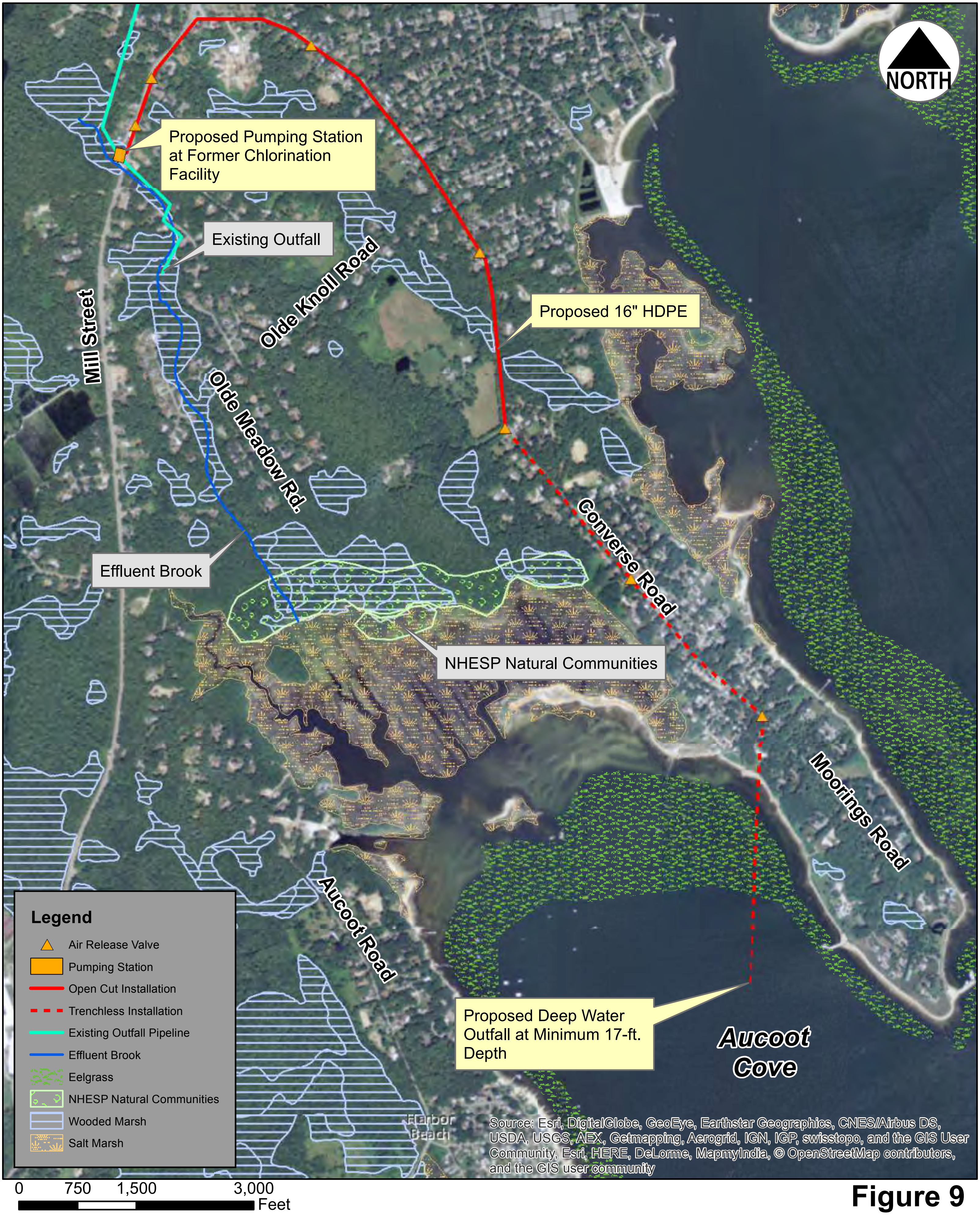


Figure 9

Town of Marion, MA
Outfall Alternative 3C

CDM Smith

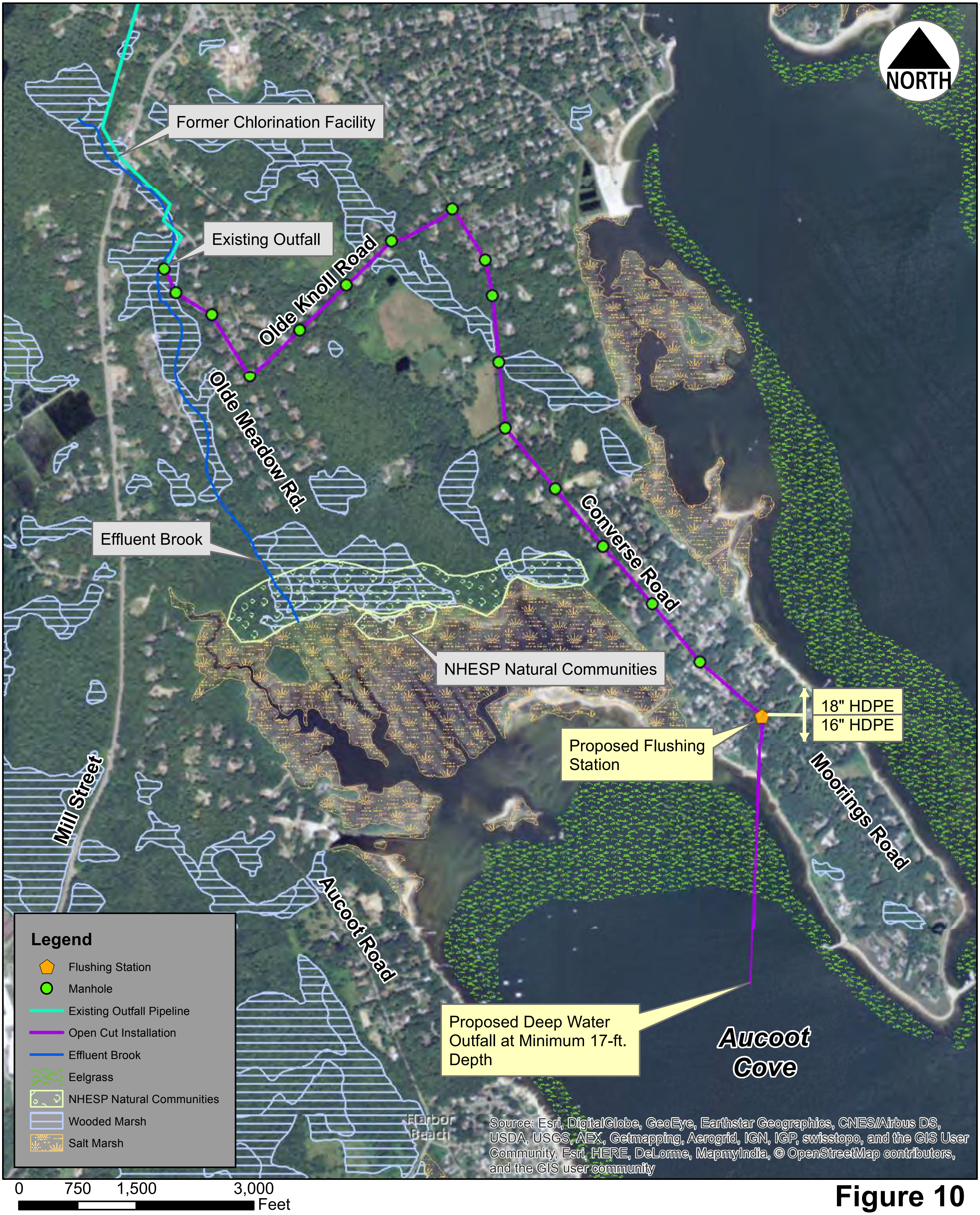
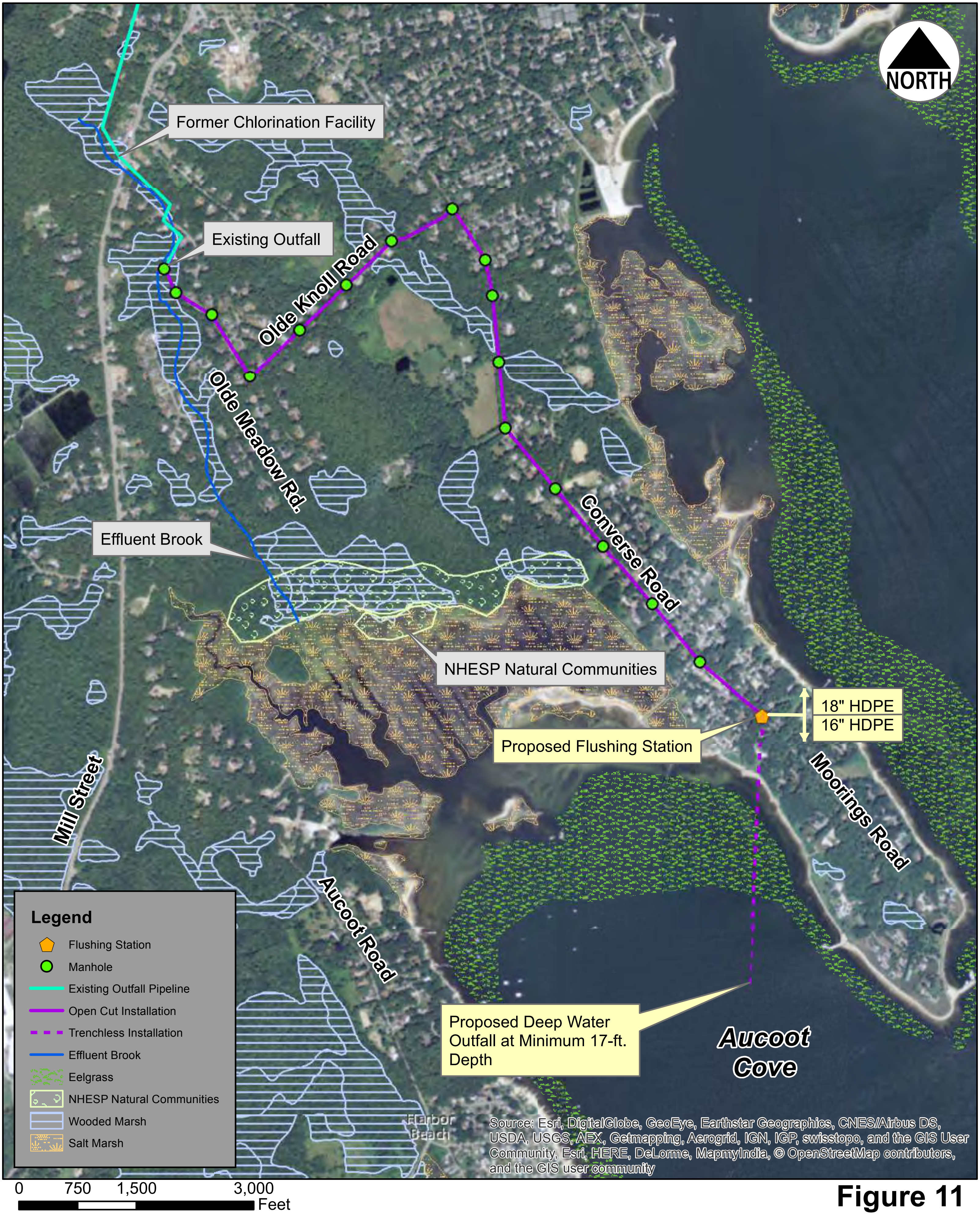


Figure 10

Town of Marion, MA
Outfall Alternative 4A

CDM Smith



CDM Smith

Figure 11

Town of Marion, MA
Outfall Alternative 4B

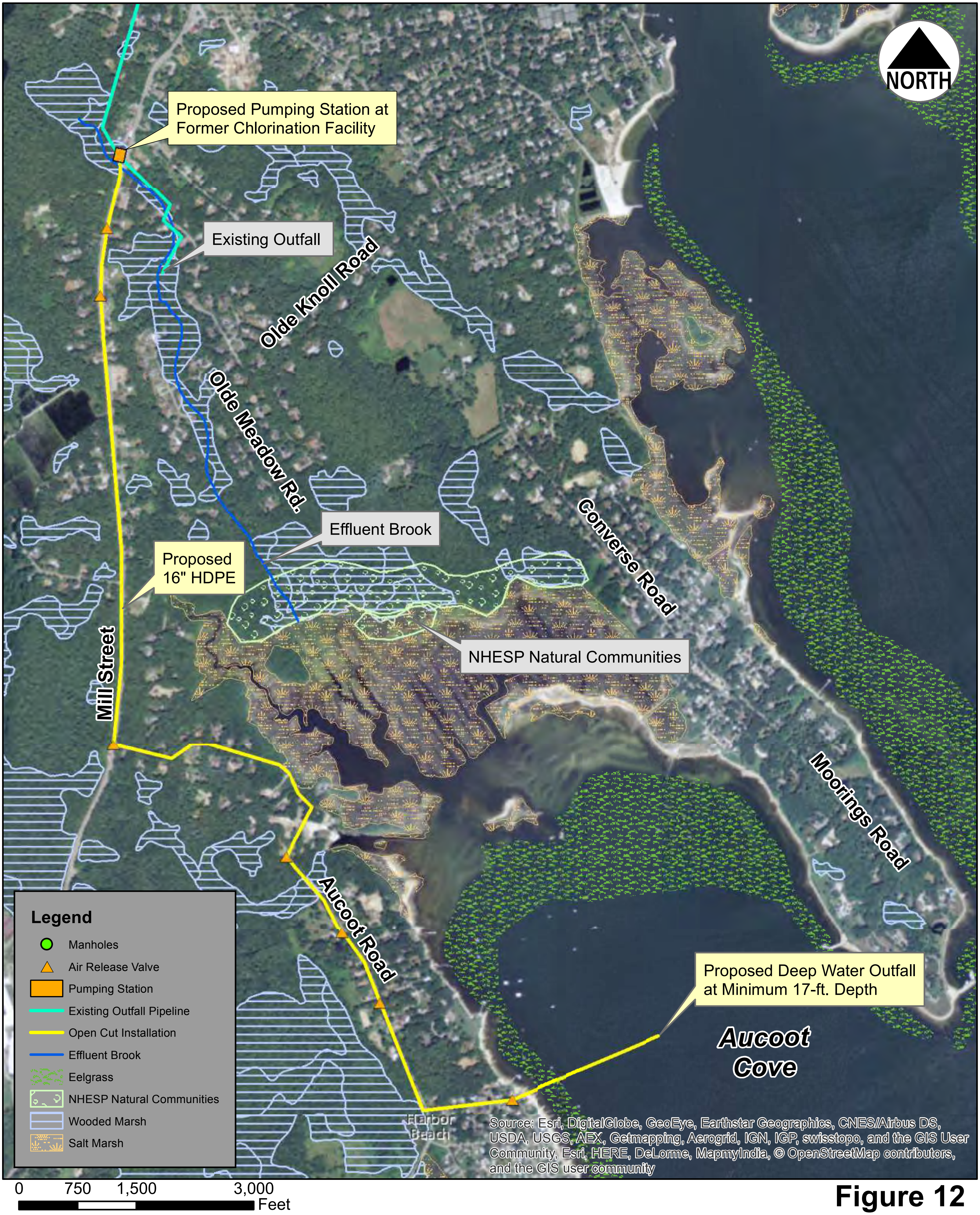


Figure 12
Town of Marion, MA
Outfall Alternative 5A

CDM Smith

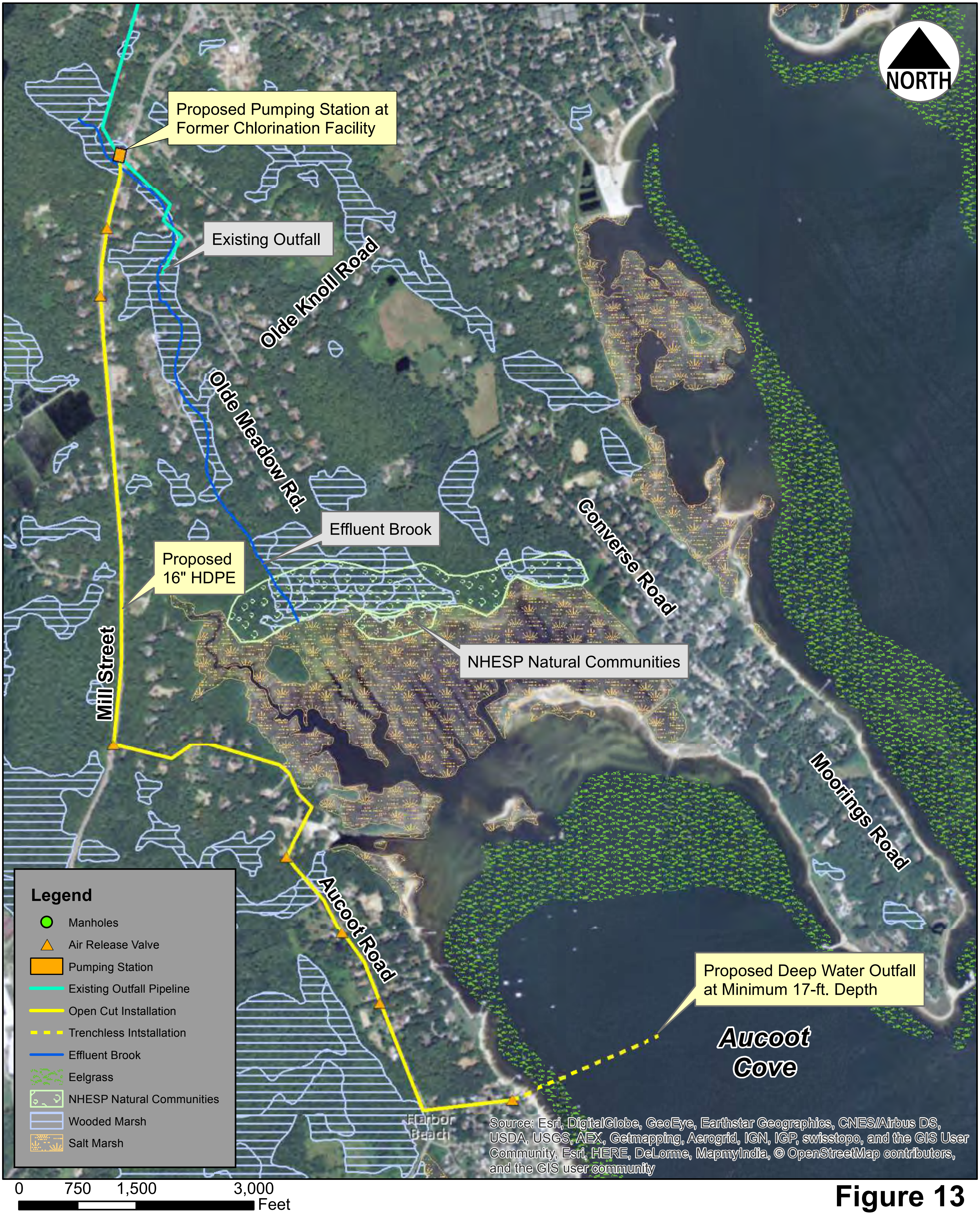


Figure 13

Town of Marion, MA
Outfall Alternative 5B

Appendix A

List of Potential Environmental Permitting Requirements – Effluent Discharge Pipe and Ocean Outfall, Marion, Massachusetts

