Appendix 5.1: TASA Technical Memos

TASA TM-1

April 11, 2019

To:	Town of Falmouth, MA	Ref. No.:	11153041
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CC:	File; Project Team		

Subject: **South Coast Embayments – Preliminary Evaluations and Notice of Project Change Project**

Teaticket / Acapesket Study Area, Flow, and N Load Evaluation – Technical Memorandum No. 1 (TASA TM-1)

1. Purpose of Memo

The purpose of this Technical Memorandum is to summarize the flow and nitrogen load evaluation that was conducted for the Teaticket Acapesket Study Area (TASA).

1.1 References and Design Guidelines

The following references and design guidelines were used to develop this memorandum:

References:

- Final Great, Green, and Bournes Pond Embayment Systems Total Maximum Daily Loads for Total Nitrogen (Report #96-TMDL-6 Control #181.0), prepared by the Commonwealth of Massachusetts Executive Office of Environmental Affairs, Massachusetts Department of Environmental Protection, Bureau of Resource Protection and Division of Watershed Management, dated April 6, 2006 (referred to as 'TMDL Report' in this memo).
- Linked Watershed-Embayment Model to Determine Critical Nitrogen Loading Thresholds for Great/Perch Pond, Green Pond and Bournes Pond, Falmouth Massachusetts; prepared by Howes B., J.S Ramsey, S.W. Kelley, R. Samimy, D. Schlezinger, E. Eichner. Massachusetts Estuaries Project, Massachusetts Department of Environmental Protection. Dated April 2005 (referred to as 'MEP Report' in this memo).
- Property Type Classification Codes Non-Arm's Length Codes and Sales Report Spreadsheet Specifications, Division of Local Services MA Department of Revenue, dated June 2016 (referred to as 'Land Use Codes' in this memo)

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<u>Design Guidelines:</u>

- New England Interstate Water Pollution Control Commission, TR-16: Guides for the Design of Wastewater Treatment Works, 2011 Edition as Revised in 2016.

2. TASA Study Area

2.1 TASA Nitrogen Loading Background

The Teaticket Acapesket Study Area (TASA), which is illustrated in Figure 1, includes properties in both the Great Pond watershed and the Green Pond watershed as well as parcels that flush directly into Vineyard Sound.

The Great Pond System watershed is divided into 24 sub-watersheds. The Green Pond System watershed is divided into four sub-watersheds. Sub-watersheds are further broken down depending on the length of time anticipated for the groundwater to reach a waterbody. LT10 is used to indicate a less than 10 year travel time and GT10 is used to indicate a greater than 10 year travel time. The TMDL Report presents target threshold watershed loads for two waterbodies in the Great Pond System (Great Pond and Perch Pond) and for the one waterbody in the Green Pond System (Green Pond). The TMDL allocation outlines the maximum nitrogen loading that the waterbody may receive while maintaining its water quality standards and designated uses. Table 1 outlines the TMDL for the three waterbodies and the sub-watersheds that make up each waterbody. The sub-watersheds for Great Pond, Perch Pond and Green Pond are shown in Figure 2.

Table 1 Great, Perch, and Green Pond Total Maximum Daily Loads (TMDL)

Major Watershed	Waterbody Segment[1]	Description[1]	TMDL (kg/d)[2]
Great Pond System	Great Pond	From the inlet of Coonamessett River to Vineyard Sound (excluding Perch Pond), Falmouth	22.50
	Perch Pond	Connects to northwest end of Great Pond, west of Keechipam Way, Falmouth	0.59
Green Pond System	Green Pond	East of Acapesket Road, outlet to Vineyard Sound, Falmouth	46.26

Sources:

1. Massachusetts Year 2016 Integrated List of Waters.
2. Table 5 – 'The Total Maximum Daily Loads (TMDL) for the Great, Green, and Bournes Pond Embayment Systems, represented as the sum of the calculated target threshold loads (from controllable watershed sources), atmospheric deposition, and sediment sources (benthic flux)' of the 'Final Great, Green and Bournes Pond Embayment Systems Total Maximum Daily Loads for Total Nitrogen' (report #96-TMDL-6 Control #181.0), dated April 6, 2006.

The largest source of controllable nitrogen for both watersheds comes from on-site septic systems. Nitrogen from septic systems enters the waterbody through two freshwater sources:

- Direct groundwater discharge into the watershed (groundwater load) – this represents septic system loads that flow directly through groundwater to the main embayment waterbody.
- Surface water discharge (surface water load) – this represents septic system loads from up-gradient watersheds that enters a stream and/or pond via groundwater before discharging into the main

embayment waterbody. During the development of the MEP Report, flow and nitrogen concentrations measurements were taken at stream gauges along Route 28 in order to quantify surface water discharge loads into the larger waterbodies. Surface water loads are represented as a point discharge from an upper watershed into the main waterbody.

Other sources of nitrogen to the watersheds include natural background, land use (stormwater runoff and fertilizers), atmospheric deposition, the Massachusetts Military Reservation Wastewater Treatment Facility (WWTF) plume and nutrient-rich embayment sediments. Of these sources, land use, septic system, and WWTF sources are considered “controllable” sources of nitrogen.

Since the majority of the controllable nitrogen in both watersheds comes from septic systems, the MEP Report includes a scenario demonstrating how the TMDL can theoretically be met through the removal of septic system loads only (Table 2). Under this scenario 50% of the surface water load to Great Pond—introduced into the waterbody through the Coonamessett River—would need to be removed, in addition to 100% of the load from Great Pond (Great Pond GT 10W, Great Pond GT 10E, and Great Pond LT10 sub-watersheds) and Perch Pond (Perch Pond LT10, Perch Pond GT10, Mares Pond, and Spectacle Pond sub-watersheds) to meet the TMDLs for Great Pond and Perch Pond.

The MEP scenario indicates that the Green Pond TMDL could be met through removal of approximately 73% of the septic load from Green Pond (Green Pond sub-watershed) and with no removal of surface water septic load.

Table 2 Great, Perch, and Green Pond Threshold Septic Loads

Major Watershed	Subwatershed / Surface Body Source	Present Septic Load (kg/d)[1]	Threshold Septic Load (kg/d)[1]	Threshold Septic Load % Change[1]
Great Pond System	• Great Pond	21.28	0.00	-100.0%
	• Perch Pond	4.47	0.00	-100.0%
	• Coonamessett River	15.08	7.54	-50.0%
Green Pond System	• Green Pond	16.62	4.43	-73.4%
	• Backus Brook	2.08	2.08	0.0%

Notes:

1. Source: Table VIII-2 – ‘Comparison of sub-embayment watershed septic loads (attenuated) used for modeling of present and threshold loading scenarios of the Ashumet Valley Systems. These loads do not include direct atmospheric deposition (onto the sub-embayment surface), benthic flux, runoff, or fertilizer loading terms of the Great/Perch Pond, Green Pond, and Bournes Pond’, MEP Report, dated April 2005.

2.2 Wastewater Infrastructure in TASA

The majority of the parcels in TASA rely on on-site disposal systems (septic systems) for wastewater treatment. According to Massachusetts Alternative Septic System Test Center records, 12 properties in TASA are serviced by Innovative and Alternative (I/A) systems.

A portion of the Great Pond watershed (outside of the TASA) has been sewered as part of the Little Pond Service Area (LPSA) project. The number of sewered parcels in the Great Pond watershed is summarized in Table 3. There are currently no sewered parcels in the Green Pond watershed.

Table 3 Number of Sewered Parcels in the Great Pond Watershed

Great Pond Subwatershed	Number of Sewered Parcels
Great Pond GT10W	7
Great Pond LT10	245
Lower Coonamessett River	0
Perch Pond LT10	1
Total	**253**

Notes:

1. Source: LPSA Sewer Connections

2.3 Number of Parcels in TASA

The overall number of parcels in TASA is summarized in Table 4. Parcels are subdivided into developed parcels, developable parcels, and undevelopable parcels.

Table 4 Number of Parcels in the Teaticket Acapesket Study Area[1]

Major Watershed	Subwatershed	Number of Developed Parcels[4]	Number of Developable Parcels[2]	Number of Undevelopable Parcels[3]	Total Number of Parcels
Great Pond	Great Pond GT10W	9	0	0	9
	Great Pond LT10	1,033	40	45	1,118
	Lower Coonamessett River	24	3	0	27
	Perch Pond LT10	179	8	2	189
Green Pond	Green Pond	509	27	13	549
Outside of a Watershed (Flushes Directly to Vineyard Sound)		67	3	12	82
Total		**1,821**	**81**	**72**	**1,974**

1. Source: Town of Falmouth Assessors data dated 2018.
2. Parcels with Land Use Codes 130 (Residential – Developable Land), 131 (Residential – Potentially Developable Land), 390 (Commercial – Developable Land).
3. Parcels with Land Use Code 132 (Residential – Undevelopable Land), 392 (Commercial – Undevelopable Land), 442 (Industrial – Undevelopable Land).
4. Parcels with all other Land Use Codes.

The overall number of parcels in TASA is divided by Land Use Codes into “residential” (defined as Land Use Code 1), “commercial” (defined as Land Use 3) and “other parcels” (defined as all other Land Use Codes) in Table 5. Figure 4 shows a land use map of the TASA.

Table 5 Number of Parcels in the Teaticket Acapesket Study Area – Types of Parcels[1]

Major Watershed	Subwatershed	Number of Residential Parcels[2]	Number of Commercial Parcels[3]	Number of Parcels Classified as "Other"[4]	Total Number of Parcels
Great Pond	Great Pond GT10W[2]	3	1	5	9
	Great Pond LT10	1,069	16	33	1,118
	Lower Coonamessett River	18	4	5	27
	Perch Pond LT10	175	4	10	189
Green Pond	Green Pond	536	3	10	549
Outside of a Watershed (Flushes Directly to Vineyard Sound)		79	1	2	82
Total		**1880**	**29**	**65**	**1,974**

Notes:

1. Source: Town of Falmouth Assessors data dated 2018.
2. Parcels with Land Use Code 1 (Residential Land Use Code).
3. Parcels with Land Use Code 3 (Commercial Land Use Code).
4. Parcels with all other Land Use Codes.

Table 6 compares the number of parcels in the Study Area to the number of overall parcels in each of the sub-watersheds for the Great Pond System (including the Great Pond Watershed, Coonamessett River, and Perch Pond Watershed) and the Green Pond System. As noted in Table 4, TASA also includes parcels outside of the watersheds that flush directly to Vineyard Sound.

Table 6 Percentage of Great Pond System and Green Pond System Parcels in TASA

Major Watershed	Subwatershed	Number of Parcels in TASA[1]	Number of Currently Sewered Parcels in Great Pond System[3]	Number of Overall Parcels in the Subwatershed[2]	% of Sub-watershed Parcels in TASA and LPSA
Great Pond System	Great Pond LT 10	1,118	245	1,518	90%
	Great Pond GT 10W	9	7	187	9%
	Great Pond GT 10E	0	0	119	0%
	Deer Pond	0	0	50	0%
	Great Pond Total	**1,127**	**252**	**1,874**	**74%**
	Perch Pond GT 10	0	0	76	0%
	Perch Pond LT 10	189	1	298	64%
	Mares Pond LT10	0	0	56	0%
	Mares Pond GT10	0	0	44	0%
	Spectacle Pond	0	0	58	0%
	Perch Pond Total	**189**	**1**	**532**	**36%**
	Coonamessett Pond GT10	0	0	18	0%
	Coonamessett Pond LT10	0	0	192	0%
	Round Pond	0	0	29	0%
	Deep Pond GT10	0	0	5	0%
	Deep Pond LT10	0	0	67	0%
	Crooked Pond GT10	0	0	24	0%
	Crooked Pond LT10	0	0	187	0%
	Shallow Pond	0	0	33	0%
	Round Pound (South)	0	0	78	0%
	Jenkins Pond	0	0	340	0%
	Flax Pond GT10	0	0	34	0%
	Flax Pond LT10	0	0	138	0%
	Upper Coonamessett River GT10	0	0	152	0%
	Upper Coonamessett River LT10	0	0	488	0%
	Lower Coonamessett River	27	0	1,220	3%
	Coonamessett River Total	**27**	**0**	**3005**	**1%**
Green Pond System	Green Pond	549	0	1007	55%
	Mill Pond	0	0	180	0%
	Backus Brook LT10	0	0	216	0%
	Backus Brook GT10	0	0	141	0%
	Green Pond System Total	**549**	**0**	**1544**	**36%**

Notes:
1. Source: Town of Falmouth Assessors data dated 2018.
2. Source: MEP Rainbow Spreadsheet.
3. Parcels in the Great Pond watershed sewered as part of the Little Pond Service Area (LPSA) project.

3. TASA – Sub Areas

For the purpose of greater flexibility in the evaluation TASA is broken out into “sub-areas” as follows.

- Sub-Area 1: This sub-area includes the portion of TASA west of Great Pond. The sub-area is primarily comprised of residential parcels in the Great Pond watershed and also includes commercial, industrial, and municipal parcels.
- Sub-Area 2: This sub-area includes the portion of TASA north of Emerson Street and east of Great Pond. The sub-area is primarily comprised of residential parcels in the Great Pond and Green Pond watersheds. The sub-area also contains commercial parcels primarily bordering Route 28/Teaticket Highway.
- Sub-Area 3: This sub-area includes the portion of TASA south of Emerson Street and is primarily comprised of residential parcels. A portion of the parcels in Sub-Area 3 are located in the Great Pond and Green Pond watersheds. The MEP report indicates that the remainder of the parcels in this area flush directly to Vineyard Sound. The entirety of Sub-Area 3 is located in a low elevation within the 100 year flood zone. A high percentage of the parcels in Sub-Area 3 are seasonal properties.
- Sub-Area 4 (LPSA): This sub-area includes the parcels in the Great Pond watershed that were sewered during the LPSA project. Since these properties are already sewered they are not included in the wastewater flow analysis. The parcels are included in the nutrient load analysis so that the overall load reduction to the Great Pond watershed (through both the sewered parcels in LPSA and the parcels proposed for sewering as part of TASA) can be evaluated.

The four sub-areas are shown in Figure 3. TASA will be further broken out into individual potential sewersheds in Technical Memorandum 2.

4. Estimated TASA Wastewater Flows

4.1 Available Water Data

Water usage data, provided by the Falmouth Water Department, for the years 2014 through 2016 was used to develop estimated wastewater flows for the Study Area. The majority of the parcels in TASA have water data (provided by the Falmouth Water Department) associated with them. Seventeen residential parcels (LUC 101) do not have associated water data. A review of aerial photography and assessor’s data indicates that the majority of these properties have houses constructed on them. It was assumed that the parcels are served by private wells. An average residential water use of 151 gallons per day (gpd), based on MEP assumptions, was assigned to these parcels.

A review of the land use codes for remaining (non-residential) parcels without an assigned water use (32 parcels total) indicates that the majority of these parcels are not anticipated to have a water use (examples of land use codes include open space properties or accessory land with an improvement such as a garage). No water use flow was assigned to non-residential properties that do not have an associated water use.

Town-provided water use information per parcel was joined by account number to the Town's most current parcel data (January 8, 2018) through GIS. Once all the water use information from the Town was associated to the appropriate parcel in GIS, the total water usage for the TASA was totaled and compared to the raw water data provided by the Town for this area. The difference in total water usage between the two data sets was less than 1% and was attributed to joining the water data with multiple accounts to one parcel location in the GIS. (i.e: a parcel that contains a condominium).

4.2 Current Water Usage and Wastewater Flows

As outlined in Section 4.1, water usage data, provided by the Falmouth Water Department, for the years 2014 through 2016 was used to develop an estimated wastewater flow for each developed parcel in the TASA. A 90% conversion factor (which is consistent with the conversion factor used in the MEP reports) was used to convert water usage to wastewater flow (Table 7). As outlined in the 'Wastewater and Nutrient Management Services Existing Collection System Evaluation Technical Memorandum S-3,' prepared by GHD and dated 2013, a peak instantaneous flow factor of 3.4 was established for the Falmouth collection system using the 2011 TR-16 guidelines and plant data.

Table 7 Existing Water Usage and Wastewater Flow in TASA[6]

Major Watershed	Subwatershed	Average Water Usage (gpd)[1]	Average Wastewater Flow (gpd)[4,5]	Peak Instantaneous WW Generation (gpd)[2,3,5]
Great Pond	Great Pond GT10W[7]	3,700	3,300	11,200
	Great Pond LT10	139,000	125,100	425,300
	Lower Coonamessett River	7,200	6,500	22,100
	Perch Pond LT10	25,900	23,300	79,200
Green Pond	Green Pond	75,200	67,700	230,200
Outside of a Watershed (flushes directly to Vineyard Sound)		14,700	13,200	44,900
Total		**266,000**	**239,000**	**813,000**

Notes:

1. Source: 2014 – 2016 water usage data, provided by the Falmouth Water Department.
2. Source: 'Wastewater and Nutrient Management Services Existing Collection System Evaluation Technical Memorandum S-3' prepared by GHD, dated 2013.
3. Peak instantaneous peaking factor = 3.4 (per S-3)
4. Wastewater Flow = Water Usage x 0.9 conversion factor
5. Flow estimates include only flow from wastewater generation. An estimate for infiltration and inflow (I/I) will be incorporated into the flow estimate once a preliminary sewer layout has been established.
6. Sub-area flow values have been rounded to the nearest hundred. Total flow values have been rounded to the nearest thousand.
7. No parcels in TASA are located in the Great Pond GT10W subwatershed.

Table 8 summarizes average wastewater flows by Sub-Area. As shown in Table 8 a substantial portion of the flow south of Emerson Street (approximately 45%) flushes directly into Vineyard Sound and is not anticipated to negatively impact the nitrogen concentrations in the Great and Green Pond watersheds.

Table 8 Average Current Wastewater Flow in TASA – By Subarea

Major Watershed	Subwatershed	Average Wastewater Flow (gpd)			
		Sub-Area 1	Sub-Area 2	Sub-Area 3	Total TASA
Great Pond	Great Pond GT10W	3,300	0	0	3,300
	Great Pond LT10	58,300	60,800	6,000	125,100
	Lower Coonamessett River	2,200	4,300	0	6,500
	Perch Pond LT10	23,300	0	0	23,300
Green Pond	Green Pond	0	60,300	7,400	67,700
Outside of a Watershed (flushes directly to Vineyard Sound)		0	0	13,200	13,200
Total		**87,000**	**125,000**	**27,000**	**239,000**

Notes:

1. Source: 2014 – 2016 water usage data, provided by the Falmouth Water Department.
2. Sub-area flow values have been rounded to the nearest hundred. Total flow values have been rounded to the nearest thousand.

4.3 Future Wastewater Flows

The following assumptions were used to develop estimated future wastewater flows for the Study Area:

- All developable residential properties are assumed to be developed into single-family residential houses. All developable commercial and industrial properties are also assumed to be developed. All developable properties were assigned the MEP average wastewater generation (using the formula water usage x 0.9 = wastewater generation) for the applicable type of property, as follows:
  - Residential Properties = 151 gpd x 0.9 = 136 gpd
  - Commercial Properties = 553 gpd x 0.9 = 553 gpd
  - Industrial Properties = 18 gpd x 0.9 = 16.2 gpd
- During Workshop No. 1 on March 16, 2018 one parcel (2 Lewis Street) was identified as a potential site for a future 40B development. The future wastewater generation for this parcel was estimated using the average housing density of two currently proposed 40B developments in Falmouth (Helmis Circle and Spring Bars Road) and the MEP average wastewater generation of single family residential properties. The housing density for the 9.19 acre parcel was assumed to be 4.2 houses per acre (38 single residential houses total).
- No other parcels were identified by the Town as significantly underdeveloped lots. No other redevelopment assumptions were included in the future flows evaluation.

Estimated future water usage and wastewater flows are summarized in Table 9.

Table 9 Future Water Usage and Wastewater Flow in TASA[5]

Major Watershed	Subwatershed	Average Water Usage (gpd)[1]	Average Wastewater Flow (gpd)[4]	Peak Instantaneous WW Generation (gpd)[2,3,4]
Great Pond	Great Pond GT10W[6]	3,700	3,300	11,200
	Great Pond LT10	149,600	134,600	457,600
	Lower Coonamessett River	8,000	7,200	24,500
	Perch Pond LT10	27,000	24,300	82,600
Green Pond	Green Pond	78,900	71,000	241,400
Outside of a Watershed (flushes directly to Vineyard Sound)		15,100	13,600	46,200
Total		**282,000**	**254,000**	**864,000**

Notes:

1. Source: 2014 – 2016 water usage data, provided by the Falmouth Water Department.
2. Source: 'Wastewater and Nutrient Management Services Existing Collection System Evaluation Technical Memorandum S-3' prepared by GHD, dated 2013.
3. Peak instantaneous peaking factor = 3.4 (per S-3)
4. Flow estimates include only flow from wastewater generation. An estimate for infiltration and inflow (I/I) will be incorporated into the flow estimate once a preliminary sewer layout has been established.
5. Sub-area flow values have been rounded to the nearest hundred. Total flow values have been rounded to the nearest thousand.
6. No parcels in TASA are located in the Great Pond GT10W subwatershed.

5. Estimated TASA Nitrogen Loads

Wastewater nitrogen loads were developed for each parcel by converting the daily estimated wastewater flow and the estimated total nitrogen concentration to a load in kilograms of nitrogen per year (kg/yr). The nitrogen concentration of septic system effluent was estimated to be 26.25 mg/L, in accordance with the MEP reports. As noted previously, a portion of the Great Pond watershed was sewered through the Little Pond Service Area (LPSA) project. The nitrogen load collected from the Great Pond watershed as part of the LPSA project is included in Table 10 in order to quantify the overall nitrogen load reduction to the Great Pond watershed through sewering (both through the constructed LPSA project and the proposed TASA project).

Table 10 – TASA Nitrogen Load – by Sub-Area

Major Watershed	Subwatershed	Sub-Area 1 Nitrogen Load (kg/yr)	Sub-Area 2 Nitrogen Load (kg/yr)	Sub-Area 3 Nitrogen Load (kg/yr)	Sub-Area 4 (LPSA) Nitrogen Load (kg/yr)	Total (TASA and LPSA) Nitrogen Load (kg/yr)
Great Pond	Great Pond GT10W	100	0	0	100	200
	Great Pond LT10	2,100	2,200	200	900	5,400
	Lower Coonamessett River	100	200	0	0	300
	Perch Pond LT10	800	0	0	0	800
	Great Pond System Total	**3,100**	**2,400**	**200**	**1,000**	**6,700**
Green Pond	Green Pond	0	2,200	300	0	2,500
	Green Pond System Total	**0**	**2,200**	**300**	**0**	**2,500**
Outside of a Watershed (flushes directly to Vineyard Sound)		0	0	500	300	800

Notes:

1. Per MEP reports, nitrogen concentration of septic system effluent is estimated at 26.25 mg/L.
2. All loads have been rounded to the nearest hundred.

Table 11 compares the septic system nitrogen load of each sub-area to the septic load that would need to be removed to meet the TMDL (per the MEP scenario outlined in Table VII-2). As demonstrated in Table 11, the TMDL for each waterbody cannot be met through sewering of TASA alone. Options to address the remainder of the septic load that needs to be removed to meet each TMDL may include additional sewering or implementation of non-traditional nitrogen removal technologies.

Table 11 – Septic System Nitrogen Load – by Sub-Area[3,4]

Waterbody	Septic System Nitrogen Load (kg/d)[2]						% of Septic Load Removed if All Parcels in All Sub-Areas Sewered[2]	% of Septic Load That Needs to Be Removed to Meet TMDL[1]
	Present Septic Load[1]	Sub-Area 1	Sub-Area 2	Sub-Area 3	Sub-Area 4 (sewered through LPSA)	Total (TASA and LPSA)		
Great Pond	21.3	6.1	6.0	0.6	2.6	15.4	**71%**	**100%**
Perch Pond	4.6	2.3	0.0	0	0	2.3	**51%**	**100%**
Coonamessett River	15.1	0.2	0.4	0	0	0.6	**4%**	**50%**
Green Pond	16.6	0	6.0	0.7	0	6.7	**41%**	**73.4%**

Notes:

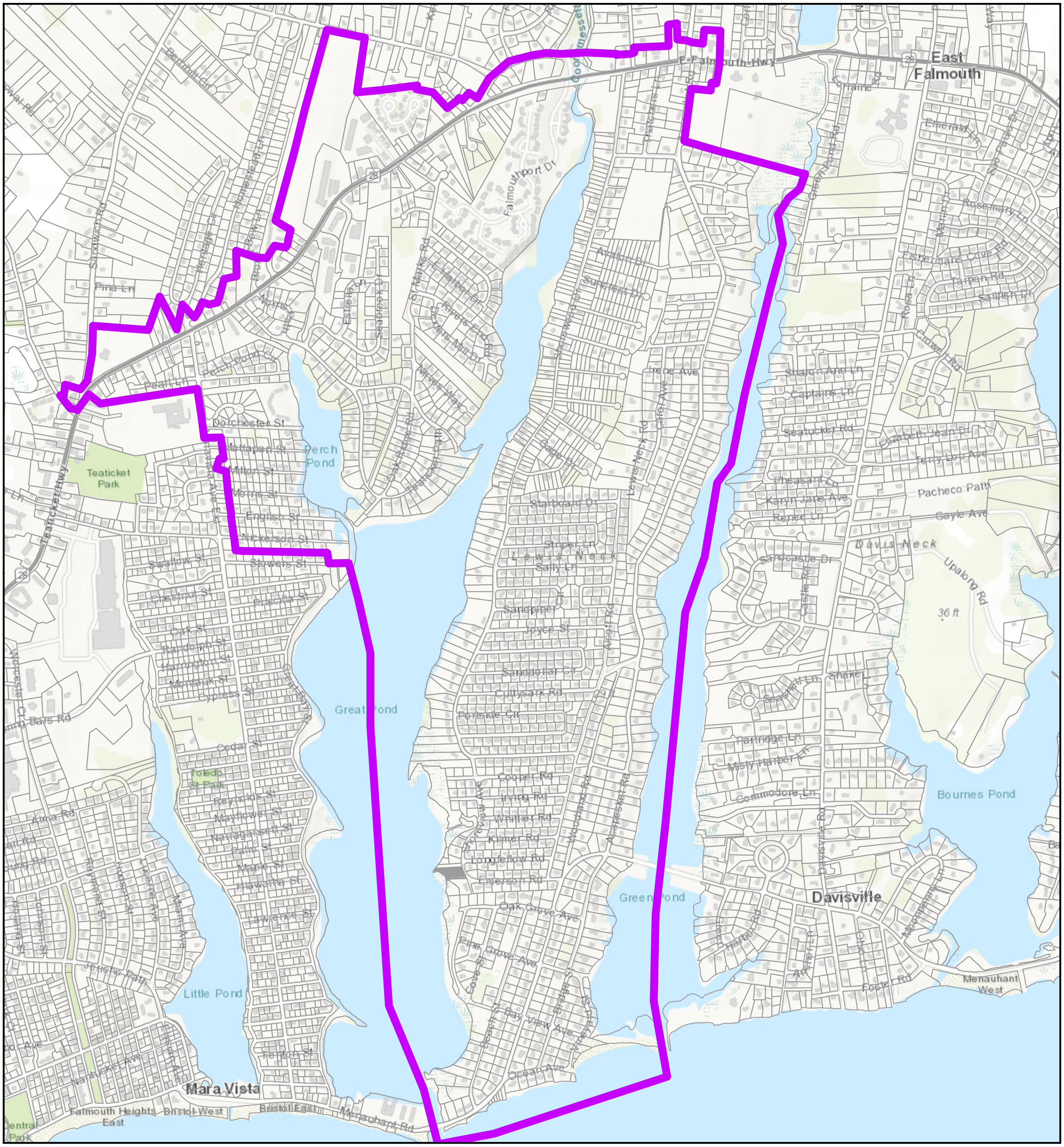
1. Source: Table VII-2 ‘Comparison of sub-embayment watershed septic loads (attenuated) used for modeling of present and threshold loading scenarios of the Ashumet Valley systems. These loads do not include direct atmospheric deposition (onto the sub-embayment surface), benthic flux, runoff, or fertilizer loading terms’ of the MEP Report, dated April 2005.
2. Analysis assumes all of the parcels in each sub-area are sewered.
3. Analysis assumes treated effluent is discharged in a watershed outside of the Great Pond and Green Pond major watersheds.
4. All nitrogen load values are rounded to the tenth.

6. Summary and Next Steps

As summarized in Table 11, although sewering the TASA would substantially reduce nitrogen loading to Great Pond and Green Pond, the TMDLs for Great Pond, Perch Pond, and Green Pond cannot be met through sewering in TASA alone. In order to meet the TMDL through removal of septic system nitrogen, additional sewering will be required in additional sub-watersheds for each waterbody and/or non-traditional nitrogen removal technologies will need to be implemented.

Technical Memorandum 2 will provide an evaluation of the collection and transmission system and a preliminary sewer layout for TASA. The flows and loads presented in Technical Memorandum 1 assume that all parcels with a water use in each sub-area are sewered. The flows and loads should be re-evaluated once a preliminary sewer layout has been developed.

Figures



LEGEND

Teaticket /Acapesket Study Area (TASA)

Paper Size ANSI A

0 800 1,600 3,200 Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

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GHD

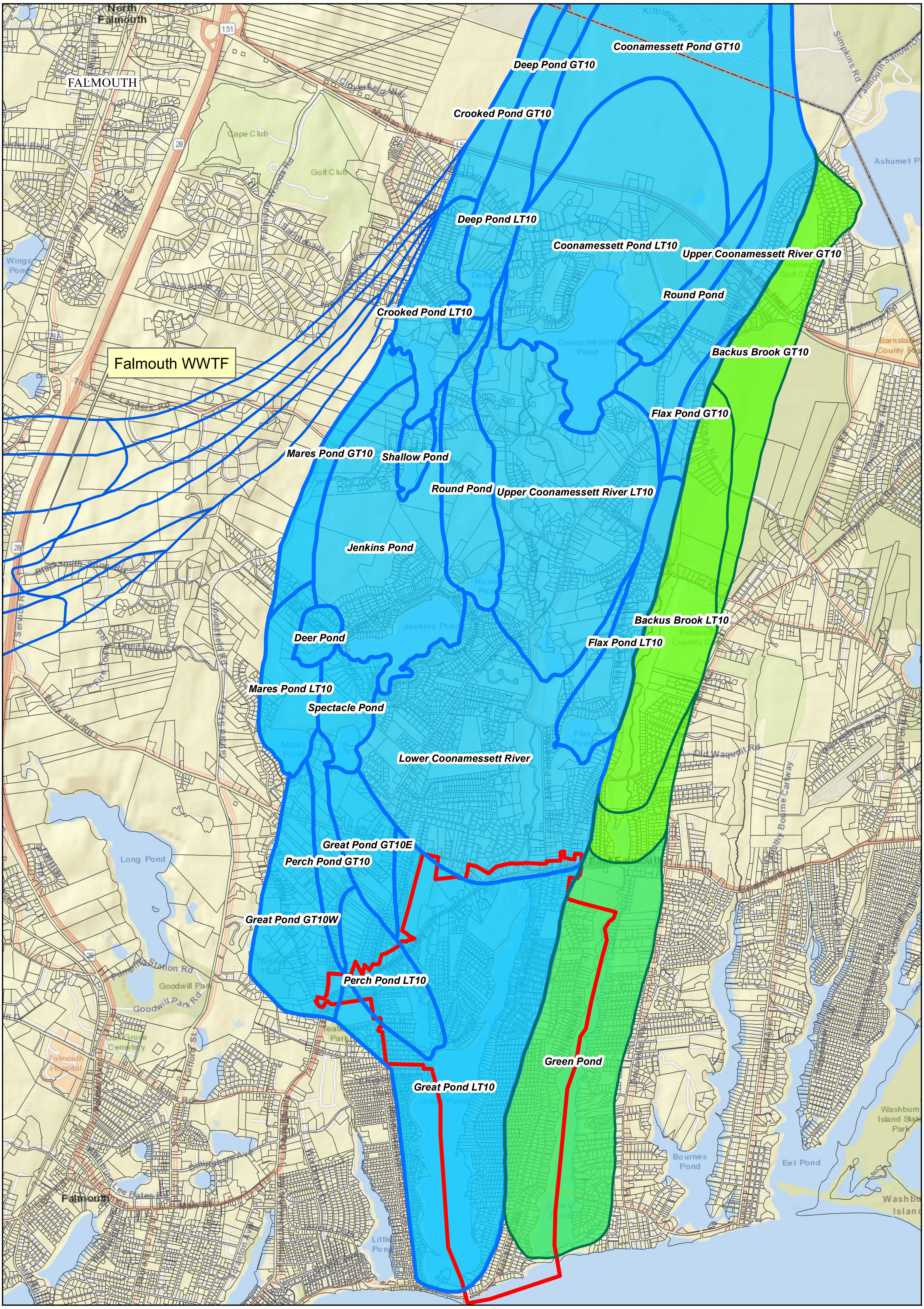
TOWN OF FALMOUTH, MA
TEATICKET/ACAPESKET PRELIMINARY
EVALUATION (TASA TM-1)

STUDY AREA LOCUS

Job Number	111-53041
Revision	-
Date	14 Feb 2019

Figure 1

G:\111\11153041 Town of Falmouth South Coast CWMP Update\GIS\Maps\MXD_Deliverables\LOCUS.mxd 180 Lonsdale Street Melbourne VIC 3000 Australia **T** 61 3 8687 8000 **F** 61 3 8687 8111 **E** melmail@ghd.com **W** www.ghd.com



LEGEND

Teaticket/Acapesket Study Area

Great Pond Watershed

Green Pond Watershed

Paper Size ANSI B

0 1,000 2,000 4,000

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1927
Grid: NAD 1927 StatePlane Massachusetts Mainland FIPS 2001

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Town of Falmouth,MA

Teaticket/Acapesket Preliminary Evaluation (TASA TM-1)

Job Number	11153041
Revision	A
Date	Feb 14, 2019

GREAT POND AND GREEN POND SYSTEMS - SUBWATERSHEDS

Figure 2

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1545 Iyannough Road, Hyannis Massachusetts 02601 USA **T** 1 508 362 5680 **F** 1 508 362 5684 **E** hyamail@ghd.com **W** www.ghd.com

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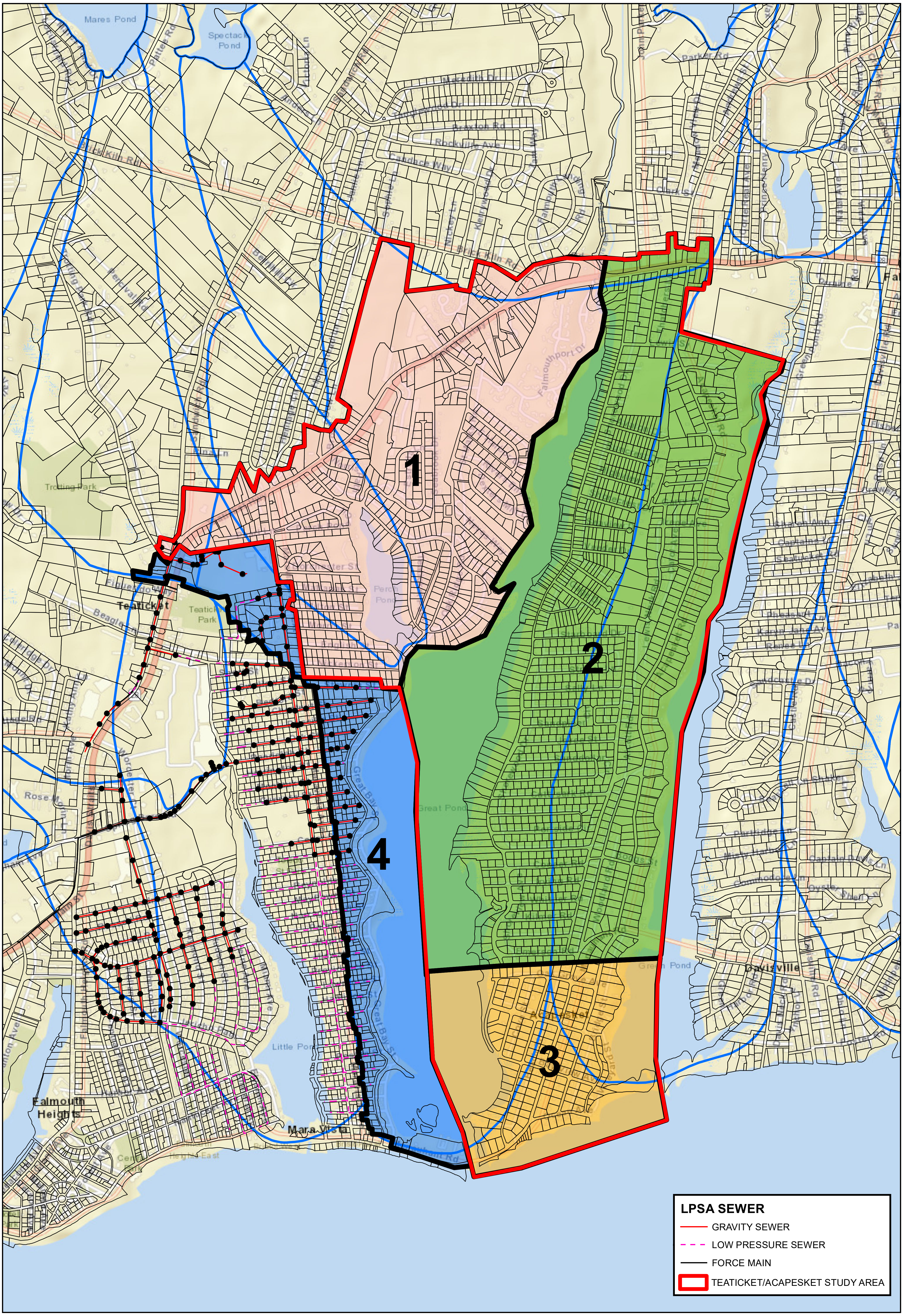

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Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1927
Grid: NAD 1927 StatePlane Massachusetts Mainland FIPS 2001

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Town of Falmouth,MA
Teaticket/Acapesket Preliminary Evaluation (TASA TM-1)

Job Number	11153041
Revision	A
Date	Feb 20, 2019

TEATICKET/ACAPESKET STUDY AREA SUB-AREAS

Figure 3

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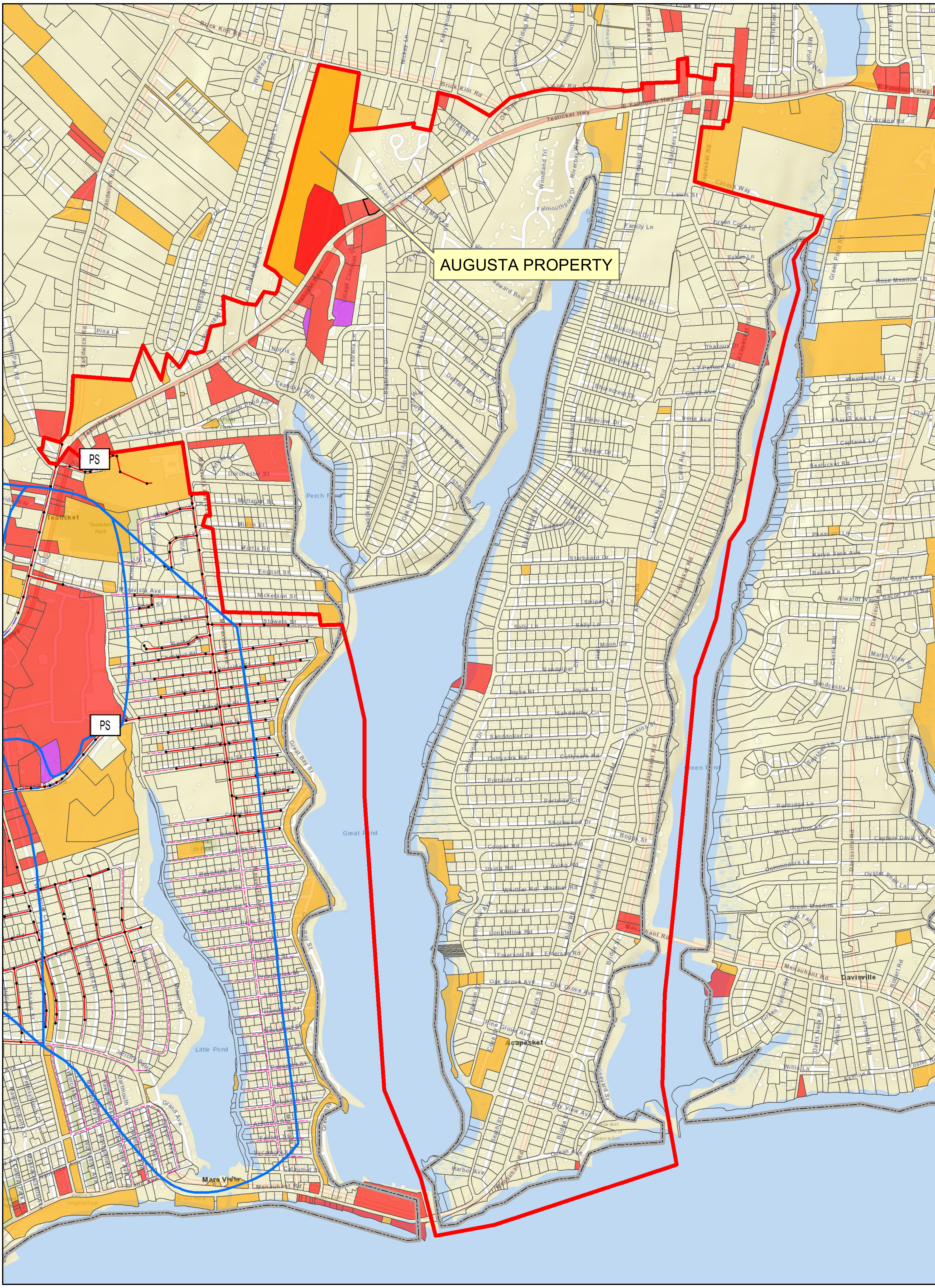

LEGEND

Teaticket/Acapesket Study Area

Gravity Sewer

Low Pressure Sewer

Force Main

Landuse

Landuse selection

Residential

Commercial

Industrial

Forest

Agriculture

Exempt

Paper Size ANSI B

0 500 1,000 2,000

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1927
Grid: NAD 1927 StatePlane Massachusetts Mainland FIPS 2001

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GHD

Town of Falmouth,MA

Teaticket/Acapesket Preliminary Evaluation (TASA TM-1)

Job Number	11153041
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LAND-USE MAP

Figure 4

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April 11, 2019

To:	Town of Falmouth, MA	Ref. No.:	11153041
From:	Anastasia Rudenko, P.E., BCEE, ENV SP	Tel:	774-470-1637
	J. Jefferson Gregg, P.E., BCEE		774-470-1640
CC:	File; Project Team		

Subject: **South Coast Embayments – Preliminary Evaluations and Notice of Project Change Update Project**

Teaticket / Acapesket Study Area Technical Memorandum No. 2 - Collection System Evaluation – Teaticket / Acapesket Study Area (TASA TM-2)

1. Purpose of Memo

The purpose of this Technical Memorandum is to summarize the collection and transmission system evaluation that was conducted for the Teaticket/Acapesket Study Area (TASA), which includes portions of the Great and Green Pond watersheds. This Memorandum also presents modifications to existing force mains.

1.1 References and Design Guidelines

The following references and design guidelines were used to develop this memorandum:

References:

- TASA TM No. 1 – South Coast Embayments Preliminary Evaluations and Notice of Project Change Project – Draft Service Area, Flow and N Load Evaluation, prepared by GHD, dated May 2018 (TASA TM-1)
- FEMA Flood Insurance Rate Map (FIRM) Number 25001C0729J, effective date July 16, 2014
- FEMA FIRM Number 25001C0737J, effective date July 16, 2014
- 'Woods Hole and Shiverick's Pond Force Mains – Contract No.3A' drawings prepared by Camp Dresser & McKee Inc., dated January 1983
- 'Route 28 Force Main – Contract No. 3B' drawings, prepared by Camp Dresser & McKee Inc., dated January 1983
- 'Woods Hole Sewer Force Main Rehabilitation' as-built drawings, prepared by Tata & Howard, dated September 2012
- Technical Memorandum PS-2 – New Lift Stations and Force Mains Basis of Design Little Pond Service Area, Town of Falmouth – Table 5, prepared by GHD and dated October 2. 2014

- 'Town of Falmouth, MA Woods Hole Force Main Evaluation,' prepared by GHD and dated May 26, 2017
- 'Altitude and Configuration of the Water Table, Western Cape Cod Aquifer, Massachusetts, March 1993, USGS Open-File Report 94-462'

<u>Design Guidelines:</u>

- New England Interstate Water Pollution Control Commission, TR-16: Guides for the Design of Wastewater Treatment Works, 2011 Edition as Revised in 2016

2. Background Information

This memorandum documents the development and evaluation of two preliminary collection system alternatives for the TASA. Preliminary collection system layouts were developed for the following two alternatives:

- A combination of gravity sewer plus low pressure sewers.
- Only gravity sewers.

Background information on both types of sewer systems is provided in this section. This memorandum will also provide cost estimates for modifications to the existing force main systems for the Jones-Palmer, Shivericks Pond, and Woods Hole lift stations. The evaluated modifications include:

- Addition of a second force main from Jones-Palmer Lift Station to the WWTF.
- Addition of a second force main from Shivericks Pond Lift Station to the Jones-Palmer Lift Station.
- Replacement of a segment of the existing Woods Hole ductile iron (DI) force main in the vicinity of Oyster Pond with HDPE.

2.1 Gravity Sewers and Lift Stations

A gravity sewer is installed at a constant slope, between manholes, until its depth becomes so great that a lift station is needed to "lift" the flow to a wastewater treatment plant, lift station, or another gravity sewer. In flat terrain, multiple lift stations may be required before the flow reaches a treatment facility. In most situations, homes along a gravity sewer connect into the system with gravity service connections from the building to the collector sewer. However, houses that are below the street elevation will require small pumps and a small diameter force main (1 to 2 inches) to discharge into the collector sewer.

2.2 Low Pressure Sewers with Packaged Grinder Pump Systems

For the purposes of this evaluation, buildings connected to the low pressure system need to have a grinder pump which is housed in a pump chamber (packaged grinder pump system). Wastewater flows from the building to the chamber by gravity, is "shredded," and then pumped into a common pressure sewer main in the street. The flow is discharged to a gravity main or directly into a lift station or treatment facility. This type of system is typically considered in areas where:

- There is a need to minimize excavation depths;
- There is a need to minimize the number of lift stations; or
- It is difficult to site a traditional gravity sewer and/or lift stations(s).

3. Potential Lift Station Sites

Potential lift station locations were identified based on siting considerations and during progress meetings with the Town. Each location was evaluated to identify potential advantages and constraints, as summarized in this section. The scope called for the identification of up to 20 lift station sites as part of this process.

3.1 Potential Lift Station Characterization Parameters

Potential advantages and constraints for each proposed lift station site were assessed based on the following parameters:

- Ownership
- Parcel Size
- Elevation
- Estimated Depth to Groundwater
- Proximity to Special Flood Hazard Areas (SFHAs)
- Proximity to Wetland Resource Areas
- Proximity to Vernal Pools

Each parameter is described in further detail below.

3.1.1 Ownership

In order to construct a lift station on an identified parcel, the Town either needs to own the parcel or to negotiate an agreement with the owner of the parcel to allow construction. Lift stations on parcels not owned by the Town would require either a property purchase or obtainment of an easement or taking, resulting in an increase to the overall cost of the lift stations construction.

The following types of parcels were considered advantageous in the analysis:

- Town-owned municipal parcels.
- Town-owned right-of-ways.

The following types of parcels were considered as having potential constraints in the analysis since they would require a property purchase, obtainment of an easement or a taking:

- Developable parcels (including currently undeveloped association-owned parcels).
- Undevelopable parcels.
- Non Town-owned right-of-ways.

3.1.2 Parcel Size

Parcel size (available area) and shape impacts the favorability of a site. Larger parcels are more favorable since they allow for greater flexibility in the layout of a lift station.

For this analysis, properties that are greater than 0.25 acres were considered advantageous while properties less than 0.25 acres were considered as having a potential constraint for the site.

3.1.3 Elevation

The topography of the TASA varies between sea level (0 feet) and approximately 45 feet above sea level. Lift stations are typically sited at low points in topography in order to maximize the extent of the area that can be served by a gravity connection. For the purposes of analysis, proposed lift station locations have been divided into two categories:

- “Low” sites—which are advantageous—are at a local low point in the surrounding topography.
- “High” sites—which have potential constraints—are at a local high point in the surrounding topography.

3.1.4 Estimated Depth to Groundwater

Groundwater in the Town of Falmouth generally flows in a southerly direction and drains into Vineyard Sound. The Groundwater contour map entitled “Altitude and Configuration of the Water Table, Western Cape Cod Aquifer, Massachusetts, March 1993, USGS Open-File Report 94-462” indicates the groundwater elevation in the TASA east of the Coonamessett River is generally less than 5 feet above Mean Sea Level (MSL). The groundwater elevation in TASA west of the Coonamessett River is generally less than 10 feet above MSL. Monitoring wells installed in the vicinity of Shorewood Drive indicate the groundwater elevation in this area is less than 3 feet above MSL. The USGS contour map was used to estimate depth to groundwater for each site. Potential lift station and deep sewer locations with groundwater in close proximity to the ground surface are anticipated to incur dewatering costs, increasing the overall construction cost and complexity of the project.

For this analysis, locations with an estimated depth to groundwater over 10 feet were considered advantageous. Sites with an estimated depth to groundwater less than 10 feet were considered as having a potential constraint.

3.1.5 Proximity to Special Flood Hazard Area (SFHA)

The Federal Emergency Management Agency (FEMA) defines the land area covered by the floodwaters of the Base Flood as a Special Flood Hazard Area (SFHA). The Base Flood is the 1-percent annual chance flooding event and is also commonly known as the 100-year flood event. The SFHA is broken down into three difference coastal flood zones, which are designated by wave height, as follows:

- AE Zone—Area with shallow flooding only, where potential for breaking waves and erosion is low. Wave height is expected to be less than 1.5 feet.
- Coastal AE Zone—Area with potential for breaking waves and erosion during the Base Flood. Wave height is expected to be 1.5 to 3.0 feet.
- VE Zone—Wave height is expected to be greater than 3.0 feet.

The Base Flood Elevation (BFE) shown on a FEMA Flood Insurance Rate Map (FIRM) includes the anticipated wave height for a given area. The AE Zone is separated from the Coastal AE Zone by a line depicting the Limit of Moderate Wave Action (LiMWA). Construction in the SFHA is regulated by the Massachusetts Building Code and would require provisions in the design to allow the lift station to withstand the Base Flood, which are anticipated to increase the construction costs of the station.

Sites outside of the SFHA were considered advantageous in this evaluation. Sites within the SFHA were considered to have potential constraints.

3.1.6 Proximity to a Wetland Resource Area

The State regulates activities that involve filling, dredging, or excavating in or near a wetland or water body. Wetland Resource Areas are protected by a 100-foot buffer zone in which landscape alterations are regulated. The regulations also govern additional construction activities, including site preparation, the removal of trees or bushes, vista pruning, and the changing of land contours. A Notice of Intent (NOI) must be filed for work in any resource area. The NOI requires a detailed description of the planned activity, and the applicant must show that if the resource area is altered, the benefits will outweigh the damage.

Sites outside the 100-foot buffer zone of a Wetland Resource Area were considered advantageous in this evaluation. Sites within the 100 foot buffer zone were considered to have potential constrains.

3.1.7 Proximity to Vernal Pools

Vernal pools are temporary bodies of freshwater that provide critical habitat for a number of vertebrate and invertebrate wildlife species. No vernal pools have been identified in the TASA.

3.1.8 Additional Considerations – Archaeological Sensitivity Assessment

An archaeological sensitivity assessment was completed for the TASA as part of this project by the Public Archaeology Laboratory (PAL). A literature search (which included the sources listed below) was conducted to identify available existing information on known cultural resources:

- Massachusetts Historical Commission – Inventory of the Historic and Archaeological Assets of the Commonwealth files.
- Archaeological resources that are listed or evaluated as eligible for listing in the State or National Registers, and surveyed properties that have not been evaluated for registration.
- Cultural Resource Management (CRM) reports salient to the Study Area.

- Town histories and historic maps.

The archaeological assessment identified five archaeological areas (four pre-contact and one post-contact), one historic architectural area, and six historic buildings in the TASA. It is recommended that the archaeological sensitivity for the TASA be further refined as project plans are finalized and specific areas are slated for ground disturbance and/or construction activities.

3.2 Potential Lift Station Locations and Sites Selected for Preliminary Layout

Potential lift station locations are shown in Figure 1. Potential advantages and constraints for the identified proposed lift station sites are summarized in Table 1 (attached). Potential advantages are highlighted in green, potential constraints are highlighted in yellow.

The sites outlined in green on Figure 1 were selected as proposed lift station locations for the preliminary collection system alternatives. These sites were chosen primarily because their location and topography maximize the portion of TASA that can potentially be served by gravity sewer. It should be noted that the majority of the proposed lift station sites are privately owned and would need to be purchased by the Town (or the Town would need to obtain an easement or taking).

4. Preliminary Collection System Layout

Section 4 outlines the methodology that was used to develop the preliminary collection system alternatives for TASA.

4.1 Gravity and Low Pressure Pipes

4.1.1 Gravity

SewerCAD was used to develop potential sanitary sewer layouts and to model the existing collection systems and proposed gravity systems. The model uses Manning’s Equation to estimate the hydraulic performance of gravity pipes/systems and is run assuming a “steady state” condition under peak flow conditions. Based on guidance from TR-16, wastewater flows are based on a combination of wastewater from sanitary sources and a conservative estimate of wet-weather flows (inflow and infiltration). Wastewater flows were established using water use data, as documented in Technical Memorandum 1. Flows are adjusted for different scenarios by applying peaking factors to represent different flow conditions (i.e. average annual, maximum month, peak hour) which can be universally applied to one, multiple, or all of the pipes in the model.

Typically 85 percent of the total pipe capacity is used to represent the design maximum flow of the pipe. When the wastewater flow exceeds this value, the pipe size is increased in the model. TR-16 requires a minimum pipe diameter of 8-inches. The pipe material selected for the model was polyvinyl chloride (PVC); currently the most common gravity pipe material because of its smooth interior surface (a Manning’s “n” value of 0.013 is used in calculations), durability, and light weight, which provides for an easier installation and lower shipping costs than other heavier materials that have been used in the past.

It is important to note that the SewerCAD model is only intended to reflect potential sewer alignments across the Study Area. During final design additional information will be required to assess the impacts of the proposed sewer layouts. This additional information includes, but is not limited to:

- Topography—The model uses contour elevations at each manhole to establish preliminary sewer profiles. A detailed survey will be required to identify localized low or high areas between the manhole locations which could potentially impact the feasibility of sewer construction. For example, a high point between manholes may result in an excessively deep excavation (greater than the maximum depth constraint) that might be impractical for a narrow road or location. Conversely, a low point between manholes may result in insufficient cover over the sewer or “daylighting.” Each of these situations would be identified during detailed design.
- Lift Station and Easement Locations—The model was developed based on the assumption that properties or easements identified as proposed lift station locations are available. If, during detailed design, it is concluded that the locations are unavailable the proposed sewer alignment will be affected.
- Utility Conflicts—Existing utilities, including stormwater systems, water mains, and gas lines may affect the sewer alignment. Existing utility locations, to the extent practical, will be identified through a detailed survey as part of final design.
- Design Decisions—Potential design decisions that can affect the preliminary sewer layout include, but are not limited to, accommodating private sewers; Town decisions to add lift stations, eliminate lift stations, or serve areas with low pressure sewers; and constructability issues.

4.2 Low Pressure

For areas where gravity sewers are not feasible or would result in excessive excavation depths, low pressure sewers are proposed. Under this approach each household served by low pressure sewers would have their own designated packaged grinder pump system. In order to develop the preliminary design layouts, low pressure sewer pipes are typically sized using a manufacturer supplied model. The Environmental-One (E-One) model software has been used by GHD for past projects in Falmouth. This model “sizes” the pressure sewers to maintain an adequate velocity throughout the system based on inputs for estimated flow values, estimated property connections, and pipe lengths. Low pressure sewer systems also require air relief valves at system high points and cleanouts, which are placed at the terminal ends of each main and at all sewer main intersections. Trenching depths are minimized by following the area’s topography and maintaining a minimum 4-foot depth of cover. Flows from low pressure sewer connections have been incorporated into the SewerCAD model by applying the flows to the nearest downstream gravity manhole as a “point load”.

4.3 Force Mains

For the calculation of friction loss and velocities in pressure pipes, SewerCAD utilizes the Hazen-Williams equation. When calculating friction loss, the model automatically identifies the length of the line representing the force main and begins its evaluation using a 4-inch DI pipe. Four-inch pipe is the minimum size considered for a force main in the model. Design velocities between 3- and 5-feet per second are considered acceptable. If the velocity exceeds 5 ft/s in the model, a larger pipe size is used. A Hazen-Williams coefficient of 120 was considered for this evaluation, which is typical for DI pipe. This coefficient represents the roughness of the pipe interior, similar to the Manning's coefficient.

Minor losses are the losses that are developed through bends and other fittings in the force main, as well as pipe entrances and contractions, and any other interior pipe condition that disrupts uniform flow in the pipeline. When the model calculates headloss, it calculates loss in feet for each force main.

The total loss of total dynamic head is the sum of the friction loss, minor loss, and static loss. The static loss is simply the difference in elevation between the water surface in the wet well and the invert of the structure it pumps into. The lift station needs to be able to convey the peak flow rate entering the station at the calculated dynamic head. When this condition is met, the system should function as designed.

4.4 Collection System Preliminary Design Parameters

Collection system preliminary design parameters are discussed below. Based on discussions with the Town, the use of gravity sewer was maximized in both conceptual layout alternatives. In order to maximize the number of properties connected to the system it was assumed that properties that cannot be serviced with a gravity connection, due to topography, will be served by a low pressure sewer in both alternatives. Houses that are adjacent to a gravity sewer but are located at a lower elevation than the sewer will require a packaged grinder pump system and small diameter service lateral (force main) to connect to a gravity service lateral for the system.

4.4.1 Alternative 1 – Gravity and Low Pressure Alternative

Based on TR-16 requirements, the following design parameters were established for this project:

- Pipe Slope, Minimum: 0.5% or 0.005 ft/ft (6-inch drop for every 100-foot span)
- Pipe Slope, Maximum: 10.0% or 0.100 ft/ft
- Minimum Cover: 4 feet
- Maximum Cover: 20 feet

TR-16 requires a minimum slope of 0.40% for an 8-inch pipe. A slightly steeper slope of 0.5% was used in the conceptual layouts to allow for some adjustment during detailed design and construction.

Wastewater flows established in TASA TM-1 using Town water data were input into the model. Flows were assigned to each parcel in the planning area. After the initial collection system layout was created, individual sewersheds were created that encapsulated groups of parcels. For modeling purposes, the flow of these parcels was summed and then assigned to a nearby manhole in each sewershed. This flow value was entered into each manhole as the sanitary flow. A peaking factor of 3.4 (as established in Technical Memorandum 1) was applied to simulate peak hour flows and to ensure that pipe sizes were adequate for such flows.

It is noted that minimal infiltration and inflow (I/I) will be evident in new infrastructure and proper municipal inspection of service laterals, but experience indicates that I/I increases during the long lifetime of a collection system. The TR-16 I/I factor of 500 gpd/in-dia/mile is used to model potential I/I throughout the life of the system.

The preliminary collection system layout for Alternative 1 is shown in Figure 2 and summarized in Table 2.

Table 2 – Alternative 1 Collection System Summary

Peninsula	Proposed Lift Station	Length of Gravity Sewer (LF)	Number of Parcels Connected to Gravity Sewer[1]	Number of Parcels Connected to Low Pressure Sewer	Average WW Flow (gpd)
Maravista	Spring Bars LS (Connection to Existing LS)	4,860	130	0	13,124
	Alphonse LS (Connection to Existing LS)	4,060	57	13	10,822
Teaticket	Saint Marks Road LS	12,940	203	18	23,787
	Falmouthport LS	6,280	18	0	17,615
	Teaticket LS	5,160	91	8	10,830
	Broken Bow Lane LS	2,530	46	3	6,716
	Village Commons LS	1,830	16	0	3,519
Acapesket	Shorewood LS	19,870	361	0	34,146
	Bridge Street LS	7,350	168	62	24,021
	Falmouth Hwy LS	8,560	120	37	19,255
	POD LS	9,260	232	0	16,467
	Acapesket LS	1,500	59	16	11,936
	Sykes Lane LS	2,460	67	15	11,944
	Garry Ave LS	3,700	52	1	7,331
Subtotal – Maravista Peninsula (Connection to Existing Lift Stations)		8,920	187	13	23,946
Subtotal – Teaticket Peninsula		28,740	374	29	62,467
Subtotal – Acapesket Peninsula		52,700	1,059	131	125,100
Total – All Lift Stations		**90,360**	**1,620**	**173**	**211,513**

Notes:

1. Houses that are adjacent to a gravity sewer but are located at a lower elevation than the sewer will require a small pump and small diameter force main to connect to the system.
2. Sewer lengths rounded to the nearest tens.

4.4.2 Alternative 2- Optimized Gravity Alternative

Alternative 2 was developed to maximize the gravity portion of the system. Based on discussions with the Town a maximum cover of 22 feet was established as a constraint. All other design parameters are consistent with Alternative 1.

The preliminary collection system layout for Alternative 2 is shown in Figure 3 and summarized in Table 3. Portions of the collection system anticipated to be deeper than 20 feet are indicated on the figure.

Table 3 – Alternative 2 Collection System Summary

Peninsula	Proposed Lift Station	Length of Gravity Sewer (LF)	Number of Parcels Connected to Gravity Sewer[1]	Number of Parcels Connected to Low Pressure Sewer	Average WW Flow (gpd)
Maravista	Spring Bars LS (Connection to Existing LS)	4,890	130	0	13,124
	Alphonse LS (Connection to Existing LS)	4,060	57	13	10,822
Teaticket	Saint Marks Road LS	12,940	203	18	23,787
	Falmouthport LS	6,280	18	0	17,615
	Teaticket LS	5,160	91	8	10,830
	Broken Bow Lane LS	2,730	46	3	6,716
	Village Commons LS	1,830	16	0	3,519
Acapesket	Shorewood LS	19,870	361	0	34,146
	Bridge Street LS	10,610	192	38	24,021
	Falmouth Hwy LS	9,850	157	0	19,255
	POD LS	9,260	232	0	16,467
	Acapesket LS	3,730	75	0	11,936
	Sykes Lane LS	2,460	67	15	11,944
	Garry Ave LS	3,700	53	0	7,331
Subtotal – Maravista Peninsula (Connection to Existing Lift Stations)		8,950	187	13	23,946
Subtotal – Teaticket Peninsula		28,940	374	29	62,467
Subtotal – Acapesket Peninsula		59,480	1,137	53	125,100
Total – All Lift Stations		**97,370**	**1,698**	**95**	**211,513**

Notes:

1. Houses that are adjacent to a gravity sewer but are located at a lower elevation than the sewer will require a small pump and small diameter force main to connect to the system.
2. Sewer lengths rounded to the nearest tens.

5. Force Main Evaluations

Section 5 summarizes the conceptual design for the force main from TASA to the WWTF and provides cost estimates for modifications to the existing force main systems for the Jones-Palmer, Shivericks Pond, and Woods Hole lift stations.

5.1 Direct Force Main to WWTF

The conceptual force main layout for TASA is outlined in Figure 4. The majority of the wastewater collected by the TASA collection system will be conveyed to the Falmouth WWTF through new dual force mains from a proposed booster lift station on the Augusta parcel. The conceptual force main layout includes one aerial crossing of the Coonamessett River.

5.2 Connections to Existing Gravity Sewer

A portion of the flow collected in the TASA will be conveyed to two existing lift stations—Alphonse Street and Spring Bars Road. The estimated wastewater flow from TASA conveyed to the two existing stations is based on a combination of sanitary flow and an allowance for infiltration/inflow (I/I). Wastewater flow estimates were developed as part of Technical Memorandum 1.

Table 4 summarizes the design capacities of the two stations, projected flows to the stations from the existing collection system, and proposed flow from TASA. As outlined in Table 4, the two existing lift stations have adequate capacity to convey the proposed flow from TASA.

Table 4 – Existing Lift Station Capacity

Lift Station	Design Capacity (gpm)[1]	Estimated Peak Flow + Infiltration (gpm)[1]	TASA Peak Flow + Infiltration (gpm)	Estimated Peak Flow (with TASA Flow) + Infiltration (gpm)	Remaining Pump Capacity (with TASA Flow) (gpm)
Alphonse Street Lift Station	120	60	30	90	30
Spring Bars Road Lift Station	400	370	30	400	0

Notes:
1. Reference: Technical Memorandum PS-2 – New Lift Stations and Force Mains Basis of Design Little Pond Service Area, Town of Falmouth – Table 5, prepared by GHD and dated October 2, 2014.
2. Flow estimates have been rounded to the nearest ten.

5.3 Existing Force Main Modifications

The Town requested additional preliminary cost estimations for three of their major force main systems which convey wastewater flow from the Jones-Palmer, Shivericks Pond, and Woods Hole lift stations.

The existing Woods Hole Lift Station pumps wastewater through a 12-inch DI force main which follows the Shining Sea Bikeway along the coast north into Falmouth and along Palmer Avenue. Due to corrosion of the exterior of the pipe, approximately 1,200 feet of the Woods Hole force main was rehabilitated using cured-in-place lining.

At the intersection with Katherine Lee Bates Road, the Shivericks Pond 14-inch DI force main meets the Woods Hole force main, increases to a 16-inch DI force main, and continues north to the Jones-Palmer Lift Station. The Jones-Palmer Lift Station force main conveys flow from the Jones-Palmer Lift Station (including flow from Woods Hole and Shivericks Pond systems) north to the WWTF through an 18-inch DI force main.

The three force mains are critical components of Falmouth's wastewater infrastructure which currently do not have any redundancy. In order to provide redundancy for critical portions of the existing collection system, capital costs were estimated for the following:

- A second 18-inch force main from Jones-Palmer Lift Station to the WWTF.
- A second force main from Shivericks Pond Lift Station to Jones-Palmer Lift Station, matching the 14-inch DI force main on Katherine Lee Bates Road and the 16-inch DI force main on Palmer Avenue.

Additionally, the cost to replace a portion of the Woods Hole force main from the Woods Hole Lift Station to the intersection of the Shining Sea Bikeway and Elm Road with a corrosion-resistant material (HDPE) was evaluated. Due to the length of the Woods Hole force main a dual force main should be considered for this location in the future. The cost estimate does not include replacement of the existing 8-inch and 10-inch force mains under Eel Pond. Due to the complexity and anticipated high cost of replacing the existing infrastructure under Eel Pond the Town should consider alternative options for replacement of this portion of infrastructure, including an alternative force main route around Eel Pond (along Millfield Street and School Street).

A conceptual layout for the proposed existing force main modifications is shown in Figure 5. Capital costs for the proposed conceptual existing force main modifications, which are presented in Table 5, are the total estimated project costs with allowances for construction costs including: a 30 percent construction contingency, 10 percent engineering design, 2 percent fiscal/legal/permitting/administrative costs, 4 percent survey and soil boring costs (for design), 12 percent construction administration and Resident Project Representative (RPR) costs, and 5 percent police detail costs. Because of the conceptual nature of this evaluation, a 30 percent contingency is carried as no detailed design has been performed. During final design a reduced contingency will be carried for variability in the bidding climate, project changes before bidding, easements, residential property restoration, and change orders due to unforeseen conditions. Project costs are presented in 2018 dollars. Once the construction timeframe is known, project costs should be adjusted to the mid-point of construction.

