Town of Mashpee, MA

Town of Mashpee Existing WWTF Evaluation

FINAL REPORT

September 2017

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1. Introduction

1.1 Background

The Town of Mashpee, MA has been involved in a watershed nitrogen management planning effort since the late 1990's. The planning effort has recently culminated in a Town-wide Watershed Nitrogen Management Plan (WMNP) for the watersheds of Popponesset Bay, Waquoit Bay East, and the remainder of the Town of Mashpee.

Upon the completion of the Town's May 2015 WNMP Final Recommended Plan/Final Environmental Impact Report (FRP/FEIR) and receipt of the July 2015 Certificate of the Secretary of Energy and Environmental Affairs, the Town is starting the implementation of their approved plan.

As part of the recommendations from the approved plan, potential sewer extensions to two private wastewater treatment facilities (WWTF)—Mashpee Commons and a new WWTF at Wampanoag Village—were identified in Phase 1 of the implementation schedule.

In order to consider these further, an updated evaluation of the two facilities and neighboring areas was required to evaluate the feasibility of utilizing the facilities, either at their current capacity or at a feasible expanded capacity.

1.2 Purpose of Report

The purpose of this report is to evaluate two private WWTFs (Mashpee Commons WWTF and Wampanoag WWTF) located within the Town of Mashpee for possible sewer extensions and to consider their available capacity relative to the capacity needs of the areas identified for sewering as part of the recommended plan. The report also examines potential changes to nitrogen loading impacts from these facilities and discusses the approach to using Computer Aided Design (CAD) modeling (i.e. SewerCAD/SewerGems) to layout potential sewers that could serve these facilities and a potential new WWTF within the same watershed. A preliminary sewer layout plan is provided (in Attachment 1) for areas scheduled to be connected to the proposed Site 4 facility during Phase 1.

1.3 Scope and Limitations

In February 2016, the Town retained GHD to perform a capacity evaluation of the Wampanoag and Mashpee Commons WWTFs and collection systems for implementation of the initial phase of the WNMP.

The Scope of Services (as amended) for this work included the following:

Task 1 – Review Data. GHD reviewed available design, construction, and operation data for both facilities.

Task 2 – Evaluate Treatment Capacity for Each Major Treatment Component of the Wampanoag WWTF. GHD reviewed the treatment capacity of the following major treatment processes at the facility based on available design information:

- Pretreatment/Primary Settling
- Flow Equalization
- Biological Treatment
- Secondary Settling
- Denitrification

- Effluent Dosing and Discharge Facilities
- Odor Control Facilities
- Effluent Disinfection

Task 3 – Evaluate Treatment Capacity for Each Major Treatment Component of the Mashpee Commons WWTF. GHD reviewed the treatment capacity of the following major treatment processes at the facility based on available design information:

- Pretreatment/Primary Settling
- Flow Equalization
- Biological Treatment
- Denitrification
- Effluent Dosing and Discharge Facilities
- Effluent Disinfection

Task 4 – Identify and Evaluate Pump Station Locations for the New Service Areas. For proposed pump station locations, GHD reviewed site ownership, location, elevation, distance from sensitive receptors, flood plains and wetlands (based on available MassGIS data), and potential need for easements. One preliminary pump station layout is provided in this report based on similar designs and size requirements as used in other communities.

Task 5 – Perform a Hydraulic Capacity Analysis. A hydraulic capacity analysis was conducted for the portions of the existing private gravity collection system that are expected to be impacted by the potential new sewers. The analysis was based on a SewerCAD type of program for gravity sewers and did not include verification of the collection system record drawings through fieldwork or survey.

Task 6 – Develop Collection System Alternatives. Estimates of probable construction, operations, and maintenance costs were developed for the preliminary collection system layout. At the Town's request, gravity collection systems were utilized to the maximum feasible extent. Cost development was based on the preliminary sewer layouts using GIS/CAD based on GIS topographic information, available MassGIS data, and available utility information.

Task 7 – Prepare Plans and Profiles. Base plans were developed for each service area. Plans and profiles were developed based on the output of the SewerCAD model results and available GIS information.

Task 8 – Review Effluent Nitrogen Concentration Issues. Issues related to the effluent nitrogen concentration from each of the WWTFs following the connection of these additional areas and their impacts to coastal embayments (specifically the Mashpee River Watershed) were reviewed. A nitrogen load analysis was conducted based on an assumed performance level of the WWTFs.

Task 9 – Mashpee Commons WWTF Site Visit. A site visit was conducted at the Mashpee Commons WWTF to review major as-built conditions of the process mechanical equipment. The site visit did not include confined space entry or entry to other non-standard accessible locations.

Task 10 – Identify Data Gaps. The need for additional influent and effluent sampling based on available existing data was evaluated.

Task 11 – Site 4 Preliminary Phase 1 Sewer Model. The Phase 1 sewer model around Site 4 was updated and expanded to reflect the sewershed outlined in the FRP/FEIR dated May 2015. The analysis results were based on a SewerCAD model. Preliminary sewer plans and profiles were provided for the proposed layout and potential pump station locations were identified.

Task 12 – Prepare a Draft Project Report. A draft report was prepared presenting the findings of the evaluation.

Task 13 – Meetings.

1.4 Summary of Prior Reports

The following documents were provided to GHD and were used in the development of this report:

- 'Addition to Sewage Disposal System Mashpee Shopping Center for Fields Point Manufacturing Corp' prepared by Whitney & Basset Architects & Engineers, Revised February 26, 1968
- 'Fields Point Corporate Proposed Utility Plan', prepared by Dufresne-Henry, dated April 1986
- 'Mashpee Commons South Street and Market Street Profiles', prepared by Dufresne-Henry, dated August 27, 1986
- 'Fields Point Corporation Mashpee Commons Sewage Treatment Plant', prepared by Dufresne-Henry, dated November 1986
- 'Mashpee Commons Main Street Profile', prepared by Dufresne-Henry, dated August 27, 1986
- 'North Market Street Mashpee, Massachusetts' Prepared by Dufresne-Henry, dated April 1993
- 'North Market Street Mashpee, Massachusetts Prepared for Fields Point Limited Partnership' Drawing Number U2 – Sewer Profiles, Pump Sta. Details', prepared by Dufresne-Henry, Inc., dated April 1993 (1993 North Market Street Drawings)
- 'The Neighborhoods of Mashpee Commons Draft Environmental Impact Report and Final Development of Regional Impact Submittal EOEA Number 5913, prepared by the Mashpee Commons Limited Partnership, dated March 15, 2000. (MCLP DEIR)
- 'Final Environmental Impact Report (FEIR), Mashpee Commons Neighborhoods File No. 1247 – Chapter 8: Water Resources,' prepared by Sanborn, Head & Associates Consulting Engineers & Scientists, dated December 29, 2000
- 'Mashpee Commons Limited Partnership Municipal Area Sewer Extension' prepared by Dufresne-Henry, dated September 2004
- 'Town of Mashpee, Popponesset Bay & Waquoit Bay East Watersheds Needs Assessment Report – Final Report', prepared by Stearns & Wheler, dated April 2007
- 'Mashpee Commons Limited Partnership (MCLP) Mashpee, Massachusetts Contract Drawings – Phase 3 WWTP Upgrade' DEP Submission Drawings (Not for Construction), prepared by Stantec Consulting Services Inc., dated June 2007 (Phase 3 Design Drawings)
- 'Wastewater Treatment Plant Modifications Mashpee Commons WWTP, Mashpee, Massachusetts Engineer's Report (WP68 Permit), prepared by Stantec Consulting Services Inc., dated August 2007 (2007 Engineers Report)
- 'Jobs & Whiting Road Commercial Space – Mashpee Planning Board Special Permit Application', prepared by the Jobs/Whiting Trust, dated December 2007
- Proposed Wastewater Treatment Facility Wampanoag Village' drawings, prepared by the Norfolk-Ram Group, LLC, dated February 2011

- ‘Proposed Sewer Design System for “Wampanoag Village” Meetinghouse Road Mashpee, Massachusetts’ drawings, prepared by the Norfolk Ram Group, LLC, dated April 15, 2011
- ‘Mashpee Commons Limited Partnership Mashpee, Massachusetts Contract Drawings Phase 3 WWTP Upgrade’ – Issued for Permit, prepared by Stantec Consulting Services Inc., dated August 2013 (Phase 3 Permit Drawings)
- ‘Town of Mashpee Final Recommended Plan / Final Environmental Impact Report’, prepared by GHD, dated May 2015
- Operations data collected and compiled by MCLP WWTF staff over the time period January 2013 – 2016

The following documents were requested, but not provided to GHD:

- As-Built Drawings of the collection, treatment, and recharge facilities, and related upgrade for the Mashpee Commons WWTF.
- Copies of manufacturer’s data for the Mashpee Commons WWTF and Wampanoag WWTF.
- Equipment Operations & Maintenance Manuals for the Mashpee Commons WWTF and Wampanoag WWTF.

1.5 Report Organization

The Town of Mashpee Existing WWTF Evaluation Report is divided into 7 Sections:

Section 1 presents general introductory information about the project.

Section 2 describes the effluent limits and performance requirements for both facilities (Wampanoag WWTF and MCLP WWTF) and summarizes the known planning efforts associated with each facility.

Section 3 provides a treatment capacity analysis for each major treatment process at the Wampanoag WWTF and MCLP WWTF.

Section 4 reviews the estimated flows for each facility, compares various approaches used for estimating wastewater flows for new/future developments, and identifies future expansion potential based on the available information for each system.

Section 5 describes the Study Areas, estimated flow from each study area, and evaluates the capacity of the existing collection system(s) to take additional flow from the Study Areas. The Section also discusses collection system options, pump station considerations and estimated costs for proposed collection systems to serve these, and a potential facility at Site 4. The conceptual plans and profiles were provided under separate cover.

Section 6 updates the WNMP nitrogen loading based on the location of proposed recharge and the updated flows.

Section 7 summarizes the findings of the report and identifies recommendations and next steps.

1.6 References

The following industry design guidelines were used in the development of this report:

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

2. Commonwealth of Massachusetts Department of Environmental Protection Division of Watershed Permitting, Guidelines for the Design, Construction, Operation and Maintenance of Small Wastewater Treatment Facilities with Land Disposal, Revised November 2014

This report: has been prepared by GHD for the Town of Mashpee and may only be used and relied on by the Town of Mashpee for the purpose agreed between GHD and the Town of Mashpee as set out in this report.

GHD otherwise disclaims responsibility to any person other than the Town of Mashpee arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report). GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by the Town of Mashpee, Mashpee Commons Limited Partnership, the Wampanoag Tribe, and others who provided information to GHD (including Government authorities), which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

GHD has prepared the preliminary cost estimates set out in section 5 of this report ("Cost Estimate") using information reasonably available to the GHD employee(s) who prepared this report; and based on assumptions and judgments made by GHD.

The Cost Estimate has been prepared for the purpose of evaluating collection system alternatives and must not be used for any other purpose.

The Cost Estimate is a preliminary estimate only. Actual prices, costs and other variables may be different from those used to prepare the Cost Estimate and may change. Unless as otherwise specified in this report, no detailed quotation has been obtained for actions identified in this report. GHD does not represent, warrant or guarantee that the project can or will be undertaken at a cost which is the same or less than the Cost Estimate.

Where estimates of potential costs are provided with an indicated level of confidence, notwithstanding the conservatism of the level of confidence selected as the planning level, there remains a chance that the cost will be greater than the planning estimate, and any funding would not be adequate. The confidence level considered to be most appropriate for planning purposes will vary depending on the conservatism of the user and the nature of the project. The user should therefore select appropriate confidence levels to suit their particular risk profile.

2. Background

Section 2 summarizes the performance requirements and available planning information for the Wampanoag WWTF and the Mashpee Commons Limited Partnership (MCLP) WWTF.

2.1 Wampanoag WWTF

The Wampanoag WWTF is a private wastewater treatment facility, located on Great Neck Road North and owned by the Wampanoag Tribe. The facility has a design flow of 40,000 gallons per day (gpd) and was constructed in 2011 to serve the wastewater needs of the proposed Wampanoag Village housing development. This development has not yet been constructed and the WWTF currently does not receive any flow. The wastewater treatment facility is a Rotating Biological Contactor (RBC) system with subsurface effluent disposal and provisions for denitrification and disinfection.

2.1.1 Effluent Limits and Performance Requirements

The Wampanoag WWTF is currently governed by MassDEP Permit No. 918-0, dated July 4, 2011 (“Wampanoag WWTF Permit”). The expiration date of the permit was extended for four years beyond its listed expiration date of 2016 by the Permit Extension Act (Section 173 of Chapter 240 of the Acts of 2010). The facility is permitted to treat wastewater generated by the proposed 119-bedroom Mashpee Wampanoag Village housing development (which has not been constructed yet) and 25,000 gpd of off-site nitrogen mitigation flow. To date, the facility does not receive any flow.

The Wampanoag WWTF permit outlines effluent requirements for two phases of development, as follows:

- **Phase 1** - the effluent flow is restricted to 9,900 gpd and construction is limited to 90 bedrooms. The Permit notes that “the permittee has provided calculations to show that the discharge will not exceed 10 mg/L at the property boundary”. Effluent limits are not specifically outlined for Phase 1.
- **Phase 2** - the effluent flow limit is 39,999 gpd.

As outlined in the Permit, Phase 1 was intended to serve as an interim treatment facility. The Wampanoag WWTF Permit requires that a nitrogen reduction strategy be developed to offset the expected nitrogen load of the housing development prior to the construction of the 91st bedroom in the development, which signifies the end of Phase 1. The Permit notes that a potential nitrogen reduction strategy is to accept and treat sewage from local properties in the Popponesset Bay watershed currently serviced by on-site sewage disposal systems. The Permit stipulates that the proposed nitrogen offset plan needs to be submitted to and approved by MassDEP prior to the implementation of Phase 2.

Conditions of the discharge permit for Phase 2 are summarized in Table 1. In addition, the facility is required to monitor Fecal Coliform monthly but does not have an effluent disinfection limit.

Table 1 Wampanoag WWTF Phase 2 - Permit Effluent Limits

Effluent Characteristics	Discharge Limitations
Flow	39,999 gpd
Oil and Grease	15 mg/L
Total Suspended Solids (TSS)	30 mg/L
Total Nitrogen (NO2 + NO3 + TKN)	10 mg/L
Nitrate-Nitrogen	10 mg/L
Biochemical Oxygen Demand, 5-day @ 20 deg C (BOD5)	30 mg/L
Notes: (1) Permit limits as listed in MassDEP Permit No 918-0, dated July 4, 2011.	

2.1.2 Known Wampanoag WWTF Planning Efforts

As outlined in the Wampanoag WWTF Permit, the original design intent for the facility was for Phase 1 of the development to be served by an interim treatment system. However, due to available funding through the American Recovery and Reinvestment Act (ARRA) in 2010, the facility was constructed to treat the entire Phase 2 design flow without constructing the interim (Phase 1) treatment system.

Since the Wampanoag Tribe, to our knowledge, has not prepared an update to the planning efforts described above, this analysis is based on the wastewater flow designations outlined in the WWTF design drawings. The 2011 drawings designate 15,000 gpd for the proposed Wampanoag Village and 25,000 gpd to be available for off-site nitrogen mitigation at the end of Phase 2.

It should be noted that the Tribe is currently involved in ongoing litigation concerning its status regarding Land-in-Trust. The findings of the litigation may affect whether the Wampanoag WWTF is subject to the terms of the MassDEP permit.

2.2 Mashpee Commons WWTF

The MCLP WWTF is a private wastewater treatment facility located on Great Neck Road South, south of the Mashpee rotary. The facility currently serves Mashpee Commons and three municipal buildings on Frank E. Hicks Road (police department, fire department, and the Senior Citizen's Center) and the Public Library.

The WWTF was originally constructed in 1988 as an RBC wastewater treatment facility with effluent filtration, UV disinfection, and effluent disposal via open sand beds. The facility has been upgraded several times. During its last major upgrade project (Phase 3), which was completed in 2014, the treatment process was upgraded to a Membrane Bioreactor (MBR) designed to treat a peak day flow of 180,000 gpd.

2.2.1 Effluent Limits and Performance Requirements

The Mashpee Commons WWTF (MCLP WWTF) is governed by MassDEP Permit No. 306-4, dated July 14, 2009 ("MCLP Permit"). The expiration date of the permit was extended for four years beyond its listed expiration date of 2014 by the Permit Extension Act (Section 173 of Chapter 240 of the Acts of 2010). The Permit authorizes discharges from the wastewater treatment facility at Mashpee Commons, which is defined in the Permit as "a mixed use retail facility including 340,458 square feet of retail space, 42 residential bedrooms, municipal buildings and 100,000 gpd of proposed expansion". Conditions of the discharge permit are summarized in Table 2. The facility is required to monitor UV intensity daily, but does not have any effluent disinfection limits outlined in the MCLP Permit.

Table 2 Mashpee Commons WWTF Permit Effluent Limits

Effluent Characteristic	Discharge Limitations
Flow	180,000 gpd
Oil and Grease	15 mg/L
Total Suspended Solids (TSS)	30 mg/L
Total Nitrogen (NO2 + NO3 + TKN)	10 mg/L
Nitrate-Nitrogen	10 mg/L
Biochemical Oxygen Demand, 5-day @ 20 deg C	30 mg/L
Source: MassDEP Permit No 306-4, dated July 14, 2009.	

2.2.2 Known Planning Efforts

Available planning documents for the MCLP WWTF were reviewed to determine the methodology used to establish present and future design flows, and to identify future anticipated flows at the facility.

MCLP Draft Environmental Impact Report (March 2000)

The "Neighborhoods of Mashpee Commons Draft Environmental Impact Report and Final Development of Regional Impact Submittal" (MCLP DEIR), dated March 2000 indicates that the design flow at the MCLP WWTF is planned to increase over four phases (Table 3).

According to the MCLP DEIR, the design Title 5 peak flow for the proposed full Mashpee Commons development is 250,000 gpd[1]. To date, Phases 1 to 3 have been constructed. The current design flow of the MCLP WWTF is 180,000 gpd.

Table 3 MCLP WWTF Construction Phases

Phase	Design Flow (gpd)	Year of Construction
Phase 1	38,000	1985
Phase 2	80,000	1993
Phase 3	180,000	2007
Phase 4	250,000	Not yet constructed
Source: 'The Neighborhoods of Mashpee Commons Draft Environmental Impact Report and Final Development of Regional Impact" Submittal EOEA Number 5913, prepared by the Mashpee Commons Limited Partnership, dated March 15, 2000. (MCLP DEIR)		

MCLP Final Environmental Impact Report (December 2000)

The Final Environmental Impact Report (FEIR), dated December 2000, outlines a modified flow and nitrogen load development methodology based on the comments received during the review of the MCLP DEIR. The revised methodology is intended to "focus on the more realistic annual average flow rates for both existing and future properties when evaluating the "no net increase" requirement

[1] The final anticipated buildout flow for the MCLP WWTF has been modified several times in the planning documents. The most recently available planning document (the 2007 Phase 3 design drawings) indicates that the "buildout" flow for the MCLP WWTF is 280,000 gpd. As discussed later in this report the 280,000 gpd flow is based on an estimated future maximum flow that could be treated at the existing MCLP WWTF site, and to our knowledge is not based on a development "buildout" flow projection for Mashpee Commons.

of the [Cape Cod] Commission.” The FEIR indicates that the full buildout of Mashpee Commons was expected to occur by 2015.

According to the report, the storage, treatment, and disposal capacity of the WWTF will be designed based on peak monthly average flows. Peak monthly flows for the three target buildout dates are outlined in Table 4; the Table also presents the properties that were anticipated to be connected to the Mashpee Commons collection system as part of this planning effort. The FEIR outlines nine municipal and quasi-municipal properties intended to be connected to the MCLP WWTF and designates a flow allocation for “non-specific off-site flow”. The non-specific wastewater flow was allocated to serve off-site properties in the watershed that are currently on septic systems.

The FEIR for the Mashpee Commons Neighborhoods states that MCLP intends to transfer its WWTP and sewer system to the Town “whenever the Town is prepared to accept the ownership and operational responsibilities for the system”.

Table 4 MCLP Design Flows as Outlined in FEIR

	Property	2005 Buildout Flow (gpd)	2010 Buildout Flow (gpd)	2015 Buildout Flow (gpd)
Commercial Properties	Village Center	29,200	29,200	29,200
	North Market Street 1	2,700	2,700	2,700
	East Steeple Street	4,800	10,100	10,600
	Jobs Fishing Road	2,800	9,300	9,300
	North Market Street 2	4,000	5,300	5,300
	Trout Pond	0	1,200	2,900
Hotels	East Steeple Street Hotel	13,200	13,200	13,200
	Jobs Fishing Road Hotel	15,400	15,400	15,400
Residential Properties	Village Center	8,300	12,200	12,200
	Whiting Road	23,600	31,100	31,100
	East Steeple	3,400	5,400	5,400
	Jobs Fishing	12,200	19,800	29,700
	Trout Pond	0	7,900	14,500
Municipal and Quasi-Municipal Properties	Police Department	800	900	900
	Fire Department	700	700	800
	Mashpee Public Library	200	300	300
	Library Extension	400	400	400
	Coombs Elementary School	2,800	3,000	3,100
	Quashnet Middle School	3,100	3,300	3,500
	Homeyer Village	2,500	2,700	2,800
	Boys and Girls Club	300	300	300
	Christ the King Church	1,800	1,800	1,900
	Non-Specific Off-Site	7,400	13,800	17,500
Total		**139,600**	**190,000**	**213,00**

Source: ‘Final Environmental Impact Report (FEIR), Mashpee Commons Neighborhoods File No. 1247 – Chapter 8: Water Resources,’ prepared by Sanborn, Head & Associates Consulting Engineers & Scientists, dated December 29, 2000

MCLP Phase 3 Planning Documents

The MCLP Phase 3 upgrade, which was designed in 2007, is based on Title 5 design flows, as outlined in Table 5.

Flow was allocated for the four municipal buildings listed in Table 5 (Fire Department, Police Station, Senior Center, and Public Library). All four municipal buildings have been connected to the MCLP WWTF.

The 2007 Phase 3 Design Drawings show space allocated for the future proposed layout for Phase 4 and indicate that the design capacity of the Phase 4 upgrade will be 280,000 gpd. The 2013 Phase 3 Permit Drawings were also reviewed during this evaluation. The Permit Drawings outline the Phase 3 process equipment layout but do not outline any design flows.

The Phase 3 WP-68 report indicates that planning of the proposed Mashpee Commons area expansion is in the preliminary planning stages and that the anticipated 100,000 gpd expansion for Phase 4 is based primarily on the hydrogeological evaluation for the site (the overall effluent disposal capacity of the site has been estimated to be 280,000 gpd). MCLP has indicated that the site is limited by effluent disposal capacity and has estimated that the treatment capacity of the remaining treatment processes could be doubled to roughly 550,000 gpd while staying within the existing property footprint. An alternate effluent disposal site would need to be identified if the capacity of the facility were to exceed 280,000 gpd.