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Technical Memorandum

To: *Shawn Syde*

From: *Danielle Gallant, QEP and Magdalena Lofstedt, PWS*

Date: *April 7, 2016*

Subject: *List of Potential Environmental Permitting Requirements – Effluent Discharge Pipe and Ocean Outfall, Marion, Massachusetts*

CDM Smith, Inc. (CDM Smith) is working with the Town of Marion (the Town) to design a new wastewater effluent discharge pipe with associated appurtenances, including new manholes, sewerage pumping stations, and a new effluent ocean outfall with an associated rip rap energy dissipating pad. The proposed effluent pipe will extend from the existing effluent pipe to its discharge point within Aucoot Cove. The proposed project is in the conceptual design phase, and the Town is evaluating a number of proposed project routes and construction specifications. An overview of potential project alternatives is provided in Figure 2 of the "Town of Marion Wastewater Treatment Plant Outfall Alternatives – Analysis of Alternatives" memorandum ("Outfall Memorandum") prepared by CDM Smith, dated April 7, 2016. For the purposes of this environmental permitting list, the following assumptions were made:

- The installation of the effluent discharge pipe within the existing roadway and rights of ways will be completed via a combination of open cut trench and horizontal directional drilling (HDD) construction.
- Underwater construction will be done by either a combination of open cut and direct lay methods or by HDD. HDD would be used to reach the proposed outfall location to avoid significant impacts to Aucoot Cove and associated resource areas (i.e., jurisdictional wetland resource areas, including eelgrass beds).
- There will be a riprap energy dissipating pad installed at the terminus of the salt marsh outfall.
- The deep water ocean outfall will consist of multiple diffusers.

The effluent discharge pipe and associated ocean outfall work is anticipated to take place within the following wetland resources areas: Land Under the Ocean, Coastal Beach, Coastal Bank, Salt Marsh, Land Containing Shellfish, Land Subject to Coastal Storm Flowage (LSCSF), Inland Bank, Bordering Vegetated Wetlands (BVW), and the 200-foot Riverfront Area (RFA). These wetland resource areas are subject to jurisdiction per the Massachusetts Wetlands Protection Act (WPA), and the Town of Marion Wetlands Protection Standards. Work associated with the effluent discharge pipe and ocean outfall is also jurisdictional per the U.S. Army Corps of Engineers (USACE) and the Massachusetts Department of

Environmental Protection (MassDEP) per Sections 404 and 401 of the Clean Water Act, respectively, Section 10 of the Rivers and Harbors Act, MassDEP's Chapter 91 Waterways Program, as well as the Massachusetts Environmental Policy Act. This work will also occur within state listed Estimated Habitats of Rare Wildlife, Priority Habitat of Rare Species, and designated Natural Communities all regulated by the Natural Heritage and Endangered Species Program (NHESP), as well as known eelgrass beds regulated by the Massachusetts Division of Marine Fisheries, USACE, MassDEP, and the Marion Conservation Commission. In addition, there are several known federally listed threatened or endangered species that occur within the project area, therefore project review under Section 7 of the Endangered Species Act (ESA) with United States Fish and Wildlife Service (USFWS) will also be required.

Below is a detailed description of five (5) alternative routes that the Town is evaluating. Table 1 summarizes the anticipated environmental permits and approvals that will be required for each alternative. The subsequent section describes the federal, state, and local environmental regulations and their applicability to Alternatives 1 through 5.

Alternative Routes

Below is a description of proposed alternative routes and their potential environmental permitting implications.

Alternatives 1A and 1B

Alternatives 1A and 1B consist of installing the new effluent pipe within cross-country areas and existing roadways. This alternative proposes the start of the new effluent pipe from just north of the northern end of Olde Logging Road, at the existing outfall pipe, to the northern end of Olde Meadow Road. The proposed pipe is routed within Olde Meadow Road to the southern terminus of Olde Meadow Road. From here, the pipe then continues cross-country heading southwest where it joins Effluent Brook at its terminus at the northern border of the existing salt marsh. Contrary to the other alternatives, these are the only alternatives that do not have an ocean outfall; effluent will discharge into a tidal channel.

Alternative 1A will be constructed using open-trench construction for the entirety of the proposed route (see Figure 3 of the Outfall Memorandum). Impacts resulting from this alternative will include approximately 24,000 square feet (sf) of BVW and Salt Marsh. Impacts to LSCSF, RFA, Natural Communities and Priority Habitat of threatened or endangered species will also occur.

Alternative 1B will be constructed using a combination of open-trench construction and HDD. The open trench construction would occur from the northern point of the effluent pipe and continue until the northern edge of BVW at the southern end of Olde Meadow Road. From this location to the outfall, the pipe installation will consist of HDD (see Figure 4 of the Outfall Memorandum). Impacts from this alternative will include approximately 12,000 sf of BVW. Impacts to LSCSF, RFA and Priority Habitat of threatened or endangered species will also occur.

Alternatives 2A and 2B

Alternatives 2A and 2B, consist of installing the new effluent pipe within cross-country areas and existing roadways, and an open ocean outfall in Aucoot Cove. Alternatives 2A and 2B propose a similar effluent pipe installation through the majority of the route described in Alternative 1 above, however the terminus of the pipe extends south-eastward, avoiding a portion of BVW, and then continues south through the salt marsh into the open ocean.

Alternative 2A will be constructed using open trench construction and direct lay of the pipe for the entirety of the route (see Figure 5 of the Outfall Memorandum). This alternative will alter approximately 64,140 sf of BVW, Salt Marsh, Land Under Ocean, Land Containing Shellfish, and vegetated shallows (eel grass beds). Impacts to LSCSF, RFA, Natural Communities, and Priority Habitat of threatened or endangered species are also proposed.

Alternative 2B will be constructed using open trench construction from the existing outfall, along the existing roadway, where it then turns south-eastward to just north of the edge of the Natural Communities (Coastal Forest). At this location, HDD installation of the pipe will begin and extend for approximately 5,800 to the open ocean outfall (see Figure 6 of the Outfall Memorandum). Alternative 2B will alter approximately 12,000 sf of BVW. Impacts to LSCSF, RFA and Priority Habitat of threatened or endangered species are also proposed.

Alternatives 3A through 3C

Alternative 3 consists of installing the new effluent pipe within existing roadways and Aucoot Cove via open cut and direct lay methods. This alternative requires a new pumping station, located at the former chlorination facility. From the pumping station the force main will run northeast on Route 6 to its intersection with Converse Road. The proposed route will then continue southerly on Converse Road to the intersection of Converse Road and Wianno Road. At this intersection, the force main will be directed southwest to a deep water terminus in Aucoot Cove.

Alternative 3A proposes installation of the new effluent pipe and ocean outfall via open-cut construction for the entirety of the proposed route (see Figure 7 of the Outfall Memorandum). This alternative will impact approximately 24,960 sf of Coastal Bank, Coastal Beach, Land Under Ocean, Land Containing Shellfish, and vegetated shallows (eelgrass beds). RFA, LSCSF and Priority Habitat of threatened or endangered species will also be impacted as a result of this alternative.

Alternative 3B consists of installing the new effluent pipe within existing roadways, as well as Aucoot Cove. The proposed pipe will follow the same route as described in alternative 3A above. This alternative will be constructed using a combination of open cut installation and HDD. The pipe will be installed via open cut construction for the majority of the route, with HDD being utilized for cross-country/open ocean areas in the southern portion of the proposed project route (see Figure 8 of the Outfall Memorandum).

Alternative 3B will impact approximately 400 sf of Land Under Ocean and Land Containing Shellfish. This alternative will also impact LSCSF, RFA and Priority Habitat of threatened or endangered species.

Alternative 3C proposed changes only in installation of proposed pipe within the roadway. Therefore, no variation in proposed impacts to resource areas is anticipated, and are summarized in Alternative 3B above.

Alternatives 4A and 4B

Alternatives 4A and 4B have the same ocean outfall location from Converse Road as Alternative 3, however the proposed effluent pipe will begin at the existing outfall pipe. At the existing outfall, the proposed effluent pipe will continue cross-country to the northern terminus of Olde Logging Road. The proposed pipe will continue within Olde Logging Road, and turn east onto Olde Knoll Road. The pipe will then turn eastward on Olde Logging Road until the intersection of Olde Knoll Road and Converse Road. The pipe continues south down Converse Road until the intersection of Converse Road and Wianno Road. The pipe will then travel cross-country/under the ocean to the proposed outfall location.

Alternative 4A will be constructed using a combination of open cut and direct lay installation. The pipe will be constructed via open trench construction from the northern start of the proposed pipe to the start of the cross-country/open ocean route. From this location the proposed pipe will be installed via a combination of underwater open cut and direct lay (see Figure 10 of the Outfall Memorandum).

Alternative 4A will impact approximately 27,840 sf of BVW, Land Under Ocean, Land Containing Shellfish, vegetated shallows (eelgrass beds), Coastal Beach, and Coastal Bank. In addition, this route will impact RFA, LSCSF and Priority Habitat of threatened or endangered species.

Alternative 4B will be constructed using a combination of open trench construction and HDD. The pipe will be installed via open trench for the majority of the proposed route, changing to HDD installation at the cross-country/open ocean portion of the route (see Figure 11 of the Outfall Memorandum).

Alternative 4B will impact approximately 400 sf of Waters of the U.S. (Land Under Ocean and Land Containing Shellfish) as well as LSCSF, RFA and Priority Habitat of threatened or endangered species.

Alternative 5A and 5B

Alternative 5 proposes installation of the new effluent pipe within existing roadways, as well as within Aucoot Cove. The route of Alternative 5 will begin at the location of the former chlorination facility and continue southerly on Route 6 to its intersection with Indian Cove Road. The pipe will then be directed east on Indian Cove Road and then south into the Town of Mattapoisett, on Aucoot Road, to the intersection of Aucoot Road and North Road. The final segment of pipe will be constructed east on North Road and continue to a deep water outfall in Aucoot Cove.

Alternative 5A will be constructed using a combination of open cut and direct lay installation (see Figure 12 of the Outfall Memorandum). This alternative will impact approximately 17,150 sf of BVW, Land Under Ocean, Land Containing Shellfish, vegetated shallows (eelgrass beds), Coastal Beach, and Coastal

Bank. Additionally, this alternative will impact RFA, LSCSF and Priority Habitat of threatened or endangered species.

Alternative 5B will be constructed using a combination of open trench construction and HDD. The pipe will be installed via open trench for the majority of the proposed route, changing to HDD installation at the cross-country/open ocean portion of the route (see Figure 13 of the Outfall Memorandum). This alternative will impact approximately 400 sf of Land Under Ocean and Land Containing Shellfish. LSCSF, RFA and Priority Habitat of threatened or endangered species will also be impacted as a result of this alternative.

Description of Applicable Permits

There are a number of federal, state and local permits required to implement the proposed new effluent discharge alternatives. Because implementing any of the five alternatives directly impacts wetland resource areas and Waters of the U.S., a number of environmental permits and approvals will be required.

During preliminary design of the recommended alternative, a more detailed evaluation will be needed to determine what permits and approvals are required and then a permitting strategy to effectively navigate the approval process will be prepared. This “permitting plan” will be incorporated into the preliminary design report for the project.

The following section identifies federal, state and local permits/approvals required to work in or adjacent to regulated natural resources.

Potential Federal Permits/Approvals

U.S. Army Corps of Engineers (USACE)

The USACE regulates Waters of the U.S. and their associated wetlands through Section 404 of the Clean Water Act. The installation of the effluent discharge pipe, new ocean outfall and associated rip rap energy dissipating pad will require approval by USACE. Cumulative impacts, including up to ½ acre (21,000 sf) of impacts to Tidal Waters of the U.S (excluding wetlands), 1000 sf of impacts to Tidal Waters of the US within Special Aquatic Sites (SAS) (e.g. wetlands), and 100 sf of impacts to SAS including vegetated shallows (e.g. eel grass beds) qualify as a Pre-Construction Notification (PCN) General Permit (GP) 9 Activity (Utility Line Activities) [Sections 10 and 404] pursuant to the Department of Army New England GP, (effective date: March 9, 2015). A USACE PCN takes approximately one month to prepare and a maximum of 90 days to approve.

If the preferred alternative proposes impacts to 100 sf or more of existing vegetated shallows such as eel grass beds, regulated as SAS by the USACE, 1,000 sf of impacts to tidal wetlands (salt marsh) regulated as SAS by the USACE, or more than ½ acre (21,000 sf) of impacts to Tidal Waters of the U.S., then the project will require an Individual Permit (IP) from the USACE. The permit process will begin with a pre-application meeting with all project stakeholders. Once the permit application is complete and submitted to USACE, a public interest review period of 30 days begins with the issuance of a public notice. During the public interest review period, the application is analyzed for concurrence with CFR 40 Part 230 Section 404(b)(1) Guidelines for Section 404 Activities. These guidelines are designed to avoid unnecessary filling of wetlands and waterways and prohibit discharges:

- Where less environmentally damaging, practicable, alternatives exist;
- Which result in violations of State or Federal Water Quality Standards, the Endangered Species Act, and the Marine Sanctuaries Act;
- Which cause or contribute to significant degradation of waters and wetlands;
- If all appropriate and practical mitigation has not been taken; or
- If there is not sufficient information to determine compliance with the guidelines.

After the determination is made on concurrence with the 404(b)(1) Guidelines, the applicant can respond to any requests for additional information. If required, a public hearing is held and all necessary information incorporated in the permit application. At this time, the USACE can make a final determination. The IP process can take approximately 9 months to 1 year to complete.

As a part of the PCN and IP processes, several Massachusetts state agencies and federal agencies will be contacted to solicit input on potential project impacts to their associated protected resources. These include:

- The USFWS will be notified to confirm that the project will have “no effect” or “not likely to effect” a federally listed threatened or endangered species. Additional information on this process is included below.
- The National Marine Fisheries Service will be contacted to provide input on potential impacts to existing protected species and essential fish habitat. Additional details on coordination with the Department of Fish and Game is included below.
- The Massachusetts Office of Coastal Zone Management (CZM) will be contacted to ensure consistency with their Coastal Zone Management Policy Guide. Additional information on CZM consistency is detailed below.
- The State Historic Preservation Officer (SHPO) at the Massachusetts Historical Commission (MHC), the Massachusetts Board of Underwater Archaeological Resources (BUAR), and the Tribal Historic Preservation Officers (THPO) for the Wampanoag Tribe of Gay Head (Aquinnah), and Mashpee Wampanoag Tribe will be notified to confirm that the project will not affect cultural resources. Further details on these agency requirements are included below.

U.S. Fish and Wildlife Service

Section 10 of the Endangered Species Act (ESA) is designed to regulate a wide range of activities affecting plants and animals designated as Endangered or Threatened, and the habitats upon which they depend. With some exceptions, the ESA prohibits activities affecting these protected species and their habitats unless authorized by a permit from the U.S. Fish and Wildlife Service (USFWS) or the National Marine Fisheries Service (NMFS). Permitted activities are designed to be consistent with the conservation of the protected species.

Section 7 of the ESA requires Federal agencies to consult with the USFWS to ensure that actions they fund, authorize, permit, or otherwise carry out will not jeopardize the continued existence of any listed species or adversely modify designated critical habitats. The proposed project area is known habitat for several federally protected threatened or endangered species including the roseate tern (*Sterna dougallii*), the red knot (*Calidris canutus*), and the northern long-eared bat (*Myotis septentrionalis*). Coordination with USFWS will be required to confirm that the project will have "no effect" or is "not likely to effect" a federally listed threatened or endangered species. The coordination process can take approximately 30-60 days.

U.S. Environmental Protection Agency (USEPA)

The U.S. Environmental Protection Agency (EPA) regulates point source discharges of pollutants to waters of the United States through the National Pollutant Discharge Elimination System (NPDES) process. All of the project alternatives will require a NPDES Construction General Permit (CGP) for total land disturbance of equal to or greater than one acre, and for stormwater discharges to Waters of the U.S. Pursuant to the requirements of the CGP, the project proponent, or designee, will prepare a Storm Water Pollution Prevention Pollution Plan (SWPPP) to document stormwater control measures during the construction periods for the projects. Following completion of the SWPPP, the proponent or designee will complete and submit to EPA a Notice of Intent to discharge stormwater. The selected contractor will be responsible for obtaining the NPDES CGP and preparing the SWPPP after award of the contract. There is no review time for a NPDES CGP permit. The eNOI has to be submitted at least 14 days prior to start of construction.