The following basis of design was used for the cost estimates:

- Dual force main routing is assumed to match existing force main sizing and routing.
- It was assumed dual force main connections would consist of buried valves and interior piping. No vaults or other sub-surface structures were included in the cost estimates.
- It was assumed that the Jones-Palmer dual force main would connect to the manhole where the existing force main transitions to gravity (upstream of the WWTF). Additional interconnection options will be evaluated as part of Technical Memorandum No. 4 – WWTF Evaluation, which will increase the cost estimate.
- It was assumed that the Shivericks Pond dual force main is routed directly to Jones-Palmer Lift Station. An interconnection with the Woods Hole force main was not included in the cost estimate.
- It was assumed that the new Woods Hole HDPE force main ties into the existing force main infrastructure underneath Eel Pond.
- Trench paving only was assumed. Full width overlay or other road reconstruction options were not factored into the cost estimates.

Table 5 – Conceptual Force Main Modifications Cost Estimates

	Redundant Force Main from Jones-Palmer Lift Station to WWTF[2]	Redundant Force Main from Shivericks Pond Lift Station to Jones-Palmer Lift Station	Replacement HDPE Force Main from Woods Hole Life Station to Oyster Pond
Construction Costs (2018$)	$8,500,000	$2,300,000	$2,500,000
Total Capital Costs (2018$)	$11,200,000	$3,040,000	$3,310,000
Construction Costs – Midpoint of Construction	TBD	TBD	TBD
Total Capital Costs – Midpoint of Construction	TBD	TBD	TBD

Notes:

1. All cost are shown in 2018 dollars. Once a construction timeframe is known for the project, costs should be adjusted to the mid-point of construction.
2. Cost estimate for the Jones-Palmer force main will be updated as part of Technical Memorandum No. 4 as additional modifications at the WWTF will be necessary to accommodate the TASA flow and second force main.
3. TBD = To Be Determined
4. It was assumed that the new HDPE force main ties into the existing force main infrastructure underneath Eel Pond.
5. Estimated project costs does not include utility relocation.

6. Summary

Two sewer alternatives were developed for the TASA. Both alternatives were designed to maximize the number of properties served by gravity. Alternative 1 was developed with a maximum depth of 20 feet. Alternative 2 was developed with a maximum depth of 22 feet. All other design variables remained consistent between the two alternatives.

Comparison of the alternatives indicates that an additional 78 parcels can be served by gravity if the maximum depth is increased to 22 feet (Alternative 2). In this alternative, six manholes would be constructed between 20 and 22 feet deep. Conceptual level cost estimates will be developed for both options as part of TASA TM-5 to determine which alternative is more cost-effective.

Table and Figures

Table 1 – Proposed Lift Station Sites
Figure 1 – Potential Lift Station Locations
Figure 2 – Collection System Alternative 1
Figure 3 – Collection System Alternative 2
Figure 4 – Conceptual Force Main Route – TASA to Falmouth WWTF
Figure 5 – Existing Force Main System Modifications – Conceptual Layout

DRAFT

Table 1 Proposed Lift Station Sites

#	Location	Elevation	Ownership	Parcel Size – Greater than 0.25 acres?	Estimated Depth to Groundwater (ft) – Greater than 10 feet?	Entire property Outside of a Special Flood Hazard Area?[1]	Entire Property Outside of a Wetland Resource Area?	Vernal Pool on Property?	Proposed LS Name in Conceptual Layouts	Comments
1	0 Bridge Street	High	Private (Developable)	Yes	No	No	Yes	No	Bridge Street LS	
2	"0" Acapesket Road (4 parcels between Jenkins Street and Bogs Street)	Low	Private (Developable)	Yes	Yes	Yes	Yes	No	Acapesket Road LS	
3	"0" Acapesket Road (parcel between Green Cove Land and Sykes Lane)	Low	Private (Developable)	Yes	Yes	Yes	Yes	No	Sykes Lane LS	
4	333 Shorewood Drive	Low	Private	Yes	No	No	No	No	Shorewood Drive LS	
5	ROW at Shorewood Drive and Starboard Drive ("Pile of Dirt" - POD Site)	Low	Right of Way (Town-owned)	No	No	No	Yes	No	POD LS	
6	35 East Falmouth Highway	Low	Private	Yes	No	Yes	Yes	No	Falmouth Hwy LS	Parcel is surrounded on three sides by an undeveloped municipal parcel (0 Shorewood Drive) which provides a buffer
7	34 Sandy Reach (Falmouthport)	Low	Private	Yes	No	No	No	No	Falmouthport LS	
8	0 Saint Marks Road (7 parcels)	High	Private (Developable)	Yes	Yes	Yes	Yes	No	Saint Marks Road LS	
9	20 Village Commons Drive	Low	Private	Yes	No	Yes	Yes	No	Village Common Lane LS	
10	0 Broken Bow Lane	Low	Private (Developable)	Yes	No	Yes	Yes	No	Broken Bow Lane LS	
11	60 Teaticket Path	Low	Private (Developable)	Yes	No	Yes	Yes	No	Teaticket Path LS	
12	632 Teaticket Highway ("Augusta" Parcel)	High	Municipal	Yes	Yes	Yes	Yes	No	Augusta (Booster) Station	
13	35 Garry Avenue	High	Private	Yes	Yes	Yes	Yes	No	Garry Avenue LS	
14	Oxbow Street Right of Way	Low	Right of Way	Yes	No	No	No	No	Not Used in Conceptual Layout	
15	30 Estrella Lane	High	Private (Undevelopable)	No	Yes	Yes	Yes	No	Not Used in Conceptual Layout	Parcel is at high point of surrounding streets – would serve a minimal sewershed.
16	7 Moniz Way	High	Private (Undevelopable)	No	Yes	Yes	Yes	No	Not Used in Conceptual Layout	
17	246 Teaticket Path	Low	Private (Undevelopable)	Yes	No	No	Yes	No	Not Used in Conceptual Layout	
18	27 – 35 Playstead Land	Low	Private (Undevelopable)	Yes	No	No	Yes	No	Not Used in Conceptual Layout	No paved access to properties. Access road in SFHA – lift station would be inaccessible by road during a 100 year flood event.
19	Right of Way on Alcott Road between Shorewood Drive and Cooper Road	Low	Right of Way	No	No	Yes	Yes	No	Not Used in Conceptual Layout	
20	Carol Avenue (no street number listed in assessor data)	Low	Municipal	Yes	Yes	Yes	Yes	No	Not Used in Conceptual Layout	No existing paved road access to parcel
21	0 East Harbor Drive	Low	Private (Undevelopable)	No	Yes	Yes	Yes	No	Not Used in Conceptual Layout	Small parcel close in close proximity to developed residential parcels

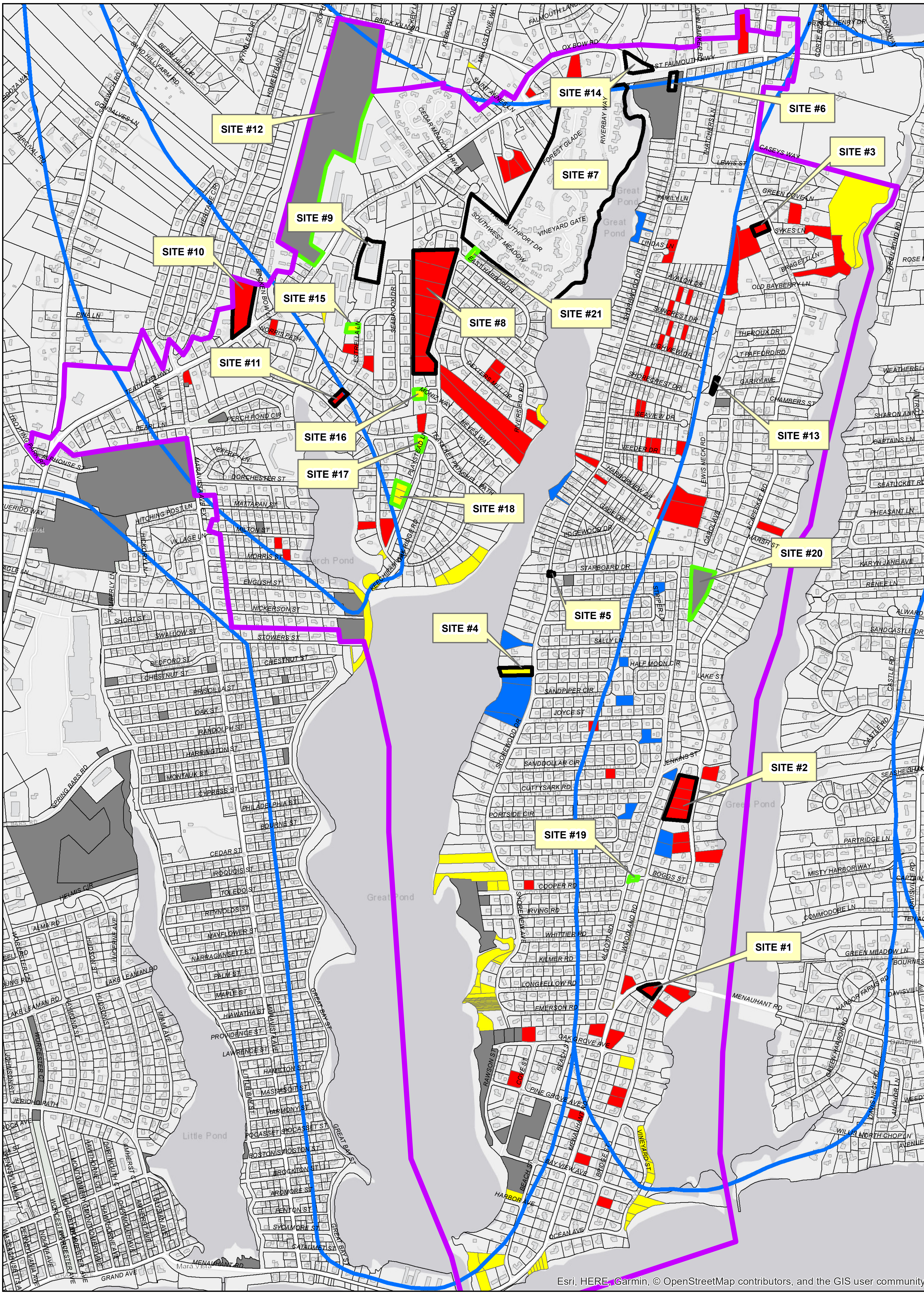
Legend:

Potential advantages highlighted in green.

Potential constraints highlighted in yellow.

Notes:

1. All potential lift station locations are outside the VE zone.



LEGEND

- Teaticket /Acapesket Study Area (TASA)
- Potential Lift Station Locations Identified By WQMC
- Undevelopable Parcel
- Developable Parcel
- Municipal Parcel
- Watershed Boundary
- Parcel Boundary
- Potential Lift Station Location-Selected for Conceptual Collection System Layout
- Potential Lift Station Location-Not Selected for Conceptual Collection System Layout

Paper Size ARCH D

0 450 900 1,800 Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

GHD

Town of Falmouth, MA
Teaticket/Acapesket Preliminary Evaluation (TASA TM-2)

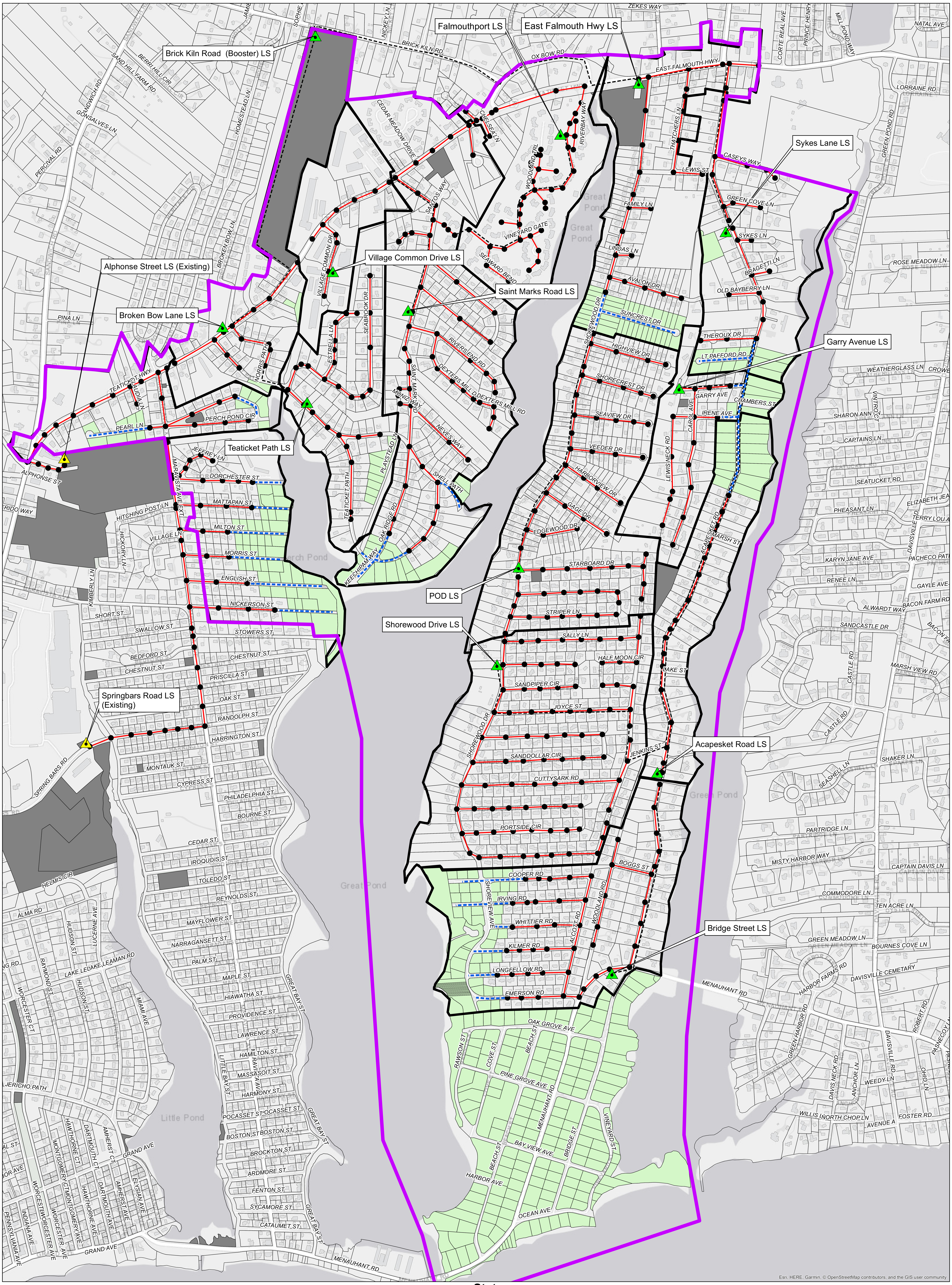
POTENTIAL LIFT STATION LOCATIONS

Job Number	111-53041
Revision	A
Date	14 Feb 2019

Figure 1

G:\111\11153041 Town of Falmouth South Coast CWMP Update\GIS\Maps\MXD_Deliverables\July 2018\111-53041-F01.mxd

1545 Iyannough Road, Hyannis Massachusetts 02601 USA T 1 508 362 5680 F 1 508 362 5684 E hyamail@ghd.com W www.ghd.com



LEGEND

- Teaticket/Acapesket Study Area (TASA)
- Sewershed Boundary
- Parcels Not Connected to Gravity Sewer
- Gravity Sewer MH
- Low Pressure Sewer
- Gravity Sewer
- Force Main

Status

- Existing Lift Station
- Proposed Lift Station
- Municipal Parcel

Paper Size ARCH D

0 420 840 1,680 Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

GHD

Town of Falmouth, MA
Teaticket/Acapesket Preliminary Evaluation (TASA TM-2)

COLLECTION SYSTEM ALTERNATIVE #1
(MAXIMUM DEPTH 20 FT)

Job Number	111-53041
Revision	A
Date	14 Feb 2019

Figure 2

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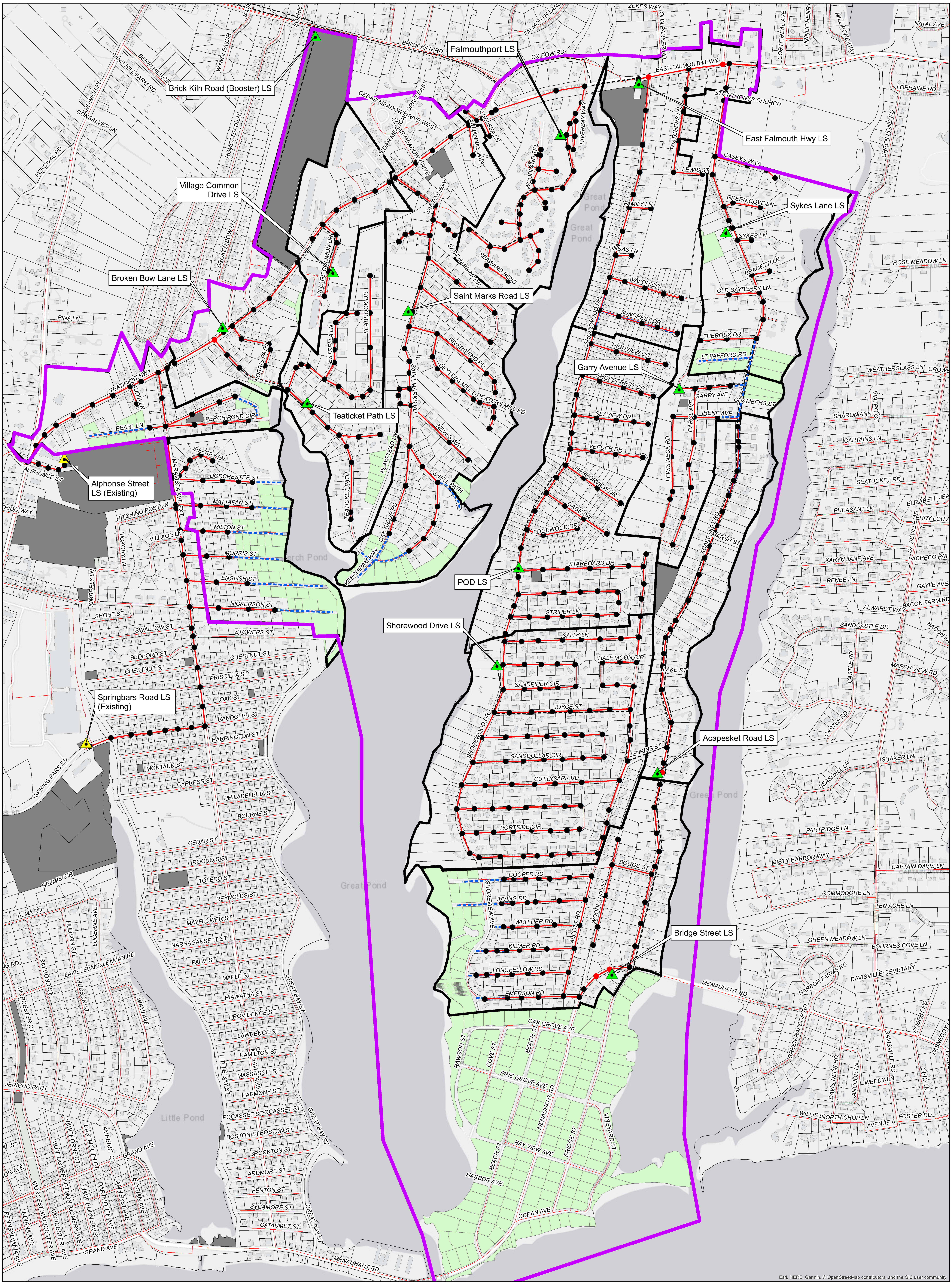

LEGEND

- Teaticket/Acapesket Study Area (TASA)
- Sewershed Boundary
- Parcels Not Connected to Gravity Sewer
- Municipal Parcel
- Gravity Sewer
- Low Pressure Sewer
- Force Main
- Gravity Sewer MH Less Than or Equal to 20 Ft Depth
- Gravity Sewer MH Greater Than 20 Ft Depth
- Existing Lift Station
- Proposed Lift Station

Paper Size ARCH D

0 420 840 1,680 Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

GHD

Town of Falmouth, MA
Teaticket/Acapesket Preliminary Evaluation (TASA TM-2)

COLLECTION SYSTEM ALTERNATIVE #2 (MAXIMUM DEPTH 22 FT)

Job Number	111-53041
Revision	A
Date	14 Feb 2019

Figure 3

G:\111\11153041 Town of Falmouth South Coast CWMP Update\GIS\Maps\MXD_Deliverables\July 2018\111-53041-F03.mxd

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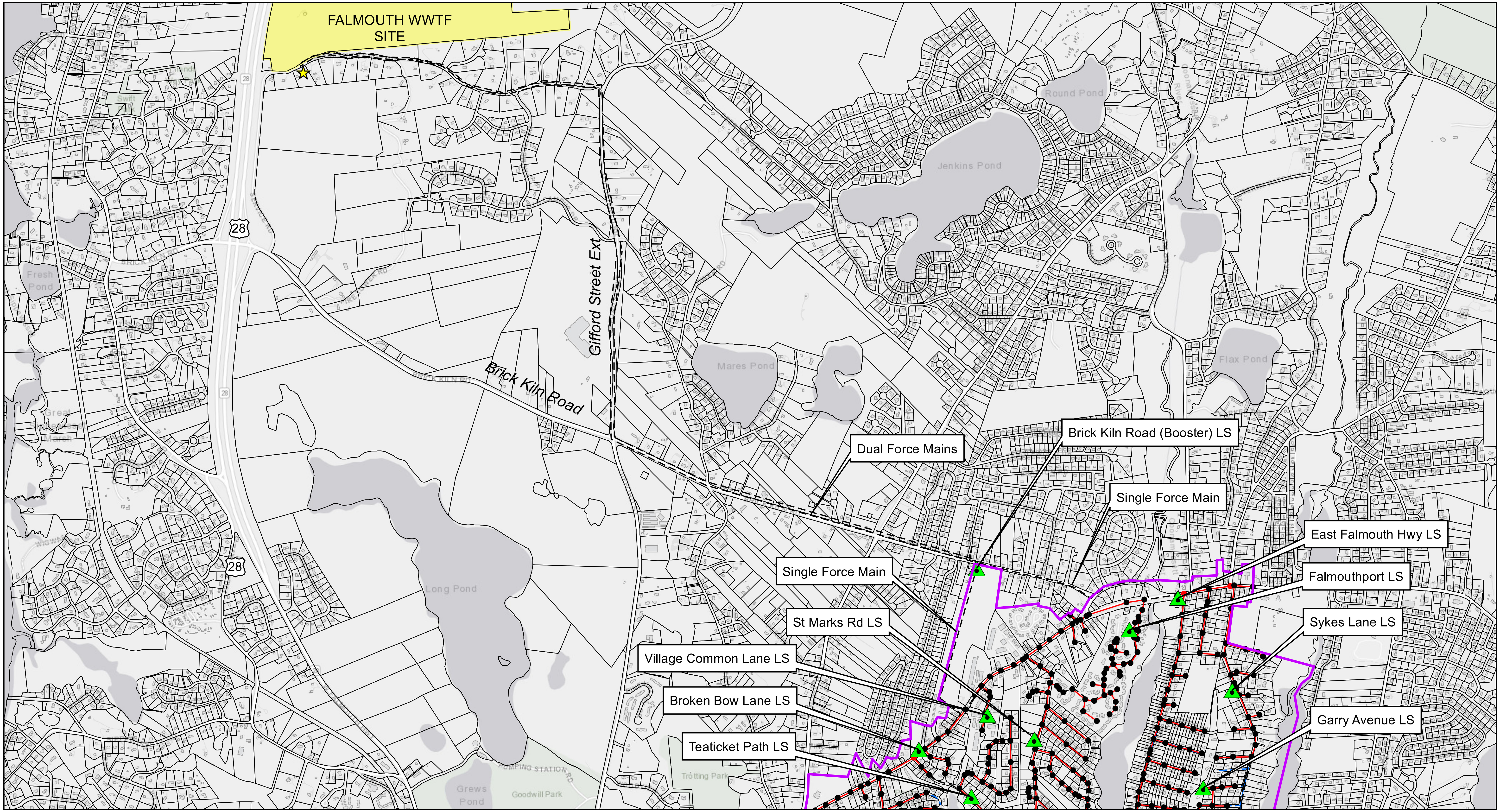

Paper Size ANSI B

0 750 1,500 3,000 Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

LEGEND

- Lift Station
- Gravity MH < 20 Ft Depth
- Gravity MH > 20 Ft Depth
- Gravity Sewer
- Teaticket/Acapesket Study Area (TASA)
- Force Main
- Low Pressure Sewer

Town of Falmouth, MA
Teaticket/Acapesket Preliminary Evaluation (TASA TM-2)

CONCEPTUAL FORCE MAIN ROUTE
TASA TO FALMOUTH WWTF

Job Number	111-53041
Revision	A
Date	14 Feb 2019

Figure 4

G:\111\11153041 Town of Falmouth South Coast CWMP Update\GIS\Maps\MXD_Deliverables\July 2018\111-53041-F04.mxd

Data source: Data Custodian, Data Set Name/Title, Version/Date. Created by:jjobrien

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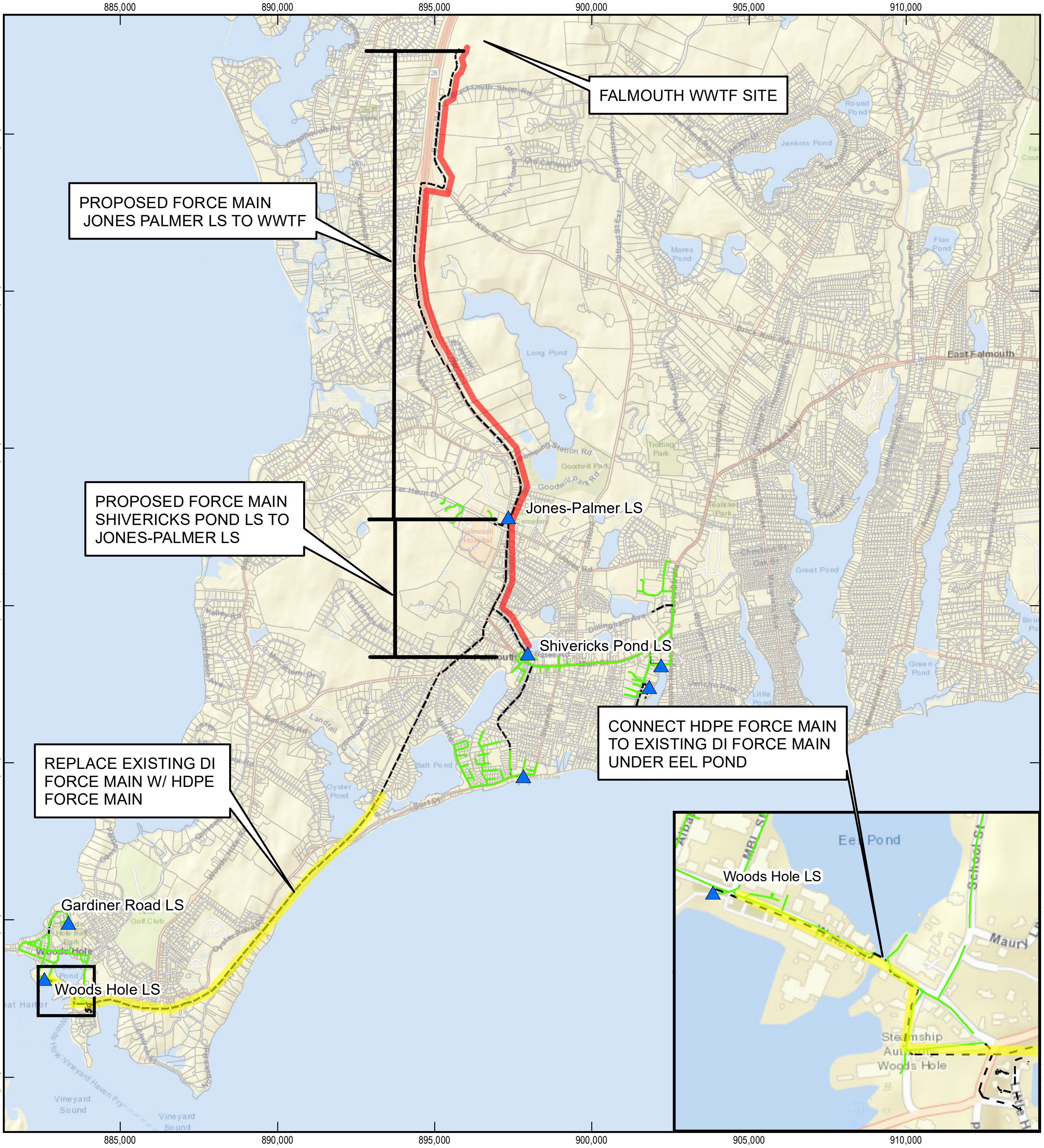

Legend (Proposed)

Replace Existing Force Main

Install New Force Main

Legend (Existing)

FORCE MAIN

GRAVITY

Paper Size ANSI A

0 810 1,620 2,430 3,240 4,050

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

GHD

Town of Falmouth, MA
Teaticket/Acapasket Preliminary Evaluation (TASA TM-2)

Job Number	11134465
Revision	A
Date	Feb 14, 2019

EXISTING FORCE MAIN SYSTEM
MODIFICATIONS-CONCEPTUAL LAYOUT

Figure 5

Created by:jjobrien

1545 Iyannough Road, Hyannis Massachusetts 02601 USA T 1 508 362 5680 F 1 508 362 5684 E hyamail@ghd.com W www.ghd.com

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TASA TM-3

April 11, 2019

To:	Town of Falmouth, MA	Ref. No.:	11153041
From:	Anastasia Rudenko, P.E., BCEE, ENV SP	Tel:	774-470-1637
	J. Jefferson Gregg, P.E., BCEE		774-470-1640
CC:	File; Project Team		

Subject: **South Coast Embayments – Preliminary Evaluations and Notice of Project Change Project**

Teaticket / Acapesket Study Area Discharge Technologies Evaluation – Technical Memorandum No. 3 (TASA TM-3)

1. Purpose of Memo

The purpose of TASA Technical Memorandum 3 (TASA TM-3) is to summarize the updated discharge technologies evaluation that was conducted for the Teaticket/Acapesket Study Area (TASA), which includes portions of the Great and Green Pond watersheds.

1.1 References, Regulations, and Design Guidelines

The references, regulations, and design guidelines listed below were used to develop this memorandum. Documents are referred to by the abbreviation indicated in parenthesis for the remainder of the memorandum.

References:

- TASA TM No. 6 – South Coast Embayments Preliminary Evaluations and Notice of Project Change Project – Draft Outfall Conceptual Cost Evaluation, prepared by GHD, dated November 2018 (TASA TM-6)
- TASA TM No. 5 – South Coast Embayments Preliminary Evaluations and Notice of Project Change Project – Draft Hydraulic Load Tests at the Augusta Parcel an Allen Parcel, prepared by GHD, dated November 2018 (TASA TM-5)
- TASA TM No. 4 – South Coast Embayments Preliminary Evaluations and Notice of Project Change Project – Draft WWTF Evaluation, prepared by GHD, dated October 2018 (TASA TM-4)
- TASA TM No. 1 – South Coast Embayments Preliminary Evaluations and Notice of Project Change Project – Draft Service Area, Flow and N Load Evaluation, prepared by GHD, dated May 2018 (TASA TM-1)

- Shared Wastewater Management Study – Towns of Bourne, Falmouth, Mashpee, Sandwich and Joint Base Cape Cod, prepared by Wright Pierce, dated November 2017 (2017 Wright Pierce JBCC Study)
- Recharge Beds 14 and 15 Operations and Maintenance Manual, prepared by GHD, dated June 2017
- Modified Individual Groundwater Discharge Permit No. 168-5, effective date December 22, 2015 (2015 Permit)
- Comprehensive Wastewater Management Plan and Final Environmental Impact Report and Targeted Watershed Management Plan – Little Pond, Great Pond, Green Pond, Bournes Pond, Eel Pond, and Waquoit Bay Watersheds and Recommendations for West Falmouth Harbor Watershed, prepared by GHD, dated September 2013. (2013 CWMP/FEIR/TWMP)
- Technical Memorandum No. 7 – Evaluation and Summary of Committee’s Preferred Option, prepared by Stearns & Wheler, dated October 20, 2010 (2010 TM-7)
- Technical Memorandum No. 1 – Cost Summaries for Alternative Wastewater Management Scenarios, prepared by Stearns & Wheler, dated October 7, 2010 (2010 TM-1)
- Comprehensive Wastewater Management Plan Review Committee Findings (revised 8/31/10, 9/7/10) Part 1: Design and Technical Principles (2010 CWMP Committee Findings)
- Groundwater Modeling to Support Comprehensive Wastewater Management Planning Guidance Document and Case Study Report, prepared by Stearns & Wheler GHD and Watershed Hydrogeologic Inc., dated June 2009 (2009 GW Modeling Report)
- Little Pond, Great Pond, Green Pond, Bournes Pond, Eel Pond, and Waquoit Bay Watersheds Alternatives Screening Analysis Report, prepared by GHD (formally Stearns & Wheler, LLC), dated November 2007 (2007 ASAR)
- Little Pond, Great Pond, Green Pond, Bournes Pond, Eel Pond and Waquoit Bay Watersheds Needs Assessment Report, prepared by Stearns & Wheler, dated October 2007 (2007 NAR)
- Falmouth Wastewater Treatment Facility Operation & Maintenance Manual – Draft, prepared by MaGuire Group Inc., dated 2007 (2007 Draft WWTF O&M Manual)
- Falmouth Country Club Conservation Restriction’, dated 2007 (FCC Restriction)
- Plan of Land Prepared for Town of Falmouth in Falmouth, MA Being a Subdivision of Lot 1 as Shown on L.C. Plan 39540-A, prepared by Holmes and McGrath, Inc., dated May 25, 2005 (2005 Allen Parcel Plan of Land)
- Article 5 and the Vote Thereon at the Special Town Meeting Held in Falmouth April 13, 2004
- Quitclaim Deed for the Augusta Parcel, dated August 2003
- Cape Cod Commission Modification of Development of Regional Impact Decision for the Falmouth Country Club – TR96021, dated 1999 Article 20 and the Vote Thereon at the Annual Town Meeting Convened at Falmouth, Massachusetts April 7, 2003
- Falmouth Conservation Commission Order of Conditions, dated 1997

- Plan of Land in Falmouth, drawing, prepared by Philip D. Holmes, dated December 1976
- Plan of Land for the Northern Trust Company in East Falmouth, Mass drawing, prepared by Philip D. Holmes, dated December 1976
- Sewage Disposal in Falmouth Massachusetts – III. Predicted Effects of Inland Disposal and sea Outfall on Groundwater, Robert Meade and Ralph Vaccaro, published in the Boston Society of Civil Engineers October 1971 (Meade & Vaccaro, 1971)
- Sewage Disposal in Falmouth Massachusetts – II. Predicted Effect of the Proposed Outfall, W. Redwood Wright, Dean F. Bumpus and Ralph Vaccaro, published in the Boston Society of Civil Engineers October 1971 (Bumpus, Wright & Vaccaro, 1971)
- Sewage Disposal in Falmouth Massachusetts – I. “Expert” Opinion and Public Policy, W. Redwood Wright, Dean F. Bumpus and Ralph Vaccaro, published in the Boston Society of Civil Engineers October 1971 (Bumpus & Vaccaro, 1971)
- Cape & Vineyard Electric Company Right of Way for Falmouth – Hatchville 23 k.v. Electric Line Through the Town of Falmouth drawing, prepared by Walter E Howlet B. Associates, revised March 19, 1965

Regulations:

- 314 CMR 5.00 – Groundwater Discharge Permits, effective December 2, 2016 (314 CMR 5.00)
- 310 CMR 27.00 – Underground Injection Control Regulations, dated September 23, 2016 (310 CMR 27.00)
- 314 CMR 4.00 – Massachusetts Surface Water Quality Standards, effective December 6, 2013 (314 CMR 4.00)
- 314 CMR 20.00 – Reclaimed Water Permit Program and Standards, effective March 20, 2009 (314 CMR 20.00)
- Massachusetts Ocean Sanctuaries Act (M.G.L. c132A) (Massachusetts Ocean Sanctuaries Act)
- 40 CFR Part 147 - State, Tribal and EPA-Administered Underground Injection Control Programs (40 CFR Part 147)

Design Guidelines:

- Guidelines for the Design, Construction, Operation, and Maintenance of Small Wastewater Treatment Facilities with Land Disposal, prepared by MassDEP, as revised in July 2018. (2018 MassDEP Small WWTF Design Guidelines)
- Massachusetts Department of Environmental Protection (MassDEP) Standard Design Guidelines for Shallow UIC Class V Injection Wells’ https://www.mass.gov/service-details/standard-design-guidelines-for-shallow-uic-class-v-injection-wells, 2018 (MassDEP Shallow Injection Well Guidelines)
- New England Interstate Water Pollution Control Commission, TR-16: Guides for the Design of Wastewater Treatment Works, 2011 Edition as revised in 2016. (TR-16)

- 'Title 5 Pressure Distribution Design Guidance', prepared by MassDEP, dated 2002

2. Discharge Technology Evaluation Update

Effluent discharge technology options were initially evaluated as part of the 2007 Alternatives Screening Analysis Report (ASAR). This section documents the updated discharge technology evaluation that was conducted based on a review of regulations and industry design guidelines (as of August 2018). To maintain consistency throughout the document each discharge technology is referred to by the name used in the 2018 MassDEP Small WWTF Design Guidelines.

2.1 Land Application

The 2018 MassDEP Small WWTF Design Guidelines list the following technologies as approved methods of effluent land disposal:

- Open sand beds
- Leaching facilities (leaching pits, leaching trenches, leaching chambers)
- Drip dispersal or other approved subsurface methods
- Reclaimed water uses consistent with MassDEP policies (including spray irrigation)

Alternative methods of effluent disposal through land application, including well injection and wick wells, are allowable on a case-by-case basis provided that adequate pilot test results at the proposed discharge site (performed with MassDEP approval) are provided or adequate experience at similar locations exists. Discharge sites within a Zone II need to meet a higher effluent standard than a site outside of a Zone II. All discharge sites currently under consideration are outside of a Zone II.

2.1.1 Open Sand Beds

Open sand beds—also known as sand infiltration beds, surface infiltration beds, recharge beds, or rapid infiltration beds—are open basins designed to allow treated effluent to flow across the bottom of the basin and infiltrate through the sand bed and the unsaturated zone to the groundwater. Open sand beds are typically operated year-round. Bed operation and maintenance (O&M) is relatively simple because the bed is exposed at the surface and the sand surface can be raked or replaced if the sand becomes plugged with effluent solids, debris, or vegetative growth. The Town currently has 15 open sand beds at the Falmouth WWTF.

A hydraulic loading rate of 5 gallons per day per square foot (gpd/sf) of bed area is typically allowed by MassDEP unless hydrogeologic tests demonstrate a greater infiltration loading capacity at the specific site. Advantages and disadvantages of open sand beds are summarized below in the following table.

Table 2.1 Open Sand Beds Summary

Discharge Technology	Advantages	Disadvantages
Open Sand Beds	• Relatively high hydraulic loading rates on sites with good permeability and sufficient depth to groundwater. • Town of Falmouth currently uses these at the WWTF and is familiar with the technology.	• Large surface footprint requirements, which may have a visual and environmental impact. • No secondary uses of the land. • Effluent disinfection required. • If the discharge site is located in a nitrogen-sensitive watershed, effluent will likely need to be treated to a very low effluent nitrogen limit. • If discharge site is located in a Zone II protection area, effluent filtration is required per 314 CMR 5.00. An effluent Total Organic Carbon (TOC) limit will also need to be met.

2.1.2 Spray Irrigation

A spray irrigation system is comprised of effluent pumps, distribution piping, and a spray system consisting of risers and spray nozzles. Treated effluent is pumped through distribution lines and discharged via spray nozzles to the surrounding surface area. Spray irrigation systems may be suitable for golf courses and in large remote fields during the growing season. During the winter (non-growing season) effluent would need to be stored or discharged at a different location through an alternate technology.

314 CMR 20.00 – Reclaimed Water Program and Standards, which was most recently updated in 2009, classifies irrigation uses into three categories:

- Class A – Locations where individual members of the public are likely to come into contact with the reclaimed water.
- Class B – Locations where individual members of the public are not likely to come into contact with the reclaimed water.
- Class C – Agricultural irrigation with restrictions and silviculture (growing and cultivation of trees).

Spray irrigation was utilized at the Falmouth WWTF for a period of time starting in 1988 and has been discontinued. The spray irrigation system had a design average annual capacity of 0.5 mgd and was designed to be used from March to November. Two storage ponds at the WWTF were intended to provide storage capacity for effluent during winter when the system was not in use. It should be noted that if spray irrigation was reinstated at a new location, an additional disinfection process would be required to meet the fecal coliform requirement for Class A or Class B reclaimed water.

The Application rate for non-golf course areas is typically 2 inches per acre per week. Application rates for golf courses are typically based on turf management needs. Effluent disposal through spray irrigation is limited to the growing season.

Advantages and disadvantages of spray irrigation systems are summarized in the following table.

Table 2.2 Spray Irrigation Summary

Discharge Technology	Advantages	Disadvantages
Spray Irrigation	• Land can be utilized for secondary uses such as golf courses. • Reduces potable water demands. • Provides nitrogen uptake by plant life and reduces need for fertilizers at golf courses. • Falmouth has past experience with this type of system.	• Difficult to secure suitable and authorized locations for spray irrigation. • Limited cold weather use due to potential freezing. • Limited use during site's secondary use (for example spray irrigation on a golf course fairway likely cannot be operating when people are golfing). • Spray nozzles subject to clogging. • Requires secondary method of discharge or storage during winter months or during the site's secondary use. • If discharge site is located in a nutrient-sensitive watershed, effluent will likely need to be treated to a very low effluent nitrogen limit. • If discharge site is located in a Zone II protection area, effluent filtration and disinfection are required per 314 CMR 05. An effluent TOC limit will also need to be met. • Discharge needs to meet reclaimed water effluent limits. • Pressurized systems often require more complex system balancing, and equalization/storage may be necessary to provide equal distribution throughout the system.

2.1.3 Leaching Facilities

Leaching facilities—also known as subsurface infiltration—typically utilize pump and piping systems to pressure dose sub-surface infiltration areas (trenches, chambers, or pits) which percolate to groundwater. Maintenance and cleaning of these systems is more difficult than surface systems because the infiltration area is below-grade and effluent solids cannot be easily removed. Leaching facilities can have secondary uses, such as parking lots, lawns, playing fields, and recreational areas.

The Town currently operates this type of facility at the New Silver Beach WWTF, which has leaching trenches installed below a school youth soccer field.

Hydraulic loading rates of 2½ to 3 gallons per day per square foot (gpd/sf) of bed area are typically allowed by MassDEP (depending on the configuration of the system), unless hydrogeologic tests demonstrate a greater infiltration loading rate at the specific site.

Advantages and disadvantages of leaching facilities are summarized in the following table.

Table 2.3 Leaching Facilities Summary

Discharge Technology	Advantages	Disadvantages
Leaching Facilities	• Located below-grade allowing for secondary uses, such as parking lots or municipal recreational areas. • Disinfection is typically not required prior to discharge unless the discharge site is within a water supply area. • Town currently operates this type of system at New Silver Beach WWTF.	• Larger land areas are required due to lower hydraulic loading rates than open sand beds. • Effluent filtration is typically required to minimize bed clogging over time. • If discharge site is located in a nutrient-sensitive watershed, effluent will likely need to be treated to a very low effluent nitrogen limit. • If discharge site is located in a Zone II protection area, effluent filtration and disinfection are required per 314 CMR 05. An effluent TOC limit will also need to be met. • Pressurized systems often require more complex system balancing, and equalization/storage may be necessary to provide equal distribution throughout the system.

2.1.4 Drip Dispersal

Drip dispersal systems—also known as subsurface dispersal systems or drip irrigation systems—consist of shallow subsurface perforated tubing. These types of systems were developed based on drip irrigation systems typically utilized to irrigate agricultural areas or plants. The systems can also be potentially used in lawns and wooded areas, if sited properly, or installed below the root zone (similar to a leaching facility). Treated effluent is pumped through the tubes under pressure and discharged slowly through the emitters into the ground (either within the root zone or below the root zone). Tubing is typically installed with a vibratory plow or trencher, and requires minimal disturbance to the surface.

A hydraulic loading rate of 1.5 gallons per day per square foot (gpd/sf) of drip dispersal area is typically allowed by MassDEP unless hydrogeologic tests demonstrate a greater infiltration loading capacity at the specific site. The hydraulic loading rate assumes that the system is installed in a rectangular configuration with emitters evenly spaced at 2 feet on center and tubing 4 feet on center with the area between the tubing used as reserve area. Effluent typically needs to be filtered to avoid clogging the drip emitters. MassDEP does not require disinfection for drip dispersal systems outside of a Zone II or Interim Wellhead Protection Area.

Advantages and disadvantages of drip dispersal are summarized in the following table.

Table 2.4 Drip Dispersal Summary

Discharge Technology	Advantages	Disadvantages
Drip Dispersal	• Can be utilized in various terrain conditions and land uses. • Tubing is typically installed at a shallow depth using relatively simple construction techniques. • Low delivery rate minimizes water table impacts. • Located below-grade allowing for secondary uses such as municipal recreational areas, or below parking areas (although not as common).	• Large area required due to low hydraulic loading rate. • Effluent filtration is typically required to minimize clogging. • Periodic back-flushing required. • Limited cold weather use due to potential freezing (if located within the root zone). • Tubing is typically installed at a shallow depth and the above-grade surface needs to be protected from heavy loading. • If discharge site is located in a nutrient-sensitive watershed, effluent will likely need to be treated to a very low effluent nitrogen limit. • If discharge site is located in a Zone II protection area, effluent filtration and disinfection are required per 314 CMR 05. An effluent TOC limit will also need to be met. • Pressurized systems often require more complex system balancing, and equalization/storage may be necessary to provide equal distribution throughout the system.

2.1.5 Well Injection

A well injection system pumps treated effluent through wells that extend into permeable and saturated geologic strata. When discharged into saturated strata, the discharge process is the reverse of extracting water from a well. Well injection systems are regulated under the Underground Injection Control (UIC) Program, which is administered by MassDEP.

The well injection rate depends on site conditions such as the groundwater depth, geologic conditions, and effluent characteristics. Potential concerns of well injection include the mounding of groundwater at low elevations. Extensive hydrogeologic testing is necessary to confirm the suitability of this technology for an application.

Well injection of treated municipal wastewater effluent has been implemented on a limited basis throughout the United States; and limited information exists on the proper siting, design, construction, and operation of the wells. Pilot tests at the Hyannis Water Pollution Control Facility (WPCF) in 2003 indicated that injection wells can become plugged with biological growth if the effluent is not properly chlorinated. Discussions with MassDEP identified minimal support for the development of this technology because it utilizes chlorination. Chlorination can create secondary impacts to the groundwater through the formation of disinfection byproducts—such as total organic halide (TOX) and trihalomethanes (THMs)—that can pose potential health risks. If the effluent is discharged directly to the groundwater, 314 CMR 5.10.4C requires that the effluent meet the same requirements as a discharge to a Zone II or Interim Wellhead Protection Area with a two-year groundwater travel time to the source.

Advantages and disadvantages of well injection systems are summarized in the following table.

Table 2.5 Well Injection Summary

Discharge Technology	Advantages	Disadvantages
Well Injection	• Small surface footprint with minimal surface disturbance. • Wells could be installed in several discrete locations to minimize groundwater mounding.	• Effluent filtration and chlorination is typically required to minimize clogging/biofouling. • Formation of disinfection byproducts after chlorination can pose potential health risks. • Effluent needs to meet very stringent discharge requirements for deep well systems (equivalent to a Zone II with a two-year ground water travel time). • Limited successful installations in the United States and limited performance data available. • Extensive hydrogeologic testing required. • Pressurized systems often require more complex system balancing and equalization/storage may be necessary.

2.1.6 Wick Wells

Wick wells typically utilize large diameter (3 to 6 feet) well casings filled with stone. Treated effluent is discharged into the unsaturated zone and infiltrates into the underlying aquifer. Wick wells have been implemented on a limited basis in the United States, with three permitted locations in Massachusetts. Extensive hydrogeologic testing is necessary to confirm the suitability of this technology for an application.

Advantages and disadvantages of wick wells are summarized in the following table.

Table 2.6 Wick Wells Summary

Discharge Technology	Advantages	Disadvantages
Wick Wells	• Small surface footprint with minimal surface disturbance. • Wells could be installed in several discrete locations to minimize groundwater mounding.	• Effluent filtration is typically required to minimize clogging. • Limited successful installations in the United States and limited performance data available. • Extensive hydrogeologic testing required. • Depending on well spacing, may require a more complex system for flow distribution and system balancing.

2.2 Outfall

The Massachusetts Oceans Sanctuary Act (M.G.L. c132A) regulations establish state environmental policy to be enforced in the five Massachusetts Ocean Sanctuary areas (consisting of the Cape Cod Ocean Sanctuary, the Cape Cod Bay Ocean Sanctuary, the Cape and Islands Ocean Sanctuary, the North Shore

Ocean Sanctuary, and the South Essex Ocean Sanctuary). Historically, municipal wastewater discharges into ocean sanctuaries were specifically precluded under these regulations, unless the discharge was approved and licensed prior to December 1971. In 2014, the Massachusetts Oceans Sanctuary Act was amended to outline prerequisites that must be met for new or modified discharges into ocean sanctuaries. An extensive permitting process is anticipated for the approval of a new municipal wastewater discharge into an ocean sanctuary.

Advantages and disadvantages of outfalls are summarized in the following table.

Table 2.7 Outfall

Discharge Technology	Advantages	Disadvantages
Outfall	• Relatively small land area required. • Effluent discharge occurs outside of a watershed, greatly reducing nitrogen loading impacts to coastal embayments. • Greater dilution provided by discharging into a large surface water body.	• Extensive permitting process anticipated. • Potential reduction in aquifer recharge. • Public perception.

2.3 Anticipated Effluent Limits for Discharge Technology Options

Future effluent permit requirements are anticipated to be dictated by the location of the effluent discharge sites and the type of technology implemented for discharge. Anticipated effluent permit requirements for the discharge technologies outlined in this memo are outlined in the following table.

Table 2.8 Anticipated WWTF Effluent Discharge Limits

Parameter	Open Sand Beds – Outside of a Zone II(1)	Subsurface Effluent Disposal – Outside of a Zone II(1)(2)	Spray Irrigation – Class A, Outside of a Zone II(3)	Deep Well Injection(4)	Outfall – Class SA(5)
BOD (mg/L)	30	30	10	10	30
TSS (mg/L)	30	30	5	5	30
TN (mg/L)(6)	3	3	3	3 – 5[8]	3
TP (mg/L)	TBD[7]	TBD[7]	TBD[7]	TBD[7]	TBD[7]
Fecal Coliform (#/100 mL)	200	N/A	14	14	TBD
Enterococci (#/100 mL)	N/A	N/A	N/A	N/A	TBD
Total Organic Carbon, mg/L	N/A	N/A	N/A	1	N/A

Notes:

1. Source: Guidelines for the Design, Construction, Operation and Maintenance of Small Wastewater Treatment Facilities with Land Disposal, prepared by MassDEP, revised July 2018.
2. Category includes leaching facilities, drip dispersal, shallow well injection, and wick wells.
3. Source: 314 CMR 20 – Reclaimed Water Permit Program and Standards, effective March 2009.
4. Source: 314 CMR 5.00 – Groundwater Discharge Permits, effective December 2, 2016.
5. Ocean outfall discharge pre-requisites are outlined in Part I Title XIX Chapter 132A Section 16. New ocean outfall discharges are required to receive advanced treatment and disinfection. Effluent parameters are not specified in the regulations. Anticipated requirements are assumed for this project. Class SA, High Quality Waters are designated per 314 CMR 4.00.
6. Anticipated effluent TN limit of 3 mg/L is anticipated for a discharge into a nutrient impaired watershed. Per the Shared Wastewater Management Study Towns of Bourne, Falmouth, Mashpee, Sandwich and Joint Base Cape Cod report prepared by Wright-Pierce and dated November 2017, anticipated effluent TN limit for Joint Base Cape Cod discharge is 10 mg/L for open sand beds and wick wells and 5 mg/L for deep well injection and outfall (Class SB).
7. A TP effluent limit may be required depending on discharge site location.
8. Effluent limit of 5 mg/L assumes that deep well injection discharge is deep enough to emerge beyond all nitrogen sensitive embayments/watersheds.

2.4 Technologies Recommended for Conceptual Layout Development

Based on the evaluation summarized in Section 2, the following discharge technologies are recommended for incorporation into the conceptual layouts developed as part of this project:

1. Open sand beds are recommended for conceptual layout development due to their relatively high hydraulic loading capacity, which requires less land area than other land-based options. Additionally, the Town of Falmouth currently uses this technology at the Falmouth WWTF and is familiar with the technology.

2. Leaching facilities are recommended for conceptual layout development in areas with a potential secondary use (for example under fairways in a golf course or under public parks/ballfields) due to their minimal visual impact. Additionally, the Town of Falmouth currently uses this technology at the New Silver Beach WWTF and is familiar with the technology.

3. Ocean outfalls are recommended for conceptual layout development due to the relatively small land area required for this technology, relatively high disposal capacity, and the ability to discharge

outside a nutrient impacted watershed, reducing the nitrogen loading impacts to coastal embayments.

3. Discharge Site Evaluation

This section summarizes the discharge site evaluation that was conducted for the following eight sites:

- Falmouth WWTF – existing WWTF open sand beds
- Joint Base Cape Cod (JBCC)
- Allen Parcel (previously identified as “Site 4” in 2013 CWMP/FEIR/TWMP)
- Augusta Parcel (previously identified as “Site Z” in the 2007 ASAR)
- Buzzards Bay Outfall
- Falmouth Country Club (previously identified as Sites A, B, and 2B in the 2013 CWMP/FEIR/TWMP)
- Nobska Point Outfall
- Land and property adjacent to Recharge Beds 14 and 15

A general description of each site is provided in this section.

3.1 Existing WWTF Open Sand Beds (With Enhanced Nitrogen Removal)

The Falmouth WWTF currently operates under Modified Groundwater Discharge Permit No. 168-5, effective date December 12, 2015 (2015 Permit). The WWTF has 15 effluent disposal open sand beds. Open sand beds 1 through 13 are located within the West Falmouth Harbor watershed. Recharge Beds 14 and 15 are located outside of the West Falmouth Harbor watershed. The 2015 Permit allocates effluent flow limits by watershed.

The following table compares the permitted flow that can be discharged to the existing open sand beds with their design flow. As shown in the table, the overall permitted flow is less than the design flow of the open sand beds. It should be noted that the actual infiltration capacity of existing open sand beds 9 through 15 is substantially greater than design and permitted capacity, based on operational experience since bed startup.

Table 3.1 Open Sand Bed Design Flow vs. Permitted Flow

	Annual Average Design Capacity – All Beds Operating (mgd)	Annual Average Permitted Capacity[2] (mgd)	Un-utilized Design Capacity Due to Permit Nitrogen Load Limit (mgd)
Open Sand Beds 1 through 13	0.75[3]	0.45	0.3
Recharge Beds 14 and 15	0.26[1]	0.26	0
Total	**1.01[1]**	**0.71**	**0.3**

Notes:

1. Source: Recharge Beds 14 and 15 Operations and Maintenance Manual, prepared by GHD, dated June 2017.
2. Source: Modified Individual Groundwater Discharge Permit No. 168-5, effective date 12/22/15.
3. Design hydraulic capacity for Sand Beds 1 through 13 = 1.1 gpd/sf per the 'Falmouth Wastewater Treatment Facility Operation & Maintenance Manual – Draft', prepared by Maguire Group Inc., dated February 2007.

The 2015 WWTF permit limits the cumulative nitrogen annual load that can be discharged within the West Falmouth Harbor watershed to 4,109 pounds per calendar year (assuming that sewer is not extended within the West Falmouth Harbor watershed and no other nitrogen reductions are credited within this watershed). This load limit is based on an average annual flow of 450,000 gpd of flow discharged to sand beds 1 through 13 at a concentration of 3 mg/L. To maintain the existing load limit, the effluent nitrogen concentration would need to be reduced if additional flow was discharged to sand beds 1 through 13. The relationship between decreasing nitrogen concentration and additional flow is linear. If the effluent nitrogen concentration was consistently reduced to 2.5 mg/L, then the flow to the beds could potentially be increased to 0.54 mgd; if it were reduced to 2 mg/L, then the flow to the beds could potentially be increased to 0.68 mgd. The WWTF would need to consistently meet an effluent limit of less than 1.8 mg/L in order to utilize the design capacity of Discharge Beds 1 through 13 (0.75 mgd) and meet the existing load limit of 4,109 lb/yr without any additional nitrogen removal from the West Falmouth Harbor watershed. Options for enhanced nitrogen removal (less than 3 mg/L) are discussed in TASA TM-4.

3.2 Joint Base Cape Cod (JBCC)

The WWTF is located at the southeast corner of the JBCC and provides wastewater treatment for the existing five military branch's joint-use facilities. The WWTF is currently owned by the United States Air Force, and operated and maintained by the Massachusetts Air National Guard.

The 2017 Wright Pierce JBCC Study provides an engineering assessment for a potential partnership for shared wastewater management options at JBCC between the four towns of Bourne, Falmouth, Mashpee, and Sandwich. As part of the study, each of the four participating towns provided capacity requests by wastewater type. The Town of Falmouth requested the following capacity allocations for its study basis:

- Mid-term allocation – 490,000 gpd (annual average) of effluent is treated at the Falmouth WWTF and sent to the JBCC disposal system
- Long-term allocation – 1,490,000 gpd (annual average) of effluent is treated at the Falmouth WWTF and sent to the JBCC disposal system

Both the mid-term and long-term allocations represent un-allocated flows since the location within the Town that the flow is derived from is not specified.

The 2017 Wright Pierce JBCC study assumed that a new 18-inch force main would be constructed in Route 28, Route 151, and Turpentine Road with a connection to the existing JBCC collection system, a total new force main distance of 6.7 miles. The evaluation also provided preliminary cost estimates for the infrastructure that would be required to allow JBCC to function as a regional facility—including expansion of the existing WWTF, development of collection system infrastructure from each community to JBCC, and development of additional effluent disposal capacity. The mid-term allocation requested by the Town of Falmouth for JBCC (490,000 gpd annual average flow) would provide adequate effluent disposal capacity for TASA (330,000 gpd estimated annual average flow) if regionalization of the facility proceeds. An agreement with JBCC and the participating Towns would be required for this option.

3.3 Allen Parcel (Site 4)

The Allen Parcel is approximately 70 acres and is Town-owned. Fourteen acres in the southwest corner of the parcel—labeled as 'Lot 3' on the '2005 Allen Parcel Plan of Land'—has been identified for general municipal use and is located in the Great Pond watershed. The property has a 100-foot wide utility easement along the western boundary of the property.

Subsurface investigation by means of a test pit in 2010 indicated that this site is in the outwash plain with sandy soils. A percolation test conducted in 2010 indicated that the underlying soils have a percolation rate of less than 2 minutes per inch. Based on the results of the percolation test, a design loading rate of 7 gpd/sf for open sand beds was assumed in the Falmouth 2013 CWMP/TWMP/FEIR. As summarized in TASA TM-5, subsurface field investigations conducted in Fall of 2018 confirmed that a design hydraulic loading rate of 7 gpd/sf is appropriate for this site. A layer of unsuitable material was discovered during the 2018 field investigation, approximately 10-inches thick at approximately 28-inches to 38-inches below grade. This layer would need to be excavated and removed if the site were developed for effluent disposal.

JBCC maintains several monitoring wells on the Allen Parcel due to its proximity to the Ashumet Valley Plume. Coordination with Air Force Civil Engineer Center at JBCC will be required if this site is pursued as a proposed effluent disposal location.

3.4 Augusta Parcel

The Augusta Parcel is approximately 20 acres and is Town-owned. It is surrounded by residential and commercial properties and is located in the Great Pond watershed. There is a topographic drop-off on the southeast portion of this property that would need to be taken into account in recharge area design. The site was previously identified as a potential site for a new WWTF. The site was also designated for a proposed booster station in the TASA collection system conceptual layout which will collect untreated wastewater from the area and pump it to the Falmouth WWTF on Blacksmith Shop Road.

The deed to the Augusta Parcel limits the sewer service area of future wastewater treatment and disposal on the property to the Teaticket Water Quality Improvement District (TWQID). The TWQID is generally defined as the Maravista area between Little Pond and Green Pond, generally bounded by Sandwich Road on the west, Brick Kiln Road on the north, and Acapesket Road on the east. The deed map showing the TWQID is provided in Attachment 1. TWQID encompasses the majority of TASA, as well as an area north of Route 28 which is not currently under consideration for sewering, and the Maravista peninsula, most of which was connected to the sewer system under the Little Pond Sewer Service Area Project. Hydraulic load testing, conducted in Fall of 2018, indicated that a design hydraulic loading rate of 7 gpd/sf was appropriate for open sand beds on this parcel. Soil borings indicated that soils at this location are generally homogenous and comprised of sand with traces of gravel and finer grained materials.

3.5 Buzzards Bay Outfall

Construction of an ocean outfall in Buzzards Bay, extending approximately 4,400 feet into Buzzards Bay and sized for 4 mgd, was evaluated as part of this project. The proposed outfall was modeled using high-resolution ocean model data for the greater Buzzards Bay area and a plume tracking model to simulate plume dispersion at the potential discharge location. A year-long simulation of effluent transport indicated that the potential outfall is anticipated to have a negligible effect on the TN concentrations in Buzzards Bay and West Falmouth Harbor. The Buzzards Bay modeling, conceptual layout, and conceptual level cost estimate are discussed in further detail in the Buzzards Bay Outfall Technical Memorandum (TASA TM-6).

3.6 Falmouth Country Club (FCC)

The FCC site is comprised of multiple parcels with a combined area of approximately 150 acres. The site is Town-owned and a portion of the site is presently utilized as a municipal golf course. The FCC Conservation Restriction (FCC Restriction), dated November 2007, notes that portions of the property may be used for effluent disposal.

In 2009, the Town was awarded a grant from the Cape Cod Water Protection Collaborative to complete groundwater modeling for a proposed recharge at the Falmouth Country Club site using a calibrated sub-regional model based on the USGS regional model of the Sagamore Lens. The study included particle tracking and groundwater mounding analysis for recharge at the FCC site. Particle tracking indicated that recharge at the FCC could flow to the Bournes Pond, Green Pond, Eel Pond, and Waquoit watersheds, depending on location and volume of discharge on the site.

Field investigations indicated that subsurface conditions were generally homogenous and comprised of sand and gravel with lenses of finer grained materials. The predominance of sand suggests suitable conditions for infiltration of treated wastewater. Depth to groundwater at the site is approximately 35 feet below ground surface (approximately 16 feet above mean sea level).

The investigations indicated that a design loading rate of 11 gpd/sf for open sand beds could be used for this site, with the understanding that additional hydrogeological investigations may be needed once the open sand basin locations are identified at the site and a full environmental impact analysis is completed. The conservative design loading rate was derived using the lowest average infiltration rate observed during the investigations and EPAs recommended design factor (10 percent of the infiltration test rate). The loading rate of 11 gpd/sf is over two times the loading rate typically allowable by MassDEP (5 gpd/sf), and MassDEP has indicated they would allow a design loading rate of 7 gpd/sf until performance testing (after WWTF implementation with actual treated water from a WWTF) proved that a higher rate was warranted.

Effluent disposal at the Falmouth Country Club was included in the Preferred Option (Scenario 1E) outlined in the 2013 CWMP/FEIR/TWMP. The Preferred Option, shown in Figure 1, included areas designated for subsurface leaching trenches on the western portion of the FCC (Site 2B) and open sand beds on the eastern portion of the property (Sites A1 and B).

A portion of the FCC is located over the Ashumet Valley plume, which is being managed by JBCC. Coordination with the Air Force Civil Engineer Center at JBCC will be required if this site is pursued as a proposed effluent disposal location.

3.7 Nobska Point Outfall

Construction of an ocean outfall at Nobska Point in Woods Hole, extending approximately 2,000 feet into Vineyard Sound and sized for 4 mgd, was first evaluated as part of wastewater planning efforts in the 1970's. Nobska Point was selected as the proposed outfall location due to its high dilution and mixing potential, and proximity to a WWTF to be built on Fay Road.

Data collected through drifter buoys, drift bottles, and seabed drifters was used to determine flushing and tidal characteristics of the site. The evaluation, summarized in a series of articles published in the Journal of Boston Society of Civil Engineers in 1971 (Bumpus & Vaccaro, 1971; Bumpus, Wright & Vaccaro, 1971; Meade & Vaccaro, 1971), indicated that the outfall should extend more than a quarter mile off of Nobska Point to avoid flow towards the beaches or into Woods Hole Passage. The evaluation concluded that *"the minimum return to shores, harbors and estuaries, coupled with maximum dilution and minimum length of outfall pipe, could be achieved at a distance of 0.375 miles south of Nobksa Point (700 yards 141 deg T) from Nobska Lighthouse."*

As part of the original evaluation, a treatment facility was proposed in Woods Hole, near Nobska Point, on land owned by the Marine Biological Laboratory.

A potential ocean outfall at Nobska Point was reexamined as part of the CWMP process in 2011. Conceptual costs for this 2011 analysis were prepared assuming the use of a 3-foot diameter HDPE force main, installed from the existing Falmouth WWTF at Blacksmith Shop Road to Nobska Point. Costs were based on construction unit costs for the 1,300-foot long outfall constructed at the Seabrook, NH WWTF in 1994.

The conceptual route to Nobska Point as part of the 2011 evaluation followed the same route as the force main from the Woods Hole Lift Station to the WWTF. However, the Town has expressed concerns about the high coastal erosion vulnerability of this route. As part of TASA TM-3, it is recommended that further discussion be held on an alternate route for the force main, if this is to be further evaluated.

3.8 Land and Property Adjacent Recharge Beds 14 and 15 to the North

A currently privately-owned parcel that is adjacent to the existing WWTF property, combined with available area adjacent to Recharge Beds 14 and 15, was identified as a potential recharge location. The privately-owned parcel is approximately 4.7 acres. This parcel abuts Town-owned property, Recharge Beds 14 and 15 (Recharge Beds 14 and 15 are located on a parcel previously identified at "Site 7" in the 2013 CWMP/FEIR/TWMP), and the Falmouth WWTF. These beds, constructed on the eastern portion of the Site 7 property in 2015, have a rated maximum month capacity of 0.47 mgd. If the site containing Recharge Beds 14 and 15 is expanded through purchase or acquisition, the location has the potential to add additional sand beds on the expanded property.