Table 5 MCLP WWTF Phase 3 Design Flows

Phase	Property	Title 5 Flow (gpd)
Phase 1 & 2	Mashpee Commons properties connected prior to 2007	44,177
Phase 1 & 2	North Market Street properties connected prior to 2007	11,086
Phase 1 & 2	Municipal Building – Fire Department	640
Phase 1 & 2	Municipal Building – Police Department	685
Phase 1 & 2	Municipal Building – Senior Center	650
Phase 1 & 2	Municipal Building – Public Library	1,500
Phase 1 & 2	Mashpee Commons Phase I	15,678
Phase 1 & 2	Mashpee Commons Phase II	5,584
Phase 3	Mashpee Commons Phase II – Frank E. Hicks Road	10,296
Phase 3	Jobs Fishing Road Phase I	13,200
Phase 3	Jobs Fishing Road Phase II	13,860
Phase 3	Whiting Road Phase III	11,880
Phase 3	Whiting Road Phase IV	19,140
Phase 3	Whiting Road Phase V	12,760
Phase 3	Whiting Road Phase VI	5,940
Phase 3	Whiting Road Phase VII	7,260
Phase 3	Misc./Unidentified Flow Allowance	5,664
Total Design Flow		180,000
Source: Wastewater Treatment Plant Modifications Mashpee Commons WWTP, Mashpee, Massachusetts Engineer's Report (WP68 Permit) – Table 2, prepared by Stantec Consulting Services Inc., dated August 2007 (2007 Engineers Report)		

Other Known Planning Efforts

1. Jobs & Whiting Neighborhood

A permit application was submitted to the Mashpee Planning Board for the ‘Jobs & Whiting Road Commercial Space’ in December 2007 following the Mashpee Zoning Board of Appeals February 1, 2007 approval of a Chapter 40B Comprehensive Permit for the 382-unit residential portion of the development. The Permit Application outlines the following Title 5 flows for the proposed development:

- Title 5 Flow = 99,500 gpd
- Peak Hour Flow = 398,200 gpd

The total Title 5 flow for the Jobs & Whiting properties indicated in the Permit Application is higher than the flow allocated for the two properties in the 2007 Phase 3 design (the overall flows increase by 15,510 gpd). It is presumed that the Permit Application flow reflects updated plans for the development. The Permit Application also noted that a portion of the new flow from the Jobs & Whiting properties is intended to flow to the existing Steeple Street Pump Station and that the remainder is expected to flow to a new pump station. MCLP has indicated that the proposed location of the pump station is near the Quashnet River; however plans have not yet been developed for the station.

The new pump station capacity, which includes an allowance for future capacity, is listed as 432,000 gpd. The document notes that the capacity of the existing pump station on Steeple Street will need to be evaluated to determine whether it can handle the additional flow or if an upgrade is required.

MCLP has indicated that the Jobs & Whiting neighborhood was permitted in seven phases. Phase 1A, which consists of eight buildings with 52 overall units, is expected to be constructed by the end of 2018. A construction date is not known for subsequent phases.

2. Northbridge Assisted Living Facility

The Town has also been notified that the MCLP WWTF has reserved 8,180 gpd for the recently constructed Northbridge Assisted Living Facility. The Northbridge development is located within the Trout Pond neighborhood. It should be noted that flow was not allocated for the Trout Pond neighborhood in the Phase 3 flow allocation.

3. MCLP Master Plan Update

MCLP has indicated that Mashpee Commons is currently updating its Master Plan and that the revised Mashpee Commons development projections are anticipated to require the entire capacity of the future Phase 4 facility (280,000 gpd).

Summary of Known MCLP WWTP Planning Efforts

Available planning documents for the MCLP WWTF are summarized below:

- The MCLP DEIR outlined anticipated buildout flows for the MCLP WWTF, which were developed based on Title 5 design flows. The anticipated buildout flow is listed as 250,000 gpd.
- Based on review comments received during the review of the MCLP DEIR, the anticipated buildout flows were revised in the MCLP FEIR to reflect “the more realistic annual average flow rates for both existing and future properties.” The anticipated buildout flow listed in the MCLP FEIR is 213,000 gpd.

- The 2007 Phase 3 design drawings outline design flows for the Phase 3 upgrade, which are based on Title 5 design flows (since the 2013 Permit Phase 3 drawings do not list any design flows, the 2007 design drawings are the most recent available planning document for the facility). The Phase 3 design drawings list the design flow of the facility as 180,000 gpd and indicates the future Phase 4 design capacity is anticipated to be 280,000 gpd. The Phase 4 design capacity represents the anticipated maximum effluent disposal capacity on the MCLP WWTF site and is not, to our knowledge, based on a buildout flow projection. MCLP has indicated that the site is limited by effluent disposal capacity and has estimated that the treatment capacity of the remaining treatment processes could be doubled to roughly 550,000 gpd within the existing footprint of the property. An alternate effluent disposal site would need to be identified if the facility's capacity were to exceed 280,000 gpd.
- Phase 1A of the Jobs & Whiting neighborhood is anticipated to be completed in 2018. Phase 1A consists of eight buildings with 52 overall units that will be connected to the MCLP WWTF.
- MCLP has reserved 8,180 gpd for the recently constructed Northbridge Assisted Living facility.
- MCLP is currently updating its Master Plan and has indicated that the entire capacity of the future Phase 4 facility (280,000 gpd) is anticipated to be required by the revised Mashpee Commons development plant.

Since MCLP, to our knowledge, has not prepared a formal update to the planning efforts described above, this analysis is based on the design flows listed in the 2007 Phase 3 design drawings, which is the most recent planning document available.

3. Treatment Facility Evaluation

A capacity evaluation was conducted to determine the theoretical maximum treatment capacity of the Wampanoag WWTF and MCLP WWTF. Major unit processes were evaluated against current industry design practices, as published in the following references:

- *Commonwealth of Massachusetts Department of Environmental Protection Division of Watershed Permitting, Guidelines for the Design, Construction, Operation and Maintenance of Small Wastewater Treatment Facilities with Land Disposal, Revised November 2014 (referred to in this document as the" small WWTF guidelines").* This reference is applicable to facilities with a design flow between 10,000 and 150,000 gpd and was used to evaluate the Wampanoag WWTF.
- *New England Interstate Water pollution Control Commission, TR-16: Guides for the Design of Wastewater Treatment Works, 2016 Edition (referred to in this document as "TR-16").* This reference is applicable to facilities with a design flow greater than 150,000 gpd and was used to evaluate the MCLP WWTF.

The discussion of each major unit process at each facility is divided into two sections, as follows:

A. **Industry Guidelines** outlines industry sizing and capacity guidelines for the unit process based on the above references.

B. **Installed Capacity** describes the capacity of the infrastructure installed at each WWTF and provides an analysis of the theoretical maximum unit capacity for the process as allowable by industry guidelines referenced above.

3.1 Wampanoag WWTF Treatment Capacity Analysis

The Wampanoag WWTF is an RBC-type system with a design flow of 40,000 gpd. Design criteria for the facility are outlined in Table 6.

Table 6 Wampanoag WWTF – Design Criteria

Design Effluent Characteristic	Permit Limit (mg/L)	Design Target (mg/L)
Total Biochemical oxygen demand (BOD)	< 30	< 20
Total Suspended Solids (TSS)	< 30	< 10
Total Nitrogen (TN)	< 10	< 5
Nitrate Nitrogen (as Nitrogen)	< 10	< 5
Ammonia-Nitrogen	N/A	< 2
Source: 'Proposed Wastewater Treatment Facility Wampanoag Village', prepared by Norfolk-Ram, dated February 2011		

The facility's major treatment processes are outlined in Figure 1. Each major treatment process is evaluated against the guidelines outlined in the "small WWTF guidelines" in this section.

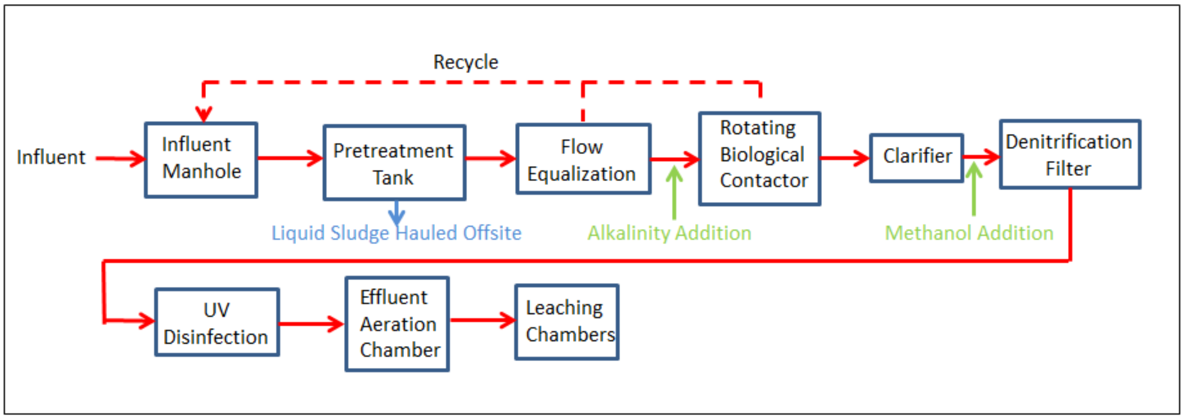


Figure 1 Wampanoag WWTF Process Schematic

3.1.1 Wampanoag WWTF - Process Redundancy Requirements

The "small WWTF guidelines" specifies that multiple treatment units be provided whenever the average daily flow exceeds 40,000 gpd. The Wampanoag WWTF has a design flow of 40,000 gpd, which is based on Title 5 flows. In accordance with the "small WWTF guidelines", Title 5 flows represent maximum day flows. The average daily flow of the facility is anticipated to be less than 40,000 gpd; therefore, the facility does not meet this flow threshold.

3.1.2 Wampanoag WWTF - Pretreatment / Primary Settling

The Wampanoag WWTF has two 20,000 gallon septic tanks that serve as pre-treatment tanks. The facility was also designed to utilize the septic tanks for sludge storage.

A. Industry Guidelines

The "small WWTF guidelines" outlines the following minimum recommended primary settling capacities based on the intended configuration of the system as follows:

- At least 75% of the estimated design flow if either garbage grinders are used within the collection system or the septic tanks are being used for sludge storage.
- At least 100% of the estimated design flow if both garbage grinders are used within the collection system and the septic tanks are being used for sludge storage.

Based on the 75% guidance and the permitted design flow of 39,999 gpd, the Wampanoag WWTF requires approximately 30,000 gallons of primary settling volume (when considering sludge storage without garbage grinders).

B. Installed Capacity

The Wampanoag WWTF has two septic tanks that provide a combined storage capacity for 40,000 gallons, which is equal to 100% of the design flow. The 100% capacity is assumed to have been installed to allow both sludge storage and garbage grinders to be used within the system.

If it is assumed that garbage grinders will not be installed in the future development, the facility currently has approximately 25% excess storage capacity and could theoretically provide preliminary treatment for a design flow up to 53,000 gpd (40,000 existing storage capacity/75% minimum recommended primary settling capacity for systems that use the septic tanks for sludge storage).

Table 7 Wampanoag WWTF - Primary Settling Unit Process Analysis

Capacity	Unit Capacity
MassDEP Recommended Capacity (75% of design flow)[1]	39,999 gpd x 0.75 = 30,000 gpd
WWTF Capacity	40,000 gpd (100% of design flow)
Maximum Unit Treatment Capacity per DEP Guidelines[1]	53,000 gpd
Source: ‘Proposed Wastewater Treatment Facility Wampanoag Village’, prepared by Norfolk-Ram, dated February 2011 Notes: 1. Capacity assumes no garbage grinders are installed in the future development.	

C. Summary

The septic tank pretreatment system has adequate capacity for the design flow of the facility. If garbage grinders are not installed in the future development (which has not been constructed yet), the process could potentially treat a higher flow of up to 53,000 gpd. If garbage grinders are installed in the future development, the unit process capacity of the pretreatment system is 40,000 gpd.

3.1.3 Wampanoag WWTF - Flow Equalization

The Wampanoag WWTF utilizes one 20,000 gallon septic tank for flow equalization. Flow enters the flow equalization tank by gravity.

A. Industry Guidelines

For treatment plants with a design flow less than 40,000 gpd, the “small WWTF guidelines” recommends that flow equalization capacity be provided for 50% of the design flow.

B. Installed Capacity

The minimum flow equalization capacity required at the Wampanoag WWTF is 20,000 gpd (39,999 (design flow) x 50% = 19,999 gpd (~20,000 gpd)). It should be noted that this facility is on the threshold of the capacity range of 40,000 gpd to 100,000 gpd, which allows for a 33% capacity.

As outlined in Table 8, the system is designed to provide the required capacity. If the facility were modified to treat greater than 40,000 gpd, the equalization requirement could be reduced to 33%, allowing the current configuration to be used for flows up to 61,000 gpd (20,000 gpd / 33%).

Table 8 Wampanoag WWTF - Flow Equalization Unit Process Analysis

Parameter	Unit Capacity
MassDEP Recommended Capacity (for flows less than 40,000 gpd)	20,000 gpd
Flow Equalization Capacity	20,000 gpd
Source: ‘Proposed Wastewater Treatment Facility Wampanoag Village’, prepared by Norfolk-Ram, dated February 2011	

C. Summary

The facility has adequate flow equalization capacity for the design flow of the facility. The facility's design flow is just under the "small WWTF guidelines" flow threshold, which allows facilities with design flows over 40,000 gpd to reduce their flow equalization capacity (to 33% instead of 50%).

If the facility were modified to treat a greater flow, the equalization capacity requirements of the facility could be reduced and the unit process capacity of the existing infrastructure would increase to 61,000 gpd.

3.1.4 Wampanoag WWTF - Biological Treatment

The Wampanoag WWTF has an RBC for nitrogen removal, which is divided into four stages as outlined in Table 9. The design drawings indicate that the RBC was designed to meet a target effluent BOD limit of 20 mg/L (which is lower than the current permit limit of 30 mg/L). To maintain consistency with the design, the lower effluent BOD target (20 mg/L) was also used to evaluate the installed capacity of the existing equipment at the Wampanoag WWTF in this report. The design drawings indicate that the facility has two RBC pumps, each with a rated capacity of 150 gpm at 21 total dynamic head (TDH).

Table 9 Wampanoag WWTF RBC Surface Area

Parameter	Surface Area (Square Feet)
Stage 1	36,000
Stage 2	22,950
Stage 3	27,900
Stage 4	27,900
Total Effective Surface Area	114,750
Sources: Proposed Wastewater Treatment Facility Wampanoag Village' drawings, prepared by Norfolk-Ram, dated February 2011	

A. Industry Guidelines

The "small WWTF guidelines" outlines the following sizing criteria, which are applicable to the Wampanoag WWTF:

- The organic loading rate allowable for an effluent BOD target of 20 mg/L is 1.25 lb soluble BOD5/day/1000 SF.
- Since the WWTF utilizes a septic tank for pre-treatment, the required surface area for BOD removal needs be increased by 50%.
- The "small WWTF guidelines" recommends additional surface area be provided for systems that require nitrification. A typical design value is 0.24 lbs ammonia removed/day/1000 SF.
- For systems designed for nitrification and denitrification, the RBC should have a minimum of four stages.
- First stage loading should not exceed 4.0 lbs soluble BOD5/day/1000 SF.

B. Installed Capacity

As outlined in Table 10, the MassDEP recommended minimum RBC media surface area for the Wampanoag WWTF, which has septic tank pretreatment and nitrification, is 113,770 SF. The maximum flow that can theoretically be treated with the installed surface area (114,750 SF) is approximately 40,300 gpd, which is slightly greater than the design flow of the facility.

Table 10 Wampanoag WWTF – Minimum RBC Sizing per "small WWTF guidelines"

Parameter	Calculation	Minimum Surface Area Required per the "small WWTF guidelines"
Surface Area Required for SBOD Removal • Influent Soluble BOD Loading = 45 lb/d[(1)] • Allowable Loading for effluent BOD design limit of 20 mg/L = 1.25 lbs soluble BOD/day/1000 SF	[(45 lb/d) x (1,000 SF)]/ (1.25 lbs soluble BOD/day/1000 SF) = 36,000 SF	36,000 SF
Surface Area Required for Processes with Septic Tank Pre-Treatment • Required Surface Area increase for WWTFs with septic tank pre-treatment = 50% increase in surface Area[(2)]	(36,000 SF) x 50%	18,000 SF
Surface Area Required for Nitrification • Influent Ammonia Loading = 14.3 lb/d[(1)] • Allowable Ammonia Loading = 0.24 lb/d/1000 SF[(2)]	[(14.3 lb/d)/(0.24 lb/d/1000 SF)] * 1000 SF	59,770 SF
Total Required Surface Area For RBC with Septic Tank Pretreatment and Nitrification (36,000 +18,000 + 59,770)		**113,770 SF**
Sources: 1. 'Proposed Wastewater Treatment Facility Wampanoag Village', prepared by Norfolk-Ram, dated February 2011 – Sheet 7 2. Commonwealth of Massachusetts Department of Environmental Protection Division of Watershed Permitting, Guidelines for the Design, Construction, Operation and Maintenance of Small Wastewater Treatment Facilities with Land Disposal, Revised November 2014		

C. Summary

The facility has adequate biological treatment capacity for the design flow of the facility. The maximum flow that can theoretically be treated with the RBC is 40,300 gpd, which is slightly higher than the design flow of the facility. The RBC feed pumps have adequate capacity for the design flow.

3.1.5 Wampanoag WWTF - Secondary Settling

The Wampanoag WWTF has one 10-foot diameter secondary clarifier, which was designed for a peak surface overflow rate of 600 gpd/sf. The clarifier is 10 feet deep. The facility has two airlift blowers for sludge and scum air lift in the clarifier. A capacity evaluation was not conducted on the air lift system as this is not considered a major process.

A. Industry Guidelines

MassDEP recommends that secondary clarifiers be sized for a surface overflow rate (SOR) of less than 1,000 gpd/sf. The selection of a design SOR is highly dependent on how well the sludge is anticipated to settle in the clarifier. Small treatment facility clarifiers typically experience a poorer settling rate than larger facilities. Typical clarifier depth range is 10 to 15 feet.

B. Installed Capacity

The secondary clarifier was sized based on a SOR of 600 gpd/sf, which is lower than the maximum allowable SOR. If the anticipated SOR was raised to 1,000 gpd/sf (the maximum allowable SOR) the secondary clarifier could theoretically process a higher flow, up to 78,500 gpd (Table 11). However, due to the poor settling rate typically experienced in small treatment facilities, lack of redundancy in the system, and shallow depth of the existing clarifier, testing would need to be conducted to determine if the process can accommodate more flow.

Table 11 Wampanoag WWTF - Secondary Clarifier Unit Process Analysis

Parameter	Design Capacity	Theoretical Unit Capacity
Clarifier Surface Area (1)	78.5 SF	78.5 SF
Maximum SOR (2)	600 GPD(1)	1,000 GPD(2)
Theoretical Flow Capacity (would require testing to confirm)	47,100 GPD	78,500 GPD

Sources:

1. 'Proposed Wastewater Treatment Facility Wampanoag Village', prepared by Norfolk-Ram, dated February 2011 – Sheet 7
2. Commonwealth of Massachusetts Department of Environmental Protection Division of Watershed Permitting, Guidelines for the Design, Construction, Operation and Maintenance of Small Wastewater Treatment Facilities with Land Disposal, Revised November 2014

C. Summary

The facility has adequate secondary treatment capacity. Based on the design SOR the secondary clarifier can theoretically treat up to 47,100 gpd. The "small WWTF guidelines" allows for a secondary clarifier to be operated at a higher SOR; however, testing would need to be conducted so the SOR can be increased without adversely affecting the process.

3.1.6 Wampanoag WWTF - Denitrification

The WWTF has a two-cell denitrification filter with an overall surface area of 34.8 square feet. The process was designed to treat 50,000 gpd. The design drawings show two 55 gallon methanol storage drums and two positive displacement methanol feed pumps with a range of 0 to 30 gpd. The design drawings show two filter blowers.

A. Industry Guidelines

The Wampanoag WWTF design utilizes a de-nitrification filter, which has an allowable loading rate of 1.0 gpm/SF per the "small WWTF guidelines".

B. Installed Capacity

The denitrification filter loading at design (max day) flow is 0.8 gpm/SF (loading rate = (40,000 gpd / (34.8 SF x 1440 min/d)).

Based on the "small WWTF guidelines" allowable loading rate, the filters could theoretically treat 50,000 gpd of flow (Table 12). It should be noted that the process is designed with no redundancy and would exceed the allowable loading rate if one of the two cells were out of service at design flow.

Table 12 Wampanoag WWTF - Denitrification Filter Unit Process Analysis

Parameter	Unit Capacity
WWTF Surface Area (Overall) (1)	34.8 sf
MassDEP Allowable Loading Rate (2)	1.0 gpm/sf
Maximum Unit Capacity per the “small WWTF guidelines”	50,000 gpd
Sources: 1. ‘Proposed Wastewater Treatment Facility Wampanoag Village’, prepared by Norfolk-Ram, dated February 2011 – Sheet 7 2. Commonwealth of Massachusetts Department of Environmental Protection Division of Watershed Permitting, Guidelines for the Design, Construction, Operation and Maintenance of Small Wastewater Treatment Facilities with Land Disposal, Revised November 2014 (referred to in this document as “small WWTF guidelines”).	

C. Summary

The facility has adequate denitrification capacity for the design flow of the facility. The theoretical maximum capacity of the process is 50,000 gpd, but includes no redundancy and theoretically could only operate at 25,000 gpd with one filter unit out of service, which would only be sufficient to treat the average annual flow.

3.1.7 Wampanoag WWTF - Effluent Disinfection

The design drawings specify a UV system rated for 34.7 gallons per minute (gpm) (50,000 gpd).

A. Industry Guidelines

The “small WWTF guidelines” states that an ultraviolet (UV) disinfection system should consist of multiple banks of lamp modules and be able to continuously disinfect a wastewater treatment facility’s peak flow with one bank out of service. The facility’s permit requires that UV intensity be monitored daily, but does not include an effluent disinfection limit.

B. Installed Capacity

The design drawings specify a UV system rated for 34.7 gpm. The drawings do not indicate the configuration of the system and do not provide a capacity with one bank out of service.

Table 13 Wampanoag WWTF - Effluent Disinfection Unit Process Analysis

Parameter	Unit Capacity
UV Rated Capacity	34.7 gpm
Maximum Unit Capacity (unknown configuration)	50,000 gpd
Source: ‘Proposed Wastewater Treatment Facility Wampanoag Village’, prepared by Norfolk-Ram, dated February 2011	

C. Summary

The specified capacity of the UV disinfection system is 50,000 gpd. However, the capacity with one bank out of service is not listed in the design drawings. Therefore, it cannot be determined whether adequate disinfection capacity exists at the facility with one bank out of service based on the available documentation.

3.1.8 Wampanoag WWTF - Post Aeration

The WWTF utilizes a 2,500 gallon septic tank as a post-aeration tank. The WWTF design calculations indicate that the post aeration system was designed to provide 15 minutes of aeration time for every 60 minutes.

A. Industry Guidelines

The "small WWTF guidelines" does not provide guidelines for sizing mechanical post aeration systems. A 15 minute retention time is commonly recommended by post aeration system manufacturers.

B. Installed Capacity

The design calculations indicate that the post aeration system could theoretically aerate 60,000 gpd of flow (Table 14).

Table 14 Wampanoag WWTF - Post Aeration Unit Process Analysis

Parameter	Unit Capacity
Installed Capacity (2)	2,500 gpd
Detention Time (1)	60 minutes
Maximum Unit Capacity	60,000 gpd

Source:
1. 'Proposed Wastewater Treatment Facility Wampanoag Village', prepared by Norfolk-Ram, dated February 2011.
2. Commonwealth of Massachusetts Department of Environmental Protection Division of Watershed Permitting, Guidelines for the Design, Construction, Operation and Maintenance of Small Wastewater Treatment Facilities with Land Disposal, Revised November 2014.