Potential State Permitting Requirements

Massachusetts Environmental Policy Act

The Massachusetts Environmental Policy Act (MEPA) applies to projects in Massachusetts that exceed defined thresholds and involve state agency action (i.e., projects that are either proposed by a state agency or require a permit, financial assistance, and/or land transfer from one or more state agencies). Projects that fall within MEPA jurisdiction are generally reviewed in a two-step process, beginning with the filing of an Environmental Notification Form (ENF), followed by an Environmental Impact Report (EIR) if needed. All alternatives of the proposed project will require the filing of an ENF and an EIR due to the proposed open ocean outfall and required compliance with the Ocean Sanctuaries Act described below. Additionally, depending on the preferred alternative the following thresholds per 301 CMR 11.03 may be exceeded, which will also trigger the filing of an EIR:

- Provided that a Permit is required, alteration of one or more acres of salt marsh or bordering vegetated wetlands [301 CMR 11.03 (3)(a)(1)(a)];

- Alteration requiring a variance in accordance with the Wetlands Protection Act [301 CMR 11.03 (3)(a)(2)]
- Provided that a Permit is required alteration of ten or more acres of any other wetlands [301 CMR 11.03 (3)(a)(1)(b)];

An EIR takes approximately 3 months to prepare and a month and a half to approve.

Depending on the preferred alternative one or several of the following thresholds per 301 CMR 11.03 will be exceeded requiring the filing of an ENF:

- Alteration of designated significant habitat [301 CMR 11.03 (2)(b)(1)];
- Alteration of greater than two acres of disturbance of designated priority habitat, as defined in 321 CMR 10.02, that results in a take of state-listed endangered or threatened species or species of special concern [301 CMR 11.03 (2)(b)(2)];
- Alteration of coastal dune, barrier beach or coastal bank [301 CMR 11.03 (3)(b)(1)(a)];
- Alteration of 1,000 or more sf of salt marsh or outstanding resource waters [301 CMR 11.03 (3)(b)(1)(c)];
- Alteration of 5,000 or more sf of bordering or isolated vegetated wetlands [301 CMR 11.03 (3)(b)(1)(d)];
- New fill or structure or Expansion of existing fill or structure, except a pile-supported structure, in a velocity zone or regulatory floodway [301 CMR 11.03 (3)(b)(1)(e)];
- New discharge or Expansion in discharge to a surface water of: 100,000 or more gpd of sewage [301 CMR 11.03 (5)(b)(4)(b)(i)];
- New discharge or Expansion in discharge to a surface water of: any amount of sewage, industrial waste water or untreated stormwater requiring a variance from applicable water quality regulations [301 CMR 11.03 (5)(b)(4)(b)(iii)].

An ENF takes approximately two months to prepare and at least 5 weeks to approve.

Massachusetts Department of Fish and Game

Division of Marine Fisheries

As part of the Massachusetts Wetlands Protection Act Notice of Intent (NOI) permitting process, the Massachusetts Division of Marine Fisheries (MassDMF) will be sent an electronic file of the NOI if work will occur below the mean high water line. MassDMF provides comment on the NOI on potential impacts to fishery resources including land containing shellfish, essential fish habitat, and vegetated shallows. The coordination process can take approximately 30 days.

Natural Heritage and Endangered Species Program (NHESP)

Aucoot Cove and its surrounding area is designated as Estimated Habitat of Rare Species and Priority Habitat of Rare Species. Additionally, two Natural Communities (a Coastal Forest/woodland and a Sea-Level Fen) have been identified within the potential project area. As a part of the Marion and/or Mattapoisett Conservation Commission NOI permitting process detailed below, coordination with the NHESP will be required to ensure that the project will not result in a “take” on a state listed threatened or endangered species, or a state designated Natural Community. NHESP coordination can take approximately 30-60 days, but may require additional time if the agency determines that additional investigative work is required. The NOIs submitted to the Marion and/or Mattapoisett Conservation Commission/or the Variance Request can be reviewed jointly with NHESP.

Massachusetts Department of Environmental Protection

Wetlands and Waterways Program

Variance Request

In general, work within jurisdictional wetland resource areas is regulated by the local Conservation Commissions. However, when the proposed project is not a limited project, and impacts exceed 5,000 sf of impact to jurisdictional wetland resource areas, the project will need to be reviewed and approved by MassDEP. The installation of the effluent discharge pipe is not designated as a limited project per 310 CMR 10.24 (7) or 310 CMR 10.53 (3), because the outfall pipe is considered a drainage pipe outside of a closed circuit utility system. Therefore, this work may require review and approval by MassDEP, instead of the Conservation Commission(s), depending on which alternative is chosen. If the proposed project alternative impacts greater than 5,000 sf of jurisdictional wetland resource areas, than a Variance Request will need to be filed with MassDEP requesting a variance from the WPA. The Variance Request cannot be reviewed by MassDEP until a MEPA certificate for the Environmental Impact Report (EIR) has been issued. If a variance is required, the process starts with filing a Notice of Intent with the Marion and/or Mattapoisett Conservation Commission, followed by filing a Request for a Superseding Order of Conditions from MassDEP, and ultimately a Request for a Variance. The Commissioner will grant the Variance when he or she finds that:

- There are no reasonable conditions or alternatives that would allow the project to proceed in compliance with 310 CMR 10.21 through 10.60;
- That mitigating measures are proposed that will allow the project to be conditioned so as to contribute to the protection of the interests identified in M.G.L.c. 131, § 40; and
- That the variance is necessary to accommodate an overriding community, regional state or national public interest; or that it is necessary to avoid an Order that so restricts the use of property as to constitute an unconstitutional taking without compensation.

In addition to the proposed effluent pipe work, the work associated with the ocean outfall is under the jurisdiction of the MassDEP per Section 401 of the Clean Water Act, and the MassDEP Chapter 91 Waterways Program. Proposed project work regulated by the abovementioned programs is detailed below.

A Request for a Variance takes approximately three months to prepare and approval time is approximately one to two years.

Section 401 Water Quality Certification

The 401 Water Quality Certification (WQC) regulates the discharge of dredged or fill material, dredging, and dredged material disposal in Waters of the U.S. within the Commonwealth. The proposed project work, including both the installation of new effluent discharge pipe and its associated ocean outfall, may exceed thresholds that will trigger the need for a WQC. These thresholds include:

- Any activity in an area subject to 310 CMR 10.00: Wetlands Protection which is also subject to 33 U.S.C. 1251, et seq. and will result in the loss of more than 5,000 square feet cumulatively of bordering or isolated vegetated wetlands and land under water, except for Ecological Restoration project not requiring a Water Quality Certification application pursuant to 314 CMR 9.03 (8). [314 CMR 9.04 (1)]; and
- Any activity resulting in the discharge of dredged or fill material in any salt marsh, except for an Ecological Restoration project not requiring a Water Quality Certification application pursuant to 314 CMR 9.03(8). [314 CMR 9.04 (8)]; and
- Any activity subject to an individual Section 404 permit by the Corps of Engineers, except for an Ecological Restoration Project not requiring a Water Quality Certification application pursuant to 314 CMR 9.03(8). [314 CMR 9.04(9)].
- Any dredging or dredged material re-use or disposal of 100 cubic yards or greater. (314 CMR 9.04 (12).

The exceedance of any of the above-mentioned thresholds will be dependent upon which project alternative is chosen. A 401 WQC takes approximately one month to prepare and can take up to 6 months for approval.

Chapter 91 Waterways License

The Massachusetts Waterways Regulations administer the provisions of MGL c. 91, the Massachusetts Public Waterfront Act. Chapter 91 preserves the rights of the public to have access to tidelands and waterways of the Commonwealth, and regulates activities on both coastal and inland waterways. The proposed ocean outfall will impact resources regulated through the Chapter 91 Waterways program, therefore the Town will be required to file for a Water Dependent Chapter 91 Waterways License. A Chapter 91 Waterways License can take approximately one month to prepare, and a minimum of 6 months for approval.

Massachusetts Office of Coastal Zone Management (CZM)

Coastal Zone Management Policy Guide

CZM is the policy and planning agency for coastal and ocean issues. Any project located within CZM jurisdiction which requires a federal permit or is considered a federal action requires federal consistency review with the CZM Policy Guide. This consistency review will occur during the USACE PCN or Individual

Permit process. In addition to CZM Policy Guide consistency review, coordination with the Massachusetts Board of Underwater Archaeological Resources (BUAR) is also required through this agency for the USACE process.

The Department of Conservation and Recreation (DCR)

Ocean Sanctuaries Act

The project proposes an ocean outfall within Aucoot Cove, which is part of the ocean area within the Cape and Islands Ocean Sanctuary regulated by the Ocean Sanctuaries Act (OSA). The OSA regulates any proposed work within the ocean and on the ocean floor of a designated ocean sanctuary. DCR does not issue any licenses or permits but acts through the regulatory process of other agencies. Ocean Sanctuaries staff comment on Massachusetts Environmental Policy Act (MEPA) filings and on MassDEP Chapter 91 license applications during the respective public comment periods.

The proposed project work triggers the need to comply with the OSA due to the installation of a new open ocean outfall that will discharge wastewater treatment plant effluent from the Marion Wastewater Treatment Plant.

According to the 2014 amendment to the Ocean Sanctuaries Act (Section 16G, Chapter 259 of Acts of 2014), a new or modified discharge may be approved to an ocean sanctuary only if clauses 1 through 10, inclusive, are met:

- The new or modified discharge shall be consistent with the intent and purpose of the act. Any discharge shall meet the water quality standards of the receiving water body and the standards of the act to protect the appearance, ecology and marine resources of the waters of the sanctuary.
- The new or modified discharge shall meet the United States Environmental Protection Agency's approved TMDL, if any, on the receiving water body.
- The applicant shall have adopted and implemented a plan approved by the department requiring the pretreatment of all commercial and industrial wastes discharged to the POTW.
- The applicant shall have adopted and implemented a program for water conservation according to the guidelines established by the water resources commission.
- The applicant shall have adopted and implemented a plan, approved by the department, to control and minimize inflow and infiltration.
- The applicant shall have adopted and implemented a plan, approved by the department, to control any combined sewer overflows.
- The new or modified discharge shall not significantly affect the quality or quantity of existing or proposed water supplies by reducing ground or surface water replenishment.
- The new or modified discharge is consistent with the policies and plans of the Massachusetts Coastal Zone Management Program.

- The new or modified discharge and treatment plans are consistent with all applicable federal, state and local laws, ordinances, by-laws, rules and regulations protecting the environment, including but not limited to, the requirements of chapters 21, 91, 130 and 131.
- The proposed discharge and outfall structure will not adversely impact marine fisheries or interfere with fishing grounds or the normal operation of fishing vessels.

In addition to meeting the requirements in clauses 1 through 10, new discharges in the Cape and Islands Ocean Sanctuary, the Cape Cod Ocean Sanctuary and the Cape Cod Bay Ocean Sanctuary shall receive advanced treatment, disinfection and such other treatment to remove nutrients, pathogens or other pollutants to avoid degradation of the ecology, appearance and marine resources of the designated sanctuary and to meet water quality standards and any applicable TMDLs. Chlorinated disinfection shall not occur unless it is followed by dechlorination prior to discharge.

Documentation for a new or modified discharge shall, at a minimum, include:

- A final Comprehensive Wastewater Management Plan (CWMP) approved by the department and a Final Environmental Impact Report and MEPA certificate;
- An evaluation of the receiving water body, including a benthic survey and fish habitat evaluation;
- A minimum of 24 months of baseline nutrient related water quality monitoring;
- Development of a site specific hydrodynamic model illustrating tides, bathymetry, mixing zones and seasonal variations; and
- A hydrologic evaluation of the aquifer, including evaluation of the effects of the new or modified discharge on the recharge of the affected aquifer.

Massachusetts Historical Commission

The Historic Preservation Act (the Act) requires that project areas be evaluated to determine the presence of cultural resources. Compliance with the Act, Section 106, and Chapter 254 is required prior to the start of construction. As a part of the MEPA process and USACE permitting process, the MHC will be contacted to determine if the project will affect any significant cultural or archaeological resources. This coordination can take approximately one month.

Tribal Historic Preservation Officers (THPO)

As a part of the MEPA process and USACE permitting the THPO's for the Wampanoag Tribe of Gay Head (Aquinnah), and Mashpee Wampanoag Tribe will be contacted to determine if the project will affect any significant cultural or archaeological resources. This coordination takes approximately one month.

Massachusetts Department of Transportation

Massachusetts Department of Transportation (MassDOT) regulates work within State-owned roadways. Mill Street (Route 6), is owned by MassDOT and work within this road will require the procurement of a

MassDOT Access permit to complete the proposed work. This coordination can take approximately one month to prepare and one to three months to obtain the permit.

Potential Local Permitting Requirements

Marion Conservation Commission

The Marion Conservation Commission regulates all proposed work (up to 5,000 sf of impacts to jurisdictional areas) within wetland resource areas subject to jurisdiction under the WPA. The effluent discharge pipe and associated ocean outfall work is anticipated to take place within the following regulated wetland resources areas: Land Under the Ocean, Coastal Beach, Coastal Bank, Salt Marsh, Land Containing Shellfish, Land Subject to Coastal Storm Flowage (LSCSF), Inland Bank, Bordering Vegetated Wetlands (BVW), and the 200-foot Riverfront Area. The proposed project will require the filing of a NOI with the Marion Conservation Commission. This permitting process takes two to three months.

Mattapoisett Conservation Commission

The Mattapoisett Conservation Commission regulates all proposed work (up to 5,000 sf of impacts to jurisdictional areas) within wetland resource areas subject to jurisdiction under the WPA. The effluent discharge pipe and associated ocean outfall work is anticipated to take place within the following regulated wetland resources areas: Land Under the Ocean, Coastal Beach, Coastal Bank, Land Containing Shellfish, and Land Subject to Coastal Storm Flowage (LSCSF). Depending upon the proposed alternative, joint filing of a NOI with both the Marion Conservation Commission and the Mattapoisett Conservation Commission may be required. This permitting process takes two to three months. Close coordination with the Conservation Commission is recommended due to the proposed project work adjacent to eelgrass beds.

The table provided on the following page outlines the potential permitting needs of the above described alternatives.

Summary

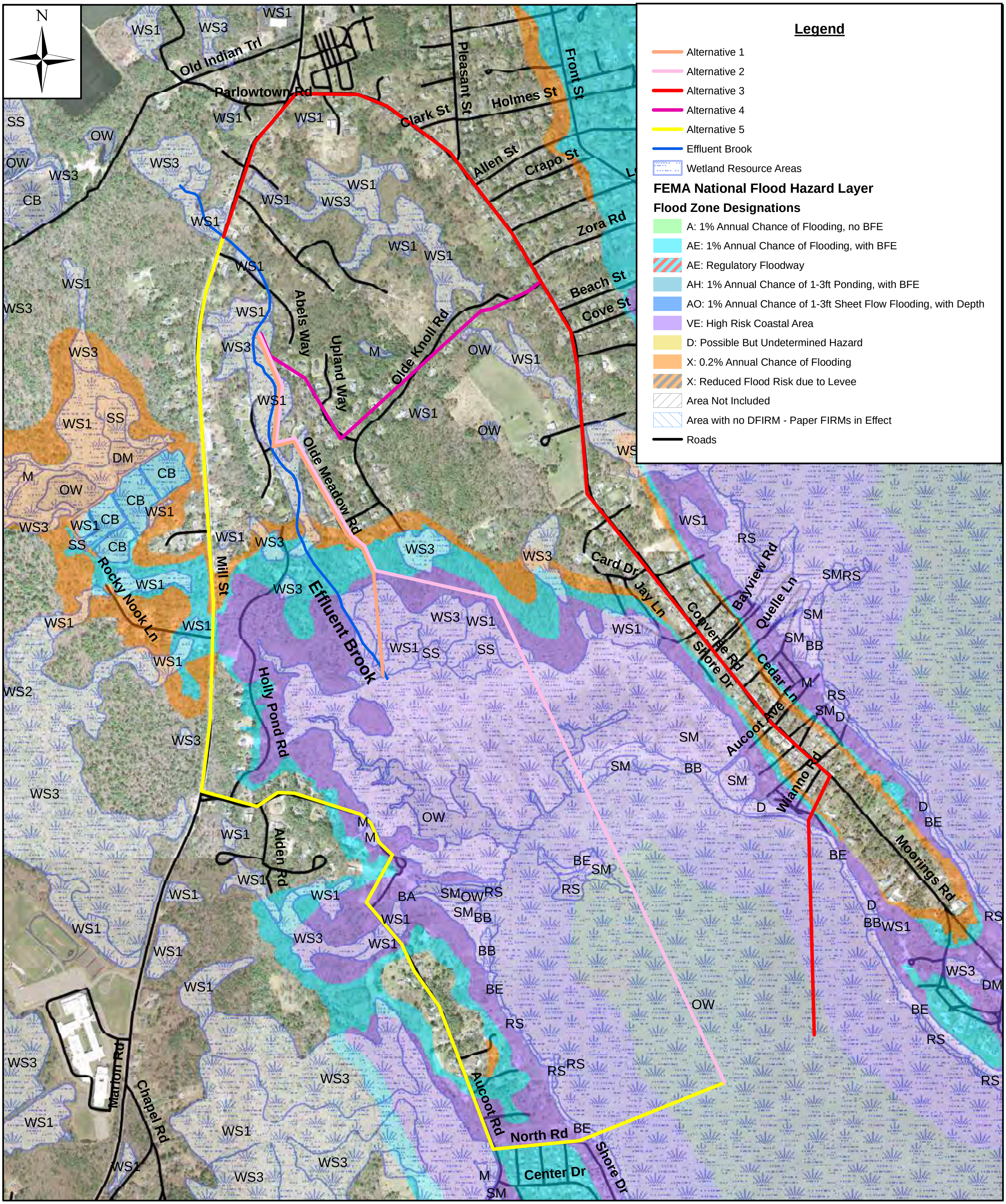
As outlined above, the project alternatives will require a number of environmental permits and approvals.