Groundwater modeling and particle tracking of treated water recharge at the Recharge Beds 14 and 15 site, conducted in 2011, indicated that effluent is anticipated to recharge into Buzzards Bay. The model indicated that a portion of the treated effluent would pass through Crocker Pond, a small freshwater kettle pond to the west of Recharge Beds 14 and 15, which is phosphorus limited. An evaluation, conducted by EcoLogic in 2013, indicated that the aquifer soils downstream of Recharge Beds 14 and 15 have a large capacity to sequester phosphorus from the groundwater and significantly retard migration of phosphorus downstream to the kettle pond. The absorptive capacity of the soil was estimated as 100 to 1,400 years of phosphorus discharge, depending on the level of effluent treatment. Additionally, the University of Massachusetts Dartmouth School of Marine Science and Technology (SMAST) conducted an evaluation assessing coastal down-gradient impacts of Recharge Beds 14 and 15 wastewater effluent discharge. The study concluded that, based on the amount of nitrogen entering Buzzards Bay and its low nitrogen levels in the study region, it is unlikely that the anticipated maximum nitrogen load from Recharge Beds 14 and 15 will be detectable.

Additional modeling would be required to determine the impact of additional flow if new beds were developed adjacent to Recharge Beds 14 and 15. The privately-owned property (if purchased) would also need to be rezoned to Public Use and would require a site hearing to be conducted by MassDEP in accordance with M.G.L. c 83 s6.

A second option for this site is to expand Recharge Beds 14 and 15 to the North within the boundaries of the existing Town-owned parcel. Performance testing could also be conducted at existing Recharge Beds 14 and 15 to evaluate the ability to request an increase in the rated capacity of the open sand beds.

4. Discharge Capacity Evaluation

This section summarizes the discharge capacity evaluation that was developed to establish the maximum capacity of each site. Maximum capacities were established based on estimated available area and discharge method. The following basis of design was used to estimate discharge capacity for each site:

- Disposal area divided into four areas.
- Disposal system sized to handle maximum month flow with one area out of service.
- 100 percent redundancy provided for average annual flow.
- 100 foot buffer maintained from property boundaries.

The discharge capacity for each site is outlined in the following table.

Table 4.1 Estimated Maximum Month Discharge Capacity of Discharge Technologies[1]

Discharge Site	Estimated Available Area (acres)	Open Sand Beds (mgd) / Estimated Footprint (acres)[7]	Drip Dispersal (mgd)/ Estimated Footprint (acres)[8]	Leaching Trenches (mgd) / Estimated Footprint (acres)[9]	Spray Irrigation (mgd) / Estimated Footprint (acres)[10]
New Site Abutting Recharge Beds 14 and 15	1.7[2]	0.27 MGD (1.7 acres)	0.09 MGD (1.7 acres)	0.17 MGD (1.7 acres)	0.01 MGD (1.7 acres)
Allen Parcel	12.6[3]	2.88 MGD (12.6 acres)	0.62 MGD (12.6 acres)	1.23 MGD (12.6 acres)	0.07 MGD (12.6 acres)
Augusta Parcel	19.4[4]	0.67 MGD (2.9 acres)	0.67 MGD (13.7 acres)	0.67 MGD (6.8 acres)	0.11 MGD (19.4 acres)
Falmouth Country Club – A1	6.6[5]	1.51 MGD (6.6 acres)	0.32 MGD (6.6 acres)	0.65 MGD (6.6 acres)	0.04 MGD (6.6 acres)
Falmouth Country Club - B	4.9[5]	1.12 MGD (4.9 acres)	0.24 MGD (4.9 acres)	0.48 MGD (4.9 acres)	0.03 MGD (4.9 acres)
Falmouth Country Club – 2B	22.0 [5,6]	N/A	1.08 MGD (22 acres)	2.16 MGD (22 acres)	0.13 MGD (22 acres)

Notes:

1. All discharge sites designed to handle maximum month conditions with one bed out of service.
2. Acreage for New Site Abutting Recharge Beds 14 and 15 includes development of available space on the existing WWTF site (immediately north of the existing open sand beds) and the New Site Abutting Recharge Beds 14 and 15 (immediately north of the existing open sand beds). Per discussions with the WQMC working group portions of both properties west of the existing open sand beds are not included in the estimated available area.
3. Allen Parcel estimated available area per Table 4-3 of the 2013 CWMP/TWMP/FEIR. Discharge capacity based on available land area.
4. Potential available area of Assessor's Map 34-04-036-000C1. A deed restriction on the property limits the disposal capacity to flow from the Teaticket Water Quality Improvement District (TWQID). The estimated acreage required to provide recharge capacity for the estimated TWQID maximum month flow (0.67 mgd) is outlined for each type of technology.
5. Potential available area per 2013 CWMP/FEIR/TWMP Table 4.3 and Figure 4-6.
6. Available area based on estimation of surface area under fairways not including the greens.
7. Estimated Hydraulic loading rate = 7 gpd/sf.
8. Hydraulic loading rate per MassDEP 2018 guidelines = 1.5 gpd/sf. Calculations assume that drip tubing is placed 4-feet on center.
9. Hydraulic loading rate per MassDEP 2018 guidelines = 3 gpd/sf.
10. Hydraulic loading rate per MassDEP 2018 guidelines = 2 inches/acre/week when system is in use. It is assumed that spray dispersal can only be utilized for nine months of the year.

All of the discharge sites have adequate effluent disposal capacity for TASA if they are developed with open sand beds.

5. Conceptual Layout Development

Six site/technology combinations were selected by the Town and the Water Quality Management Committee Working Group (WQMC) for conceptual layout development (a combination is defined as a single technology used on a single site):

- Allen Parcel with open sand beds.
- Augusta Parcel with open sand beds.[1]
- Falmouth Country Club with leaching trenches on the western portion of the property.
- Site abutting Recharge Beds 14 and 15 (Site 7) to the north with open sand beds.
- Nobska Point ocean outfall.
- Buzzards Bay ocean outfall.

Although a conceptual layout was not developed for effluent recharge at JBCC to serve TASA, JBCC was evaluated as part of another study and the Town continues to consider it as a future long-term option for effluent disposal.

5.1 Conceptual Layout – Sizing Criteria

All of the conceptual layouts were sized to recharge the anticipated maximum month flow from TASA with one disposal “area” out of service (0.53 mgd). Anticipated TASA flows are outlined in the following table. Additionally, each conceptual layout was also developed for a future condition, which maximized the recharge capacity of the site.

Table 5.1 Anticipated Flow from TASA

Parameter	TASA Flow (mgd)[1]	TASA Flow + I/I (mgd)[2]
Average Annual	0.25	0.33
Maximum Month	0.46	0.53
Maximum Day	0.48	0.56
Peak Instantaneous	0.86	0.94

Notes:
1. Source: TASA TM-1 – South Coast Embayments Preliminary Evaluations and Notice of Project Change Update Project – Draft Service Area, Flow and N Load Evaluation prepared by GHD, dated May 2018.
2. I/I allocation based on estimated TASA gravity sewer length and an infiltration rate of 500 gpd/sf. I/I allocation will need to be re-calculated once the TASA collection system conceptual layout has been finalized.

5.2 Conceptual Force Main Routing

Conceptual force main routing for the layouts is shown in Figure 2 and Figure 3. The distance from the WWTF to each discharge location along the conceptual force main routes is summarized in the following table.

[1] One additional conceptual layout was developed for sub-surface leaching trenches at the Augusta Parcel.

Table 5.2 Distance from Falmouth WWTF to Proposed Discharge Site Locations

Conceptual Force Main Route	Approximate Distance from Falmouth WWTF to Discharge Site along Conceptual Force Main Route (miles)
Falmouth WWTF to Allen Parcel	4
Falmouth WWTF to Falmouth Country Club	5
Falmouth WWTF to Buzzards Bay Outfall	2
Falmouth WWTF to Augusta Parcel	4
Falmouth WWTF to Site Abutting Recharge Beds 14 and 15 to the North	<1
Falmouth WWTF to Nobska Point Ocean Outfall	7
Falmouth WWTF to existing JBCC Collection System[1]	7

Notes:

1. Per the conveyance route outlined in the Shared Wastewater Management Study Towns of Bourne, Falmouth, Mashpee, Sandwich, and Joint Base Cape Cod, prepared by Wright-Pierce, dated November 2017.

5.3 Conceptual Effluent Disposal Layout

A description of the conceptual layouts is summarized below.

5.3.1 Allen Parcel

A conceptual layout for the Allen Parcel with sand beds is shown on Figure 4. The following basis of design was used to develop the layout:

- Open Sand Beds 1 through 4 are sized to recharge TASA flow during maximum month conditions with one open sand bed out of service.
- A 50-foot buffer is maintained from the utility easement to provide a visual barrier.
- A 20-foot buffer is maintained around the perimeter of Lot 3 for the development of a future service road.
- Open Sand Beds 5 through 15 provide a conceptual layout for future expanded conditions to maximize the available area of the site.
- The open sand beds are sized based on a design hydraulic loading rate of 7 gpd/sf.
- The conceptual layout assumes that the open sand beds will be fed by gravity from a distribution box at the end of the effluent force main.

5.3.2 Augusta Parcel

A conceptual layout for the Augusta Parcel with open sand beds is shown in Figure 5. The following basis of design was used to develop the layout:

- Open Sand Beds 1 through 4 are sized to recharge TASA flow during maximum month conditions with one open sand bed out of service.
- A 100-foot buffer is maintained from the property boundaries.

- Open Sand Bed 5 provides a conceptual layout for future expanded conditions to maximize the amount of flow allowable per the deed restriction for the Teaticket Water Quality Improvement District (see Attachment No. 1).
- The open sand beds are sized based on a design hydraulic loading rate of 7 gpd/sf.
- The conceptual layout assumes that the open sand beds will be fed by gravity from a distribution box at the end of the effluent force main.

A second conceptual layout for the site with leaching trenches was developed and is shown in Figure 6. A leaching trench system could allow the site to maintain a secondary use such as a park or ballfield, if desired. The following basis of design was used to develop the layout:

- Leaching trench system is sized to recharge TASA flow during maximum month conditions with one leaching trench area out of service.
- The leaching trench system is sized based on a design hydraulic loading rate of 3 gpd/sf.
- The leaching trench system will be fed through a pressurized distribution system from an effluent lift station located on the site.

5.3.3 Falmouth Country Club

A conceptual layout for leaching trenches in the western portion of the FCC (Site 2B) is shown in Figure 7. The following basis of design was used to develop the layout:

- Leaching trench system is sized to recharge TASA flow during maximum month conditions with one area out of service.
- The conceptual layout was restricted to the Green Pond watershed (no effluent recharge on the eastern portion of the property which is primarily in the Bournes Pond watershed) to reduce transfer of flow and nitrogen load between watersheds.
- The leaching trench system is sized based on a design hydraulic loading rate of 3 gpd/sf.
- The leaching trench system will be fed through a pressurized distribution system from an effluent lift station located on the site.
- Leaching trench areas 1 through 4 are sized to recharge TASA flow during maximum month conditions with one leaching trench area out of service.
- Leaching trench areas 5 through 12 provide a conceptual layout for future expanded conditions to maximize the potentially available area of the site.

5.3.4 Site Abutting Recharge Beds 14 and 15

A conceptual layout for the site with open sand beds is shown in Figure 8. The following basis of design was used to develop the layout:

- Open Sand Beds 16 and 17 are sized to recharge TASA flow during maximum month conditions with all beds operational. If an open sand bed needs to be taken down for maintenance during maximum month conditions, the existing open sand beds at the WWTF are anticipated to be used for additional capacity during this condition.

- A 100-foot buffer is maintained from the property boundaries.
- The open sand beds were sized based on a design hydraulic loading rate of 7 gpd/sf.
- The conceptual layout assumes that the open sand beds will be fed through the existing gravity line used to convey flow to existing Open Sand Beds 14 and 15.

5.3.5 Nobska Point Ocean Outfall

The basis of design for the potential Nobska Point Ocean Outfall is outlined in TASA TM-6 and summarized below:

- 3-foot-diameter High Density Polyethylene (HDPE) force main from the Falmouth WWTF to Nobska Point (approximately 7.3 miles) sized to convey 4 mgd annual average flow.
- 3-foot-diameter HDPE outfall (approximately 2,000 feet from Nobska Point into Vineyard Sound) based on the original study.
- The existing open sand beds at the Falmouth WWTF provide redundancy to the system. If the ocean outfall needed to be taken offline for a period of time, effluent flow would be diverted to the existing open sand beds.

5.3.6 Buzzards Bay Ocean Outfall

The basis of design for the potential Buzzards Bay Ocean Outfall is outlined in TASA TM-6 and summarized below:

- 2-foot diameter High Density Polyethylene (HDPE) force main from the Falmouth WWTF to a location off Chapoquoit Road (approximately 2 miles) sized to convey 4 mgd annual average flow.
- 2-foot diameter HDPE outfall (approximately 4,380 feet from Chapoquoit Road into Buzzards Bay).
- The existing open sand beds at the Falmouth WWTF provide redundancy to the system. If the ocean outfall needed to be taken offline for a period of time, effluent flow would be diverted to the existing open sand beds.

A smaller diameter outfall is used for the Buzzards Bay Ocean Outfall (2-foot diameter) than the Nobska Point Ocean Outfall (3-foot diameter) due to the reduced head losses from the shorter force main length.

6. Summary

Four different potential discharge sites and two potential ocean outfalls were evaluated, and discharge capacities were developed for each option. It is estimated that all of the potential sites have adequate effluent disposal for the estimated TASA flows if they are developed utilizing open sand beds. The basis of design for the seven conceptual layouts developed as part of this project are summarized in the following table.

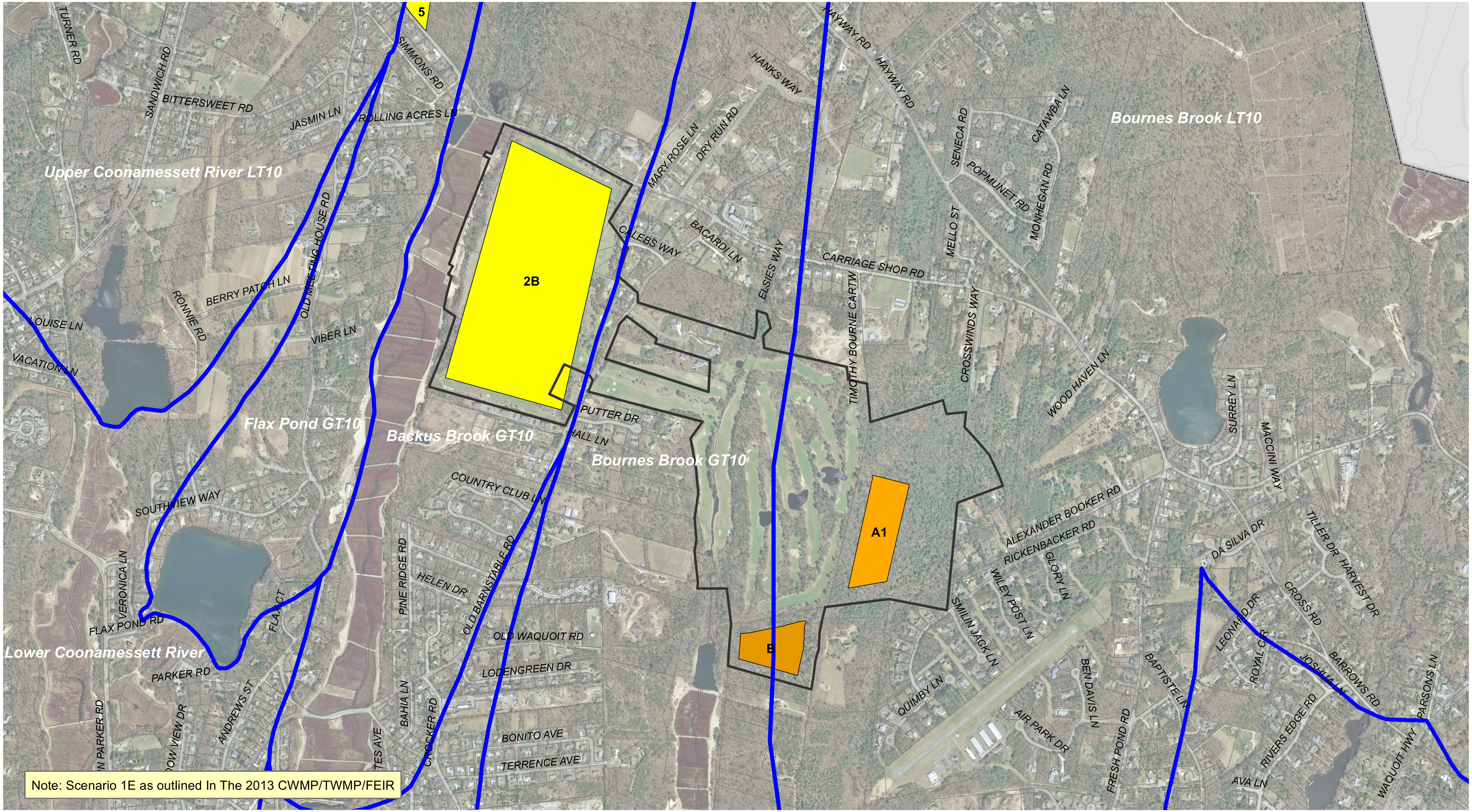
Table 6.1 Effluent Disposal Conceptual Layouts – Basis of Design

Site	Technology	Design Maximum Month Flow (mgd)	Hydraulic Loading Rate (gpd/sf)	Design Capacity
New Site Abutting Recharge Beds 14 & 15	Open Sand Beds	0.53	7	Sized to handle TASA maximum month flow with all beds in service (existing open sand beds at WWTF provide redundancy in the system).
Allen Parcel	Open Sand Beds	0.53	7	Sized to handle TASA maximum month flow with one bed out of service.
Falmouth Country Club	Leaching Trenches	0.53	3	Sized to handle TASA maximum month flow with one bed out of service.
Augusta Parcel	Open Sand Beds	0.53	7	Sized to handle TASA maximum month flow with one bed out of service.
Augusta Parcel	Leaching Trenches	0.53	3	Sized to handle TASA maximum month flow with one bed out of service.
Nobska Point	Ocean Outfall	7	N/A	Sized based on Town-wide flow documented in the 2007 Needs Assessment Report and 1968 Whitman Howard Report.
Buzzards Bay	Ocean Outfall	7	N/A	Sized based on Town-wide flow documented in the 2007 Needs Assessment Report and 1968 Whitman Howard Report.

In the next phase of this project, cost estimates will be developed for four to five conceptual plans identifying sewer service area, collection/transmission system (and lift station sites), discharge plan, and WWTF upgrades.

Figures

Figure 1 – Falmouth Country Club – (Scenario 1E)
Figure 2 – Conceptual Layout – Force Main Routing for Effluent Discharge Scenarios
Figure 3 – Conceptual Latyou – Force Main – Routing for JBCC Effluent Disposal Scenario
Figure 4 – Conceptual Layout – Open Sand Beds at Allen Parcel
Figure 5 – Conceptual Layout – Open Sand Beds at Augusta Parcel
Figure 6 – Conceptual Layout – Leaching Trench Effluent Disposal System at Augusta Parcel
Figure 7 – FCC Effluent Leaching Trenches – Conceptual Layout
Figure 8 – Open Sand Beds at New Site Abutting Recharge Beds 14 & 15



Paper Size ANSI B

0 0.0375 0.075 0.15

Miles

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1927
Grid: NAD 1927 StatePlane Massachusetts Mainland FIPS 2001

N

LEGEND

- Falmouth Country Club Site (As Outlined in 2007 ASAR)
- Potential Open Sand Beds
- MEP Watershed Boundary
- Potential Subsurface Leaching Facilty

TOWN OF FALMOUTH, MA
Teaticket/Acapesket Preliminary Evaluation TASA (TM-3)

Job Number	111-53041
Revision	A
Date	30 Oct 2018

FALMOUTH COUNTRY CLUB -
(SCENARIO 1E)

Figure 1

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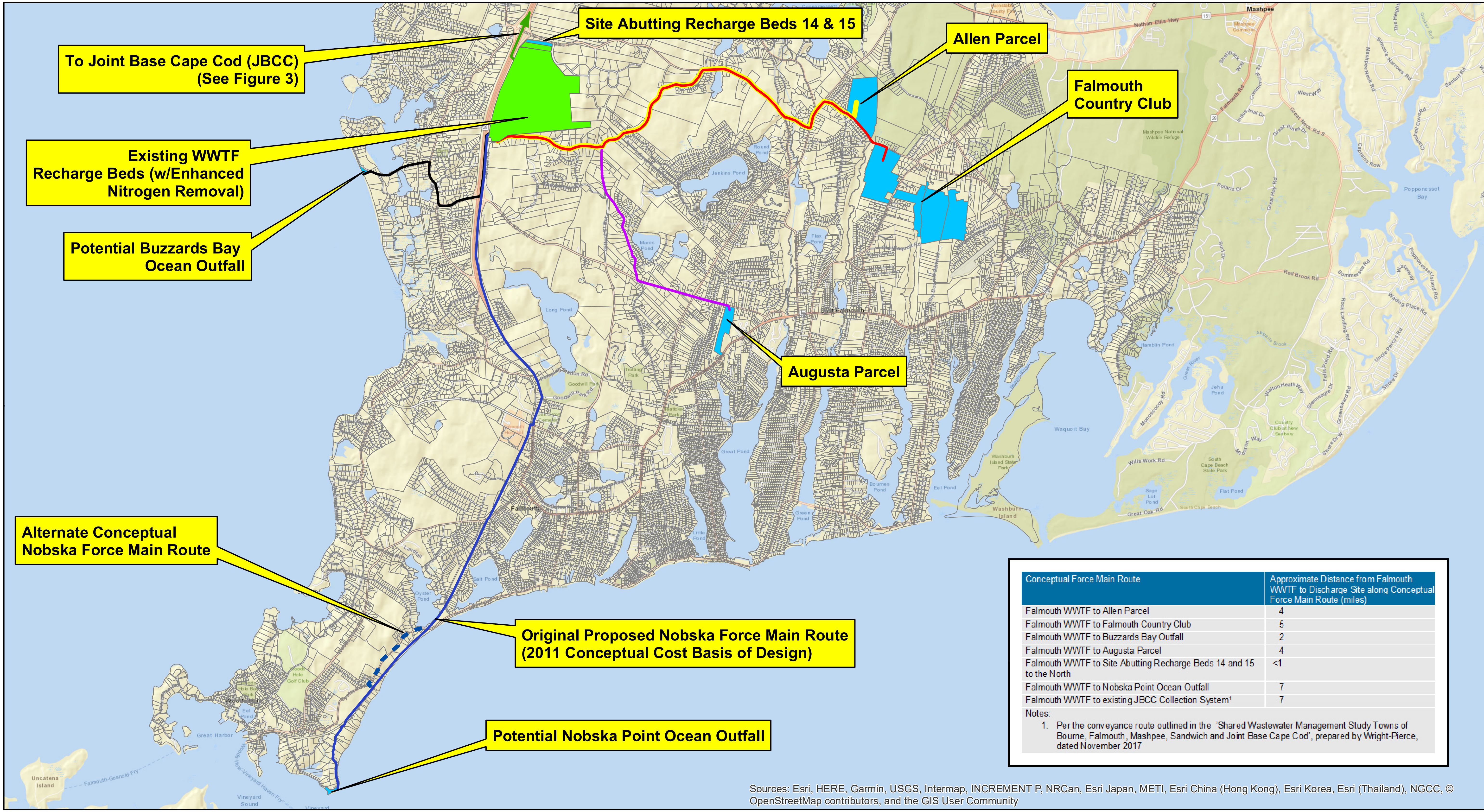
1545 Iyannough Road, Hyannis Massachusetts 02601 USA T 1 508 362 5680 F 1 508 362 5684 E hyamail@ghd.com W www.ghd.com



Conceptual Force Main Route	Approximate Distance from Falmouth WWTF to Discharge Site along Conceptual Force Main Route (miles)
Falmouth WWTF to Allen Parcel	4
Falmouth WWTF to Falmouth Country Club	5
Falmouth WWTF to Buzzards Bay Outfall	2
Falmouth WWTF to Augusta Parcel	4
Falmouth WWTF to Site Abutting Recharge Beds 14 and 15 to the North	<1
Falmouth WWTF to Nobska Point Ocean Outfall	7
Falmouth WWTF to existing JBCC Collection System[1]	7

Notes:

1. Per the conveyance route outlined in the 'Shared Wastewater Management Study Towns of Bourne, Falmouth, Mashpee, Sandwich and Joint Base Cape Cod', prepared by Wright-Pierce, dated November 2017

Paper Size ANSI B

0 2,250 4,500 9,000 Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

LEGEND

Conceptual Force Main Routes

- Allen Parcel
- Augusta Parcel
- Potential Buzzards Bay Outfall
- Existing Effluent Disposal Parcel
- Falmouth Country Club
- To Joint Base Cape Cod
- Potential Nobska Point Outfall
- Potential Effluent Discharge Site

GHD

TOWN OF FALMOUTH, MA
Teaticket/Acapesket Preliminary Evaluation TASA (TM-3)

Job Number 111-53041
Revision A
Date 01 Nov 2018

Conceptual Layout - Force Main Routing For Effluent Discharge Scenarios

Figure 2

G:\111\11153041 Town of Falmouth South Coast CWMP Update\GIS\Maps\MXD_Deliverables\TASA TM-3\111-53041-F02 EFF_DIS.mxd

Data source: Data Custodian, Data Set Name/Title, Version/Date. Created by:jjobrien

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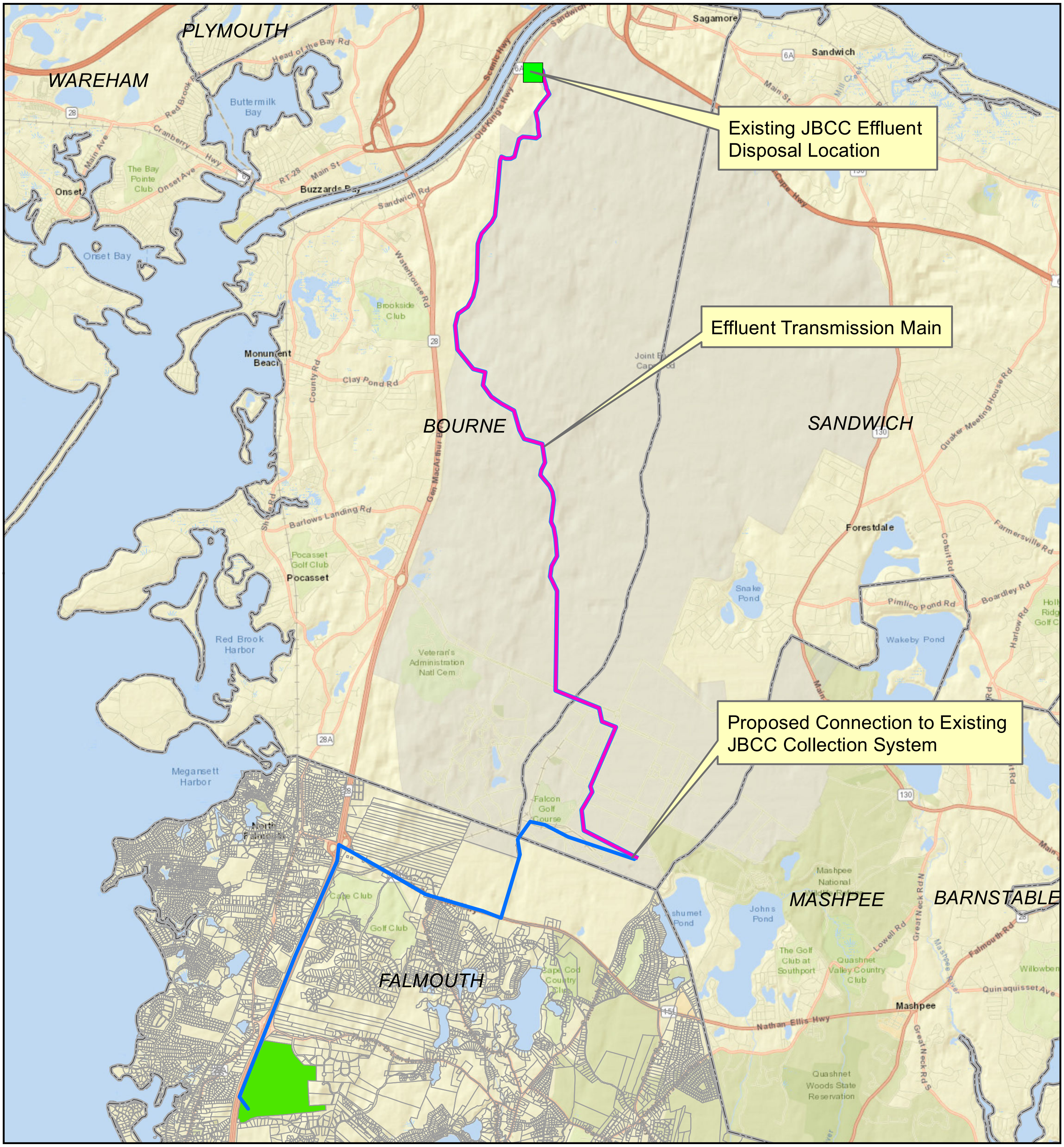

LEGEND

- Effluent Transmission Main
- Proposed JBCC Force Main
- Town Boundary
- Falmouth WWTF Site

Note 1: Proposed Routing as Outlined in the "Shared Wastewater Mangement Study Towns of Bourne, Falmouth, Mashpee, Sandwich and Joint Base Cape Cod

Paper Size ANSI A

0 2,950 5,900 11,800 Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

GHD

TOWN OF FALMOUTH, MA
Teaticket / Acapesket Preliminary Evaluation TASA (TM-3)

CONCEPTUAL LAYOUT - FORCE MAIN - ROUTING FOR JBCC EFFLUENT DISPOSAL SCENARIO

Job Number	111-53041
Revision	-
Date	30 Oct 2018

Figure 3

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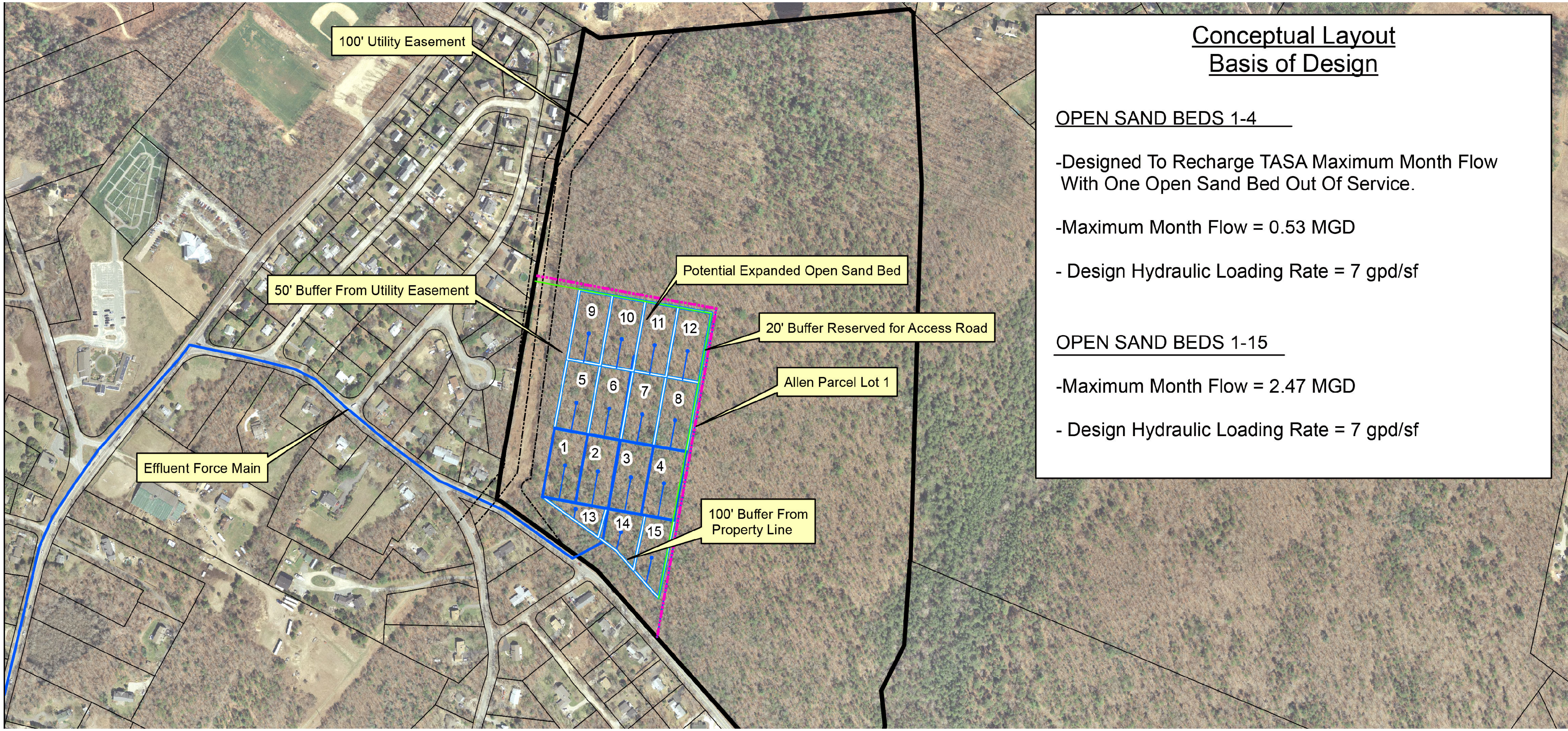

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Not To Scale

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

LEGEND

- Access road
- Lot 3 Border (General Municipal Use -Plan 2005)
- Conceptual Force Main
- Allen Parcel Lot 1 Border
- Conceptual Open Sand Beds
- Utility Easement
- Potential Expanded Open Sand Bed

TOWN OF FALMOUTH, MA
Teaticket/Acapesket Preliminary Evaluation (TASA TM-3)

Conceptual Layout - Open Sand Beds at Allen Parcel

Job Number	111-53041
Revision	A
Date	14 Feb 2019

Figure 4

G:\111\11153041 Town of Falmouth South Coast CWMP Update\GIS\Maps\MXD_Deliverables\TASA TM-3\111-53041-F04_Allen_old revision.mxd

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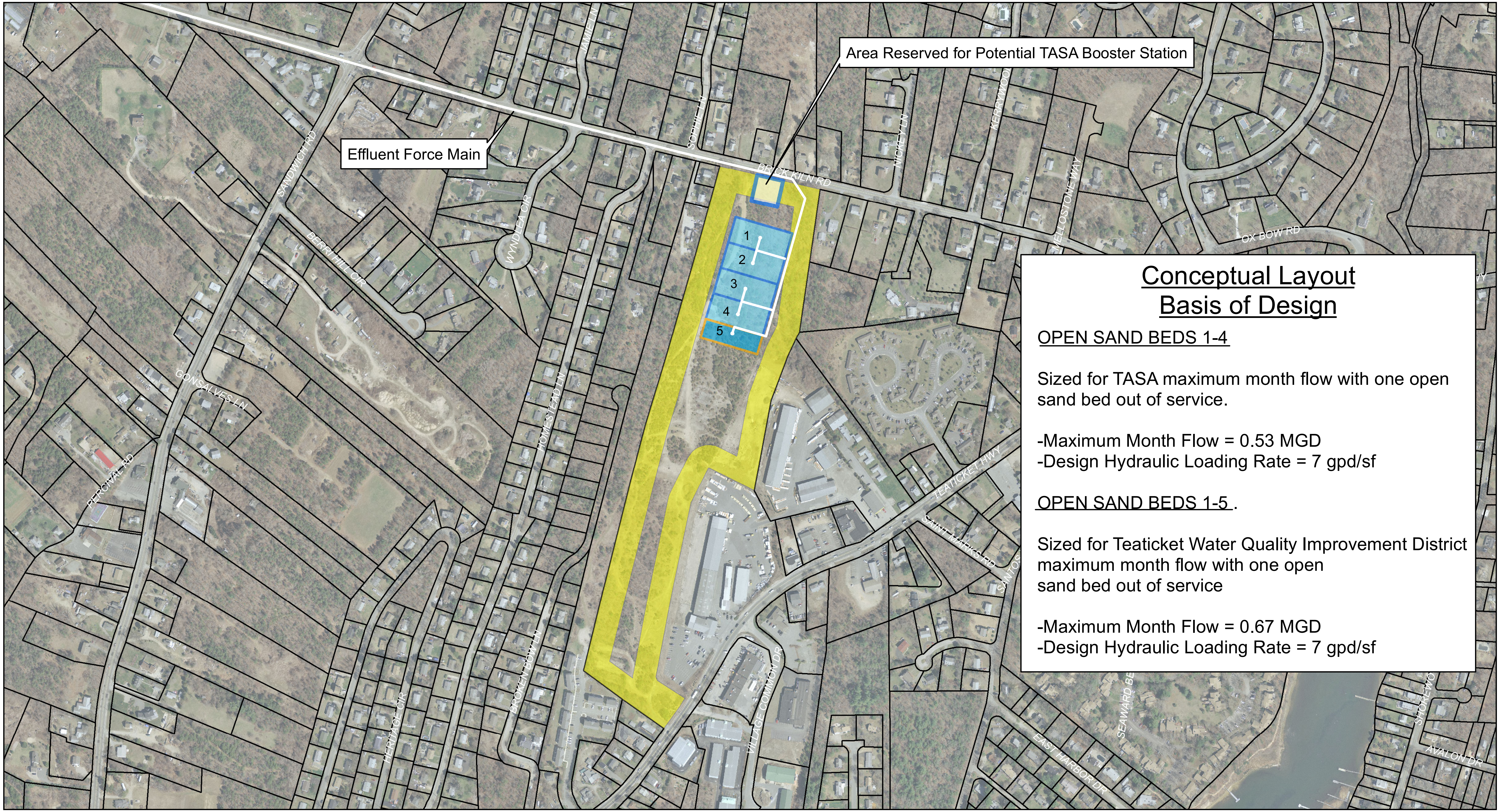

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Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

LEGEND

- Lift Station
- Conceptual Open Sand Bed for TASA
- Potential Expanded Open Sand Bed
- 100 Foot Buffer From Property Boundaries

TOWN OF FALMOUTH, MA
Teaticket/Acapesket Preliminary Evaluation (TASA TM-3)

Conceptual Layout - Open
Sand Beds at Augusta Parcel

Job Number	111-53041
Revision	A
Date	14 Feb 2019

Figure 5

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Data source: Data Custodian, Data Set Name/Title, Version/Date. Created by:jjobrien

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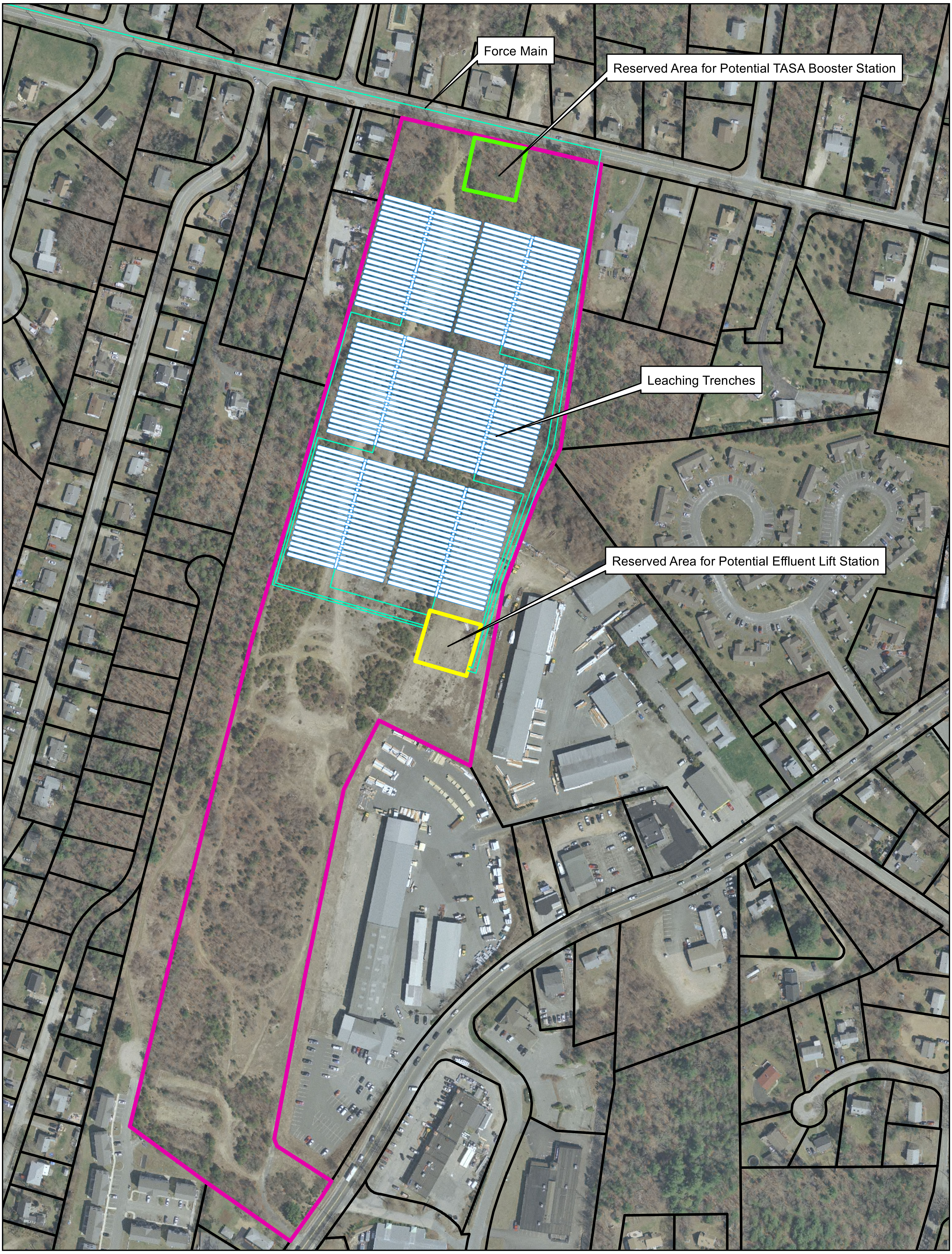

LEGEND

Property Boundary
Force Main
Conceptual Leaching Trench
Effluent Lift Station
TASA Booster Station

Paper Size ANSI B

0 75 150 300
Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: NAD 1983 2011
Grid: NAD 1983 2011 StatePlane Massachusetts Mnld FIPS 2001 FtUS

N

TOWN OF FALMOUTH, MA
Teaticket/Acapesket Preliminary Evaluation TASA (TM-3)

Job Number	111-53041-F06
Revision	A
Date	30 Oct 2018

Conceptual Layout - Leaching Trench Effluent Disposal System at Augusta Parcel

Figure 6

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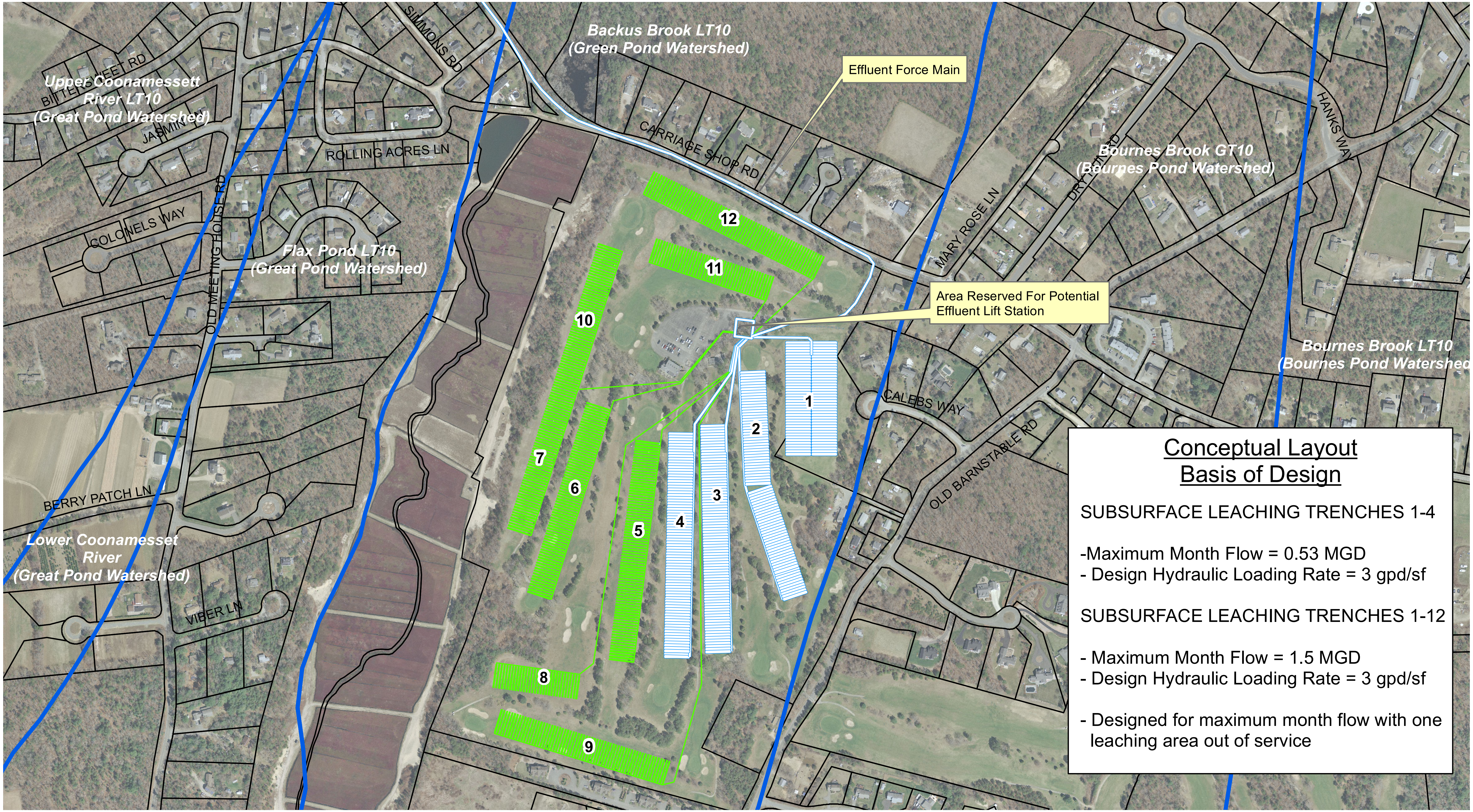

Conceptual Layout
Basis of Design

SUBSURFACE LEACHING TRENCHES 1-4

- Maximum Month Flow = 0.53 MGD
- Design Hydraulic Loading Rate = 3 gpd/sf

SUBSURFACE LEACHING TRENCHES 1-12

- Maximum Month Flow = 1.5 MGD
- Design Hydraulic Loading Rate = 3 gpd/sf

- Designed for maximum month flow with one leaching area out of service

Paper Size ANSI B

0 90 180 360
Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

Legend

- Potential Future Infiltration Trench
- Conceptual Infiltration Trench for TASA
- MEP Watershed Boundary
- Parcel Boundary

GHD

TOWN OF FALMOUTH, MASSACHUSETTS
Teaticket/Acapesket Preliminary Evaluation (TASA TM-3)

Falmouth Country Club Effluent Leaching Trenches - Conceptual Layout

Job Number	111-53041
Revision	A
Date	14 Feb 2019

Figure 7

G:\111\11153041 Town of Falmouth South Coast CWMP Update\GIS\Maps\MXD_Deliverables\TASA TM-3\111-53041-F07.mxd

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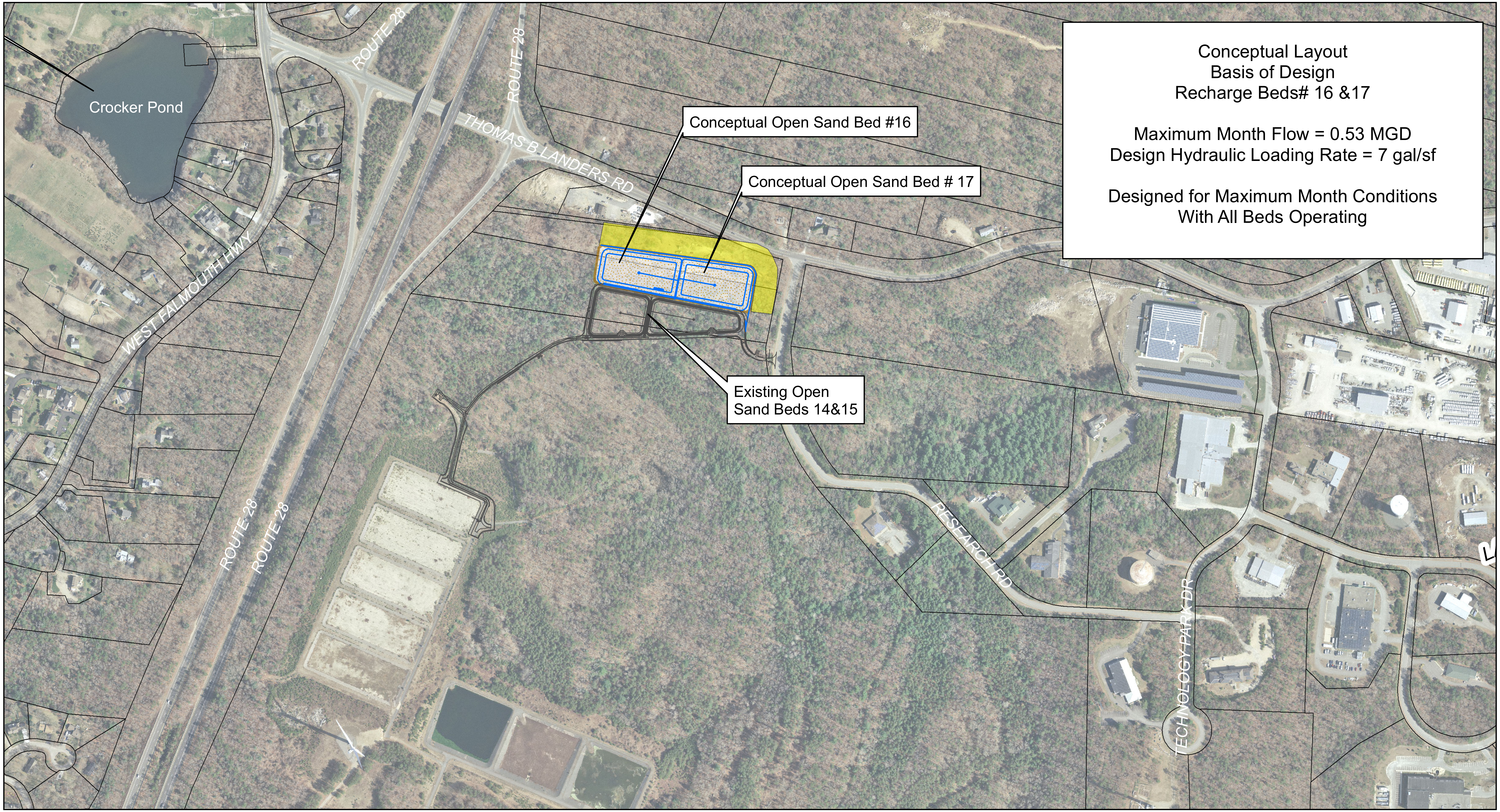

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Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

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LEGEND

Conceptual Open Sand Bed

100 Foot Buffer From Property Boundary

TOWN OF FALMOUTH, MA
Teaticket/Acapesket Preliminary Evaluation TASA (TM-3)

Job Number	111-53041
Revision	A
Date	01 Nov 2018

Conceptual Layout- Open Sand Beds
at New Site Abutting Recharge Beds 14 & 15

Figure 8

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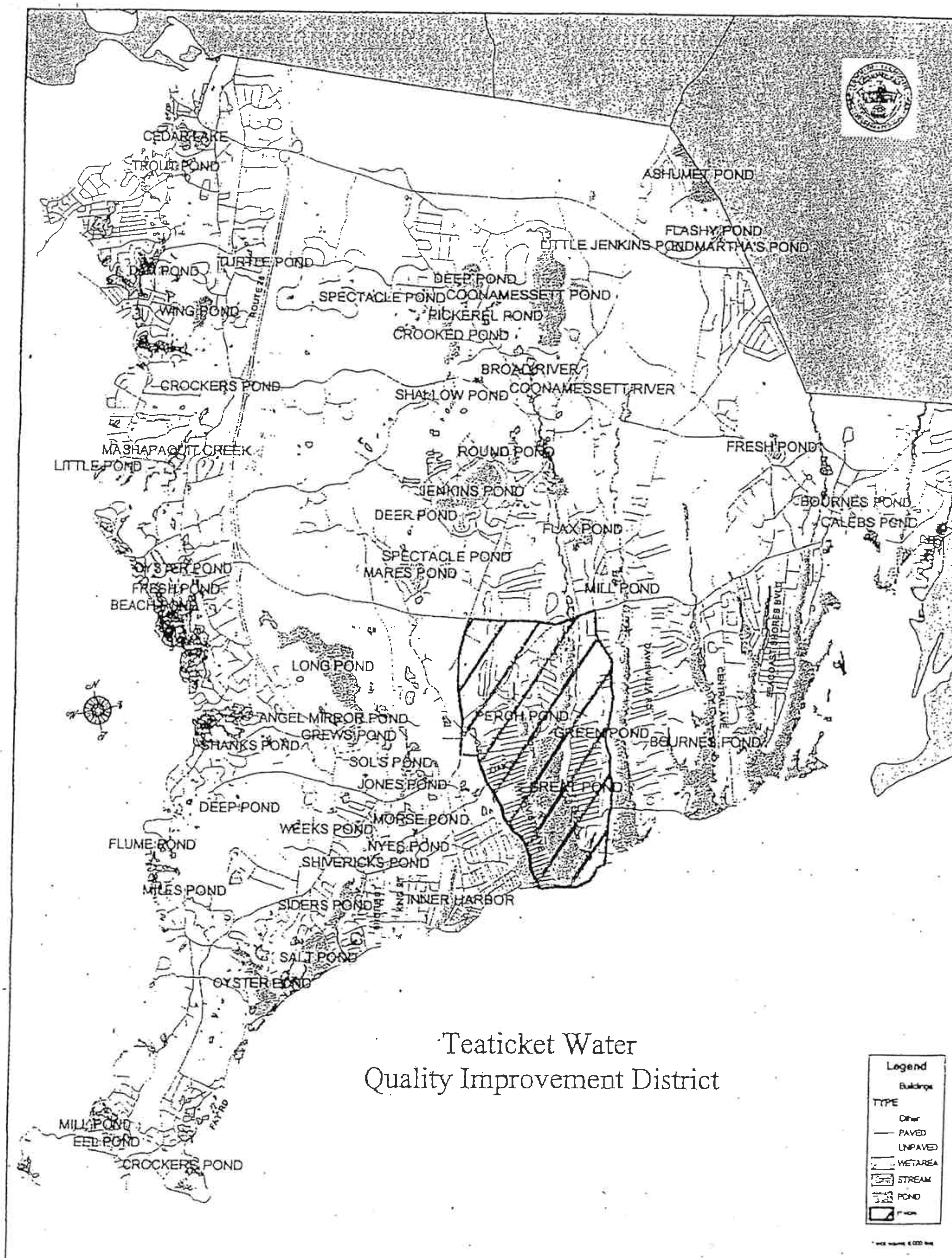
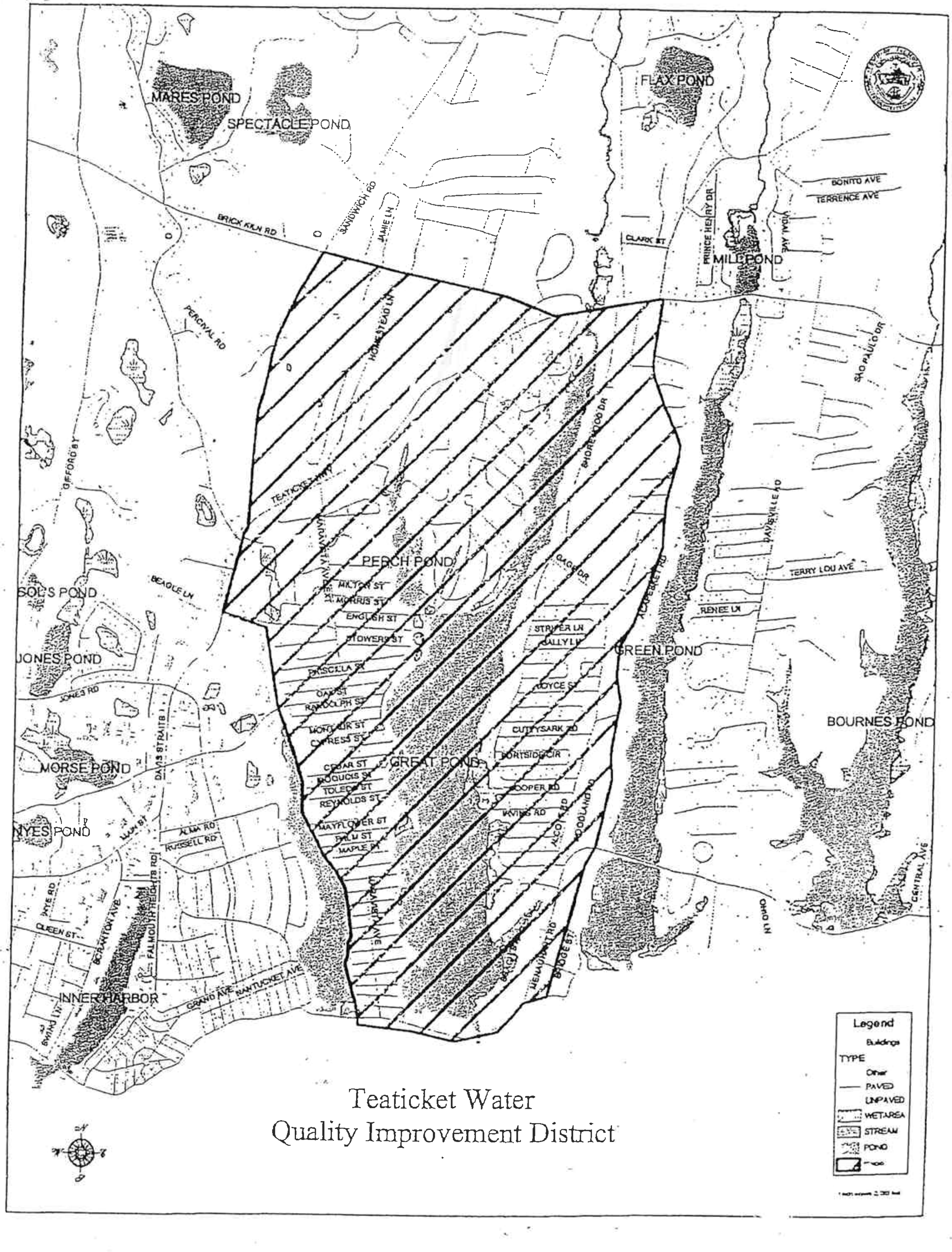
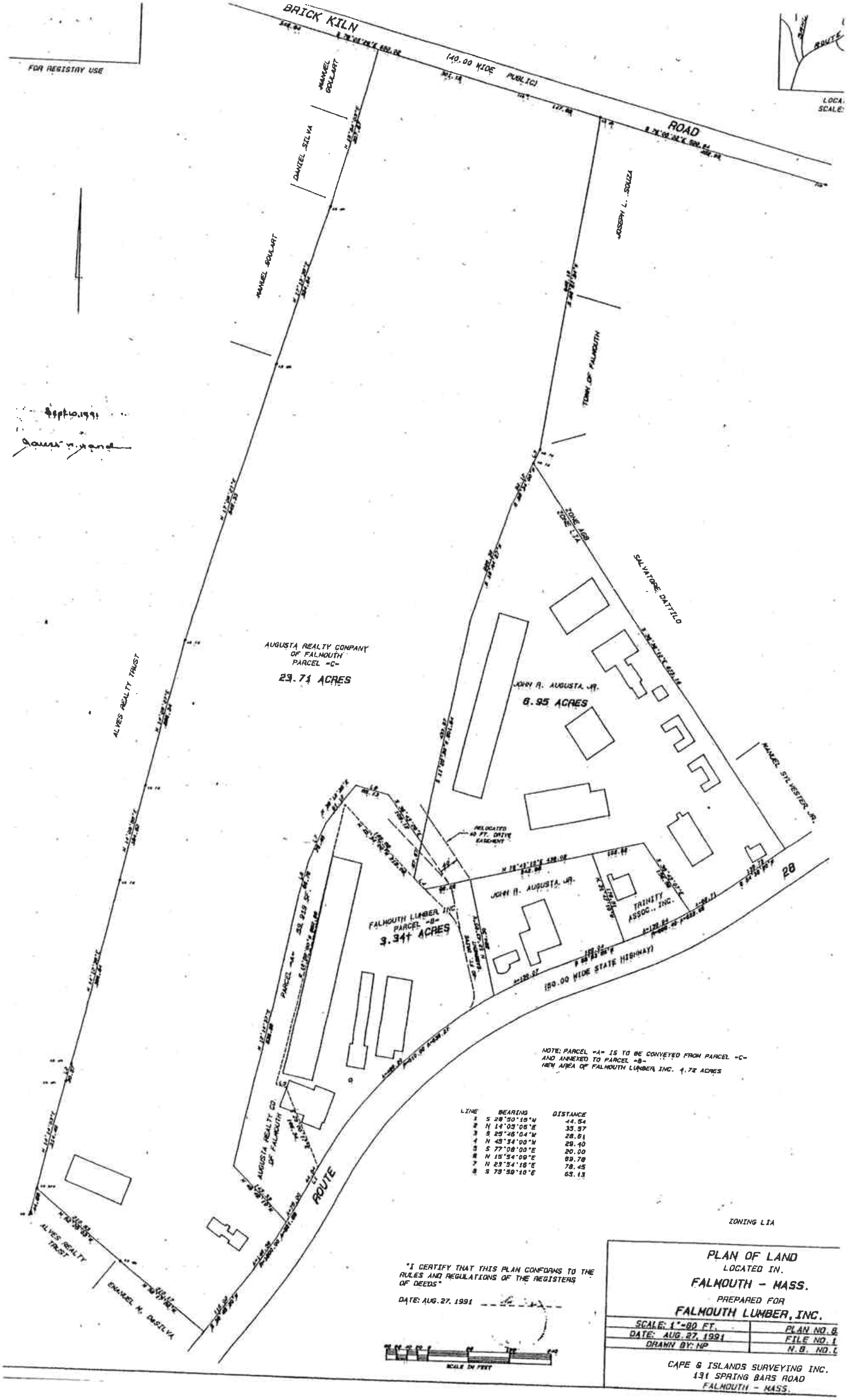
Attachment No. 1

Teaticket Water Quality Improvement District Map


CEDAR LAKE
TROUT POND
ASHUMET POND
FLASHY POND
LITTLE JENKINS POND
MARTHA'S POND
DEEP POND
TURTLE POND
ROUTE 28
SPECTACLE POND
COONAMESSETT POND
WING POND
PICKEREL POND
CROOKED POND
BROAD RIVER
CROCKERS POND
SHALLOW POND
COONAMESSETT RIVER
MASHAPAQUIT CREEK
ROUND POND
FRESH POND
LITTLE POND
JENKINS POND
BOURNES POND
DEER POND
CALEBS POND
FLAX POND
SPECTACLE POND
OYSTER POND
MARES POND
FRESH POND
MILL POND
BEACH POND
LONG POND
ANGEL MIRROR POND
PERCH POND
GREWS POND
GREEN POND
BOURNES POND
SHANKS POND
SOL'S POND
JONES POND
GREAT POND
DEEP POND
MORSE POND
WEEKS POND
NYES POND
FLUME POND
SHIVERICKS POND
MILES POND
SIDERS POND
INNER HARBOR
KING ST
SALT POND
OYSTER POND
CENTRAL AVE
Teaticket Water
Quality Improvement District
MILL POND
EEL POND
CROCKERS POND
Legend
Buildings
TYPE
Other
PAVED
UNPAVED
WETAREA
STREAM
POND



Teaticket Water
Quality Improvement District



LINE	BEARING	DISTANCE
1	S 28°50'18"W	44.54
2	N 14°03'06"E	35.57
3	S 25°46'04"W	28.61
4	N 45°34'00"W	29.40
5	S 77°08'00"E	20.00
6	N 15°54'09"E	69.78
7	N 23°54'16"E	78.45
8	S 75°50'10"E	65.13

Sept 10, 1991

"I CERTIFY THAT THIS PLAN CONFORMS TO THE RULES AND REGULATIONS OF THE REGISTERS OF DEEDS"

DATE: AUG. 27, 1991

PLAN OF LAND
LOCATED IN
FALMOUTH - MASS.
PREPARED FOR
FALMOUTH LUMBER, INC.

SCALE: 1"=80 FT.	PLAN NO. 8
DATE: AUG. 27, 1991	FILE NO. 1
DRAWN BY: NP	N.B. NO. [illegible]

CAPE & ISLANDS SURVEYING INC.
131 SPRING BARS ROAD
FALMOUTH - MASS.

April 11, 2019

To:	Town of Falmouth, MA	Ref. No.:	11153041
From:	Anastasia Rudenko, P.E., BCEE, ENV SP	Tel:	774-470-1637
	Marc Drainville, P.E., BCEE, LEED AP		774-470-1640
CC:	File; Project Team		

Subject: **South Coast Embayments – Preliminary Evaluations and Notice of Project Change Update Project**

WWTF Evaluation – Teaticket / Acapesket Study Area Technical Memorandum No. 4 (TASA TM-4)

1. Purpose of Memo

The purpose of this Technical Memorandum is to summarize the capacity evaluation that was conducted for the Falmouth Wastewater Treatment Facility (WWTF). The evaluation includes an assessment of the facility's existing capacity and identifies potential treatment upgrades required to treat flow from the Teaticket/Acapesket Study Area (TASA), which includes portions of the Great and Green Pond watersheds. The evaluation also includes an assessment of the existing operations building facilities and identifies potential upgrades for improved operations at the facility regardless of treatment expansion.

1.1 References, Regulations, and Design Guidelines

The references, regulations, and design guidelines listed below were used to develop this memorandum. Documents are referred to by the abbreviation indicated in parenthesis for the remainder of the memorandum.