C. Summary

The facility has adequate post aeration capacity for the design flow of the facility. Based on the design calculations, the process can treat up to 60,000 gpd.

3.1.9 Wampanoag WWTF - Effluent Disposal

Flow is pumped to the effluent disposal system by an effluent pump chamber. The design drawings indicate the effluent pump chamber has two submersible pumps, each rated for 150 gpm with a total dynamic head of approximately 10 feet. The facility has a leaching system made up of two halves. Each half is comprised of 11 trenches, with 16 chambers in each trench (352 chambers overall).

A. Industry Guidelines

The design drawings establish that the percolation rate of the site is less than two minutes/inch. The allowable design loading rate for the site, per "small WWTF guidelines", is 3.0 gpd/sf.

B. Installed Capacity

The design drawings indicate that the effluent pump chamber was designed to deliver eight doses of effluent per day (each day is 5,000 gallons). The design discharge rate for the pumps is 143 gpm.

The theoretical maximum unit capacity of the Wampanoag WWTF is 42,100 gpd (14,033SF x 3.0 gpd/SF). However, if the effective leaching area called out on the design drawings is used (13,860), this would equal 41,580 gpd.

Table 15 Wampanoag WWTF - Effluent Disposal Unit Process Analysis

Parameter	Unit Capacity
Effective Leaching Area	13,860 sf
Maximum Allowable Loading Rate	3.0 gpd/sf
Maximum Unit Capacity[1]	41,580 gpd
Source: 'Proposed Wastewater Treatment Facility Wampanoag Village', prepared by Norfolk-Ram, dated February 2011 Notes: 1. Maximum unit capacity based on the design drawings.	

C. Summary

The facility has adequate disposal capacity for the design flow of the facility. Based on the effective leaching area listed in the design drawings, the maximum unit capacity of the effluent disposal system is 41,580 gpd. The effluent pumps have adequate capacity for the design flow.

3.1.10 Wampanoag WWTF - Odor Control Facilities

The 2011 drawings do not show accommodations for odor control facilities.

3.1.11 Wampanoag WWTF - Summary

Table 16 summarizes the unit process capacity analysis for the Wampanoag WWTF. Plant capacity is determined based on the most limiting process. The currently limiting factor is flow equalization; however, if the design flow of the facility increased over to 40,000 gpd, the capacity of this process could be increased to 61,000 gpd. Therefore, the actual most limiting process at the facility is the RBC. If the design flow of the facility were increased, a second RBC would likely need to be installed. Due to the limited available space in the existing Process Building, a new Process Building would be needed to house the equipment.

Table 16 Wampanoag WWTF - Unit Process Capacity Analysis

Parameter	Maximum Unit Capacity (gpd)	Expansion Potential
Primary Settling	53,000	Unit process could be expanded through the installation of additional septic tanks in the yard.
Flow Equalization	40,000 (61,000)	Per the "small WWTF guidelines", if the design flow of the facility increases to over 40,000 gpd, the required flow equalization capacity decreases (from 50% to 33%) and the existing infrastructure unit capacity would increase to 61,000 gpd.
Biological Treatment	40,300	A second RBC would need to be installed, likely in a new process building.
Secondary Settling	47,100	Testing would be required to determine if increasing the SOR of the existing clarifier would adversely impact the process.
Denitrification	50,000	A second denitrification filter would need to be installed, likely in a new process building.
Post Aeration	60,000	Unit process could be expanded through the installation of an additional tank in the yard.
Effluent Disinfection	50,000	A second UV disinfection unit would need to be installed, likely in a new process building
Effluent Disposal	41,580	Additional leaching chamber capacity would need to be installed. A hydrogeological evaluation would be needed to determine if adequate effluent disposal capacity exists on the site.

3.2 MCLP WWTF Treatment Capacity Analysis

The most recent upgrade to the MCLP WWTF (Phase 3) was completed in 2014. The upgrade converted the WWTF from an RBC facility to a membrane biological reactor (MBR) facility. Design flows are outlined in Table 17.

Table 17 MCLP Phase 3 Design Flows

Parameter	Flow (gpd)
Peak Day	180,000
Maximum Month	140,000
Average Day (Summer)	120,000
Average Day (Winter)	70,000
Source: 'Wastewater Treatment Plant Modifications Mashpee Commons WWTP, Mashpee, Massachusetts Engineer's Report (WP68 Permit), prepared by Stantec Consulting Services Inc., dated August 2007 (2007 Engineers Report)	

Figure 2 outlines the major processes at the MCLP WWTF. Raw wastewater from the Mashpee Commons collection system flows to the Steeple Street Pump Station and is pumped to the MCLP WWTF. Each major process within the facility is outlined in the sections below.

As outlined previously, the "small WWTF guidelines" are intended to only be applicable for small facilities with a design flow between 10,000 gpd and 150,000 gpd. During its first two phases the MCLP WWTF design flow was within this range. However, during the last upgrade, the design flow of the MCLP WWTF has surpassed this range. The upgraded processes are evaluated using the guidelines outlined in TR-16, which is the document applicable for facilities with a design over 150,000 gpd with the exception of the effluent recharge facilities (open sand beds) which are evaluated on the 'small WWTF guidelines' since TR-16 does not cover this type of process. The blower system was not evaluated due to the proprietary nature of the design.

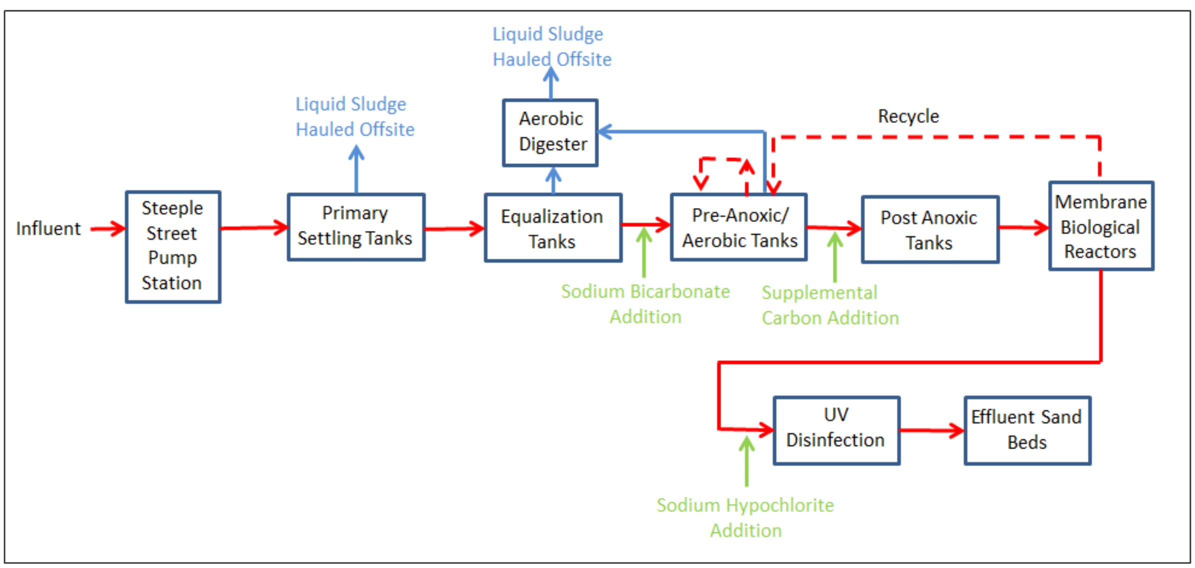


Figure 2 MCLP WWTF Process Schematic

3.2.1 MCLP WWTF Secondary Treatment Process Influent Characteristics

The GE Operations and Maintenance Manual does not include design loading information for the MBR process. For the purposes of this analysis, design loads to the secondary treatment process, which were estimated based on available information, are summarized in Table 18.

Table 18 MCLP WWTF – Influent Design Criteria

Influent Parameters	Average Daily			Maximum Month			
	Concentration (mg/L)[1]	Load – Summer (lb/d)[2]	Load – Winter (lb/d)[2]	Concentration (mg/L)[1]	Load (lb/d)[4]	Concentration (mg/L)[1]	Load (lb/d)[5]
BOD5	345	345	201	640	747	880	1321
TSS	120	120	70	240	280	430	646

Source:

1. 'ZeeWeed 500D Membrane System Vendor Data Manual, prepared by GE and dated December 2013.
2. Calculated using the average daily summer flow (120,000 gpd) listed in 'Wastewater Treatment Plant Modifications Mashpee Commons WWTP, Mashpee, Massachusetts Engineer's Report (WP68 Permit), prepared by Stantec Consulting Services Inc., dated August 2007 (2007 Engineers Report)
3. Calculated using the average daily winter flow (70,000 gpd) listed in 'Wastewater Treatment Plant Modifications Mashpee Commons WWTP, Mashpee, Massachusetts Engineer's Report (WP68 Permit), prepared by Stantec Consulting Services Inc., dated August 2007 (2007 Engineers Report)
4. Calculated using the maximum month flow (140,000 gpd) listed in 'Wastewater Treatment Plant Modifications Mashpee Commons WWTP, Mashpee, Massachusetts Engineer's Report (WP68 Permit), prepared by Stantec Consulting Services Inc., dated August 2007 (2007 Engineers Report)
5. Calculated using the maximum flow (180,000 gpd) listed in 'Wastewater Treatment Plant Modifications Mashpee Commons WWTP, Mashpee, Massachusetts Engineer's Report (WP68 Permit), prepared by Stantec Consulting Services Inc., dated August 2007 (2007 Engineers Report)

The Biowin model outputs in the WP-68 application indicates the following reductions, which are within the typical range listed in TR-16, are anticipated from the preliminary treatment system:

- BOD = 20% reduction (TR-16 typical primary clarifier BOD removal efficiency is listed as 25% to 40%)
- SS = 50% reduction (TR-16 typical primary clarifier TSS removal efficiency is listed as 40% to 70%)

MBR influent concentrations and loads, based on the reduction assumptions outlined above, and the facilities influent design criteria are shown in Figures 3 through 6. The data indicates the secondary process may be approaching its design capacity on a TSS loading basis.

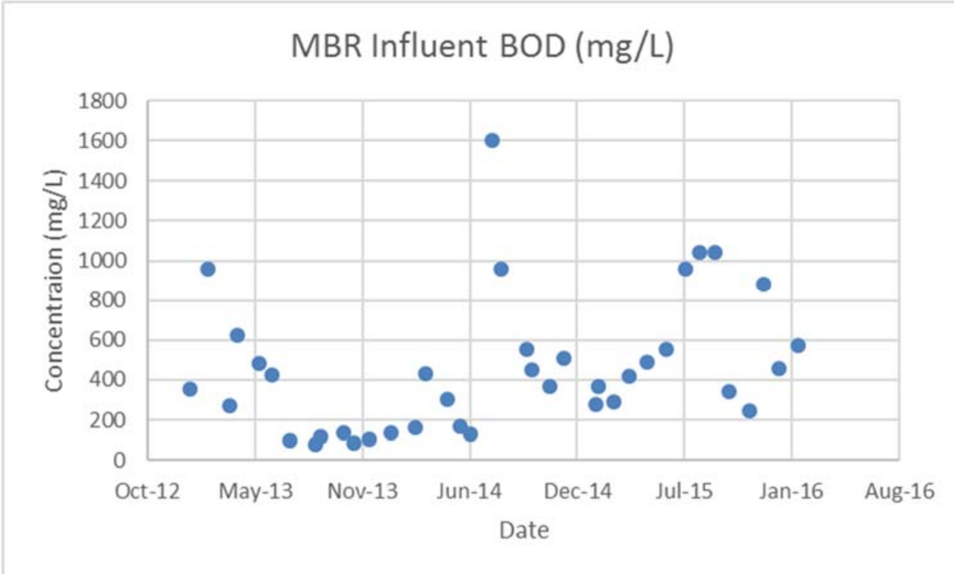


Figure 3 MCLP WWTF January 2013 – January 2016 MBR Influent BOD Concentration Data

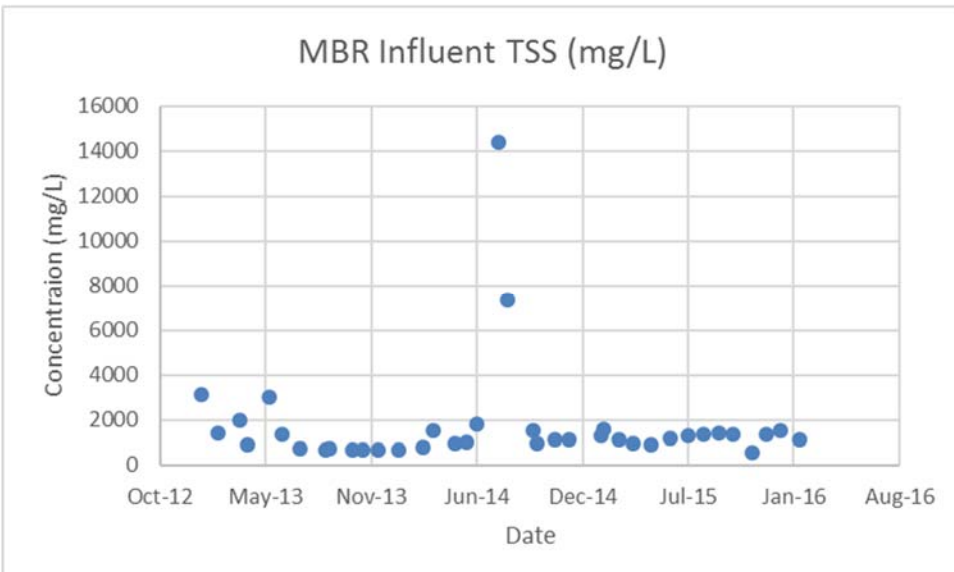


Figure 4 MCLP WWTF January 2013 – January 2016 MBR Influent TSS Concentration Data

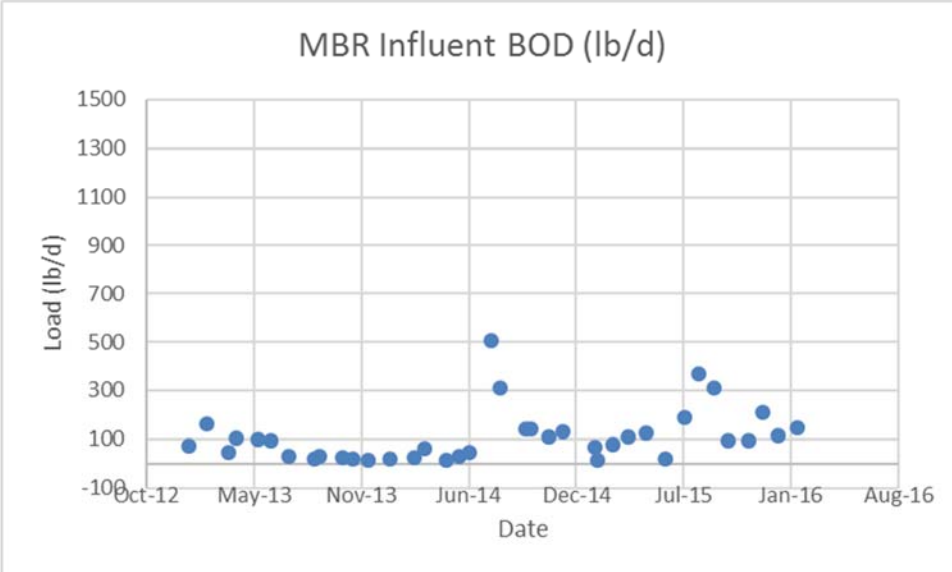


Figure 5 MCLP WWTF January 2013 – January 2016 MBR Influent BOD Load Data

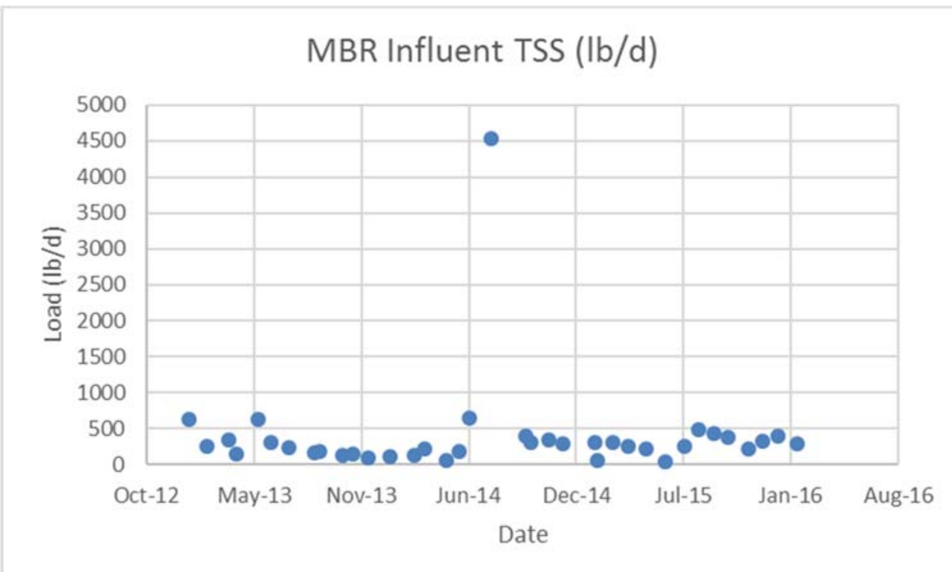


Figure 6 MCLP WWTF January 2013 – January 2016 MBR Influent TSS Load Data

3.2.2 MCLP WWTF Effluent Characteristics

Table 19 outlines the facility's design criteria, as outlined in the GE Operations and Maintenance Manual.

Table 19 MCLP WWTF – Secondary Treatment Capacity

Effluent Parameters (mg/L)	Average Daily Flow	Maximum Month Flow	Maximum Day Flow
BOD5	≤ 5	≤ 5	≤ 5
TSS	≤ 5	≤ 5	≤ 5
Turbidity	≤ 1	≤ 1	≤ 1
NH3-N	≤ 1	≤ 1	≤ 1
TN	≤ 3	≤ 10	≤ 10
Source: 'ZeeWeed 500D Membrane System Vendor Data Manual, prepared by GE and dated December 2013.			

The staff at the MCLP WWTF conducts routine sampling and laboratory analysis of the effluent discharged by the facility in accordance with the monitoring and reporting requirements specified by MassDEP in the discharge permit. Table 20 outlines the WWTFs effluent sampling schedule.

Table 20 MCLP WWTF Effluent Sampling Schedule

Sampling Period	Parameter
Daily	• Flow • pH • UV Intensity
Monthly	• Total Suspended Solids (TSS) • Total Solids (TS) • Oil & Grease • BOD5 • Nitrate Nitrogen • Total Nitrogen (NO2 + NO3 + TKN)
Quarterly	• Total Phosphorus (as P)
Source: 1. 'MassDEP Permit No 306-4, dated July 14, 2009.	

Historical plant performance monitoring data, collected and compiled by the Mashpee Commons WWTF staff over the 36-month period of January 2013 to January 2016, was analyzed to determine current operating conditions and performance for the Mashpee Commons WWTF. As outlined in Table 21, the facility is currently using approximately 40% of its design capacity and has approximately 110,000 gpd of remaining capacity on a peak day basis.

Table 21 MCLP WWTF – Current Plant Flows

Parameter	Phase 3 Design Flow (gpd) [1]	Current Plant Flow (gpd) [2]	Peaking Factor	% Capacity Used[3]
Average Annual	N/A	29,000	1.0	16%
Maximum Month	140,000	40,000	1.4	22%
Peak Day	180,000	70,000	2.4	38%
Sources: 1. 'Mashpee Commons Limited Partnership (MCLP) Mashpee, Massachusetts Contract Drawings – Phase 3 WWTP Upgrade' DEP Submission Drawings (Not for Construction), prepared by Stantec Consulting Services, Inc., dated June 2007 (Phase 3 Drawings). 2. January 2013 – January 2016 MCLP WWTF Data. 3. Of peak day design flow of 180,000 gpd.				

Plant data is summarized in Figures 7, 8, and 9. Observations based on analysis of the data are summarized below:

- With the exception of one exceedance of effluent TSS and effluent TN in April 2014, the facility has consistently been able to meet the effluent limits outlined in its discharge permit between January 2013 and January 2016.
- The average annual TN effluent concentrations in 2013 and 2015 were 3.4 mg/L and 3.2 mg/L respectively. Due to the TN exceedance in April 2014, the average annual TN concentration in 2014 was 9.4 mg/L.