In general, Alternatives 1A & B, 2A & B, 3A, 4A, and 5A will require more permitting effort than Alternatives 3B, 4B, and 5B, as they will require a Variance from the WPA for impacts to resource areas exceeding the 5,000 sf threshold, as well as preparation of an Environmental Impact Report (EIR), which is necessary for any project that proposes a new open ocean outfall.

Alternatives 3B, 4B, and 5B are likely to require less permitting effort than the other Alternatives. These alternatives will minimize impacts to environmental resource areas by utilizing existing roadway rights of ways, and HDD for installation of the effluent discharge pipe and the open ocean outfall.

	Environmental Permitting Needs	Alts. 1A and 1B	Alts. 2A and 2B	Alt. 3A	Alts. 3B and 3C	Alt. 4A	Alt. 4B	Alt. 5A	Alt. 5B
Federal Approvals	USACE Pre-Construction Notification *(Approximately 3-4 Months)**	X			X		X		X
	USACE Individual Permit *(Approximately 9 Months to 1 Year)**		X	X		X		X	
	USFWS Agency Coordination *(Approximately 1-2 Months)**	X	X	X	X	X	X	X	X
	NPDES Construction General Permit (GP) *(Contractor Obtains)*	X	X	X	X	X	X	X	X
State Approvals	MEPA Certificate (ENF) *(Approximately 1 month)**	X	X	X	X	X	X	X	X
	MEPA Certificate (EIR) *(Approximately 1.5 Months)**	X	X	X	X	X	X	X	X
	Mass DMF *(Approximately 1-2 Months)**	X	X	X	X	X	X	X	X
	Natural Heritage and Endangered Species Program *(Approximately 1 Month)**	X	X	X	X	X	X	X	X
	MassDEP Variance from the WPA *(Approximately 1-2 Years)**	X	X	X		X		X	
	MassDEP 401 WQC *(Approximately 6 Months)**	X	X	X	X	X	X	X	X
	MassDEP Chapter 91 Waterways License *(Approximately 6 Months)**	X	X	X	X	X	X	X	X
	Massachusetts Historical Commission *(Approximately 1 Month)**	X	X	X	X	X	X	X	X
	Tribal Head Preservation Office's *(Approximately 1-2 Months)**	X	X	X	X	X	X	X	X
	MassDOT Highway Access Permit *(Approximately 1-3 Months)**			X	X			X	X
Local Approvals	Marion and/or Mattapoisett Conservation Commission Order of Conditions *(Approximately 2-3 Months)**	X	X	X	X	X	X	X	X

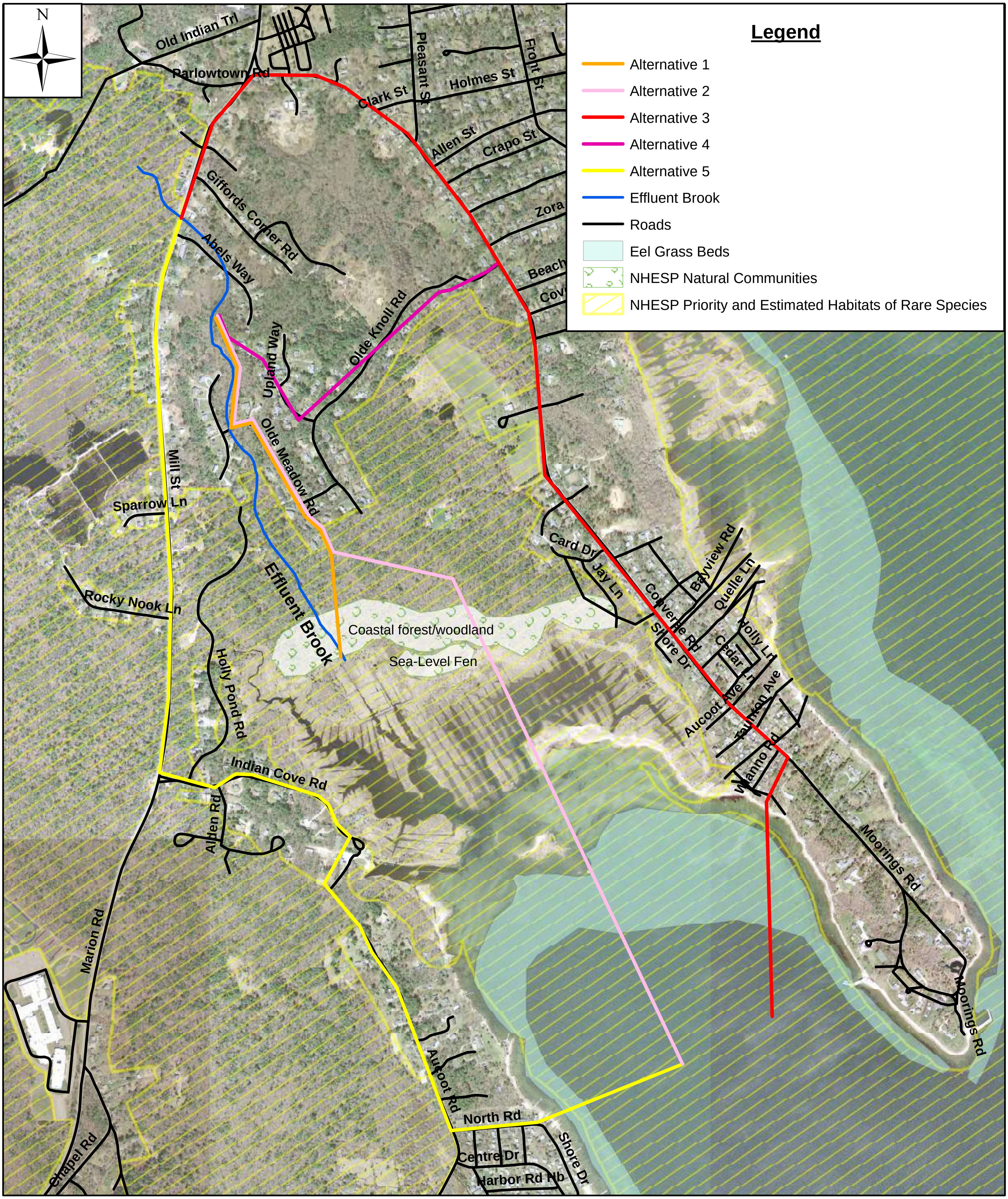
*Durations represent approximate agency approval times only.



Feet
0 250 500 1,000 1,500 2,000

Wetland Resource Areas
Marion New Effluent Discharge Pipe and Ocean Outfall
Town of Marion, MA

CDM Smith



Feet
0 245 490 980 1,470 1,960

Threatened and Endangered Species Habitat
Marion New Effluent Discharge Pipe and Ocean Outfall
Town of Marion, MA

CDM Smith

Appendix B

Site Visit Photos

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Photos from CDM Smith's Site Visit on November 19, 2014

18" RCP discharge into Effluent Brook.

Effluent Brook crossing beneath Olde Meadow Road.

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Effluent Brook through wooded area.

Effluent Brook at beginning of wetlands.

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Appendix C

Hydraulic Calculations

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Present Day Condition Calculations

Tailwater El: 15.0 (NAVD88)

Peak Flow: 1.18 mgd

MARION, MASS. - OUTFALL EXTENSION - HYDRAULIC GRADE LINE/PUMP STATION SIZINC
ALTERNATIVE 1A

Flow Rate	
1.18	*MGD*

100-year flood elevation:

Tailwater Elev
15

	US STA	DS STA	Forcemain (FM), Gravity (G), Or Gravity under Head (P)	Material	Starting Water Surface Elevation	Flow Rate	US Surface Elev	DS Surface Elev	Pipe Length	Pipe Diameter	Velocity	Manning's n Value	HGL Slope	Friction Losses	Gravity Direct Loss Input	Force Main Loss Coefficients				Minor Losses	Total Losses	U/S Water Surface Elevation	Freeboard
Calc ID:						Q	USSE		L	D	v	n	HGLS	FL		k				ML	TL	USWSEL	F
units:						*ft³/sec*			*ft*	*in*	*ft/sec*			*ft*	*ft*					*ft*			*ft*
GRAVITY	0	245	G	RC	18.33	1.83	28.00	28.00	245	18	1.03	0.013	0.0003	0.074	0.10					0.10	0.17	18.50	9.50
	245	490	G	RC	18.15	1.83	28.00	28.00	245	18	1.03	0.013	0.0003	0.074	0.10					0.10	0.17	18.33	9.67
	490	735	G	RC	17.98	1.83	28.00	27.00	245	18	1.03	0.013	0.0003	0.074	0.10					0.10	0.17	18.15	9.85
	735	985	G	RC	17.80	1.83	27.00	27.00	250	18	1.03	0.013	0.0003	0.076	0.10					0.10	0.18	17.98	9.02
	985	1235	G	RC	17.63	1.83	27.00	27.00	250	18	1.03	0.013	0.0003	0.076	0.10					0.10	0.18	17.80	9.20
	1235	1484	G	RC	17.45	1.83	27.00	26.00	249	18	1.03	0.013	0.0003	0.075	0.10					0.10	0.18	17.63	9.37
	1484	1695	G	RC	17.24	1.83	26.00	29.00	211	18	1.03	0.013	0.0003	0.064	0.15					0.15	0.21	17.45	8.55
	1695	1986	G	RC	17.00	1.83	29.00	27.00	291	18	1.03	0.013	0.0003	0.088	0.15					0.15	0.24	17.24	11.76
	1986	2277	G	RC	16.76	1.83	27.00	24.00	291	18	1.03	0.013	0.0003	0.088	0.15					0.15	0.24	17.00	10.00
	2277	2568	G	RC	16.52	1.83	24.00	21.00	291	18	1.03	0.013	0.0003	0.088	0.15					0.15	0.24	16.76	7.24
	2568	2859	G	RC	16.29	1.83	21.00	20.00	291	18	1.03	0.013	0.0003	0.088	0.15					0.15	0.24	16.52	4.48
	2859	3150	P	HDPE	16.16	1.83	20.00	16.00	291	18	1.03	-	-	0.116		0.60				0.01	0.13	16.29	3.71
	3150	3413	P	HDPE	16.04	1.83	16.00	15.00	263	18	1.03	-	-	0.105		0.60				0.01	0.11	16.16	-0.16
	3413	3599	P	HDPE	15.96	1.83	15.00	13.00	186	18	1.03	-	-	0.074		0.60				0.01	0.08	16.04	-1.04
	3599	3784	P	HDPE	15.88	1.83	13.00	10.00	185	18	1.03	-	-	0.074		0.60				0.01	0.08	15.96	-2.96
	3784	4052	P	HPDE	15.76	1.83	10.00	8.00	268	18	1.03	-	-	0.107		0.60				0.01	0.12	15.88	-5.88
	4052	4320	P	HPDE	15.64	1.83	8.00	6.00	268	18	1.03	-	-	0.107		0.60				0.01	0.12	15.76	-7.76
	4320	4588	P	HPDE	15.53	1.83	6.00	5.00	268	18	1.03	-	-	0.107		0.60				0.01	0.12	15.64	-9.64
	4588	4888	P	HPDE	15.40	1.83	5.00	5.00	300	18	1.03	-	-	0.120		0.60				0.01	0.13	15.53	-10.53
	4888	5188	P	HDPE	15.00	1.83	5.00	4.00	300	18	1.03	-	-	0.120	0.25	0.60	1.00			0.28	0.40	15.40	-10.40

Equations Used		
GENERAL	*GRAVITY*	*FORCEMAIN*
Area [A] = $\pi*(D/24)^2$	Hydraulic Radius [R_h] = $A/(D*\pi)$	FL = $(Q*60*7.4805)^{1.85}*10.44/(C^{1.85}*D^{4.865})$
v=Q/A	HGLS = $((Q*n)/(1.486*A*R_h^{2/3}))^2$	ML = $\Sigma k*v^2/64.4$
F = USSE-USWSEL	FL = L*HGLS	
	ML = direct entry of value	

Manning's n Values	
RC	0.013
HDPE	0.011

Hazen-Williams's Rougness Factor [C]	
HDPE	100

Pump Sizing	
U/S Static	
D/S Static	
Total Losses	
Total Dynamic Head	**N/A**

Note: Assumes full pipe flow for all gravity applications. Calculated depth of flow and velocity in 18" gravity pipe with slope =.005 at 1.2 MGD are 3.49 ft/sec and 6", respectively.

MARION, MASS. - OUTFALL EXTENSION - HYDRAULIC GRADE LINE/PUMP STATION SIZING
ALTERNATIVE 1B

Flow Rate	
1.18	*MGD*

100-year flood elevation:

Tailwater Elev
15

	US STA	DS STA	Forcemain (FM), Gravity (G), Or Gravity under Head (P)	Material	Starting Water Surface Elevation	Flow Rate	US Surface Elev	DS Surface Elev	Pipe Length	Pipe Diameter	Velocity	Manning's n Value	HGL Slope	Friction Losses	Gravity Direct Loss Input	Force Main Loss Coefficients				Minor Losses	Total Losses	U/S Water Surface Elevation	Freeboard
Calc ID:						Q	USSE		L	D	v	n	HGLS	FL		k				ML	TL	USWSEL	F
units:						*ft³/sec*			*ft*	*in*	*ft/sec*			*ft*	*ft*					*ft*			*ft*
GRAVITY	0	245	G	RC	18.29	1.83	28.00	28.00	245	18	1.03	0.013	0.0003	0.074	0.10					0.10	0.17	18.46	9.54
	245	490	G	RC	18.11	1.83	28.00	28.00	245	18	1.03	0.013	0.0003	0.074	0.10					0.10	0.17	18.29	9.71
	490	735	G	RC	17.94	1.83	28.00	27.00	245	18	1.03	0.013	0.0003	0.074	0.10					0.10	0.17	18.11	9.89
	735	985	G	RC	17.76	1.83	27.00	27.00	250	18	1.03	0.013	0.0003	0.076	0.10					0.10	0.18	17.94	9.06
	985	1235	G	RC	17.59	1.83	27.00	27.00	250	18	1.03	0.013	0.0003	0.076	0.10					0.10	0.18	17.76	9.24
	1235	1484	G	RC	17.41	1.83	27.00	26.00	249	18	1.03	0.013	0.0003	0.075	0.10					0.10	0.18	17.59	9.41
	1484	1695	G	RC	17.20	1.83	26.00	29.00	211	18	1.03	0.013	0.0003	0.064	0.15					0.15	0.21	17.41	8.59
	1695	1986	G	RC	16.96	1.83	29.00	27.00	291	18	1.03	0.013	0.0003	0.088	0.15					0.15	0.24	17.20	11.80
	1986	2277	G	RC	16.72	1.83	27.00	24.00	291	18	1.03	0.013	0.0003	0.088	0.15					0.15	0.24	16.96	10.04
	2277	2568	G	RC	16.48	1.83	24.00	21.00	291	18	1.03	0.013	0.0003	0.088	0.15					0.15	0.24	16.72	7.28
	2568	2859	G	RC	16.25	1.83	21.00	20.00	291	18	1.03	0.013	0.0003	0.088	0.15					0.15	0.24	16.48	4.52
	2859	3150	P	HDPE	16.12	1.83	20.00	16.00	291	18	1.03	-	-	0.116		0.60				0.01	0.13	16.25	3.75
	3150	3413	P	HDPE	16.01	1.83	16.00	15.00	263	18	1.03	-	-	0.105		0.60				0.01	0.11	16.12	-0.12
	3413	3599	P	HDPE	15.92	1.83	15.00	13.00	186	18	1.03	-	-	0.074		0.60				0.01	0.08	16.01	-1.01
	3599	3784	P	HDPE	15.84	1.83	13.00	10.00	185	18	1.03	-	-	0.074		0.60				0.01	0.08	15.92	-2.92
	3784	5189	P	HPDE	15.00	1.83	10.00	4.00	1405	18	1.03	-	-	0.561	0.25	0.60	1.00			0.28	0.84	15.84	-5.84

Equations Used		
GENERAL	*GRAVITY*	*FORCEMAIN*
Area [A] = π*(D/24)^2	Hydraulic Radius [R_h] =A/(D*π)	FL = (Q*60*7.4805)$^{1.85}$*10.44/(C$^{1.85}$*D$^{4.865}$)
v=Q/A	HGLS =((Q*n)/(1.486*A*$R_h^{2/3}$))^2	ML = Σk*v^2/64.4
F = USSE-USWSEL	FL = L*HGLS	
	ML = direct entry of value	

Manning's n Values	
RC	0.013
HDPE	0.011

Hazen-Williams's Rougness Factor [C]	
HDPE	100

Note: Assumes full pipe flow for all gravity applications. Calculated depth of flow and velocity in 18" gravity pipe with slope =.005 at 1.18 MGD are 3.49 ft/sec and 6", respectively.