References:

- 'TASA Technical Memorandum No. 1 – South Coast Embayments Preliminary Evaluations and Notice of Project Change Update Project – Draft Service Area, Flow and N Load Evaluation' prepared by GHD, dated August 2018 (2018 TASA TM-1)
- 'Shared Wastewater Management Study – Towns of Bourne, Falmouth, Mashpee, Sandwich and Joint Base Cape Cod' prepared by Wright Pierce, dated November 2017 (2017 Wright Pierce JBCC Study)
- 'Town of Falmouth Recharge Beds 14 & 15 Operations and Maintenance Manual', prepared by GHD, dated June 2017 (Recharge Beds 14&15 O&M Manual)
- 'Modified Individual Groundwater Discharge Permit No. 168-5', effective date December 22, 2015 (2015 Permit)

GHD
1545 Iyannough Road Hyannis Massachusetts 02601 USA
T 774 470 1630 **F** 774 470 1631 **W** www.ghd.com

- ‘Project Manual for Wastewater Treatment Facility Phase II – Town of Falmouth’ prepared by GHD, dated May 2015. (2015 Phase II O&M Manual)
- ‘Nitrogen Removal Optimization Planning, Falmouth Flow and Nitrogen Planning Memo’ prepared by GHD, dated September 2013 (2013 Nitrogen Optimization Study)
- ‘Comprehensive Wastewater Management Plan and Final Environmental Impact Report, and Targeted Watershed Management Plan – Little Pond, Great Pond, Green Pond, Bournes Pond, Eel Pond, and Waquoit Bay Watersheds and Recommendations for West Falmouth Harbor Watershed’ prepared by GHD, dated September 2013. (2013 CWMP/FEIR/TWMP)
- ‘Town of Falmouth, Massachusetts Wastewater Treatment Facility Improvements Falmouth Contract No. WW-03-01-C Record Drawings’ prepared by Maguire Group Inc., dated August 2013 (2013 WWTF Improvements Record Drawings)
- ‘Technical Memorandum S-3 – Wastewater and Nutrient Management Services, Existing Collection System Evaluation’ prepared by GHD, dated March 2013. (2013 TM S-3)
- ‘Technical Memorandum WW-1 – Wastewater and Nutrient Management Services, Existing WWTF and Vent Evaluation’, prepared by GHD, dated March 2013. (2013 TM WW-1)
- ‘Falmouth Wastewater Treatment Facility Operation & Maintenance Manual (Draft)’, prepared by Maguire Group Inc., dated February 2007. (2007 Draft WWTF O&M Manual)
- ‘Town of Falmouth Wastewater Treatment Facility Improvements’ drawings, prepared by Maguire Group, dated February 2003. (2003 WWTF Improvement Drawings)
- ‘Town of Falmouth, Massachusetts Contract Documents and Specifications for the Construction of the Falmouth Waste Water Treatment Facility Improvements’, prepared by Maguire Group Inc., dated February 2003. (2003 WWTF Specifications)

Regulations:

- Part I Title XIX Chapter 132A Section 16G – Publicly Owned Treatment Works; New or Modified Discharge into Ocean Sanctuaries; Prerequisites, Commonwealth of Massachusetts, 2018.
- 314 CMR 5 – Groundwater Discharge Program, prepared by Massachusetts Department of Environmental Protection (MDEP), as revised in December 2016.
- 314 CMR 20 – Reclaimed Water Permit Program and Standards, prepared by Massachusetts Department of Environmental Protection (MDEP), as revised in March 2009.

Design Guidelines:

- Guidelines for the Design, Construction, Operation, and Maintenance of Small Wastewater Treatment Facilities with Land Disposal, prepared by Massachusetts Department of Environmental Protection (MDEP), as revised in July 2018. (2018 MassDEP Small WWTF Design Guidelines)
- New England Interstate Water Pollution Control Commission, TR-16: Guides for the Design of Wastewater Treatment Works, 2011 Edition as revised in 2016 (2016 TR-16)

2. Background Information

2.1 Falmouth Wastewater Treatment Facility

The Falmouth Wastewater Treatment Facility (WWTF) was constructed in 1986. The facility originally utilized a lagoon treatment system. In 2005, the treatment process at the WWTF was upgraded from a lagoon treatment system to a Sequencing Batch Reactor (SBR) process followed by denitrification. In 2013, a Nitrogen Optimization Study (2013 Nitrogen Optimization Study) and a WWTF condition evaluation (2013 TM WW-1) were conducted by GHD. Major process-related recommendations from the studies that were implemented in 2015 include:

- Installation of additional diffuser assemblies in each existing SBR basin to increase the treatment capacity of the existing system
- Addition of a flow-paced caustic (sodium hydroxide) feed system to provide more reliable alkalinity and pH to the SBRs
- Installation of manufacturer standard analyzer assemblies for the denitrification filters to provide continuous analysis of filter influent and effluent nitrate-nitrogen
- Optimization of the methanol (carbon) injection location
- Installation of a second Blended Sludge Tank to provide operational flexibility to fill one tank and let the other settle, resulting in a cleaner decant sent to the SBRs
- Modifications to the effluent distribution structure to improve the WWTF effluent flow measurement system

Two additional open sand beds, Recharge Beds 14 and 15, were constructed in 2015-2016 to provide effluent disposal capacity for flow from the Little Pond Service Area (LPSA) outside of the West Falmouth Harbor watershed. In 2016 the WWTF started receiving flow from the LPSA.

3. WWTF Treatment Capacity

3.1 Permitted Capacity

The WWTF currently operates under Modified Groundwater Discharge Permit No. 168-5, effective date December 12, 2015 (2015 Permit). The WWTF has 15 effluent disposal open sand beds (recharge beds). Recharge Beds 1 through 13 are located within the West Falmouth Harbor watershed. Recharge Beds 14 and 15 are located outside of the West Falmouth Harbor watershed. The Permit allocates effluent flow limits by watershed. The following table compares the permitted flow that can be discharged to the existing recharge beds with the design flow of the facility. As shown in the table, the overall permitted flow of the facility is less than the design flow of the facility.

Table 3.1 WWTF Permitted Flow vs Design Flow

	Permitted Discharge Flow – Recharge Beds 1 through 13[1]	Permitted Discharge Flow – Recharge Beds 14 and 15[1]	Total Permitted Discharge Flow	Design Flow[2]
Average Daily Flow (mgd)	0.45	0.26	0.71	1.2
Maximum Day Flow (mgd)	0.80	0.47	1.27	2.2

Notes:

1. Source: 'Modified Individual Groundwater Discharge Permit No. 168-5', effective date 12/22/15.
2. Source: 'Falmouth Wastewater Treatment Facility Operation & Maintenance Manual (Draft)', prepared by Maguire Group Inc., dated February 2007.

3.2 Hydraulic (Design) Capacity

The design capacity of the WWTF is compared to future influent flows in the following table. Future flows are defined as the existing flows at the WWTF (as of 2013 – prior to the connection of LPSA), anticipated flow from LPSA (since all anticipated connections from LPSA have not been made yet this flow is kept separate from existing WWTF flows in this evaluation), and anticipated flow from TASA (if the entire Study Area is sewered).

Table 3.2 WWTF Influent Flows

Flow (mgd)	WWTF Prior to LPSA Connection[1,]	LPSA[2]	TASA[3]	Total Future	Design Flows[4]	Total Permitted Discharge Flow[5]
Average daily flow	0.36	0.26	0.32	0.94	1.2	0.71
Maximum day flow	0.68	0.47	0.55	1.7	2.2	1.27
Peak hour flow	1.22	0.88	0.93	3.03	4.3	N/A

Notes:

1. WWTF flows prior to LPSA connection were calculated based on the influent wet well flow measured flows from November 2010 through November 2012.
2. Source: 'Town of Falmouth Recharge Beds 14 & 15 Operations and Maintenance Manual', prepared by GHD, dated June 2017.
3. Technical Memorandum No. 1 – South Coast Embayments Preliminary Evaluations and Notice of Project Change Update Project – Draft Service Area, Flow and N Load Evaluation prepared by GHD, dated August 2018 plus and estimated allocation for I/I (I/I allocation will be updated once TASA conceptual layout is finalized).
4. Source: Falmouth Wastewater Treatment Facility Operation & Maintenance Manual (Draft), prepared by Maguire Group Inc., dated February 2007.
5. Source: Modified Individual Groundwater Discharge Permit No. 168-5, effective date 12/22/15.

Based on the hydraulic capacity outlined in the Town of Falmouth Wastewater Treatment Facility Improvements drawings dated February 2003 and developed by the Maguire Group, the facility will still be below capacity on a hydraulic basis with the addition of TASA. However, it should be noted that the proposed future flow is above the current permitted flow of the facility. A modification to the Town's groundwater discharge permit would be required in order to increase discharge volume and to address any change in discharge location.

3.3 Sequencing Batch Reactor (SBR) Load Capacity

Aqua-Aerobic Systems, Inc., the developer of the process model for the WWTFs existing Sequencing Batch Reactor (SBR), was contacted to determine the maximum capacity of the current system to treat future flows. Aqua-Aerobic indicated that the existing tanks and equipment are near capacity with the additional flow anticipated from LPSA with minimal excess capacity while maintaining the current effluent limits contained within the equipment's performance guarantee. Additional diffusers were installed in the existing SBR tanks in 2015 to increase the capacity of the existing system. Aqua-Aerobics has indicated that an additional (third) tank will be required to treat the anticipated load from TASA. The additional tank will also provide operational flexibility.

3.4 Condition Evaluation

A condition evaluation of existing equipment is beyond the scope of this project. However, since the existing equipment at the Falmouth WWTF will be approaching its design life approximately when TASA is anticipated to be constructed, it is recommended that the condition of each process within the WWTF be evaluated in order to identify condition-related upgrades.

4. Treatment Upgrade Requirements

Future effluent permit requirements are anticipated to be dictated by the location of future effluent discharge sites and the type of technology implemented for discharge.

Anticipated effluent permit requirements for the effluent discharge technologies evaluated as part of this project are outlined in the following table and compared to the original WWTF design parameters.

(continued)

Table 4.1 Anticipated WWTF Effluent Discharge Limits

Parameter	Open Sand Beds – Outside of a Zone II (1)	Subsurface Effluent Disposal – Outside of a Zone II (1)(2)	Class A Spray Irrigation – Outside of a Zone II (3)	Deep Well Injection (4)	Outfall – Class SA (5)	Existing WWTF Recharge Beds (additional flow with enhanced N removal)	Original WWTF Design Parameters (9)
BOD (mg/L)	30	30	10	10	30	30	10
TSS (mg/L)	30	30	5	5	30	30	5
TN (mg/L) (6)	3	3	3	3 – 5 [8]	3	<3 [10]	3
TP (mg/L)	TBD [7]	TBD [7]	TBD [7]	TBD [7]	TBD [7]	TBD [7]	N/A
Fecal Coliform (#/100 mL)	200	N/A	14	14	TBD	200	200
Enterococci (#/100 mL)	N/A	N/A	N/A	N/A	TBD	N/A	N/A
Total Organic Carbon, mg/L	N/A	N/A	N/A	1	N/A	N/A	N/A

Notes:

1. Source: Guidelines for the Design, Construction, Operation and Maintenance of Small Wastewater Treatment Facilities with Land Disposal, prepared by MassDEP, revised July 2018.
2. Category includes subsurface infiltration beds, subsurface dispersal, shallow well injection and wick wells.
3. Source: 314 CMR 20 – Reclaimed Water Permit Program and Standards, effective March 2009.
4. Source: 314 CMR 5.00 – Groundwater Discharge Permits, effective December 2, 2016.
5. Ocean outfall discharge pre-requisites are outlined in Part I Title XIX Chapter 132A Section 16. New ocean outfall discharges are required to receive advanced treatment and disinfection. Effluent parameters are not specified in the regulations. Anticipated requirements are assumed for this project.
6. Anticipated effluent TN limit of 3 mg/L is anticipated for a discharge into a nutrient impaired watershed. Per the Shared Wastewater Management Study Towns of Bourne, Falmouth, Mashpee, Sandwich and Joint Base Cape Cod report prepared by Wright-Pierce and dated November 2017, anticipated effluent TN limit for Joint Base Cape Cod discharge is 10 mg/L for open sand beds and wick wells and 5 mg/L for deep well injection and outfall (Class SB).
7. A phosphorus effluent limit may be required depending on discharge site location.
8. Effluent limit of 5 mg/L assumes that deep well injection discharge is deep enough to emerge beyond all nitrogen sensitive embayments/watersheds.
9. Town of Falmouth, Massachusetts Contract Documents and Specifications for the Construction of the Falmouth Waste Water Treatment Facility Improvements, prepared by MaGuire Group Inc., dated February 2003.
10. The current load limit is based on an average annual flow of 450,000 gpd of flow discharge to sand beds 1 through 13 at a concentration of 3 mg/L. To maintain the existing load limit the effluent nitrogen concentration would need to be reduced if additional flow was discharged to sand beds 1 through 13. This would require a permit modification.

Anticipated major process upgrades to meet the effluent discharge requirements outlined in Table 4-1 are summarized in Table 4-2.

Table 4.2 Major Process Upgrades to Meet Effluent Discharge Requirements

	Preliminary Treatment, Influent Pumping[3]	Sequencing Batch Reactors[4]	Denitrification Filters[5]	Ultraviolet Disinfection[5]	Effluent Distribution and Metering[3]
Open Sand Beds – Outside of a Zone II	N/A	Install additional tank and associated process equipment	N/A	N/A	N/A
Subsurface Effluent Disposal[1] – Outside of a Zone II	N/A	Install additional tank and associated process equipment	N/A	N/A	N/A
Spray Irrigation – Outside of a Zone II	N/A	Install additional tank and associated process equipment	N/A	Upgrade disinfection process to achieve higher level of treatment.	N/A
Deep Well Injection	N/A	Install additional tank and associated process equipment	N/A	Upgrade disinfection process to achieve higher level of treatment.	N/A
Outfall – Class SA	N/A	Install additional tank and associated process equipment	N/A	TBD	N/A
Existing WWTF Recharge Beds (if enhanced N removal is required by permit modification)	N/A	Add enhanced N removal process[2]	Add enhanced N removal process[2]	N/A	N/A

Notes:
1. Category includes subsurface infiltration beds, subsurface dispersal, shallow well injection, and wick wells.
2. Enhanced N removal process options discussed in Section 5.
3. Process is sized based on flow.
4. Process is sized based on flow, load, and permit requirements.
5. Process is sized based on flow and permit requirements.

5. Treatment Upgrade Options for Enhanced Nitrogen Removal

5.1 Enhanced Nitrogen Removal Background

The WWTF permit limits the cumulative nitrogen annual load that can be discharged within the West Falmouth Harbor watershed to 4,109 pounds per calendar year. In accordance with the 2015 Permit "the obligation to exercise best efforts to limit the annual nitrogen load to 4,109 pounds assumes that the sewer is not extended within the West Falmouth Harbor." In the future a watershed permit could be obtained for West Falmouth Harbor, potentially allowing the Town to receive credit for alternative nitrogen removal technologies within the watershed. The 4,109 lb/yr load limit is based on an average annual flow of 450,000 gpd of flow discharged to sand beds 1 through 13 at a concentration of 3 mg/L. To maintain the existing load limit (without extending sewer to the West Falmouth Harbor watershed or obtaining alterative nitrogen removal credits in the watershed) the effluent nitrogen concentration would need to be reduced if additional flow was discharged to sand beds 1 through 13. A permit modification would also be required.

Two options to enhance the existing wastewater treatment facility performance to reduce effluent nitrogen to below 3 mg/L at the WWTF are discussed below.

5.2 Lignocellulosic Media (“Wood Chip”) Treatment Systems

“Wood chip” based treatment systems can theoretically be used as a nitrogen polishing technology to remove nitrates from nitrified treatment facility effluent. Effluent total nitrogen measurements are comprised of two components—total kjeldahl nitrogen (TKN) and nitrate/nitrite (NOx). The Falmouth WWTF SBR system is designed to produce an effluent nitrogen concentration of 3 mg/L (typical breakdown is 1.5 mg/L TKN and 1.5 mg/L NOx). Falmouth WWTF data indicates that, on average, effluent nitrate is typically less than 0.5 mg/L and effluent TKN is typically greater than 1.5 mg/L. Wood chip based systems only break down nitrates, indicating that an effluent total nitrogen concentration of 1.5 mg/L could theoretically be achieved if all of the nitrate in the effluent is broken down and the effluent TKN concentration is 1.5 mg/L or less. Since the effluent TKN at the Falmouth WWTF is frequently greater than 1.5 mg/L it is not anticipated that an effluent nitrogen concentration of 1.5 mg/L could be met at the facility with this technology.

Two configurations for a “wood chip” treatment system are described below.

- In a “layer cake” approach a layer of lignocellulosic media (such as wood chips, mulch, or sawdust) is installed in a layer beneath an effluent disposal bed. The wood chip layer provides an additional opportunity for nitrate breakdown as effluent percolates through the wood chip layer. This technology could be considered for a new effluent disposal system where the wood chip layer can be incorporated during construction. Discussions with MassDEP on an appropriate sampling point for the effluent (which typically needs to be sampled prior to discharge) would be required. This type of system is being piloted with on-site systems on Cape Cod through a grant from EPAs Southeast New England Coastal Watershed Restoration Program. The system has not been sized or piloted for larger municipal type systems.
- In a “filter unit” approach a saturated tank of lignocellulosic media could be added as a treatment step following the denitrification filters (prior to effluent discharge). This approach would provide a more easily accessible effluent sampling point prior to effluent discharge. A smaller version of this system (the NITREX™ system) is designed to be added to the end of an innovative/alternative (I/A) septic system. Conversations with the Massachusetts Alternative Septic System Test Center (MASSTC) indicate that this technology could theoretically be implemented on a larger scale as an up-flow system at the end of a treatment process for nitrate reduction. Sizing for the system would be based on a 40% porosity and minimum one day residence time.

As of the writing of this memorandum, no testing has been conducted for a “wood chip” treatment system at the end of a WWTF treatment process. Due to the high effluent TKN concentration and low nitrate concentration at the WWTF, this technology is not anticipated to consistently achieve an effluent nitrogen level of less than 3 mg/L.

5.3 Membrane Treatment

Membrane treatment processes can remove nitrogen and other organic molecules (such as total organic carbon) through filtration. Organic molecules greater in size than the membrane pores are rejected based on size exclusion.

Pressure-driven membrane processes are typically grouped into the following categories, in order of decreasing pore size:

- Microfiltration (MF)
- Ultrafiltration (UF)
- Nanofiltration (NF)
- Reverse Osmosis (RO)

MF and UF are typically used to remove particles, while NF and RO are used to remove dissolved materials. It is anticipated that filtration through an RO membrane will be required to meet an effluent nitrogen level of less than 3 mg/L. While RO membranes are effective in removing dissolved constituents, they are not designed for removal of suspended material, and MF is typically required as a pre-treatment step for solids removal. Feed water with a low suspended solids concentration is critical for successful implementation of the RO process. By removing suspended and colloidal solids from treated wastewater, the MF protects the RO system from excessive fouling from particulate matter. An option for the Falmouth WWTF is to convert the existing SBR tanks to a Membrane Biological Reactor (MBR) system to provide secondary treatment and the required solids reduction prior to RO within one process.

The RO process produces two streams—permeate (filtered effluent) and an RO concentrate stream.

1. RO Permeate: RO permeate is a de-mineralized stream with a depressed pH (potentially as low as 5, depending on feed water composition) and virtually no alkalinity or hardness remaining due to the removal of dissolved metals through the filtration process. The stream needs to be “stabilized” to recover hardness and prevent corrosion of downstream systems. Stabilization could include several steps to reduce the corrosive properties of the RO permeate stream. One method of hardness adjustment is to allow a fraction of the MF filtrate flow to bypass the RO system, and to blend the two streams together to recover hardness. Chemical addition may be required to adjust the pH of the final effluent to meet the effluent permit range of 6.5 to 8.5.
2. RO Concentrate: A major challenge of RO is handling the concentrated reject water. RO recovery is typically 80 to 85% in a wastewater application, resulting in a 10 to 15% reject stream. Options to treat the reject water include:
    - Evaporation—A mechanical vapor compression (MVC) evaporator or thermal evaporator could potentially be used to remove water from the RO concentrate stream. An evaporator converts the water portion of stream to water vapor, further concentrating the remaining components of the waste stream. The remaining components can be further dewatered with a centrifuge and belt filter press or hauled off-site in a liquid state. This technology is typically used in industrial applications.
    - Discharge to less nutrient-sensitive watershed—RO concentrate can potentially be blended and discharged to a less nutrient-sensitive watershed. For example the Orange County Water District’s

Groundwater Replenishment System discharges RO concentrate through an ocean outfall. If the shared wastewater management strategy at Joint Base Cape Cod (JBCC) proceeds, RO concentrate could potentially be blended with wastewater flow and sent to JBCC, which does not discharge to an identified nitrogen-sensitive watershed and is anticipated to have a higher effluent nitrogen limit than the Falmouth WWTF (JBCC's current effluent limit is 10 mg/L).

6. Operations Building Analysis

6.1 Background

The Operations Building at the Falmouth WWTF was originally constructed in the 1980's. The building served as the single location for office, electrical, and process equipment for the original plant. This building has continued to serve as the main office area and also has some functions related to process and maintenance.

6.2 Needs Identification

The overall condition of the Operations Building was assessed by GHD during a site visit in May 2018. Overall the building was found to have a sufficient structural "skeleton" and a functional layout concerning access, safety, and flexibility. The facility has limited storage capacity.

As part of the assessment, each of the buildings systems and rooms were evaluated to identify needs. The Town then provided prioritization of the needs identified. The following table summarizes these needs and prioritizations.

(continued)

Table 6.1 Operations Building Condition Assessment Summary

	Item	Included in Conceptual Layout #			
		1	2	3	4
Priority 1	Replace existing HVAC system with a unit that includes air conditioning.	Yes	Yes	Yes	Yes
	Replace existing windows.	Yes	Yes	Yes	Yes
	Replace or cover existing façade.	Yes	Yes	Yes	Yes
	Replace existing roof.	Yes	Yes	Yes	Yes
	Remove non-functioning solar collection system.	Yes	Yes	Yes	Yes
	Replace all exterior doors, including overhead door.	Yes	Yes	Yes	Yes
	Reconfigure existing roof access hatch.	Yes	Yes	Yes	Yes
	Add a new conference/training room.	Yes	Yes	Yes	Yes
	Add a new RTU/PLC room.	Yes	Yes	Yes	Yes
	Separate Control Room from other building functions.	Yes	Yes	Yes	Yes
Priority 2	Add a Chief Operator's Office.	Yes	Yes	Yes	Yes
	Upsize and renovate existing break room.	Yes	Yes	Yes	Yes
	Renovate rest rooms.	Yes	Yes	Yes	Yes
	Demolish un-used equipment in the basement.	Yes	Yes	Yes	Yes
	Rework equipment hatch to the basement.	Yes	Yes	Yes	Yes
	Add a file/book storage room.	Yes	Yes	Yes	Yes
Priority 3	Rework electrical switchboard and remove old buckets.	Yes	Yes		Yes
	Add separate garage space.[1]				
	Relocate electrical room (potentially to the basement).	Yes	Yes		Yes
	Add a larger men's room.	Yes	Yes	Yes	Yes

Notes:

1. Garage space budget cost developed as a separate cost item.

6.3 Conceptual Layouts

As part of the evaluation, four layouts were developed to address prioritized needs (see Attachment).

1. Option 1 – Rear / West Expansion
2. Option 2 – Front / East Expansion
3. Option 3 – Side / South West Expansion
4. Option 4 – Side / South East Expansion

A potential storage garage layout was developed (see Attachment).

Budgetary costs for the four options and a separate garage are outlined in Table 6-2. Capital costs are the total estimated project costs with allowances for construction costs including:

- A 30 percent construction contingency;
- 25 percent engineering design and construction;
- 15 percent fiscal/legal/permitting/administrative costs; and
- 2 percent survey, hazardous materials survey and soil boring costs (for design)

Because of the conceptual nature of this evaluation, a 30 percent contingency is carried as no detailed design has been performed. During final design a reduced contingency will be carried for variability in the bidding climate, project changes before bidding, and change orders due to unforeseen conditions. Project costs are presented in 2018 dollars (ENR index = 10959). Once the construction timeframe is known, project costs should be adjusted to the mid-point of construction.

Table 6.2 Operations Building Expansion/Upgrade – Budgetary Costs

Layout	Budgetary Construction Cost	Budgetary Capital Cost
Conceptual Layout 1	$3,000,000	$4,300,000
Conceptual Layout 2	$3,000,000	$4,300,000
Conceptual Layout 3	$2,700,000	$3,800,000
Conceptual Layout 4	$3,100,000	$4,400,000
Separate Garage Space	$800,000	$1,100,000

Notes:

1. All costs are shown in 2018 dollars (ENR index = 10959). Once a construction timeframe is known for the project, costs should be adjusted to the mid-point of construction.

7. Summary and Next Steps

7.1 Capacity Analysis

The facility is currently close to capacity on a load basis. An additional SBR tank and associated process equipment will be required to treat the proposed flow from TASA. Additionally, it is anticipated that an upgraded disinfection system will be required if spray dispersal or deep well injection is utilized for effluent disposal. In order to meet the load limit for the existing sand beds enhanced nitrogen removal would be required if additional flow were sent to the existing open sand beds. A permit modification would also be required prior to sending additional flow to any existing or new discharge site.

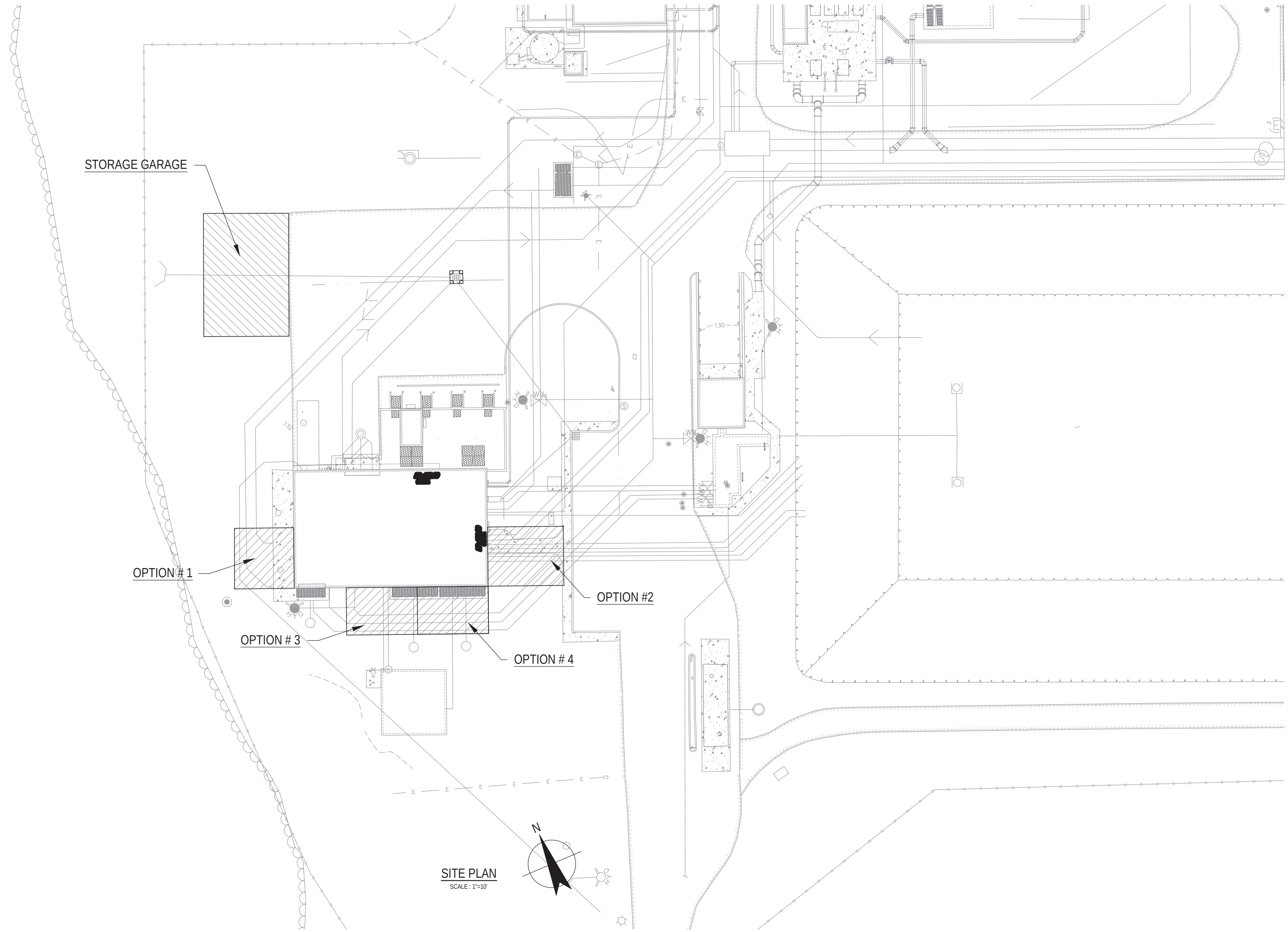
In the next phase of this project cost estimates will be developed for four to five conceptual plans identifying sewer service area, collection/transmission system (and lift station site), discharge plan, and WWTF upgrades. It is also recommended that a condition evaluation be conducted at the Falmouth WWTF.

7.2 Operations Building Analysis

Potential improvements to the layout of the existing Operations Building were identified and prioritized as part of this evaluation. Four conceptual layouts were developed to address the identified needs. A conceptual layout for a separate storage garage was also developed. The Operations Building analysis is independent of the other evaluations included in this memorandum. The conceptual layouts should be reviewed and a decision made on which conceptual layout to move forward with as part of a separate design project.

Attachment

Operations Building – Conceptual Designs



No	Revision	Drawn	Job Manager	Project Director	Date
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0	FOR CONSTRUCTION		MRD	WRH	1/15

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SCALE 1"=40' AT ORIGINAL SIZE

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Date	
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Client **TOWN OF FALMOUTH, MA**

Project **WASTEWATER TREATMENT FACILITY PHASE 2 IMP.**

Title **SITE PLAN**

Contract No. WW-14-03

Original Size **Arch D** **Drawing No: 111-53041-C01** **Rev: 1**

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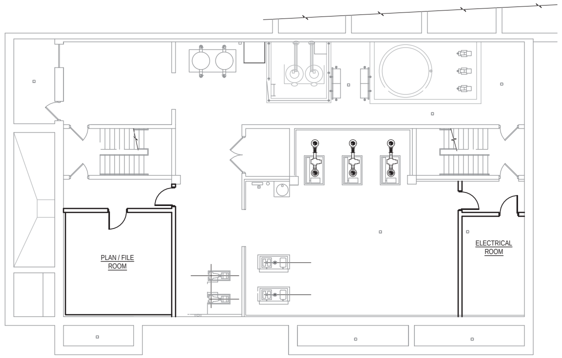

LOWER LEVEL PLAN OPTION #1

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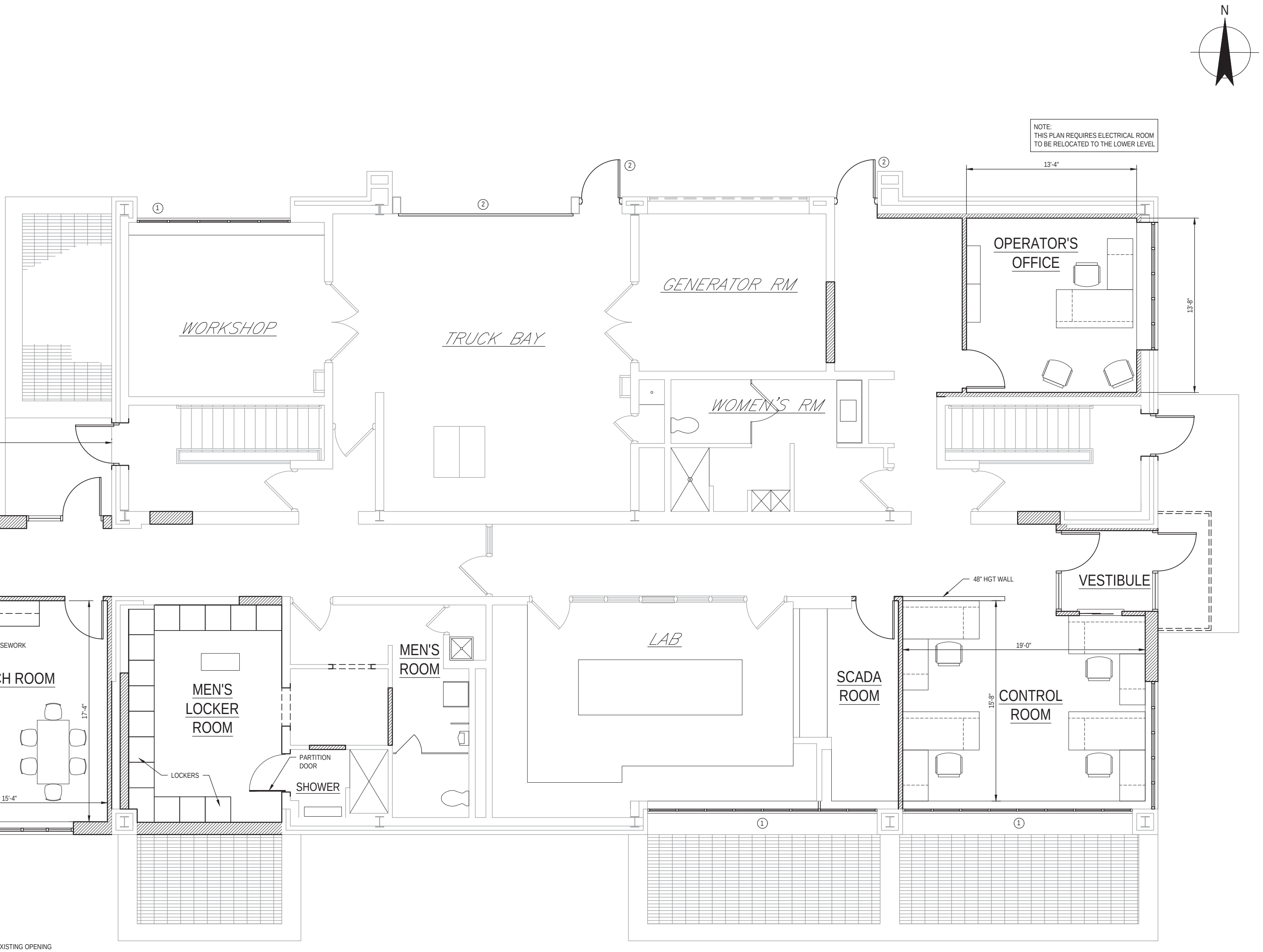

LEGEND

= EXISTING WALLS

= PROPOSED WALLS

(1) = NEW WINDOWS IN EXISTING OPENING

(2) = NEW DOOR IN EXISTING OPENING

FIRST FLOOR PLAN OPTION #1

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Drafting Check | Design Check

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Project **OPERATIONS BUILDING ADDITIONS AND ALTERATIONS**

Title **FIRST FLOOR PLAN OPTION #1**

Contract No.

Original Size **Arch D**

Drawing No: 111-53041-A1

Rev:

Plot Date: 18 July 2018 - 8:46 AM Plotted by: Mike Simizon Cad File No: N:\US\Hyannis\Projects\111\11153041 Town of Falmouth South Coast CWMP Update\CADD\Drawings\Architectural\111-53041-A01.dwg

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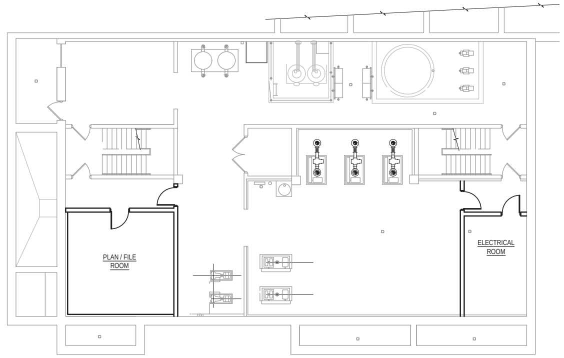

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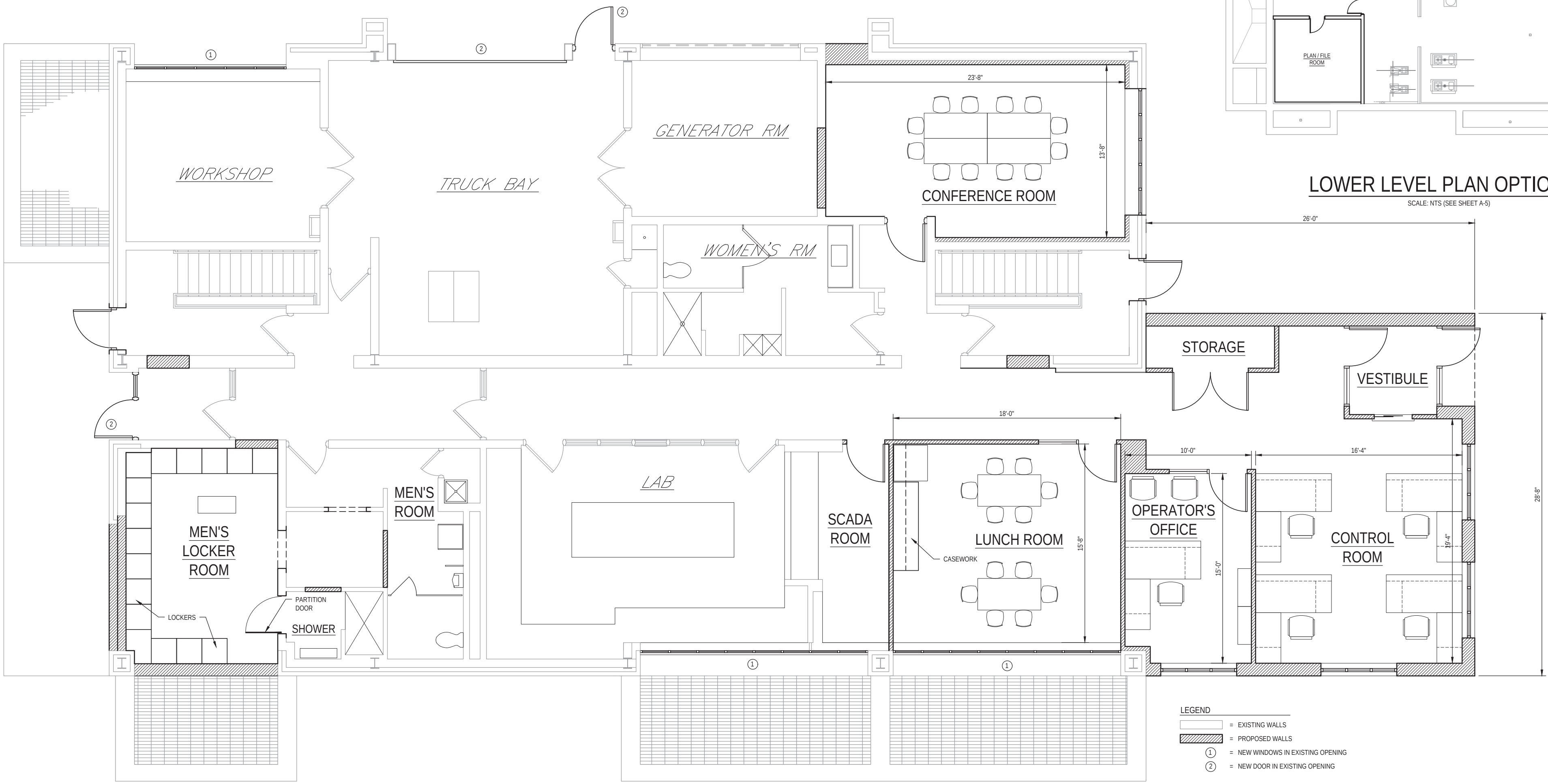

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2 = NEW DOOR IN EXISTING OPENING

FIRST FLOOR PLAN OPTION #2

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Title FIRST FLOOR PLAN OPTION #2
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Original Size Arch D
Drawing No: 111-53041-A2
Rev:

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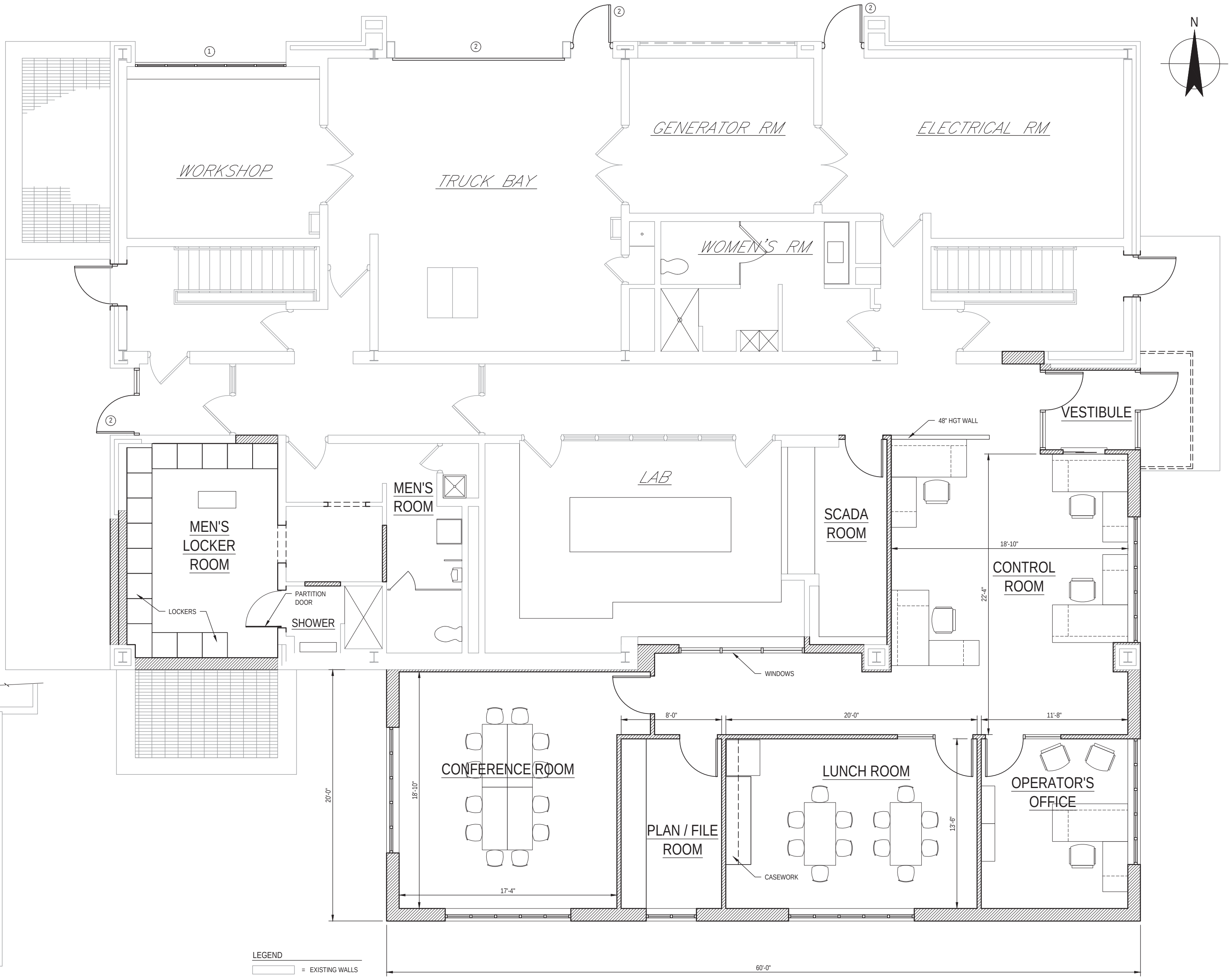

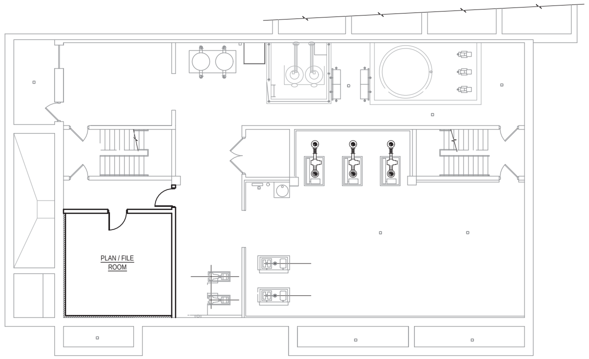

LEGEND

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= PROPOSED WALLS

(1) = NEW WINDOWS IN EXISTING OPENING

(2) = NEW DOOR IN EXISTING OPENING

LOWER LEVEL PLAN OPTION #3

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FIRST FLOOR PLAN OPTION #3

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Project **OPERATIONS BUILDING ADDITIONS AND ALTERATIONS**

Title **FIRST FLOOR PLAN OPTION #3**

Contract No.

Original Size Arch D

Drawing No: **111-53041-A3**

Rev:

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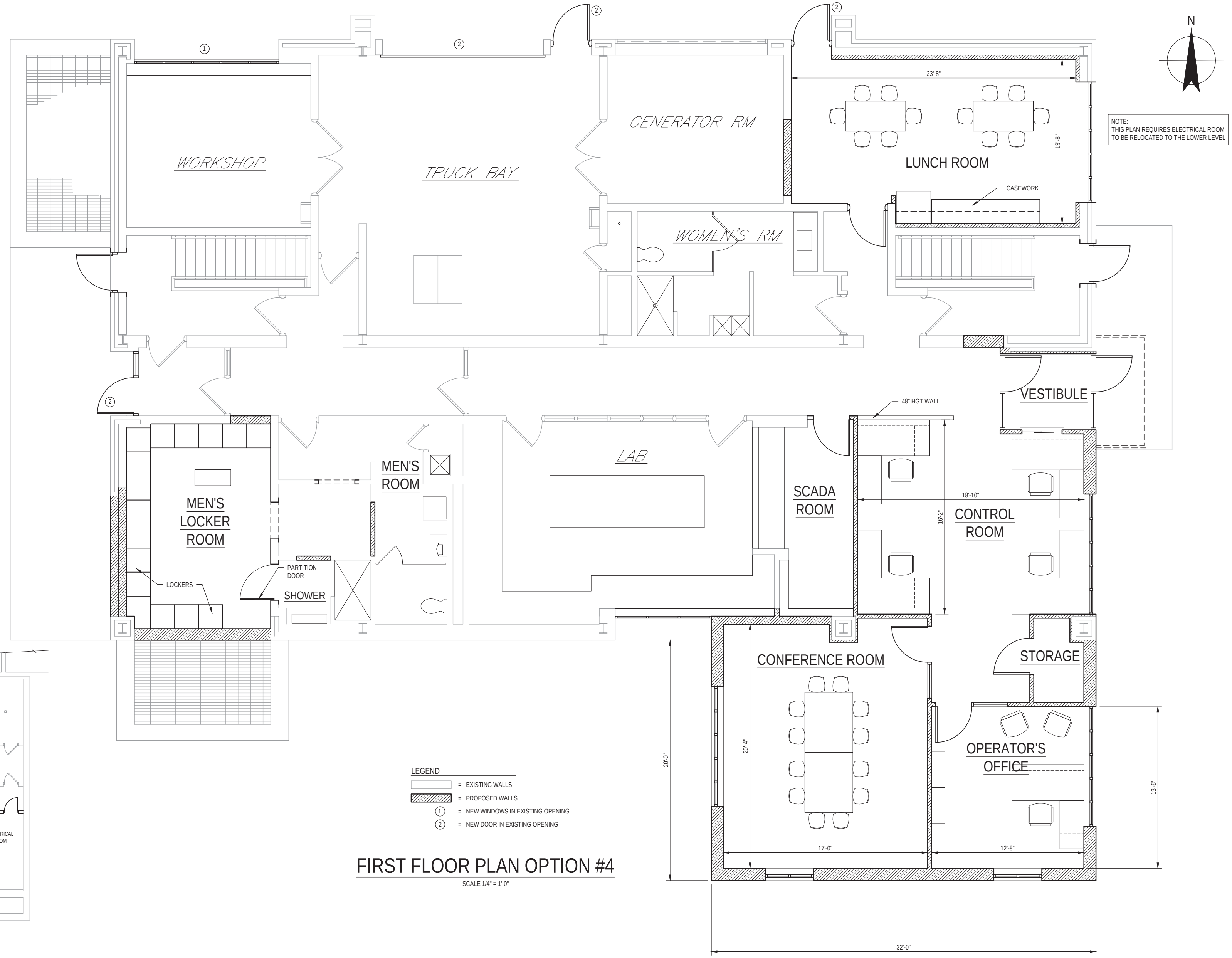

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FIRST FLOOR PLAN OPTION #4

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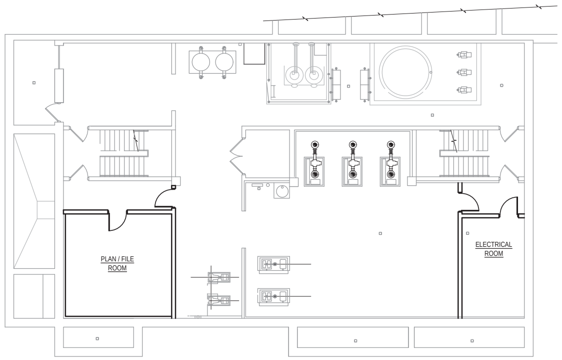

LOWER LEVEL PLAN OPTION #4

SCALE: NTS (SEE SHEET A-5)

No	Revision	Note: * indicates signatures on original issue of drawing or last revision of drawing	Drawn	Job Manager	Project Director	Date
1						
0						

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E hyamail@ghd.com **W** www.ghd.com

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Approved (Project Director)
Date
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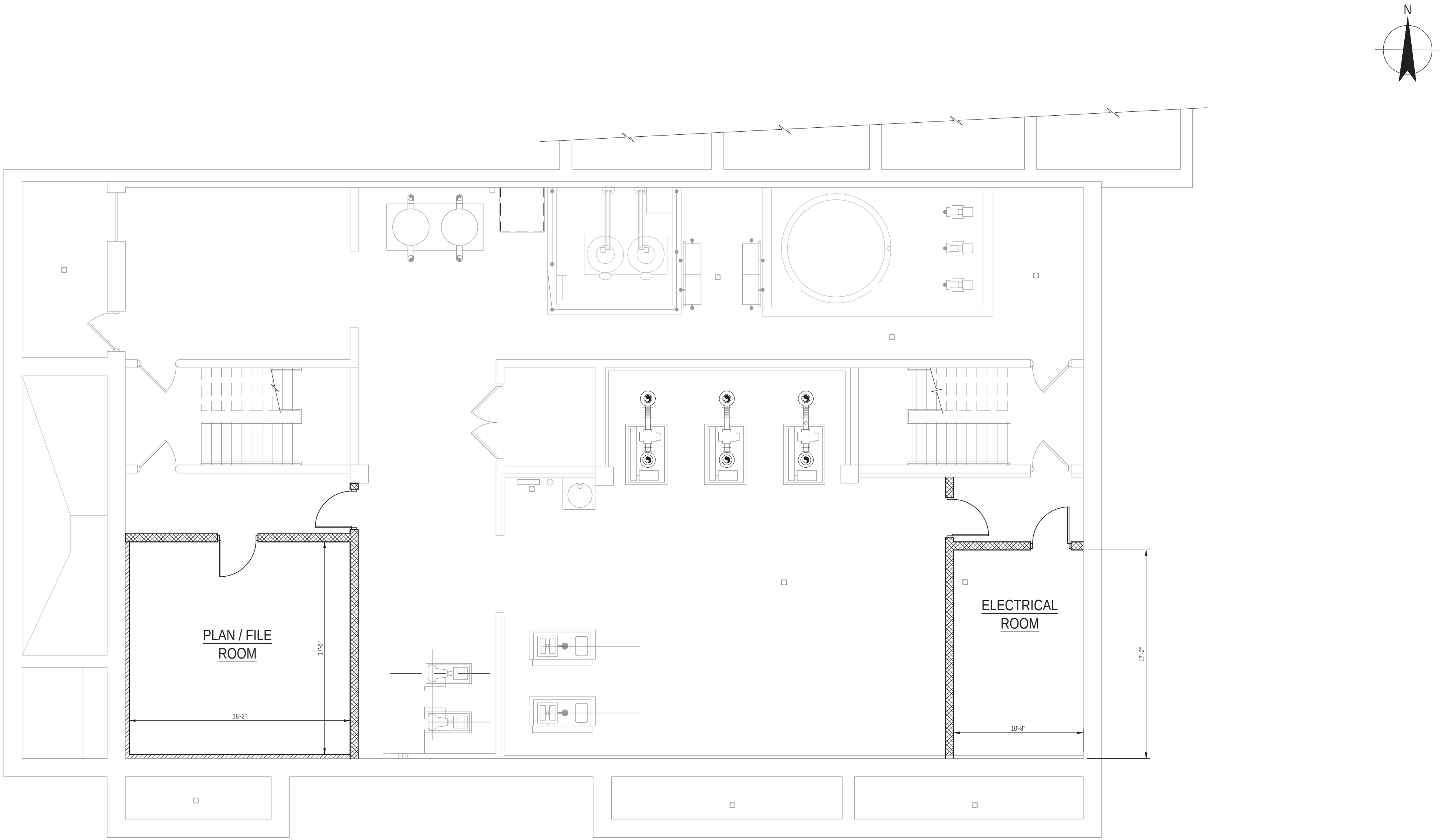
Client **TOWN OF FALMOUTH, MA**
Project **OPERATIONS BUILDING ADDITIONS AND ALTERATIONS**
Title **FIRST FLOOR PLAN OPTION #4**
Contract No.
Original Size **Arch D** **Drawing No: 111-53041-A4** **Rev:**

Plot Date: 18 July 2018 - 8:48 AM Plotted by: Mike Simizon Cad File No: N:\US\Hyannis\Projects\111\11153041 Town of Falmouth South Coast CWMP Update\CADD\Drawings\Architectural\111-53041-A01.dwg



LOWER LEVEL PLAN OPTIONS #1, 2, 4

SCALE 1/4" = 1'-0"

LEGEND

= EXISTING WALLS

= PROPOSED WALLS

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Project **OPERATIONS BUILDING ADDITIONS AND ALTERATIONS**
Title **LOWER LEVEL PLAN OPTION #3**
Contract No.
Original Size **Arch D** **Drawing No: 111-53041-A5** **Rev:**

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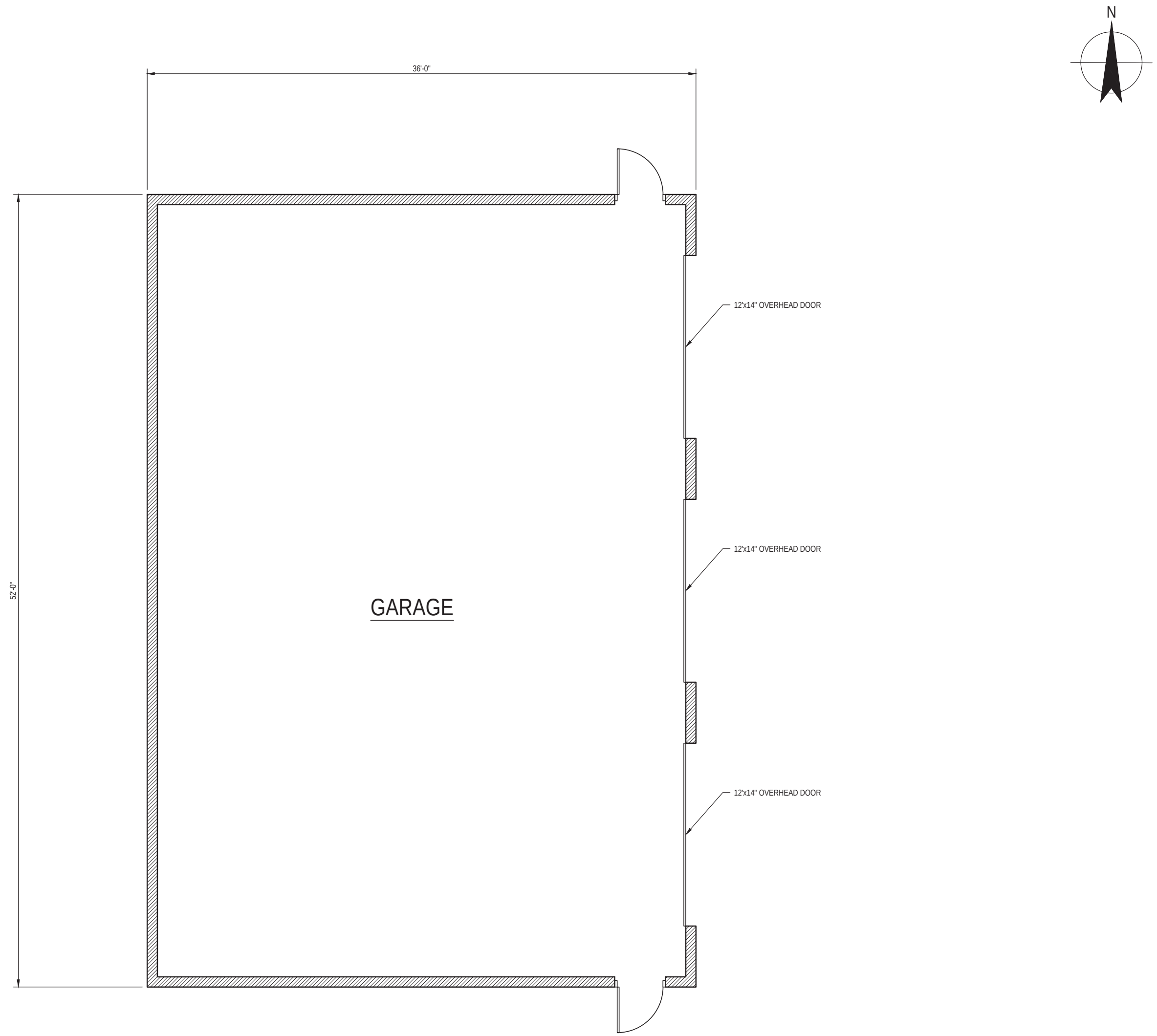

FLOOR PLAN

SCALE 1/4" = 1'-0"

No	Revision	Drawn	Job Manager	Project Director	Date
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NOTES: UNDERGROUND FACILITIES, STRUCTURES, AND UTILITIES HAVE BEEN PLOTTED FROM AVAILABLE SURVEYS AND RECORDS, AND THEREFORE THEIR LOCATIONS MUST BE CONSIDERED APPROXIMATE ONLY. THERE MAY BE OTHERS, THE EXISTENCE OF WHICH IS PRESENTLY NOT KNOWN. ANYONE USING UTILITY INFORMATION AND DATA PROVIDED HEREIN SHALL CALL DIG SAFE AT 811 SEVENTY TWO (72) HOURS, 3 BUSINESS DAYS IN ADVANCE TO VERIFY THE LOCATION OF UTILITIES PRIOR TO START OF CONSTRUCTION.

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Date
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Client **TOWN OF FALMOUTH, MA**
Project **OPERATIONS BUILDING ADDITIONS AND ALTERATIONS**
Title **GARAGE PLAN**

Contract No.

Original Size **Arch D** **Drawing No: 111-53041-A6** **Rev:**

Plot Date: 18 July 2018 - 9:49 AM Plotted by: Mike Simizon Cad File No: N:\US\Hyannis\Projects\111\11153041 Town of Falmouth South Coast CWMP Update\CADD\Drawings\Architectural\111-53041-A01.dwg

April 11, 2019

To:	Town of Falmouth, MA	Ref. No.:	11153041
From:	Anastasia Rudenko, P.E., BCEE, ENV SP	Tel:	774-470-1637
	J. Jefferson Gregg, P.E., BCEE		774-470-1640
	Adam Scicchitano, EIT		
CC:	File; Project Team		

Subject: **South Coast Embayments – Preliminary Evaluations and Notice of Project Change Update Project**

Teaticket / Acapesket Study Area Technical Memorandum No. 5 - Hydraulic Load Tests at the Augusta Parcel and Allen Parcel (TASA TM-5)

1. Purpose of Memo

The purpose of this Technical Memorandum is to summarize the field investigations that were completed to determine the suitability of two potential effluent disposal sites for receiving and infiltrating effluent from a wastewater treatment facility (WWTF). The two sites are the Augusta Parcel and Allen Parcel (Figures 1 and 2).

2. Background

The Town is currently undertaking a preliminary evaluation of sewer project alternatives for the Teaticket Acapesket Study Area (TASA) which includes portions of the Great Pond watershed and the Green Pond watershed. Field investigations were conducted for two potential effluent disposal sites chosen by the Town and the Water Quality Management Committee (WQMC)—the Augusta Parcel and the Allen Parcel. Both parcels were identified as potential effluent disposal locations in the 2007 ASAR. Both sites are municipally owned and were purchased by the Town as possible treated wastewater recharge sites. A single percolation test was conducted at the Allen Parcel in 2010. Prior to the field investigations outlined in this memorandum, no sub-surface investigations had been conducted on the Augusta Parcel.

3. References

The references and guidelines listed below were used to develop this memorandum. Documents are referred to by the abbreviation indicated in parenthesis for the remainder of the memorandum.

<u>References:</u>

- “TASA TM-3 – Draft Teaticket/Acapesket Study Area Discharge Technologies Evaluation – Rev 1”, prepared by GHD and dated November 2018 (2018 TASA TM-3)
- “Technical Memorandum No. 8 – Hydraulic Load Tests at Sites 7 and 10”, prepared by GHD and dated August 2011 (2011 TM-8)
- “Little Pond, Great Pond, Green Pond, Bournes Pond, Eel Pond and Waquoit Bay Watersheds Alternatives Screening Analysis Report” prepared by GHD (formally Stearns & Wheler), dated November 2007 (2007 ASAR)
- “Ground-water Recharge Area and Travel Times to Pumped Wells, Ponds, Streams, and Coastal Water Bodies, Cape Cod, Massachusetts” prepared by USGS and dated 2004 (2004 USGS Cape Cod Groundwater Contours Map)

<u>Guidelines:</u>

- “Guidelines for the Design, Construction, Operation and Maintenance of Small Wastewater Treatment Facilities with Land Disposal”, prepared by MassDEP and revised in July 2018 (2018 MassDEP Small WWTF Guidelines)
- “Process Design Manual – Land Treatment of Municipal Wastewater Effluents (EPA/625/R-06/016)”, prepared by EPA, dated September 2006 (EPA/625/R-06/016)

4. Field Investigations

Subsurface field investigations were conducted at the Augusta and Allen Parcel in September and October 2018. A work plan, consistent with the 2018 MassDEP Small WWTF Guidelines, was developed for the field investigations and submitted to MassDEP. The work plan is attached as Appendix A. The field investigations consisted of groundwater monitoring well installations, soil borings, percolation tests, and hydraulic load tests. Each investigation is outlined in this section.

4.1 Groundwater Monitoring Well Installations and Soil Borings

A groundwater monitoring well (GMW) was installed at each site in proximity to the hydraulic load test location for the parcel. The two GMWs are identified as AU-1 (Augusta Parcel) and AL-1 (Allen Parcel). In addition, a GMW was also installed at the south end of the Augusta Parcel (AU-2) to measure groundwater elevation near the eastern property boundary which borders a private business (Falmouth Lumber). Due to the proximity to an abutter and the topographical drop-off at this location, it is recommended that groundwater elevation be monitored at AU-2 in order to characterize the depth to groundwater at this location throughout the year.

Groundwater elevations were measured throughout the field investigations at each site. The groundwater data collected from the monitoring wells during installation is summarized in Table 4.1.

Table 4.1 – Groundwater Monitoring Well Installation Data

	GMW AU-1	GMW AU-2	GMW AL-1
Well Depth (ft)	42	39	46.5
Well Screen Depth – Measured from Ground Elevation (ft)	27-37	24 - 34	31.5 – 41.5
Groundwater Depth – Measured from Ground Elevation (ft)	32	29	36

Soil boring information was collected during the installation of each well. The soil boring logs (Appendix B) indicate sandy soil with traces of gravel at both parcels.

4.2 Percolation Test

Two percolation tests (TP-1 and TP-2) were conducted at the Augusta Parcel and one percolation test (TP-3) was conducted at the Allen Parcel. A deep observation hole was also dug at each percolation test site. The deep observation holes at the Augusta Parcel (Figures 4.1 and 4.2) indicated that at least four feet of naturally occurring pervious material exists at both sites. The percolation rate at both holes was less than two minutes per inch.

At the Allen Parcel (Figure 4.3) a layer of unsuitable material[1], approximately 10-inches thick from 28-inches to 38-inches below the surface, was observed in the deep observation hole. The deep observation hole indicated at least of four feet of naturally occurring pervious material at the site under the unsuitable material. The unsuitable materials failed a percolation test. A second percolation test, which was conducted once the unsuitable material was excavated, indicated a percolation rate of less than two minutes per inch. The unsuitable material would need to be removed if this site was developed for effluent disposal.

Percolation test findings for both sites are included in Appendix C. Percolation test results from field investigations conducted in 2010 on the Allen Parcel are included in Appendix D. The 2010 percolation test results indicated a percolation rate of less than two minutes per inch. Unsuitable material was not found at this test location. Since one percolation test had already been conducted at the site it was decided that only one additional percolation test was needed as part of the 2018 field investigations.

[1] Unsuitable refers to soils that are not conducive for the installation of a land-based effluent discharge system due to their low percolation rates.

Figure 4.1 – Deep Observation Hole at TP-1 (Augusta Parcel)

Figure 4.2 – Deep Observation Hole at TP-2 (Augusta Parcel)

Figure 4.3 – Deep Observation Hole at TP-3 (Allen Parcel)

4.3 Hydraulic Loading Tests (HLT)

Hydraulic loading tests are essentially large-scale percolation tests. They are conducted by applying large quantities of water to a basin with a specific surface area to allow the calculation of an infiltration rate in units of minutes per inch and in gallons per day per square foot. Hydraulic load testing was conducted at the Augusta Parcel and Allen Parcel. The testing at each parcel was conducted over three days. MassDEP has previously indicated that, if hydraulic load testing indicates a high infiltration rate (above 7 gpd/sf), the agency would allow a design loading rate of 7 gpd/sf for open sand beds until performance testing with actual treated effluent from a WWTF proved that a higher rate was warranted. The design loading rate of 7 gpd/sf is an increase over the 5 gpd/sf allowable by MassDEP if a hydraulic load test was not completed.

4.3.1 Test Preparation

The first infiltration test was conducted at the Augusta Parcel from September 25 to 27, 2018. The Town's Water Department provided equipment and manpower to install a 10-foot diameter basin. The basin consisted of a prefabricated 10-foot diameter, 36-inch tall, 3/16-inch thick aluminum "ring" which served as the walls of the basins. The excavation was advanced to a depth below the top layer of organic and silty material (including the layer of unsuitable material at the Allen Parcel).

The basin was supplied with water from a fire hydrant located on the south side of Brick Kiln Road, approximately 200 feet from the test site. A two-inch hose was laid from the hydrant to the basin. The Falmouth Water Department provided a meter and a backflow preventer at the hydrant connection. Flow was controlled using a valve at the backflow preventer. (See Figure 4.4)

Figure 4.4 – Installation of Test Basin at the Augusta Parcel

Once testing was completed at the Augusta Parcel the basin was moved to and installed at the Allen Parcel. Testing was conducted at the Allen Parcel from October 2 to 4, 2018. At this location the basin was supplied with water from a fire hydrant located on Alderberry Lane, approximately 1,000 feet from the basin.

Groundwater monitoring was conducted throughout the testing period by taking a manual reading from the adjacent monitoring well at the beginning and end of each day of testing and by installing an electronic data logger in the well during hydraulic load testing. Water level was measured by the data logger in one-minute intervals. Rainfall during the testing period was measured through a rain gauge installed at the test site.

The testing schedule is outlined in Table 4.2.

Table 4.2 – Testing Schedule

Parcel	Basin Set and Saturation	Constant Head Test	Falling Head Test and Basin Removal
Augusta Parcel	Tuesday 9/25/18	Wednesday 9/26/18	Thursday 9/27/18
Allen Parcel	Tuesday 10/2/18	Wednesday 10/3/18	Thursday 10/4/18

Precipitation data from the week prior to the test through the week after is summarized in Figure 4.5. The precipitation that occurred during the testing period slightly increased the soil moisture, thereby reducing the empty pore spaces in the soil and making the test findings more conservative.