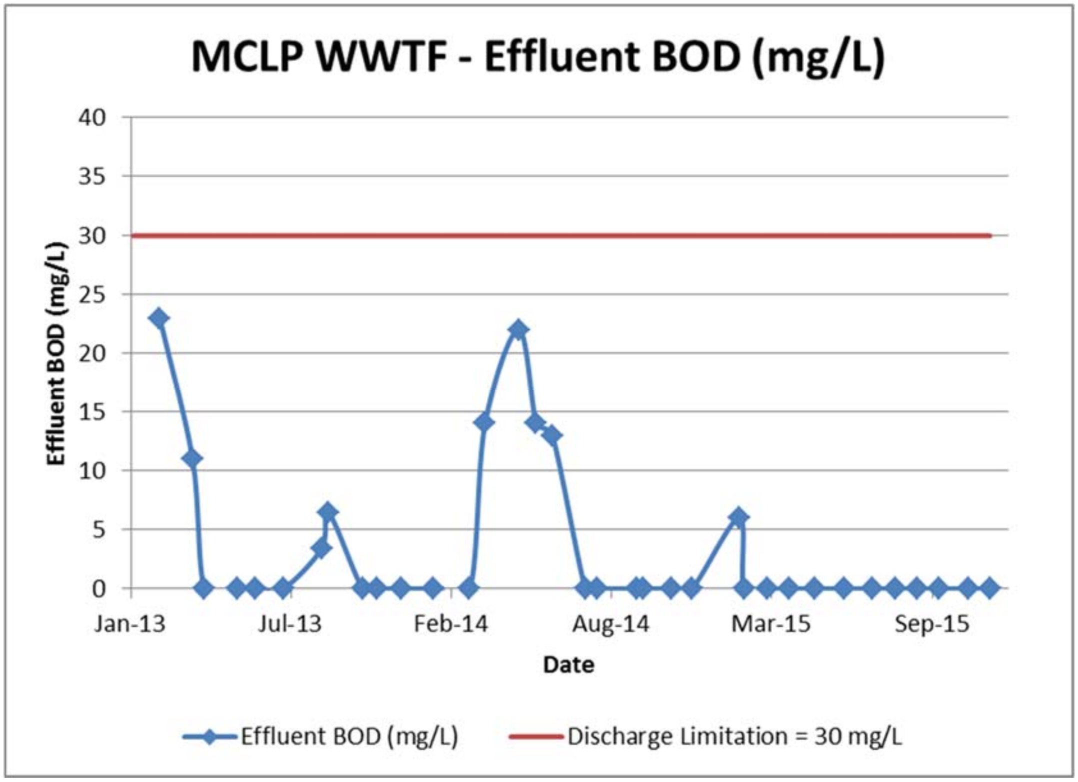


Figure 7 MCLP WWTF January 2013 – January 2016 Effluent BOD Concentration Data

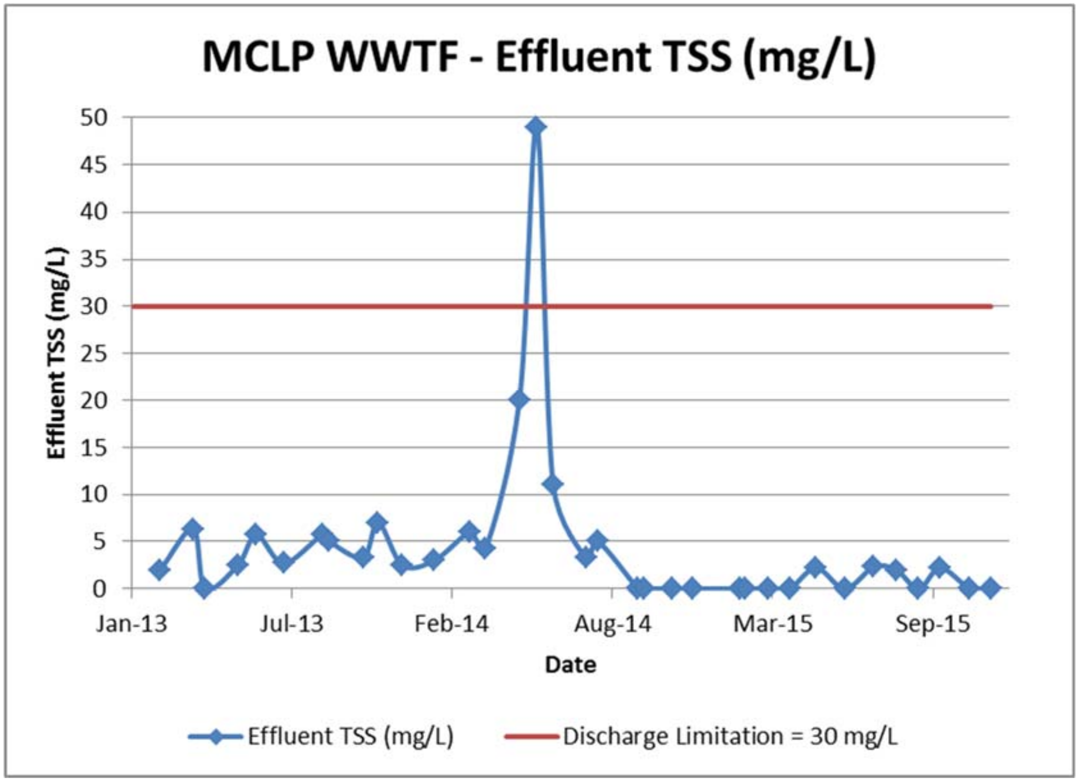


Figure 8 MCLP WWTF January 2013 – January 2016 Effluent TSS Concentration Data

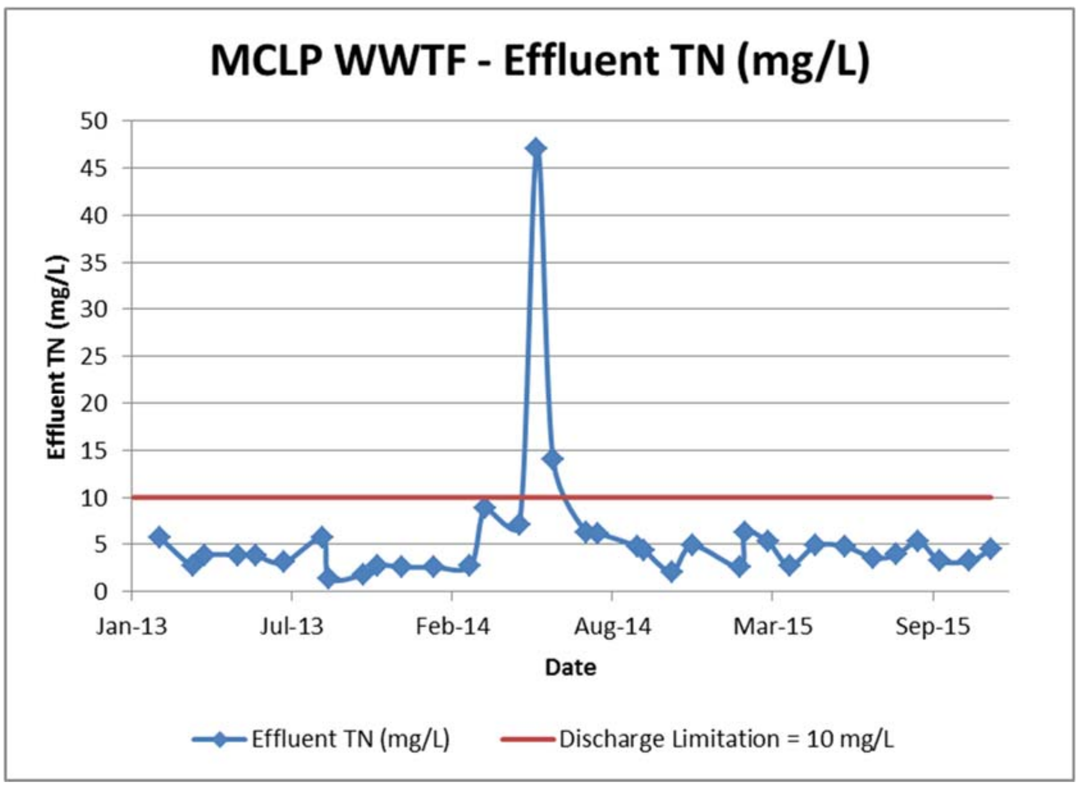


Figure 9 MCLP WWTF January 2013 – January 2016 Effluent TN Concentration Data

3.2.3 MCLP WWTF - Pretreatment/Primary Settling

The MCLP WWTF utilizes three septic tanks in series for primary treatment. The primary treatment system has a combined capacity of 70,000 gallons. The tanks rely on settling for solids separation and do not have any skimming mechanism for scum collection. MCLP WWTF staff have indicated that the septic tanks are pumped out approximately five times a year.

A. Industry Guidelines

TR-16 recommends that preliminary screening equipment be provided at all facilities in order to protect downstream pumps and treatment facilities. The recommended fine screen spacing for MBR facilities is one to three millimeters. Additionally, the GE proposal, included with the 2007 Engineer's Report, recommends a one to two millimeter screen be installed upstream of the MBR equipment. MCLP WWTF staff indicated that GE (the MBR equipment manufacturer) did not require a fine screen for this facility and that the septic tanks used for primary treatment work adequately in this application.

TR-16 does not provide sizing guidelines for septic tank pretreatment systems. The septic tanks were installed in a previous phase when the design flow fell within the "small WWTF guidelines" flow range and were reused in the Phase 3 configuration.

The "small WWTF guidelines" recommend that septic tank pretreatment systems that are also used for sludge storage be sized based on 75% of the design flow; however, they specifically note that this is not applicable for large treatment facilities (systems exceeding 150,000 gpd). Since the upgrade was approved by MassDEP it is assumed these maximum design flow guidelines are not applicable to upgrades and retrofits.

B. Installed Capacity

The septic tanks have an effective liquid capacity of roughly 40% (70,000 gpd/180.000 gpd = 40%), which is significantly less than the 75% of design flow recommended in the "small WWTF guidelines".

C. Summary

The facility uses septic tanks for pretreatment, which is a type of preliminary treatment not covered in TR-16. If the guidelines in the "small WWTF guidelines" are used, the facility has significantly less effective liquid capacity than is recommended for systems that utilize septic tanks for sludge storage.

Preliminary fine screening is typically recommended for MBR facilities. However MCLP staff have indicated that GE did not require a preliminary screen to be installed upstream of the MBR and that the existing septic tank pretreatment system works adequately.

3.2.4 MCLP WWTF - Flow Equalization

The MCLP WWTF has two flow equalization tanks, with a combined capacity of 58,000 gpd (33% of the design flow). The 2007 GE Proposal (included in the 2007 Engineers Report) states that flow equalization is required for any flow conditions greater than a maximum daily flow of 180,000 gpd for more than 24 continuous hours. A duplex pumping system is used to pump wastewater from the flow equalization tank to the pre-anoxic zone.

A. Industry Guidelines

TR-16 recommends flow equalization prior to MBR equipment in order to provide "steady state" conditions. If flow equalization is provided, it is recommended that either aeration or mechanical equipment be used to maintain adequate mixing in the tanks, to prevent deposition of solids, and to prevent the wastewater from becoming septic. The 2007 drawings do not show a mechanical mixing system in the flow equalization tanks.

TR-16 does not include any sizing requirement for flow equalization tanks.

B. Installed Capacity

The flow equalization tanks were installed under a previous phase when the design flow fell within the “small WWTF guidelines” flow range and were reused in the Phase 3 configuration. The “small WWTF guidelines” recommend that equalization systems be sized for 33% of the design flow; however they specifically note that this is not applicable for large treatment facilities (systems exceeding 150,000 gpd).

C. Summary

TR-16 does not provide sizing guidance for flow equalization tanks. However, based on 2013-2016 plant data, the flow equalization system appears to be adequately sized for the flow that the facility is currently treating. As flow approaches design (180,000 gpd peak flow), the performance and condition of the membranes should be evaluated to determine whether additional screening is necessary to protect the membranes.

3.2.5 MCLP WWTF - Membrane Bioreactor

The facility has two process trains for biological treatment. In each treatment train, wastewater flows through a pre-anoxic and aerobic zone, and a post-anoxic zone prior to continuing by gravity to one of two GE ZeeWeed 500D membrane cassettes. Denitrification is achieved in the MBR. Wastewater is recirculated from the aeration zone to the pre-anoxic zone and recycled from the membrane tanks to the head of the aerobic zone. Waste Activated Sludge (WAS) is pumped from the biological process to an aerobic digester. A recycle pump recirculates wastewater from each membrane tank to the head of the aerobic zone in each process train.

A. Industry Guidelines

MBR capacity is determined by its design flux, which is the maximum flow through a membrane. Since MBRs are proprietary systems, the design flux is typically determined by the manufacturer. The typical effluent quality for an MBR with both a pre- and post-anoxic zone is under 3 mg/L for total nitrogen.

The system’s design MLSS is 10,000 to 12,000 mg/L, which is within the typical MLSS range listed in TR-16 (4,000 to 15,000 mg/L). It should be noted that recent GHD experience has indicated that operating an MBR for an extended period (multiple months) at 10,000 mg/L can lead to the increased risk of thickened solids deposits on the membrane fibers, ultimately shortening the life of the membrane. In a recent installation the thickened solids appeared to have captured other forms of debris in the wastewater, such as hair, which could impact the air scouring process and possibly damage the individual membrane fibers. GHD typically recommends that MBR processes be designed for an average month MLSS of 8,000 mg/L and a maximum month MLSS of 10,000 mg/L.

The GE Proposal states that the system is designed to treat nitrogen down to a concentration equal to or less than 3 mg/L under average daily flow conditions and a concentration equal to or less than 10 mg/L under peak day conditions.

B. Installed Capacity

Secondary treatment design flows are summarized in Table 22.

Table 22 MCLP WWTF – Secondary Treatment Capacity

Parameter	Flow Capacity (gpd)
Summer Average Daily Flow	120,000
Winter Average Daily Flow	70,000
Peak Day Flow	180,000
Source: 'Wastewater Treatment Plant Modifications Mashpee Commons WWTP, Mashpee, Massachusetts Engineer's Report (WP68 Permit), prepared by Stantec Consulting Services Inc., dated August 2007 – 'GE Proposal for a Z-MOD™-: Wastewater Treatment System Retrofit for Mashpee Commons Take II'	

C. Summary

The MBR process is a proprietary system. Adequate information has not been provided to independently verify the unit process capacity of the system. It should be noted that the design MLSS is higher than the range GHD would typically design for. The impact of additional flow to the process will need to be evaluated by the design engineer and manufacturer. Additionally the Biowin model developed for the original design would likely need to be rerun to assess remaining process capacity.

The proposed Phase 4 layout indicates that an additional MBR will be installed when the facility is upgraded to 280,000 gpd.

3.2.6 MCLP WWTF - Effluent Disinfection

The MCLP WWTF has a horizontal bulb UV disinfection system with four double banks (eight bulbs overall). MCLP WWTF staff indicated that, due to the previous system's failure, the UV system was replaced prior to the Phase 3 upgrade (around 2012). The system was sized with the upcoming Phase 3 upgrade (280,000 gpd) in mind.

A. Industry Guidelines

TR-16 recommends that a UV system be capable of providing the minimum UV dose at design average and peak flow conditions. The document acknowledges site-specific variability complicating the establishment of a guideline design dosage and outlines typical dosage ranges based on required transmittance, effluent Total Suspended Solids (TSS) concentration, and effluent fecal coliform requirements. Based on the 30 mg/L effluent TSS Permit requirement and a fecal coliform effluent standard of 200 fecal coliform/100 mL, the typical UV dose range outlined in TR-16 for the system is 35,000 to 40,000 uWs/cm2.

B. Installed Capacity

MCLP staff have indicated that the design capacity of the UV system is 180,000 gpd. No information on the system was provided to verify the design capacity.

C. Summary

MCLP staff have indicated that the design capacity of the UV system is 180,000 gpd and that additional disinfection capacity would need to be installed to reach the Phase 4 upgrade flow.

3.2.7 MCLP WWTF - Effluent Dosing and Discharge Facilities

The MCLP WWTF has a 15,000 gallon dosing chamber. According to the design calculations in the 2007 Engineers Report the Effluent Dosing Tank has a design flow of 180,000 gpd. The dosing pumps are operated on a float system which limits the flow pumped to the original sand beds to 80,000 gpd, and the flow to the sand beds that were added as part of the Phase 3 upgrade to 100,000 gpd. Effluent is pumped from the Effluent Dosing Tank to approximately 36,000 square feet of open sand beds. MCLP staff have indicated that a hydrogeological evaluation has the effluent disposal capacity at the property limited to 280,000 gpd.

A. Industry Guidelines

MassDEP outlines design loading rates based on the results of a site percolation test. The design loading rate for a percolation rate of 2 min/inch is 5 gpd/sf.

B. Installed Capacity

Based on the percolation rate listed in the 2007 drawings of 2 min/inch and the associated MassDEP design loading rate, the effluent disposal system has a design capacity of 180,000 gpd.

C. Summary

Based on the available information, the facility appears to have adequate capacity for its design flow. Both the effluent dosing tank and discharge facilities would need to be upgraded if additional flow were treated at the facility. MCLP staff have indicated that the theoretical effluent disposal capacity of the site is 280,000 gpd.

3.2.8 MCLP WWTF - Odor Control Facilities

The 2007 drawings do not show any accommodations for odor control facilities.

3.2.9 MCLP WWTF – Summary

Based on the available documents it appears that all of the processes at the MCLP WWTF are sized adequately to treat a design flow of 180,000 gpd. The Phase 3 drawings outline proposed upgrades that would be required to increase the capacity of the facility to 280,000 gpd. MCLP WWTF have indicated that a new Process Building and secondary treatment train would need to be installed to achieve the Phase 4 capacity.

Table 23 MCLP WWTF – Unit Process Capacity Analysis

Parameter	Maximum Unit Capacity (gpd)	Expansion Potential
Primary Settling	N/A[1]	Unit process could potentially be expanded through the installation of additional septic tanks in the yard.
Flow Equalization	N/A[2]	Unit process could potentially be expanded through the installation of additional tankage in the yard.
Biological Treatment	180,000[4]	The proposed Phase 4 layout indicates that an additional MBR will be installed when the facility is upgraded to 280,000 gpd. The existing Process Building footprint will need to be expanded to house the equipment.
Effluent Disinfection	180,000[3]	Additional disinfection capacity would need to be installed to treat the Phase 4 upgrade flow.
Effluent Dosing and Disposal	180,000	The proposed Phase 4 layout indicates that an additional 100,000 gpd of effluent disposal capacity will be added to the facility when the facility is upgraded to 280,000 gpd. MCLP staff have indicated that the effluent disposal capacity of the site is 280,000 gpd. The proposed Phase 4 layout does not indicate any additional effluent dosing capacity will be installed during the upgrade.

Notes:

1. Industry design guidelines do not provide sizing guidelines for septic tank pretreatment for facilities with a design flow over 150,000 gpd.
2. Industry design guidelines do not provide sizing guidelines for flow equalization for facilities with a design flow over 150,000 gpd.
3. MCLP staff have indicated that the design capacity of the existing UV system is 180,000 gpd. No information on the system was provided to verify the design capacity.
4. Maximum unit capacity as listed in the 'Wastewater Treatment Plant Modifications Mashpee Commons WWTP, Mashpee, Massachusetts Engineer's Report (WP68 Permit), prepared by Stantec Consulting Services Inc., dated August 2007 (2007 Engineers Report). The design engineer and manufacturer needs to evaluate the unit process capacity of the existing system.

3.2.10 MCLP Pump Stations

The MCLP collection system has two pump stations:

- North Market Street
- Steeple Street

North Market Street Pump Station

North Market Street is a duplex pump station that was constructed in the mid 1990's. The pump station has two submersible pumps, each rated to pump 105 gpm at 21 feet Total Dynamic Head (TDH). The pump station serves a supermarket and several retail stores. As shown in Table 24, the installed pumps have adequate capacity to accommodate the anticipated flow north of the pump station outlined in the FEIR—the existing North Market Street development, the proposed North Market Street Phase 2, and the two schools. An evaluation of the working volume provided in the wet well indicates that the pump station was designed to accommodate the FEIR flows north of the pump station.

It should be noted that the analysis in this section is based on available development information for North Market Street Phase 2. The Town has recently been made aware that the development plans for Phase 2 have been modified; however, updated plans have not yet been provided. The capacity of the North Market Street Pump Station should be re-evaluated when updated plans are made available by MCLP.

Table 24 North Market Street Pump Station Capacity

Parameter	Peak Day Flow (gpd)
North Market Street (1)	11,800
North Market Street Phase II (2)	12,700
Quashnet School (2)	10,100
Coombs School (2)	23,300
Total Flow	57,900
Total Pump Capacity (1)	151,200

Sources:

1. 'North Market Street Mashpee, Massachusetts Prepared for Fields Point Limited Partnership' Drawing Number U2 – Sewer Profiles, Pump Sta. Details', prepared by Dufresne-Henry, Inc., dated April 1993 (1993 North Market Street Drawings)
2. 'The Neighborhoods of Mashpee Commons Draft Environmental Impact Report and Final Development of Regional Impact Submittal EOEA Number 5913, prepared by the Mashpee Commons Limited Partnership, dated March 15, 2000. (MCLP DEIR)

Steeple Street Pump Station

Steeple Street is a pre-packaged pump station that was constructed in the mid-1980's. The pump station has two suction lift pumps, each rated to pump 300 gpm at 51 feet of TDH. Flow from the Mashpee Commons collection system flows by gravity to the pump station and is pumped to the head of the Mashpee Commons WWTF.

The pump station wet well is sized to provide an approximate 15 minute detention time for a peak flow of 180,000 gpd.

The Jobs & Whiting Permit Application specified that a portion of the proposed flow from these developments will flow to the Steeple Street Pump Station. The document also indicates that a new pump station will be needed to convey the flow from the remainder of the development.

MCLP Pump Station Summary

Based on available planning documents, the North Market Street Pump Station has adequate capacity to convey the following flows, which the FEIR indicates are intended to be connected to the MCLP WWTF:

- North Market Street Phase I (constructed)
- North Market Street Phase II (proposed)
- Quashnet School (proposed for connection)
- Coombs School (proposed for connection)

The Steeple Street Pump Station wet well is sized to provide an adequate detention time for 180,000 gpd. The Jobs & Whiting Permit Application indicates an additional pump station will be required in its vicinity to convey future flows to the MCLP WWTF.

4. Estimated WWTF Flows

The design documents for the Wampanoag WWTF and MCLP WWTF indicate that both facilities were designed based on Title 5 design flows. This section outlines the current flows to each facility, establishes a theoretical design capacity, and discusses the expansion potential for each site.

4.1 Wampanoag WWTF

4.1.1 Current Flows

The Wampanoag WWTF is not yet operational and does not treat any flow.

4.1.2 Theoretical Design Capacity

In lieu of actual flow data, MassDEP guidelines[2] allow WWTF design flows to be established through the following two methodologies:

1. Establishment of design flows using State Environmental Code (Title 5) flows.
2. Establishment of design flows using water use data from known similar establishments (metered flows).

The Wampanoag WWTF design flow was established using Title 5 flows, which are intended to represent wastewater flows within the Commonwealth of Massachusetts and incorporate a safety factor to cover this substantial range. As outlined in Technical Bulletin 91-001, prepared by the Cape Cod Commission, Title 5 flows "are purposely inflated to ensure that the systems avoid hydraulic failure and assimilate maximum flows."

Analysis of metered data from known similar establishments can help establish whether local water usage is equivalent to or less than Title 5 flows. If water usage data indicates that local wastewater flow is less than Title 5 flows, excess capacity may exist in the facility design.

Wastewater flows derived from metered flows were compared to Title 5 design flows to determine if excess capacity may exist in the Wampanoag WWTF design.

A. Title 5 Design Flows

The Basis of Design for the Wampanoag WWTF is summarized in the 2011 'Proposed Wastewater Treatment Facility Wampanoag Village' drawings prepared by Norfolk-Ram. Design flows are based on Title 5 design flows (Table 25).

[2] Massachusetts Department of Environmental Protection - Guidelines for the Design, Construction, Operation and Maintenance of Small Wastewater Treatment Facilities with Land Disposal, Revised November 2014

Table 25 Wampanoag Village Design Flows

Building Type	Number of Units	Number of Bedrooms	Title 5 Design Flow (gpd)
2 Bedroom Single Family Home	21	42	4,620
3 Bedroom Single Family Home	15	45	4,950
2 Bedroom Duplex Unit	16	32	3,520
Subtotal (44 Bldgs)	**52**	**119**	**13,090**
On-Site Reserve and Infiltration/Inflow			1,910
Subtotal for Project Site Design Flow			**15,000**
Reserve for Off-Site Nitrogen Mitigation			25,000
Total WWTF Design Flow			**40,000**
Source: 'Proposed Wastewater Treatment Facility Wampanoag Village', prepared by Norfolk-Ram, dated February 2011 – Sheet 7 'Design Calculations'			

B. Metered Flows Analysis

An analysis was conducted to establish design flows for the 119-bedroom Wampanoag Village using metered data.

Establishment of Average Wastewater Flows Utilizing Metered Water Data

The 'Watershed Nitrogen Management Plan (WNMP) – Needs Assessment Report,' prepared by Stearns & Wheler, LLC (now GHD Inc.) in 2007, established average residential wastewater flows for the Town of Mashpee. The report used water use data from 1997 thru 1999 to estimate average Town wastewater flows consistent with the Massachusetts Estuaries Project (MEP).

Updated water use data from 2009 thru 2011 was used in the development of the Cape Cod Commission's 208 Plan and incorporated into their Multi-Variant Planner (MVP) tool. Average wastewater flows were established for "single family" residential properties and "all" residential properties. "All" residential properties include single-family residential houses, condominiums, and multi-family housing.

Water use data from both data sets is summarized in Table 26. In order to be conservative, the 2009-2011 water use data value of 48 gpd/bedroom for all residential properties was used in this evaluation.