Pump Sizing	
U/S Static	
D/S Static	
Total Losses	
Total Dynamic Head	**N/A**

MARION, MASS. - OUTFALL EXTENSION - HYDRAULIC GRADE LINE/PUMP STATION SIZING
ALTERNATIVES 2A & 2B

Flow Rate	
1.18	*MGD*

100-year flood elevation:

Tailwater Elev
15

	US STA	DS STA	Forcemain (FM), Gravity (G), Or Gravity under Head (P)	Material	Starting Water Surface Elevation	Flow Rate	US Surface Elev	DS Surface Elev	Pipe Length	Pipe Diameter	Velocity	Manning's n Value	HGL Slope	Friction Losses	Gravity Direct Loss Input	Force Main Loss Coefficients				Minor Losses	Total Losses	U/S Water Surface Elevation	Freeboard
Calc ID:						Q	USSE		L	D	v	n	HGLS	FL		k				ML	TL	USWSEL	F
units:						*ft^3/sec*			*ft*	*in*	*ft/sec*			*ft*	*ft*					*ft*			*ft*
GRAVITY	0	245	G	RC	24.99	1.83	28.00	28.00	245	18	1.03	0.013	0.0003	0.074	0.10					0.10	0.17	25.16	2.84
	245	490	G	RC	24.81	1.83	28.00	28.00	245	18	1.03	0.013	0.0003	0.074	0.10					0.10	0.17	24.99	3.01
	490	735	G	RC	24.64	1.83	28.00	27.00	245	18	1.03	0.013	0.0003	0.074	0.10					0.10	0.17	24.81	3.19
	735	985	G	RC	24.46	1.83	27.00	27.00	250	18	1.03	0.013	0.0003	0.076	0.10					0.10	0.18	24.64	2.36
	985	1235	G	RC	24.29	1.83	27.00	27.00	250	18	1.03	0.013	0.0003	0.076	0.10					0.10	0.18	24.46	2.54
	1235	1484	G	RC	24.11	1.83	27.00	26.00	249	18	1.03	0.013	0.0003	0.075	0.10					0.10	0.18	24.29	2.71
	1484	1695	G	RC	23.90	1.83	26.00	29.00	211	18	1.03	0.013	0.0003	0.064	0.15					0.15	0.21	24.11	1.89
	1695	1986	G	RC	23.66	1.83	29.00	27.00	291	18	1.03	0.013	0.0003	0.088	0.15					0.15	0.24	23.90	5.10
	1986	2277	G	RC	23.42	1.83	27.00	24.00	291	18	1.03	0.013	0.0003	0.088	0.15					0.15	0.24	23.66	3.34
	2277	2568	P	HDPE	23.30	1.83	24.00	21.00	291	18	1.03	-	-	0.116		0.60				0.01	0.13	23.42	0.58
	2568	2859	P	HDPE	23.17	1.83	21.00	20.00	291	18	1.03	-	-	0.116		0.60				0.01	0.13	23.30	-2.30
	2859	3150	P	HDPE	23.04	1.83	20.00	16.00	291	18	1.03	-	-	0.116		0.60				0.01	0.13	23.17	-3.17
	3150	3413	P	HDPE	22.93	1.83	16.00	15.00	263	18	1.03	-	-	0.105		0.60				0.01	0.11	23.04	-7.04
	3413	3599	P	HDPE	22.85	1.83	15.00	13.00	186	18	1.03	-	-	0.074		0.60				0.01	0.08	22.93	-7.93
	3599	3784	P	HDPE	22.76	1.83	13.00	10.00	185	18	1.03	-	-	0.074		0.60				0.01	0.08	22.85	-9.85
	3784	4076	P	HPDE	22.63	1.83	10.00	11.00	292	18	1.03	-	-	0.117		0.60				0.01	0.13	22.76	-12.76
	4076	4368	P	HPDE	22.51	1.83	11.00	10.00	292	18	1.03	-	-	0.117		0.60				0.01	0.13	22.63	-11.63
	4368	4660	P	HPDE	22.38	1.83	10.00	9.00	292	18	1.03	-	-	0.117		0.60				0.01	0.13	22.51	-12.51
	4660	4952	P	HPDE	22.26	1.83	9.00	9.00	292	18	1.03	-	-	0.117		0.60				0.01	0.13	22.38	-13.38
	4952	5244	P	HPDE	22.13	1.83	9.00	10.00	292	18	1.03	-	-	0.117		0.60				0.01	0.13	22.26	-13.26
	5244	5536	P	HPDE	22.00	1.83	10.00	12.00	292	18	1.03	-	-	0.117		0.60				0.01	0.13	22.13	-12.13
	5536	5828	P	HPDE	21.86	1.83	12.00	13.00	292	18	1.03	-	-	0.117		0.60	0.30	0.50		0.02	0.14	22.00	-10.00
	5828	12673	P	HDPE	15.00	1.83	13.00	-17.00	6845	16	1.31	-	-	4.847	2.00	0.60				2.02	6.86	21.86	-8.86

Equations Used

GENERAL

Area [A] = $\pi*(D/24)^2$

v=Q/A

F = USSE-USWSEL

GRAVITY

Hydraulic Radius $[R_h] = A/(D*\pi)$

HGLS $=((Q*n)/(1.486*A*R_h^{2/3}))^2$

FL = L*HGLS

ML = direct entry of value

FORCEMAIN

FL = $(Q*60*7.4805)^{1.85}*10.44/(C^{1.85}*D^{4.865})$

ML = $\Sigma k*v^2/64.4$

Manning's n Values	
RC	0.013
HDPE	0.011

Hazen-Williams's Rougness Factor [C]	
HDPE	100

Note: Assumes full pipe flow for all gravity applications. Calculated depth of flow and velocity in 18" gravity pipe with slope =.005 at 1.2 MGD are 3.49 ft/sec and 6", respectively.

Pump Sizing	
U/S Static	
D/S Static	
Total Losses	
Total Dynamic Head	**N/A**

MARION, MASS. - OUTFALL EXTENSION - HYDRAULIC GRADE LINE/PUMP STATION SIZING
ALTERNATIVES 3A, 3B, AND 3C

Flow Rate	
1.18	*MGD*

100-year flood elevation:	Tailwater Elev
	15

	US STA	DS STA	Forcemain (FM), Gravity (G), Or Gravity under Head (P)	Material	Starting Water Surface Elevation	Flow Rate	US Surface Elev	DS Surface Elev	Pipe Length	Pipe Diameter	Velocity	Manning's n Value	HGL Slope	Friction Losses	Gravity Direct Loss Input	Force Main Loss Coefficients				Minor Losses	Total Losses	U/S Water Surface Elevation	Freeboard
Calc ID:						Q	USSE		L	D	v	n	HGLS	FL		k				ML	TL	USWSEL	F
units:						*ft³/sec*			*ft*	*in*	*ft/sec*			*ft*	*ft*					*ft*			*ft*
FROM PUMP STATION	0	1330	FM	HDPE		1.83			1330	16	1.31	-	-	0.942		0.60				0.02	0.96		
	1330	1554	FM	HDPE		1.83			224	16	1.31	-	-	0.159		0.60				0.02	0.17		
	1554	2178	FM	HDPE		1.83			624	16	1.31	-	-	0.442		0.60				0.02	0.46		
	2178	2984	FM	HDPE		1.83			806	16	1.31	-	-	0.571		0.60				0.02	0.59		
	2984	3408	FM	HDPE		1.83			424	16	1.31	-	-	0.300		0.60				0.02	0.32		
	3408	4457	FM	HDPE		1.83			1049	16	1.31	-	-	0.743		0.60				0.02	0.76		
	4457	4760	FM	HDPE		1.83			303	16	1.31	-	-	0.215		0.60				0.02	0.23		
	4760	5800	FM	HDPE		1.83			1040	16	1.31	-	-	0.736		0.60				0.02	0.75		
	5800	7278	FM	HDPE		1.83			1478	16	1.31	-	-	1.047		0.60				0.02	1.06		
	7278	7810	FM	HDPE		1.83			532	16	1.31	-	-	0.377		0.60				0.02	0.39		
	7810	9460	FM	HDPE		1.83			1650	16	1.31	-	-	1.168		0.60				0.02	1.18		
	9460	10054	FM	HDPE		1.83			594	16	1.31	-	-	0.421		0.60				0.02	0.44		
	10054	13269	FM	HDPE		1.83			3215	16	1.31	-	-	2.276		0.60				0.02	2.29		
	13269	14365	FM	HDPE		1.83			1096	16	1.31	-	-	0.776		0.60	0.30	0.50		0.04	0.81		
	14365	17851	FM	HDPE		1.83			3486	16	1.31	-	-	2.468	2.00	0.60				2.02	4.48		

Equations Used

GENERAL

Area [A] = $\pi*(D/24)^2$

v=Q/A

F = USSE-USWSEL

GRAVITY

Hydraulic Radius [R_h] =$A/(D*\pi)$

HGLS =$((Q*n)/(1.486*A*R_h^{2/3}))^2$

FL = L*HGLS

ML = direct entry of value

FORCEMAIN

FL = $(Q*60*7.4805)^{1.85}*10.44/(C^{1.85}*D^{4.865})$

ML = $\Sigma k*v^2/64.4$

Manning's n Values	
RC	0.013
HDPE	0.011

Hazen-Williams's Rougness Factor [C]	
HDPE	100

Pump Sizing	
U/S Static	28
D/S Static	45
Total Losses	14.90
Total Dynamic Head	**31.90**

MARION, MASS. - OUTFALL EXTENSION - HYDRAULIC GRADE LINE/PUMP STATION SIZING
ALTERNATIVE 4A AND 4B

Flow Rate	
1.18	MGD

Tailwater Elev	
100-year flood elevation:	15

	US STA	DS STA	Forcemain (FM), Gravity (G), Or Gravity under Head (P)	Material	Starting Water Surface Elevation	Flow Rate	US Surface Elev	DS Surface Elev	Pipe Length	Pipe Diameter	Velocity	Manning's n Value	HGL Slope	Friction Losses	Gravity Direct Loss Input	Force Main Loss Coefficients				Minor Losses	Total Losses	U/S Water Surface Elevation	Freeboard
Calc ID:						Q	USSE		L	D	v	n	HGLS	FL		k				ML	TL	USWSEL	F
units:						ft^3/sec			ft	in	ft/sec			ft	ft					ft			ft
GRAVITY	0	316	G	RC	27.83	1.83	28.00	28.00	316	18	1.03	0.013	0.0003	0.095	0.10					0.10	0.20	28.03	-0.03
	316	563	G	RC	27.66	1.83	28.00	32.00	247	18	1.03	0.013	0.0003	0.075	0.10					0.10	0.17	27.83	0.17
	563	786	G	RC	27.49	1.83	32.00	30.00	223	18	1.03	0.013	0.0003	0.067	0.10					0.10	0.17	27.66	4.34
	786	1044	G	RC	27.31	1.83	30.00	34.00	258	18	1.03	0.013	0.0003	0.078	0.10					0.10	0.18	27.49	2.51
	1044	1275	G	RC	27.14	1.83	34.00	32.00	231	18	1.03	0.013	0.0003	0.070	0.10					0.10	0.17	27.31	6.69
	1275	1636	G	RC	26.93	1.83	32.00	30.00	361	18	1.03	0.013	0.0003	0.109	0.10	0.30				0.10	0.21	27.14	4.86
	1636	1936	G	RC	26.74	1.83	30.00	32.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	26.93	3.07
	1936	2236	G	RC	26.55	1.83	32.00	35.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	26.74	5.26
	2236	2536	G	RC	26.35	1.83	35.00	38.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	26.55	8.45
	2536	2836	G	RC	26.16	1.83	38.00	38.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	26.35	11.65
	2836	3136	G	RC	25.97	1.83	38.00	38.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	26.16	11.84
	3136	3436	G	RC	25.78	1.83	38.00	35.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	25.97	12.03
	3436	3736	G	RC	25.59	1.83	35.00	34.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	25.78	9.22
	3736	4036	G	RC	25.40	1.83	34.00	35.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	25.59	8.41
	4036	4134	G	RC	25.27	1.83	35.00	35.00	98	18	1.03	0.013	0.0003	0.030	0.10					0.10	0.13	25.40	9.60
	4134	4299	G	RC	25.12	1.83	35.00	34.00	165	18	1.03	0.013	0.0003	0.050	0.10	0.30				0.10	0.15	25.27	9.73
	4299	4575	G	RC	24.93	1.83	34.00	33.00	276	18	1.03	0.013	0.0003	0.083	0.10					0.10	0.18	25.12	8.88
	4575	4836	G	RC	24.75	1.83	33.00	36.00	261	18	1.03	0.013	0.0003	0.079	0.10					0.10	0.18	24.93	8.07
	4836	5021	G	RC	24.60	1.83	36.00	38.00	185	18	1.03	0.013	0.0003	0.056	0.10					0.10	0.16	24.75	11.25
	5021	5321	G	RC	24.41	1.83	38.00	38.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	24.60	13.40
	5321	5621	G	RC	24.22	1.83	38.00	37.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	24.41	13.59
	5621	5801	G	RC	24.06	1.83	37.00	35.00	180	18	1.03	0.013	0.0003	0.054	0.10					0.10	0.15	24.22	12.78
	5801	6049	G	RC	23.89	1.83	35.00	33.00	248	18	1.03	0.013	0.0003	0.075	0.10					0.10	0.17	24.06	10.94
	6049	6270	G	RC	23.72	1.83	33.00	30.00	221	18	1.03	0.013	0.0003	0.067	0.10					0.10	0.17	23.89	9.11
	6270	6570	G	RC	23.53	1.83	30.00	26.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	23.72	6.28
	6570	6870	G	RC	23.34	1.83	26.00	25.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	23.53	2.47
	6870	7170	G	RC	23.15	1.83	25.00	28.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	23.34	1.66
	7170	7470	G	RC	22.96	1.83	28.00	30.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	23.15	4.85
	7470	7770	G	RC	22.77	1.83	30.00	35.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	22.96	7.04
	7770	7978	G	RC	22.61	1.83	35.00	35.00	208	18	1.03	0.013	0.0003	0.063	0.10					0.10	0.16	22.77	12.23
	7978	8278	G	RC	22.41	1.83	35.00	35.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	22.61	12.39
	8278	8578	G	RC	22.22	1.83	35.00	35.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	22.41	12.59
	8578	8878	G	RC	22.03	1.83	35.00	35.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	22.22	12.78
	8878	9178	G	RC	21.84	1.83	35.00	33.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	22.03	12.97
	9178	9478	G	RC	21.65	1.83	33.00	31.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	21.84	11.16
	9478	9778	G	RC	21.46	1.83	31.00	28.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	21.65	9.35
	9778	10078	G	RC	21.27	1.83	28.00	26.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	21.46	6.54
	10078	10378	G	RC	21.08	1.83	26.00	25.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	21.27	4.73
	10378	10678	G	RC	20.89	1.83	25.00	24.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	21.08	3.92
	10678	10978	G	RC	20.70	1.83	24.00	22.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	20.89	3.11
	10978	11278	G	RC	20.51	1.83	22.00	20.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	20.70	1.30
	11278	11578	G	RC	20.32	1.83	20.00	20.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	20.51	-0.51
	11578	11878	G	RC	20.13	1.83	20.00	23.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	20.32	-0.32
	11878	12178	G	RC	19.94	1.83	23.00	24.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	20.13	2.87
	12178	12478	G	RC	19.75	1.83	24.00	25.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	19.94	4.06
	12478	12778	G	RC	19.56	1.83	25.00	27.00	300	18	1.03	0.013	0.0003	0.091	0.10					0.10	0.19	19.75	5.25
	12778	12905	P	HDPE	19.48	1.83	27.00	30.00	127	18	1.03	-	-	0.051		0.60	0.30	0.50		0.02	0.07	19.56	7.44
	12905	16391	P	HDPE	15.00	1.83	30.00	-17.00	3486	16	1.31	-	-	2.468	2.00	0.50				2.01	4.48	19.48	10.52

Equations Used		
GENERAL	*GRAVITY*	*FORCEMAIN*
Area [A] = $\pi*(D/24)^2$	Hydraulic Radius [R_h] =$A/(D*\pi)$	FL = $(Q*60*7.4805)^{1.85}*10.44/(C^{1.85}*D^{4.865})$
v=Q/A	HGLS =$((Q*n)/(1.486*A*R_h^{2/3}))^2$	ML = $\Sigma k*v^2/64.4$
F = USSE-USWSEL	FL = L*HGLS	
	ML = direct entry of value	

Manning's n Values	
RC	0.013
HDPE	0.011

Hazen-Williams's Rougness Factor [C]	
HDPE	100

Pump Sizing	
U/S Static	
D/S Static	
Total Losses	
Total Dynamic Head	**N/A**

Note: Assumes full pipe flow for all gravity applications. Calculated depth of flow and velocity in 18" gravity pipe with slope =.005 at 1.2 MGD are 3.49 ft/sec and 6", respectively.