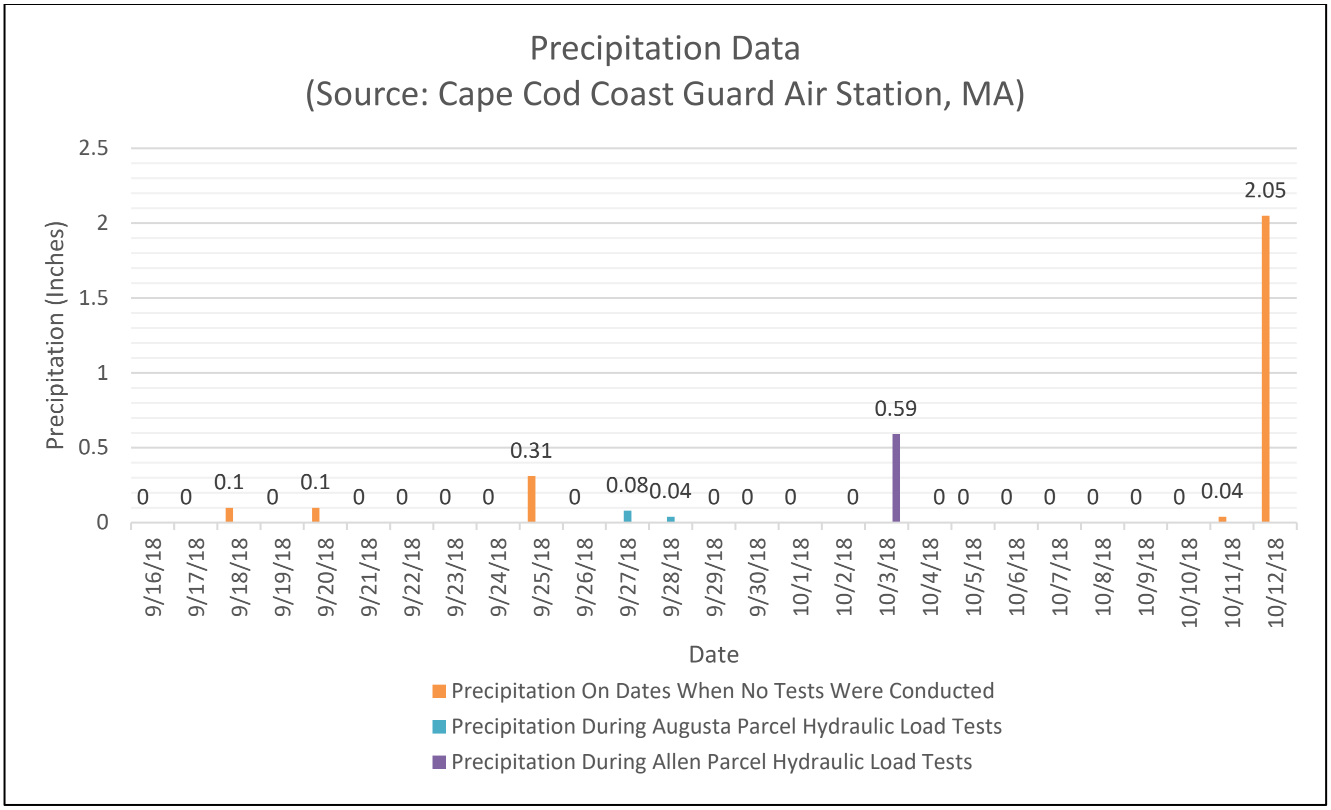


Figure 4.5 – Precipitation Data (Source: Cape Cod Coast Guard Air Station, MA)

The basin testing consisted of three phases—saturation, constant head testing, and falling head testing. Each phase is described below.

4.3.2 Saturation

The soils at the test sites were saturated prior to the testing by supplying water to the basin for a 24-hour period. Flow into the basin was sustained at a rate that would keep a generally constant level of water in the test basin. As saturation progressed, the infiltration rate of the underlying soils declined as the amount of empty pore space for water to enter in the soils was reduced. By the end of the day, it was possible to adjust the flow into the basin to a rate that would maintain the saturated conditions overnight without flooding the basin or allowing it to go dry.

4.3.3 Constant Head Test

The constant head test was started after the basin had been saturated for 24 hours. The constant head test was performed by supplying enough flow to maintain a constant water level in the basin over a six to eight hour test period. The flow rate was measured hourly and the monitoring well water levels were measured on a per minute basis throughout the testing. In the constant head tests, the infiltration capacity was measured in gallons per minute (gpm) as water was added to the 10-foot infiltration basin at the same rate that the water infiltrated out of the basin. Flow rate was measured at the end of the fire hose using containers of known volume and a stopwatch. By dividing the rate of flow by the square foot area of the basin, an infiltration capacity in gallons per day per square foot could be calculated. To be conservative, the infiltration rate for the constant head test was established as the lowest infiltration rate observed during the test. At the end of the day of constant head testing, the basin was again supplied with a constant flow overnight to maintain saturated conditions in the underlying soils.

Figure 4.6 – Constant Head Test

4.3.4 Falling Head Test

The falling head tests were performed after the constant head tests. These tests were performed by filling the basin to a specific level and then stopping the water supply and observing the rate at which the water level in the basin dropped. The falling head tests were repeated as many times as possible during the course of the day. Five tests were performed at the Augusta parcel: one test of 14-inches, and four tests of 10-inches. Three tests were performed at the Allen parcel: one test of 10-inches, one test of 13- inches, and one test of 12- inches. In the falling head tests, infiltration is measured in inches per hour. This result is converted to gallons per day per square foot. In a falling head test infiltration rates are typically highest at the beginning of the test due to the relatively large static pressure of the water and drops as the static pressure decreases with water level. The infiltration rate for each test was established as the average infiltration rate observed during the test.

5. Findings and Conclusions

5.1 Subsurface Conditions

The 2004 USGS Cape Cod Groundwater Contour Map indicates that groundwater at both parcels generally flows towards the south.

The subsurface conditions observed at the Augusta Parcel were generally homogenous, comprised of sand with traces of gravel and finer grained materials. The predominance of the sand suggests suitable conditions for infiltration of treated wastewater.

The subsurface conditions observed at the Allen Parcel were generally homogeneous, comprised of sand with traces of gravel and finer grained materials. A layer of compact silty clay (approximately 10-inches thick) is present at approximately 28-inches to 38-inches below grade. The impermeable layer was encountered in both the deep observation hole and at the hydraulic load test site. Unsuitable material was not found at the test location used in 2010. The 2018 test location is located approximately 250 feet to the north of the 2010 test location.

The subsurface investigations indicate that the silty layer is likely shallow. Further subsurface investigations would be required to define the full area extent of the layer. The predominance of sand below the layer of unsuitable material suggests suitable conditions for infiltration of treated wastewater if the layer of impermeable silty material was removed.

(continued)

Figure 5.1 – Allen Parcel Subsurface Conditions

5.2 Constant Head Test Results

The first set of testing at each basin was constant head testing. As described above, during the constant head tests, water was added to the basins at approximately the same rate that it infiltrated into the ground.

The data collected for the two constant head tests is outlined in Tables 5.1 and 5.2, and summarized in Table 5.3. In accordance with EPA guidelines the allowable infiltration rate was calculated as 10% of the observed infiltration rate. The calculated allowable infiltration rate for the constant head tests was calculated as 16 gpd/sf for the Augusta Parcel and 13 gpd/sf for the Allen Parcel, based on the lowest infiltration rate observed during testing at each site.

Table 5.1 – Augusta Parcel – Constant Head Testing Data[1]

Time	Flow (gpm)	Water Level[2] (Inches)	Average Infiltration Rate (gpd/sf)
8:15 a.m.	13.5	6.6	247
9:00 a.m.	13.9	9.9	254
10:00 a.m.	13.3	12.5	243
11:00 a.m.	13.8	12.3	252
12:38 p.m.	9.8	10.3	179
1:30 p.m.	9.1	9.4	166

Time	Flow (gpm)	Water Level[2] (Inches)	Average Infiltration Rate (gpd/sf)
2:30 p.m.	9.1	8.9	166
3:30 p.m.	**8.9**	**8.3**	**163**
Average	11.4	N/A	209

Notes:
1. Constant heat testing at the Augusta Parcel took place on Wednesday September 26, 2018.
2. Water level in the test basin. Zero inches of water level was set at a point above the highest point of the test basin floor.
3. Lowest infiltration rate observed indicated with **bold text.**

Table 5.2 – Allen Parcel – Constant Head Testing Data[1]

Time	Flow (gpm)	Water Level[2] (Inches)	Average Infiltration Rate (gpd/sf)
8:00 a.m.	8.3	7.3	152
9:00 a.m.	8.3	7.5	152
10:00 a.m.	8.4	7.9	153
11:00 a.m.	8.3	8.1	151
12:00 p.m.	7.4	7.6	136
1:00 p.m.	7.4	7.4	136
2:00 p.m.	7.5	7.3	138
3:00 p.m.	7.4	7.5	136
4:00 p.m.	**7.2**	**7.6**	**133**
Average	7.8	N/A	143

Notes:
1. Constant heat testing at the Allen Parcel took place on Wednesday October 3, 2018.
2. Water level in the test basin. Zero inches of water level was set at a point above the highest point of the test basin floor.
3. Lowest infiltration rate observed indicated with **bold text.**

Table 5.3 – Constant Head Testing Results Summary

Parcel	Flow Rate (gpm)[1]	Flow Rate (gpd)	Basin Area (SF)	Infiltration Capacity (gpd/sf)	Allowable Loading Rate (gpd/sf)[2]
Augusta Parcel	8.9	12,816	79	163	16
Allen Parcel	7.2	10,368	79	133	13

Notes:
1. To be conservative the final stabilized flow rate is equal to the lower flow rate recorded during the constant head test.
2. Per EPA guidance (EPA/625/R-06/016), allowable loading rate is calculated as 7-10% of measured basin infiltration rates. Consistent with previous evaluations conducted in Falmouth, 10% was used for this evaluation.

5.3 Falling Head Test Results

For the falling head tests, the basin was filled to a measured mark. The water supply was then turned off and the rate at which the water level in the basin dropped was recorded. The falling head tests were repeated as many times as possible during the course of the day, with the basin being refilled to the measured mark at the beginning of each test run. The results of the falling head tests are presented in Appendix E and the average infiltration from each test is summarized in Tables 5.4 and 5.5. The average infiltration rate in gpd/sf for the Augusta Parcel was determined to be 138 gpd/sf (five tests completed). The average infiltration rate in gpd/sf for the Allen Parcel was determined to be 130 gpd/sf (three tests completed). Based on these infiltration rates and the EPA criteria of using 10 percent of the loading test rate, a design rate of 14 gpd/sf is considered appropriate for the Augusta Parcel and 13 gpd/sf is appropriate for the Allen Parcel.

Figure 5.2 – Falling Head Test – Allen Parcel

Table 5.4 – Falling Head Testing Results Summary – Augusta Parcel

Test Number	Average Observed Infiltration Rate (gpd/sf)	Average Allowable Loading Rate (gpd/sf)[1]
1	147	15
2	116	12
3	142	14
4	147	15
5[2]	283	28
Average	**138**	**14**

Notes:

1. Per EPA guidance (EPA/625/R-06/016), allowable loading rate is calculated as 7-10% of measured basin infiltration rates. Consistent with previous evaluations conducted in Falmouth, 10% was used for this evaluation.
2. Test number 5 was considered an outlier and was not used in the calculation to establish the average observed infiltration rate and average allowable loading rate.

Table 5.5 – Falling Head Testing Results Summary – Allen Parcel

Test Number	Average Observed Infiltration Rate (gpd/sf)	Average Allowable Loading Rate (gpd/sf)
1	152	15
2	131	13
3	108	11
Average	**130**	**13**

Notes:

1. Per EPA guidance (EPA/625/R-06/016), allowable loading rate is calculated as 7-10% of measured basin infiltration rates. Consistent with previous evaluations conducted in Falmouth, 10% was used for this evaluation.

5.4 Hydraulic Load Test Findings Summary

MassDEP has previously indicated that, if hydraulic load testing indicates a high infiltration rate (above 7 gpd/sf), the agency would consider a design loading rate of 7 gpd/sf for open sand beds until performance testing with actual treated effluent from a WWTF proved that a higher rate was warranted. The design loading rate of 7 gpd/sf is an increase over the 5 gpd/sf rate allowable by MassDEP if a hydraulic load test was not completed.

For the constant head test, an infiltration rate of 163 gpd/sf was calculated for the Augusta Parcel and 133 gpd/sf was calculated for the Allen Parcel. Applying an EPA design factor of 10 percent results in an average design factor of 16 gpd/sf for the Augusta Parcel and 13 gpd/sf for the Allen Parcel.

In the falling head test, an infiltration rate of 138 gpd/sf was calculated for the Augusta Parcel and 130 gpd/sf was calculated for the Allen Parcel. Applying the same EPA design factor of 10 percent results in an average design factor or 14 gpd/sf for the Augusta Parcel and 13 gpd/sf for the Allen Parcel.

Groundwater elevation measurements throughout both tests varied by less than two inches throughout the testing. It is recommended that groundwater modeling be conducted for each site at the proposed effluent flow if design proceeds.

Based on this analysis, a design rate of 7 gpd/sf is planned to be used to design potential open sand beds at both sites. Once a design has been developed for each parcel, additional infiltration/percolation testing will need to be conducted in the location of the proposed effluent disposal beds and witnessed by MassDEP.

6. Summary

Field investigations consisting of groundwater monitoring well installations, soil borings, percolation tests, and hydraulic load tests were conducted at the Augusta and Allen Parcels in September and October 2018. The field investigations indicate a high infiltration rate capacity at both sites (14 gpd/sf at the Augusta parcel and 13 gpd/sf at the Allen Parcel). Past experience, and the field investigation findings, indicate that a design hydraulic loading rate of 7 gpd/sf should be used for the design of open sand beds at both sites. The sites will require additional infiltration/percolation testing witnessed by MassDEP once a design is developed for the sites. Once the sites are constructed it is recommended that performance testing with actual treated effluent from a WWTF be conducted to evaluate the ability to request an increase in the rated capacity of the open sand beds.

Figures

Parcels

Road Ownership
<all other values>
COUNTY; TOWN
PRIVATE
STATE

Water Features
POND
STREAM

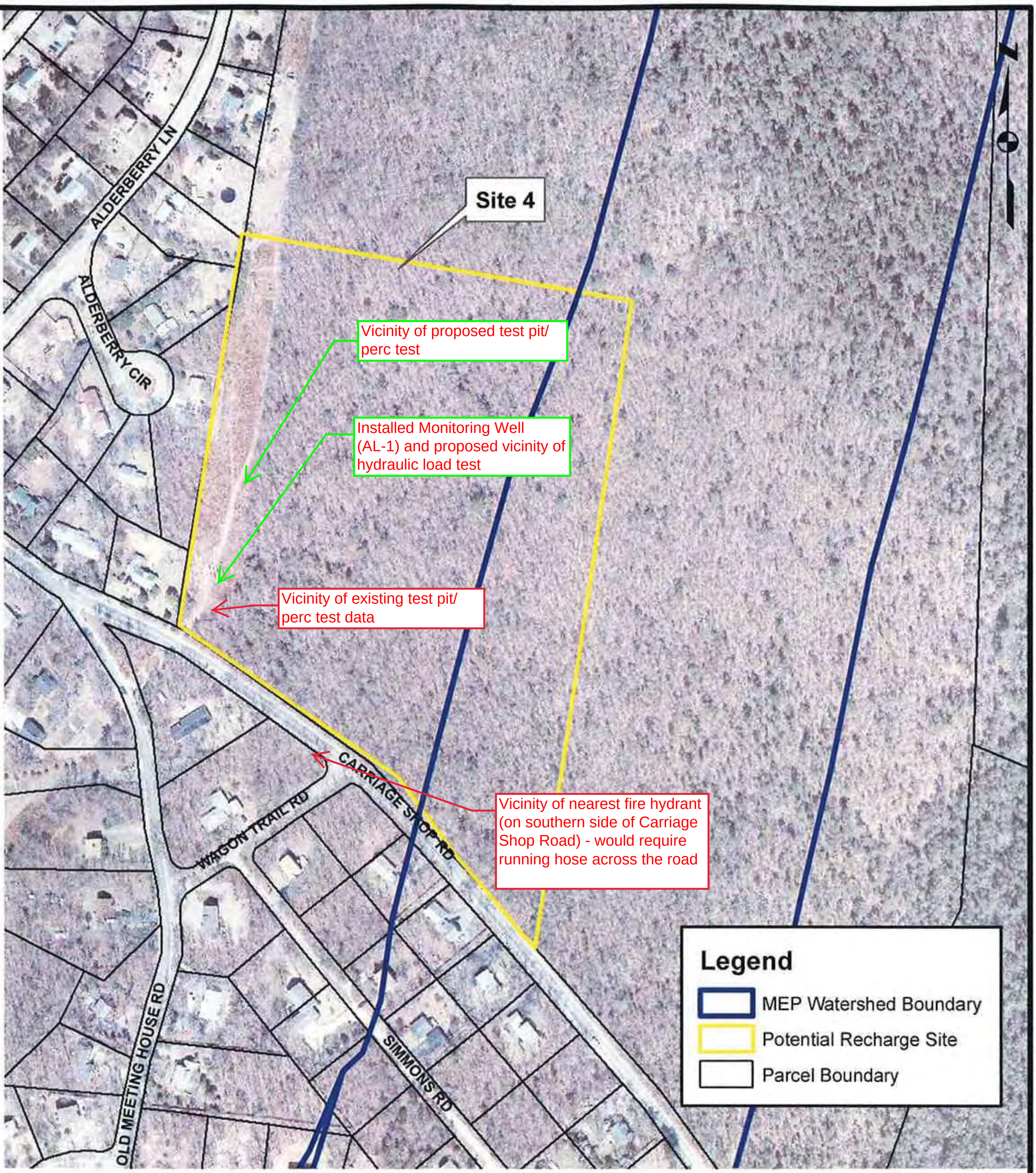
Town Boundry

Disclaimer
The Town of Falmouth makes no claims,no representations and no warranties, express or implied, concerning the validity (express or implied), the reliability or the accuracy of the GIS data and/or GIS products furnished by the Town, including the implied validity of any uses of such data.Parcel lines are graphic representations only.
Planimetric features derived from 3/05 Aerials.
Prepared by Falmouth G.I.S.

1" = 300 ft

Town of Falmouth

August 7, 2018



Data Source: MassGIS/Town of Falmouth GIS Department

1 inch = 250 feet

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STEARNS & WHELER LLC
Environmental Engineers & Scientists
HYANNIS, MASSACHUSETTS
Phone: 508 362-5680
Fax: 508 362-5684

Job No.: 71045

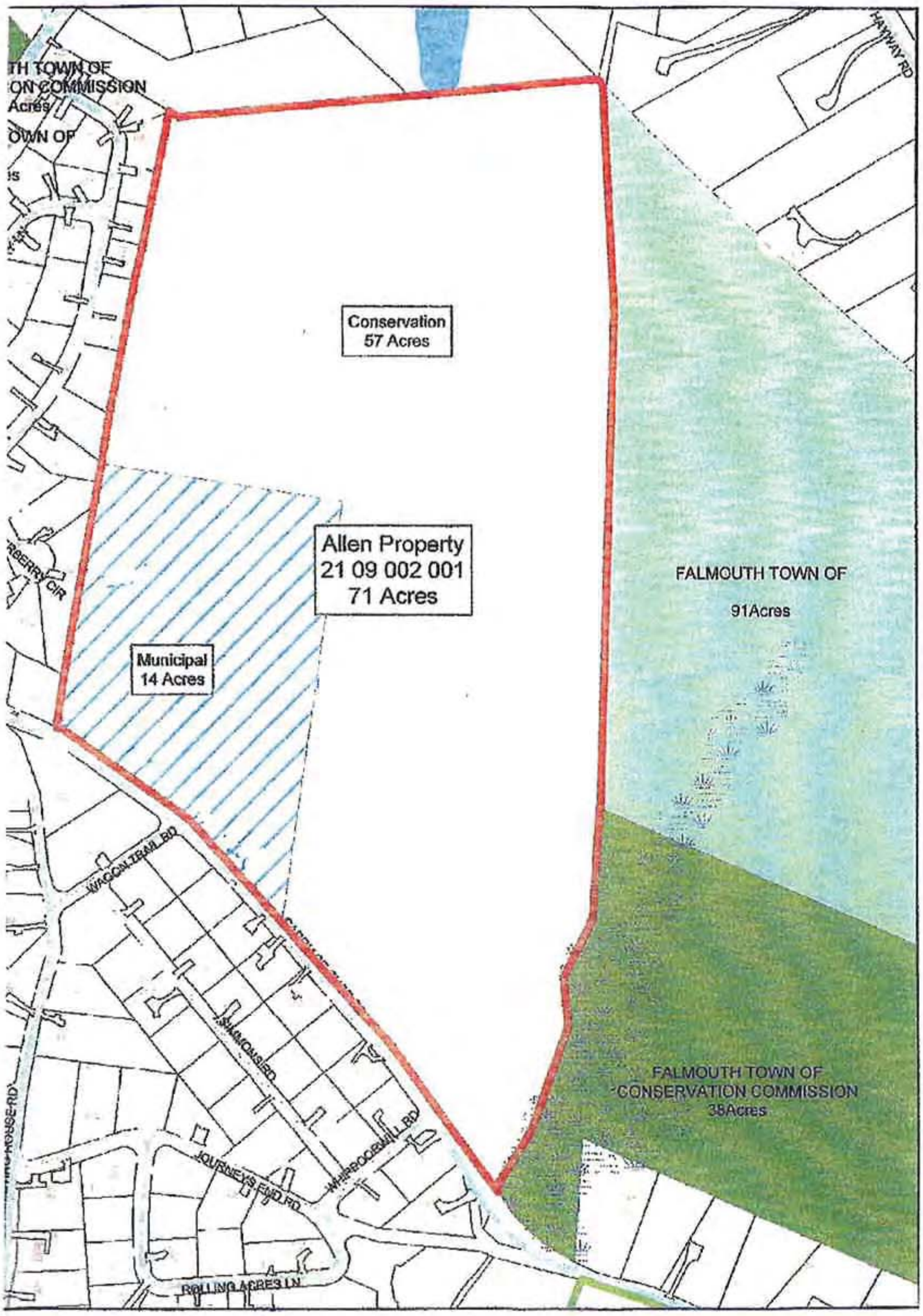
Date: 7/06/2010

Town of Falmouth, Massachusetts
CWMP

Allen Property and Site 4
Aerial View

FIGURE 8

2004 Special April Town Meeting



Land Acquisition - Jurisdiction Selectmen / Conservation
MAPNO 21 09 002 001

Article
5

Data Source: MassGIS/Town of Falmouth
GIS Department

Location: GIS File Location: I:\Jobs\70000\71045 Falmouth\Figures\Technical Memo Figures 6_21_2010\July_6_Figures\71045F05.mxd

STEARNS & WHELER LLC
Environmental Engineers & Scientists
HYANNIS, MASSACHUSETTS

Job No.: 71045
Date: 7/06/2010

Town of Falmouth, Massachusetts
CWMP

Allen Property
From 2004 Special Town Meeting

FIGURE 7

Appendices

Appendix A: 2018 Falmouth Hydraulic Load Test Work Plan

Technical Memorandum

August 22, 2018

To:	Town of Falmouth, MA	Ref. No.:	11153041
From:	Anastasia Rudenko, P.E., BCEE, ENV SP	Tel:	774-470-1637
	J. Jefferson Gregg, P.E., BCEE		774-470-1640
CC:	File; Project Team		

Subject: South Coast Embayments – Preliminary Evaluations and Notice of Project Change Update Project

Hydraulic Load Testing – Work Plan for the Augusta Parcel and Allen Parcel

1. Introduction

The Town of Falmouth (Town) is undertaking a Comprehensive Wastewater Management Planning (CWMP) Project to develop strategies for collecting, treating, and disposing of wastewater. A key element of that planning process is identifying sites in the Town where the treated water can be recharged to the groundwater.

The Hydraulic Load Test Work Plan outlined in this memorandum was developed in accordance with the guidance provided in the "Guidelines for the Design, Construction, Operation and Maintenance of Small Wastewater Treatment Facilities with Land Disposal" prepared by MassDEP and revised in July 2018, and is consistent with the methodology used for hydraulic load testing that was conducted in Falmouth in 2011.

2. Work Plan

2.1 Monitoring Well Construction

One monitoring well will be located in the vicinity of each hydraulic load test site, as detailed below:

- Each well shall be drilled to 10-feet below the water table. Ten feet of screening shall be installed straddling the water table.
- Split spoon sampling will be conducted at 5-foot intervals.
- Wells shall be finished with a riser, locking cover, and bentonite seal.
- Well construction and soil details shall be provided by the well installation contractor.
- Boring logs shall include:
  - o Boring name
  - o Location (with swing ties or other means to locate)

- Blow counts, soil types, and depths
- Estimated depth of groundwater

2.2 Basin Construction

A pre-manufactured, round, 10-foot diameter, 30-inch deep, test ring will be placed in a cleared area within an excavated basin. The basin depth will depend on the soil conditions. The top one to two feet of soil will be removed to remove topsoil, roots, etc. in the area. The ring will be pushed into the subsoil to the point of refusal by a loader bucket to seal the ring into the ground. At least 22 inches of the inside of the ring should remain exposed. Backfill around the outside of the ring to stabilize, as needed.

2.3 Water Conveyance

A nearby fire hydrant will be located and used to supply the water through a flow meter and backflow preventer. Water will be conveyed from the hydrant to the test basin using flexible fire hose. A valve at the end of the hose will regulate the flow to the basin. If no hydrant is available within a reasonable distance of the proposed site, a water truck or rented storage tank could be used.

2.4 Basin Testing

Prior to initiating testing at each site, the soils at each basin will be saturated.

A. Basin Saturation

The basin will be flooded to in order to saturate the soils of the test basin. It is anticipated that the saturation period will last 24 hours. Water will be applied day and night at a low enough rate that the basin will not flood. Basin saturation is intended to simulate a loading condition prior to testing.

B. Infiltration Testing

Two (2) types of hydraulic tests will be completed to provide information regarding the infiltration capacity of the soils as described below:

i. Constant Head Test

In this test, water is continuously added to the basin and allowed to infiltrate. The flow rate into this basin is adjusted so that the input equals the output and the water level in the basin remains constant. The influent will be measured in gallons per minute. Influent flow measurements will be converted and divided by the surface area of the basin to establish a gallon per day per square foot value. This test will be conducted over a four- to six-hour period of time to estimate the infiltration capacity of the soil, starting after the water level and flow rate have stabilized.

ii. Falling Head Test

After the constant head test, water will be applied to the basin during the overnight period at a slow rate to sustain saturation of the soil but not flood the basin. The falling head test will be started by adding water to the basin to a depth of 10 inches (approximately 500 gallons). The water supply will be shut off and the rate of infiltration in minutes per inch will be measured until the basin is nearly drained. The test will be repeated with as many cycles as can be completed in a one-day test period (minimum of three tests will be conducted). Results will be averaged. If more than six tests are completed, the highest and lowest values will be eliminated to remove outliers and the remaining values will be averaged. All data will be presented graphically over time. The rate of recharge in minutes per inch will then be mathematically converted to an infiltration rate in gallons per day per square foot.

iii. During the basin saturation and testing, the groundwater levels in the monitoring well will be monitored on an hourly basis.

2.5 Percolation Tests and Test Pits

Additional site evaluations will be conducted, including a percolation test and soil profile, by a certified soil evaluator in accordance with 310 CMR 15.

2.6 Data Evaluation and Report Preparation

The results of the hydraulic load testing will be evaluated to estimate the infiltration capacity of the site (in gallons per day per square foot). Precipitation data, from the week prior to and after the test, will be obtained from local meteorological data collection stations. A letter report will be prepared describing the testing, presenting the results, and providing an estimated design loading rate for future use in the wastewater planning process. Metrological data will be included along with a summary of the impact of any precipitation.

2.7 Documentation of Modifications

If field conditions warrant modifications to the above outlined procedures, those modifications will be documented in the letter report.

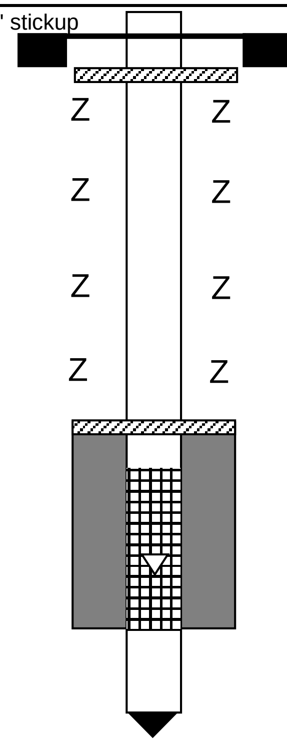
Appendix B: Boring Logs September 2018

Cape Cod Test Boring	Project	Boring No. B1 (Augusta Parcel)
5 Rayber Road, Orleans, MA 02653	GHD, Inc.	
(508) 240-1000	Falmouth Wastewater Treatment Facility	Sheet 1 of 1
div. Desmond Well Drilling, Inc.	Falmouth, MA	

Driller:	Patrick Desmond	Boring location:	N41.57927°; W070.58396°
Helper:	Ryan Whitlatch	Ground Surface Elevation:	
Inspector:	Craig Curtin	Date start: 9/6/2018	Date end: 9/6/2018

Sampler consists of a two inch split spoon driven using a 140 lb. hammer falling thirty inches	Notes:	Auger Size: 6 1/4" x 4" H.S.A Casing Size: 2"x44' SCH40 PVC FJT Screen Size: 2"x10'X.010 SCH40 PVC FJT

Depth (FT)	Sample NO	PEN/REC	DEPTH/FT	BLOWS 6"	Sample Description	Well Installation
2						2' stickup
0						
-2						
-4	1	24/16	3 - 5	3-6-8-9	F-M-C brown sand; trace gravel.	
-6					Dry.	
-8	2	24/18	8 - 10	4-6-5-6	F-M-C brown sand; trace gravel.	
-10					Dry.	
-12						
-14	3	24/17	13 - 15	3-3-4-4	F-M-C brown sand; trace gravel.	
-16					Dry.	
-18	4	24/16	18 - 20	2-3-5-4	F-M-C brown sand; trace gravel.	
-20					Dry.	
-22						
-24	5	24/20	23 - 25	4-4-6-5	F-M brown sand; little C brown	
-26					sand. Dry.	
-28	6	24/18	28 - 30	3-6-8-7	F-M brown sand; trace C brown	
-30					sand. Dry.	
-32						
-34	7	24/16	33 - 35	2-6-6-7	F-M brown sand; trace C brown	
-36					sand. Wet.	
-38	8	24/14	38 - 40	6-9-15-17	F-M brown sand; trace C brown	
-40					sand. Wet.	
-42						
-44						
-46						
-48						
-50						Bentonite seal: 2' to 3'
-52						Bentonite seal: 24' to 25'
-54						Settling tube: 37' to 42'
-56						Well screen: 27' to 37'
-58						Well Depth: 42'
-60						Static: 32'
-62						End of boring: 45'
-64						End of sample: 40'
-66						Measurements from grade



Granular Soils BLOWS/FT	DENSITY	Cohesive Soils BLOWS/FT	DENSITY	Proportions Used	Well Installation Key
0 - 4	V. LOOSE	> 2	V. SOFT	Trace 0 - 10%	- CONCRETE
4 - 10	LOOSE	2 - 4	SOFT	Little 10 - 20%	- SAND PACK
10 - 30	M. DENSE	4 - 8	M. STIFF	Some 20 - 35%	Z - SOIL BACKFILL
30 - 50	DENSE	8 - 15	STIFF	And 35 - 50%	- BENTONITE
> 50	V. DENSE	15 - 30	V. STIFF		- SCREEN
		> 30	HARD		- APPROX. WATER LEVEL

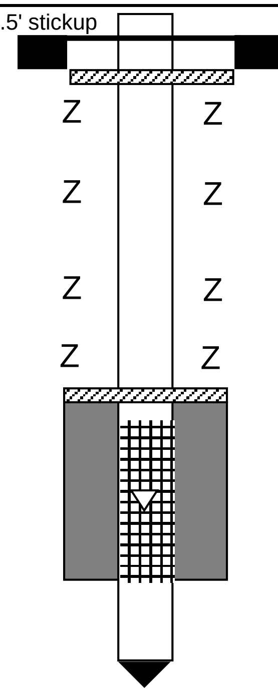
CAPE COD TEST BORING **BORING NO. B1 (Augusta Parcel)**

Cape Cod Test Boring	Project	Boring No. B2 (Augusta Parcel)
5 Rayber Road, Orleans, MA 02653	GHD, Inc.	
(508) 240-1000	Falmouth Wastewater Treatment Facility	Sheet 1 of 1
div. Desmond Well Drilling, Inc.	Falmouth, MA	

Driller:	Patrick Desmond	Boring location:	N41.57570°; W070.58483°
Helper:	Ryan Whitlatch	Ground Surface Elevation:	
Inspector:	Craig Curtin	Date start: 9/6/2018	Date end: 9/6/2018

Sampler consists of a two inch split spoon driven using a 140 lb. hammer falling thirty inches	Notes:	Auger Size: 6 1/4" x 4" H.S.A Casing Size: 2"x45' SCH40 PVC FJT Screen Size: 2"x10'X.010 SCH40 PVC FJT

Depth (FT)	Sample NO	PEN/REC	DEPTH/FT	BLOWS 6"	Sample Description	Well Installation
2						2.5' stickup
0						
-2						
-4	1	24/18	3 - 5	3-11-15-17	F-M-C brown sand; trace gravel.	
-6					Dry.	
-8	2	24/20	8 - 10	2-4-6-8	F-M-C brown sand; trace gravel.	
-10					Dry.	
-12						
-14	3	24/17	13 - 15	3-4-4-5	F-M-C brown sand; trace gravel.	
-16					Dry.	
-18	4	24/17	18 - 20	2-4-6-7	F-M-C brown sand; trace gravel.	
-20					Dry.	
-22						
-24	5	24/17	23 - 25	4-4-5-5	F-M brown sand; trace C brown	
-26					sand. Dry.	
-28	6	24/20	28 - 30	6-9-8-7	F-M-C brown sand; trace gravel.	
-30					Wet.	
-32						
-34	7	24/14	33 - 35	4-7-8-9	F-M-C brown sand. Wet.	
-36						
-38						
-40						
-42						
-44						
-46						
-48						
-50						Bentonite seal: 2' to 3'
-52						Bentonite seal: 22' to 23'
-54						Settling tube: 34' to 39'
-56						Well screen: 24' to 34'
-58						Well Depth: 39'
-60						Static: 29'
-62						End of boring: 40'
-64						End of sample: 35'
-66						Measurements from grade



Granular Soils BLOWS/FT	DENSITY	Cohesive Soils BLOWS/FT	DENSITY	Proportions Used
0 - 4	V. LOOSE	> 2	V. SOFT	Trace 0 - 10%
4 - 10	LOOSE	2 - 4	SOFT	Little 10 - 20%
10 - 30	M. DENSE	4 - 8	M. STIFF	Some 20 - 35%
30 - 50	DENSE	8 - 15	STIFF	And 35 - 50%
> 50	V. DENSE	15 - 30	V. STIFF	
		> 30	HARD	

Well Installation Key
- CONCRETE
- SAND PACK
Z - SOIL BACKFILL
- BENTONITE
- SCREEN
- APPROX. WATER LEVEL

CAPE COD TEST BORING **BORING NO. B2 (Augusta Parcel)**

Cape Cod Test Boring	Project	Boring No. B3 (Allen Parcel)
5 Rayber Road, Orleans, MA 02653	GHD, Inc.	
(508) 240-1000	Falmouth Wastewater Treatment Facility	Sheet 1 of 1
div. Desmond Well Drilling, Inc.	Falmouth, MA	

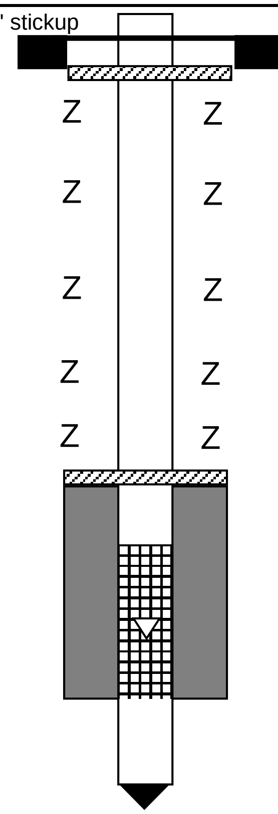
Driller:	Patrick Desmond	Boring location:	N41.60536°; W070.56221°
Helper:	Ryan Whitlatch	Ground Surface Elevation:	
Inspector:	Craig Curtin	Date start: 9/10/2018	Date end: 9/10/2018

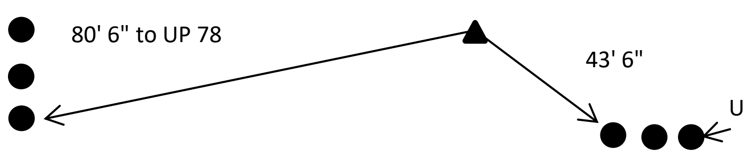
Sampler consists of a two inch split spoon driven using a 140 lb. hammer falling thirty inches	Notes:	Auger Size: 6 1/4" x 4" H.S.A Casing Size: 2"x48.5' SCH40 PVC FJT Screen Size: 2"x10'X.010 SCH40 PVC FJT

Depth (FT)	Sample NO	PEN/REC	DEPTH/FT	BLOWS 6"	Sample Description
2					
0					
-2					
-4	1	24/13	3 - 5	3-6-6-6	F-M-C brown sand; trace gravel.
-6					Dry.
-8	2	24/12	8 - 10	3-8-7-8	F-M-C brown sand; little gravel.
-10					Dry.
-12					
-14	3	24/12	13 - 15	2-3-3-4	F-M-C brown sand; trace gravel.
-16					Dry.
-18	4	24/12	18 - 20	3-5-5-6	F-M-C brown sand; trace gravel.
-20					Dry.
-22					
-24	5	24/13	23 - 25	3-4-5-6	F-M-C brown sand; trace gravel.
-26					Dry.
-28	6	24/14	28 - 30	3-5-5-6	F-M-C brown sand; trace gravel.
-30					Dry.
-32					
-34	7	24/15	33 - 35	4-5-5-7	F-M brown sand; trace C brown
-36					sand; trace gravel. Dry.
-38	8	24/9	38 - 40	3-4-5-9	F-M-C brown sand. Wet.
-40					
-42					
-44	9	24/7	43 - 45	3-3-3-5	F-M-C brown sand. Wet.
-46					
-48					

Well Installation

2' stickup





Bentonite seal: 2' to 3'
Bentonite seal: 27' to 28'
Settling tube: 41.5' to 46.5'
Well screen: 31.5' to 41.5'
Well Depth: 46.5'
Static: 36'
End of boring: 50'
End of sample: 45'
Measurements from grade

Granular Soils BLOWS/FT	DENSITY	Cohesive Soils BLOWS/FT	DENSITY	Proportions Used
0 - 4	V. LOOSE	> 2	V. SOFT	Trace 0 - 10%
4 - 10	LOOSE	2 - 4	SOFT	Little 10 - 20%
10 - 30	M. DENSE	4 - 8	M. STIFF	Some 20 - 35%
30 - 50	DENSE	8 - 15	STIFF	And 35 - 50%
> 50	V. DENSE	15 - 30	V. STIFF	
		> 30	HARD	

Well Installation Key
- CONCRETE
- SAND PACK
Z - SOIL BACKFILL
- BENTONITE
- SCREEN
- APPROX. WATER LEVEL

CAPE COD TEST BORING **BORING NO. B3 (Allen Parcel)**

Appendix C: 2018 Percolation Test Results – Augusta Parcel and Allen Parcel

Commonwealth of Massachusetts
City/Town of Falmouth

Form 11 - Soil Suitability Assessment for On-Site Sewage Disposal

A. Facility Information

Owner Name

Brick Kiln Road (Augusta Parcel)
Street Address

Map/Lot #

Falmouth
City

MA
State

02536
Zip Code

B. Site Information

1. (Check one) ☒ New Construction ☐ Upgrade ☐ Repair

2. Soil Survey Available? ☐ Yes ☐ No If yes:

Source

254A, 665
Soil Map Unit

TP-1 Merrimac, TP-2 Udipsamments
Soil Name

Soil Limitations

Glacial Outwash
Soil Parent material

Landform

3. Surficial Geological Report Available? ☐ Yes ☐ No If yes:

Year Published/Source

Map Unit

Description of Geologic Map Unit:

4. Flood Rate Insurance Map Within a regulatory floodway? ☐ Yes ☐ No

5. Within a velocity zone? ☐ Yes ☐ No

6. Within a Mapped Wetland Area? ☐ Yes ☐ No If yes, MassGIS Wetland Data Layer:

Wetland Type

7. Current Water Resource Conditions (USGS):

Month/Day/ Year

Range: ☐ Above Normal ☐ Normal ☐ Below Normal

8. Other references reviewed:

Commonwealth of Massachusetts
City/Town of Falmouth

Form 11 - Soil Suitability Assessment for On-Site Sewage Disposal

C. On-Site Review *(minimum of two holes required at every proposed primary and reserve disposal area)*

Deep Observation Hole Number: TP-1 (Hole #) 09/26/2018 (Date) 8:30 AM (Time) Clouds/Rain (Weather) ______ (Latitude) ______ (Longitude:)

1. Land Use: Vacant Lot with some vegetation (e.g., woodland, agricultural field, vacant lot, etc.) Pines, shrubs (Vegetation) ______ (Surface Stones (e.g., cobbles, stones, boulders, etc.)) 3-5% (Slope (%))

   Description of Location: ______

2. Soil Parent Material: Glacial outwash ______ (Landform) ______ (Position on Landscape (SU, SH, BS, FS, TS))

3. Distances from: Open Water Body ______ feet Drainage Way ______ feet Wetlands ______ feet

   Property Line ______ feet Drinking Water Well ______ feet Other ______ feet

4. Unsuitable Materials Present: ☐ Yes ☒ No If Yes: ☐ Disturbed Soil ☐ Fill Material ☐ Weathered/Fractured Rock ☐ Bedrock

5. Groundwater Observed: ☐ Yes ☒ No If yes: ______ Depth Weeping from Pit ______ Depth Standing Water in Hole

Soil Log

Depth (in)	Soil Horizon /Layer	Soil Texture (USDA	Soil Matrix: Color-Moist (Munsell)	Redoximorphic Features			Coarse Fragments % by Volume		Soil Structure	Soil Consistence (Moist)	Other
				Depth	Color	Percent	Gravel	Cobbles & Stones			
0-4	Ap	LS	10YR 4/4	-	-	-	-	-	Granular	Friable	
6-20	Bw	CS	10YR 5/6	-	-	-	10	3	SG	LOOSE	
20-40	C1	M/CS	2.5Y 6/4	-	-	-	5	-	SG	LOOSE	
40-132	C2	CS	2.5Y 8/3	-	-	-	5	3	SG	LOOSE	

Additional Notes: ______

Commonwealth of Massachusetts
City/Town of Falmouth

Form 11 - Soil Suitability Assessment for On-Site Sewage Disposal

C. On-Site Review *(minimum of two holes required at every proposed primary and reserve disposal area)*

Deep Observation Hole Number: TP-2 (Hole #) 9/26/18 (Date) 10:00 AM (Time) Clouds/Rain (Weather) ______ (Latitude) ______ (Longitude:)

1. Land Use: Vacant lot with some vegetation (e.g., woodland, agricultural field, vacant lot, etc.) Pines, shrubs (Vegetation) ______ (Surface Stones (e.g., cobbles, stones, boulders, etc.)) 3-5% (Slope (%))

   Description of Location: ______

2. Soil Parent Material: Glacial outwash ______ (Landform) ______ (Position on Landscape (SU, SH, BS, FS, TS))

3. Distances from: Open Water Body ______ feet Drainage Way ______ feet Wetlands ______ feet
   Property Line ______ feet Drinking Water Well ______ feet Other ______ feet

4. Unsuitable Materials Present: ☐ Yes ☒ No If Yes: ☐ Disturbed Soil ☐ Fill Material ☐ Weathered/Fractured Rock ☐ Bedrock

5. Groundwater Observed: ☐ Yes ☒ No If yes: ______ Depth Weeping from Pit ______ Depth Standing Water in Hole

Soil Log

Depth (in)	Soil Horizon /Layer	Soil Texture (USDA)	Soil Matrix: Color-Moist (Munsell)	Redoximorphic Features			Coarse Fragments % by Volume		Soil Structure	Soil Consistence (Moist)	Other
				Depth	Color	Percent	Gravel	Cobbles & Stones			
0-4	Ap	LS	10YR 4/4	-	-	-	-	-	Granular	Friable	
4-16	Bw	MS	10YR 5/6	-	-	-	5	3	SG	LOOSE	
16-32	C1	MS	10YR 6/4	-	-	-	5	-	SG	LOOSE	
32-124	C2	CS	2.5Y 6/4	-	-	-	10	3	SG	LOOSE	

Additional Notes:

Commonwealth of Massachusetts
City/Town of Falmouth

Form 11 - Soil Suitability Assessment for On-Site Sewage Disposal

D. Determination of High Groundwater Elevation

1. Method Used: Obs. Hole #_____ Obs. Hole #_____

	Obs. Hole #_____	Obs. Hole #_____
☐ Depth observed standing water in observation hole	_____ inches	_____ inches
☐ Depth weeping from side of observation hole	_____ inches	_____ inches
☐ Depth to soil redoximorphic features (mottles)	_____ inches	_____ inches
☐ Depth to adjusted seasonal high groundwater (S_h) (USGS methodology)	_____ inches	_____ inches

_______________ Index Well Number _______________ Reading Date

$S_h = S_c - [S_r \times (OW_c - OW_{max})/OW_r]$

Obs. Hole/Well# _____ S_c _____ S_r _____ OW_c _____ OW_{max} _____ OW_r _____ S_h _____

2. Estimated Depth to High Groundwater: _____ inches

E. Depth of Pervious Material

1. Depth of Naturally Occurring Pervious Material

a. Does at least four feet of naturally occurring pervious material exist in all areas observed throughout the area proposed for the soil absorption system?

☒ Yes ☐ No

b. If yes, at what depth was it observed (exclude A and O Horizons)? Upper boundary: _____ inches Lower boundary: _____ inches

c. If no, at what depth was impervious material observed? Upper boundary: _____ inches Lower boundary: _____ inches

Commonwealth of Massachusetts
City/Town of Falmouth

Form 11 - Soil Suitability Assessment for On-Site Sewage Disposal

F. Certification

I certify that I am currently approved by the Department of Environmental Protection pursuant to 310 CMR 15.017 to conduct soil evaluations and that the above analysis has been performed by me consistent with the required training, expertise and experience described in 310 CMR 15.017. I further certify that the results of my soil evaluation, as indicated in the attached Soil Evaluation Form, are accurate and in accordance with 310 CMR 15.100 through 15.107.

Todd MacDonald Todd MacDonald — 09/27/18
Signature of Soil Evaluator — Date

SE14157 — 12/1/2020
Typed or Printed Name of Soil Evaluator / License # — Expiration Date of License

Name of Approving Authority Witness — Approving Authority

Note: In accordance with 310 CMR 15.018(2) this form must be submitted to the approving authority within 60 days of the date of field testing, and to the designer and the property owner with Percolation Test Form 12.

Field Diagrams: Use this area for field diagrams:

Commonwealth of Massachusetts
City/Town of Falmouth
Percolation Test
Form 12

Percolation test results must be submitted with the Soil Suitability Assessment for On-site Sewage Disposal. DEP has provided this form for use by local Boards of Health. Other forms may be used, but the information must be substantially the same as that provided here. Before using this form, check with the local Board of Health to determine the form they use.

Important: When filling out forms on the computer, use only the tab key to move your cursor - do not use the return key.

A. Site Information

Owner Name:

Street Address or Lot #: Brick Kiln Road

City/Town: Falmouth

State: MA

Zip Code: 02536

Contact Person (if different from Owner):

Telephone Number:

B. Test Results

	Date: 09/26/18	Time: 9:25 AM	Date: 09/26/18	Time: 10:20 AM
Observation Hole #	TP-1		TP-2	
Depth of Perc	43-61		46-64	
Start Pre-Soak	9:25		10:20	
End Pre-Soak	9:31:45		10:30:05	
Time at 12”	-		-	
Time at 9”	-		-	
Time at 6”	-		-	
Time (9”-6”)	-		-	
Rate (Min./Inch)	<2 MIN/INCH		<2 MIN/INCH	
	Test Passed: ☒		Test Passed: ☒	
	Test Failed: ☐		Test Failed: ☐	

Test Performed By: Todd MacDonald, BSC Group

Board of Health Witness:

Comments:

Test results for TP-1 25 gallons passed in 6 min 45 sec, TP-2 25 gallons passed in 10 min 5 sec

Appendix D: 2010 Percolation Test Results – Allen Parcel

Commonwealth of Massachusetts
City/Town of
Percolation Test
Form 12

Percolation test results must be submitted with the Soil Suitability Assessment for On-site Sewage Disposal. DEP has provided this form for use by local Boards of Health. Other forms may be used, but the information must be substantially the same as that provided here. Before using this form, check with the local Board of Health to determine the form they use.

Important: When filling out forms on the computer, use only the tab key to move your cursor - do not use the return key.

A. Site Information

Town of Falmouth
Owner Name

off Carriage Shop Road (Allen Property Site 4)
Street Address or Lot #

East Falmouth	Massachusetts	02536
City/Town	State	Zip Code
Nate Weeks, P.E.	(508) 362-5680	
Contact Person (if different from Owner)	Telephone Number	

B. Test Results

	Date	Time	Date	Time
	June 14, 2010	10:30 AM		
Observation Hole #	TP-3			
Depth of Perc	56"			
Start Pre-Soak	10:55			
End Pre-Soak	11:10			
Time at 12"				
Time at 9"				
Time at 6"				
Time (9"-6")				
Rate (Min./Inch)	< 2 mins./inch			
	Test Passed: ☒ Test Failed: ☐		Test Passed: ☐ Test Failed: ☐	

Brian G. Yergatian, P.E.
Test Performed By:

Witnessed By:

Comments:

TP-3: greater than 24 gallons drained during Pre-Soak

Commonwealth of Massachusetts
City/Town of

Form 11 - Soil Suitability Assessment for On-Site Sewage Disposal

A. Facility Information

Town of Falmouth
Owner Name

off Carriage Shop Road
Street Address

Map/Lot #

East Falmouth | Massachusetts | 02536
City | State | Zip Code

B. Site Information

1. (Check one) ☒ New Construction ☐ Upgrade ☐ Repair

2. Published Soil Survey Available? ☒ Yes ☐ No — If yes: Web Soil Survey accessed 6/14/2010 (Year Published) ___ (Publication Scale) 265A (Soil Map Unit)

   Enfield silt loam, 0 to 3% slopes
   Soil Name ___ Soil Limitations

3. Surficial Geological Report Available? ☐ Yes ☐ No — If yes: ___ Year Published ___ Publication Scale ___ Map Unit

   Geologic Material ___ Landform

4. Flood Rate Insurance Map

   Above the 500-year flood boundary? ☒ Yes ☐ No — Within the 100-year flood boundary? ☐ Yes ☒ No

   Within the 500-year flood boundary? ☐ Yes ☒ No — Within a velocity zone? ☐ Yes ☒ No

5. Wetland Area: National Wetland Inventory Map ___ Map Unit ___ Name

   Wetlands Conservancy Program Map ___ Map Unit ___ Name

6. Current Water Resource Conditions (USGS): ___ Month/Year — Range: ☐ Above Normal ☐ Normal ☐ Below Normal

7. Other references reviewed: ___

Commonwealth of Massachusetts
City/Town of

Form 11 - Soil Suitability Assessment for On-Site Sewage Disposal

C. On-Site Review *(minimum of two holes required at every proposed primary and reserved disposal area)*

Deep Observation Hole Number: TP-3 | June 14, 2010 (Date) | 10:30 AM (Time) | 60 degrees, overcast (Weather)

1. Location

   Ground Elevation at Surface of Hole: ______ Location (identify on plan): ______

2. Land Use: power line easement (e.g., woodland, agricultural field, vacant lot, etc.) | none (Surface Stones) | 5 to 10% (Slope (%))

   grass, brush (Vegetation) | outwash plain (Landform) | valley (Position on Landscape (attach sheet))

3. Distances from: Open Water Body ______ feet | Drainage Way ______ feet | Possible Wet Area ______ feet

   Property Line ______ feet | Drinking Water Well ______ feet | Other ______ feet

4. Parent Material: Glacial outwash | Unsuitable Materials Present: ☐ Yes ☒ No

   If Yes: ☐ Disturbed Soil ☐ Fill Material ☐ Impervious Layer(s) ☐ Weathered/Fractured Rock ☐ Bedrock

5. Groundwater Observed: ☐ Yes ☒ No | If yes: ______ Depth Weeping from Pit ______ Depth Standing Water in Hole

   Estimated Depth to High Groundwater: ______ inches ______ elevation

C. On-Site Review (continued)

Deep Observation Hole Number: TP-3

Depth (in.)	Soil Horizon/ Layer	Soil Matrix: Color-Moist (Munsell)	Redoximorphic Features (mottles)			Soil Texture (USDA)	Coarse Fragments % by Volume		Soil Structure	Soil Consistence (Moist)	Other
			Depth	Color	Percent		Gravel	Cobbles & Stones			
0 - 6	A	10YR 2/1				SL			granular		
6 - 32	Bw	10YR 5/6				SL			blocky		
32 - 94	C	2.5Y 6/4				C. Sand	25%		single grain		

Additional Notes:

No groundwater or redoximorphic features observed.

Commonwealth of Massachusetts
City/Town of

Form 11 - Soil Suitability Assessment for On-Site Sewage Disposal

D. Determination of High Groundwater Elevation

1. Method Used:

☐ Depth observed standing water in observation hole	A. ______ inches	B. ______ inches
☐ Depth weeping from side of observation hole	A. ______ inches	B. ______ inches
☐ Depth to soil redoximorphic features (mottles)	A. ______ inches	B. ______ inches
☐ Groundwater adjustment (USGS methodology)	A. ______ inches	B. ______ inches

2. ______ Index Well Number ______ Reading Date ______ Index Well Level

______ Adjustment Factor ______ Adjusted Groundwater Level

E. Depth of Pervious Material

1. Depth of Naturally Occurring Pervious Material

a. Does at least four feet of naturally occurring pervious material exist in all areas observed throughout the area proposed for the soil absorption system?

☒ Yes ☐ No

b. If yes, at what depth was it observed? Upper boundary: ______ inches Lower boundary: ______ inches

Commonwealth of Massachusetts
City/Town of

Form 11 - Soil Suitability Assessment for On-Site Sewage Disposal

F. Certification

I certify that I am currently approved by the Department of Environmental Protection pursuant to 310 CMR 15.017 to conduct soil evaluations and that the above analysis has been performed by me consistent with the required training, expertise and experience described in 310 CMR 15.017. I further certify that the results of my soil evaluation, as indicated in the attached Soil Evaluation Form, are accurate and in accordance with 310 CMR 15.100 through 15.107.

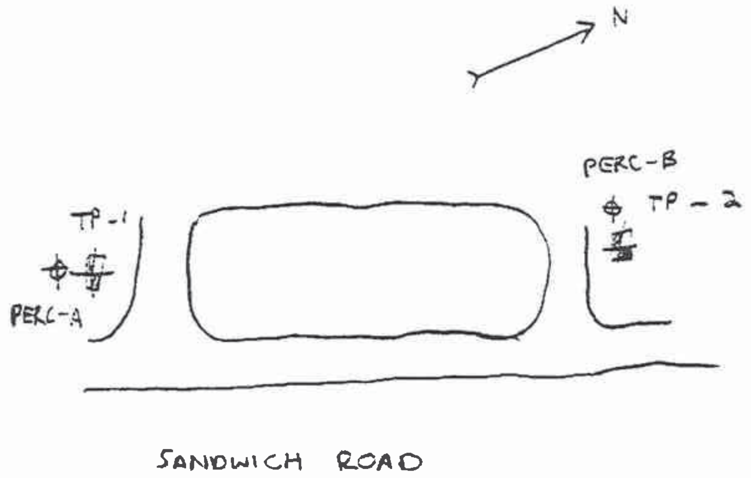
	June 14, 2010
Signature of Soil Evaluator	Date
Brian G. Yergatian, P.E.	October 2005
Typed or Printed Name of Soil Evaluator / License #	Date of Soil Evaluator Exam
Name of Board of Health Witness	Board of Health

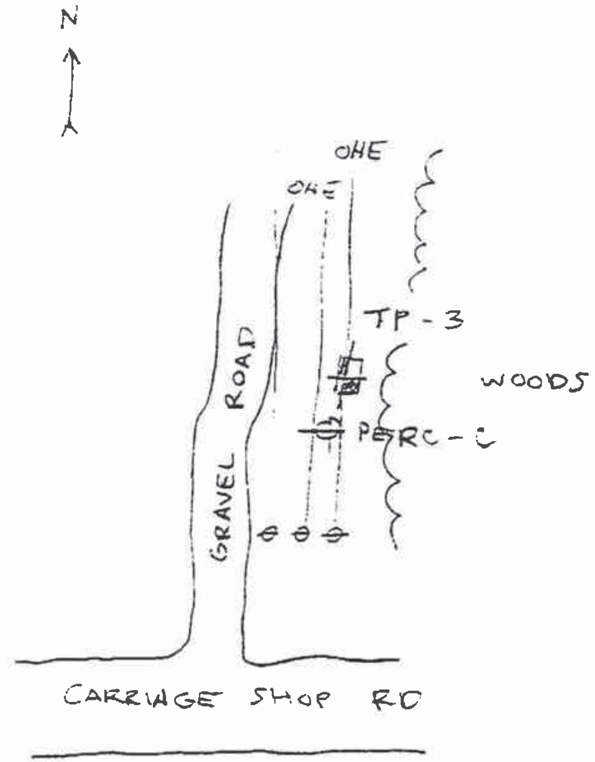
Note: In accordance with 310 CMR 15.018(2) this form must be submitted to the approving authority within 60 days of the date of field testing, and to the designer and the property owner with Percolation Test Form 12.

Commonwealth of Massachusetts
City/Town of
Form 11 - Soil Suitability Assessment for On-Site Sewage Disposal

Field Diagrams

Use this sheet for field diagrams:





Commonwealth of Massachusetts
City/Town of Falmouth

Form 11 - Soil Suitability Assessment for On-Site Sewage Disposal

A. Facility Information

Owner Name

Carriage Shop Road (Allen Property)
Street Address

Map/Lot #

Falmouth
City

MA
State

02536
Zip Code

B. Site Information

1. (Check one) ☒ New Construction ☐ Upgrade ☐ Repair

2. Soil Survey Available? ☐ Yes ☐ No If yes:

   Source

   265A
   Soil Map Unit

   TP-3 Enfield
   Soil Name

   Soil Limitations

   Glacial Outwash
   Soil Parent material

   Landform

3. Surficial Geological Report Available? ☐ Yes ☐ No If yes:

   Year Published/Source

   Map Unit

   Description of Geologic Map Unit:

4. Flood Rate Insurance Map Within a regulatory floodway? ☐ Yes ☐ No

5. Within a velocity zone? ☐ Yes ☐ No

6. Within a Mapped Wetland Area? ☐ Yes ☐ No If yes, MassGIS Wetland Data Layer:

   Wetland Type

7. Current Water Resource Conditions (USGS):

   Month/Day/ Year

   Range: ☐ Above Normal ☐ Normal ☐ Below Normal

8. Other references reviewed:

Commonwealth of Massachusetts
City/Town of Falmouth

Form 11 - Soil Suitability Assessment for On-Site Sewage Disposal

C. On-Site Review *(minimum of two holes required at every proposed primary and reserve disposal area)*

Deep Observation Hole Number: TP-3 (Hole #) | 09/26/2018 (Date) | 12:00 PM (Time) | Clouds/Rain (Weather) | ______ (Latitude) | ______ (Longitude:)

1. Land Use: Power Line right of way with vegetation (e.g., woodland, agricultural field, vacant lot, etc.) | Pines, shrubs (Vegetation) | ______ Surface Stones (e.g., cobbles, stones, boulders, etc.) | 3-5% Slope (%)

   Description of Location: ______

2. Soil Parent Material: Glacial outwash | ______ Landform | ______ Position on Landscape (SU, SH, BS, FS, TS)

3. Distances from: Open Water Body ______ feet | Drainage Way ______ feet | Wetlands ______ feet
   Property Line ______ feet | Drinking Water Well ______ feet | Other ______ feet

4. Unsuitable Materials Present: ☐ Yes ☒ No | If Yes: ☐ Disturbed Soil ☐ Fill Material ☐ Weathered/Fractured Rock ☐ Bedrock

5. Groundwater Observed: ☐ Yes ☒ No | If yes: ______ Depth Weeping from Pit ______ Depth Standing Water in Hole

Soil Log

Depth (in)	Soil Horizon /Layer	Soil Texture (USDA	Soil Matrix: Color-Moist (Munsell)	Redoximorphic Features			Coarse Fragments % by Volume		Soil Structure	Soil Consistence (Moist)	Other
				Depth	Color	Percent	Gravel	Cobbles & Stones			
0-8	Ap	LS	10YR 2/1	-	-	-	-	-	Granular	Friable	
8-12	B1	LS	10YR 4/6	-	-	-	3	3	MASSIVE	FIRM	
12-28	B2	LS	10YR 5/6	-	-	-	3	3	MASSIVE	FIRM	
28-38	C1	SiL	2.5Y 6/8	-	-	-	3	3	MASSIVE	V. FIRM	
38-120	C2	MS	2.5Y 6/6	-	-	-	5	3	SG	LOOSE	

Additional Notes:

Commonwealth of Massachusetts
City/Town of Falmouth

Form 11 - Soil Suitability Assessment for On-Site Sewage Disposal

C. On-Site Review *(minimum of two holes required at every proposed primary and reserve disposal area)*

Deep Observation Hole Number: ______ Hole # ______ Date ______ Time ______ Weather ______ Latitude ______ Longitude:

1. Land Use: ______ (e.g., woodland, agricultural field, vacant lot, etc.) ______ Vegetation ______ Surface Stones (e.g., cobbles, stones, boulders, etc.) ______ Slope (%)

   Description of Location: ______

2. Soil Parent Material: ______ ______ Landform ______ Position on Landscape (SU, SH, BS, FS, TS)

3. Distances from: Open Water Body ______ feet Drainage Way ______ feet Wetlands ______ feet

   Property Line ______ feet Drinking Water Well ______ feet Other ______ feet

4. Unsuitable Materials Present: ☐ Yes ☐ No If Yes: ☐ Disturbed Soil ☐ Fill Material ☐ Weathered/Fractured Rock ☐ Bedrock

5. Groundwater Observed: ☐ Yes ☐ No If yes: ______ Depth Weeping from Pit ______ Depth Standing Water in Hole

Soil Log

Depth (in)	Soil Horizon /Layer	Soil Texture (USDA)	Soil Matrix: Color-Moist (Munsell)	Redoximorphic Features			Coarse Fragments % by Volume		Soil Structure	Soil Consistence (Moist)	Other
				Depth	Color	Percent	Gravel	Cobbles & Stones			

Additional Notes:

______

D. Determination of High Groundwater Elevation

1. Method Used: Obs. Hole #_____ Obs. Hole #_____

	Obs. Hole #_____	Obs. Hole #_____
☐ Depth observed standing water in observation hole	_____ inches	_____ inches
☐ Depth weeping from side of observation hole	_____ inches	_____ inches
☐ Depth to soil redoximorphic features (mottles)	_____ inches	_____ inches
☐ Depth to adjusted seasonal high groundwater (S_h) (USGS methodology)	_____ inches	_____ inches

Index Well Number ______________________ Reading Date ______________________

$S_h = S_c – [S_r \times (OW_c – OW_{max})/OW_r]$

Obs. Hole/Well# _____ S_c _____ S_r _____ OW_c _____ OW_{max} _____ OW_r _____ S_h _____

2. Estimated Depth to High Groundwater: _____ inches

E. Depth of Pervious Material

1. Depth of Naturally Occurring Pervious Material

a. Does at least four feet of naturally occurring pervious material exist in all areas observed throughout the area proposed for the soil absorption system?

☒ Yes ☐ No

b. If yes, at what depth was it observed (exclude A and O Horizons)? Upper boundary: _____ inches Lower boundary: _____ inches

c. If no, at what depth was impervious material observed? Upper boundary: _____ inches Lower boundary: _____ inches

Form 11 - Soil Suitability Assessment for On-Site Sewage Disposal

F. Certification

I certify that I am currently approved by the Department of Environmental Protection pursuant to 310 CMR 15.017 to conduct soil evaluations and that the above analysis has been performed by me consistent with the required training, expertise and experience described in 310 CMR 15.017. I further certify that the results of my soil evaluation, as indicated in the attached Soil Evaluation Form, are accurate and in accordance with 310 CMR 15.100 through 15.107.

Todd MacDonald Todd MacDonald
Signature of Soil Evaluator

09/27/18
Date

SE14157
Typed or Printed Name of Soil Evaluator / License #

12/1/2020
Expiration Date of License

Name of Approving Authority Witness

Approving Authority

Note: In accordance with 310 CMR 15.018(2) this form must be submitted to the approving authority within 60 days of the date of field testing, and to the designer and the property owner with Percolation Test Form 12.

Field Diagrams: Use this area for field diagrams:

Commonwealth of Massachusetts
City/Town of Falmouth
Percolation Test
Form 12

Percolation test results must be submitted with the Soil Suitability Assessment for On-site Sewage Disposal. DEP has provided this form for use by local Boards of Health. Other forms may be used, but the information must be substantially the same as that provided here. Before using this form, check with the local Board of Health to determine the form they use.

Important: When filling out forms on the computer, use only the tab key to move your cursor - do not use the return key.

A. Site Information

Owner Name

Carriage Shop Road (Allen Propoerty)

Street Address or Lot #

Falmouth	MA	02536
City/Town	State	Zip Code

Contact Person (if different from Owner)

Telephone Number

B. Test Results

	09/26/18	12:15 PM	09/26/18	12:35 PM
	Date	Time	Date	Time
Observation Hole #	TP-3 (C-1 Layer)		TP-3 (C-2 Layer)	
Depth of Perc	12-30		26-44	
Start Pre-Soak	12:15		12:35	
End Pre-Soak	12:30		12:43:25	
Time at 12”	-		-	
Time at 9”	-		-	
Time at 6”	-		-	
Time (9”-6”)	-		-	
Rate (Min./Inch)	N/A		<2 MIN/INCH	
	Test Passed: ☐		Test Passed: ☒	
	Test Failed: ☒		Test Failed: ☐	

Todd MacDonald, BSC Group

Test Performed By:

Board of Health Witness

Comments:

Test results for Test 1 (C-1 layer) abandoned as <1" passed during 15 min pre soak. Test 2 (C-2layer) 25 gallons passed in 6 min 45 sec, TP-2 25 gallons passed in 10 min 5 sec.