Using this methodology, the 119 bedrooms for the proposed Wampanoag Village represent an average flow of 5,700 gpd.

Table 26 Mashpee Water Use and Wastewater Flow Comparison

Data Set	Water Use (gpd/property)	Wastewater Flow(1) (gpd/property)	Wastewater Flow(1) (gpd/bedroom)(3)
1997 - 1999 water use data (WMNP) – residential properties	155	140	50
2009 - 2011 water use data (CCC) – all residential properties (2)	147	133	48
2009 - 2011 water use data (CCC) – single family residential properties	118	106	38

Notes:

1. Ninety percent of a property's water use is estimated to become wastewater.
2. All residential properties include single family residential houses, condominiums, and multi-family housing.
3. Average number of bedrooms per property in Mashpee = 2.8 (2010-2014 American Community Survey 5-year estimates).

Establishment of Method 2 Peaking Factors

The Town has indicated that the population of the proposed Wampanoag Village is expected to be primarily made up of year-round residents. The Town of Mashpee has one private WWTF that is serving a primarily year-round residential development (Southport). The peaking factors for the Southport WWTF are outlined in Table 27. Due to the similar population composition of the two developments, the Southport WWTF peaking factors are considered to be representative of the peaking factors for Wampanoag WWTF.

Table 27 Mashpee Operating Wastewater Treatment Facilities Peaking Factors

	Southport Flow (gpd)	Wampanoag Village Flow (gpd)	Wastewater Flow (gpd/bedroom)[2]
Average Flow (gpd)	30,000	5,700	48
Maximum Month Peaking Factor[1]	1.3	7,500	62
Peak Day Peaking Factor[1]	1.6	9,200	77

Source: 'Town of Mashpee, Popponesset Bay & Waquoit Bay East Watersheds Needs Assessment Report – Final Report', prepared by Stearns & Wheler, dated April 2007

Notes:

(1) Peaking Factors are based on flow data from October 2004 – October 2005 and are the ratio of maximum month or peak flow to average annual flow.

(2) Per bedroom wastewater flows derived from Cape Cod Commission 2009-2011 water use data for the Town of Mashpee.

As shown in Table 28, wastewater flows established using metered flow data are slightly lower than Title 5 flows. Metered flow data indicates that a small amount of excess capacity potentially exists in the Total Design Flow. Using this methodology, the facility has the capacity to treat up to 28,950 gpd (25,000 gpd + 3,950 gpd) of off-site flow on a peak day basis.

Table 28 Title 5 and Metered Flow Comparison

Parameter	Method 1 – Title 5 Flows (100 gpd/bedroom)		Method 2 – Metered Flows (77 gpd/bedroom)	
	Number of Bedrooms	Flow (gpd)	Number of Bedrooms	Flow (gpd)
Wampanoag Village	119	13,090	119	9,180
On-Site Reserve and I/I	N/A	1,910	N/A	1,910
Reserve for Off-Site Nitrogen Mitigation	227	25,000	326	25,000
Excess Flow Capacity		0	51	3,910
Total Design Flow	346	40,000	496	40,000

Notes:

1. Average number of bedrooms per property in Mashpee = 2.8 (2010-2014 American Community Survey 5-year estimates).

C. Summary

The Wampanoag WWTF design was based on Title 5 flows. The design designates 25,000 gpd for off-site nitrogen mitigation.

Wastewater flows established using metered flows are slightly lower than Title 5 flows. Based on metered flow, the facility has an additional 3,910 gpd of excess capacity in addition to the 25,000 gpd reserve for off-site nitrogen mitigation (28,910 gpd total).

4.1.3 Expansion Potential

As indicated by the treatment capacity analysis in Section 3, the overall capacity of the Wampanoag WWTF is 40,000 gpd and the most limiting process at the facility is the RBC. Due to limited space in the existing Process Building, expanding the capacity of the facility would likely require an addition to the existing building or a new Process Building. The site appears to have adequate space for additional process buildings; however, a more detailed site evaluation and a hydrogeological evaluation are required to determine the maximum process capacity of the site.

This report assumes that the design flow of the facility will remain at 40,000 gpd (peak day flow) and that 28,953 gpd (peak day flow) of the overall flow is available for off-site nitrogen mitigation.

4.2 MCLP WWTF

4.2.1 Current Flows

As outlined in Section 3.2 an analysis of plant data from January 2013 – January 2016 indicates that the facility is currently using 40% of its peak design capacity, which translates to a remaining peak day capacity of 110,000 gpd.

4.2.2 Theoretical Design Capacity

Design flows from the last major upgrade to the MCLP facility (Phase 3), as summarized in the 2007 Engineers Report, are outlined in Table 29.

Table 29 Mashpee Commons WWTF Phase 3 Design Flows and Loads

Parameter	Flow (mgd)
Average Day (Summer)	120,000
Average Day (Winter)	70,000
Maximum Month	140,000
Maximum Week	160,000
Peak Day	180,000
Peak Hour	180,000
Source: 'Wastewater Treatment Plant Modifications Mashpee Commons WWTP, Mashpee, Massachusetts Engineer's Report (WP68 Permit), prepared by Stantec Consulting Services Inc., dated August 2007	

MCLP has not fully developed all of the properties that were identified for connection during Phase 3.

5. Study Area and Collection System Evaluations

Three Study Areas were identified for potential connection to the two private WWTFs (Study Areas A, B, and C shown in Attachment 1). Study Area A is in close proximity to the Wampanoag WWTF and Study Areas B and C are in close proximity to MCLP WWTF. This section describes each Study Area and outlines the methodology used to estimate future flows from each Study Area and the collection system considerations in connecting these areas to the WWTF. The extent of each Study Area was reviewed with the Town and revised based on the Town's comments concerning future anticipated land development. The scope was expanded to also consider collection system needs for Site 4 which would serve to address flows unable to be handled at either the Wampanoag or Mashpee Commons facility.

5.1 Wampanoag WWTF Study Areas

5.1.1 Study Area Composition

As outlined in Section 3, the Wampanoag WWTF design includes a reserve capacity of 25,000 gpd (peak) for off-site nitrogen mitigation. The metered flow analysis (Section 4.1.3) indicates that an additional 3,910 gpd of peak capacity could exist within the design capacity of the facility. Due to its proximity, a portion of the area designated for treatment at Site 4 in the WMNP could potentially utilize the off-site reserve capacity at the Wampanoag WWTF for treatment.

The Wampanoag WWTF Study Area, Study Area A, is an expansion of the WMNP "Subarea" Q, as shown in Figure 6-4 in the FRP/FEIR (see Attachment 2). Subarea Q was scheduled to be connected to the Wampanoag WWTF during Phase 1. As part of this evaluation, several adjacent parcels to the original Area Q were identified for consideration to be treated at the Wampanoag WWTF.

For the purpose of greater flexibility in the evaluation, the total Study Area is further broken out as follows (Attachment 3):

- Study Area Q-A1 includes residential properties south of the Wampanoag WWTF along Meetinghouse Road.
- Study Area Q-A2A includes residential properties north of the Wampanoag WWTF, primarily along the east side of Meetinghouse Road.
- Study Area Q-A2B includes residential properties and one large commercial property north of the Wampanoag WWTF, along the east side of Great Neck Road North.
- Study Area Q-A3 includes residential, commercial, municipal properties, and State/Town owned land north of the Wampanoag WWTF, along Route 130/Main Street.

5.1.2 Estimated Wampanoag WWTF Study Area Flows

As an update to the data used as part of the WMNP, Cape Cod Commission (CCC) water use data was used to develop an estimated wastewater flow for each parcel in the Study Area, using a 90% conversion factor from water to wastewater. As discussed in Section 4.1.3 the peaking factors for the Southport WWTF are considered representative of peaking factors in the Wampanoag WWTF Study Area. Estimated flows for each Wampanoag WWTF Study Area are outlined in Table 30.

Table 30 Estimated Wampanoag WWTP Study Area Flows

Study Area	Future Average Flow (gpd) [1]	Future Peak Day Flow (gpd) [2]
Q-A1	1,300	2,100
Q-A2A	5,900	9,500
Q-A2B	7,400	11,800
Q-A3	6,700	10,800
Total	21,300	34,200

Sources:
1. Cape Cod Commission Water Use Data multiplied by a 0.9 water to wastewater conversion factor.
2. Peak Day Peaking Factor = 1.6

5.1.3 Study Area Connections to Wampanoag WWTF

The Wampanoag WWTF design has 25,000 gpd of peak design flow designated for off-site nitrogen mitigation on a peak day basis. The metered flow analysis (Section 4.1.3) indicates that an additional 3,910 gpd of peak capacity could exist within the design capacity of the facility. This reserve capacity of 28,910 gpd could treat approximately 87% of the estimated Wampanoag WWTF Study Area flow.

One option, which would maximize the flow sent to the Wampanoag WWTF, would be to connect Study Areas Q-A1, Q-A2A, and Q-A3 (peak day flow of 21,400 gpd) to the Wampanoag WWTF. Under this scenario, the feasibility of sending Study Area Q-A2B flow to either Mashpee Commons WWTF, the proposed Site 4 WWTF, or another facility would need to be assessed.

All of the preliminary collection system layouts in the Wampanoag WWTF Study Area have been designed with the flexibility to flow to either the Wampanoag Village WWTF or the proposed Site 4 WWTF. Depending on the timing (and phasing) of the Wampanoag Village housing development construction, additional Study Areas could potentially be initially connected to the Wampanoag WWTF and could later be diverted to the Site 4 WWTF as the flow from the Wampanoag Village to the Wampanoag WWTF increases. This arrangement would allow the flow from these Study Areas to be treated by a WWTF prior to the construction of the Site 4 facility.

5.2 MCLP WWTF Study Areas

5.2.1 Study Area Composition

The two MCLP WWTF Study Areas are a subsection of the CWMP Subareas P1 and P2, as shown in Figure 6-4 in the FRP/FEIR (see Attachment 2). Subareas P1 and P2 are scheduled to be connected to the Mashpee Commons WWTP during Phase 2 with the exception of P2A-S, which is proposed to be connected to Site 6. Area B is located north of Mashpee Commons and is bordered by Nathan Ellis Highway (Route 151), Great Neck Road North, and Lowell Road. Study Area C is located south of Mashpee Commons primarily along Falmouth Road and Great Neck Road South. The Study Areas around the MCLP WWTF (identified above) are divided as follows:

- Study Area P1-B1 includes the Quashnet School and residential neighborhoods north of Mashpee Commons.
- Study Area P1-B2 consists of the Coombs School and Sandpiper Village Condominiums (existing MCLP flows have been subtracted out of the estimated flows for P1-B2).
- Study Area P1-B3 consists of Mashpee Commons properties not yet connected to the MCLP WWTF (existing MCLP flows have been subtracted out of the estimated flows for P1-B3).

- Study Area P1-B4 consists primarily of residential neighborhoods, a medical office building, and the Town's "Kids Klub" daycare center, north of Mashpee Commons and west of Great Neck Road.
- Study Area P2-C1 consists primarily of residential properties south of Mashpee Commons and between Falmouth Road and Great Neck Road South.
- Study Area P2-C2 consists primarily of residential properties south of Mashpee Commons and east of Great Neck Road South.
- Study Area P2-C3 consists of residential and commercial/industrial properties south of Mashpee Commons and along Falmouth Road.
- Study Area P2-C4 consists of residential properties south of Mashpee Commons and east of Great Neck Road South.
- Study Area P2-C5 consists of residential properties south of Mashpee Commons along Great Neck Road South.
- Study Area P2-C6 consists primarily of commercial and industrial properties south of Mashpee Commons and primarily between Commercial Street and Great Hay Road.

5.2.2 Establishing Study Area Peaking Factors

Peaking factors were established for the commercial and residential properties in the MCLP WWTF Study Areas using the methodology outlined below.

A. Commercial/Industrial Properties

The majority of properties currently connected to the MCLP WWTF are commercial properties. The current peaking factor (2.5) observed at the MCLP WWTF (derived from 2013-2016 flow data) was used to approximate the peaking factor for commercial properties in the MCLP WWTF Study Area.

B. Residential Properties

Phase 3 peak day wastewater flows (developed by Stantec) were established using Title 5 flows. To maintain consistency with this approach, MCLP WWTF Study Area peaking factors were established using Title 5 flows as a peak day flow. The methodology used to establish the residential peaking factor of 2.3 is outlined Table 31.

Table 31 MCLP WWTF Study Areas – Residential Peaking Factor

Parameter	
Average Wastewater Flow [1]	132 gpd/property
Average Number of Bedrooms per Property [2]	2.8 bedrooms/property
Average Flow/Bedroom	47.4 gpd/bedroom
Peak Day Flow/Bedroom [3]	110 gpd/bedroom
Peak Day/Average Peaking Factor	2.3

Source:

1. Cape Cod Commission Water Use Data multiplied by a 0.9 water to wastewater conversion factor.
2. Average number of bedrooms per property in Mashpee = 2.8 (2010-2014 American Community Survey 5-year estimates).
3. State Environmental Code (Title 5) flows.

5.2.3 Estimated MCLP WWTP Study Area Flows

Cape Cod Commission water use data was used to develop an estimated wastewater flow for each parcel in the Study Area, using a 90% conversion factor from water to wastewater. Estimated flows for the MCLP WWTF Study Areas are outlined in Table 32.

Table 32 Estimated MCLP WWTP Study Area Flows

Study Area	Future Average Flow (gpd) [1]	Future Maximum Day Flow (gpd) [2,3]
P1-B1	36,400	84,600
P1-B2	5,900	14,900
P1-B4	4,100	9,700
P2-C1	8,500	19,800
P2-C2	11,600	27,000
P2-C3	51,600	122,300
P2-C4	6,800	15,800
P2-C5	13,400	31,000
P2-C6	32,400	82,000
B (Total)	46,400	109,200
C (Total)	124,300	297,900
Total	**170,700**	**407,100**

Sources:

1. Cape Cod Commission Water Use Data multiplied by a 0.9 water to wastewater conversion factor.
2. Residential Peak Day Peaking Factor = 2.3
3. Commercial Peak Day Peaking Factor = 2.5
4. Study Area P1-B3 consists entirely of Mashpee Commons properties and in not included in this analysis

5.2.4 Study Area Connections to MCLP WWTP

The Phase 3 design flows, developed by Stantec, indicate reserved capacity to treat flow from four municipal buildings. The four municipal buildings specified in the design flows (Police Station, Fire Station, Library, and Senior Center) have been connected to the MCLP WWTP. As previously discussed, not all of the properties specified in the Phase 3 flow allocation have been constructed yet.

The 2013-2016 plant data indicates that the facility is currently using approximately 40% of its design capacity on a peak day basis. A portion of the MCLP WWTF Study Areas could be connected to the existing MCLP WWTF if MCLP was willing to delay the construction/connection of a portion of the development slated for Phase 3.

5.3 Site 4 Preliminary Phase 1 Study Areas

In 2005, a SewerCAD model was developed for Popponesset Bay as part of the 'Sewer Modeling and Preliminary Design Evaluations Guidance Document and Case Study Report', prepared by Stearns & Wheler. Since the development of the model, the preliminary layouts for several of the sewersheds, including Site 4, were modified during the development of the WNMP. As part of this project the existing SewerCAD model for three "new" Study Areas (D, E, and F) was updated to reflect the preliminary layout outlined in the WMNP. Study Area D is comprised of portions of the WMNP "Subarea" S1 and S2. Study Area E is made up of portions of the WMNP "Subarea" S1, S and F.

Study Area F is comprised of portions of “Subareas” S and S2. The Study Areas were broken down as follows (Attachment 4):

- Study Area D1 includes residential properties south of Falmouth Road, primarily along Quinaquisset Avenue, Orchard Road, and Mashpee Neck Road.
- Study Area D2 primarily includes residential properties along both sides of Falmouth Road.
- Study Area D3 primarily includes residential properties along Falmouth Road and Meetinghouse Road. The Area also includes the proposed Site 4 facility.
- Study Area E1 primarily includes residential properties to the east of Simon Narrows Road.
- Study Area E2 primarily includes residential properties to the east of Mashpee Neck Road and to the west of Simon Narrows Road.
- Study Area E3 primarily includes residential properties along Captains Row.
- Study Area F1 primarily includes residential properties to the south of Main Street and to the east of Noisy Hole Road.
- Study Area F2 primarily includes residential properties along Main Street, north of Amos Pond.
- Study Area F3 primarily includes residential properties along Main Street.
- Study Area F4 primarily includes residential properties north of Main Street and east of Cotuit Road.
- Study Area F5 primarily includes residential properties north of Main Street, along Cotuit Road.
- Study Area F6 primarily includes residential properties along Main Street and South Sandwich Road.
- Study Area F7 primarily includes residential properties north of Main Street and west of Cotuit Road.

5.4 Collection System Alternatives

The layout and design of a collection system depends on several factors. Key factors include the type of collection system technology, the topography of the service area, utilities located in the road right-of-way (ROW), groundwater elevations, and the location of the treatment and treated water recharge sites(s).

Three collection system technologies which can be fairly easily connected to existing wastewater treatment infrastructure—gravity, vacuum, and low pressure—were considered in this evaluation to serve the Study Areas. Each technology is described in further detail below.

5.4.1 Gravity Sewers and Pump Stations

The most prevalent type of collection system in general use is a traditional gravity sewer. This type of system involves the installation of sewers at a constant downhill gradient. The sewer line is designed at a slope sufficient to maintain a velocity within the sewer line that ensures that solids stay suspended within the waste stream. The minimum size of a typical sanitary sewer is 8 inches. The pipe size increases proportionally with the expected wastewater flow. The sewer is installed at a constant slope until its depth becomes so great that a sewage pump station is needed to “lift” the flow to a wastewater treatment plant or to another gravity sewer. In flat terrain, several pump stations may be required before the flow is pumped to 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. Houses that are below the street elevation may require the use of small pumps and a small diameter force main (1 to 2 inches) for discharging to the collector sewer.

The installation cost and ease of construction of a gravity sewer depends upon the topography within a particular area and on specific soil types. In areas where topography is consistently increasing or decreasing, the sewers can be installed close to minimum depth. In very hilly areas, deep sewers and/or pump stations may be required. This can significantly increase construction costs when compared with other options.

Advantages of gravity sewers include the following:

- A properly designed and installed gravity sewer requires little maintenance.
- A gravity system can be easily expanded to serve additional areas. Additional capacity can be provided to accept future flow without affecting performance.
- The potential for odors in a properly designed gravity sewer is low.
- A gravity system is reliable, since it is not dependent upon electrical power for operation. When pump stations are used on collector sewers, electrical generators are provided to supply power during a power outage.

Disadvantages of gravity sewers include:

- Gravity sewers are installed at a constant slope, and thus can require deep excavations as the topography changes. Construction with trenchless technologies is generally difficult as constant grades are required. Construction is generally disruptive to traffic patterns and surface infrastructure, as they are often located within the paved roadway to avoid conflicts with water and gas utilities that are typically located closer to the shoulder(s) of the road.
- Pump stations are required to transport the sewage out of low points in topography.
- Feasibility may be limited by availability of appropriate pump station locations.
- Capital and operation and maintenance costs increase with each pump station required.
- Pump stations tend to increase the potential for odor emissions.
- Improper installation techniques could result in infiltration and exfiltration.

5.4.2 Pressure Sewers with Grinder Pumps

A pressure sewer system requires the installation of a grinder pump to serve each building or group of buildings. Wastewater flows by gravity into a pump chamber, where the sewage is shredded and pumped into a pressure sewer, eventually discharging to a gravity main or directly to a pump station or treatment facility. This type of technology has become more widely used over the past 20 years, and is particularly suited to areas where there is a need to minimize excavation depths.

The typical pressure in this type of system is 5 to 40 pounds per square inch (psi). Pressure systems can be expanded to serve additional areas. Typically, systems can be expanded to serve additional homes; however, there are design limitations, and the overall expansion capability tends to be less than that of a gravity sewer.

When connecting pressure sewer lines to a gravity line or directly to a pump station, odor control for larger systems may be required at the discharge point to mitigate odors created in the pressure sewer pipe. Also, manholes at the discharge point should be protected from corrosion resulting from high hydrogen sulfide concentrations.

Advantages of a pressure sewer include the following:

- The collection main is installed at a relatively shallow depth and is independent of grade changes. This allows shallower excavation, lower piping installation costs, and less overall disruption to the area due to a shorter construction period.
- A pressure sewer can serve areas of hilly terrain or marginal slope.
- The pressure sewer piping (beyond the pumping chamber) is not susceptible to infiltration, unlike gravity sewers.
- The shredding action of the pump eliminates the need for a larger size collection system. Pressure sewers tend to be much smaller diameter than a typical sanitary sewer, ranging from 1-1/4 inch to 6 inches, depending upon the expected design flow.
- The pressure sewer mains (at shallower depths than gravity) are easier to locate in road shoulders to minimize construction in roads where space is available.
- Some portion of pressure sewers could be installed with trenchless technologies, thus reducing general disruptions experienced during construction.
- Homes with generators can maintain operation of their pumps as long as their home electrical system is set up to accommodate the load (this is becoming more common with the larger number of power outages in recent years).

Disadvantages to this type of system include the following:

- Each building or group of buildings in the system would have to be equipped with a pump unit, which increases operation and maintenance requirements. Towns that operate their own systems typically have to maintain an inventory of pumps/parts for these units to minimize disruption of services; otherwise it becomes the homeowner’s responsibility to have their system maintained.
- Each pump unit is dependent upon electrical power for proper operation; since the pumps are located at individual homes, municipal backup electrical power is typically not provided. Storage capacity is typically built into each pump chamber (capacity depends on manufacturer). However, in a prolonged power outage, it would be possible for the wastewater flow to exceed this capacity and backup into the pipelines within the structures. This can be remediated by providing electrical connections on each pump unit to allow a service crew to connect a portable generator and pump out each unit during times of prolonged power outage. Another option is to install a larger capacity unit or a dual tank system, thus providing more storage.
- This system is more sensitive to seasonal flow conditions than a gravity sewer. In areas with extreme seasonal fluctuations, minimum flow conditions must be carefully quantified to be sure the sewage flow can properly travel through the system. If inadequate flow exists, solids can harden within the sewer and cause blockages.
- There is a potential for exfiltration of sewage into the surrounding soil if leaks or breaks occur in the pipeline.

- Training would be required to familiarize operating staff with maintenance of the pumps and pressure sewers.
- Ownership considerations need to be clearly defined early in the selection and design process. Costs for systems will depend on who owns, operates, and maintains the grinder pump. Easements may also be required to address maintenance and emergency power issues.
- Odors can be released at the point of discharge into a gravity sewer or pump station.

5.4.3 Vacuum Sewers

Vacuum sewers are typically 4 to 12 inches in diameter, and therefore are typically smaller in diameter than traditional gravity sewers and rely upon a vacuum created within the pipeline to draw the sewage towards a pump station. Vacuum pumps located at a vacuum station draw air out of the sewer, creating a vacuum inside the sewer. Sewage from individual homes flows by gravity to a vacuum valve pit located on each property like a grinder or STEP system or at the property line. Flows from larger facilities, such as hotels/motels, restaurants, apartments, condominium and large commercial facilities are handled by buffer tanks instead of valve pits; however, the operational aspects are similar and rely on the same type of vacuum valve, usually in configurations of two or more. As sewage fills a chamber in the bottom of the valve pit or buffer tank, a sensor activates an automatic vacuum valve. When the valve opens, sewage is drawn into the sewer because of the pressure difference between the sewer and atmospheric pressure outside the valve. Each subsequent opening of the valve draws the sewage (and air) further downstream until it reaches the vacuum station, where it is pumped from the receiving tank to a gravity sewer, another pump station, or treatment facility.