MARION, MASS. - OUTFALL EXTENSION - HYDRAULIC GRADE LINE/PUMP STATION SIZINC
ALTERNATIVES 5A AND 5B

Flow Rate	
1.18	*MGD*

Tailwater Elev	
100-year flood elevation:	15

	US STA	DS STA	Forcemain (FM), Gravity (G), Or Gravity under Head (P)	Material	Starting Water Surface Elevation	Flow Rate	US Surface Elev	DS Surface Elev	Pipe Length	Pipe Diameter	Velocity	Manning's n Value	HGL Slope	Friction Losses	Gravity Direct Loss Input	Force Main Loss Coefficients				Minor Losses	Total Losses	U/S Water Surface Elevation	Freeboard
Calc ID:						Q	USSE		L	D	v	n	HGLS	FL		k				ML	TL	USWSEL	F
units:						*ft^3/sec*			*ft*	*in*	*ft/sec*			*ft*	*ft*					*ft*			*ft*
FROM PUMP STATION	0	1296	FM	HDPE		1.83			1296	16	1.31	-	-	0.918		0.60				0.02	0.93		
	1296	2659	FM	HDPE		1.83			1363	16	1.31	-	-	0.965		0.60				0.02	0.98		
	2659	3569	FM	HDPE		1.83			910	16	1.31	-	-	0.644		0.60				0.02	0.66		
	3569	4159	FM	HDPE		1.83			590	16	1.31	-	-	0.418		0.60				0.02	0.43		
	4159	5297	FM	HDPE		1.83			1138	16	1.31	-	-	0.806		0.60				0.02	0.82		
	5297	6107	FM	HDPE		1.83			810	16	1.31	-	-	0.574		0.60				0.02	0.59		
	6107	7738	FM	HDPE		1.83			1631	16	1.31	-	-	1.155		0.60				0.02	1.17		
	7738	8443	FM	HDPE		1.83			705	16	1.31	-	-	0.499		0.60				0.02	0.52		
	8443	8855	FM	HDPE		1.83			412	16	1.31	-	-	0.292		0.60				0.02	0.31		
	8855	9908	FM	HDPE		1.83			1053	16	1.31	-	-	0.746		0.60				0.02	0.76		
	9908	10088	FM	HDPE		1.83			180	16	1.31	-	-	0.127		0.60	0.30			0.02	0.15		
	10088	10376	FM	HDPE		1.83			288	16	1.31	-	-	0.204		0.60	0.15			0.02	0.22		
	10376	10619	FM	HDPE		1.83			243	16	1.31	-	-	0.172		0.60	0.15			0.02	0.19		
	10619	11341	FM	HDPE		1.83			722	16	1.31	-	-	0.511		0.60	0.15			0.02	0.53		
	11341	12085	FM	HDPE		1.83			744	16	1.31	-	-	0.527		0.60	0.30			0.02	0.55		
	12085	12464	FM	HDPE		1.83			379	16	1.31	-	-	0.268		0.60				0.02	0.28		
	12464	13037	FM	HDPE		1.83			573	16	1.31	-	-	0.406		0.60				0.02	0.42		
	13037	14705	FM	HDPE		1.83			1668	16	1.31	-	-	1.181		0.60				0.02	1.20		
	14705	16200	FM	HDPE		1.83			1495	16	1.31	-	-	1.059		0.60	0.30			0.02	1.08		
	16200	18278	FM	HDPE		1.83			2078	16	1.31	-	-	1.471	2.00	0.60				2.02	3.49		

Equations Used

GENERAL	*GRAVITY*	*FORCEMAIN*
Area [A] = π*(D/24)^2	Hydraulic Radius [R_h] =A/(D*π)	FL = (Q*60*7.4805)$^{1.85}$*10.44/(C$^{1.85}$*D$^{4.865}$)
v=Q/A	HGLS =((Q*n)/(1.486*A*$R_h^{2/3}$))^2	ML = Σk*v^2/64.4
F = USSE-USWSEL	FL = L*HGLS	
	ML = direct entry of value	

Manning's n Values	
RC	0.013
HDPE	0.011

Hazen-Williams's Rougness Factor [C]	
HDPE	100

Pump Sizing	
U/S Static	28
D/S Static	20
Total Losses	15.30
Total Dynamic Head	**7.30**

Future Condition Calculations

Tailwater El: 21.9 (NAVD88)

Peak Flow: 1.3 mgd

MARION, MASS. - OUTFALL EXTENSION - HYDRAULIC GRADE LINE/PUMP STATION SIZING
ALTERNATIVE 1A

Flow Rate	
1.3	*MGD*

100-year flood elevation:	Tailwater Elev
	21.9

	US STA	DS STA	Forcemain (FM), Gravity (G), Or Gravity under Head (P)	Material	Starting Water Surface Elevation	Flow Rate	US Surface Elev	DS Surface Elev	Pipe Length	Pipe Diameter	Velocity	Manning's n Value	HGL Slope	Friction Losses	Gravity Direct Loss Input	Force Main Loss Coefficients				Minor Losses	Total Losses	U/S Water Surface Elevation	Freeboard
Calc ID:						Q	USSE		L	D	v	n	HGLS	FL		k				ML	TL	USWSEL	F
units:						*ft^3/sec*			*ft*	*in*	*ft/sec*			*ft*	*ft*					*ft*			*ft*
GRAVITY	0	245	G	RC	25.39	2.01	28.00	28.00	245	18	1.14	0.013	0.0004	0.090	0.10					0.10	0.19	25.58	2.42
	245	490	G	RC	25.20	2.01	28.00	28.00	245	18	1.14	0.013	0.0004	0.090	0.10					0.10	0.19	25.39	2.61
	490	735	G	RC	25.01	2.01	28.00	27.00	245	18	1.14	0.013	0.0004	0.090	0.10					0.10	0.19	25.20	2.80
	735	985	G	RC	24.82	2.01	27.00	27.00	250	18	1.14	0.013	0.0004	0.092	0.10					0.10	0.19	25.01	1.99
	985	1235	G	RC	24.63	2.01	27.00	27.00	250	18	1.14	0.013	0.0004	0.092	0.10					0.10	0.19	24.82	2.18
	1235	1484	G	RC	24.43	2.01	27.00	26.00	249	18	1.14	0.013	0.0004	0.091	0.10					0.10	0.19	24.63	2.37
	1484	1695	G	RC	24.21	2.01	26.00	29.00	211	18	1.14	0.013	0.0004	0.077	0.15					0.15	0.23	24.43	1.57
	1695	1986	G	RC	23.95	2.01	29.00	27.00	291	18	1.14	0.013	0.0004	0.107	0.15					0.15	0.26	24.21	4.79
	1986	2277	G	RC	23.69	2.01	27.00	24.00	291	18	1.14	0.013	0.0004	0.107	0.15					0.15	0.26	23.95	3.05
	2277	2568	P	HDPE	23.54	2.01	24.00	21.00	291	18	1.14	-	-	0.139		0.60				0.01	0.15	23.69	0.31
	2568	2859	P	HDPE	23.39	2.01	21.00	20.00	291	18	1.14	-	-	0.139		0.60				0.01	0.15	23.54	-2.54
	2859	3150	P	HDPE	23.24	2.01	20.00	16.00	291	18	1.14	-	-	0.139		0.60				0.01	0.15	23.39	-3.39
	3150	3413	P	HDPE	23.10	2.01	16.00	15.00	263	18	1.14	-	-	0.126		0.60				0.01	0.14	23.24	-7.24
	3413	3599	P	HDPE	23.00	2.01	15.00	13.00	186	18	1.14	-	-	0.089		0.60				0.01	0.10	23.10	-8.10
	3599	3784	P	HDPE	22.90	2.01	13.00	10.00	185	18	1.14	-	-	0.088		0.60				0.01	0.10	23.00	-10.00
	3784	4052	P	HPDE	22.76	2.01	10.00	8.00	268	18	1.14	-	-	0.128		0.60				0.01	0.14	22.90	-12.90
	4052	4320	P	HPDE	22.62	2.01	8.00	6.00	268	18	1.14	-	-	0.128		0.60				0.01	0.14	22.76	-14.76
	4320	4588	P	HPDE	22.48	2.01	6.00	5.00	268	18	1.14	-	-	0.128		0.60				0.01	0.14	22.62	-16.62
	4588	4888	P	HPDE	22.33	2.01	5.00	5.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	22.48	-17.48
	4888	5188	P	HDPE	21.90	2.01	5.00	4.00	300	18	1.14	-	-	0.143	0.25	0.60	1.00			0.28	0.43	22.33	-17.33

Equations Used		
GENERAL	*GRAVITY*	*FORCEMAIN*
Area [A] = π*(D/24)^2	Hydraulic Radius [R_h] =A/(D*π)	FL = (Q*60*7.4805)$^{1.85}$*10.44/(C$^{1.85}$*D$^{4.865}$)
v=Q/A	HGLS =((Q*n)/(1.486*A*$R_h^{2/3}$))^2	ML = Σk*v^2/64.4
F = USSE-USWSEL	FL = L*HGLS	
	ML = direct entry of value	

Manning's n Values	
RC	0.013
HDPE	0.011

Hazen-Williams's Rougness Factor [C]	
HDPE	100

Pump Sizing	
U/S Static	
D/S Static	
Total Losses	
Total Dynamic Head	**N/A**

Note: Assumes full pipe flow for all gravity applications. Calculated depth of flow and velocity in 18" gravity pipe with slope =.005 at 1.2 MGD are 3.49 ft/sec and 6", respectively.

MARION, MASS. - OUTFALL EXTENSION - HYDRAULIC GRADE LINE/PUMP STATION SIZING
ALTERNATIVE 1B

Flow Rate	
1.3	*MGD*

100-year flood elevation:

Tailwater Elev
21.9

	US STA	DS STA	Forcemain (FM), Gravity (G), Or Gravity under Head (P)	Material	Starting Water Surface Elevation	Flow Rate	US Surface Elev	DS Surface Elev	Pipe Length	Pipe Diameter	Velocity	Manning's n Value	HGL Slope	Friction Losses	Gravity Direct Loss Input	Force Main Loss Coefficients				Minor Losses	Total Losses	U/S Water Surface Elevation	Freeboard
Calc ID:						Q	USSE		L	D	v	n	HGLS	FL		k				ML	TL	USWSEL	F
units:						*ft^3/sec*			*ft*	*in*	*ft/sec*			*ft*	*ft*					*ft*			*ft*
GRAVITY	0	245	G	RC	25.34	2.01	28.00	28.00	245	18	1.14	0.013	0.0004	0.090	0.10					0.10	0.19	25.53	2.47
	245	490	G	RC	25.15	2.01	28.00	28.00	245	18	1.14	0.013	0.0004	0.090	0.10					0.10	0.19	25.34	2.66
	490	735	G	RC	24.96	2.01	28.00	27.00	245	18	1.14	0.013	0.0004	0.090	0.10					0.10	0.19	25.15	2.85
	735	985	G	RC	24.77	2.01	27.00	27.00	250	18	1.14	0.013	0.0004	0.092	0.10					0.10	0.19	24.96	2.04
	985	1235	G	RC	24.58	2.01	27.00	27.00	250	18	1.14	0.013	0.0004	0.092	0.10					0.10	0.19	24.77	2.23
	1235	1484	G	RC	24.39	2.01	27.00	26.00	249	18	1.14	0.013	0.0004	0.091	0.10					0.10	0.19	24.58	2.42
	1484	1695	G	RC	24.16	2.01	26.00	29.00	211	18	1.14	0.013	0.0004	0.077	0.15					0.15	0.23	24.39	1.61
	1695	1986	G	RC	23.90	2.01	29.00	27.00	291	18	1.14	0.013	0.0004	0.107	0.15					0.15	0.26	24.16	4.84
	1986	2277	G	RC	23.65	2.01	27.00	24.00	291	18	1.14	0.013	0.0004	0.107	0.15					0.15	0.26	23.90	3.10
	2277	2568	P	HDPE	23.49	2.01	24.00	21.00	291	18	1.14	-	-	0.139		0.60				0.01	0.15	23.65	0.35
	2568	2859	P	HDPE	23.34	2.01	21.00	20.00	291	18	1.14	-	-	0.139		0.60				0.01	0.15	23.49	-2.49
	2859	3150	P	HDPE	23.19	2.01	20.00	16.00	291	18	1.14	-	-	0.139		0.60				0.01	0.15	23.34	-3.34
	3150	3413	P	HDPE	23.05	2.01	16.00	15.00	263	18	1.14	-	-	0.126		0.60				0.01	0.14	23.19	-7.19
	3413	3599	P	HDPE	22.95	2.01	15.00	13.00	186	18	1.14	-	-	0.089		0.60				0.01	0.10	23.05	-8.05
	3599	3784	P	HDPE	22.85	2.01	13.00	10.00	185	18	1.14	-	-	0.088		0.60				0.01	0.10	22.95	-9.95
	3784	5189	P	HPDE	21.90	2.01	10.00	21.90	1405	18	1.14	-	-	0.671	0.25	0.60	1.00			0.28	0.95	22.85	-12.85

Equations Used		
GENERAL	*GRAVITY*	*FORCEMAIN*
Area [A] = π*(D/24)^2	Hydraulic Radius [R_h] =A/(D*π)	FL = (Q*60*7.4805)$^{1.85}$*10.44/($C^{1.85}$*$D^{4.865}$)
v=Q/A	HGLS =((Q*n)/(1.486*A*$R_h^{2/3}$))^2	ML = Σk*v^2/64.4
F = USSE-USWSEL	FL = L*HGLS	
	ML = direct entry of value	

Manning's n Values	
RC	0.013
HDPE	0.011

Hazen-Williams's Rougness Factor [C]	
HDPE	100

Pump Sizing	
U/S Static	
D/S Static	
Total Losses	
Total Dynamic Head	**N/A**

Note: Assumes full pipe flow for all gravity applications. Calculated depth of flow and velocity in 18" gravity pipe with slope =.005 at 1.18 MGD are 3.49 ft/sec and 6", respectively.

MARION, MASS. - OUTFALL EXTENSION - HYDRAULIC GRADE LINE/PUMP STATION SIZING
ALTERNATIVES 2A & 2B

Flow Rate	
1.3	*MGD*

100-year flood elevation:

Tailwater Elev
21.9

	US STA	DS STA	Forcemain (FM), Gravity (G), Or Gravity under Head (P)	Material	Starting Water Surface Elevation	Flow Rate	US Surface Elev	DS Surface Elev	Pipe Length	Pipe Diameter	Velocity	Manning's n Value	HGL Slope	Friction Losses	Gravity Direct Loss Input	Force Main Loss Coefficients				Minor Losses	Total Losses	U/S Water Surface Elevation	Freeboard
Calc ID:						Q	USSE		L	D	v	n	HGLS	FL		k				ML	TL	USWSEL	F
units:						*ft³/sec*			*ft*	*in*	*ft/sec*			*ft*	*ft*					*ft*			*ft*
	U/S of Rte 6		P	HDPE	33.99	2.01	40.00	35.00	3300	18	1.14	-	-	1.576		0.60				0.01	1.59	35.57	4.43
			P	HDPE	32.78	2.01	35.00	28.00	1400	16	1.44	-	-	1.186		0.60				0.02	1.21	33.99	1.01
GRAVITY	0	245	P	HDPE	32.65	2.01	28.00	28.00	245	18	1.14	-	-	0.117		0.60				0.01	0.13	32.78	-4.78
	245	490	P	HDPE	32.52	2.01	28.00	28.00	245	18	1.14	-	-	0.117		0.60				0.01	0.13	32.65	-4.65
	490	735	P	HDPE	32.39	2.01	28.00	27.00	245	18	1.14	-	-	0.117		0.60				0.01	0.13	32.52	-4.52
	735	985	P	HDPE	32.26	2.01	27.00	27.00	250	18	1.14	-	-	0.119		0.60				0.01	0.13	32.39	-5.39
	985	1235	P	HDPE	32.13	2.01	27.00	27.00	250	18	1.14	-	-	0.119		0.60				0.01	0.13	32.26	-5.26
	1235	1484	P	HDPE	32.00	2.01	27.00	26.00	249	18	1.14	-	-	0.119		0.60				0.01	0.13	32.13	-5.13
	1484	1695	P	HDPE	31.89	2.01	26.00	29.00	211	18	1.14	-	-	0.101		0.60				0.01	0.11	32.00	-6.00
	1695	1986	P	HDPE	31.74	2.01	29.00	27.00	291	18	1.14	-	-	0.139		0.60				0.01	0.15	31.89	-2.89
	1986	2277	P	HDPE	31.59	2.01	27.00	24.00	291	18	1.14	-	-	0.139		0.60				0.01	0.15	31.74	-4.74
	2277	2568	P	HDPE	31.43	2.01	24.00	21.00	291	18	1.14	-	-	0.139		0.60				0.01	0.15	31.59	-7.59
	2568	2859	P	HDPE	31.28	2.01	21.00	20.00	291	18	1.14	-	-	0.139		0.60				0.01	0.15	31.43	-10.43
	2859	3150	P	HDPE	31.13	2.01	20.00	16.00	291	18	1.14	-	-	0.139		0.60				0.01	0.15	31.28	-11.28
	3150	3413	P	HDPE	30.99	2.01	16.00	15.00	263	18	1.14	-	-	0.126		0.60				0.01	0.14	31.13	-15.13
	3413	3599	P	HDPE	30.89	2.01	15.00	13.00	186	18	1.14	-	-	0.089		0.60				0.01	0.10	30.99	-15.99
	3599	3784	P	HDPE	30.79	2.01	13.00	10.00	185	18	1.14	-	-	0.088		0.60				0.01	0.10	30.89	-17.89
	3784	4076	P	HPDE	30.64	2.01	10.00	11.00	292	18	1.14	-	-	0.139		0.60				0.01	0.15	30.79	-20.79
	4076	4368	P	HPDE	30.49	2.01	11.00	10.00	292	18	1.14	-	-	0.139		0.60				0.01	0.15	30.64	-19.64
	4368	4660	P	HPDE	30.34	2.01	10.00	9.00	292	18	1.14	-	-	0.139		0.60				0.01	0.15	30.49	-20.49
	4660	4952	P	HPDE	30.19	2.01	9.00	9.00	292	18	1.14	-	-	0.139		0.60				0.01	0.15	30.34	-21.34
	4952	5244	P	HPDE	30.04	2.01	9.00	10.00	292	18	1.14	-	-	0.139		0.60				0.01	0.15	30.19	-21.19
	5244	5536	P	HPDE	29.88	2.01	10.00	12.00	292	18	1.14	-	-	0.139		0.60				0.01	0.15	30.04	-20.04
	5536	5828	P	HPDE	29.72	2.01	12.00	13.00	292	18	1.14	-	-	0.139		0.60	0.30	0.50		0.03	0.17	29.88	-17.88
	5828	12673	P	HDPE	21.90	2.01	13.00	-17.00	6845	16	1.44	-	-	5.798	2.00	0.60				2.02	7.82	29.72	-16.72

Equations Used

GENERAL

Area [A] = $\pi*(D/24)^2$

v=Q/A

F = USSE-USWSEL

GRAVITY

Hydraulic Radius [R_h] =A/(D*π)

HGLS =$((Q*n)/(1.486*A*R_h^{2/3}))^2$

FL = L*HGLS

ML = direct entry of value

FORCEMAIN

FL = $(Q*60*7.4805)^{1.85}*10.44/(C^{1.85}*D^{4.865})$

ML = $\Sigma k*v^2/64.4$

Manning's n Values	
RC	0.013
HDPE	0.011

Hazen-Williams's Rougness Factor [C]	
HDPE	100

Pump Sizing	
U/S Static	
D/S Static	
Total Losses	
Total Dynamic Head	**N/A**

Note: Assumes full pipe flow for all gravity applications. Calculated depth of flow and velocity in 18" gravity pipe with slope =.005 at 1.2 MGD are 3.49 ft/sec and 6", respectively.