Appendix E: 2018 Falling Head Test Results

Appendix E Falling Head Testing Results Hydraulic Loading Tests Town of Falmouth

Table 1 Augusta Parcel – Test No. 1

Inches	Time (Minutes, Cumulative)	Minutes/Inch	Inches/Hour	Average Infiltration Rate (gpd/sf)
14	0	N/A	N/A	N/A
13	5	5.0	12.0	180
12	10	5.0	12.0	180
11	15	5.0	12.0	180
10	20	5.0	12.0	180
9	26	6.0	10.0	150
8	32	6.0	10.0	150
7	40	8.0	7.5	112
6	47	7.0	8.6	128
5	56	9.0	6.7	100
4	64	8.0	7.5	112
Average	**N/A**	**6.4**	**9.8**	**147**

1. Falling head testing at the Augusta Parcel took place on Thursday September 27, 2018.

Table 2 Augusta Parcel – Test No. 2

Inches	Time (Minutes, Cumulative)	Minutes/Inch	Inches/Hour	Average Infiltration Rate (gpd/sf)
10	0	N/A	N/A	N/A
9	6.00	6.0	10.0	150
8	12.12	6.1	9.8	147
7	18.75	6.6	9.0	135
6	26.00	7.3	8.3	124
5	33.62	7.6	7.9	118
4	41.75	8.1	7.4	110
3	50.83	9.1	6.6	99
2	61.20	10.4	5.8	87
1	72.60	11.4	5.3	79
Average	**N/A**	**8.1**	**7.8**	**116**

1. Falling head testing at the Augusta Parcel took place on Thursday September 27, 2018.

Table 3 Augusta Parcel – Test No. 3

Inches	Time (Minutes, Cumulative)	Minutes/Inch	Inches/Hour	Average Infiltration Rate (gpd/sf)
10	0	N/A	N/A	N/A
9	6.00	6.0	10.0	150
8	11.00	5.0	12.0	180
7	16.75	5.8	10.4	156
6	22.20	5.5	11.0	165
5	28.40	6.2	9.7	145
4	34.80	6.4	9.4	140
3	42.57	7.8	7.7	116
2	50.47	7.9	7.6	114
1	58.30	7.8	7.7	115
Average	**N/A**	**6.5**	**9.5**	**142**

1. Falling head testing at the Augusta Parcel took place on Thursday September 27, 2018.

Table 4 Augusta Parcel – Test No. 4

Inches	Time (Minutes, Cumulative)	Minutes/Inch	Inches/Hour	Average Infiltration Rate (gpd/sf)
10	0	N/A	N/A	N/A
9	4.58	4.6	13.1	196
8	9.61	5.0	11.9	178
7	14.98	5.4	11.2	167
6	20.91	5.9	10.1	151
5	27.04	6.1	9.8	146
4	33.41	6.4	9.4	141
3	40.61	7.2	8.3	125
2	48.58	8.0	7.5	113
1	57.33	8.8	6.9	103
Average	**N/A**	**6.4**	**9.8**	**147**

1. Falling head testing at the Augusta Parcel took place on Thursday September 27, 2018.

Table 5 Augusta Parcel – Test No. 5

Inches	Time (Minutes, Cumulative)	Minutes/Inch	Inches/Hour	Average Infiltration Rate (gpd/sf)
10	0	N/A	N/A	N/A
9	4.58	2.3	26.1	390
8	9.61	2.5	24.0	359
7	14.98	2.1	28.8	431
6	20.91	3.2	18.6	278
5	27.04	3.7	16.1	241
4	33.41	3.7	16.2	243
3	40.61	4.2	14.2	212
2	48.58	4.4	13.5	203
1	57.33	4.6	13.0	194
Average	**N/A**	**3.4**	**18.9**	**283**

1. Falling head testing at the Augusta Parcel took place on Thursday September 27, 2018.

Table 6 Allen Parcel – Test No. 1

Inches	Time (Minutes, Cumulative)	Minutes/Inch	Inches/Hour	Average Infiltration Rate (gpd/sf)
10	0	N/A	N/A	N/A
9	6.07	6.1	9.9	148
8	12.32	6.3	9.6	144
7	18.00	5.7	10.6	158
6	26.02	8.0	7.5	112
5	32.42	6.4	9.4	140
4	36.62	4.2	14.3	214
3	41.02	4.4	13.6	204
2	49.17	8.2	7.4	110
1	55.60	6.4	9.3	140
Average	**N/A**	**6.2**	**10.2**	**152**

1. Falling head testing at the Allen Parcel took place on Thursday October 4, 2018.

Table 7 Allen Parcel – Test No. 2

Inches	Time (Minutes, Cumulative)	Minutes/Inch	Inches/Hour	Average Infiltration Rate (gpd/sf)
13	0	N/A	N/A	N/A
12	5.00	5.0	12.0	180
11	10.52	5.5	10.9	163
10	16.82	6.3	9.5	142
9	22.70	5.9	10.2	153
8	28.98	6.3	9.6	143
7	35.06	6.1	9.9	148
6	42.31	7.3	8.3	124
5	49.56	7.3	8.3	124
4	58.19	8.6	7.0	104
3	67.62	9.4	6.4	95
2	76.87	9.3	6.5	97
1	85.95	9.1	6.6	99
Average	**N/A**	**7.2**	**8.7**	**131**

1. Falling head testing at the Allen Parcel took place on Thursday October 4, 2018.

Table 8 Allen Parcel – Test No. 3

Inches	Time (Minutes, Cumulative)	Minutes/Inch	Inches/Hour	Average Infiltration Rate (gpd/sf)
12	0	N/A	N/A	N/A
11	5.82	5.8	10.3	154
10	12.82	7.0	8.6	128
9	20.67	7.9	7.6	114
8	28.34	7.7	7.8	117
7	36.49	8.2	7.4	110
6	44.86	8.4	7.2	107
5	54.53	9.7	6.2	93
4	65.11	10.6	5.7	85
3	76.31	11.2	5.4	80
2	85.93	9.5	6.2	93
1	94.45	8.5	7.0	105
Average	**N/A**	**8.6**	**7.2**	**108**

1. Falling head testing at the Allen Parcel took place on Thursday October 4, 2018.

April 11, 2019

To:	Town of Falmouth, MA	Ref. No.:	11153041
From:	Anastasia Rudenko, P.E., BCEE, ENV SP	Tel:	774-470-1637
	J. Jefferson Gregg, P.E., BCEE		774-470-1640
CC:	File; Project Team		

Subject: **South Coast Embayments – Preliminary Evaluations and Notice of Project Change Update Project**

Teaticket / Acapesket Study Area Technical Memorandum No. 6 Outfall Conceptual Cost Evaluation (TASA TM-6)

1. Purpose of Memo

The purpose of this Technical Memorandum is to summarize the basis of design for conceptual layout, and conceptual level cost estimate development for two potential ocean outfalls—Nobska Point and Buzzards Bay.

1.1 References and Design Guidelines

The following references and design guidelines were used to develop this memorandum:

References:

- ‘Comprehensive Wastewater Management Plan and Final Environmental Impact Report and Targeted Watershed Management Plan – Little Pond, Great Pond, Green Pond, Bournes Pond, Eel Pond, and Waquoit Bay Watersheds and Recommendations for West Falmouth Harbor Watershed’ prepared by GHD, dated September 2013. (“2013 CWMP/FEIR/TWMP”)
- ‘Sewage Disposal in Falmouth Massachusetts – III. Predicted Effects of Inland Disposal and sea Outfall on Groundwater’, Robert Meade and Ralph Vaccaro, published in the Boston Society of Civil Engineers October 1971 (“Meade & Vaccaro, 1971”)
- ‘Sewage Disposal in Falmouth Massachusetts – II. Predicted Effect of the Proposed Outfall’, W. Redwood Wright, Dean F. Bumpus and Ralph Vaccaro, published in the Boston Society of Civil Engineers October 1971 (“Bumpus, Wright & Vaccaro, 1971”)
- ‘Sewage Disposal in Falmouth Massachusetts – I. “Expert” Opinion and Public Policy’, W. Redwood Wright, Dean F. Bumpus and Ralph Vaccaro, published in the Boston Society of Civil Engineers October 1971 (“Bumpus & Vaccaro, 1971”)

Design Guidelines:

- 'ASCE MOP 108: Pipeline Design for Installation by Horizontal Direction Drilling, Second Edition' (ASCE MOP 108)
- 'TR-16 Guides for the Design of Wastewater Treatment Works', 2011 Edition as Revised in 2016, prepared by NEIWCC (TR-16)

2. Background

The long-term option of treated (tertiary) effluent discharge through an ocean outfall is being evaluated for effluent disposal in the Town of Falmouth. Ocean outfalls are a proven technology that have the advantage of bypassing nutrient impacted watersheds, estuaries, and coastal ponds (as opposed to land-based discharge options). Ocean outfalls are currently being utilized by multiple Massachusetts communities, including Boston, Dartmouth, Fall River, and New Bedford.

The Town of Falmouth operated an ocean outfall for the village of Woods Hole from 1946 through 1985 that discharged chlorinated raw sewage into Great Harbor. In the mid-1980's the Falmouth Wastewater Treatment Facility (WWTF) was constructed on Blacksmith Shop Road and discharge from the Woods Hole outfall was discontinued.

Construction of an ocean outfall at Nobska Point in Woods Hole, extending approximately 2,000 feet into Vineyard Sound, was first evaluated as part of wastewater planning efforts in the 1970's. At this time, a treatment facility was proposed in Woods Hole, near Nobska Point. Town Meeting approved this approach but it failed at a subsequent ballot vote. A potential ocean outfall at Nobska Point was re-examined as part of the CWMP process in 2011 and conceptual costs were developed for a force main from the Falmouth WWTF on Blacksmith Shop Road to Nobska Point with an ocean outfall off Nobska Point. This memorandum describes the methodology used to update the 2011 Nobska Point conceptual costs to account for cost inflation and applicable changes in technology and materials.

In 2018, GHD subcontracted with J. Churchill, G. Cowles, and J. Rheuban to develop a hydrodynamic ocean model to simulate the plume dispersion for a potential ocean outfall location on Buzzards Bay. The methodology used to develop the model and findings is outlined in the attached report 'Assessing the Effect of Ocean Effluent Discharge off of West Falmouth, MA' (see Appendix A). As outlined in the report, the results of the year-long effluent tracking simulations indicate a negligible (less than one percent) effect on total nitrogen concentrations in Buzzards Bay and West Falmouth Harbor from a potential ocean outfall 4,380 linear feet from the shoreline with an effluent nitrogen concentration of 3 mg/L. The proposed length, established through modeling, was used in the development of a conceptual basis of design for an ocean outfall into Buzzards Bay off of West Falmouth Harbor.

3. Conceptual Cost Evaluation

3.1 Force Main – Sizing Criteria

The determination of force main size is primarily a function of the flow rate and anticipated head-losses along the force main route. The conceptual layout for the potential Nobska Point Ocean Outfall and the potential

Buzzards Bay Ocean Outfall were sized to convey an annual average flow of 4 mgd. Peaking factors and design flows are outlined in Table 1.

Table 1 Ocean Outfall Flow Basis of Design

Parameter	Peaking Factor	Flow (mgd)
Annual Average Flow	1	4
Peak Day Flow	1.9	7.6
Peak Instantaneous Flow	3	12
Notes: 1. Peaking factors established using the TR-16 Figure 2-1: Ratio of Extreme Flow to Average Daily Flow		

Table 2 outlines the force main basis of design for both scenarios. As indicated in Table 2, a larger force main diameter was selected for the Nobska Point outfall due to the larger head losses anticipated along the longer force main conveyance route for this option. A more detailed analysis of pipe size should be completed during final design.

Table 2 Ocean Outfall Basis of Design

	Length of PVC Force Main (lf)	Length of HDPE Ocean Outfall (lf)	Nominal Pipe Diameter (ft)
WWTF to Nobska Point	38,700 lf (7.3 miles)	2,000 lf (0.4 miles)	3
WWTF to Buzzards Bay	11,600 lf (2.2 miles)	4,380 lf (0.8 miles)	2

3.2 Construction Methods

As part of developing conceptual costs, marine construction methods needed to be reviewed. This section summarizes the two construction methods that were evaluated as part of the conceptual cost development. Field investigations and borings will be required to determine the appropriate method of construction for the potential ocean outfalls.

3.2.1 Open Cut Excavation

An ocean outfall can be installed through an open cut excavation using a mechanical dredger (such as a bucket, clamshell, or backhoe dredger) suitable for detailed excavation. A mechanical dredger is typically mounted on a floating barge or jack-up platform. The dredger picks up sediment from the seabed and loads the excavated (dredged) material onto hopper barges for transport and deposition at the relocation site. Use of multiple hopper barges allow for continuous dredging to be maintained. A mechanical dredger has the ability to dredge a wide range of materials, including those that contain debris or (for large machines) boulders; as well as excavate smaller, defined areas such as a trench. Difficult materials, such as stiff clays and weak, weathered, or fractured rocks, can be dredged by larger dredgers.

As the excavation progresses, the pipe is laid in the trench with the trench excavation leading the pipe by about 500 feet. As each segment of pipe is placed the flanged connections are bolted by divers. Concrete collars are installed on the pipe to sink the pipe and anchor it into the trench.

After the entire pipe is placed in the trench, hopper barges carry backfill ballast to the trench and the appropriately graded material is placed over the pipe by the hopper dredge. Hydrographic surveys are performed during this operation to ensure required coverage is maintained.

3.2.2 Horizontal Directional Drilling (HDD)

Horizontal Directional Drilling (HDD) is a multi-step trenchless method of installing underground pipe, which has the potential to minimize construction disturbance to the ecologically sensitive shoreline environment. The HDD drilling operation is initiated with a pilot hole drilled along the pipe's design route. The progress and alignment of the pilot hole is closely monitored using electronic survey tools. Spoils (excavated materials) from the drilled path are conveyed along the drill pipe to the surface at the rig. Next the hole is reamed, enlarging the pilot excavation to its final required diameter for the carrier pipe installation. In general, the final excavation is approximately 150 percent of the installed carrier pipe's volume. A drilling fluid, typically comprised of a mixture of fresh water and bentonite, is used to stabilize the bore hole and remove soil cuttings. Water needs to be provided, typically through a fire hydrant or water storage facility. Once the hole is stable, the pipe pullback commences. The pipeline pullback does not stop until the carrier pipe is in its final position.

HDD can be set up to pull pipe from either the shore or the water. In a shore based operation, a drill pit is established in the parking lot area. A jack-up (lift) barge, stabilized by an anchor, is mobilized to the offshore location to serve as the platform for the offshore drilling rig.

In order to be conservative with respect to the expectations of HDD it was assumed, for the purposes of this evaluation, that the maximum overall length of an HDD installation is 3,500 feet. Although a longer length of HDD installation may be feasible, longer installations are anticipated to limit the number of contractors that can complete the installation. Utilizing HDD for a portion of the outfall installation would, at a minimum, avoid impacts associated with construction through the beach and surf zone.

The carrier pipe size and material of the pipe determines the minimum radius of curvature that the routing needs to maintain. After the maximum length of HDD installation is reached, the outfall would reach the seabed. The remaining outfall pipe would be constructed using excavation and backfill techniques (open cut excavation). Concrete collars will be placed on each pipe segment of the open cut excavation at designated intervals to weight the pipe.

Subsurface geotechnical investigations along the proposed ocean outfall route are needed to assess the risk of implementing this construction technology.

3.3 Nobska Point – Updated Cost Evaluation

A conceptual level cost estimate for a potential outfall at Nobska Point was developed in 2011 as part of the CWMP planning effort. The following basis of design was used to develop the 2011 conceptual cost:

- Effluent is pumped from the Falmouth WWTF on Blacksmith Shop Road to Nobska Point by an effluent lift station located at the Falmouth WWTF.
- Three-foot diameter Polyvinyl Chloride (PVC) force main from the Falmouth WWTF to Nobska Point (approximately 7.3 miles).

- Three-foot diameter High Density Polyethylene (HDPE) outfall (approximately 2,000 feet from Nobska Point to Vineyard South).
- Open cut installation (outfall buried and armored as needed).
- The outfall would extend directly into the sound from Nobska Point to a tee and the outfall diffusers would be placed at right angles to the currents to maximize mixing.
- Costs were based on 1994 construction unit costs for the Seabrook, NH WWTF.
- The cost estimate included final disinfection at a second effluent lift station at Nobska Point.

The 2011 conceptual cost estimate was updated to account for inflation to 2018 dollars. A second conceptual cost estimate was developed for an HDD installation of the outfall. Subsurface investigations are required to determine if HDD is an appropriate technology for this site or to determine the maximum feasible extent of an HDD installation. The following basis of design was used to develop the cost estimates:

- Effluent is pumped from the Falmouth WWTF on Blacksmith Shop Road to Nobska Point by an effluent lift station located at the Falmouth WWTF. Final disinfection occurs at the Falmouth WWTF (no remote disinfection process).
- Force main to be placed next to the existing wastewater force mains that extend from Woods Hole to the Falmouth WWTF on Blacksmith Shop Road.
- Three-foot nominal diameter PVC force main from the Falmouth WWTF to Nobska Point (approximately 7.3 miles). Transition from force main to ocean outfall occurs in an air release valve vault (no secondary effluent lift station). Three-foot nominal diameter HDPE (High Density Polyethylene) force main extends approximately 2,000 feet from Nobska Point to Vineyard Sound. A more detailed analysis of pipe size should be completed during final design.
- One cubic yard of rock excavation was assumed for every 30 linear feet of pipe laid.
- Force main from Falmouth WWTF to Nobska Point is installed through open cut trench installation.
- To minimize risk from adverse weather conditions, construction occurs between May and September.

Both cost estimates were developed based on construction unit costs for the Rehoboth Beach Outfall Project, which was constructed in 2017. Capital costs for both alternatives, which are presented in Table 3, are the total estimated project costs with allowances for construction costs including: a 30 percent construction contingency; 10 percent engineering design; 15 percent fiscal/legal/permitting/construction administration and Resident Representative (RPR) costs; and 2 percent survey, soil boring, and field investigation costs (for design). Because of the conceptual nature of this evaluation a 30 percent construction contingency is carried as no detailed design has been performed and no soil borings/field investigations have been conducted. During final design a reduced contingency will be carried for variability in the bidding climate, project changes before bidding, potential easements, and change orders due to unforeseen conditions. Project costs are presented in 2018 dollars. Once the construction timeframe is known, project costs should be adjusted to the mid-point of construction.

Table 3 Nobska Point Conceptual Cost Estimate[1,2,4,5,6]

	Nobska Point Ocean Outfall – HDD Installation	Nobska Point Ocean Outfall – Open Cut Trench Installation
Construction Total	**$50.2 M**	**$66.3 M**
Design and Administration Allowance	$13.6 M	$17.9 M
Total Capital Costs (2018$)	**$63.8 M**	**$84.2 M**
Total Capital Costs – Midpoint of Construction	**TBD[3]**	**TBD[3]**

Notes:
1. Limited real estate in Route 28 if proposed dual force mains are installed is anticipated to increase the cost of the FM to Nobska Point.
2. All costs are shown in 2018 dollars (ENR = 10959). Once a construction timeframe is known for the project, costs should be adjusted to the mid-point of construction.
3. TBD = To Be Determined when midpoint of construction is known..
4. Estimated Capital Costs does not include utility relocation or dewatering due to high groundwater.
5. Total Capital Costs includes allowances for construction costs such as: a 30% construction contingency, a 10% engineering design, 15% fiscal/legal/permitting/construction administration and Resident Project Representative (RPR) costs, and 2 percent survey, soil boring and field investigations.
6. Linear distances are estimated based on straight line distances along the conceptual route. No surveys or subsurface investigations have been conducted as part of this project.

3.4 Buzzards Bay Cost Evaluation

Two conceptual level cost estimates were developed for a potential ocean outfall from the Falmouth WWTF extending approximately 4,380 feet into Buzzards Bay. The following basis of design was used to develop the conceptual level cost estimate:

- Effluent is pumped from the Falmouth WWTF to a location off Chapoquoit Road to Buzzards Bay by an effluent lift station located at the Falmouth WWTF. Final disinfection occurs at the Falmouth WWTF (no remote disinfection process).
- The Effluent Force Main is routed on Brick Kiln Road West of Route 28, West Falmouth Highway, and Chapoquoit Road.
- Two-foot nominal diameter PVC force main from the Falmouth WWTF to Chapoquoit Road (approximately 2 miles). Transition from force main to ocean outfall occurs in an air release valve vault (no secondary effluent lift station). Two-foot nominal diameter HDPE force main extends approximately 4,380 feet from Chapoquoit Road to Buzzards Bay. A more detailed analysis of pipe size should be completed during final design.
- One cubic yard of rock excavation was assumed for every 30 linear feet of pipe laid.
- Force main from Falmouth WWTF to Chapoquoit Road is installed through open cut trench excavation.
- The maximum length of HDD installation (for the HDD installation alternative) is 3,500 feet.
- No hazardous materials or other materials that require special handling are encountered.

- To minimize risk from adverse weather conditions, construction occurs between May and September.

Conceptual cost estimates were developed for both: 1) an HDD/open cut excavation installation, and 2) a completely open cut excavation installation. Subsurface investigations are required to determine if HDD is an appropriate technology for this site or to determine the maximum feasible extent of an HDD installation. Both cost estimates were developed based on construction unit costs for the Rehoboth Beach Outfall Project, which was constructed in 2017. Capital costs for both alternatives, which are presented in Table 4, are the total estimated project costs with allowances for construction costs including: a 30 percent construction contingency; 10 percent engineering design; 15 percent fiscal/legal/permitting/construction administration and Resident Representative (RPR) costs and 2 percent survey, soil boring, and field investigations costs (for design). Because of the conceptual nature of this evaluation a 30 percent construction contingency is carried as no detailed design has been performed and no soil borings have been conducted. During final design a reduced contingency will be carried for variability in the bidding climate, project changes before bidding, potential easements, and change orders due to unforeseen conditions. Project costs are presented in 2018 dollars. Once the construction timeframe is known, project costs should be adjusted to the mid-point of construction.

Table 4 Buzzards Bay Conceptual Cost Estimate[1,3,4,5]

	Buzzards Bay Ocean Outfall – HDD Installation	Buzzards Bay Ocean Outfall – Open Cut Trench Installation
Construction Total	**$55.1 M**	**$96.2 M**
Design and Administration Allowance	$14.9 M	$26.0 M
Total Capital Costs (2018$)	**$70.0 M**	**$122.2 M**
Total Capital Costs – Midpoint of Construction	**TBD[2]**	**TBD[2]**

Notes:

1. All costs are shown in 2018 dollars (ENR = 10959). Once a construction timeframe is known for the project, costs should be adjusted to the mid-point of construction.
2. TBD = To Be Determined when midpoint of construction is known..
3. Estimated Capital Costs does not include utility relocation or dewatering due to high groundwater.
4. Total Capital Costs includes allowances for construction costs such as: a 30% construction contingency, a 10% engineering design, 15% fiscal/legal/permitting/construction administration and Resident Project Representative (RPR) costs, and 2 percent survey, soil boring and field investigations.
5. Linear distances are estimated based on straight line distances along the conceptual route. No surveys or subsurface investigations have been conducted as part of this project.

4. Summary and Next Steps

The long-term option of treated (tertiary) effluent discharge through an ocean outfall is being evaluated as an option for effluent disposal in the Town of Falmouth. The basis of design that was used to develop conceptual cost estimates for a potential ocean outfall off of Nobska Point into Vineyard Sound and off of Chapoquoit Road into Buzzards Bay is summarized in Table 5.

Table 5 Ocean Outfall Conceptual Layout - Basis of Design

Route	WWTF to Nobska Point	WWTF to Buzzards Bay
Length of FM on Land	38,700 lf (7.3 miles)	11,600 lf (2.2 miles)
Length of FM Under Ocean	2,000 lf (0.4 miles)	4,380 lf (0.8 miles)
Total	**40,700 lf (7.7 miles)**	**15,980 lf (3.0 miles)**

Conceptual cost estimates for both options are summarized in Table 6.

Table 6 Conceptual Cost Estimate Summary

	Nobska Point Ocean Outfall – HDD Installation[1,6]	Nobska Point Ocean Outfall – Open Cut Trench Installation[1,6]	Buzzards Bay Ocean Outfall – HDD/Ocean Outfall Installation[6]	Buzzards Bay Ocean Outfall – Open Cut Trench Installation[6]
Construction Total	**$50.2 M**	**$66.3 M**	**$55.1 M**	**$96.2 M**
Design and Administration Allowance	$13.6 m	$17.9 M	$14.9 M	$26.0 M
Total Capital Costs (2018$)[2,4,5]	**$63.8 M**	**$84.2 M**	**$70.0 M**	**$122.2 M**
Total Capital Costs – Midpoint of Construction	**TBD[3]**	**TBD[3]**	**TBD[3]**	**TBD[3]**

Notes:

1. Limited real estate in Route 28 if proposed dual mains are installed is anticipated to increase the costs of the FM to Nobska Point.
2. All costs are shown in 2018 dollars (ENR = 10959). Once a construction timeframe is known for the project, costs should be adjusted to the mid-point of construction.
3. TBD = To Be Determined when midpoint of construction is known.
4. Estimated Capital Costs does not include utility relocation or dewatering due to high groundwater.
5. Total Capital Costs includes allowances for construction costs such as: a 30% construction contingency, a 10% engineering design, 15% fiscal/legal/permitting/construction administration and Resident Project Representative (RPR) costs, and 2 percent survey, soil boring and field investigations.
6. Linear distances are estimated based on straight line distances along the conceptual route. No surveys or subsurface investigations have been conducted as part of this project.

Conceptual cost estimates are developed for two installation technologies—open cut excavation and horizontal directional drilling (HDD). HDD installation is anticipated to be more cost-effective than open cut excavation, but the technology carries larger risks due to unanticipated subsurface conditions. Field investigation and borings will be required to assess the feasibility of installing the ocean outfall using HDD and, if feasible, the extent that HDD may be used. The following next steps are recommended in the evaluation of an ocean outfall:

- Complete a desktop geological study through the review of available subsurface information.

- Conduct subsurface investigations along the proposed ocean outfall path to assess the feasibility of completing construction with HDD.
- Develop a hydrodynamic model for the potential ocean outfall off of Nobska Point.

Appendix A

Assessing the Effect of Ocean Effluent Discharge off of West Falmouth,MA

Assessing the Effect of Effluent Discharge from Proposed Ocean Outfall Sites off of West Falmouth MA

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Final Report – April 11, 2019

Executive Summary

The town of Falmouth MA lies on the southwestern, down-gradient side of a sole source aquifer, the Sagamore Lens, which feeds multiple Falmouth watersheds flowing to coastal embayments of Buzzards Bay and Vineyard and Nantucket Sounds. This trend of ground water flow poses a challenge in choosing a site to discharge the town's treated wastewater. Wastewater discharged at a ground-based site within the town limits will, almost certainly, be carried in the down-gradient ground water flow into a coastal embayment. Currently, the primary site at which the Town of Falmouth disposes treated wastewater is at the Town's Waste Water Treatment Facility in West Falmouth, which lies in the watershed of West Falmouth Harbor. It is well established that West Falmouth Harbor has experienced significant degradation due to the influx of nutrients carried in the treated wastewater discharged at the treatment facility. As part of Falmouth's current wastewater planning process, potential upland discharge sites throughout town are being evaluated. Several have been found to be problematic because of the projected impact on coastal estuaries.

An alternative to upland discharge of treated wastewater is an ocean outfall. Historically, this option has been widely used by municipalities in southeastern New England, and is still employed (for example, by New Bedford, Dartmouth, and Boston). An ocean outfall was also used in Woods Hole from 1946 through 1985.

As part of a study of wastewater discharge options, the Town of Falmouth contracted with GHD and its consultants, J. Churchill, G. Cowles and J. Rheuban, to evaluate the potential impact of effluent discharge from an ocean outfall in Buzzards Bay. Two candidate outfall locations were considered. One was 4380 ft from the West Falmouth shoreline and the other was roughly 900 ft further offshore. High-resolution modeling was conducted to track a hypothetical effluent discharged from these two sites. The discharge was set at a rate of 4 million gallons per day and at a total nitrogen concentration of 3 mg/L (by permit, the maximum concentration of total nitrogen currently allowed in effluent discharged by the Town of Falmouth).

The results of the year-long, effluent-tracking simulations reveal that the hypothetical discharge from either candidate outfall would have negligible effects on total nitrogen concentrations in surrounding waters. The projected impact of the discharge on the total nitrogen concentration in either Buzzards Bay or West Falmouth Harbor is considerably less than 1 percent. Furthermore, because the candidate outfalls are situated in an area of vigorous ocean mixing, the simulations show that the discharged effluent is rapidly diluted upon release. This is a

critical finding as national shellfish sanitation regulations generally prohibit shellfish harvest within the area where the effluent dilution from an outfall is less than 1000:1. The simulations indicate that 1000:1 effluent dilution is achieved within 600 ft from either proposed outfall location, and far distant from any shellfish beds. Importantly, the simulations show little difference in the impact of effluent discharge from one outfall vs the other, making the more inshore site, which would be less expensive to install, a viable option for replacing the current ground-based discharge of wastewater at the West Falmouth Treatment Facility and eliminating this large source of nitrogen pollution to West Falmouth Harbor.

1. Introduction – Goals and Approach

Currently, the wastewater effluent processed by the Town of Falmouth MA at two Waste Water Treatment Facilities (WWTFs) is discharged into ground-based infiltration basins (open sand beds at the West Falmouth WWTF and leaching trenches at the North Falmouth WWTF). The open sand beds of the West Falmouth WWTF, which services most of the properties connected to the town's sewer lines and processes septage as well, are located less than 1 mile from West Falmouth Harbor (Figure 1). The effluent enters a ground water flow directed towards West Falmouth Harbor and Buzzards Bay, and has degraded the water quality of West Falmouth Harbor. In 2005, Falmouth upgraded the plant to tertiary treatment and reduced the concentration of nitrogen in the effluent to 3 mg/L. Falmouth is now exploring options to remove all future discharge of treated effluent from the ground water and West Falmouth Harbor.

As part of the investigation into alternate methods of wastewater effluent disposal, GHD and subcontractors, J. Churchill, G. Cowles and J. Rheuban, were contracted to explore the possibility of open-water effluent discharge at an ocean outfall within Buzzards Bay. The area considered for siting an outfall was directly offshore of West Falmouth (Figure 1). The GHD team was charged with the principal goals of:

1. Selecting suitable open-water outfall sites based on the potential effect of disposal on coastal and estuarine concentrations of total nitrogen (TN).
2. Undertaking high-resolution modeling to estimate the coastal and estuarine TN concentrations resulting from effluent discharge of 3 mg/L at the chosen sites.

In working towards Goal 1, the team utilized high resolution bathymetry of the area of Buzzards Bay off of West Falmouth with the aim of selecting candidate outfall sites in an area of deep water (to maximize initial dilution) and sufficiently distant from the coast so that the effluent discharged from the sites would have a negligible effect on nutrient concentrations in sensitive areas (e.g., West Falmouth Harbor). As detailed below, two locations were selected as potential open-water outfall sites.

A high-resolution water quality model was employed to achieve Goal 2. The model utilized velocity fields generated by a high-resolution hydrodynamic model to track the movement and mixing of TN in effluent (hereafter 'effluent TN') set out from the candidate outfall sites. To assess the possible effect of discharge from the candidate sites on existing TN concentrations, the effluent TN concentrations estimated by the model were compared with recent measurements of TN concentrations acquired in Buzzards Bay and West Falmouth Harbor.

This report details the data and methodology employed by the GHD team (Section 2 and Appendix 1), offers a detailed description of the findings focusing on the effects of the proposed open-water discharge on TN concentrations in Buzzards Bay and West Falmouth Harbor (Section 3), and concludes with a summary of the findings (Section 4).

2. Methods

2.1 Measured Total Nitrogen Concentrations

The measurements of TN concentrations used in this study came from two principal sources: Massachusetts Estuary Project (MEP; see Howes et al., 2006 and http://www.smast.umassd.edu/Coastal/research/estuaries/estuaries.html) and the Buzzards Bay Coalition online Bay Health data directory (http://www.savebuzzardsbay.org/bay-health/)

As part of data acquisition for both sources, water samples were collected over the summer months (primarily in July-September) and principally during the last three hours of an outgoing tide. The samples were either filtered on site or after immediate transport to a laboratory. The typical sample analysis procedure was carried out as follows. Inorganic nutrients, nitrate and nitrite, (NO_3^- and NO_2^-) were analyzed spectrophotometrically by automated Cd reduction (Johnson and Petty, 1983). Ammonium (NH_4^+) was measured using the phenol hypochlorite method (Strickland and Parsons, 1972). Total dissolved nitrogen (TDN; the sum of NO_3^- + NO_2^- + NH_4^+) was measured as nitrate following persulfate digestion (D'Elia and Steudler, 1977). Particulate organic nitrogen (PON) was measured by elemental analysis (Sharp, 1974). The methodology is outlined in a Quality Assurance Project Plan that has been approved by the Massachusetts Department of Environmental Protection and the U.S. Environmental Protection Agency (Williams and Neill, 2014).

Detection limits for nitrogen measurements in coastal waters vary depending on the methods and instruments used for analysis. In West Falmouth Harbor, TN has commonly been determined from measurements of various nitrogen species including some combination of nitrate+nitrite (detection limit 0.0014 mg/L), ammonium (detection limit 0.0014 mg/L), total dissolved nitrogen (detection limit 0.007 mg/L), and particulate organic nitrogen (detection limit 0.007 mg/L) (Gunn et al. 1994). The best estimate of the overall detection limit for TN from West Falmouth Harbor is then in the range of 0.0014-0.007 mg/L, with the limit of a particular sample depending on the relative contribution of each nitrogen species.

TN concentrations in the interior of Buzzards Bay were characterized using the data from the Buzzards Bay Coalition mid-bay buoy (BBC in Figure 1). TN concentrations from samples taken at the buoy are available online (http://www.savebuzzardsbay.org/subembayments/center-bay-buoy/) in the form of yearly averages spanning the period 2007-2017.

Characterization of recent (2016-2018) TN concentrations within West Falmouth Harbor was carried out using data from MEP sampling at seven sites within the harbor (Table 1, Figure 2).

Longer-term (1992-2017) data of TN concentration within West Falmouth Harbor were obtained from the Buzzards Bay Coalition website (see above), which lists yearly averages of TN concentration at the seven MEP sampling sites (Figure 2).

2.2 Bathymetry Data

The choice of candidate open-water outfall locations was guided by high resolution bathymetry data compiled by the US Geological Survey (USGS). The data represent the bottom depth at a horizontal resolution of 10 m (32.8 ft), and are produced using public domain depth measurements as well as measurements from LiDaR (Light Detection and Ranging) surveys (Andrews et al., 2018). In coastal waters, the fine resolution of the USGS bathymetry is principally a product of the LiDaR depth data, which are typically acquired from aircraft and have an accuracy of ± 15 cm (6 inches) and maximum vertical extent of roughly 130 ft.

The USGS data show complex bathymetry in the area considered for open-water outfall (Figure 3). Prominent features are Gifford Ledge, extending offshore of West Falmouth, and an unnamed deep (>50 ft) basin to the north. As detailed below (Section 3.1), the basin was chosen as the area to be considered for siting an open-water outfall. It is situated offshore of a coastal escarpment that extends roughly 3500 ft from shore (Figure 4).

2.3 Hydrodynamic Model

As noted in the Introduction, the modeling component of this project was carried out in two parts. In the first, flows in the region of interest (Buzzards Bay and the estuaries near the proposed open-water outfalls) were simulated with a high-resolution hydrodynamic model. The second part entailed using the modeled flow fields to simulate the transport and mixing of effluent discharged at the proposed outfalls. This section presents an overview of the hydrodynamic model (with further details presented in Appendix 1). Details of the plume tracking model are presented in Section 2.4.

2.3.1 Model Description

The hydrodynamic modeling was carried out using the Finite-Volume Community Ocean Model (FVCOM: Chen et al., 2003, 2006; Cowles, 2008), an open source model system with over 4000 registered users that has been applied in a wide array of coastal and open ocean studies. FVCOM operates by solving the equations of motion on an unstructured grid, with triangular elements that can be aligned with coastline and bathymetric irregularities. To produce a 3-dimensional solution, FVCOM employs a sigma-coordinate system, in which the vertical component of the model domain is divided into a fixed number of layers that follow changes in model terrain. Layer thickness is thus proportional to water depth.

The modeling of this project was carried out using a regional FVCOM-based model known as the Southeastern Massachusetts-FVCOM (SEMASS-FVCOM), which includes the Massachusetts and Rhode Island coastal zones as well as Long Island Sound (Figure 5). SEMASS-FVCOM has been employed, and its output velocity fields extensively evaluated, by co-PIs Cowles and Churchill for recent studies of tidal energy in the Massachusetts coastal zone (Hakim et al., 2013; Cowles et al., 2017) and the dispersal of bay scallop larvae in Buzzards Bay (Liu et al., 2015). Churchill and Cowles are currently using SEMASS-FVCOM in two NOAA-funded studies. One is aimed at assessing the impact of climate change on the delivery of lobster larvae to suitable juvenile habitat off of southern New England. The other is directed at quantifying the impact of municipal sewage discharge on coastal acidification, focusing on effluent released by the towns of New Bedford, Fairhaven and Wareham MA.

In the regions of interest, the SEMASS-FVCOM grid scales are small (Figure 5), giving highly resolved modeled velocity fields. In the area considered for the proposed outfalls, for example, the horizontal grid spacing is of order 350 ft (giving modeled velocities of the same spacing). Grid spacing is smaller close to shore and in the estuaries, equaling 100-200 ft in West Falmouth Harbor. In the vertical, SEMASS-FVCOM is divided into 20 equally spaced layers, giving a fine vertical resolution of velocity in coastal water (i.e., of 0.5 ft in 10 ft of water).

As demonstrated by the modeled surface velocity fields of maximum flood and ebb tides on 14-15 July 2015 (Figure 6), the model captures the complexities of the flows over West Falmouth Harbor and the near-shore waters to the west.

<u>2.3.2 Model Forcing and Execution</u>

As detailed in Appendix 1, the hydrodynamic model simulations were generated with a full suite of natural forcings, including tides, surface wind stress, surface heat flux and fresh water runoff. For this project, the model was executed to produce velocity fields, saved at hourly intervals, throughout 2015. These fields were then used by the water quality model to simulate the movement and mixing of effluent discharged from the proposed ocean outfall, as described in the section below.

Further details of the operation of the hydrodynamic model and examples of model verification using field data are given in Appendix 1.

2.4 Water Quality Model

The water quality model is formulated to operate on a grid linked with the hydrodynamic model grid (Figure 5). In the horizontal plane, the grid is formed by a series of polygon-shaped cells, which give the grid something of a honeycomb appearance (Figure 7). In the vertical plane, each cell is divided into 20 evenly spaced layers (corresponding to the sigma-layers of the hydrodynamic model). This arrangement gives a concentration field with a fine vertical resolution. For example, a cell with a water depth of 10 ft would consist of layers with a thickness of 0.5 ft. The effluent concentration is considered to be uniform within each layer of each cell.

The advective transport (carried by water velocities) of effluent across the grid cell boundaries and between the grid cell layers is determined using the velocities from the hydrodynamic model. Diffusive transport across the cell and layer boundaries is also incorporated into the model.

In mathematical terms, the effluent-tracking simulations operate by solving the diffusion-advection equation in three dimensions with a source term (applied at the outfall). Denoting the effluent TN concentration as C_E, the equation is expressed as:

$$\underbrace{\frac{\partial C_E}{\partial t}}_{A} = \underbrace{-\left[u\frac{\partial C_E}{\partial x} + v\frac{\partial C_E}{\partial x} + w\frac{\partial C_E}{\partial z}\right]}_{B} + \underbrace{K_H\left[\frac{\partial^2 C_E}{\partial x^2} + \frac{\partial^2 C_E}{\partial y^2}\right]}_{C} + \underbrace{K_V\frac{\partial^2 C_E}{\partial z^2}}_{D} + \underbrace{SS}_{E}$$

where: x, y and z are the east, north and vertical coordinates, respectively; t is time; u, v and w are the east, north and vertical velocity components; K_H and K_V are the horizontal and vertical diffusivities; and SS is the source of TN introduced at the outfall.

Solving for the change in effluent TN concentration (term A above) in each layer of each cell is done by determining the advective fluxes (term B) of TN through the horizontal and vertical boundaries of each layer of each cell. Also included are the diffusive TN fluxes through the layer's horizontal (term C) and vertical (term D) boundaries. The advective fluxes are determined with the velocities output from the hydrodynamic model. In determining the horizontal diffusive fluxes, K_H is set to a uniform value of 0.2 $m^2\ s^{-1}$. Values for K_V are taken from the output of the hydrodynamic model (K_V depends on the vertical shear of the horizontal velocity) with a minimum value of 0.3 $\times 10^{-2}\ m^2\ s^{-1}$ imposed.

The input of effluent TN (term E) occurs in the control volume encompassing the outfall (Figure 7) at a rate (mass per unit time) of $V^*C_{discharge}$, where $C_{discharge}$ is the concentration of TN emerging from the outfall and V is the volume rate of discharge. It is assumed that the discharged TN is initially mixed vertically and horizontally in the model cell containing the outfall. It is also assumed that all effluent TN passing through the model's oceanic (open) boundary (Figure 5) is lost to the system (i.e., does not return to the model domain). As the model boundaries are far from the areas of interest (i.e. eastern Buzzards Bay and the estuaries of West Falmouth), this boundary condition has no appreciable effect on the modeled TN concentrations in these areas.

The model was executed in monthly increments for all of 2015. The final concentration field of each month was used as the initial concentration field for the subsequent month. The model time step was set at 20 s. In solving the equation for each time step, the velocities and K_V values output by the hydrodynamic model (at hourly intervals) were interpolated to center of each time step.

The model code was formulated (in MATLAB) by project team members Churchill, Cowles and Rheuban for use in a MIT Sea Grant-funded project aimed at quantifying the impact of municipal effluent discharge on the carbonate system of coastal waters. As part of this project, the code has been subject to considerable testing (i.e., by comparison of modeled and observed effluent concentration patterns). In the simulations for this project, testing was done for mass conservation (that the accumulated effluent TN in the model domain equaled the amount discharged minus the amount lost at the open oceanic boundary) and to ensure the spatially averaged TN concentrations in each region matched the TN mass in the region divided by the region's volume.

It's important to note that the effluent TN was considered to be conservative, and not subject to natural processes that would tend to extract TN from the water column (i.e., transfer through the air-water interface or biological uptake and transfer to the sediments). The modeled concentrations of effluent TN thus represent an upper limits for TN released from the open-water outfall.

2.5 Selection of Proposed Outfall Sites

In addressing Goal 1 (Section 1), the region deemed most suitable for siting an outfall was confined to the basin immediately north of Gifford Ledge (Figure 3). The basin was deemed to be an ideal area for a near-bottom outfall for two principal reasons. One is its proximity to shore, an important factor when considering outfall construction costs. The second is its depth of greater than 50 ft (Figures 3 and 4), which will allow for considerable initial dilution of discharge effluent through vertical mixing.

Two locations within the basin, both at roughly 52 ft depth, were chosen as candidate outfall sites. Designated as D1 and D2, the sites are roughly 4380 and 5250 ft, respectively, from shore (Figure 3 and 4).

2.6 Discharge Parameters

As noted above (Section 2.4) the rate at which effluent TN is discharged at each outfall site (i.e. the mass rate of discharge in units of mass/time) is a product of two parameters: the volume rate of discharge (V) and the TN concentration contained in the discharge ($C_{discharge}$). In executing the effluent transport simulations, $C_{discharge}$ was set to 3 mg/L, the effluent nitrogen concentration set by permit for the Falmouth WWTF. In accordance with the scope of the project as specified by the Town of Falmouth, V was 4 million gallons per day (MGD), which is roughly twice the current rate of effluent discharge by the town.

It should be noted that because the model system is linear, the simulated effluent TN concentrations are proportional to the mass rate of discharge. For example, with $C_{discharge}$ unchanged, the simulated TN concentrations for a volume rate of discharge of 2 MGD would be half of the concentrations shown here (determined with a 4 MGD discharge rate).

In assessing the viability of the outfall options proposed here, it's useful to compare their site and discharge characteristics with those of existing outfalls situated in Buzzards Bay. At present there are two such outfalls. One, operated by the City of New Bedford MA, is sited off of Clarks Point, New Bedford, while the other is operated by the Town of Dartmouth MA and situated off of Mishaum Point in Dartmouth. Both facilities discharge a wastewater effluent that has received secondary treatment. By contrast, the Falmouth WWTF is a tertiary treatment facility, designed to produce a lower TN effluent concentration than that of a secondary treatment facility. Compared with the projected 4 MGD flow rate through the proposed Falmouth WWTF ocean outfall, the volume rate of release is slightly greater for the Town of Dartmouth discharge (at 4.2 MGD), and is significantly greater for the New Bedford discharge (at 30 MGD). With regard to location, both the New Bedford and Dartmouth outfalls are situated closer to shore and in significantly shallower water (<25 ft depth) than the proposed D1 and D2 sites (which are at >50 ft depth). Because of the difference in release depth, initial dilution through vertical mixing should be less effective at the existing outfalls than at D1 and D2. A detailed comparison of the existing outfalls with D1 and D2 is given in Appendix 2.

3. Results

3.1 Existing Total Nitrogen Concentrations in Buzzards Bay and West Falmouth Harbor

The means of recent TN concentrations compiled in this study show something of a steady rise going from Buzzards Bay into the interior of West Falmouth Harbor (Table 1, Figure 2). The mean TN concentration (in mg/L) increases from 0.26 at the BBC mid-bay buoy to 0.37 at the outer West Falmouth Harbor stations, and further rises to values of 0.42-0.51 and 0.48-0.86 at stations in Chapoquoit Basin and Snug Harbor, respectively. As demonstrated by Howes et al. (2006), a principal cause of this trend is the influx of groundwater-borne nutrients into West Falmouth Harbor from residential septic systems and from the Town of Falmouth's effluent discharge from the WWTF in West Falmouth (Figure 1).

In addition to this spatial trend, long-term (1992-2017) data reveal complex temporal changes in the TN concentrations within West Falmouth Harbor. These are exemplified by the yearly averaged concentrations at the MEP Snug Harbor station (WF5 in Figure 2), often referred to as the 'Sentinel Station' for the inner portion of West Falmouth Harbor. In addition to significant year-to-year variations, of up to 0.24 mg/L, these yearly averages show what appears to be a long-term rise to a maximum TN concentration of 0.79 mg/L in 2010 followed by a decline to concentrations of between 0.48 and 0.61 over the last four years (Figure 8). Yearly averaged TN concentrations at other MEP sites in West Falmouth Harbor show a similar pattern, with maximum values appearing in the 2010-2012 range.

In the sections to follow, the projected concentrations of effluent TN determined by the model simulations are compared with 2016-2018 averages of the measured TN concentrations in West Falmouth Harbor (Tables 1-2, Figure 2), as these averages best represent the existing TN concentrations, unbiased by the 2010-2012 maxima.

3.2 Effluent Transport Simulations

The presentation of the simulation results in this section begins with a description of the projected effluent TN fields resulting from discharge at the two candidate outfalls (Section 3.2.1). This is followed by a description of the manner in which effluent discharged at the two sites is projected to influence TN concentrations in Buzzards Bay (Section 3.2.2) and in West Falmouth Harbor (Section 3.2.3). Attention is given to quantifying the relative effect of discharge at the one site vs the other, i.e., addressing the question of whether or not the discharge at the more offshore site (D2) results in significant lower effluent TN concentrations in West Falmouth Harbor than projected for discharge at the more onshore site (D1). Also considered is the time required to flush effluent TN from the harbor, should it be exposed to relatively high concentrations of effluent TN (Section 3.2.4).

In discussing the model results, the **concentration of effluent TN** (i.e., the TN emanating from the outfall, which is **separate** from the **'background' concentration of TN in the receiving waters**) is denoted at $\boldsymbol{C_E}$ (see the above section). The **vertical average of** $\boldsymbol{C_E}$ is represented by $\boldsymbol{<C_E>}$, whereas the **near-surface effluent TN concentration** is denoted at $\boldsymbol{C_{E(S)}}$. **Time averages of effluent TN concentration** are represented by the operator $\mathbf{[]}_T$ (i.e. $[<C_E>]_{July}$ is the vertically averaged effluent TN concentration that is also averaged over the month of July).

3.2.1 Modeled Fields of Effluent Total Nitrogen Concentration

Monthly averaged fields of both $<C_E>$ and $C_{E(S)}$ reveal a seasonal variation in the distribution of effluent TN that is linked to the seasonal variation in the wind field over the eastern portion of Buzzards Bay (discussed further in Appendix 1). For example, the fields of $[<C_E>]_{July}$ and $[C_{E(S)}]_{July}$ (averaged over July) (Figures 9a, 10a, 11a and 12a) are noticeably skewed with higher concentrations tending to appear to the NNE of the outfall. This pattern is a result of circulation forced by the summer-time winds in eastern Buzzards Bay, which are dominated by the daily sea-breeze and are predominately from the SSW (Figure 13). The wind forcing characteristic of autumn (Figure 13) results in an effluent TN distribution noticeably different from that of the summer. For example, the $[<C_E>]_{Nov.}$ and $[C_{E(S)}]_{Nov.}$ fields have lower values than their July counterparts due to the stronger autumn winds, which result in more vigorous wind-driven flow and more rapid transport of effluent away from the outfall (Figures 9-12). Furthermore, the November-averaged effluent TN fields tend to be skewed with higher concentrations appearing to the south of the outfall, a result of the tendency for winds of November to be directed from the WNW (Figure 13).

Notable in all monthly averaged $<C_E>$ and $C_{E(S)}$ fields is a rapid decline in the concentration of effluent TN moving away from the outfall (Figures 9-12). For example, the November-averaged TN concentrations of effluent discharged from outfall D1 (i.e., $[<C_E>]_{November}$) (Figure 9b) decline from 0.0046 mg/L at the cell containing the outfall to no greater than 0.0017 mg/L at cells roughly 800 ft from the outfall. All monthly averaged fields of $<C_E>$ and $C_{E(S)}$ show a similar pattern of decline (by a factor of more than 2.5 going 800 ft from the outfall).

The rapid decline in $<C_E>$ moving away from the outfall is reflected in the fields of dilution ratio, defined here as the ratio of the $<C_E>$ to the TN concentration released at the outfall ($C_{discharge}$ = 3 mg/L for the simulations here). Mathematically, the dilution ratio averaged over some time period, T, is defined as

$$F_T = \frac{C_{discharge}}{[<C_E>]_{\mathrm{T}}},$$

It should be noted that because a change in $C_{discharge}$ will produce a corresponding change in $[<C_E>]_T$, F_T is independent of the discharged TN concentration.

National shellfish sanitation regulations generally prohibit shellfish harvest within the area of less than 1000:1 dilution from a WWTF outfall. All monthly F_T fields exhibit a rapid increase in magnitude moving away from the outfall and contain a small area over which F_T is less than the 1000:1 threshold. In all but the summer months of July-September, the region over which F_T<1000:1 is confined to the outfall cell (e.g., the $F_{November.}$ field for discharge from D1 shown in Figure 14b). In the F_T fields of the summer months, the area with dilution of less than 1000:1 is contained within two cells and extends no more than 600 ft from the outfall (e.g., the $F_{July.}$ field in Figure 14a).

Of particular importance are the low values of all the monthly averaged $\langle C_E \rangle$ and $C_{E(S)}$ fields. Even with the outfall cell included, monthly averaged $\langle C_E \rangle$ fields determined with effluent discharge at D1 never exceed 0.0056 mg/L, while the monthly averaged $\langle C_E \rangle$ fields determined with the outfall at D2 have a slightly smaller maximum of 0.0049 mg/L. These maximum values are dwarfed by the TN concentrations measured at the BBC buoy in the central region of Buzzards Bay (Table 1).

3.2.2 Effect on Total Nitrogen in Buzzards Bay

In all modeled C_E fields, the decline in magnitude is particularly steep going westward from the outfall into the interior of Buzzards Bay (Figures 9-12). For example, in the fields computed with the outfall at D1, the yearly averaged $\langle C_E \rangle$ at a site roughly ½ mile due west of the outfall is 0.001 mg/L, more than a factor of 5 lower than the yearly averaged $\langle C_E \rangle$ at the outfall. C_E is further reduced going westward to the central region of Buzzards Bay. The yearly averaged $\langle C_E \rangle$ (for discharge at D1) in the cell containing the BBC mid-bay buoy (Figure 1) is 0.00016 mg/L, more than three orders of magnitude lower than the mean TN concentration derived from samples taken at the buoy (Table 1). However, as these samples were taken near the surface and predominately during summer, a more appropriate comparison of their mean TN concentration is with the summertime average of the $C_{E(S)}$ within the model cell containing the buoy. For the discharge at D1 and D2, this concentration ($[C_{E(S)}]_{\text{June-Aug.}}$) is, respectively, 0.00024 and 0.00025 mg/L, both still three orders of magnitude less than the mean TN concentration measured at the buoy (Table 2, Figures 15 and 16). Clearly, the model simulations indicate that open-water discharge at sites D1 and D2 should have negligible effect on the TN concentrations within the central portion of Buzzards Bay.

Furthermore, the monthly averaged model results described in the section above indicate that the impact of discharge at D1 and D2 is negligible on TN concentrations throughout Buzzards Bay, even in the area near the discharge sites. In particular, the largest monthly averaged $\langle C_E \rangle$ of 0.0056 mg/L (for an outfall at D1) is 2 % of the mean TN measured at the BBC buoy (Table 1). It's noteworthy that even if the discharge TN concentration were, for some reason, temporally increased to 10 times the permitted value of 3 mg/L, the maximum $\langle C_E \rangle$ determined by the model would still be a relatively small fraction, order 20 %, of the existing TN concentration in Buzzards Bay. As demonstrated below (Section 3.2.5), the flushing near the selected outfalls is vigorous and would quickly reduce any increase in $\langle C_E \rangle$ resulting from discharge at an elevated concentration of TN.

3.2.3 Effect on Total Nitrogen in West Falmouth Harbor

A number of metrics are employed to assess the effect of effluent discharge from each of the candidate outfall locations on the mass and concentration of TN within West Falmouth Harbor. One such metric is the spatially averaged value of C_E within the harbor. This is taken as the ratio of the total mass of effluent TN within the model cells contained within the harbor (the colored cells in Figure 17) to the overall volume of these cells, which changes with the rising and falling tide.

The time series of the mean C_E in West Falmouth Harbor (Figure 18) have prominent temporal variations spanning 0.003 mg/L. These include a weak seasonal modulation (tending to be higher during the warmer months) as well as changes occurring on a roughly weekly time scale. These variations are presumably the result of varying wind conditions and the attendant changes in the delivery of effluent TN to West Falmouth Harbor.

Despite these variations, the magnitude of the mean C_E in West Falmouth Harbor is always minute compared with the recent measurements of TN concentration in the harbor. With the exception of a brief period of <1.5 days in early September, the mean C_E in the harbor is less than 0.003 mg/L. By contrast, the measured TN concentrations in the harbor are typically well in excess of 0.3 mg/L (Table 1, Figure 2).

Of particular note is the similarity of mean C_E in West Falmouth Harbor derived from simulations with the outfall at D1 and D2 (the blue and magenta lines in Figure 18). The mean C_E of effluent discharged from D2 is typically lower than the mean C_E of effluent from D1, but never by more than 0.001 mg/L and by less than 0.0004 mg/L over 98 % of the year-long series.

Another metric used in assessing the effect of effluent discharge on TN within West Falmouth Harbor is the projected time series of $\langle C_E \rangle$ at the Snug Harbor sentinel station (Figure 2). For discharge at either outfall location, the sentinel station $\langle C_E \rangle$ never exceeds 0.003 mg/L (Figure 19a) and is always less than 0.5 % of the mean measured concentration (averaged over 2016-2018) of TN at the sentinel stations (Figure 19b). As seen in time series of mean C_E in West Falmouth Harbor, the time series of sentinel station $\langle C_E \rangle$ derived from simulations with the outfall at D1 and D2 are closely aligned, differing by less than 0.0004 mg/L over 98 % of the series.

A final metric considered here is a comparison of the measured existing TN concentration with the projected mean $\langle C_E \rangle$ in three regions of West Falmouth Harbor: the Outer Harbor, Snug Harbor and Chapoquoit Basin (Figure 17). The mean existing TN concentration in each region is taken as the average of all Pond Watcher measurements taken within the region's border over 2016-2018 (Table 1, Figure 2). The mean existing TN concentration is greatest in Snug Harbor and smallest in the Outer Harbor (Figure 15). By contrast, the mean 'regional' $\langle C_E \rangle$ (averaged over the full year of 2015) is greatest in the Outer Harbor, owing to closer proximity to the outfall than the other two regions. For all regions, the mean $\langle C_E \rangle$ is less than 0.4 % of the mean existing TN concentration (Figure 16).

From all the metrics presented above, it is clear that the model simulations indicate that the effect of discharged effluent on TN concentrations in West Falmouth Harbor should be negligible for effluent released from either site D1 or D2.

3.2.4 Flushing Rate of West Falmouth Harbor

To explore a worst-case discharge scenario, simulations were carried out to examine the flushing of West Falmouth Harbor after being subject to outfall discharge with elevated concentrations of TN. The model setup consisted of a month-long period over which effluent with an elevated TN concentration was discharged at D1 followed by a month-long period of no effluent discharge. For the month of discharge, $C_{discharge}$ was set to 35 mg/L and the volume rate of discharge was set to 4 MGD. The simulations tracked the rise and subsequent decline (after discharge shutoff) of effluent TN contained in the harbor (in the colored cells of Figure 17). The simulations were carried out over two periods with discharge in July and November and followed by model execution without discharge in August and December.

The results show that following the discharge shutoff, the mean concentration of effluent TN within the harbor undergoes a steady decline modulated by relatively small tidal variations (Figure 20a). For both periods modeled, the total mass of effluent TN contained in the harbor is reduced by a factor of 10 roughly 3 weeks after shutoff (Figure 20b). At this 3-week after shutoff mark, the mean C_E within the harbor is roughly 0.001 mg/L in the December simulation and slightly higher, roughly 0.0016 mg/L, in the August simulation. Based on these results, it appears that TN is effectively flushed from West Falmouth (to 10 % of the original mass) over a period of 3 weeks.

3.2.5 Flushing Rate of Buzzards Bay at Site D1

In considering the worst-case discharge scenario outlined above, another concern is the rate at which effluent TN resulting from a temporary discharge at an elevated TN concentration is flushed from the area near the discharge site. It may be expected that the flushing of TN from the open water near the outfall locations would proceed relatively quickly. This expectation is borne out by the simulations described above. Following the cessation of the discharge with $C_{discharge}$ set to 35 mg/L, the time series of $\langle C_E \rangle$ at the discharge site (D1) shows a dramatic drop over the first hour (Figure 21), by a factor of roughly 2 and 4, respectively, in the August and December simulations. Following this drop, $\langle C_E \rangle$ at the discharge site continues to decline, and is reduced to 10% of its value at discharge cessation in roughly 8 and 2 days, respectively, in the August and December simulations. Based on these results, it appears that in the 'worst-case scenario' specified here, any temporary increase in effluent TN near the outfall would be quickly flushed away, in the course of a few days.

4. Summary

The outfall option considered here, relocating the Town of Falmouth effluent discharge from the West Falmouth facility to Buzzards Bay, would completely remove the discharge of treated effluent into the groundwater upstream of West Falmouth Harbor, and thus eliminate a significant source of TN to the Harbor. This study addressed the question of whether or not implementation of this option might have a significant impact on TN concentrations in areas of Buzzards Bay or its tributaries.

The results of the year-long simulations of effluent transport from two candidate outfall sites in Buzzards Bay leads to the overarching conclusion that this effluent will have negligible effects on TN concentrations in Buzzards Bay and West Falmouth Harbor. Among the notable findings supporting this conclusion are:

- Except at the candidate outfall sites, the monthly averaged concentration of effluent TN derived from the simulations never exceeds 0.004 mg/L.
- In the central portion of Buzzards Bay, the projected mean concentration of effluent TN is of order 0.00024 mg/L and less than 0.1 % of the mean existing TN concentration measured by the Buzzards Bay Coalition at a mid-bay location.
- The dilution factor of effluent TN (computed from monthly averaged TN fields generated by the model) exceeds 1000:1 at all locations except for a small area near each candidate outfall site.
- For the year-long simulation, the projected mean concentration of effluent TN carried into West Falmouth Harbor is less than 0.003 mg/L for all but a brief period, 1.5 days, when the mean concentration rises to 0.0036 mg/L.
- The concentration of effluent TN at the Snug Harbor sentinel station within West Falmouth Harbor, derived from the year-long simulation, never exceeds 0.003 mg/L and is always less than 0.5 % of the mean TN concentration measured at the station.

The overarching conclusion is reinforced by noting that because the effluent TN was considered to be conservative, and not subject to natural processes that would extract nitrogen from the water column, the modeled effluent TN concentrations should be viewed as upper limits.

Appendix 1 – Details and Testing of the Hydrodynamic Model

As noted in Section 2.3, the hydrographic modeling of this project was carried out with a model encompassing all of Buzzards Bay as well as the Massachusetts and Rhode Island coastal zones and Long Island Sound (Figure 5). The purposes of this Appendix are to describe this model, known as SEMASS-FVCOM, in much greater detail than done in the main body of the report and to present information on model assessment in the areas of interest.

A.1 Grid Setup

As part of a recent project directed at the potential effect of effluent discharge into the western portion of Cape Cod Canal, the computational mesh of SEMASS-FVCOM has been refined in Buzzards Bay and Cape Cod Canal. The refined model grid contains 284,305 elements in the horizontal and 20 evenly spaced sigma-layers in the vertical.

The model bathymetry is interpolated from a composite dataset. The majority of the model domain is encompassed by the 3-arcsecond Gulf of Maine bathymetry product (Twomey and Signell, 2013) and the 1/3-arcsecond Nantucket Inundation Digital Elevation Model (NOAA: Eakins et al., 2009). Data from a directed sounding survey are used to specify the bathymetry of the Cape Cod Canal (USACE, 2011). The coastal boundary is derived from a high-resolution (1/2 arc-second) product developed and distributed by the Massachusetts Office of Coastal Zone Management.

A.2 Boundary Forcing

The model is driven at the open boundary by sea surface elevation constructed from the six primary tidal constituents (M_2, S_2, N_2, K_1, O_1 and M_4). The phase and amplitude of these constituents and the associated regional barotropic response have been extensively evaluated in prior work (Cowles et al., 2017). Values of the salinity and temperature of water flowing into the domain are also set at the open boundary from a hindcast of a large-scale Gulf of Maine/Southern New England FVCOM-GOM model developed by Dr. Changsheng Chen of U. Mass. Dartmouth (NeCOFS, 2017).

A.3 Surface Forcing

At the surface, SEMASS-FVCOM is driven by net heat flux and surface wind stress, which are also derived from the regional 30-year FVCOM-GOM hindcast (NECOFS, 2017). The wind field in Buzzards Bay during 2015 (the year of the model runs for this project) displays a strong seasonality (Figure A1-1). The southwest sea breeze dominates the Buzzards Bay wind field from late spring to early fall. By contrast, winds from late fall to early spring are characterized by synoptic events with the strongest wind magnitudes directed from NW and NE. These characteristics of the 2015 wind field are typical of the seasonal wind field in Buzzards Bay (Liu et al., 2015).

In addition to utilizing wind and heat flux data to force the model at the surface, the model simulations also employ satellite-derived sea surface temperature (SST) derived from the NOAA

4 km-resolution product. SST is assimilated into the model using a Newtonian relaxation (nudging) approach, which adjusts the modeled SST to best match the observed SST.

A.4 Freshwater Input

Freshwater is input into the model domain at discrete points along the coastal boundary. The locations of the freshwater entry points into Buzzards Bay are based on the watershed delineations of the Buzzards Bay National Estuary Project, which established 32 watersheds draining into the bay. Currently, the only long-term record of freshwater influx into the bay is from a gauge in the Paskamansett River in Dartmouth (USGS 01105933). The freshwater flow from the other watersheds is estimated by multiplying the gauged flow of Paskamansett River by the ratio of a given watershed's area to the area of the Paskamansett River watershed. Using this method, it is estimated that the average freshwater discharge into the Bay in 2015 is 21.3 $m^3\ s^{-1}$, which is 14% below the 20-year annual mean discharge of 25.0 $m^3\ s^{-1}$. Inputs of freshwater from the major rivers outside the Bay (Connecticut, Blackstone, Pawtuxet, Taunton, Neponset, and Charles) are included in the model and are specified from hourly flow data recorded by USGS gauges (available from https://waterdata.usgs.gov/nwis).

A.5 Execution and Data Archiving

The model was executed for the period Jan 1, 2015 to Jan 1, 2016 using a time step of two seconds. The execution required 110,000 core-hours of wall time on 2.6 GHz Intel Haswell Xeons. The two-dimensional fields of sea surface height and depth-averaged velocity, and the three-dimensional fields of velocity, temperature, salinity, and the vertical turbulent eddy diffusivity and viscosity were archived at hourly intervals into NetCDF format files. The total dataset (1.5 TB in size) is accessible through the SMAST Thredds server at: http://www.smast.umassd.edu:8080/thredds/catalog/buzzards/BBC_WW/catalog.html.

A.6 Model Testing and Verification

SEMASS-FVCOM has been subject to extensive testing, principally through comparison of the model results with water levels and velocities measured at various locations within the model domain. Notable comparison studies have been carried out by Lui et al. (2015) as part of a study of the dispersal of bay scallop larvae in Buzzards Bay, by Cowles et al. (2017) as part of a study of tidal energy in the Massachusetts coastal zone, and most recently by project members Churchill, Cowles and Rheuban as part of a study of the possible impact of effluent discharge into Cape Cod Canal (Churchill et al., 2017). All studies found that the velocities and water levels produced by the model to be in close agreement with observations.

To assess the performance of the model in the areas of interest (West Falmouth Harbor and the waters of Buzzards Bay west of Falmouth MA), the model-derived velocities are compared with velocities measured by the USGS at two sites: one in the open waters of Buzzards Bay roughly 3 nautical miles SSW of the entrance to West Falmouth Harbor (hereafter, the southern Buzzards Bay site) and the second in the entrance to the Harbor (hereafter, the harbor entrance). The measurements at these sites did not overlap with our 2015 simulation period, so the comparison is directed at the velocity statistics (i.e., does the model produce velocities with statistical properties similar to those of the measured velocities?). The focus is on the mean velocity and the tidal and subtidal signals.

The model velocity statistics were derived from model time series taken from the model cell with the center closest to the location of the observed currents and from the same time period (i.e., same year days) as the observed current time series. The tidal signal of the observed and model currents was determined by decomposing the depth-averaged velocity time series into the principal tidal constituents (M2, S2, N2, K1, O1 and M4) using the MATLAB routine T-Tide (Pawlowicz et al., 2002). The harmonic constituents were then used to reconstruct the full tidal signal of the model and measured currents for the measured current time period. Properties of the subtidal velocity signal were determined from the low-pass-filtered (40-hr half-power period) vertically-averaged modeled and observed velocity time series at each measurement site.

The modeled and observed depth-averaged velocities at the southern Buzzards Bay site both have a tidal signal roughly twice as strong as the subtidal signal and very weak (order 1.5 cm/s) means (Figure A1-2). The reconstructed observed and modeled tidal velocity signals are highly correlated (R=0.88) with tidal ellipses that are nearly identical in orientation and magnitude (Figure A1-2a).

The modeled and observed depth-averaged velocities at the harbor entrance are both dominated by the tidal signal, with a subtidal signal and mean that are very small in comparison (Figure A1-3). As at the southern Buzzards Bay site, the measured and modeled tidal signal at the harbor entrance are highly correlated (R=0.90) with tidal ellipses of similar orientation and magnitude (Figure A1-3a).

Based on this comparison, and the more extensive validation work cited above, the model is deemed capable of closely reproducing the flows of Buzzard Bay, including the area of the proposed outfall sites.

Appendix 2 – Comparison of Proposed to Existing Effluent Outfalls in Buzzards Bay

As noted in Section 2.6, there are currently two municipal outfalls discharging effluent directly in Buzzards Bay. One, operated by the City of New Bedford MA, is situated off Clarks Point, New Bedford (Figure A2-1a). The other is operated by the Town of Dartmouth, MA and is sited east of Mishaum Point in Dartmouth (Figure A2-1b). Given here is a tabulated comparison of the characteristics of these outfalls with the characteristics of the candidate outfalls proposed for the Town of Falmouth (Table A2-1). In brief, both candidate outfall locations are further from shore, in deeper water, have lower flow rate and likely much lower effluent TN concentration (because of tertiary vs secondary waste water treatment) than either of the existing outfalls in Buzzards Bay.