Advantages of vacuum sewers include:

- Vacuum sewers can be installed at shallower depths than gravity as allowed by their use of a saw tooth configuration, which can reduce installation costs and excavation time.
- Because the piping must be airtight to allow proper vacuum operation, the infiltration potential tends to be low. Infiltration can occur if a pipe leaks or breaks in areas where the line is completely submerged in groundwater; however, leaks are readily apparent through the vacuum system operation records and loss in vacuum pressure.
- There are typically no power requirements at individual properties. Vacuum stations can be equipped with emergency generators at their main stations, which allow the system to remain in operation during power outages.
- The vacuum sewer mains can be located in road shoulders to minimize construction in roads where space is available.
- Vacuum sewer mains are typically smaller diameter than gravity sewer and can be installed at flatter slopes than traditional gravity.
- Saw tooth configuration can allow installation to avoid some utility conflicts, as long as there is sufficient headloss and friction loss available in the system to allow additional bends beyond the original design.

A vacuum system has the following disadvantages:

- A vacuum must be constantly maintained in the pipeline for the system to work. Malfunctions (air leaks) in the line can affect the entire system and must be fixed quickly to keep the system operational. Leaks or malfunctions may be difficult to locate.

- The potential for odor generation at the vacuum station is greater due to the vacuum pumps constantly pulling air from the system. This airflow must be treated to minimize odors.
- Operator training would be required to gain sufficient knowledge to operate and maintain the vacuum pipelines, vacuum pump stations, and emergency response procedures. The "learning curve" for these systems is much greater than other collection systems.
- This type of system is not readily adaptable to hilly terrain.
- To design a properly operating system, the design flows must be estimated as accurately as possible, and a detailed route survey must be performed. Vacuum systems are sized for specific cases and cannot be easily expanded to serve additional homes if not designed in advance to do so.
- There is a potential for frozen valve controllers, depending on the valve pit depth.
- Systems are less common in Massachusetts; therefore, contractors are less familiar with the system, which has system-specific design requirements that are very different from gravity and low pressure systems.
- Headloss and length of system limit the application of this technology.
- System is very sensitive to the types and flow variations, which can impact system performance and capacity.
- Large commercial developments (hotels, motels) typically require the use of buffer tanks, which complicate the system.
- Large flows entering vacuum systems at one location are difficult to manage and should not be located at the ends of the collection system lines.
- Gravity and pump stations should not be connected into vacuum systems; large flow rates from pump stations and infiltration/inflow from gravity systems can overwhelm the vacuum valves.
- Buffer tank installations (if needed) add to the operation and maintenance requirements (act as small pump stations).
- Influent pipe to buffer tanks (if needed) are susceptible to clogging with rags, can create backups in the gravity lines feeding the system, and can impact flow splitting in these structures.
- Seasonal use can impact the system. System relies on even distribution of air into the system to move flow, and the addition of automatic air valves to assist this can add to energy costs at the vacuum station.

5.4.4 Operations and Maintenance Considerations

Each sewer alternative has varying degrees of required operation consideration. Gravity systems are the simplest to operate. Although pump stations may be complex, equipment is centrally located (i.e. at the station) and can be designed such that station components are interchangeable.

Grinder pump and pressure sewers are also made of interchangeable components; however the equipment is spread throughout the collection system, thus alternatives with fewer grinder pumps are considered easier to operate. Grinder pumps are not maintained as frequently as pump stations, however the locations and number of units involved make them more difficult to manage, especially during times of power loss or multiple failures.

Vacuum systems are the most operator-intensive. The systems, although reliable, often require more maintenance and monitoring than a pressure sewer system when working to identify air leaks. Valve pits are analogous to grinder pumps in that they are spread throughout the system; however, electrical power is not required. Buffer tanks and the vacuum station are analogous to pump stations and require weekly or more frequent monitoring.

5.4.5 Collection System Technology Selection

Conceptual layouts were developed for each Study Area with the goal of maximizing the extent of gravity sewers and minimizing the use of pressures or vacuum. However, in some applications, structures being served by a gravity sewer may be too low to allow for simple gravity connections, and low pressure pumps may be required for individual connections. In areas where gravity sewers do not appear to be feasible (most commonly due to topographical issues), low pressure sewers are considered. Due to high maintenance requirements and limited expansion potential, vacuum sewers were not considered.

5.5 Pump Station Alternatives

The three types of pump stations most commonly used in collection systems are described below.

5.5.1 Wet Well/Dry Well Pump Stations

Wet well/ dry well pump stations are typically used for large flows collected from several smaller areas. There are several styles and options, including packaged units. Wet well/ dry well pump stations are typically the most expensive pump stations of the three considered in this report.

5.5.2 Submersible Pump Stations

Submersible pump stations are typically used for smaller flows and are relatively simple in design. The pumps and their motors are mounted in the bottom of the wet well and are accessed by slide rails. Once the pumps and motors are larger than 25 hp, they become too heavy for most operators to want to work with. Pump and motor size often defines the upper flow range used by this station type. These types of pump stations are often most suitable for small collection systems. A typical layout for a submersible pump station is shown in Attachment 5.

5.5.3 Suction Lift Pump Stations

Suction lift pump stations are becoming popular for small to intermediate flows because the pumps are located above-ground and are accessible for repairs and maintenance. The pumps are not submerged in raw wastewater and therefore do not require the hoisting or cleaning involved with submersible stations, resulting in a reduced maintenance time. They are often smaller in size than wet well/dry well configuration and come with skid mounted equipment, simplifying construction.

TR-16 provides design considerations for suction lift pump stations, such as suction piping be limited to 25 feet. Often the pumps are limited to applications of less than 100 feet total dynamic head (TDH). A typical layout for a suction lift pump station is shown in Attachment 5.

5.5.4 Pump Station Selection

Due to the relatively small flows expected from the Study Areas, preliminary pump station layouts for this report were based on a submersible pump station. A final selection of pump station technology is expected to occur as the project proceeds to final design at a later date.

5.5.5 Proposed New Pump Station Locations

Table 33 lists the estimate pump station site and if an easement may potentially be needed for construction. Proposed pump station locations are shown on Attachment 6.

Table 33 Proposed New Pump Station Summary

Pump Station Designation	New Pump Station Site	Land Use	Easement Potential
A1	380 Ashers Path East	Municipal	No
A2	50 Meeting House Road	Municipal	No
A3	2 Great Neck Road North	Municipal	No
B1	150-152 Old Barnstable Road	Municipal	No
B2	19-20-26 Frank E. Hicks Drive	Municipal	No
B4	501 Great Neck Road	Municipal	No
C1	18 Kings Court	Residential – Undevelopable	Yes
C2	0 Great Pines Drive	Residential – Open Land	Yes
C3	732 Falmouth Road	Municipal	No
C4	Amos Landing Road (Right-of-Way)	Private Road – Right-of-Way	Yes
C5	0 Great Neck Road South	Residential – Open Land	Yes
C6	117 Industrial Drive Ext.	Private Road – Right-of-Way	Yes
D1	0 Mashpee Neck Road	Municipal	No
D2	0 Falmouth Road	Municipal	No
D3	380 Ashers Path East	Municipal	No
E1	39 Spinnaker Drive	Municipal	No
E2	0 Dry Hollow Lane	Subdivision Common Land	Yes
E3	65 Treasure Lane	Residential	Yes
F1	0-NS Noisy Hole Road	Municipal	No
F2	1 Village Green Circle	Condiminium	Yes
F3	117 Main Street	Municipal	No
F4	118 Shields Road	Municipal	No
F5	125 Timberlane Drive	Municipal	No
F6	0 Hickory Circle	Municipal	No
F7	0 Sheffield Place	Municipal	No

5.6 SewerCAD Model Development

SewerCAD Version 8i by Bentley System, Inc. was used to develop a proposed layout for each Study Area. The major assumptions that were used to develop the SewerCAD model are outlined in this Section.

5.6.1 Gravity Sewer

The SewerCAD model uses Manning's Equation for gravity pipes and is run assuming a "steady state" condition under peak flow conditions. Wastewater flows are based on a combination of wastewater from sanitary sources and a conservative estimate of wet-weather flows (inflow and infiltration {I/I}) based on guidance from TR-16. 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% 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. At a minimum, TR-16 requires an 8-inch pipe diameter. The pipe material selected for the model was PVC, currently the most commonly used 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, all of which provides for an easier installation and lower shipping costs than other heavier materials that have been used in the past.

Based on TR-16 requirements, the following design parameters were used:

- Pipe Slope Minimum: 0.5% or 0.005 ft/ft
- Manholes at every junction and every 400 feet
- Minimum drop of 0.10-feet between manhole inlet and outlet
- Minimum Cover: 5-feet
- Maximum Cover: 20-feet for extreme locations

Once these design parameters are proposed, manhole elevations are input into the model and a subsequent model run verifies that all the design constraints are being met. At areas where the collection system layout is identified, the slope and cover requirements are adjusted manually so the model can maximize the amount of area covered by gravity sewers.

Each parcel in the planning area was assigned a future wastewater flow based on Cape Cod Commission future water use projections for each parcel and a 90% conversion factor. After the collection system was laid out, 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. The appropriate peaking factor was applied to simulate peak hour flows and to ensure that pipe sizes were adequate for such flows.

It is noted that minimal I/I will be evident with new infrastructure and proper municipal inspection of service laterals, but experience indicates that I/I increases during the long lifetime of a collection system which is taken into account with the I/I factor of 500 gpd/in-dia/mile (based on TR-16 recommendations) which is used in the model.

5.6.2 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 used for the force main and will begin its evaluation using a 4-inch ductile iron 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 (ft/s) 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 used for this evaluation. 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 head-losses, it factors in the bends in the force main, as well as the other conditions, and calculates loss in feet for each force main.

The total loss or total dynamic head is the sum of the friction loss, minor losses, and the static loss. The static loss is simply the difference in elevation of the water surface in the wet well and the invert of the structure into which it pumps. The pump 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.

5.7 Hydraulic Capacity Analysis

Available record drawings of existing collection system infrastructure in the vicinity of the Wampanoag WWTF and the MCLP WWTF were input into the SewerCAD model to evaluate the potential impacts of conveying proposed Study Area flows through the existing infrastructure. The capacity analysis was based on owner-provided record drawings. The collection system record drawings were not verified through field work or survey as part of this project.

5.7.1 Impact to Existing Wampanoag WWTF Collection System

The proposed Wampanoag Village collection system has not been constructed yet. The proposed collection system for Study Area A connects directly to the Wampanoag WWTF and does not utilize the proposed Wampanoag Village collection system to convey flow. Study Area A flows are expected to have a minimal impact on the future Wampanoag Village collection system.

5.7.2 Impact to Existing MCLP WWTF Collection System

The existing Mashpee Commons was modeled in SewerCAD Version 8i by Bentley System, Inc. The model was developed based on owner-provided collection system record drawings (collection system record drawings were not verified through field work or survey as part of this project). The model indicates that the gravity portion existing Mashpee Commons collection system has adequate capacity to convey the Study Area flows during peak flow events. It is anticipated that a new, larger, force main will need to be installed between the North Market Street Pump Station and the existing collection system to convey the full flow from Study Area B. Additionally, it is anticipated that both existing pump stations (North Market Street and Steeple Street) will require larger pumps to convey the flow from Study Area B and Study Area C.

5.8 Estimated Costs (Study Areas A, B, and C Only)

5.8.1 Engineer's Estimate of Probable Capital Costs

The Engineer's opinion of probable capital cost in 2017 dollars for Study Areas A, B, and C are outlined in Tables 34, 35, and 36. The costs include a 25% construction contingency allowance and a 30% allowance for legal, fiscal, and engineering services during design and construction. The contingency is provided for variability in the bidding climate, project changes before bidding, easements and residential property restoration, and change orders due to unforeseen conditions. The following basis of design was used for the costs estimates:

- Eleven new pump stations (either submersible or suction lift) will be constructed in the Study Area.
- Sewer system is primarily comprised of a gravity collection system.
- Town assumes all construction and O&M up to the property line.
- Trench paving only, full width overlay or other road reconstruction options are not factored into the cost estimates.
- Service lateral installation costs are segmented from a) main to property line, and b) property line to house.
- The number of lateral connections for each sewershed is based upon the number of parcels contained within the sewershed which are (1) projected to have a future water demand in the Cape Cod Commission data, and (2) are not currently served by an I/A system.

(continued)

Table 34 Engineers Opinion of Probable Capital Cost – Town-Owned and Maintained System, Study Area A

Study Area	A1	A2	A3	Study Area A Total
Estimated Quantities				
Gravity Mains (LF)	1,830	6,260	1,550	9,640
Low Pressure Sewer (LF)	0	0	0	0
Force Main (LF)	2,250	3,030	870	6,150
Gravity Manholes	25	56	7	88
Gravity Connections	10	42	14	66
Grinder Pumps	0	0	0	0
Pump Stations	1	1	1	3
Estimated Operations and Maintenance Costs ($/year)				
O&M Cost ($/yr)	$25,000	$27,000	$25,000	$77,000
Estimated Capital Costs (2017$)				
Capital Costs (2017$)	$3,500,000	$5,300,000	$3,100,000	$11,900,000
Extrapolated Capital Costs – Future Years[2]				
Capital Costs (2018$)	*$3,600,000*	*$5,500,000*	*$3,200,000*	*$12,300,000*
Capital Costs (2019$)	*$3,700,000*	*$5,700,000*	*$3,300,000*	*$12,700,000*
Capital Costs (2020$)	*$3,800,000*	*$5,900,000*	*$3,400,000*	*$13,100,0000*
Capital Costs (2021$)	*$3,900,000*	*$6,100,000*	*$4,500,000*	*$13,500,000*
Capital Costs (2022$)	*$4,000,000*	*$6,300,000*	*$4,600,000*	*$13,900,000*

Notes:

1. Estimated quantities were calculated using the SewerCAD model. No surveys have been conducted of the proposed Study Areas as part of this project.
2. Capital costs were extrapolated using a 3% inflation factor to demonstrate the impact of inflation on capital costs if the project was implemented in different years in Phase 1.
3. Total Capital Costs includes allowances for construction costs such as: a 25% construction contingency; 30% legal, fiscal and engineering costs; and a 5% mobilization cost. The contingency is provided for variability in the bidding climate, project changes before bidding, and change orders due to unforeseen conditions. The contingency is also intended to cover costs for limited power utility improvements if phase and voltage changes to the existing infrastructure are required, however the actual costs would not be known until the final design in these areas. Total capital costs are rounded to the nearest $100k.
4. Costs do not include the cost of final restoration of private property or the cost of attaining easements.
5. Trench work (gravity, low pressure, and force mains) in Town roads include excavation, backfill, and traffic control (but not police details). Full width overlay was not factored into the costs.
6. O&M costs include items for labor and parts to operate and maintain the proposed system. Total capital costs are rounded to the nearest $1,000.

Table 35 Engineers Opinion of Probable Capital Cost – Town-Owned and Maintained System, Study Area B

Study Area	B1	B2	B3	B4	Study Area B Total
Estimated Quantities					
Gravity Mains (LF)	11,050	1,300	N/A	2,670	15,020
Low Pressure Sewer (LF)	1,600	0	N/A	0	1,660
Force Main (LF)	340	820	N/A	2,360	3,520
Gravity Manholes	122	7	N/A	52	181
Gravity Connections	109	3	N/A	20	132
Grinder Pumps	25	0	N/A	0	25
Pump Stations	1	N/A	N/A	1	2
Estimated Operations and Maintenance Costs ($/year)					
O&M Cost ($/yr)	$41,000	$25,000	$27,000	$41,000	$159,000
Estimated Capital Costs (2017$)[2]					
Capital Costs (2017$)	$7,000,000	$800,000	N/A	$4,100,000	$11,900,000
Extrapolated Capital Costs – Future Years[2]					
Capital Costs (2018$)	*$7,200,000*	*$800,000*	*N/A*	*$4,200,000*	*$12,200,000*
Capital Costs (2019$)	*$7,400,000*	*$800,000*	*N/A*	*$4,300,000*	*$12,500,000*
Capital Costs (2020$)	*$7,600,000*	*$800,000*	*N/A*	*$4,400,000*	*$12,800,000*
Capital Costs (2021$)	*$7,800,000*	*$800,000*	*N/A*	*$4,500,000*	*$13,100,000*
Capital Costs (2022$)	*$8,000,000*	*$800,000*	*N/A*	*$4,600,000*	*$13,400,000*

Notes:

1. Estimated quantities were calculated using the SewerCAD model. No surveys have been conducted of the proposed Study Areas as part of this project.
2. Capital costs were extrapolated using a 3% inflation factor to demonstrate the impact of inflation on capital costs if the project was implemented in different years in Phase 1.
3. Total Capital Costs includes allowances for construction costs such as: a 25% construction contingency; 30% legal, fiscal and engineering costs; and a 5% mobilization cost. The contingency is provided for variability in the bidding climate, project changes before bidding, and change orders due to unforeseen conditions. The contingency is also intended to cover costs for limited power utility improvements if phase and voltage changes to the existing infrastructure are required, however the actual costs would not be known until the final design in these areas. Total capital costs are rounded to the nearest $100k.
4. Costs do not include the cost of final restoration of private property or the cost of obtaining easements.
5. Trench work (gravity, low pressure, and force mains) in Town roads includes excavation, backfill, and traffic control (but not police details). Full width overlay was not factored into the costs.
6. O&M costs include items for labor and parts to operate and maintain the proposed system. Total capital costs are rounded to the nearest $1,000.
7. The existing Mashpee Commons collection system comprises Study Areas B3 and B5. It is assumed that any additional wastewater infrastructure in these Study Areas will be privately installed by MCLP and will not represent a capital cost to the Town.

Table 36 Engineers Opinion of Probable Capital Cost – Town Owned and Maintained System, Study Area C

Study Area	C1	C2	C3	C4	C5	C6	Study Area C Total
Estimated Quantities							
Gravity Mains (LF)	4,480	6,360	10,140	4,110	4,980	4,700	34,770
Low Pressure Sewer (LF)	0	0	0	380	520	230	1,130
Force Main (LF)	2,010	2,210	5,020	5,550	4,930	2,370	22,090
Gravity Manholes	42	161	116	35	121	84	559
Gravity Connections	52	105	60	59	57	32	563
Grinder Pumps	0	0	0	5	7	1	13
Pump Stations	1	1	1	1	1	1	6
Estimated Operations and Maintenance Costs ($/yr)							
O&M Cost ($/yr)	$25,000	$25,000	$41,000	$25,000	$25,000	$27,000	$168,000
Estimated Capital Costs (2017$)							
Capital Costs (2017$)	$4,500,000	$5,400,000	$7,400,000	$5,700,000	$5,900,000	$4,700,000	$33,600,000
Extrapolated Capital Costs – Future Years							
Capital Costs (2018$)	*$4,600,000*	*$5,600,000*	*$7,600,000*	*$5,900,000*	*$6,100,000*	*$4,800,000*	*$34,600,000*
Capital Costs (2019$)	*$4,700,000*	*$5,800,000*	*$7,800,000*	*$6,100,000*	*$6,300,000*	*$4,900,000*	*$35,600,000*
Capital Costs (2020$)	*$4,800,000*	*$6,000,000*	*$8,000,000*	*$6,300,000*	*$6,500,000*	*$5,000,000*	*$36,600,000*
Capital Costs (2021$)	*$4,900,000*	*$6,200,000*	*$8,200,000*	*$6,500,000*	*$6,700,000*	*$5,200,000*	*$37,700,000*
Capital Costs (2022$)	*$5,000,000*	*$6,400,000*	*$8,400,000*	*$6,700,000*	*$6,900,000*	*$5,400,000*	*$38,800,000*

Notes:

1. Estimated quantities were calculated using the SewerCAD model. No surveys have been conducted of the proposed Study Areas as part of this project.
2. Capital costs were extrapolated using a 3% inflation factor to demonstrate the impact of inflation on capital costs if the project was implemented in different years in Phase 1.
3. Total Capital Costs includes allowances for construction costs such as: a 25% construction contingency; 30% legal, fiscal and engineering costs; and a 5% mobilization cost. The contingency is provided for variability in the bidding

climate, project changes before bidding, and change orders due to unforeseen conditions. The contingency is also intended to cover costs for limited power utility improvements if phase and voltage changes to the existing infrastructure are required, however the actual costs would not be known until the final design in these areas. Total capital costs are rounded to the nearest $100k.

4. Costs do not include the cost of final restoration of private property or the cost of obtaining easements.
5. Trench work (gravity, low pressure, and force mains) in Town roads includes excavation, backfill, and traffic control (but not police details). Full width overlay was not factored into the costs.
6. O&M costs include items for labor and parts to operate and maintain the proposed system. Total capital costs are rounded to the nearest $1,000.

5.8.2 Operations and Maintenance Costs

In general, operation and maintenance (O&M) costs were developed inclusive of labor needs, power needs, equipment replacement, and technology-specific needs. A summary of the total annual O&M costs are presented in Tables 34, 35, and 36.

6. Estimated Nitrogen Loading

As part of the development of the FRP/FEIR, during the WNMP, the estimated nitrogen load to each subwatershed was calculated. As discussed previously, the project planning area was divided into several Subareas based on potential sewershed areas established early in the planning process. The originally estimated nitrogen loads were calculated based on the level of treatment applied to a particular Subarea, and on where wastewater was recharged within any particular subwatershed. The primary means of estimating impact was conducted using the “Rainbow Spreadsheets” developed as part of the MEP and the target threshold values established by MEP as part of the TMDLs with the goal of trying to achieve TMDL compliance at the sentinel station.

As part of this evaluation, modified Subareas were established for this evaluation. A nitrogen loading analysis was conducted to determine if these shifts have a significant impact on the overall nitrogen management plan established as part of the WMNP. As discussed previously, these changes reflected the desire of the Town to provide collection system service to certain areas, however they are all areas identified for service as part of the WNMP. These revised Study Areas are now being evaluated for possible connection to the Wampanoag WWTF and Mashpee Commons WWTF. The Study Areas are shown in Attachment 3.

Tables 37 and 38 identify each of the Study Area subsets for the Wampanoag WWTF and Mashpee Commons respectively. The tables indicate where each new Study Area was originally proposed to be served by in the WNMP and where it is proposed to be served under this study. These Study Areas are described in Sections 5.1.1 and 5.1.4 of this report.

Table 37 Wampanoag WWTF Study Areas Proposed Effluent Recharge

Study Area	WNMP Proposed Effluent Recharge Location	WWTF Evaluation Proposed Effluent Recharge Location
A1	**Site 4 (Transfer Station)**	Wampanoag WWTF
A2A	Wampanoag WWTF	Wampanoag WWTF
A2B	**Mashpee Commons**	Wampanoag WWTF
A3	Wampanoag WWTF	Wampanoag WWTF

Table 38 Mashpee Commons WWTF Study Areas Proposed Effluent Recharge

Study Area	WNMP Proposed Effluent Recharge Location	WWTF Evaluation Proposed Effluent Recharge Location
Area B1	Mashpee Commons	Mashpee Commons WWTF
Area B2	Mashpee Commons	Mashpee Commons WWTF
Area B3	Mashpee Commons (Existing)[1]	Mashpee Commons WWTF
Area B4	Mashpee Commons	Mashpee Commons WWTF
Area B5	Mashpee Commons (Existing)[1]	Mashpee Commons WWTF
Area C1	Mashpee Commons	Mashpee Commons WWTF
Area C2	**Site 6**	Mashpee Commons WWTF
Area C3	Mashpee Commons	Mashpee Commons WWTF
Area C4	Mashpee Commons	Mashpee Commons WWTF
Area C5	Mashpee Commons	Mashpee Commons WWTF
Area C6	Mashpee Commons	Mashpee Commons WWTF

Notes:

1. Mashpee Commons (Existing) – indicates that these are areas already connected to the WWTF as part of the Phase 1 and 2 areas identified in Mashpee Commons approved development plan.