MARION, MASS. - OUTFALL EXTENSION - HYDRAULIC GRADE LINE/PUMP STATION SIZING
ALTERNATIVES 3A, 3B, AND 3C

Flow Rate	
1.3	*MGD*

100-year flood elevation:

Tailwater Elev
21.9

	US STA	DS STA	Forcemain (FM), Gravity (G), Or Gravity under Head (P)	Material	Starting Water Surface Elevation	Flow Rate	US Surface Elev	DS Surface Elev	Pipe Length	Pipe Diameter	Velocity	Manning's n Value	HGL Slope	Friction Losses	Gravity Direct Loss Input	Force Main Loss Coefficients				Minor Losses	Total Losses	U/S Water Surface Elevation	Freeboard
Calc ID:						Q	USSE		L	D	v	n	HGLS	FL		k				ML	TL	USWSEL	F
units:						*ft^3/sec*			*ft*	*in*	*ft/sec*			*ft*	*ft*					*ft*			*ft*
FROM PUMP STATION	0	1330	FM	HDPE		2.01			1330	16	1.44	-	-	1.126		0.60				0.02	1.15		
	1330	1554	FM	HDPE		2.01			224	16	1.44	-	-	0.190		0.60				0.02	0.21		
	1554	2178	FM	HDPE		2.01			624	16	1.44	-	-	0.529		0.60				0.02	0.55		
	2178	2984	FM	HDPE		2.01			806	16	1.44	-	-	0.683		0.60				0.02	0.70		
	2984	3408	FM	HDPE		2.01			424	16	1.44	-	-	0.359		0.60				0.02	0.38		
	3408	4457	FM	HDPE		2.01			1049	16	1.44	-	-	0.888		0.60				0.02	0.91		
	4457	4760	FM	HDPE		2.01			303	16	1.44	-	-	0.257		0.60				0.02	0.28		
	4760	5800	FM	HDPE		2.01			1040	16	1.44	-	-	0.881		0.60				0.02	0.90		
	5800	7278	FM	HDPE		2.01			1478	16	1.44	-	-	1.252		0.60				0.02	1.27		
	7278	7810	FM	HDPE		2.01			532	16	1.44	-	-	0.451		0.60				0.02	0.47		
	7810	9460	FM	HDPE		2.01			1650	16	1.44	-	-	1.398		0.60				0.02	1.42		
	9460	10054	FM	HDPE		2.01			594	16	1.44	-	-	0.503		0.60				0.02	0.52		
	10054	13269	FM	HDPE		2.01			3215	16	1.44	-	-	2.723		0.60				0.02	2.74		
	13269	14365	FM	HDPE		2.01			1096	16	1.44	-	-	0.928		0.60	0.30	0.50		0.05	0.97		
	14365	17851	FM	HDPE		2.01			3486	16	1.44	-	-	2.953	2.00	0.60				2.02	4.97		

Equations Used

GENERAL

Area [A] = $\pi*(D/24)^2$

v=Q/A

F = USSE-USWSEL

GRAVITY

Hydraulic Radius [R_h] =$A/(D*\pi)$

HGLS =$((Q*n)/(1.486*A*R_h^{2/3}))^2$

FL = L*HGLS

ML = direct entry of value

FORCEMAIN

FL = $(Q*60*7.4805)^{1.85}*10.44/(C^{1.85}*D^{4.865})$

ML = $\Sigma k*v^2/64.4$

Manning's n Values	
RC	0.013
HDPE	0.011

Hazen-Williams's Rougness Factor [C]	
HDPE	100

Pump Sizing	
U/S Static	28
D/S Static	45
Total Losses	17.44
Total Dynamic Head	**34.44**

MARION, MASS. - OUTFALL EXTENSION - HYDRAULIC GRADE LINE/PUMP STATION SIZING
ALTERNATIVE 4A AND 4B

Flow Rate	
1.3	*MGD*

100-year flood elevation:

Tailwater Elev
21.9

	US STA	DS STA	Forcemain (FM), Gravity (G), Or Gravity under Head (P)	Material	Starting Water Surface Elevation	Flow Rate	US Surface Elev	DS Surface Elev	Pipe Length	Pipe Diameter	Velocity	Manning's n Value	HGL Slope	Friction Losses	Gravity Direct Loss Input	Force Main Loss Coefficients				Minor Losses	Total Losses	U/S Water Surface Elevation	Freeboard
Calc ID:						Q	USSE		L	D	v	n	HGLS	FL		k				ML	TL	USWSEL	F
units:						*ft³/sec*			*ft*	*in*	*ft/sec*			*ft*	*ft*					*ft*			*ft*
	U/S of Rte 6		P	HDPE	34.83	2.01	40.00	35.00	3300	18	1.14	-	-	1.576		0.60				0.01	1.59	36.42	3.58
			P	HDPE	33.63	2.01	35.00	28.00	1400	16	1.44	-	-	1.186		0.60				0.02	1.21	34.83	0.17
GRAVITY	0	316	P	HDPE	33.46	2.01	28.00	28.00	316	18	1.14	-	-	0.151		0.60				0.01	0.16	33.63	-5.63
	316	563	P	HDPE	33.33	2.01	28.00	32.00	247	18	1.14	-	-	0.118		0.60				0.01	0.13	33.46	-5.46
	563	786	P	HDPE	33.22	2.01	32.00	30.00	223	18	1.14	-	-	0.106		0.60				0.01	0.12	33.33	-1.33
	786	1044	P	HDPE	33.08	2.01	30.00	34.00	258	18	1.14	-	-	0.123		0.60				0.01	0.14	33.22	-3.22
	1044	1275	P	HDPE	32.96	2.01	34.00	32.00	231	18	1.14	-	-	0.110		0.60				0.01	0.12	33.08	0.92
	1275	1636	P	HDPE	32.77	2.01	32.00	30.00	361	18	1.14	-	-	0.172		0.60	0.30			0.02	0.19	32.96	-0.96
	1636	1936	P	HDPE	32.61	2.01	30.00	32.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	32.77	-2.77
	1936	2236	P	HDPE	32.46	2.01	32.00	35.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	32.61	-0.61
	2236	2536	P	HDPE	32.30	2.01	35.00	38.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	32.46	2.54
	2536	2836	P	HDPE	32.15	2.01	38.00	38.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	32.30	5.70
	2836	3136	P	HDPE	31.99	2.01	38.00	38.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	32.15	5.85
	3136	3436	P	HDPE	31.84	2.01	38.00	35.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	31.99	6.01
	3436	3736	P	HDPE	31.68	2.01	35.00	34.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	31.84	3.16
	3736	4036	P	HDPE	31.52	2.01	34.00	35.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	31.68	2.32
	4036	4134	P	HDPE	31.47	2.01	35.00	35.00	98	18	1.14	-	-	0.047		0.60				0.01	0.06	31.52	3.48
	4134	4299	P	HDPE	31.37	2.01	35.00	34.00	165	18	1.14	-	-	0.079		0.60	0.30			0.02	0.10	31.47	3.53
	4299	4575	P	HDPE	31.22	2.01	34.00	33.00	276	18	1.14	-	-	0.132		0.60				0.01	0.14	31.37	2.63
	4575	4836	P	HDPE	31.09	2.01	33.00	36.00	261	18	1.14	-	-	0.125		0.60				0.01	0.14	31.22	1.78
	4836	5021	P	HDPE	30.99	2.01	36.00	38.00	185	18	1.14	-	-	0.088		0.60				0.01	0.10	31.09	4.91
	5021	5321	P	HDPE	30.83	2.01	38.00	38.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	30.99	7.01
	5321	5621	P	HDPE	30.68	2.01	38.00	37.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	30.83	7.17
	5621	5801	P	HDPE	30.58	2.01	37.00	35.00	180	18	1.14	-	-	0.086		0.60				0.01	0.10	30.68	6.32
	5801	6049	P	HDPE	30.45	2.01	35.00	33.00	248	18	1.14	-	-	0.118		0.60				0.01	0.13	30.58	4.42
	6049	6270	P	HDPE	30.33	2.01	33.00	30.00	221	18	1.14	-	-	0.106		0.60				0.01	0.12	30.45	2.55
	6270	6570	P	HDPE	30.18	2.01	30.00	26.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	30.33	-0.33
	6570	6870	P	HDPE	30.02	2.01	26.00	25.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	30.18	-4.18
	6870	7170	P	HDPE	29.86	2.01	25.00	28.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	30.02	-5.02
	7170	7470	P	HDPE	29.71	2.01	28.00	30.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	29.86	-1.86
	7470	7770	P	HDPE	29.55	2.01	30.00	35.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	29.71	0.29
	7770	7978	P	HDPE	29.44	2.01	35.00	35.00	208	18	1.14	-	-	0.099		0.60				0.01	0.11	29.55	5.45
	7978	8278	P	HDPE	29.29	2.01	35.00	35.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	29.44	5.56
	8278	8578	P	HDPE	29.13	2.01	35.00	35.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	29.29	5.71
	8578	8878	P	HDPE	28.98	2.01	35.00	35.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	29.13	5.87
	8878	9178	P	HDPE	28.82	2.01	35.00	33.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	28.98	6.02
	9178	9478	P	HDPE	28.67	2.01	33.00	31.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	28.82	4.18
	9478	9778	P	HDPE	28.51	2.01	31.00	28.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	28.67	2.33
	9778	10078	P	HDPE	28.36	2.01	28.00	26.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	28.51	-0.51
	10078	10378	P	HDPE	28.20	2.01	26.00	25.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	28.36	-2.36
	10378	10678	P	HDPE	28.04	2.01	25.00	24.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	28.20	-3.20
	10678	10978	P	HDPE	27.89	2.01	24.00	22.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	28.04	-4.04
	10978	11278	P	HDPE	27.73	2.01	22.00	20.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	27.89	-5.89
	11278	11578	P	HDPE	27.58	2.01	20.00	20.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	27.73	-7.73
	11578	11878	P	HDPE	27.42	2.01	20.00	23.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	27.58	-7.58
	11878	12178	P	HDPE	27.27	2.01	23.00	24.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	27.42	-4.42
	12178	12478	P	HDPE	27.11	2.01	24.00	25.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	27.27	-3.27
	12478	12778	P	HDPE	26.96	2.01	25.00	27.00	300	18	1.14	-	-	0.143		0.60				0.01	0.16	27.11	-2.11
	12778	12905	P	HDPE	26.87	2.01	27.00	30.00	127	18	1.14	-	-	0.061		0.60	0.30	0.50		0.03	0.09	26.96	0.04
	12905	16391	P	HDPE	21.90	2.01	30.00	-17.00	3486	16	1.44	-	-	2.953	2.00	0.50				2.02	4.97	26.87	3.13

Equations Used		
GENERAL	*GRAVITY*	*FORCEMAIN*
Area [A] = $\pi*(D/24)^2$	Hydraulic Radius [R_h] =$A/(D*\pi)$	FL = $(Q*60*7.4805)^{1.85}*10.44/(C^{1.85}*D^{4.865})$
v=Q/A	HGLS =$((Q*n)/(1.486*A*R_h^{2/3}))^2$	ML = $\Sigma k*v^2/64.4$
F = USSE-USWSEL	FL = L*HGLS	
	ML = direct entry of value	

Manning's n Values	
RC	0.013
HDPE	0.011

Hazen-Williams's Rougness Factor [C]	
HDPE	100

Pump Sizing	
U/S Static	
D/S Static	
Total Losses	
Total Dynamic Head	**N/A**

Note: Assumes full pipe flow for all gravity applications. Calculated depth of flow and velocity in 18" gravity pipe with slope =.005 at 1.2 MGD are 3.49 ft/sec and 6", respectively.

MARION, MASS. - OUTFALL EXTENSION - HYDRAULIC GRADE LINE/PUMP STATION SIZING
ALTERNATIVES 5A AND 5B

Flow Rate	
1.3	*MGD*

100-year flood elevation:

Tailwater Elev
21.9

	US STA	DS STA	Forcemain (FM), Gravity (G), Or Gravity under Head (P)	Material	Starting Water Surface Elevation	Flow Rate	US Surface Elev	DS Surface Elev	Pipe Length	Pipe Diameter	Velocity	Manning's n Value	HGL Slope	Friction Losses	Gravity Direct Loss Input	Force Main Loss Coefficients				Minor Losses	Total Losses	U/S Water Surface Elevation	Freeboard
Calc ID:						Q	USSE		L	D	v	n	HGLS	FL		k				ML	TL	USWSEL	F
units:						*ft³/sec*			*ft*	*in*	*ft/sec*			*ft*	*ft*					*ft*			*ft*
FROM PUMP STATION	0	1296	FM	HDPE		2.01			1296	16	1.44	-	-	1.098		0.60				0.02	1.12		
	1296	2659	FM	HDPE		2.01			1363	16	1.44	-	-	1.154		0.60				0.02	1.17		
	2659	3569	FM	HDPE		2.01			910	16	1.44	-	-	0.771		0.60				0.02	0.79		
	3569	4159	FM	HDPE		2.01			590	16	1.44	-	-	0.500		0.60				0.02	0.52		
	4159	5297	FM	HDPE		2.01			1138	16	1.44	-	-	0.964		0.60				0.02	0.98		
	5297	6107	FM	HDPE		2.01			810	16	1.44	-	-	0.686		0.60				0.02	0.71		
	6107	7738	FM	HDPE		2.01			1631	16	1.44	-	-	1.381		0.60				0.02	1.40		
	7738	8443	FM	HDPE		2.01			705	16	1.44	-	-	0.597		0.60				0.02	0.62		
	8443	8855	FM	HDPE		2.01			412	16	1.44	-	-	0.349		0.60				0.02	0.37		
	8855	9908	FM	HDPE		2.01			1053	16	1.44	-	-	0.892		0.60				0.02	0.91		
	9908	10088	FM	HDPE		2.01			180	16	1.44	-	-	0.152		0.60	0.30			0.03	0.18		
	10088	10376	FM	HDPE		2.01			288	16	1.44	-	-	0.244		0.60	0.15			0.02	0.27		
	10376	10619	FM	HDPE		2.01			243	16	1.44	-	-	0.206		0.60	0.15			0.02	0.23		
	10619	11341	FM	HDPE		2.01			722	16	1.44	-	-	0.612		0.60	0.15			0.02	0.64		
	11341	12085	FM	HDPE		2.01			744	16	1.44	-	-	0.630		0.60	0.30			0.03	0.66		
	12085	12464	FM	HDPE		2.01			379	16	1.44	-	-	0.321		0.60				0.02	0.34		
	12464	13037	FM	HDPE		2.01			573	16	1.44	-	-	0.485		0.60				0.02	0.50		
	13037	14705	FM	HDPE		2.01			1668	16	1.44	-	-	1.413		0.60				0.02	1.43		
	14705	16200	FM	HDPE		2.01			1495	16	1.44	-	-	1.266		0.60	0.30			0.03	1.30		
	16200	18278	FM	HDPE		2.01			2078	16	1.44	-	-	1.760	2.00	0.60				2.02	3.78		