References

Andrews, B.D., Baldwin, W.E., Sampson, D.W., and Schwab, W.C. (2018). Continuous bathymetry and elevation models of the Massachusetts coastal zone and continental shelf (ver. 2.0, April 2018): U.S. Geological Survey data release, https://doi.org/10.5066/F72806T7.

Chen, C., Beardsley, R., and Cowles, G. (2006). An unstructured-grid Finite-Volume Coastal Ocean Model (FVCOM) System, *Oceanography,* 19 (1), 78–89.

Chen, C., Liu, H., and Beardsley, R. C. (2003). An unstructured grid, finite-volume, three-dimensional, primitive equations ocean model: application to coastal ocean and estuaries, *Journal of Atmospheric and Oceanic Technology,* 20 (1), 159–186.

Churchill, J., Cowles, G., and Rheuban, J. (2017). Assessing the Impact of Increased Effluent Discharge into Cape Cod Canal. Report to the Buzzards Bay Coalition, 45 pp.

Cowles, G. W. (2008). Parallelization of the FVCOM coastal ocean model, *International Journal of High Performance Computing Applications,* 22 (2), 177–193.

Cowles, G.W., Hakim, A., and Churchill, J. H. (2017). A comparison of numerical and analytical predictions of the tidal stream power resource of Massachusetts, USA, *Renewable Energy*, 114, 215-228, https://doi.org/10.1016/j.renene.2017.05.003.

D'Elia, C. F., and Steudler, P. A. (1977). Determination of total nitrogen in aqueous samples using persulfate digestion, *Limnology and Oceanography,* 22, 760-764.

Eakins, B. W., Taylor, L. A., Carignan, K. S., Warnken, R. R., Lim, E., and Medley, P. R. (2009). Digital elevation model of Nantucket, Massachusetts: Procedures, data sources and analysis, National Oceanic and Atmospheric Administration, National Environmental Satellite, Data, and Information Service, National Geophysical Data Center, Marine Geology and Geophysics Division.

Gunn, E., Janik, D. S., and Costa, J. E. (1994) Quality assurance project plan for the Buzzards Bay Project. Buzzards Bay National Estuary Program report, Available at https://buzzardsbay.org/download/qappcitz1994.pdf

Hakim, A. R., Cowles, G. W., and Churchill, J. H. (2013). The impact of tidal stream turbines on circulation and sediment transport in Muskeget Channel, MA. *Marine Technology Society Journal, 47*(4), 122-136. doi:10.4031/mtsj.47.4.14.

Howes B., Kelley, S. W., Ramsey, J. S., Samimy, R., Schlezinger, D., and Eichner, E. (2006). Linked Watershed-Embayment Model to Determine Critical Nitrogen Loading Thresholds for West Falmouth Harbor, Falmouth, Massachusetts. Massachusetts Estuaries Project, Massachusetts Department of Environmental Protection.

Johnson, K. S., and Petty, R. L. (1983). Determination of nitrate and nitrite in seawater by flow injection analysis. *Limnology and Oceanography,* 28, 1260 – 1266.

Liu, C., Cowles, G. W., Churchill, J. H., and Stokesbury, K. D. (2015). Connectivity of the bay scallop (Argopecten irradians) in Buzzards Bay, Massachusetts, U.S.A. *Fisheries Oceanography, 24*(4), 364-382. doi:10.1111/fog.12114.

NECOFS (2017). Northeast Coastal Ocean Forecasting System (NECOFS) Main Portal http: //fvcom.smast.umassd.edu/necofs/. Accessed: 2017-05-20.

Pawlowicz, R., Beardsley, B., and Lentz, S. (2002). Classical tidal harmonic analysis including error estimates in MATLAB using T_TIDE, *Computers and Geosciences,* 28 (8), 929–937.

Sharp, J. H. (1974). Improved analysis for particulate organic carbon and nitrogen from seawater. *Limnology and Oceanography,* 19, 984-989.

Strickland, J. D. H., and Parsons, T. R. (1972). Determination of ammonia. *In* A Practical Handbook of Seawater Analysis, Fisheries Research Board of Canada, Bulletin 167 (Second Edition), 310 pages.

Twomey, E.R. and Signell, R.P. (2013). Construction of a 3-arcsecond digital elevation model for the Gulf of Maine, U.S. Geological Survey Open-File Report 2011–1127, 24 pp, https://pubs.usgs.gov/of/2011/1127/.

USACE, (2011). Cape Cod Canal Condition Survey, Tech. Rep. 11-1154, U.S. Army Corps of Engineers, New England District, Concord, MA.

Williams, T., and Neill, C. (2014). Buzzards Bay Coalition Citizens' Water Quality Monitoring Program, "Baywatchers", 5 Year Quality Assurance Project Plan.

City of New Bedford NPDES Permit dated September 26, 2008.

Town of Dartmouth NPDES Permit dated June 19, 2009.

Table 1. Statistics of total nitrogen (TN) concentrations measured in Buzzards Bay and West Falmouth Harbor. Values for the Snug Harbor Sentinel Station are in bold.

Location	Identifier (Figs 1 and 2)	Period of Meas.	Mean TN Conc. mg/L	Stand. Dev. mg/L
BBC Mid-Bay Buoy	BBC	2007-2017	0.266	0.049
Nashawena Road	WF1	2016-2018	0.856	0.166
Harbor Head	WF2	2016-2018	0.507	0.091
Chapoquoit Basin	WF3	2016-2018	0.425	0.077
Inner W. Fal. Harbor	WF4	2016-2018	0.477	0.104
Snug Harbor Sentinel Sta.	**WF5**	**2016-2018**	**0.553**	**0.126**
Outer W. Fal Harbor	WF6	2016-2018	0.374	0.069
Outer W. Fal. Harbor	WF7	2016-2018	0.370	0.076

Table 2. Comparison of measured concentrations of total nitrogen (TN) with projected effluent TN concentrations discharged at outfall sites D1 and D2 (Figure 3).

Site or Region	Measured TN Conc. mg/L	Modeled TN Conc. – D1 mg/L	Modeled TN Conc. – D2 mg/L
BBC Mid-Bay Buoy (Fig 1)	0.266	0.00024	0.00025
Outer Harbor (Fig. 16)	0.372	0.0013	0.0011
Chapoquoit Basin (Fig. 16)	0.466	0.0010	0.0009
Snug Harbor (Fig. 16)	0.629	0.0012	0.0011

Table A2-1. Comparison of Proposed to Existing Effluent Outfalls in Buzzards Bay.

	Proposed Outfalls (Figure 3)		**Existing Outfalls (Figure A2-1)**	
	D1	**D2**	**New Bedford**[1]	**Dartmouth**[2]
Treatment type	Tertiary	Tertiary	Advanced secondary	Secondary
TN limit	3 mg/L	3 mg/L	None, report only	None, report only
Proposed/permitted flow rate	4 MGD	4 MGD	30 MGD	4.2 MGD
Distance from shore	4380 ft	5250 ft	3050 ft from Clark pt (4600 ft from beaches)	2550 ft (to Mishaum Pt.)
Bottom depth	52 ft	52 ft	~22 ft	~ 10 ft
Industrial Input	No	No	Allowed	No
Storm water Input	No	No	Allowed	No

1

2

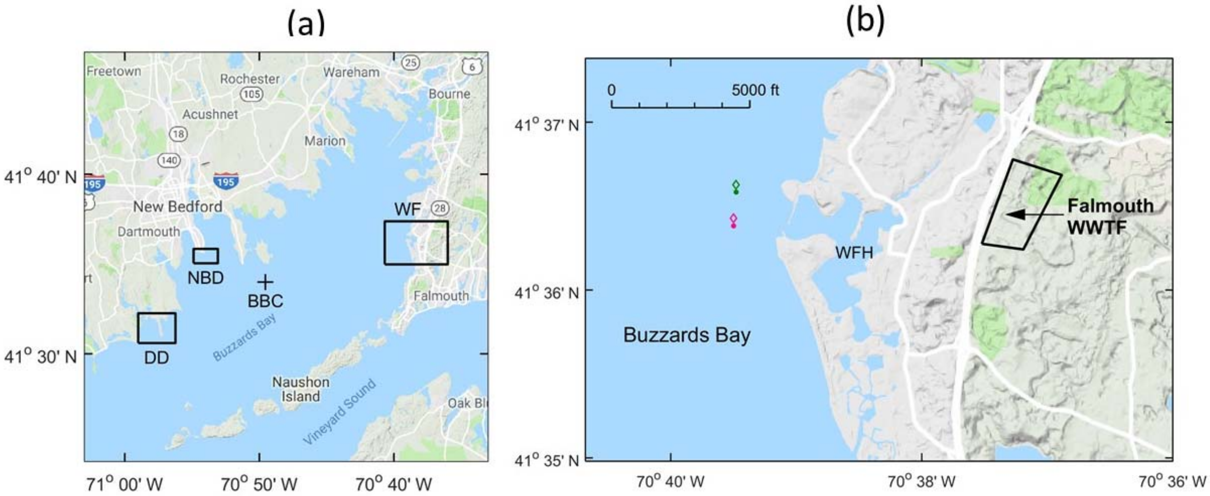


Figure 1. (a) A large-scale view of Buzzards Bay, MA showing the location of the Buzzards Bay Coalition mid-bay buoy (BBC). The box labeled NBD encloses the city of New Bedford MA waste water discharge off of Clark Point, New Bedford, whereas the box labeled DD encompasses the waste water outfall of the Town of Dartmouth MA. The box labeled WF encloses this study's area of interest. As shown in (b), this area includes the waste water treatment facility (WWTF) currently used by the town of Falmouth MA for in-ground discharge of treated effluent as well as West Falmouth Harbor (WFH) and the region of Buzzards Bay considered for open water effluent discharge (south of the entrance to WFH).

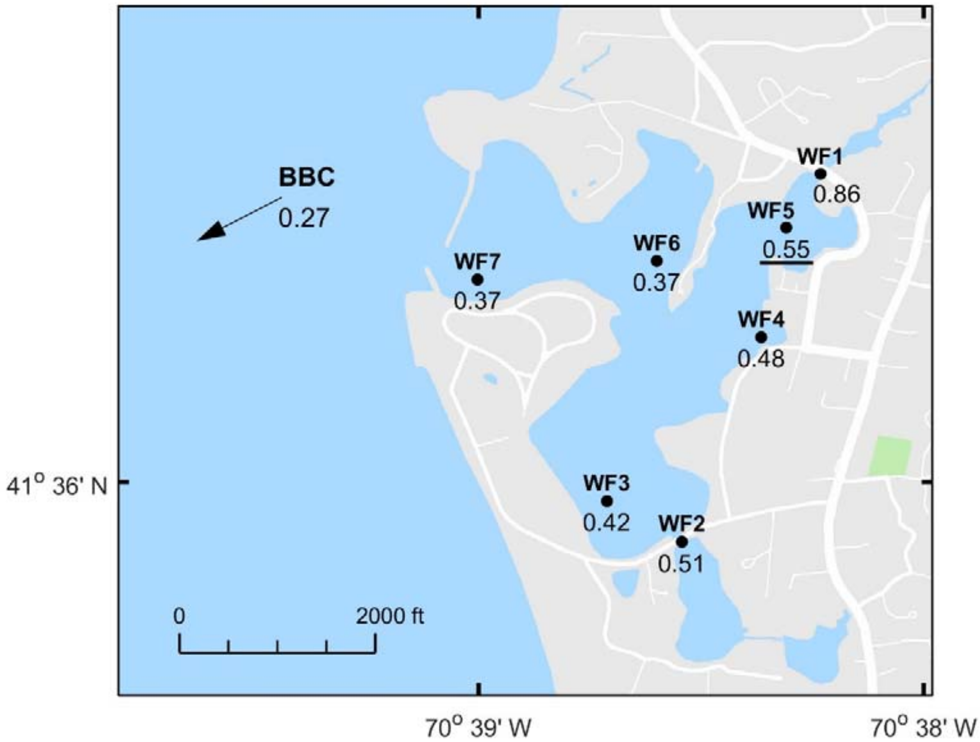


Figure 2. Averaged (over 2016-2018) concentrations of TN (mg/L) measured at various stations in West Falmouth Harbor as part of the Mass. Estuaries Project (see also Table 1). Station names are listed above each station location. The concentration at the Snug Harbor sentinel station is underlined. Also shown is the averaged (2007-2017) TN concentration at the Buzzards Bay Coalition mid-bay buoy (BBC, Figure 1)

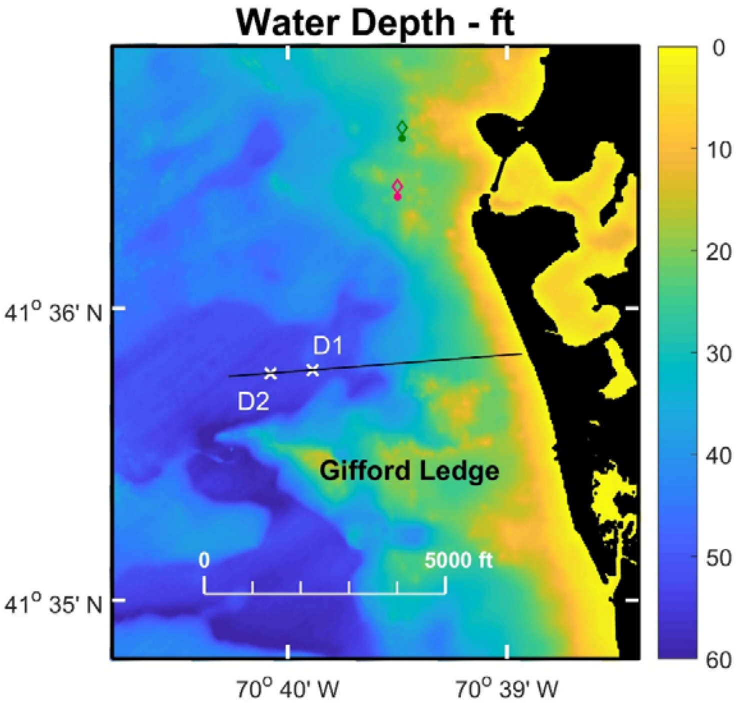


Figure 3. High-resolution bathymetry of the area of eastern Buzzards Bay considered for open water effluent discharge. The proposed discharge sites, D1 and D2, are situated within a basin with a mean depth of roughly 52 ft. Also shown are the locations of navigation buoys marking the entrance to West Falmouth Harbor.

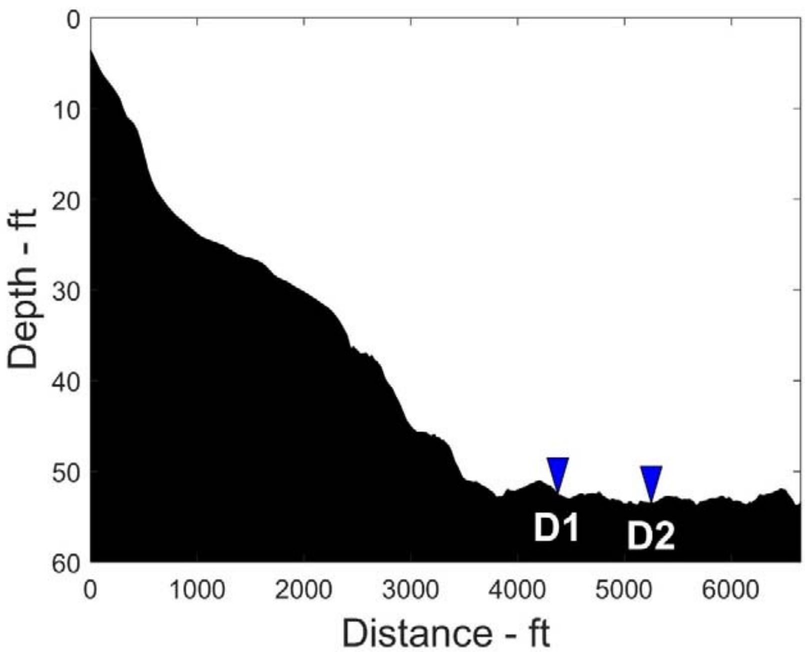


Figure 4. Profile of bottom depth along a line (shown in Figure 3) that extends offshore from West Falmouth and intercepts the proposed discharge sites D1 and D2.

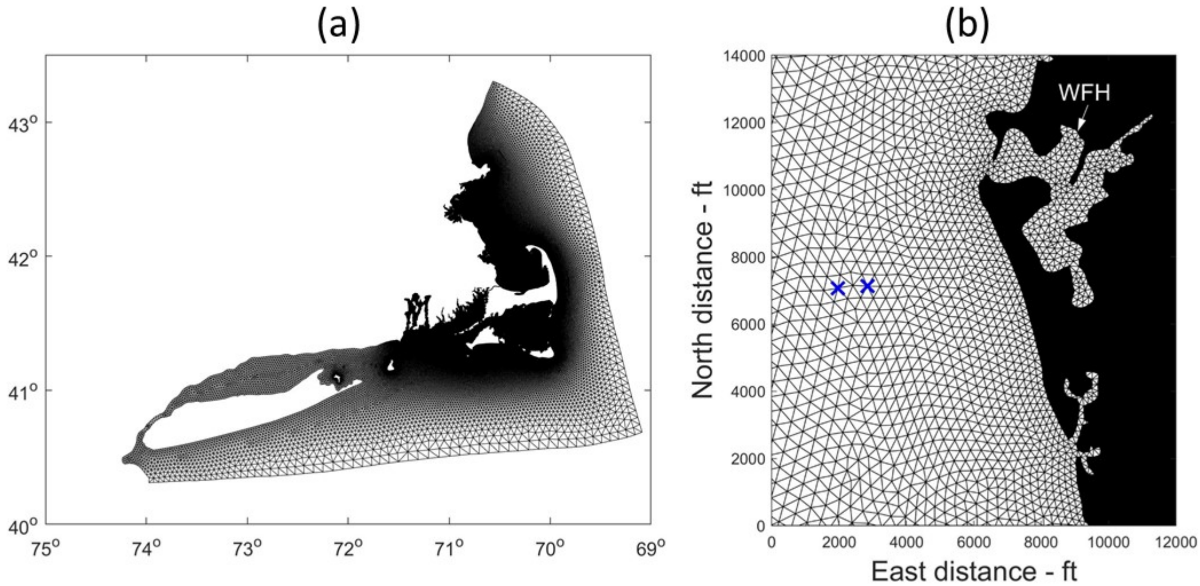


Figure 5. (a) Grid cells of the Southeastern Mass. hydrodynamic model (SEMASS-FVCOM) used to generate the velocity fields used in this study. The grid contains 284305 cells with increased resolution (smaller cell size) in the coastal zone. (b) A view of the SEMASS grid in our study region. The cells in the area of the proposed open water discharge sites, blue x's, measure roughly 350 ft in the N-S and E-W directions. The near-shore cells and those in West Falmouth Harbor (WFH) have horizontal dimensions of 100-200 ft.

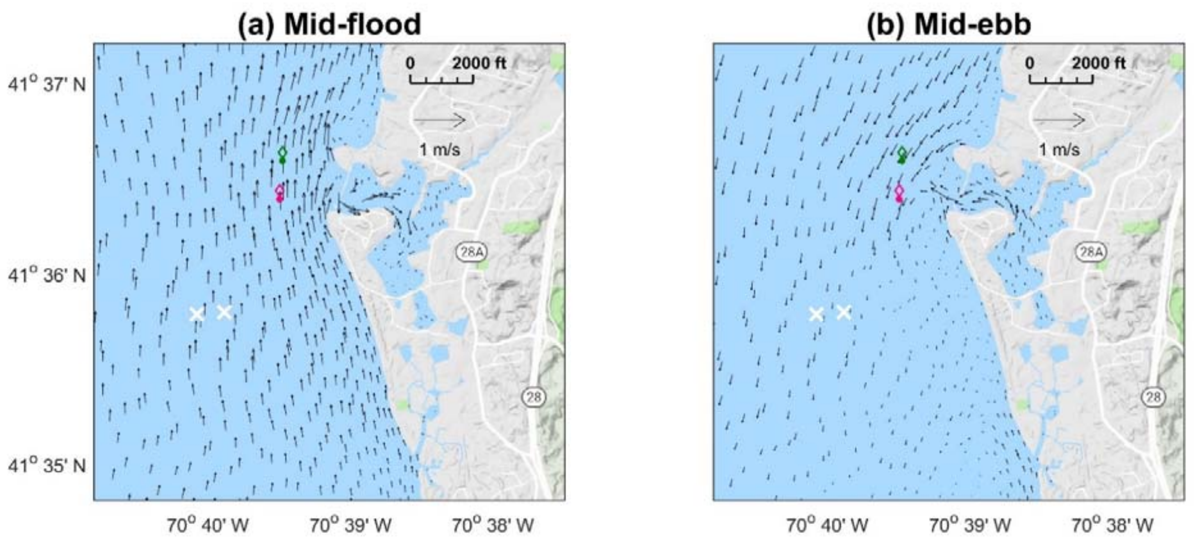


Figure 6. Sample fields of surface current (from 14-15 July 2015) during mid-flood (a) and mid-ebb (b) flow in West Falmouth Harbor. The proposed discharge sites are marked with white x's.

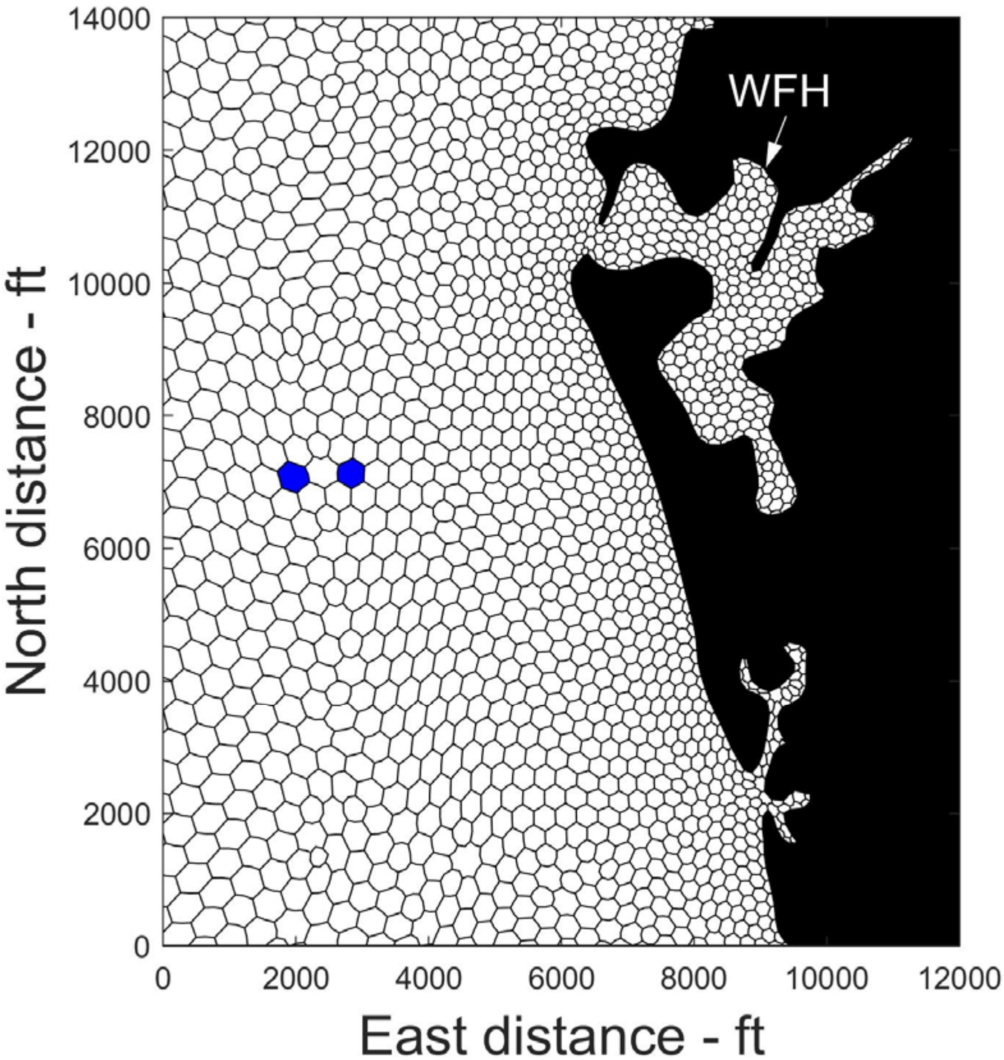


Figure 7. Cells of the water quality model grid in our region of interest. This grid is linked (but not identical) to the grid of the hydrodynamic model (Figure 5), which supplies velocities for the effluent transport simulations. The cells containing the proposed open water discharge sites, shaded in blue, measure roughly 430 ft in the N-S and E-W directions. The near-shore cells and those in West Falmouth Harbor (WFH) have horizontal dimensions of 100-200 ft.

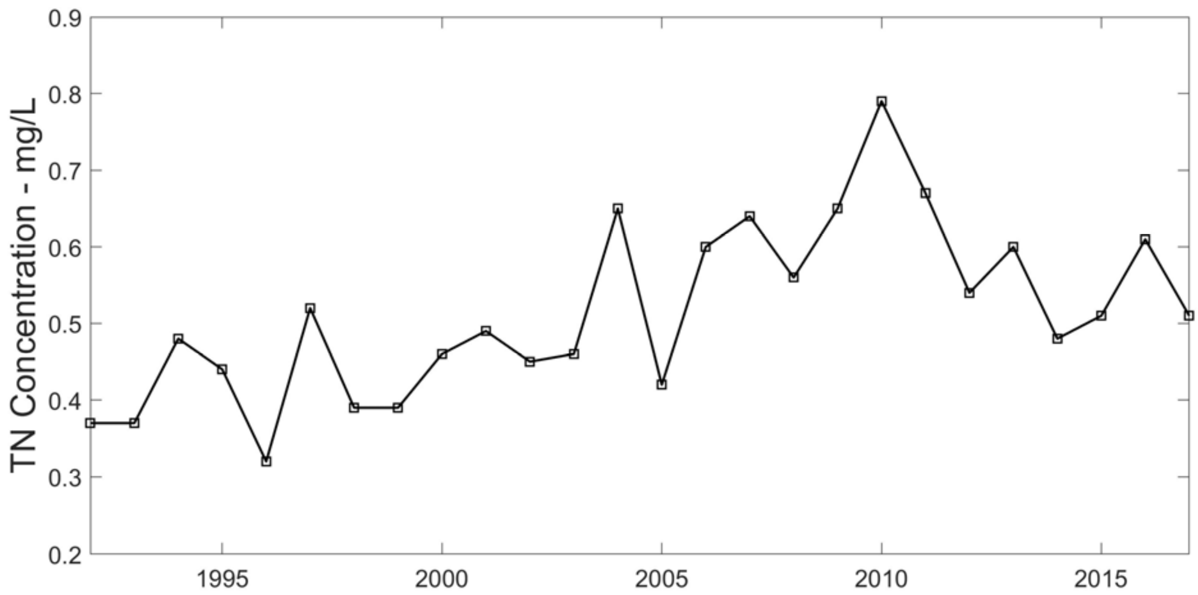


Figure 8. Yearly averaged TN concentrations at the Snug Harbor Sentinel (WF5 in Figure 2) in West Falmouth Harbor.

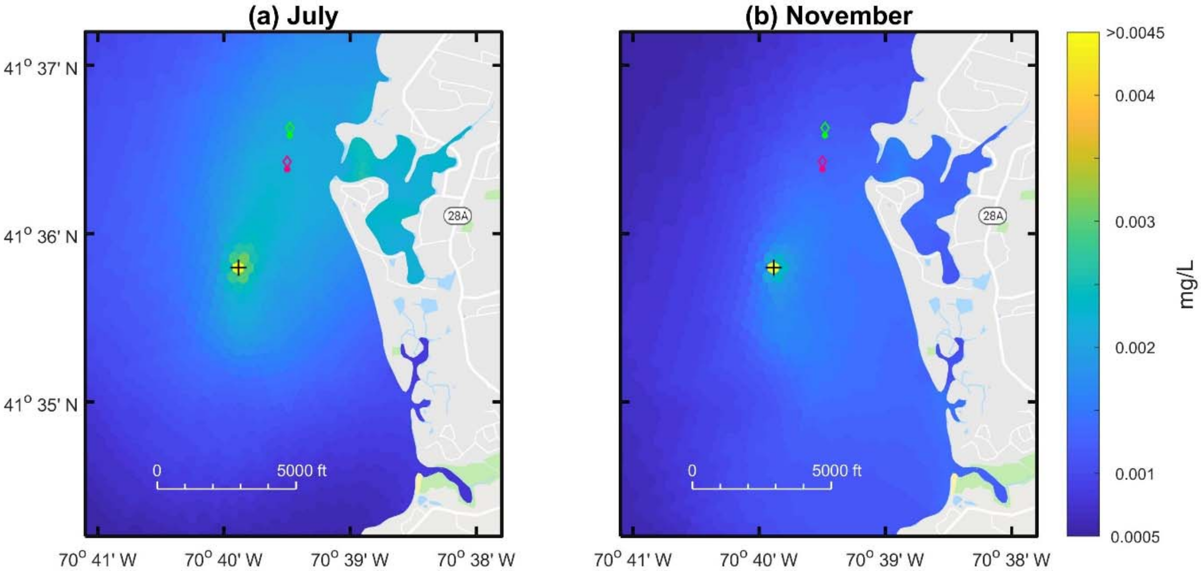


Figure 9. Fields of vertically averaged concentration of effluent TN discharged at site D1 (Figures 3 and 4) and averaged over July (a, when winds are predominately from the SW) and November (b, with predominately N winds). In both fields, the averaged effluent TN exceeds 0.0031 mg/L only in the cell containing the discharge (marked by a cross).

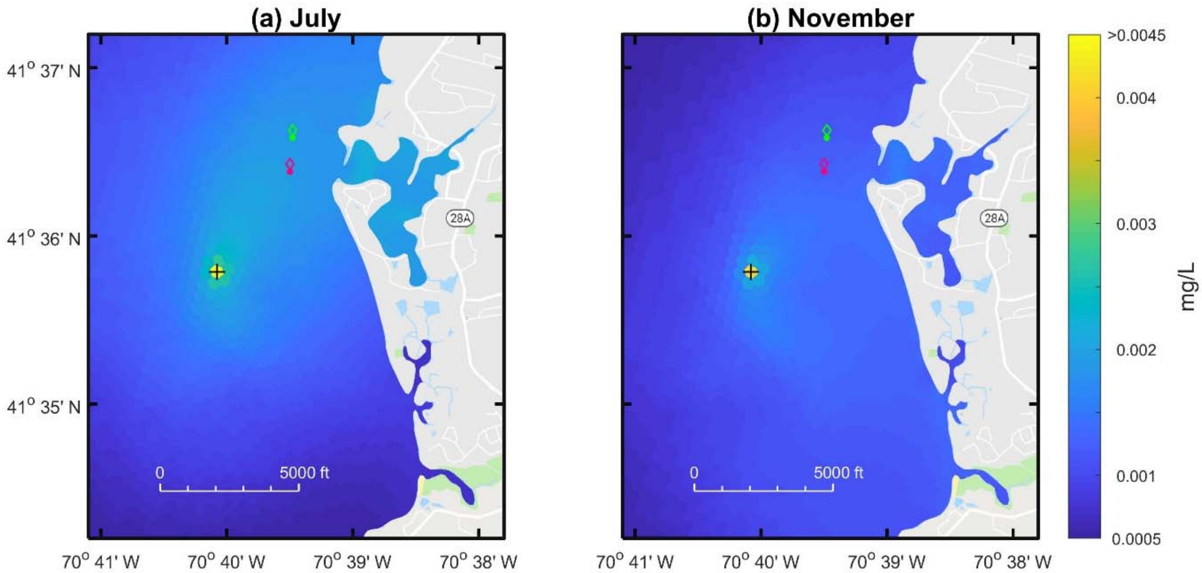


Figure 10. Same as Figure 9 except showing fields of vertically averaged concentration of effluent TN discharged at site D2 (Figures 3 and 4).

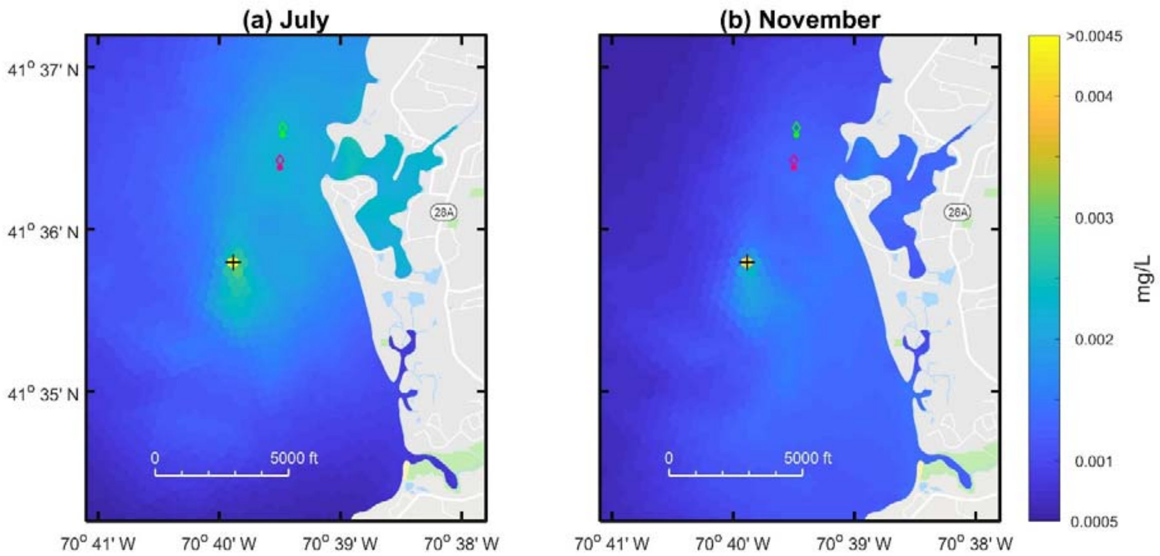


Figure 11. Fields of surface concentration of effluent TN discharged at site D1 (Figures 3 and 4) and averaged over July (when winds are predominately from the SW) and November (with predominately N winds). In both fields, the surface effluent TN exceeds 0.003 mg/L only in the cell containing the discharge (marked by a cross).

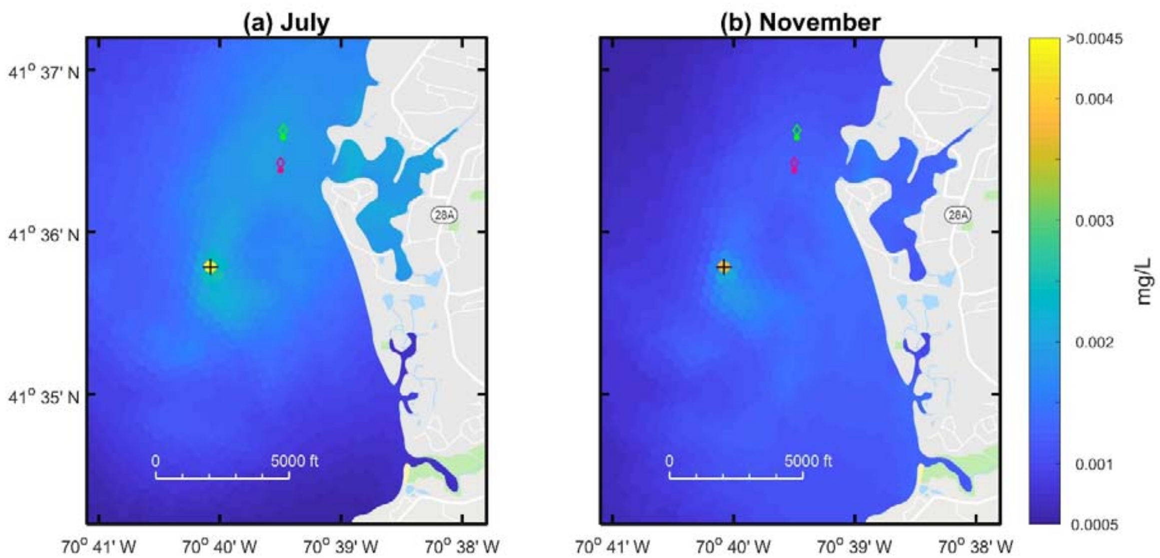


Figure 12. Same as Figure 11 except showing fields of the surface concentration of effluent TN discharged at site D2 (Figures 3 and 4).

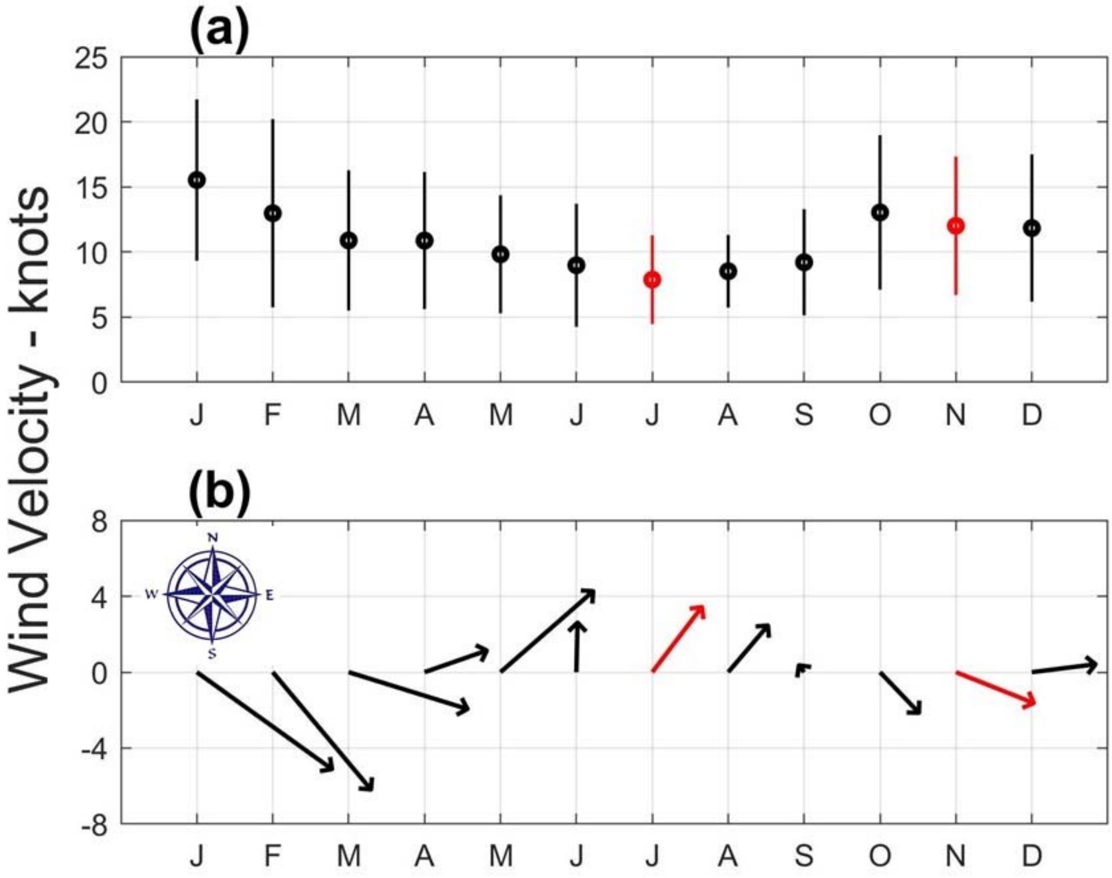


Figure 13. Graphical representation of the monthly statistics of the model winds off of West Falmouth (in the region of the proposed open water outfalls). The filled circles in (a) mark the monthly mean wind speeds, while the lines bracketing each circle span the range of ± one standard deviation about the mean winds. Shown in (b) are the monthly mean wind vectors in a geographic coordinate system. Statistics of the winds of July and November (the months of the mean TN concentration fields of Figures 9-12) are shown in red.

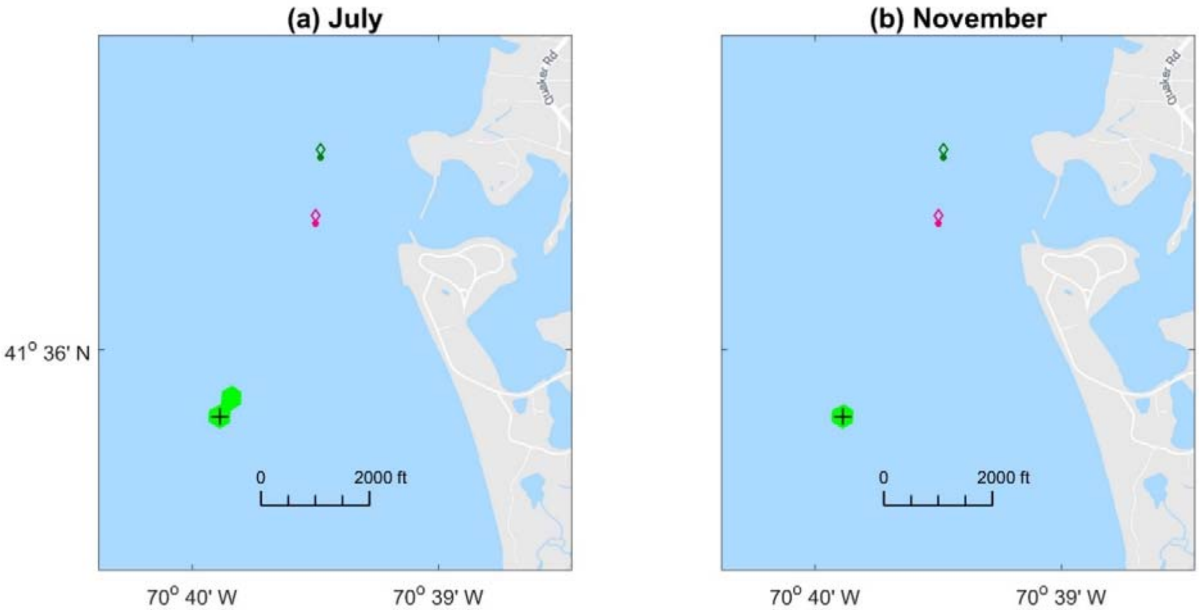
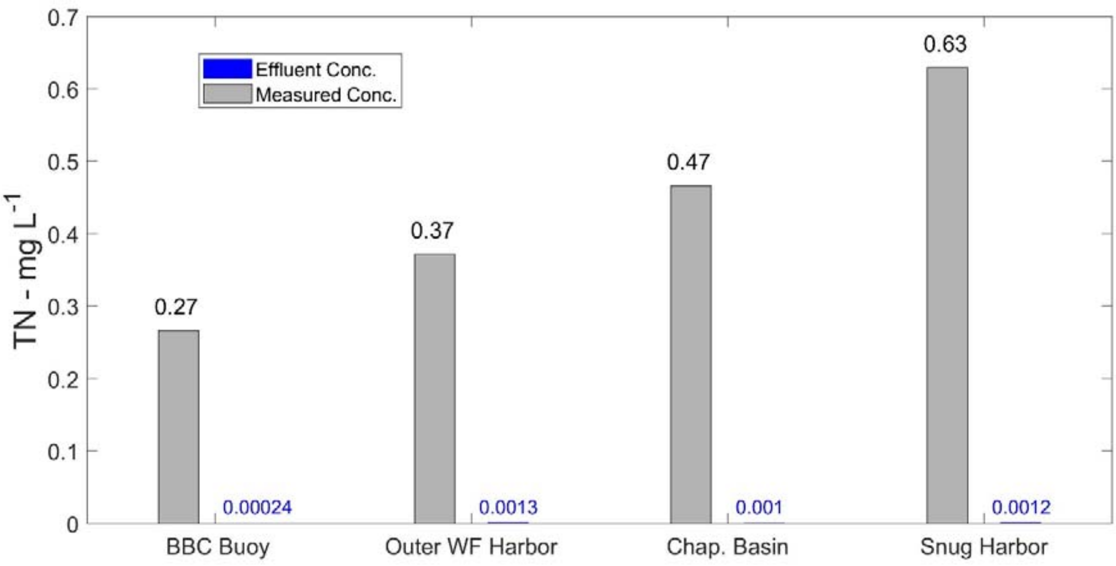


Figure 14. The green-shaded areas depict the model cells over which the dilution factor (the ratio of discharged to vertically averaged effluent TN concentration) is equal to or less than 1000:1 for the TN concentration fields of July (a) and November (b). Note that the mean dilution factor for these months exceeds 1000:1 for all but 1-2 cells at or adjacent to the discharge, which is the case for all months.



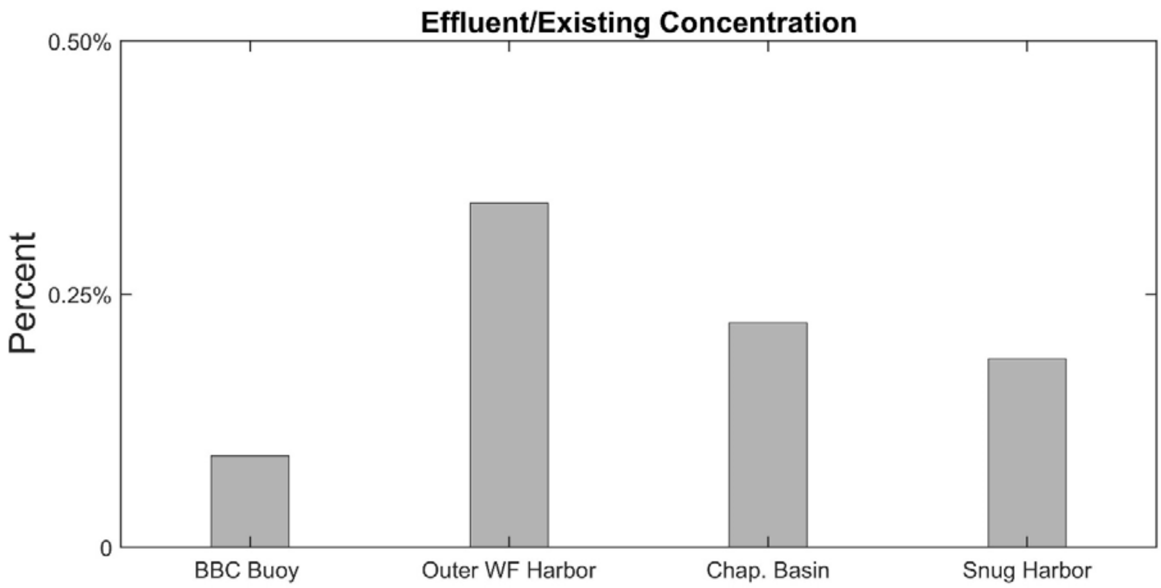


Figure 16. Ratio of the modeled effluent TN to the measured TN at the BBC buoy (Figure 1) and the indicated regions in West Falmouth Harbor (Figure 17). The ratios (in percent) were determined from the TN values shown in Figure 15.

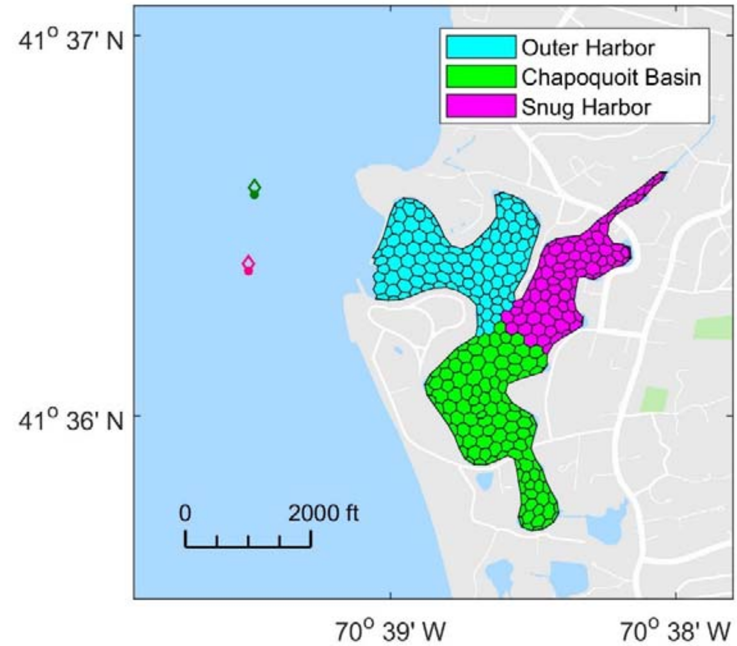


Figure 17. Water quality model cells in West Falmouth Harbor divided into three regions: Outer Harbor, Chapaquoit Basin and Snug Harbor.

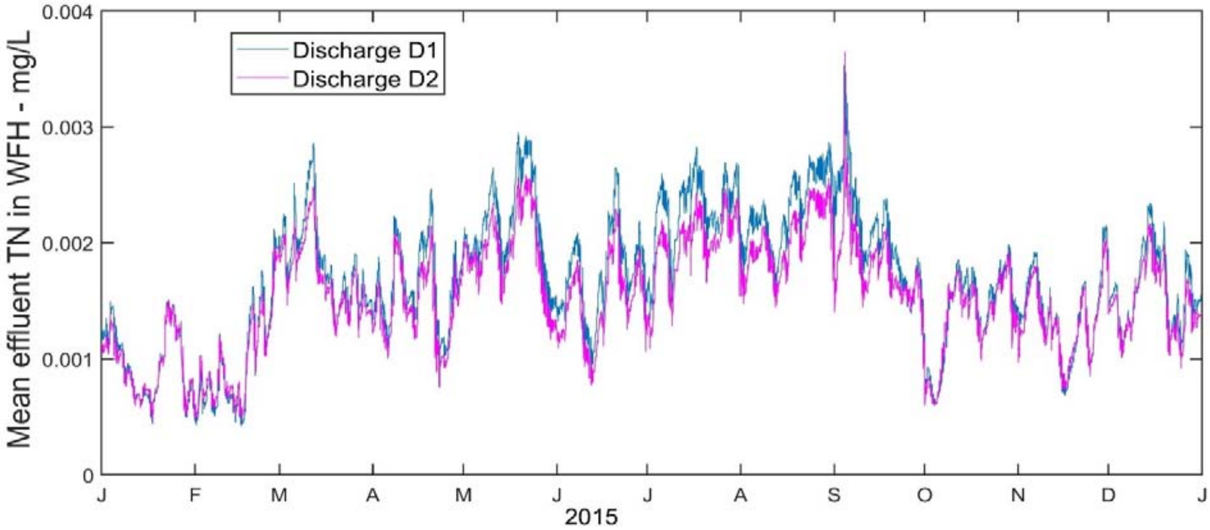


Figure 18. Year-long time series of the mean concentration of effluent TN within West Falmouth Harbor (encompassing all of the colored cells of Figure 17) resulting from effluent discharge from site D1 (blue line) and site D2 (magenta). Note that except for a brief period in early September, the mean effluent TN concentration is < 0.003 mg/L, close to the TN detection limit (see Methods).

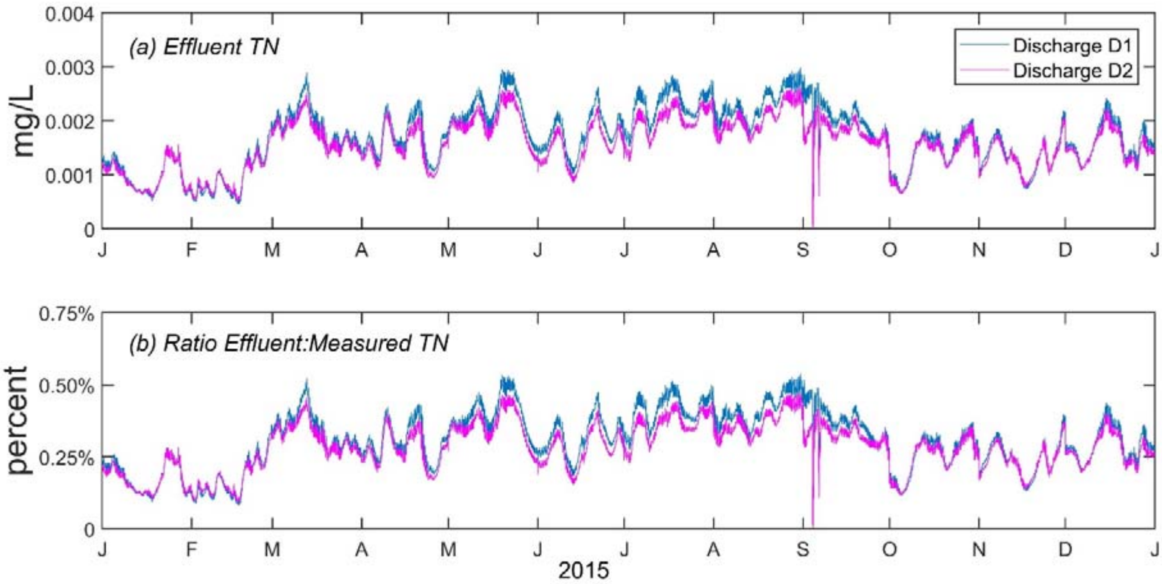


Figure 19. (a) Year-long time series of the vertically averaged effluent TN concentration at the Snug Harbor sentinel station (Figure 2) resulting from effluent discharge from site D1 (blue line) and site D2 (magenta). (b) Ratio (in percent) of the model effluent TN concentration to the mean TN concentration at the Snug Harbor sentinel station (Table 1). Note that both panels show little difference in the impact of discharge at D1 relative to D2.

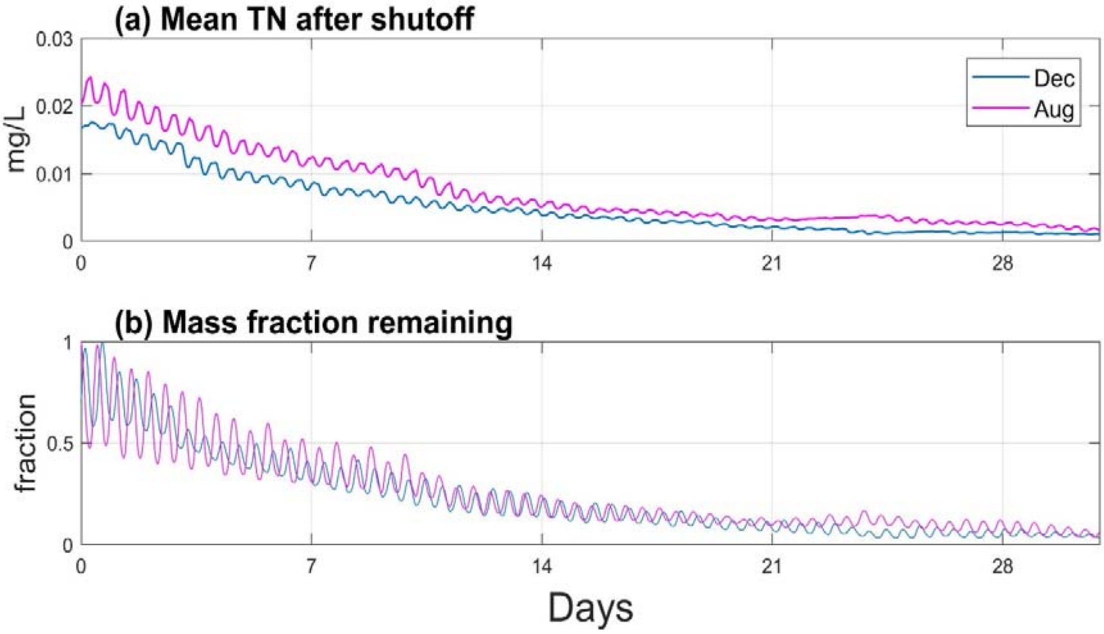


Figure 20. The results of model runs aimed at determining the flushing time of West Falmouth Harbor. (a) The mean concentration of effluent TN in West Falmouth Harbor after cessation of effluent discharge at the beginning of August (magenta) and December (blue). The starting TN fields were taken from simulations (in July and November) with discharge at site D1 set at a rate of 4 MGD and with TN concentration of 35 mg/L. (b) The fraction of TN mass in West Falmouth Harbor relative to the maximum mass after discharge cessation. For both months, the TN mass in West Falmouth Harbor is reduced by a factor of 10 in roughly 3 weeks after the cessation of discharge.

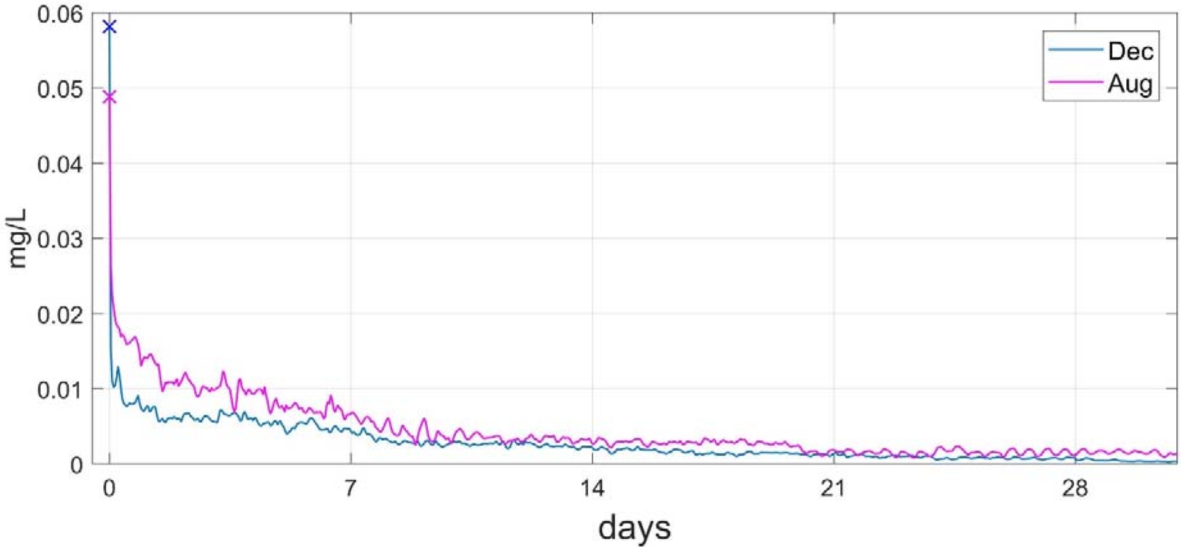


Figure 20. The vertically averaged effluent TN concentration at outfall D1 after cessation of effluent discharge at the beginning of August (magenta) and December (blue). The starting TN fields were taken from simulations (in July and November) with discharge at D1 set at a rate of 4 MGD and with TN concentration of 35 mg/L. Apparent is a rapid initial decline in effluent TN concentration immediately after the cessation of discharge. This is followed by a more gradual decline that brings the TN concentration to order 0.002 mg/L two weeks after the termination of discharge.

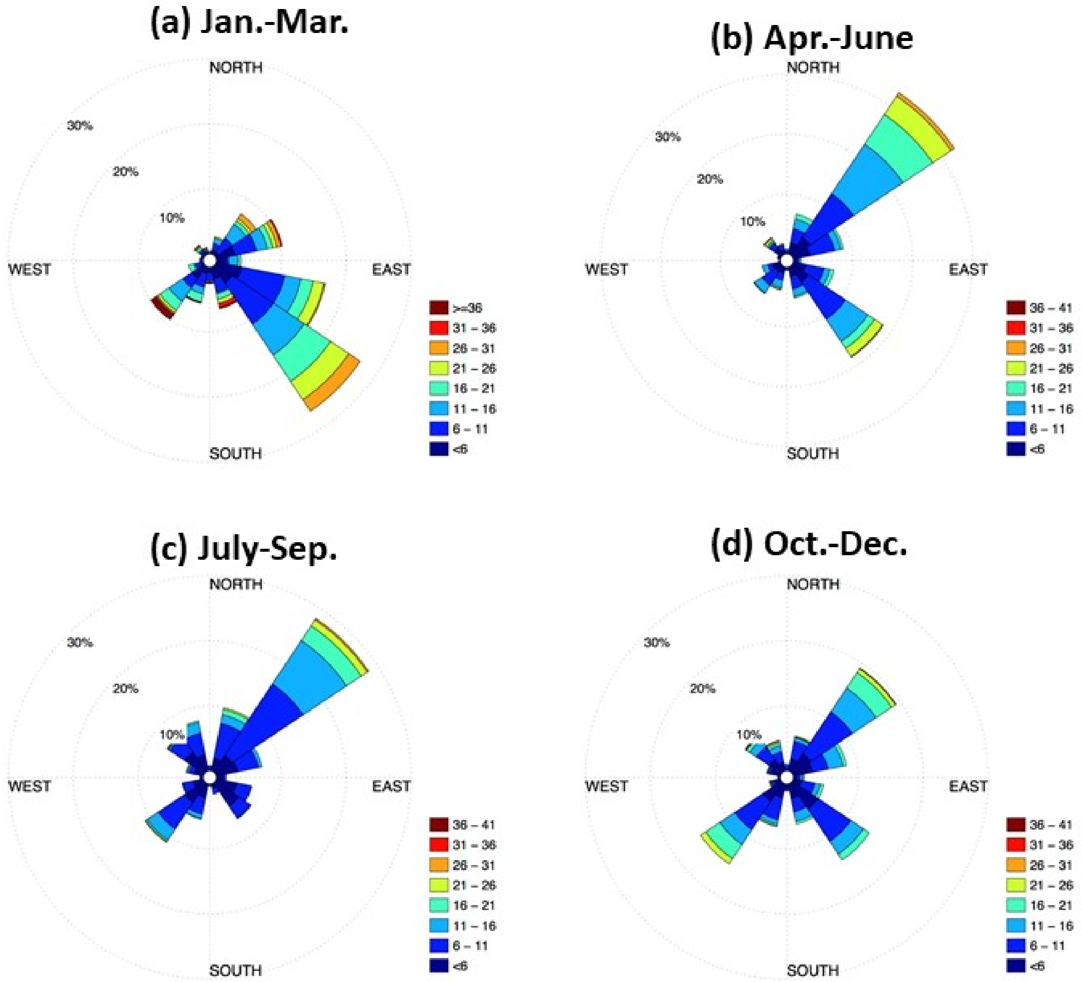


Figure A1-1. Graphical representation of the seasonal variation of the modeled wind field (knots) in eastern Buzzards Bay.

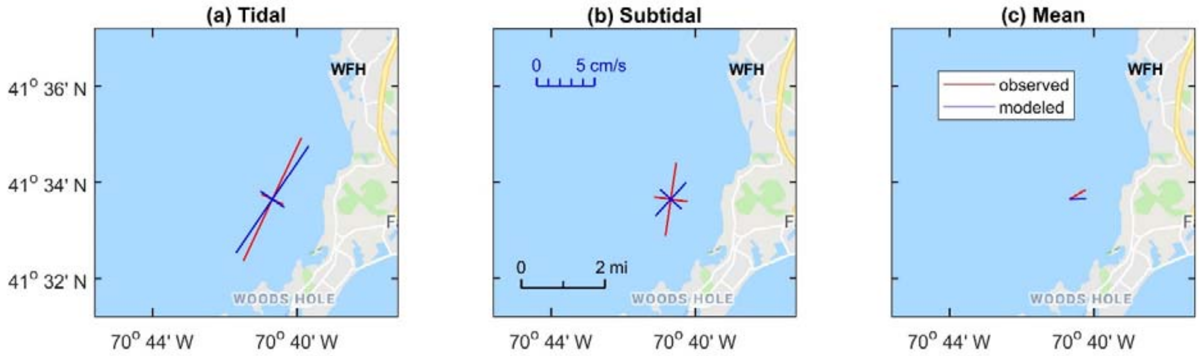


Figure A1-2. Comparison of the statistical properties of the vertically averaged modeled and measured currents at site in eastern Buzzards Bay roughly 3 nautical miles SSW of West Falmouth Harbor. The measured currents represented by the statistics are from a current meter (ADCP) deployed by the USGS over 13 August – 9 November 2009. The modeled currents are from the model cell with the center closest to the current meter location and from the period 13 August – 9 November 2015 (the year of the model simulations). Demarked by the lines in (a) and (b) are the standard deviations along the principal axes of the tidal and sub-tidal velocity signal [on the scale shown in (b)]. The overall mean velocities are shown in (c).

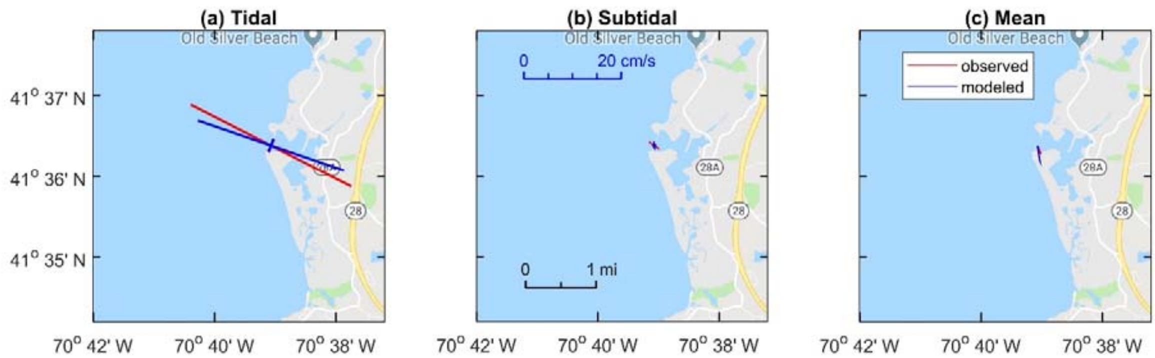


Figure A1-3. Same as Figure A1-2 except comparing the statistical properties of the vertically averaged measured (2 July – 8 September 2010) and modeled (2 July – 8 September 2015) currents at the entrance to West Falmouth Harbor.

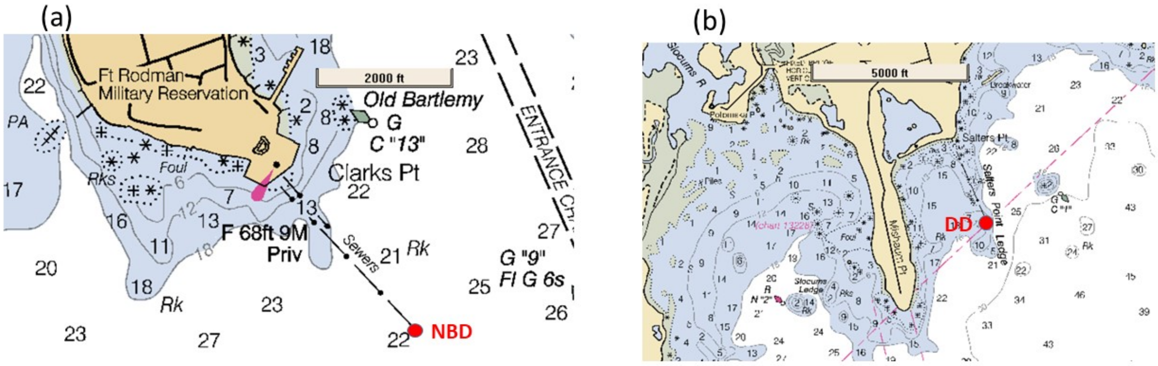


Figure A2-1. Portions of navigation charts (enclosed by the boxes in Figure 1a) of the region near Clarks Point, New Bedford MA (a) and Mishaum Point Dartmouth MA (b) (soundings in feet). The red dots in (a) and (b) mark the locations of the open water discharge of the City of New Bedford (NBD) and Town Dartmouth (DD), respectively.