As presented in the preceding tables, the proposed treatment facility for the majority of the areas in this evaluation did not change. Two Study Areas are being considered for treatment at the Wampanoag WWTF that had not been previously: A1 and A2B. A1 was originally planned for connection to Site 4, and A2B was originally planned to connect to Mashpee Commons. In addition, one Study Area is now being considered for connection to Mashpee Commons: C2. C2 was originally intended to be connected to a future facility identified as Site 6 in a later phase of the WNMP; however, the Town requested that this area be considered as another extension to the Mashpee Commons collection system during this evaluation.

Updated estimated nitrogen loads calculated as part of the evaluation are compared to nitrogen loads estimated as part of the WNMP. Updates are necessary as the projected flows have changed based on the shifting of Study Areas, CCC updated water use data, and estimated performance levels assumed. Since the WNMP examined the estimated nitrogen loading impacts from multiple facilities and Subareas that extend beyond those being considered for this evaluation, only the incremental nitrogen loading change associated with the Study Areas in this evaluation are examined. Some of the facilities (primarily Site 4 and Mashpee Commons) identified as part of the WNMP included flows from Subareas that are outside of this report's Study Areas, it is assumed there is no change in those proposed flows and loads.

Estimated effluent water quality relative to total nitrogen is identified for each facility. Those used during the WNMP reflect assumptions used. The Wampanoag WWTF quality was improved for this evaluation based on the design information contained on the approved WWTF design plans.

The following three tables present the estimated flows and nitrogen loads developed as part of the WNMP and three variations:

- Table 39: improved nitrogen removal performance at each facility from those used in WNMP to all at 5 mg/L TN.
- Table 40: improved nitrogen removal performance at each facility from those used in WNMP to all at 3 mg/L TN.

- Table 41: improved nitrogen removal performance at each facility from those used in WNMP to all at 3 mg/L TN and increases the build-out projections of the Mashpee Commons development from the permitted 180,000 gpd (Phase 3 peak flows) to the recharge capacity of the site of 280,000 gpd peak (or approximately 65,000 gpd average annual flow increase).

Table 39 Estimated Change in Nitrogen Load (WWTF Performance at 5 mg/L TN)

WWTF	Estimated Avg. Future WWTF Flow (gpd)		Estimated Eff. Quality at WWTF (mg/L TN)		Estimated Nitrogen Load in Effluent (kg/yr)		Load Change (kg/y)
	WNMP	WWTF Evaluation	WNMP	WWTF Evaluation	WNMP	WWTF Evaluation	
Wampanoag	14,400	30,700	8	5	160	210	50
Mashpee Commons [2]	315,700	357,700 [3]	5	5	2,180	2,470	290
Site 4	389,500	388,200	5	5	2,690	2,680	(10)

Notes:
1. Flows rounded to nearest hundreds. Loads rounded to the nearest 5 kg/yr.
2. Mashpee Commons flows based on Phase 3 flows.
3. Includes a small portion of flow from Study Area C2 (Subarea "P2A-S" from WNMP of approximately 11,500 gpd average annual) originally designated for treatment at Site 6 under a later phase of the WNMP implementation.

As an alternative to this approach for the evaluation, by improving the performance at each WWTF in the future, a significant load change can be obtained based on the same flows as shown in Table 39. The improved performance would be to go from 5 mg/L TN in the effluent to the limit of technology (3 mg/L TN). These improvements are reflected in Table 40.

Table 40 Estimated Change in Nitrogen Load with Improved Performance (3 mg/L TN)

WWTF	Estimated Avg. Future WWTF Flow (gpd)		Estimated Eff. Quality at WWTF (mg/L TN)		Estimated Nitrogen Load in Effluent (kg/yr)		Load Change (kg/y)
	WNMP	WWTF Evaluation	WNMP	WWTF Evaluation	WNMP	WWTF Evaluation	
Wampanoag	14,400	30,700	8	3	160	130	(30)
Mashpee Commons [2]	315,700	357,700 [3]	5	3	2,180	1,485	(695)
Site 4	389,500	388,200	5	3	2,690	1,610	(1,080)

Notes:
1. Flows rounded to nearest hundreds. Loads rounded to the nearest 5 kg/yr.
2. Mashpee Commons flows based on Phase 3 flows.
3. Includes a small portion of flow from Study Area C2 (Subarea "P2A-S" from WNMP of approximately 11,500 gpd average annual) originally designated for treatment at Site 6 under a later phase of the WNMP implementation.

In examining the potential impacts of any change in estimated nitrogen load, consideration was given to the following conditions:

- Each of the potential recharge locations (Mashpee Commons, Site 4, and the Wampanoag WWTF site) are all within the Mashpee River watershed, and therefore their impacts are influenced by whether they are in the upper watershed (Site 4 and Wampanoag) or the lower watershed (Mashpee Commons).
- Mashpee Commons potential build-out flows. The facility may ultimately construct a WWTF to collect and treat up to 280,000 gpd (peak flow) or approximately an increase of 65,000 gpd

average annual flow over the 180,000 gpd currently permitted and modeled as part of the WNMP.

Table 41 reflects the increased Mashpee Common's flow at its Phase 4 build-out potential and the impact those flow changes and improved wastewater treatment has on the total nitrogen load generated at each facility.

Table 41 Estimated Change in Nitrogen Load with Improved Performance (3 mg/L TN) and Build-out at Mashpee Commons to 280,000 gpd (peak)

WWTF	Estimated Avg. Future WWTF Flow (gpd)		Estimated Eff. Quality at WWTF (mg/L TN)		Estimated Nitrogen Load in Effluent (kg/yr)		Load Change (kg/y)
	WNMP	WWTF Evaluation	WNMP	WWTF Evaluation	WNMP	WWTF Evaluation	
Wampanoag	14,400	30,700	8	3	160	130	(30)
Mashpee Commons [2]	315,700	424,400 [3]	5	3	2,180	1,760	(422)
Site 4	389,500	388,200	5	3	2,690	1,610	(1,080)

Notes:

1. Flows rounded to nearest hundreds. Loads rounded to the nearest 5 kg/yr.
2. Mashpee Commons Flows based on Phase 3 flows.
3. Includes a small portion of flow from Study Area C2 (Subarea "P2A-S" from WNMP of approximately 11,500 gpd average annual) originally designated for treatment at Site 6 under a later phase of the WNMP implementation. Mashpee Commons flow also increase up to the recharge capacity of the site (peak of 280,000 gpd, or +66,000 gpd average annual).

As part of the WNMP, the Recommended Plan identified the alternative of recharge at the Willowbend Golf Course since the eastern subwatersheds of Popponesset Bay (Shoestring Bay) have a higher threshold for nitrogen. This alternative was proposed to take into consideration potential cost savings of recharge closer to the Site 4 WWTF. This option also took into consideration the potential benefit of shellfish aquaculture use in the Popponesset Bay watershed. However, if the shellfish program is unable to meet its nitrogen removal goals and buildout is to be achieved in the community, flow from Mashpee Commons and other facilities in the watershed would potentially need to be pumped to New Seabury (Site 7) for subsurface discharge outside of the Popponesset Bay and Waquoit Bay watersheds.

Now, as adaptive management allows for, and as Mashpee Commons plans for potential expansion to 280,000 gpd (peak flow), alternatives for treatment and recharge of wastewater are presented for consideration as part of this evaluation. Based on preliminary discussions with Mashpee Commons (Tom Feronti), the MCLP WWTF site may be able to accommodate treatment up to 550,000 gpd; however, the portion of recharge above 280,000 gpd (peak) would need to be relocated to another site. Therefore, in this evaluation, consideration is made to relocate flow to Site 4 (either all or in part) and examine its potential impact with the assumption that the shellfish program is able to meet or exceed the performance identified in the WNMP.

The WNMP Recommended Plan (with shellfish) was based on Mashpee Phase 3 flows treated to 5 mg/L TN and recharged at Mashpee Commons existing open sand beds; Site 4 flows treated to 5 mg/L TN and recharged at Willowbend Golf Course; and Wampanoag WWTF treating to 8 mg/L TN and recharging at their existing subsurface leaching facilities.

Adaptive Approach 1: Take all flows from the Study Areas proposed for Mashpee Commons, Wampanoag WWTF, and Site 4 and treat them to 5 mg/L TN and recharge all flows within the Mashpee River Upper Watershed. This could be at Site 4, or Site 4 and the Wampanoag WWTF.

Adaptive Approach 2: Take all flows from the Study Areas proposed for Mashpee Commons, Wampanoag WWTF, and Site 4 and treat them to 3 mg/L TN and recharge all flows within the Mashpee River Upper Watershed. This could be at Site 4, or Site 4 and the Wampanoag WWTF. This approach also accounts for an increase of the Mashpee Commons development to 280,000 gpd peak flow (up from the Phase 3 flows).

If, at buildout, all of the wastewater from the Study Areas in addition to the buildout flow at Mashpee Commons was treated to 3 mg/L and recharged at Site 4, and the WNMP shellfish program is successful, the TMDL goals should be achieved based on the Landuse Model "Rainbow Spreadsheets" developed by MEP. This would need to be verified through water quality monitoring and modeling through their linked model to confirm. This would also depend on several factors including potential facility expansion at Mashpee Commons, possible WWTF construction at Site 4, and effluent recharge at Site 4 to serve both areas.

The following Table 42 shows the TMDL thresholds in kg/d for the major embayment systems for the Popponesset Bay System and provides a comparison of the Recommended Plan with shellfish and the two adaptive approaches discussed above, maximizing the recharge at Site 4.

Table 42 Popponesset Bay System Nitrogen Loads with Shellfish Nitrogen Removal Credit at Build-Out

Embayment	TMDL Threshold (kg/d)	WNMP Recommended Plan (kg/d)	Adaptive Approach 1 (kg/d)	Adaptive Approach 2 (kg/d)
Popponesset Bay System (Total)	40.18	40.03	40.50	36.03
Mashpee River	16.17	16.67	21.56	17.09
Shoestring Bay Total	19.71	19.80	14.92	14.92
Ockway Bay	0.76	2.52	2.52	2.52
Popponesset Bay	3.54	0.32	0.32	0.32

Notes:

1. Popponesset Bay System (Total) includes all subwatersheds to that system.
2. Mashpee River includes: Mashpee River Upper and Lower subwatersheds.
3. Shoestring Bay Total includes: Shoestring Bay, Santuit River, and Quaker Run subwatersheds.
4. Popponesset Bay includes: Pinquickset Cove, Popponesset Creek, and Popponesset Bay subwatersheds.

As shown in Table 42, as the wastewater treatment flows approach build-out conditions for the entire service area and are recharged at Site 4 (approximately 850,000 gpd average annual), the overall load is still near the TMDL threshold for the system; however, Mashpee River subwatershed would exceed its limit. This was initially addressed in the WNMP by recharging treated effluent in the Shoestring Bay subwatershed via the Willowbend Golf Course. However, with the use of Membrane technology at Site 4 and Mashpee Commons, this load may be reduced by reducing the effluent concentration to 3 mg/L and therefore approaching or exceeding the overall nitrogen reduction goals. Due to the large number of variables that can potentially impact the nitrogen load to the embayments, the Town will have to monitor these as part of their compliance program. These variables include: will flows reach build-out, will shellfish remove more or less of the nitrogen anticipated, will there be major landuse changes in the watershed that could impact nitrogen loads from other sources, etc. As a result, the Town will need to apply adaptive management approaches or look to relocate recharge to Site 7 if TMDL compliance is lagging.

7. Summary

An evaluation was conducted of two private WWTFs located within the Town of Mashpee (Mashpee Commons WWTF and Wampanoag WWTF) to consider possible sewer extensions and their available capacity relative to the capacity needs of the areas identified for sewering as part of the Town's May 2015 WNMP Final Recommended Plan/Final Environmental Impact Report (FRP/FEIR). The analysis also examined potential changes to nitrogen loading impacts from these facilities.

A preliminary collection system was developed for three Study Areas in close proximity to the two facilities using SewerCAD Version 8i by Bentley System, Inc. and available GIS information. During the project, the Scope was amended to update and expand the existing Phase 1 sewer model around Site 4 to reflect the sewersheds outlined in the RFP/FEIR dated May 2015. The amended scope did not include the development of cost estimates for the updated Phase 1 sewer model.

Major findings of the evaluation are summarized in this Section.

7.1 Treatment Capacity Evaluation

7.1.1 Wampanoag WWTF

The Wampanoag WWTF is an RBC-type system. The facility is operational, however it does not have any properties connected to it and therefore currently does not treat any flow. The proposed collection system to serve the Wampanoag community development has not yet been constructed. The overall capacity of the facility is 40,000 gpd and the unit process capacity analysis (Table 43) indicated that the most limiting process at the facility is the RBC.

The Wampanoag WWTF Permit (MassDEP Permit No 918-0, dated July 4, 2011) requires that a nitrogen reduction strategy be developed to offset the expected nitrogen load of the housing development. The treatment facility design designates 25,000 gpd for off-site nitrogen mitigation.

The design flow is based on Title 5 flows. A metered flow analysis indicates that an additional 3,910 gpd of excess capacity may be available for off-site nitrogen mitigation (28,910 gpd overall available for off-site nitrogen mitigation). It should be noted that the Tribe is currently involved in ongoing litigation concerning it status regarding Land-in-Trust. The findings of the litigation may affect whether the Wampanoag WWTF is subject to the terms of the MassDEP permit in the future.

Table 43 Wampanoag WWTF - Unit Process Capacity Analysis

Parameter	Maximum Unit Capacity (gpd)	Expansion Potential
Primary Settling	53,000	Unit process could be expanded through the installation of additional septic tanks in the yard.
Flow Equalization	40,000 (61,000)	Per the "small WWTF guidelines", if the design flow of the facility increases to over 40,000 gpd, the required flow equalization capacity decreases (from 50% to 33%) and the existing infrastructure unit capacity would increase to 61,000 gpd.
Biological Treatment	40,300	A second RBC would need to be installed, likely in a new process building.
Secondary Settling	47,100	Testing would be required to determine if increasing the SOR of the existing clarifier would adversely impact the process.

Parameter	Maximum Unit Capacity (gpd)	Expansion Potential
Denitrification	50,000	A second denitrification filter would need to be installed, likely in a new process building.
Post Aeration	60,000	Unit process could be expanded through the installation of an additional tank in the yard.
Effluent Disinfection	50,000	A second UV disinfection unit would need to be installed, likely in a new process building.
Effluent Disposal	41,580	Additional leaching chamber capacity would need to be installed. A hydrogeological evaluation would be needed to determine if adequate effluent disposal capacity exists on the site.

7.1.2 Mashpee Commons (MCLP) WWTF

The MCLP WWTF is a membrane-type system. The facility has undergone several expansions. The current facility (Phase 3) has a design flow of 180,000 gpd (based on Title 5 flows). The facility currently treats flow from Mashpee Commons development and four municipal buildings. With the exception of one exceedance of effluent TSS and effluent TN, the facility has consistently been able to meet the effluent limits outlined in its discharge permit between January 2014 and January 2016. Plant data from the same time period indicates that the facility is currently using approximately 40% of its peak design capacity. MCLP has not fully developed all of the properties that were identified for connection to the facility in Phase 3.

Based on the available information on the facility it appears that all of the processes at the MCLP WWTF are sized adequately to treat a design flow of 180,000 gpd. The MBR is a proprietary process and not enough information was provided to adequately confirm the capacity of the process. It is recommended that a capacity evaluation be conducted by the MCLP design engineer and the MBR manufacturer prior to the connection of additional flow to the facility.

The design drawings from the most recent upgrade at the facility outline proposed upgrades, including a new Process Building and secondary treatment train that would be required to increase the capacity of the facility to 280,000 gpd. MCLP has indicated that effluent disposal is the limiting unit process for future expansions. The maximum effluent disposal capacity of the site in its current configuration is 280,000 gpd.

Table 44 MCLP WWTF - Unit Process Capacity Analysis

Parameter	Maximum Unit Capacity (gpd)	Expansion Potential
Primary Settling	N/A[1]	Unit process could potentially be expanded through the installation of additional septic tanks in the yard.
Flow Equalization	N/A[2]	Unit process could potentially be expanded through the installation of additional tankage in the yard.
Biological Treatment	180,000[4]	The proposed Phase 4 layout indicates that an additional MBR will be installed when the facility is upgraded to 280,000 gpd. The existing Process Building footprint will need to be expanded to house the equipment.
Effluent Disinfection	180,000[3]	Additional disinfection capacity would need to be installed to treat the Phase 4 upgrade flow.

Parameter	Maximum Unit Capacity (gpd)	Expansion Potential
Effluent Dosing and Disposal	180,000	The proposed Phase 4 layout indicates that an additional 100,000 gpd of effluent disposal capacity will be added to the facility when the facility is upgraded to 280,000 gpd. MCLP staff have indicated that the effluent disposal capacity of the site is 280,000 gpd. The proposed Phase 4 layout does not indicate any additional effluent dosing capacity will be installed during the upgrade.

Notes:

1. Not Applicable - Industry design guidelines do not provide sizing guidelines for septic tank pretreatment for facilities with a design flow over 150,000 gpd.
2. Not Applicable - Industry design guidelines do not provide sizing guidelines for flow equalization for facilities with a design flow over 150,000 gpd.
3. MCLP staff have indicated that the design capacity of the existing UV system is 180,000 gpd. No information on the system was provided to verify the design capacity.
4. Maximum unit capacity as listed in the 'Wastewater Treatment Plant Modifications Mashpee Commons WWTP, Mashpee, Massachusetts Engineer's Report (WP68 Permit), prepared by Stantec Consulting Services Inc., dated August 2007 (2007 Engineers Report). The design engineer and manufacturer need to evaluate the unit process capacity of the existing system.

7.2 Study Area and Collection System Evaluations

Three Study Areas were identified for potential connection to the two private WWTFs. Estimated flows from each Study Area are described below.

7.2.1 Wampanoag WWTF Study Area Estimated Flows

The Wampanoag WWTF Study Area A is an expansion of the WMNP "Subarea" Q as shown in Figure 6-4 in the FRP/FEIR. Subarea Q was scheduled to be connected to the Wampanoag WWTF during Phase 1. As part of this evaluation, several adjacent parcels to the original Area Q were identified for consideration to be treated at the Wampanoag WWTF. Estimated flows for each Wampanoag WWTF Study Area are outlined in Table 45.

Table 45 Estimated Flows for Wampanoag WWTF Study Areas

Study Area	Future Average Flow (gpd)[1]	Future Peak Flow (gpd)[2]	Study Area Description
Q-A1	1,300	2,100	Primarily residential properties south of the Wampanoag WWTF along Meetinghouse Road.
Q-A2A	5,900	9,500	Primarily residential properties north of the Wampanoag WWTF, primarily along the east side of Meetinghouse Road.
Q-A2B	7,400	11,800	Primarily residential properties and one large commercial property north of the Wampanoag WWTF, along the east side of Great Neck Road North.
Q-A3	6,700	10,800	Residential, commercial, municipal properties and State/Town owned land north of the Wampanoag WWTF, along Route 130 / Main Street.
Total	**21,300**	**34,200**	

Notes:

1. Cape Cod Commission Water Use Data multiplied by a 0.9 water to wastewater conversion factor.
2. Peak Day Peaking Factor = 1.6

The metered flow analysis indicates the Wampanoag WWTF has a reserve capacity of 28,910 gpd, which could be utilized to treat off-site flow. An option could be to maximize the flow sent to the Wampanog WWTF. This could be accomplished as follows:

- Connect Study Areas Q-A1, Q-A2A, and Q-A3 (peak day flow of 21,400 gpd) to the Wampanoag WWTF.
- Consider the feasibility of sending Study Area Q-A2B flow to either:
    - Mashpee Commons WWTF,
    - the proposed Site 4 WWTF, or
    - another facility would need to be assessed.

Depending on the timing (and phasing) of the Wampanoag Village housing development construction, additional Study Areas could potentially be connected to the Wampanoag WWTF and could later be diverted to the Site 4 WWTF as the flow from the Wampanoag Village to the Wampanoag WWTF increases. This arrangement would allow the flow from these Study Areas to be treated by a WWTF prior to the construction of the Site 4 facility.

7.2.2 MCLP WWTF Study Area Estimated Flows

The two MCLP WWTF Study Areas are a subsection of WNMP Subareas P1 and P2, as shown in Figure 6-4 in the FRP/FEIR. Estimated flows for each MCLP WWTF Study Area are outlined in Table 46.

Table 46 Estimated Flows for Wampanoag WWTF Study Areas

Study Area	Future Average Flow (gpd)[1]	Future Peak Flow (gpd)[2,3]	Study Area Description
P1-B1	36,400	84,600	Includes the Quashnet School and residential neighborhoods north of Mashpee Commons
P1-B2	5,900	14,900	Consists of the Coombs School and Sandpipe Village Condominiums (existing MCLP flows have been subtracted out of the estimated flow for P1-B2)
P1-B4	4,100	9,700	Consists primarily of residential neighborhoods, a medical office building and the Town's "Kids Klub" daycare center, north of Mashpee Commons and west of Great Neck Road
Study Area B – Subtotal	**46,400**	**109,200**	
P2-C1	8,500	19,800	Consists primarily of residential properties south of Mashpee Commons and between Falmouth Road and Great Neck Road South
P2-C2	11,600	27,000	Consists primarily of residential properties south of Mashpee Commons and east of Great Neck Road South.
P2-C3	51,600	122,300	Consists of residential and commercial/industrial properties south of Mashpee Commons and along Falmouth Road

Study Area	Future Average Flow (gpd)[1]	Future Peak Flow (gpd)[2,3]	Study Area Description
P2-C4	6,800	15,800	Consists of residential properties south of Mashpee Commons and east of Great Neck Road South.
P2-C5	13,400	31,000	Consists of residential properties south of Mashpee Commons along Great Neck Road South.
P2-C6	32,400	82,000	Consists primarily of commercial and industrial properties south of Mashpee Commons and primarily between Commercial Street and Great Hay Road.
Study Area C – Subtotal	**124,300**	**297,900**	
Study Areas B and C - Total	**170,700**	**407,100**	
Notes: 1. Cape Cod Commission Water Use Data multiplied by a 0.9 water to wastewater conversion factor. 2. Residential Peak Day Peaking Factor = 2.3 3. Commercial Peak Day Peaking Factor = 2.5			

The Phase 3 design flows, developed by Stantec, indicated reserved capacity to treat flow from four municipal properties, which have all been connected to the facility. However, not all of the Mashpee Commons properties that were included in the Phase 3 design flows have been developed and connected.