Equations Used

GENERAL

Area [A] = $\pi*(D/24)^2$

v=Q/A

F = USSE-USWSEL

GRAVITY

Hydraulic Radius [R_h] =$A/(D*\pi)$

HGLS =$((Q*n)/(1.486*A*R_h^{2/3}))^2$

FL = L*HGLS

ML = direct entry of value

FORCEMAIN

FL = $(Q*60*7.4805)^{1.85}*10.44/(C^{1.85}*D^{4.865})$

ML = $\Sigma k*v^2/64.4$

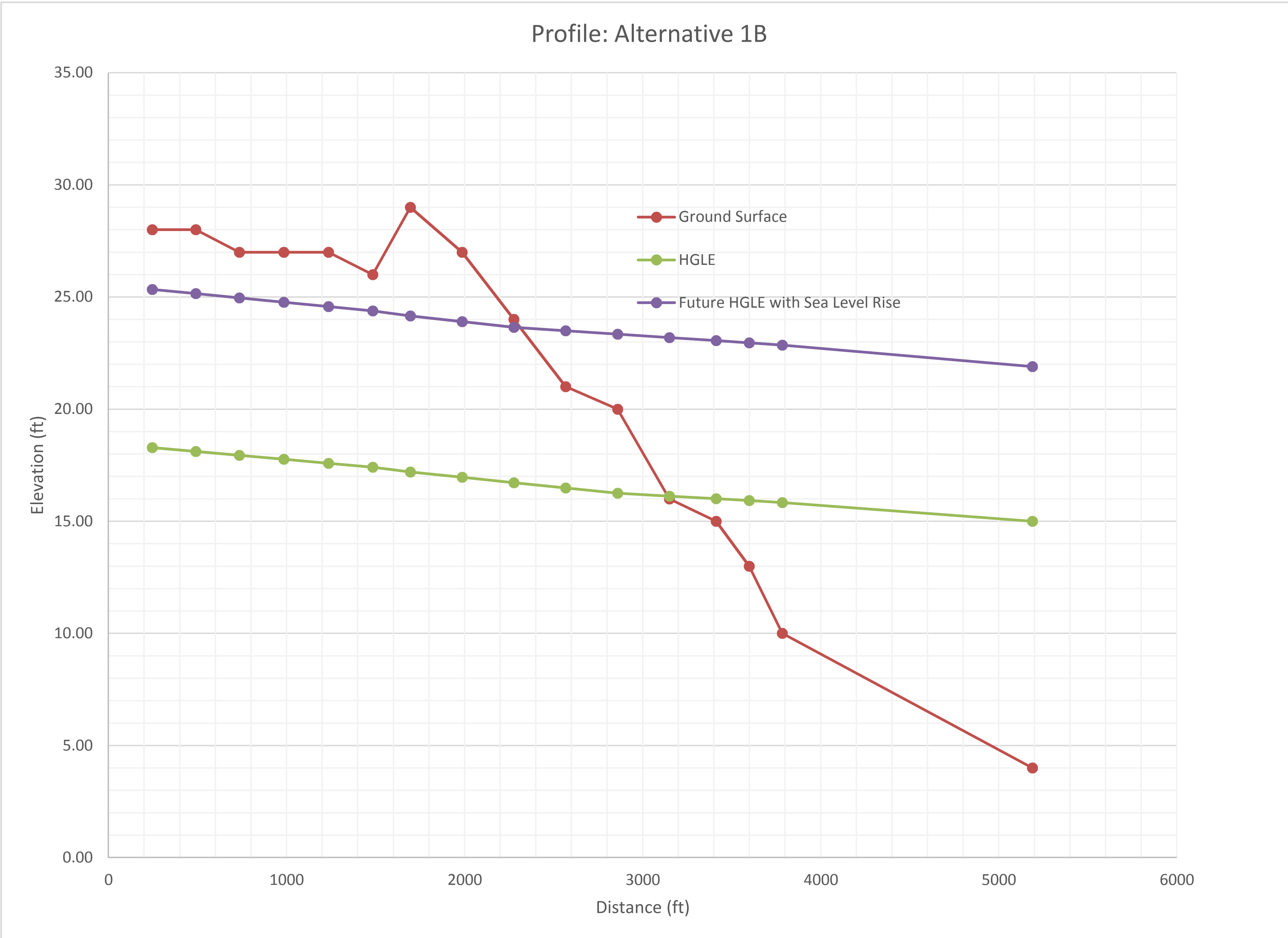
Manning's n Values	
RC	0.013
HDPE	0.011

Hazen-Williams's Rougness Factor [C]	
HDPE	100

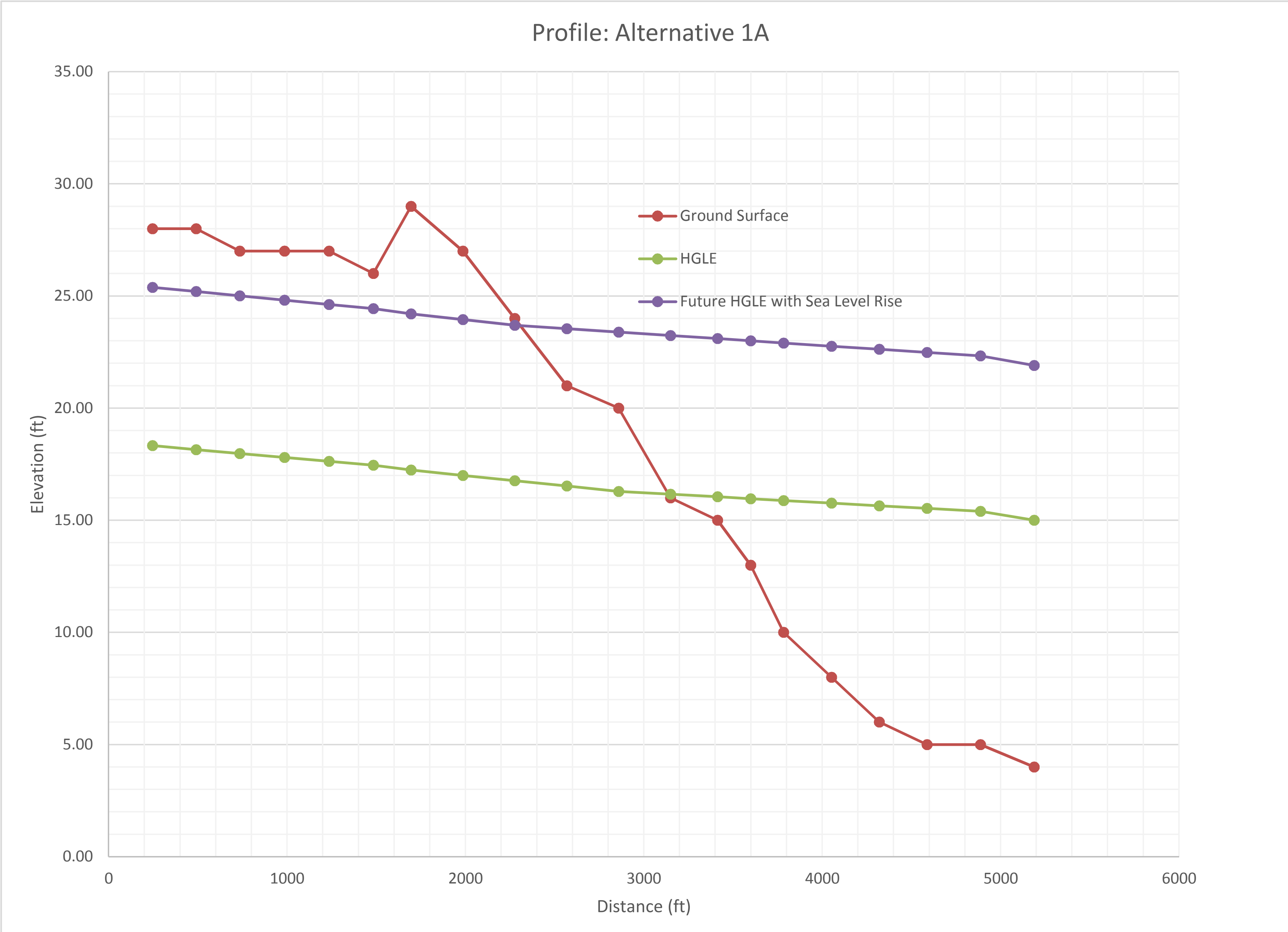
Pump Sizing	
U/S Static	28
D/S Static	21.9
Total Losses	17.91
Total Dynamic Head	**11.81**

Ground Surface Profiles

Hydraulic grade line profiles included for gravity options

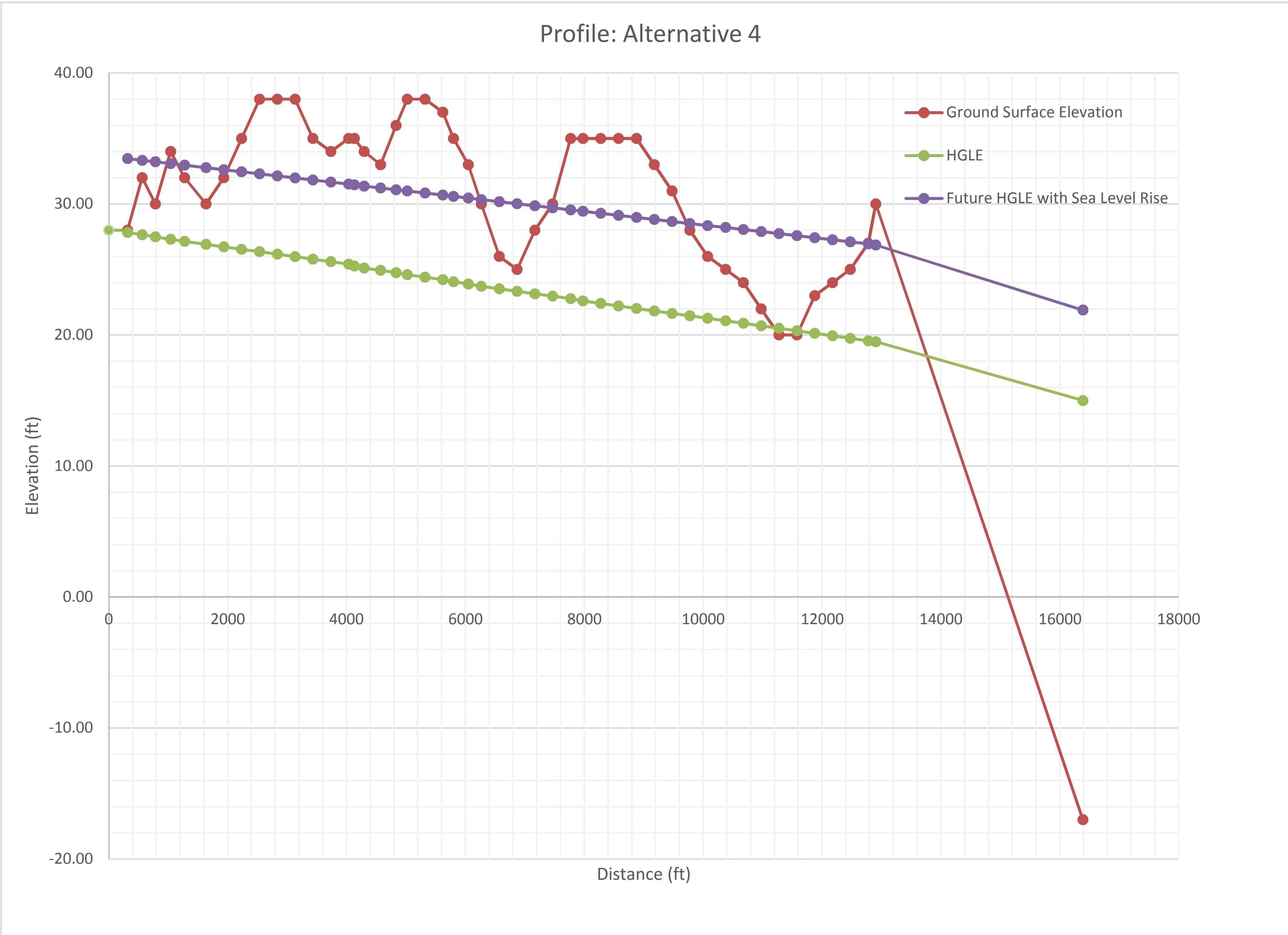


Profile: Alternative 1B
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
Elevation (ft)
0
1000
2000
3000
4000
5000
6000
Distance (ft)
Ground Surface
HGLE
Future HGLE with Sea Level Rise


Profile: Alternative 1A
Ground Surface
HGLE
Future HGLE with Sea Level Rise
Elevation (ft)
Distance (ft)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0
1000
2000
3000
4000
5000
6000


Profile: Alternative 2
40.00
30.00
20.00
10.00
0.00
-10.00
-20.00
Elevation (ft)
0
2000
4000
6000
8000
10000
12000
14000
Distance (ft)
Ground Surface
HGLE
Future HGLE with Sea Level Rise


Profile: Alternative 3
Elevation (ft)
60
50
40
30
20
10
0
-10
-20
-30
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000
Distance (ft)
Ground Surface



Profile: Alternative 4
Ground Surface Elevation
HGLE
Future HGLE with Sea Level Rise
Elevation (ft)
Distance (ft)
40.00
30.00
20.00
10.00
0.00
-10.00
-20.00
0
2000
4000
6000
8000
10000
12000
14000
16000
18000

Profile: Alternative 4
Ground Surface Elevation
Elevation (ft)
50
40
30
20
10
0
-10
-20
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000
Distance (ft)

Appendix D

Development of Opinion of Probable Costs

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Marion, Mass - Outfall Alternatives - Capital Costs

			Quantities																					
			Alternative 1				Alternative 2				Alternative 3						Alternative 4				Alternative 5			
			Alt 1A		Alt 1B		Alt 2A		Alt 2B		Alt 3A		Alt 3B		Alt 3C		Alt 4A		Alt 4B		Alt 5A		Alt 5B	
Item	Units	Unit Cost	Quantity	Cost	Quantity	Cost	Quantity	Cost	Quantity	Cost	Quantity	Cost	Quantity	Cost	Quantity	Cost	Quantity	Cost	Quantity	Cost	Quantity	Cost	Quantity	Cost
16" Diameter HDPE Pipe																								
Open Cut (Roadway)*	LF	$212									14,400	$3,052,800	14,400	$3,052,800	9,400	$1,992,800					16,200	$3,434,400	16,200	$3,434,400
Open Cut (Cross Country)	LF	$183																						
HDD	LF	$420													5,000	$2,100,000								
Underwater, Open Cut	LF	$1,350					6,900	$9,315,000			3,500	$4,725,000					3,500	$4,725,000			2,100	$2,835,000		
Underwater, Direct Lay	LF	$223																						
Underwater HDD	LF	$1,500							6,900	$10,350,000			3,500	$5,250,000	3,500	$5,250,000			3,500	$5,250,000			2,100	$3,150,000
18" Diameter HDPE Pipe																								
Open Cut (Roadway)*	LF	$230	1,500	$345,000	1,500	$345,000	2,300	$529,000	2,300	$529,000							12,300	$2,829,000	12,300	$2,829,000				
Open Cut (Cross Country)	LF	$190					3,600	$684,000	3,600	$684,000							600	$114,000	600	$114,000				
Open Cut (Swamp/marsh)	LF	$280	1,400	$392,000																				
HDD	LF	$450			1,400	$630,000																		
18" RCP																								
Open Cut (Roadway)*	LF	$140	800	$112,000	800	$112,000																		
Open Cut (Cross Country)	LF	$200	1,500	$300,000	1,500	$300,000																		
Open Cut (Swamp/marsh)	LF	$238																						
Structures																								
4' Dia. PC Manhole (5 VF)	EA	$4,183	9	$37,644	8	$33,462	9	$37,644	9	$37,644							16	$66,924	16	$66,924				
4' Dia. PC Air Release Valve Structure	EA	$7,181									6	$43,083	6	$43,083	6	$43,083					7	$50,264	7	$50,264
Flushing Connection w/ Manhole	EA	$15,000					1	$15,000	1	$15,000							1	$15,000	1	$15,000				
Pump Station	LS	$1,500,000									1	$1,500,000	1	$1,500,000	1	$1,500,000					1	$1,500,000	1	$1,500,000
Outfall																								
Headwall with Rip Rap	LS	$10,000	1	$10,000	1	$10,000																		
Deep Water Outfall Multiple Diffuser System	LS	$352,074					1	$352,074	1	$352,074	1	$352,074	1	$352,074	1	$352,074	1	$352,074	1	$352,074	1	$352,074	1	$352,074
*Includes cost of new pavement				$1,196,644		$1,430,462		$10,932,718		$11,967,718		$9,672,957		$10,197,957		$11,237,957		$8,101,997		$8,626,997		$8,171,738		$8,486,738

LF = Linear Foot
EA = Each
LS = Lump Sum

Marion, Mass - Outfall Alternatives - O&M Costs for Pumping Station

Alternative	Head @ 0.588 MGD	Input shaft HP	kW input to motor	kW-hr/yr	Cost/yr
3A	22.5	3.093	2.531	22171	$ 3,769
3B	22.5	3.093	2.531	22171	$ 3,769
3C	22.5	3.093	2.531	22171	$ 3,769
5A	5	0.687	0.562	4927	$ 838
5B	5	0.687	0.562	4927	$ 838

pump efficiency	75%	
motor efficiency	94%	
VFD efficiency	97%	
electrical cost	0.17	$/kW-Hr

Marion, Mass - Outfall Alternatives - Net Present Value

Alternative	Start of Construction	present unloaded cap costs	escalated cap costs w/ contingencies (Alt 1:2020; Alts 2-5: 2022)	present annual O*M costs	escalated annual O&M costs (Alt 1:2020; Alts 2-5: 2022)	2016 NPV total
1A[1]	2019	$ 1,196,644	$ 2,133,385	$ -	$ -	$ 1,895,485
1B[1]	2019	$ 1,430,462	$ 2,550,236	$ -	$ -	$ 2,265,851
2A[2]	2021	$ 10,932,718	$ 22,677,912	$ 1,000	$ 1,230	$ 19,011,995
2B[2]	2021	$ 11,967,718	$ 24,635,489	$ 1,000	$ 1,230	$ 20,641,766
3A[2]	2021	$ 9,672,957	$ 20,295,226	$ 3,769	$ 4,635	$ 17,087,790
3B[2]	2021	$ 10,197,957	$ 21,288,200	$ 3,769	$ 4,635	$ 17,919,390
3C[2]	2021	$ 11,237,957	$ 23,255,234	$ 3,769	$ 4,635	$ 19,566,750
4A[2]	2021	$ 8,101,997	$ 17,323,946	$ 1,000	$ 1,230	$ 14,532,638
4B[2]	2021	$ 8,626,997	$ 18,316,920	$ 1,000	$ 1,230	$ 15,364,238
5A[2]	2021	$ 8,171,738	$ 17,455,851	$ 838	$ 1,030	$ 14,639,191
5B[2]	2021	$ 8,486,738	$ 18,051,636	$ 838	$ 1,030	$ 15,138,151

[1]Capital Costs for Alternatives 1A and 1B do not include TN improvements required at WWTP
[2]Escalated Capital Costs for Alternative 2-5 include additional $2M for Ocean Sanctuaries Act studies

Note: Costs do not include any improvments to lagoons or WWTP

Construction Contingency	20%
Engineering, Permitting, and Implementation	20%
Project Contingency	10%
Inflation Rate	3%
Alt 1 - Mid-point of Construction (2020)	4
Alts 2-5 -Mid-point of Construction (2022)	6
Alts 2-5 -Start O&M (2023)	7
Evaluation Period Duration (yr)	30

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