TASA TM-7

April 11, 2019

To:	Town of Falmouth, MA	Ref. No.:	11153041
From:	Anastasia Rudenko, P.E., BCEE, ENV SP	Tel:	774-470-1637
	J. Jefferson Gregg, P.E., BCEE		774-470-1640
cc:	File; Project Team		

Subject: **South Coast Embayments – Preliminary Evaluations and Notice of Project Change Project**

Teaticket / Acapesket Study Area Conceptual Layouts and Preliminary Cost Estimates Evaluation – Technical Memorandum No. 7 (TASA TM-7)

1. Purpose of Memo

The purpose of TASA Technical Memorandum 7 (TASA TM-7) is to summarize the four conceptual layouts that were developed for the Teaticket/Acapesket Study Area (TASA), which includes portions of the Great and Green Pond watersheds.

1.1 References, Regulations, and Design Guidelines

The references, regulations, and design guidelines listed below were used in the development of this memorandum. Abbreviations (in parenthesis) are used to reference the following documents throughout the remainder of the memorandum.

<u>References:</u>

- TASA TM No. 6 – South Coast Embayments Preliminary Evaluations and Notice of Project Change Project – Draft Outfall Conceptual Cost Evaluation, prepared by GHD, dated November 2018 (TASA TM-6)
- TASA TM No. 5 – South Coast Embayments Preliminary Evaluations and Notice of Project Change Project – Draft Hydraulic Load Tests at the Augusta Parcel an Allen Parcel, prepared by GHD, dated November 2018 (TASA TM-5)
- TASA TM No. 4 – South Coast Embayments Preliminary Evaluations and Notice of Project Change Project – Draft WWTF Evaluation, prepared by GHD, dated October 2018 (TASA TM-4)
- TASA TM No. 3 – South Coast Embayments Preliminary Evaluations and Notice of Project Change Project – Draft Teaticket / Acapesket Area Discharge Technologoes Evaluation – Technical Memorandum No. 3 (TASA TM-3) – Rev 2, dated November 2018
- TASA TM No. 1 – South Coast Embayments Preliminary Evaluations and Notice of Project Change Project – Draft Service Area, Flow and N Load Evaluation, prepared by GHD, dated May 2018 (TASA TM-1)

GHD
1545 Iyannough Road Hyannis Massachusetts 02601 USA
T 774 470 1630 **F** 774 470 1631 **W** www.ghd.com

- 'Property Type Classification Codes Non-arm's Length Codes and Sales Report Spreadsheet Specifications' prepared by the Bureau of Local Assessment, revised May 2018 (Land Use Codes)
- Recharge Beds 14 and 15 Operations and Maintenance Manual, prepared by GHD, dated June 2017
- Modified Individual Groundwater Discharge Permit No. 168-5, effective date December 22, 2015 (2015 Permit)
- Comprehensive Wastewater Management Plan and Final Environmental Impact Report and Targeted Watershed Management Plan – Little Pond, Great Pond, Green Pond, Bournes Pond, Eel Pond, and Waquoit Bay Watersheds and Recommendations for West Falmouth Harbor Watershed, prepared by GHD, dated September 2013. (2013 CWMP/FEIR/TWMP)
- Little Pond, Great Pond, Green Pond, Bournes Pond, Eel Pond, and Waquoit Bay Watersheds Alternatives Screening Analysis Report, prepared by GHD (formally Stearns & Wheler, LLC), dated November 2007 (2007 ASAR)
- Little Pond, Great Pond, Green Pond, Bournes Pond, Eel Pond and Waquoit Bay Watersheds Needs Assessment Report, prepared by Stearns & Wheler, dated October 2007 (2007 NAR)
- Falmouth Wastewater Treatment Facility Operation & Maintenance Manual – Draft, prepared by MaGuire Group Inc., dated 2007 (2007 Draft WWTF O&M Manual)
- Final Great, Green, and Bournes Pond Embayment Systems Total Maximum Daily Loads for Total Nitrogen (Report #96-TMDL-6 Control #181.0), prepared by the Commonwealth of Massachusetts Executive Office of Environmental Affairs, Massachusetts Department of Environmental Protection, Bureau of Resource Protection and Division of Watershed Management, dated April 6, 2006 (TMDL Report).
- Linked Watershed-Embayment Model to Determine Critical Nitrogen Loading Thresholds for Great/Perch Pond, Green Pond and Bournes Pond, Falmouth Massachusetts; prepared by Howes B., J.S Ramsey, S.W. Kelley, R. Samimy, D. Schlezinger, E. Eichner. Massachusetts Estuaries Project, Massachusetts Department of Environmental Protection. Dated April 2005 (MEP Report).

Regulations:

- 314 CMR 5.00 – Groundwater Discharge Permits, effective December 2, 2016 (314 CMR 5.00)
- 314 CMR 4.00 – Massachusetts Surface Water Quality Standards, effective December 6, 2013 (314 CMR 4.00)
- 314 CMR 20.00 – Reclaimed Water Permit Program and Standards, effective March 20, 2009 (314 CMR 20.00)
- Massachusetts Ocean Sanctuaries Act (M.G.L. c132A) (Massachusetts Ocean Sanctuaries Act)

Design Guidelines:

- Guidelines for the Design, Construction, Operation, and Maintenance of Small Wastewater Treatment Facilities with Land Disposal, prepared by MassDEP, as revised in July 2018. (2018 MassDEP Small WWTF Design Guidelines)
- New England Interstate Water Pollution Control Commission, TR-16: Guides for the Design of Wastewater Treatment Works, 2011 Edition as revised in 2016. (TR-16)
- 'Title 5 Pressure Distribution Design Guidance', prepared by MassDEP, dated 2002

2. Conceptual Layouts

At the November 2018 progress meeting four conceptual layouts were selected by the Town and Water Quality Management Committee (WQMC) Working Group for conceptual layout development and for the development of conceptual cost estimates. Each conceptual layout identified a sewer service area, collection/transmission system (and lift station sites), effluent discharge system, and recommended wastewater treatment facility (WWTF) capacity upgrades. The sewer service area, collection/transmission system, and WWTF upgrades are the same in each of the four conceptual layouts. Therefore, each layout is simply differentiated by the location and method of effluent disposal. Layouts for each component identified above were developed on a conceptual basis with limited field work and no land survey. The conceptual layouts are subject to change as the design advances, based on the findings of additional field investigations and land survey.

The development of each conceptual layout component is described in this section.

2.1 Sewer Service Area Conceptual Layout

The Teaticket/Acapesket Study Area (TASA) includes properties in both the Great Pond watershed and the Green Pond watershed as well as parcels that flush directly into Vineyard Sound. As outlined in TASA TM-1, TASA was divided into three "sub-areas" for greater flexibility in evaluation (Figure 1).

- Sub-Area 1: This sub-area includes the portion of TASA west of Great Pond. The sub-area is primarily comprised of residential parcels in the Great Pond watershed and also includes commercial, industrial, and municipal parcels along Route 28/Teaticket Highway.
- Sub-Area 2: This sub-area includes the portion of TASA north of Emerson Street and east of Great Pond. The sub-area is primarily comprised of residential parcels in the Great Pond and Green Pond watersheds. The sub-area also contains commercial parcels primarily bordering Route 28/East Falmouth Highway.
- Sub-Area 3: This sub-area includes the portion of TASA south of Emerson Street and is primarily comprised of residential parcels. A portion of the parcels in Sub-Area 3 are located in the Great Pond and Green Pond watersheds. The MEP report indicates that the remainder of the parcels in this area flush directly to Vineyard Sound. The entirety of Sub-Area 3 is located in a low elevation within the 100-year flood zone. The impact of sea level rise in this area is also a concern. A high percentage of the parcels in Sub-Area 2 are seasonal properties.

The sewer service area for the conceptual layouts includes only Sub-Area 1 and Sub-Area 2. Sub-Area 3 was eliminated from the sewer service area due to the high number of properties in this sub-area that flush directly to Vineyard Sound (as opposed to into a nutrient-impaired estuary), the high percentage of seasonal properties and the low elevation within the 100 year flood zone.

The overall number of parcels in the sewer service area is summarized in Table 1. Parcels are subdivided into developed parcels, developable parcels, undevelopable parcels and conservation land. It was assumed that parcels classified as conservation land would not generate wastewater flow and therefore are not anticipated to connect into the collection system. Although the Land Use Code for the “Shorewood Parcel” (0 Shorewood Drive) indicates it is a vacant municipal property, this parcel was included in the count of conservation land since it has a conservation restriction preventing development. It was assumed that “undevelopable” land may be re-classified by the Town in the future, and therefore could have the potential to connect into the available collection system in the future.

Table 1 – Number of Parcels in the proposed TASA Sewer Service Area[1]

Type of Property	Number of Parcels
Developed[5]	1668
Developable[2]	72
Undevelopable[3]	51
Conservation Land[4]	1
Total	1,792

Notes:

1. Source: Town of Falmouth Assessors data, dated 2018.
2. Parcels with Land Use Codes 130 (Residential – Developable Land), 131 (Residential – Potentially Developable Land), 390 (Commercial – Developable Land).
3. Parcels with Land Use Code 132 (Residential – Undevelopable Land), 392 (Commercial – Undevelopable Land), 442 (Industrial – Undevelopable Land).
4. Conservation Land– Parcels with Land Use Codes 932 (Vacant, Conservation), 950 (Vacant, Conservation Organizations), 982 (Vacant, Conservation, Other City of Town)
5. Parcels with all other Land Use Codes.

2.2 Collection/Transmission System Conceptual Layout

SewerCAD was used to develop the TASA conceptual collection system layout. Two initial alternatives (Alternatives 1 and 2) were presented in TASA TM-3. In order to reduce the overall number of lift stations a subsequent alternative (Alternative 3) was developed which considered sewer easements through developable parcels. At the November 2018 progress meeting Alternative 3 was chosen for the conceptual layout and preliminary cost estimate development. The conceptual layout is shown in Figure 2.

The conceptual collection system is divided into eleven sewersheds and configured to maximize the number of properties served by gravity sewer. In the conceptual layout nine sewersheds are serviced by a new lift station and two sewersheds connect into existing lift stations (Alphonse Street LS and Spring Bars Road LS).

Flow from the nine sewersheds with new lift stations is conveyed to a single booster station (the Brick Kiln Road Booster Lift Station) by individual (single) force mains. Flow is proposed to be conveyed from the Brick Kiln Road Booster Lift Station to the Falmouth WWTF through dual 10-inch HDPE force mains along Brick Kiln Road to Gifford Street Extension to Locustfield Road and finally Blacksmith Shop Road, as shown in Figure 3.

The model was developed based on the assumption that properties or easements identified as proposed lift station sites are available. If, during detailed design, it is concluded that the locations are unavailable the proposed sewer alignment will be affected.

2.2.1 Connection Types

In order to maximize the amount of parcels potentially serviced through a gravity connection, a conceptual layout was developed which utilized sewer easements through currently undeveloped properties. In areas where gravity sewers are not feasible, due to topography, low pressure sewers are proposed. The number of properties connected to gravity sewer and low pressure sewer is summarized by sewershed in Table 2.

Table 2 – TASA Conceptual System Collection System Summary

Peninsula	Proposed Lift Station	Number of Parcels Connected to Gravity Sewer[1]	Number of Parcels Connected to Low Pressure Sewer	Total Number of Parcels Connected to the Collection System [2]
Maravista	Spring Bars Road LS (Connection to Existing LS)	87	43	130
	Alphonse Street LS (Connection to Existing LS)	57	13	70
	Subtotal	**144**	**56**	**200**
Teaticket	Saint Marks Road LS	187	12	199
	Falmouthport LS	27	0	27
	Teaticket Path LS	98	3	101
	Broken Bow Lane LS	48	0	48
	Village Common Drive LS	27	0	27
	Subtotal	**387**	**15**	**402**
Acapesket	Shorewood Drive LS	649	10	659
	Bridge Street LS	204	18	222
	East Falmouth Highway LS	212	21	233
	Acapesket Road LS	72	3	75
	Subtotal	**1137**	**52**	**1,189**
Total – All Lift Stations		**1,668**	**123**	**1,791**

Notes:

1. Houses that are adjacent to a gravity sewer but are located at a lower elevation than the sewer will require a small pump and small diameter force main to connect to the system.
2. Number of parcels anticipated to be sewered include all currently developed properties, developable properties, and undevelopable properties. It was assumed that the classification of properties currently designated as undevelopable may change with availability of sewer.

2.2.2 Estimated Wastewater Flows

As discussed in TASA TM-1, water usage data, provided by the Falmouth Water Department, for the years 2014 through 2016 was used to develop a current estimated wastewater flow for the sewer service area. Town-provided water use information was joined by account number to the Town's parcel data (January 8, 2018) through GIS. A 90% conversion factor was used to convert water usage into wastewater flow. Future wastewater flow estimates were developed using the methodology outlined in TASA TM-1.

Estimated future wastewater flows are summarized by sewershed in Table 3.

Table 3 – TASA Conceptual System Collection System Summary

Peninsula	Proposed Lift Station	Estimated Average Annual Flow (gpd)	Estimated Peak Instantaneous Flow (gpd)
Maravista	Spring Bars Road LS (Connection to Existing LS)	20,000	58,600
	Alphonse Street LS (Connection to Existing LS)	20,100	51,700
	Subtotal	**40,100**	**110,300**
Teaticket	Saint Marks Road LS	31,800	88,300
	Falmouthport LS	28,200	82,200
	Teaticket Path LS	17,900	51,400
	Broken Bow Lane LS	21,300	63,500
	Village Commons Lane LS	12,400	37,800
	Subtotal	**111,600**	**323,200**
Acapesket	Shorewood Drive LS	97,500	271,800
	Bridge Street LS	37,500	108,100
	East Falmouth Highway LS	56,000	167,800
	Acapesket Road LS	18,100	54,700
	Subtotal	**209,100**	**602,400**
Total – All Lift Stations		**360,800**	**1,035,900**

Notes:

1. Estimated wastewater flows developed using water use data from 2014, 2015 and 2016 and a 20% allocation to account for undesignated redevelopment and potential development of currently un-developable parcels.
2. A peaking factor of 3.4 is applied to the average annual flow to estimate peak instantaneous flow.
3. Infiltration rate assumed to be 500 gpd/in. diam/mile of gravity sewer based on 2011 TR-16 standards.
4. All flow values have been rounded to the nearest hundred.

(continued)

2.2.3 Conceptual Layout

The major components of the entire sewer service area used in the conceptual layout are summarized in Table 4.

Table 4 – TASA Conceptual System Collection System Summary

Component	Estimated Quantities
Gravity Mains	98,600 linear feet (18.7 miles)
Low Pressure Sewer	9,200 linear feet (1.7 miles)
Force Main	61,200 linear feet (11.6 miles)
Gravity Manholes	451
Gravity Connections	1,668
Grinder Pumps	123
New Lift Stations	9
Connection to Existing Lift Stations	2
Booster Lift Stations	1

Notes:
1. Estimated quantities were calculated using the SewerCAD model. No surveys have been conducted as part of this project.
2. Linear quantities have been rounded to the hundred.

2.3 WWTF Improvements Conceptual Layout

An evaluation of the existing Falmouth WWTF to treat the proposed flow from TASA on a capacity and treatment level basis was conducted as part of TASA TM-4. The evaluation indicated that the facility can handle the additional projected flow on a hydraulic basis (per the rated capacity outlined in the O&M manual). The analysis indicated that an additional (third) Sequencing Batch Reactor (SBR) would be required to treat the anticipated nutrient load generated by TASA.The Town requested that a flow allocation be incorporated into the analysis for undesignated redevelopment within the existing collection system. The existing collection system redevelopment allocation was established as 20% of the facilities 2015 permitted flow of 710,000 gpd on an annual average basis. Anticipated wastewater flows for TASA and the 20% existing collection system redevelopment allocation (designated as TASA + ESRA throughout this memorandum) are outlined in Table 5.

Table 5 – Projected TASA + Existing Collection System Redevelopment Allocation Wastewater Flows (TASA + ESRA Flow)

Parameter	TASA Wastewater Flow (mgd)	ESRA Wastewater Flow (mgd)	TASA + ESRA Wastewater Flow (mgd)
Average Annual	0.36	0.14	0.50
Maximum Month	0.59	0.26	0.85
Maximum Day	0.61	0.27	0.88
Peak Instantaneous	1.04	0.48	1.52

Notes:
1. Estimated wastewater flows for TASA were developed using water use data from 2014, 2015 and 2016 and a 20% allocation to account for undesignated redevelopment and potential development of currently un-developable parcels.

Parameter	TASA Wastewater Flow (mgd)	ESRA Wastewater Flow (mgd)	TASA + ESRA Wastewater Flow (mgd)

2. Existing system redevelopment allocation is established as 20% of the facilities 2015 permitted flow (existing system reallocation allocation = ESRA).
3. Infiltration rate assumed to be 500 gpd/in.diam/mile of gravity sewer based on 2011 TR-16 standards.

As outlined in Table 6, a preliminary process model developed by Aqua-Aerobic Systems, Inc. indicates that the addition of a third basin would provide adequate capacity for the anticipated future flow from TASA and the redevelopment allocation. A conceptual layout of the additional SBR is shown in Figure 4.

Table 6 – Sequencing Batch Reactor Three Basin System – Preliminary Design

Flow (mgd)	WWTF Prior to LPSA Connection[1]	LPSA[2]	TASA[3]	Existing Collection System Redevelopment Allocation (ESRA)[4]	Total Future Flow	3-Basin Preliminary Design Flows[5]
Average Daily Flow	0.36	0.26	0.36	0.14	1.12	1.48
Maximum Day Flow	0.68	0.47	0.61	0.27	2.03	2.2

Notes:

1. WWTF flows prior to LPSA connection were calculated based on the influent wet well flow measured flows from November 2010 through November 2012.
2. Source: 'Town of Falmouth Recharge Beds 14 & 15 Operations and Maintenance Manual', prepared by GHD, dated June 2017.
3. TASA TM-1 plus and an estimated allocation for infiltration and inflow (I/I) (I/I allocation will be updated once TASA conceptual layout is finalized).
4. Redevelopment allocation is established as 20% of the facility's 2015 permitted flow.
5. Source: 'Process Design Report – Design #132822', prepared by Aqua-Aerobic Systems, Inc., dated December 3, 2012.

Based on the effluent disposal options selected for conceptual layout development, no other major capacity or treatment level process improvements are anticipated to treat the projected flow from TASA and the redevelopment allocation. It is recommended that a condition evaluation be conducted at the facility to identify non-capacity/treatment level improvements that are likely needed (as discussed further in TASA TM-4). The condition evaluation should include an assessment of the physical state of equipment and structures.

2.4 Effluent Disposal Conceptual Layouts

In the November progress meeting four effluent disposal options were selected for conceptual layout development:

1. Open Sand Beds at the Allen Parcel.
2. Subsurface Effluent Disposal (Leaching Trenches) at the Falmouth Country Club.
3. Expanded Open Sand Beds 14 & 15 within the existing Town-owned parcel.
4. Buzzards Bay Ocean Outfall.

The basis of design for each conceptual layout is described in this section.

2.4.1 Open Sand Beds at the Allen Parcel

The conceptual layout for the Allen Parcel with Open Sand Beds is shown on Figure 5. The following basis of design was used to develop the layout.

- Open Sand Beds 1 through 4 are sized based on the anticipated flow from TASA + ESRA during maximum month conditions with one open sand bed out of service.
- A 50-foot buffer is maintained from the utility easement to provide a visual barrier.
- A 100-foot buffer is maintained from the property line on Carriage Shop Road.
- A 20-foot buffer is maintained around the perimeter of Lot 3 for the development of a future service road.
- The open sand beds are sized based on a design hydraulic loading rate of 7 gpd/sf.
- The conceptual layout assumes that the open sand beds will be fed by gravity from a distribution box at the end of the effluent force main.
- An effluent lift station located at the Falmouth WWTF conveys flow to the effluent disposal site.
- The effluent force main route follows Blacksmith Shop Road, Thomas Landers Road, Sandwich Road and Carriage Shop Road (approximately 4 miles).

As documented in TASA TM-3, the Allen Parcel has land area available for additional effluent disposal (beyond TASA +ESRA).

2.4.2 Subsurface Effluent Disposal (Leaching Trenches) at the Falmouth Country Club (FCC)

The conceptual layout for the FCC with subsurface effluent disposal (leaching trenches) is shown in Figure 6. The following basis of design was used to develop the layout.

- The leaching trench system is sized based on the anticipated flow from TASA +ESRA during maximum month conditions with one leaching area out of service.
- The leaching trench system is sized based on a design hydraulic loading rate of 3 gpd/sf.
- An effluent lift station located at the Falmouth WWTF conveys flow to the effluent disposal site.
- The leaching trench system is fed through a pressurized distribution system from an effluent lift station located on the FCC site.
- The effluent force main route follows Blacksmith Shop Road, Thomas Landers Road, Sandwich Road and Carriage Shop Road (approximately 5 miles).

As documented in TASA TM-3, the FCC has land area available for additional effluent disposal (beyond TASA +ESRA).

2.4.3 Expanded Open Sand Beds 14 & 15 (Within Existing Town-Owned Property Boundaries)

A conceptual layout was developed for expanding the existing Open Sand Beds 14 and 15 (Figure 7). The expansion is intended to maximize the recharge area available within the existing Town-owned property boundaries to the north of the existing open sand beds. The following basis of design was used to develop the layout.

- Existing Open Sand Beds 14 and 15 are expanded to the North through demolition of the existing northern berm wall of each bed.
- A 25-foot buffer is maintained from the northern property boundary to the edge of the expanded open sand beds (the minimum buffer specified in the MassDEP Small WWTF Design Guidelines). This scenario was developed to maximize the potential available discharge area on the site. In this scenario the northern access road for the beds lies within the 25-foot buffer.
- The open sand beds are sized based on a design hydraulic loading rate of 7 gpd/sf.
- The maximum month capacity gained through Expanded Open Sand Beds 14 & 15 (expanded open sand bed capacity minus existing open sand bed capacity) is estimated as 0.63 mgd on a maximum month basis with all open sand beds in service (system redundancy may be provided by existing open sand beds at the Falmouth WWTF).
- Effluent flow is conveyed to the open sand beds through the existing gravity pipe to Recharge Beds 14 and 15.

The conceptual layout was developed based on the maximum recharge area potentially available on the site. Subsequent information gathered through field investigations and survey may indicate parameters that reduce the overall potential area on the site. The maximum additional capacity from expanding Open Sand Beds 14 and 15 (0.63 mgd on a maximum month basis) is not adequate to recharge the estimated maximum month flow from TASA + ESRA (0.84 mgd) at a hydraulic loading rate of 7 gpd/sf. Additional recharge capacity at a different location would be required to recharge the entire TASA + ESRA flow at 7 gpd/sf. Alternately, performance testing could be conducted at the existing Open Sand Beds 14 and 15 to evaluate the ability to request an increase in the rated capacity of the open sand beds. Preliminary calculations indicate that Expanded Open Sand Beds 14 and 15 (as shown in the conceptual layout) have adequate capacity to recharge TASA +ESRA if the allowable hydraulic loading rate was raised to 8 gpd/sf.

2.4.4 Buzzards Bay Ocean Outfall

A conceptual layout was developed for a potential ocean outfall from the Falmouth WWTF extending approximately 4,380 feet into Buzzards Bay (Figure 9).The following basis of design was used to develop the layout.

- Effluent is pumped from the Falmouth WWTF to a location off Chapoquoit Road to Buzzards Bay by an effluent lift station located at the Falmouth WWTF. Final disinfection occurs at the Falmouth WWTF (no remote disinfection process).
- 2-foot nominal diameter PVC force main from the Falmouth WWTF along Service Road, Brick Kiln, West Falmouth Highway and Chapoquoit Road (approximately 2 miles, sized to convey 4 mgd

annual average flow). The transition from force main to ocean outfall occurs in an air release valve vault (no second effluent lift station). 2-foot nominal diameter HDPE force main extends approximately 4,380 feet from Chapoquoit Road to Buzzards Bay. A more detailed analysis of pipe size should be completed during final design.

- One cubic yard of rock excavation was assumed for every 30 linear feet of pipe laid.
- Force main from Falmouth WWTF to Chapoquoit Road is installed through open cut trench excavation.
- The maximum length of horizontal directional drilling (HDD) installation is 3,500 feet. Although HDD installation is typically a more cost effective method of installation than open cut excavation field investigations, such as survey and marine borings, are required to determine the feasibility of its use. Since no field investigations have been completed, a range of conceptual costs was developed for the ocean outfall using a combination of HDD and open cut excavation as the low end of the range and an all open cut excavation installation as the high end of the range.
- 2-foot diameter HDPE outfall (approximately 4,380 feet from Chapoquoit Road into Buzzards Bay).
- The existing open sand beds at the Falmouth WWTF provide redundancy to the system. If the ocean outfall needed to be taken offline for a period of time, effluent flow would be diverted to the existing open sand beds.

If constructed, an ocean outfall is anticipated to provide a long-term effluent disposal solution for the Town of Falmouth.

2.5 Conceptual Layouts Summary

Through discussion with the Town and WQMC Working Group four conceptual layouts were developed for the purposes of preliminary cost estimating. Each conceptual layout identifies a sewer service area, collection/transmission system WWTF capacity upgrades and discharge plan. The four conceptual layouts are summarized in Table 7.

Table 7 – Conceptual Layout Summary

<table>
<tr><th>Layout #</th><th>Sewer Service Area Collection/Transmission System Components</th><th>WWTF Upgrades</th><th>Effluent Discharge Plan</th></tr>
<tr><td>1 – Open Sand Beds at the Allen Parcel</td><td rowspan="2">TASA Sub-Areas 1 & 2
• Nine new lift stations
• Connection to two existing lift stations
• One booster lift station
• Dual force main from the booster lift station to the Falmouth WWTF</td><td rowspan="2">• Installation of an additional SBR tank and associated process equipment[1]</td><td>• Open sand beds at the Allen Parcel
• Sized to recharge maximum month flow from TASA +ESRA with one open sand bed out of service (Maximum Month Design Flow = 0.84 mgd)</td></tr>
<tr><td>2 – Subsurface Effluent Disposal at Falmouth Country Club</td><td>• Subsurface effluent disposal at the Falmouth Country Club
• Sized to recharge maximum month flow from TASA +ESRA with one leaching trench area out of service (Maximum Month Design Flow = 0.84 mgd)</td></tr>
</table>

Layout #	Sewer Service Area Collection/Transmission System Components	WWTF Upgrades	Effluent Discharge Plan
3 – Expanded Open Sand Beds 14 and 15			• Expanded open sand beds 14 and 15 • Sized to maximize the effluent disposal capacity of the site with all open sand beds in service during maximum month conditions (Maximum Month Design Flow = 0.63 mgd)
4 – Buzzards Bay Outfall			• Buzzards Bay ocean outfall • Average Design Flow = 4 mgd

Notes:
1. Recommended WWTF upgrades includes treatment capacity and process related improvements only. A condition evaluation should be conducted to assess the physical state of equipment and buildings.

3. Conceptual Cost Estimates

Preliminary level capital costs were developed for the four conceptual layouts. The Engineer's opinion of probable costs in 2018 dollars is outlined in Table 8. The costs are the total estimated project costs with allowances for construction costs including:

- 30 percent construction contingency
- 10 percent engineering design
- 2 percent fiscal/legal/permitting/administrative costs
- 15 percent construction administration and Resident Project Representative (RPR) costs
- 8 percent police detail costs for linear work.

Because of the conceptual nature of this evaluation, a 30 percent construction contingency is carried as no detailed design has been performed and no survey have been performed. During final design a reduced contingency will be carried for variability in the bidding climate, project changes before bidding, easements, residential property restoration, and change orders due to unforeseen conditions. Project costs are presented in 2018 dollars. Project costs are also presented for an estimated mid-point of construction date in 2026. Once the construction timeframe is known, project costs should be adjusted to the mid-point of construction.

The following basis of design was used to develop the conceptual cost estimates:

1. Sewer Service Area and Collection and Transmission System

- Nine new lift stations and one booster lift station are constructed in the sewer service area.
- All lift stations are assumed to be submersible lift stations.
- Maximum depth for gravity sewer was set at 22 feet.

- Force mains between the nine lift stations and the booster lift station are single force mains.
- Force main from the booster lift station to the Falmouth WWTF is a dual force main. The dual force main system is sized to provide full redundancy (each force main sized to convey full anticipated flow).
- Costs to acquire privately owned land for lift stations was estimated as the property value listed on Zillow.com (accessed November 2018) plus 20% to account for future price fluctuations.
- It was assumed that use of the property for a lift station would be negotiated for the proposed Falmouthport Lift Station, the proposed Shorewood Drive Lift Station and the proposed Village Commons Lane Lift Station. It was assumed that these properties would not be purchased by the Town.
- Properties serviced by low pressure connection have one grinder pump per building.
- Town assumes all construction costs up to the property line (installation of the lateral from the property line to each house is assumed to be the responsibility of the property owner and a cost for the installation is not included in this cost estimate).
- Grinder pumps are owned by the Town and installed by the property owner (capital costs for the grinder pumps are included in the cost estimate. The installation of the grinder pumps is assumed to be the responsibility of the property owner and no installation costs for grinder pumps are included in this cost estimate. This assumption is based on LPSA grinder pump bid prices).
- Trench paving only for Town and private roads. Full width overlay or other road reconstruction options are not factored into the cost estimates for Town and private roads.
- Full width mill and overlay is assumed for State roads only.
- The number of lateral connections for each sewershed is based on upon the number of parcels contained within the sewershed minus conservation land (land use code 932, 950, 982) and the “Shorewood Parcel” (which has a conservation restriction). It was assumed that “undevelopable” land may be re-classified in the future with the availability of sewer and may connect into the collection system in the future.
- Estimated project costs do not include utility relocation.
- Estimated costs assume that no hazardous materials or other materials that required special handling are encountered.

2. WWTF Improvements

- A third SBR will be constructed to the north of the two existing SBRs.
- A new pipe gallery will be constructed to house the electrical room, blower and pump for the third SBR.

A condition evaluation should be conducted to identify non capacity/treatment process improvements that are likely needed at the facility.

3. Effluent Disposal

- For Conceptual Layouts 1, 2, and 4 it is assumed that an effluent lift station to be constructed at the Falmouth WWTF is used to convey flow from the WWTF to the effluent disposal location.
- For Conceptual Layout 3 the existing gravity line from the Falmouth WWTF to Open Sand Beds 14 and 15 is used to convey flow from the WWTF to the effluent disposal location.
- Since no field investigations have been conducted along the potential ocean outfall route, the Conceptual Layout 4 effluent disposal costs were developed as a range. The low end of the range assumed 3,500 LF of the ocean outfall is installed using HDD and the remainder of the outfall is installed using open cut excavation. The high end of the range assumed that the entire ocean outfall is installed using open cut excavation. Field investigations are required to determine how much of the ocean outfall could feasibly be installed through HDD.
- Estimated project costs do not include utility relocation.
- Estimated costs assume that no hazardous materials or other materials that require special handling are encountered.

(continued)

Table 8 – Conceptual Layout Engineers Estimate of Probable Cost[1]

	1) Open Sand Beds at the Allen Parcel	2) Subsurface Effluent Disposal at FCC	3) Expanded Open Sand Beds 14 & 15	4) Buzzards Bay Ocean Outfall[5]
Collection System[2,3,4]	$67.9 M	$67.9 M	$67.9 M	$67.9 M
WWTF Improvements	$7.8 M	$7.8 M	$7.8 M	$7.8 M
Effluent Disposal[4]	$9.8 M	$11.9 M	$1.5 M	$55.1 M - $96.2 M
Construction Sub-Total	**$85.5 M**	**$87.6 M**	**$77.2 M**	**$130.8 M - $171.9 M**
Design	$8.6 M	$8.8 M	$7.7 M	$13.1 M - $17.2 M
Construction Phase Services	$10.3 M	$10.5 M	$9.3 M	$15.7 M - $20.6 M
Survey & Soil Borings	$3.4 M	$3.5 M	$3.1 M	$5.2 - $6.9 M
Fiscal, Legal, Police	$3.3 M	$3.4 M	$3.2 M	$3.8 M - $4.2 M
Capital Costs (2018$)	**$111.1M**	**$113.8 M**	**$100.5 M**	**$168.6 M - $220.8 M**
Capital Costs (2026$)	**$140.8 M**	**$144.2 M**	**$127.4 M**	**$213.6 M - $279.8 M**

Notes:

1. Total Capital Costs includes allowances for construction costs such as: a 30% construction contingency, 10% engineering design, 2% fiscal/legal/permitting/administrative costs, 15% construction administration and Resident Project Representative (RPR) costs, and 8% police detail costs on linear work.
2. Estimated quantities for cost estimates were calculated using the SewerCAD model. No surveys have been conducted of the proposed sewer service area as part of this project.
3. Costs do not include the cost of final restoration of private property or the cost of attaining easements.
4. Estimated costs for obtaining currently privately owned potential lift station locations are included.
5. Trench work (gravity, low pressure and force mains) in Town, private and State roads includes excavation, backfill and traffic control. Final trench repair is assumed for Town and private roads. Full width overlay is assumed for State roads only.
6. The Buzzards Bay Ocean Outfall conceptual costs are presented as a range. The low end of the range assumes 3,500 LF of the ocean outfall is installed using horizontal directional drilling (HDD) and the remainder of the outfall is installed using open cut excavation. The high end of the range assumes that the entire ocean outfall is installed using open cut excavation. Field investigations are required to determine how much of the ocean outfall could feasibly be installed through HDD.

4. Summary and Next Steps

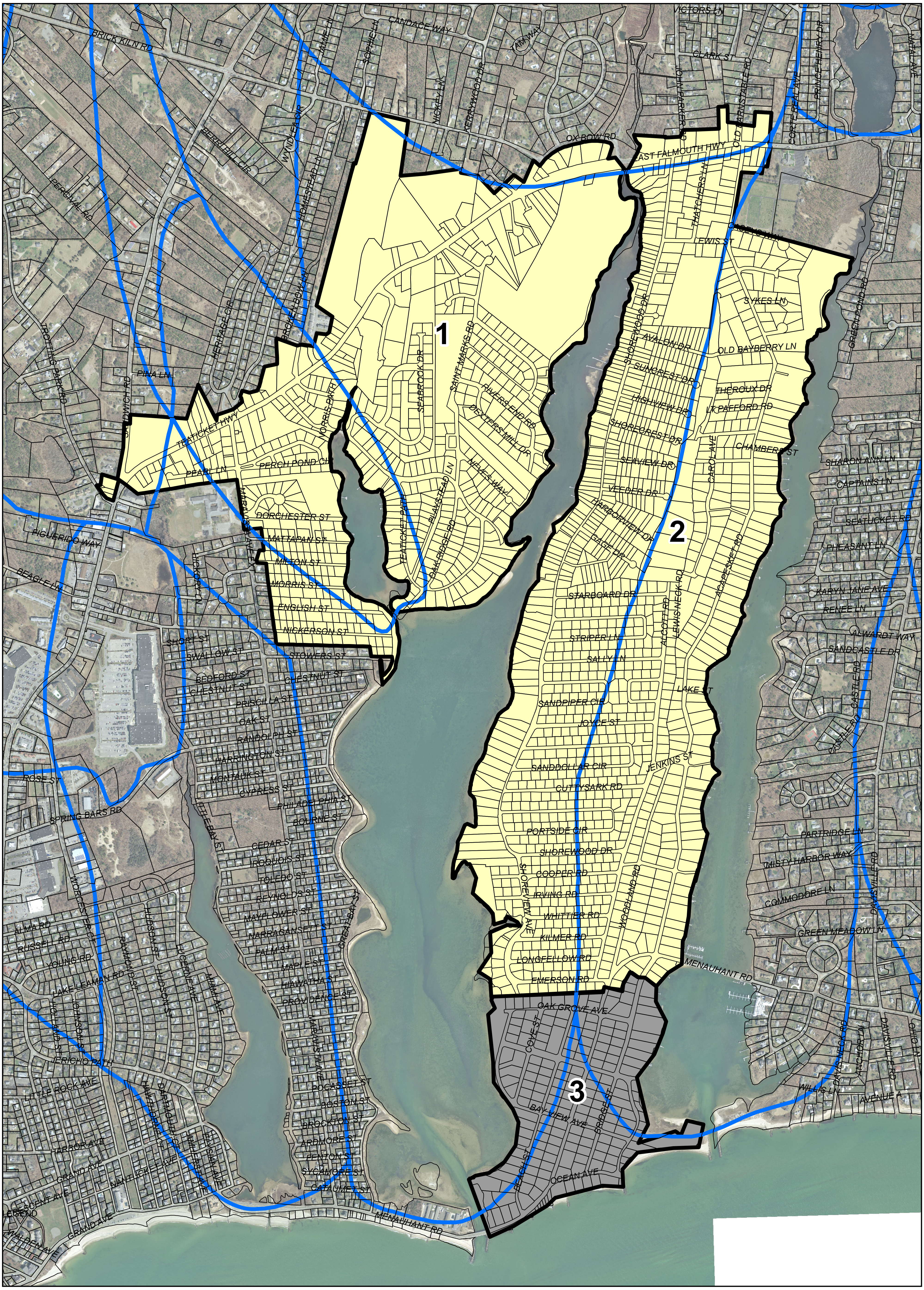
The preliminary capital cost estimates, developed for the four conceptual layouts are outlined below:

- Open Sand Beds at the Allen Parcel – $112,600,000 (2018 Dollars)
- Subsurface Effluent Disposal at the Falmouth Country Club – $115,300,000 (2018 Dollars)
- Expanded Open Sand Beds 14 and 15 – $101,900,000 (2018 Dollars)
- Buzzards Bay Outfall – $170,100,000 - $222,300,000 (2018 Dollars)

The preliminary cost estimates will be reviewed at a workshop with the Town and WQMC Working Group and revised based on workshop feedback. The finalized version of this memorandum is anticipated to be included as an Appendix to the Notice of Project Change Update Report.

Figures

Figure 1 – TASA Sub-Areas
Figure 2 – TASA Collection System Conceptual Layout
Figure 3 – Conceptual Layouts – Force Mains
Figure 4 – Conceptual Layouts – WWTF Improvements
Figure 5 – Conceptual Layout 1
Figure 6 – Conceptual Layout 2
Figure 7 – Conceptual Layout 3
Figure 8 – Conceptual Layout 4



Legend

Yellow	Teaticket/Acapesket Sewer Service Area Conceptual Layout
Blue line	MEP Watershed
Gray	Not Included in TASA Service Area Conceptual Layout

Paper Size ANSI B

0 500 1,000 2,000

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1927
Grid: NAD 1927 StatePlane Massachusetts Mainland FIPS 2001

N

Town of Falmouth,MA
Teaticket/Acapesket Preliminary Evaluation (TASA TM-7)

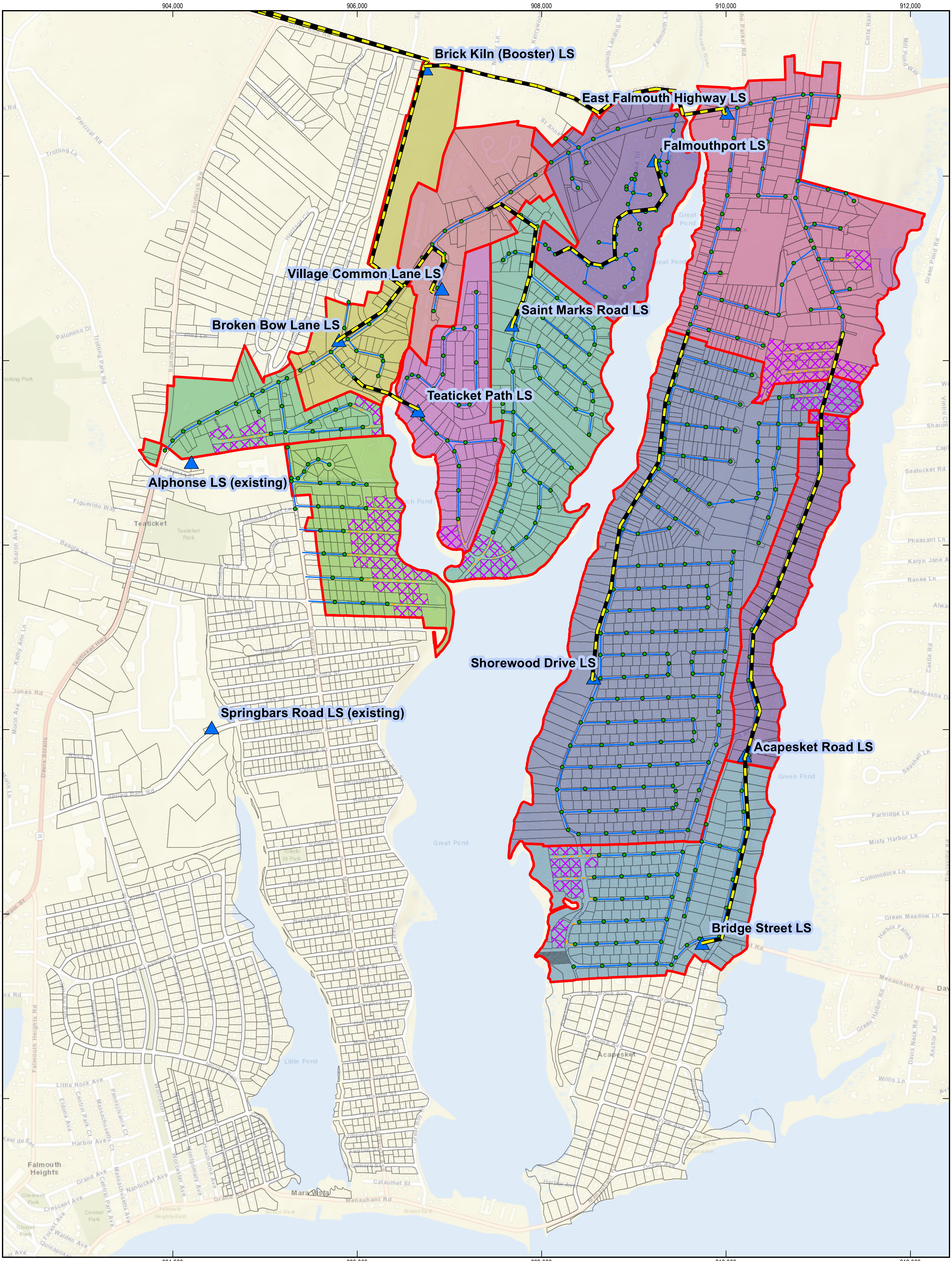
Job Number	11153041
Revision	A
Date	Feb 14, 2019

TASA SUB AREAS

Figure 1

1545 Iyannough Road, Hyannis Massachusetts 02601 USA **T** 1 508 362 5680 **F** 1 508 362 5684 **E** hyamail@ghd.com **W** www.ghd.com

G:\111\11153041 Town of Falmouth South Coast CWMP Update\GIS\Maps\MXD_Deliverables\TASA TM-7\Figures\111-53041-F01 TASA Subareas.mxd



Legend

Force Main
Lift Station
Manhole
Gravity Sewer
Low Pressure Pipe
Low Pressure Parcels
Parcels

Sewersheds

Acapesket Road LS
Alphonse Street LS
Bridge Street LS
Broken Bow Lane LS
East Falmouth Highway LS
Falmouthport LS
Saint Marks Road LS
Shorewood Drive LS
Springbars Road LS
Teaticket Path LS
Village Common Drive LS

Paper Size ANSI B

0 380 760 1,520

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

Town of Falmouth,MA
Teaticket/Acapesket Preliminary
Evaluation (TASA TM-7)

Job Number 11153041
Revision A
Date Feb 14, 2019

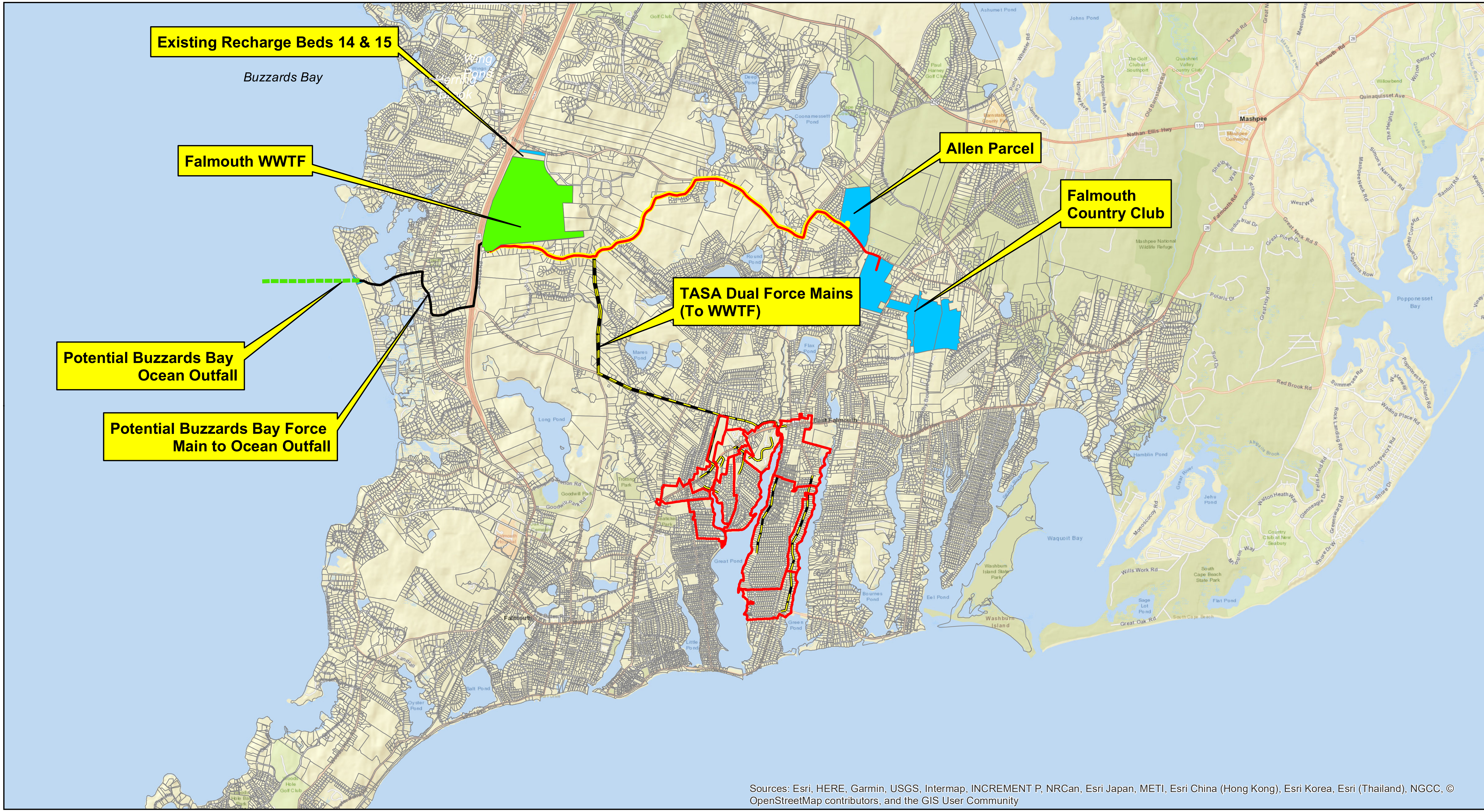
TASA COLLECTION SYSTEM
CONCEPTUAL LAYOUT

Figure 2

G:\111\11153041 Town of Falmouth South Coast CWMP Update\GIS\Maps\MXD_Deliverables\TASA TM-7\Figures\111-53041-F02

1545 Iyannough Road, Hyannis Massachusetts 02601 USA T 1 508 362 5680 F 1 508 362 5684 E hyamail@ghd.com W www.ghd.com

Data source: Data Custodian, Data Set Name/Title, Version/Date. Created by:jjobrien



Paper Size ANSI B

0 2,250 4,500 9,000 Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

LEGEND

- Allen Parcel
- Potential Buzzards Bay FM to Outfall
- Falmouth Country Club
- TASA Force Main to WWTF
- Existing Effluent Disposal Parcel
- Potential Effluent Disposal Parcel
- TASA Sewer Service Areas
- Potential Buzzards Bay Ocean Outfall

TOWN OF FALMOUTH, MA
Teaticket/Acapesket Preliminary Evaluation TASA (TM-7)

Conceptual Layouts - Force Main Routing For Effluent- Conceptual Layouts

Job Number	111-53041
Revision	A
Date	14 Feb 2019

Figure 3

G:\111\11153041 Town of Falmouth South Coast CWMP Update\GIS\Maps\MXD_Deliverables\TASA TM-7\Figures\111-53041-F03 EFF_Routing.mxd

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1545 Iyannough Road, Hyannis Massachusetts 02601 USA T 1 508 362 5680 F 1 508 362 5684 E hyamail@ghd.com W www.ghd.com

LEGEND

Structure Type

- Pipe Gallery
- SBR

Paper Size ANSI A

0 170 340 680 Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

GHD

TOWN OF FALMOUTH, MA
Teaticket / Acapesket Preliminary Evaluation TASA (TM-7)

CONCEPTUAL LAYOUT -
WWTF IMPROVEMENTS

Job Number | 111-53041
Revision | -
Date | 29 Nov 2018

Figure 4

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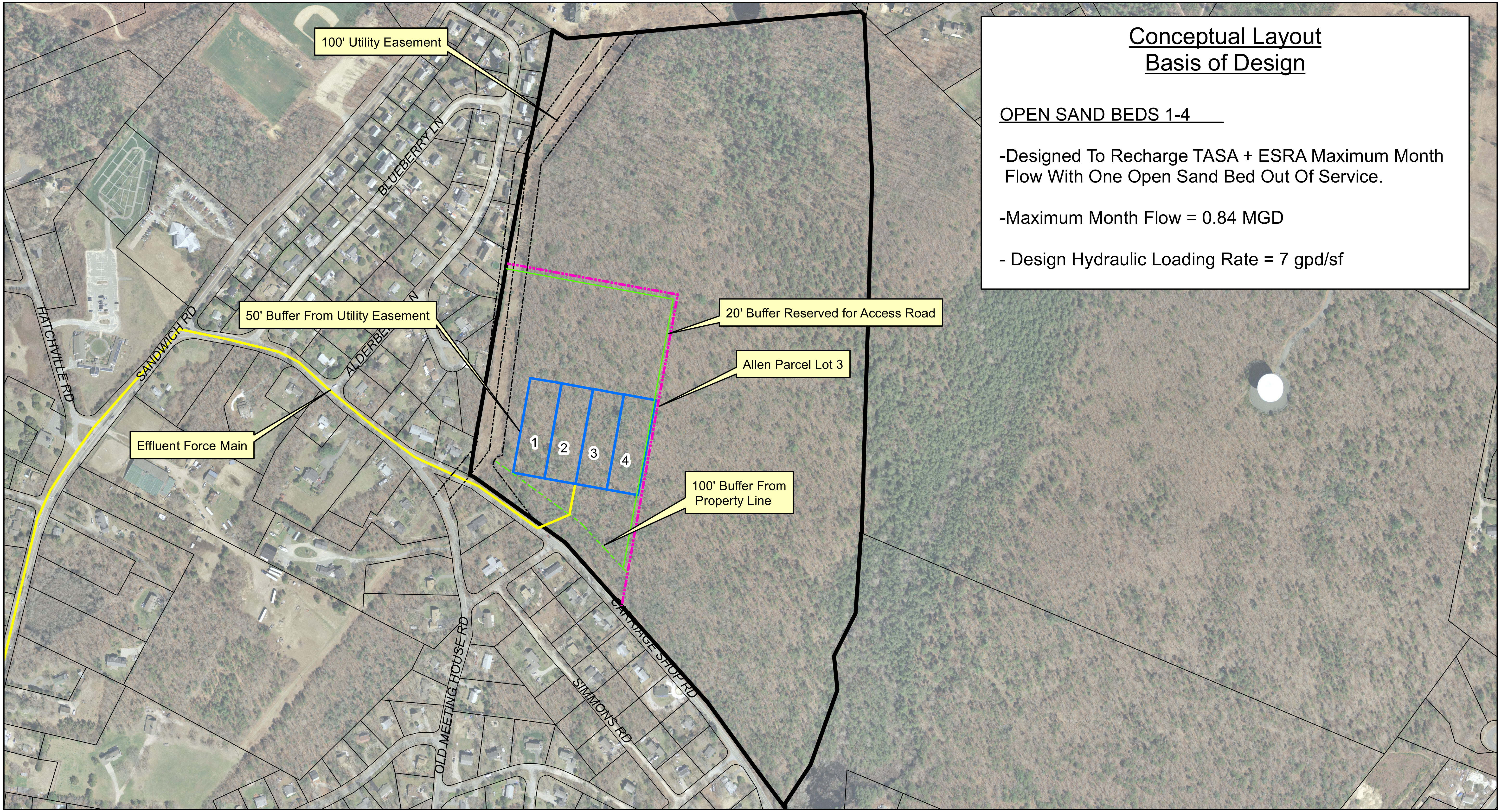


Paper Size ANSI B

0 175 350 700 Feet

N

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

LEGEND

Roads_CL	Roads_CL
Allen Parcel Lot 1 Border	Force Main
Access road	Conceptual Open Sand Beds
Lot 3 Border (General Municipal Use -Plan 2005)	Utility Easement

TOWN OF FALMOUTH, MA
Teaticket/Acapesket Preliminary Evaluation TASA (TM-7)

GHD

Job Number	111-53041
Revision	A
Date	14 Feb 2019

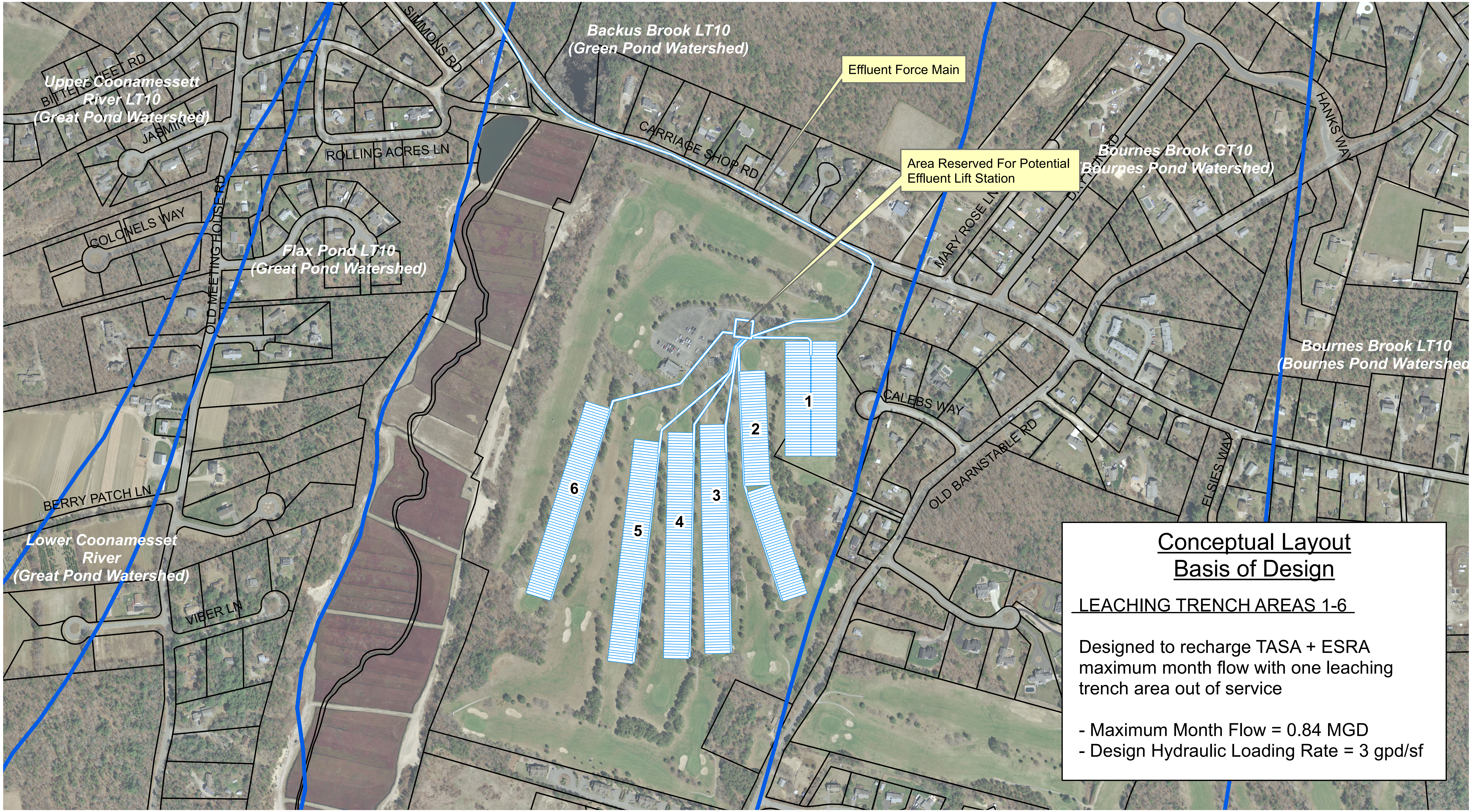
Conceptual Layout 1 - Open Sand Beds at Allen Parcel

Figure 5

G:\111\11153041 Town of Falmouth South Coast CWMP Update\GIS\Maps\MXD_Deliverables\TASA TM-7\Figures\111-53041-F05_Allen.mxd

Data source: Data Custodian, Data Set Name/Title, Version/Date. Created by:jjobrien

1545 Iyannough Road, Hyannis Massachusetts 02601 USA T 1 508 362 5680 F 1 508 362 5684 E hyamail@ghd.com W www.ghd.com



Paper Size ANSI B

0 200 400 800

Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

Legend

- Conceptual Infiltration Trench for TASA
- MEP Watershed Boundary
- Parcel Boundary

TOWN OF FALMOUTH, MASSACHUSETTS
Teaticket/Acapesket Preliminary Evaluation (TM-7)

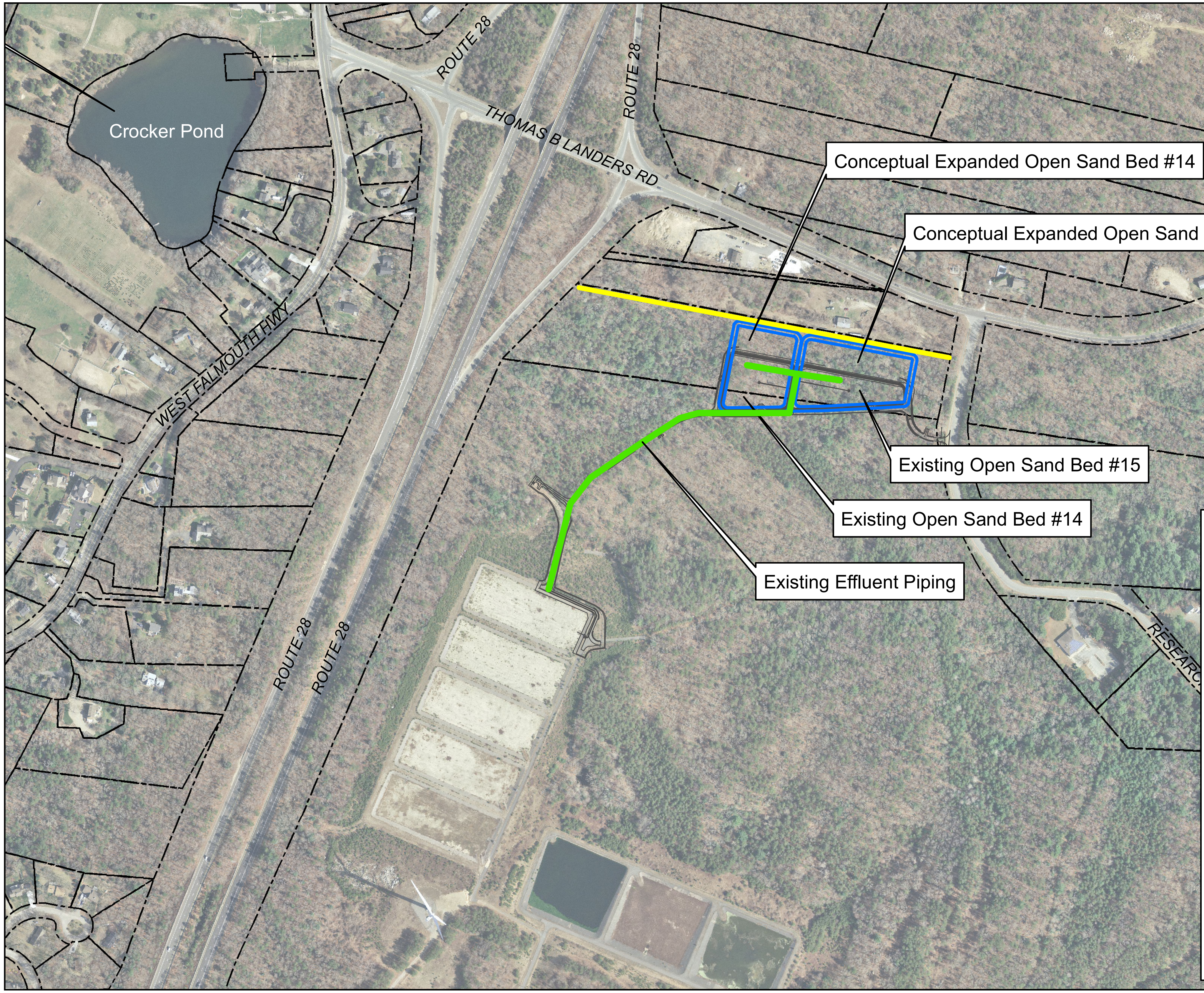
Conceptual Layout 2 - Falmouth Country Club
Effluent Leaching Trenches

Job Number	111-53041
Revision	A
Date	29 Nov 2018

Figure 6

G:\111\11153041 Town of Falmouth South Coast CWMP Update\GIS\Maps\MXD_Deliverables\TASA TM-7\Figures\111-53041-F06_FalCCC.mxd

1545 Iyannough Road, Hyannis Massachusetts 02601 USA T 1 508 362 5680 F 1 508 362 5684 E hyamail@ghd.com W www.ghd.com



CONCEPTUAL LAYOUT
BASIS OF DESIGN

EXPANDED OPEN SAND BEDS #14 & #15

Maximum Month Flow (From Expansion) = 0.63 MGD
Design Hydraulic Loading Rate = 7 gal/sf

Designed For Maximum Month Conditions
With All Beds in Service.

Notes:
1. Maximum potential available recharge area shown. Field investigations may indicate a reduced available area available for recharge.

2. Service access road located within 25 feet buffer from property boundary.

Paper Size ANSI B

0 200 400 800 Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

LEGEND
Conceptual Expanded Open Sand Bed
Existing Effluent Piping
25' Buffer From Property Line

TOWN OF FALMOUTH, MA
Teaticket/Acapesket Preliminary Evaluation TASA (TM-7)

Conceptual Layout 3 - Expanded Open Sand Beds 14 & 15

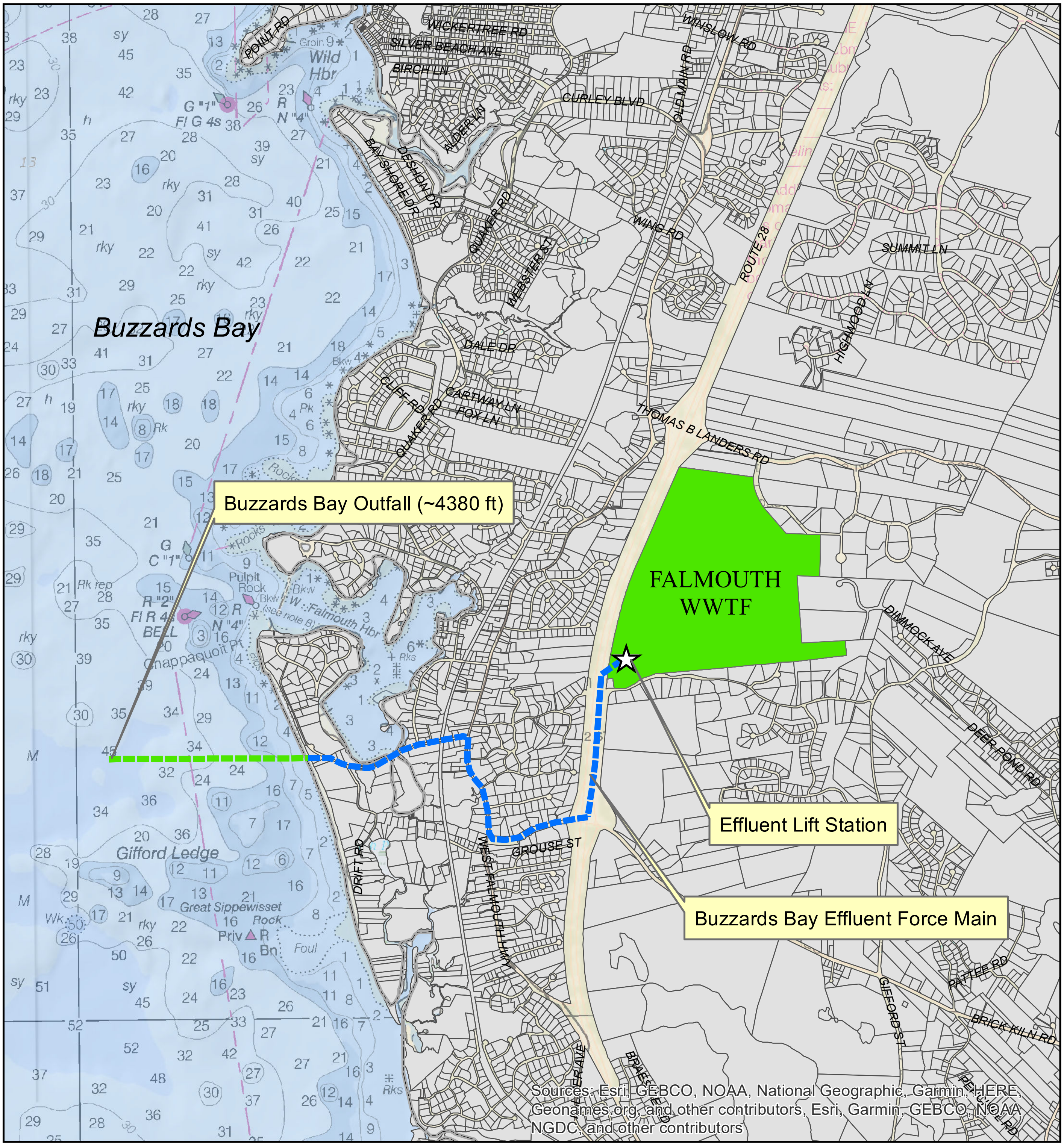
Job Number	111-53041
Revision	A
Date	14 Feb 2019

Figure 7

G:\111\11153041 Town of Falmouth South Coast CWMP Update\GIS\Maps\MXD_Deliverables\TASA TM-7\Figures\111-53041-F07_WWTF_SB.mxd

Data source: Data Custodian, Data Set Name/Title, Version/Date. Created by:jjobrien

1545 Iyannough Road, Hyannis Massachusetts 02601 USA T 1 508 362 5680 F 1 508 362 5684 E hyamail@ghd.com W www.ghd.com



LEGEND

- Outfall
- Buzzards Bay Effluent Force Main

Paper Size ANSI A

0 750 1,500 3,000 Feet

Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

GHD

TOWN OF FALMOUTH, MA
Teaticket / Acapesket Preliminary Evaluation TASA (TM-7)

CONCEPTUAL LAYOUTS -
BUZZARDS BAY OCEAN OUTFALL

Job Number	111-53041
Revision	-
Date	14 Feb 2019

Figure8

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