The 2013-2016 plant data indicates that the facility is currently using approximately 40% of its design capacity, which translates to a remaining peak capacity of 110,000 gpd. Several options could then be considered:

- A portion of the MCLP WWTF Study Area could be connected to the existing MCLP WWTF if MCLP was willing to delay the construction/connection of a portion of the development slated for Phase 3.
- The 2013-2016 plant data indicates that the facility currently has adequate capacity to treat the Quashnet and Coombs Schools.
- Coombs School could potentially be connected to the existing Mashpee Commons collection system through a gravity connection. Connection of the Quashnet School to the existing Mashpee Commons collection system would require construction of a new pump station.

It should be noted that the limited data provided by MCLP appears to indicate that the existing MBR process may be approaching its design capacity on a solids loading bases. The facilities design engineer and MBR manufacturer should confirm remaining capacity in the existing unit prior to the connection of any additional flow.

7.2.3 Existing Mashpee Commons Hydraulic Capacity Evaluation

The SewerCAD model of the existing Mashpee Commons collection system, which was developed based on owner-provided collection system record drawings, indicates that the existing Mashpee Commons gravity collection system has adequate capacity to convey the Study Area flows during peak flow events.

It is anticipated that a new, larger, force main will need to be installed between the North Market Street pump station and the existing collection system to convey the full flow from Study Area B. It is also anticipated that both existing pump stations (North Market and Steeple Street) will require larger pumps to convey the flow from Study Area B and Study Area C.

7.2.4 Site 4 Preliminary Phase 1 Study Areas

As part of this project the existing SewerCAD model for three Study Areas was updated to reflect the preliminary layout outlined in the WMNP. The Study Areas were broken down into sub-areas as outlined in Table 47.

Table 47 Site 4 Preliminary Phase 1 Study Areas

Study Area	Study Area Description
D1	Primarily residential properties south of Falmouth Road, primarily along Quinaquisset Avenue, Orchard Road, and Mashpee Neck Road.
D2	Primarily residential properties along both sides of Falmouth Road.
D3	Primarily residential properties along Falmouth Road and Meetinghouse Road. The Area also includes the proposed Site 4 facility.
E1	Primarily residential properties to the east of Simons Narrows Road.
E2	Primarily residential properties to the east of Mashpee Neck Road and to the west of Simon Narrows Road.
E3	Primarily residential properties along Captains Row.
F1	Primarily residential properties to the south of Main Street and to the east of Noisy Hole Road.
F2	Primarily residential properties along Main Street, north of Amos Pond.
F3	Primarily residential properties along Main Street.
F4	Primarily residential properties north of Main Street and east of Cotuit Road.
F5	Primarily residential properties north of Main Street, along Cotuit Road.
F6	Primarily residential properties along Main Street and South Sandwich Road.
F7	Primarily residential properties north of Main Street and west of Cotuit Road.

7.2.5 Preliminary Collection System Layout Development

A preliminary collection system design was developed for six Study Areas. Collection system layouts were designed to convey flow to the following proposed treatment facilities:

- Study Area A – Wampanoag WWTF or Site 4
- Study Area B – MCLP WWTF
- Study Area C – MCLP WWTF
- Study Area D – Proposed Site 4 Facility
- Study Area E – Proposed Site 4 Facility
- Study Area F – Proposed Site 4 Facility

Conceptual layouts were developed for each Study Area with the goal of maximizing the extent of gravity sewers and minimizing the use of pressure sewer or vacuum sewer. In areas where gravity sewer does not appear to be feasible (most commonly due to topographical issues), low pressure sewers used. Due to high maintenance requirements and limited expansion potential, vacuum sewers were not considered. Plans and profiles of the conceptual layouts have been developed and delivered to the Town under separate cover.

7.2.6 Estimated Costs

Capital cost were developed for Study Areas A, B, and C. As discussed previously, cost estimates were not included in the amended scope of work for Study Areas D, E, and F. The Engineer's opinion of probable capital costs are outlined in Tables 48, 49, and 50.

Table 48 Engineers Opinion of Probable Capital Cost – Town-Owned and Maintained System, Study Area A

Study Area	A1	A2	A3	Study Area A Total
Estimated Quantities				
Gravity Mains (LF)	1,830	6,260	1,550	9,640
Low Pressure Sewer (LF)	0	0	0	0
Force Main (LF)	2,250	3,030	870	6,150
Gravity Manholes	25	56	7	88
Gravity Connections	10	42	14	66
Grinder Pumps	0	0	0	0
Pump Stations	1	1	1	3
Estimated Operations and Maintenance Costs ($/year)				
O&M Cost ($/yr)	$25,000	$27,000	$25,000	$77,000
Estimated Capital Costs (2017$)				
Capital Costs (2017$)	$3,500,000	$5,300,000	$3,100,000	$11,900,000
Extrapolated Capital Costs – Future Years[2]				
Capital Costs (2018$)	*$3,600,000*	*$5,500,000*	*$3,200,000*	*$12,300,000*
Capital Costs (2019$)	*$3,700,000*	*$5,700,000*	*$3,300,000*	*$12,700,000*
Capital Costs (2020$)	*$3,800,000*	*$5,900,000*	*$3,400,000*	*$13,100,0000*
Capital Costs (2021$)	*$3,900,000*	*$6,100,000*	*$4,500,000*	*$13,500,000*
Capital Costs (2022$)	*$4,000,000*	*$6,300,000*	*$4,600,000*	*$13,900,000*

Notes:

1. Estimated quantities were calculated using the SewerCAD model. No surveys have been conducted of the proposed Study Areas as part of this project.
2. Capital costs were extrapolated using a 3% inflation factor to demonstrate the impact of inflation on capital costs if the project was implemented in different years in Phase 1.
3. Total Capital Costs includes allowances for construction costs such as: a 25% construction contingency; 30% legal, fiscal and engineering costs; and a 5% mobilization cost. The contingency is provided for variability in the bidding climate, project changes before bidding, and change orders due to unforeseen conditions. The contingency is also intended to cover costs for limited power utility improvements if phase and voltage changes to the existing infrastructure are required, however the actual costs would not be known until the final design in these areas. Total capital costs are rounded to the nearest $100k.
4. Costs do not include the cost of final restoration of private property or the cost of attaining easements.
5. Trench work (gravity, low pressure, and force mains) in Town roads include excavation, backfill, and traffic control (but not police details). Full width overlay was not factored into the costs.
6. O&M costs include items for labor and parts to operate and maintain the proposed system. Total capital costs are rounded to the nearest $1,000.

Table 49 Engineers Opinion of Probable Capital Cost – Town Owned and Maintained System, Study Area B

Study Area	B1	B2	B3	B4	Study Area B Total
Estimated Quantities					
Gravity Mains (LF)	11,050	1,300	N/A	2,670	15,020
Low Pressure Sewer (LF)	1,600	0	N/A	0	1,660
Force Main (LF)	340	820	N/A	2,360	3,520
Gravity Manholes	122	7	N/A	52	181
Gravity Connections	109	3	N/A	20	132
Grinder Pumps	25	0	N/A	0	25
Pump Stations	1	N/A	N/A	1	2
Estimated Operations and Maintenance Costs ($/year)					
O&M Cost ($/yr)	$41,000	$25,000	$27,000	$41,000	$159,000
Estimated Capital Costs (2017$)[2]					
Capital Costs (2017$)	$7,000,000	$800,000	N/A	$4,100,000	$11,900,000
Extrapolated Capital Costs – Future Years[2]					
Capital Costs (2018$)	*$7,200,000*	*$800,000*	*N/A*	*$4,200,000*	*$12,200,000*
Capital Costs (2019$)	*$7,400,000*	*$800,000*	*N/A*	*$4,300,000*	*$12,500,000*
Capital Costs (2020$)	*$7,600,000*	*$800,000*	*N/A*	*$4,400,000*	*$12,800,000*
Capital Costs (2021$)	*$7,800,000*	*$800,000*	*N/A*	*$4,500,000*	*$13,100,000*
Capital Costs (2022$)	*$8,000,000*	*$800,000*	*N/A*	*$4,600,000*	*$13,400,000*

Notes:

1. Estimated quantities were calculated using the SewerCAD model. No surveys have been conducted of the proposed Study Areas as part of this project.
2. Capital costs were extrapolated using a 3% inflation factor to demonstrate the impact of inflation on capital costs if the project was implemented in different years in Phase 1.
3. Total Capital Costs includes allowances for construction costs such as: a 25% construction contingency; 30% legal, fiscal and engineering costs; and a 5% mobilization cost. The contingency is provided for variability in the bidding climate, project changes before bidding, and change orders due to unforeseen conditions. The contingency is also intended to cover costs for limited power utility improvements if phase and voltage changes to the existing infrastructure are required, however the actual costs would not be known until the final design in these areas. Total capital costs are rounded to the nearest $100k.
4. Costs do not include the cost of final restoration of private property or the cost of obtaining easements.
5. Trench work (gravity, low pressure, and force mains) in Town roads includes excavation, backfill, and traffic control (but not police details). Full width overlay was not factored into the costs.
6. O&M costs include items for labor and parts to operate and maintain the proposed system. Total capital costs are rounded to the nearest $1,000.
7. The existing Mashpee Commons collection system comprises Study Areas B3 and B5. It is assumed that any additional wastewater infrastructure in these Study Areas will be privately installed by MCLP and will not represent a capital cost to the Town.

Table 50 Engineers Opinion of Probable Capital Cost – Town Owned and Maintained System, Study Area C

Study Area	C1	C2	C3	C4	C5	C6	Study Area C Total
Estimated Quantities							
Gravity Mains (LF)	4,480	6,360	10,140	4,110	4,980	4,700	34,770
Low Pressure Sewer (LF)	0	0	0	380	520	230	250
Force Main (LF)	2,010	2,210	5,020	5,550	4,930	2,370	22,090
Gravity Manholes	42	161	116	35	121	84	559
Gravity Connections	52	105	60	59	57	32	563
Grinder Pumps	0	0	0	5	7	1	13
Pump Stations	1	1	1	1	1	1	6
Estimated Operations and Maintenance Costs ($/yr)							
O&M Cost ($/yr)	$25,000	$25,000	$41,000	$25,000	$25,000	$27,000	$168,000
Estimated Capital Costs (2017$)							
Capital Costs (2017$)	$4,500,000	$5,400,000	$7,400,000	$5,700,000	$5,900,000	$4,700,000	$33,600,000
Extrapolated Capital Costs – Future Years							
Capital Costs (2018$)	*$4,600,000*	*$5,600,000*	*$7,600,000*	*$5,900,000*	*$6,100,000*	*$4,800,000*	*$34,600,000*
Capital Costs (2019$)	*$4,700,000*	*$5,800,000*	*$7,800,000*	*$6,100,000*	*$6,300,000*	*$4,900,000*	*$35,600,000*
Capital Costs (2020$)	*$4,800,000*	*$6,000,000*	*$8,000,000*	*$6,300,000*	*$6,500,000*	*$5,000,000*	*$36,600,000*
Capital Costs (2021$)	*$4,900,000*	*$6,200,000*	*$8,200,000*	*$6,500,000*	*$6,700,000*	*$5,200,000*	*$37,700,000*
Capital Costs (2022$)	*$5,000,000*	*$6,400,000*	*$8,400,000*	*$6,700,000*	*$6,900,000*	*$5,400,000*	*$38,800,000*

Notes:

1. Estimated quantities were calculated using the SewerCAD model. No surveys have been conducted of the proposed Study Areas as part of this project.
2. Capital costs were extrapolated using a 3% inflation factor to demonstrate the impact of inflation on capital costs if the project was implemented in different years in Phase 1.
3. Total Capital Costs includes allowances for construction costs such as: a 25% construction contingency; 30% legal, fiscal and engineering costs; and a 5% mobilization cost. The contingency is provided for variability in the bidding

climate, project changes before bidding, and change orders due to unforeseen conditions. The contingency is also intended to cover costs for limited power utility improvements if phase and voltage changes to the existing infrastructure are required, however the actual costs would not be known until the final design in these areas. Total capital costs are rounded to the nearest $100k.

4. Costs do not include the cost of final restoration of private property or the cost of obtaining easements.
5. Trench work (gravity, low pressure, and force mains) in Town roads includes excavation, backfill, and traffic control (but not police details). Full width overlay was not factored into the costs.
6. O&M costs include items for labor and parts to operate and maintain the proposed system. Total capital costs are rounded to the nearest $1,000.

7.3 Nitrogen Loading Analysis

As part of the development of the Town's FRP/FEIR, the estimated nitrogen load to each subwatershed was calculated. As part of the evaluation, the modified Study Areas were established for the treatment and recharge of wastewater. Since these Study Areas and points of treatment and potential recharge were modified from the original plan, a nitrogen loading analysis was conducted to determine if these shifts would have a significant impact on the overall nitrogen management plan established as part of the WMNP.

Two alternative approaches are presented in Section 6 and are summarized below:

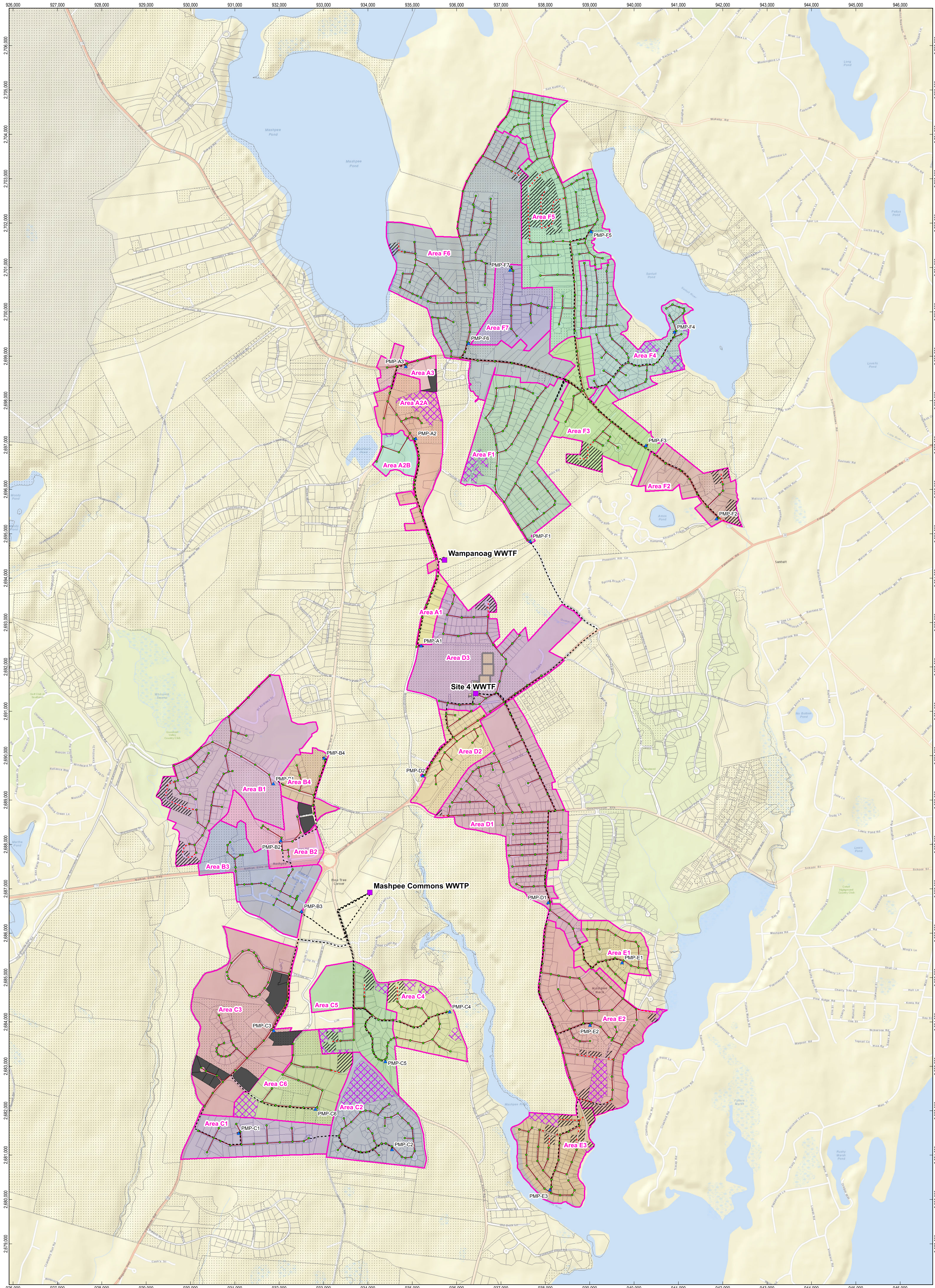
- Adaptive Approach 1: Take all flows from the Study Areas proposed for Mashpee Commons, Wampanoag WWTF, and Site 4 and treat them to 5 mg/L TN and recharge all flows within the Mashpee River Upper Watershed. This could be at Site 4, or Site 4 and the Wampanoag WWTF.
- Adaptive Approach 2: Take all flows from the Study Areas proposed for Mashpee Commons, Wampanoag WWTF, and Site 4 and treat them to 3 mg/L TN and recharge all flows within the Mashpee River Upper Watershed. This could be at Site 4, or Site 4 and the Wampanoag WWTF. This approach also accounts for an increase of the Mashpee Commons development to 280,000 gpd peak flow (up from the Phase 3 flows).

Based on these two approaches, TMDL nitrogen loading compliance was calculated and presented in Table 42. Each option approached or exceeded the TMDL compliance point for the Popponesset Bay System. However, due to the large number of variables that can potentially impact the nitrogen load to the embayments, if selected, the Town will have to monitor these as part of their compliance program. These variables include: will flows reach build-out, will shellfish remove more or less of the nitrogen anticipated, will there be major landuse changes in the watershed that could impact nitrogen loads from other sources, etc.

As a result, the Town will need to apply adaptive management approaches or look to relocate recharge to Site 7 if TMDL compliance is lagging.

Attachments

Attachment 1
Study Areas A through F



Legend

WWTP	Conduit	Parcels Not Connected (No Anticipated Future Water Use)	Sewer Sheds
Pump	Pressure Pipe	Existing I&A Parcels	Parcels
Manhole	Low Pressure Pipe	Parcels Requiring Grinder Pump	Municipal Property

Sewer Sheds

Area A3	Area B4	Area C4	Area D2	Area E3	Area F3	Area F7
Area A1	Area B1	Area C1	Area C5	Area D3	Area E4	Area F4
Area A2A	Area B2	Area C2	Area C6	Area E1	Area F1	Area F5
Area A2B	Area B3	Area C3	Area D1	Area E2	Area F2	Area F6

Paper Size ANSI D
1 inch = 1,000 feet

0 500 1,000 2,000 3,000
Feet

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

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Map of Areas A, B, C, D, E, and F

Job Number	11110939
Revision	A
Date	Jun 06, 2017

Attachment 1

C:\ArcGIS\11110987 Mashpee\- Figures -\Map of All Areas.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

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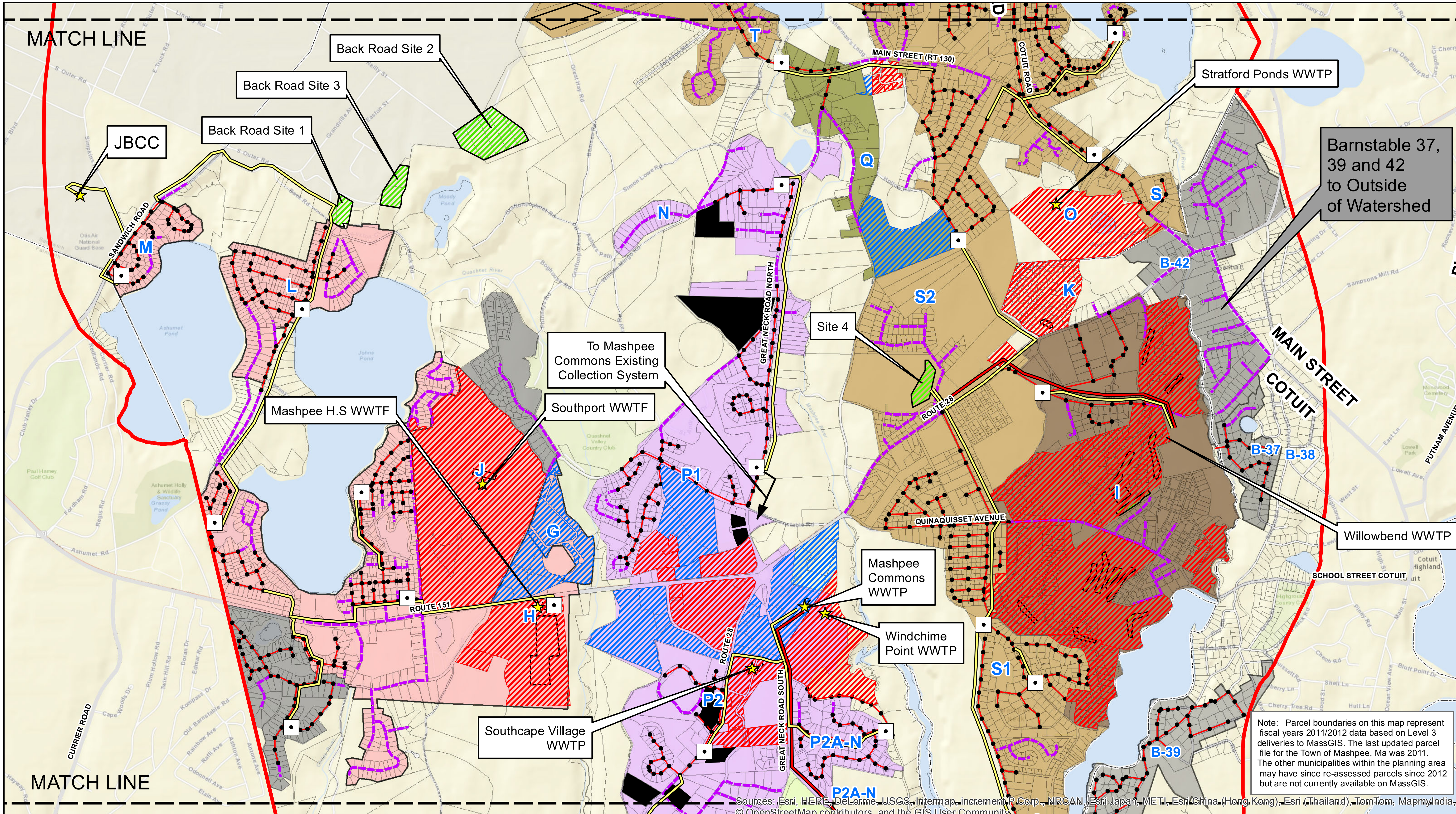
Attachment 2
Excerpts from WNMP



Paper Size ANSI B

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

N

- MMR_Site
- I/A -Large Individual or Neighborhood Only
- Proposed Treatment/Discharge Site
- Planning Area Boundary
- Parcel Boundary
- Existing Private Sewer
- Proposed Private Sewer
- Proposed Gravity MH
- Proposed Gravity Sewer
- Proposed Low Pressure Sewer
- Proposed Pump Station
- Dual Force Main
- Single Force Main
- Proposed Force Main to PS or WWTP

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Town of Mashpee Sewer Commission
Watershed Nitrogen Management Plan

Job Number	86-12001
Revision	A
Date	06 May 2015

PRELIMINARY INFRASTRUCTURE BY SUBAREA (CENTRAL PPA)

Figure 6-4

G:\86\12001\GIS\00074 Mashpee\2014\Final Report Figures\New folder\8612001F6-4_Treatment Sites.mxd

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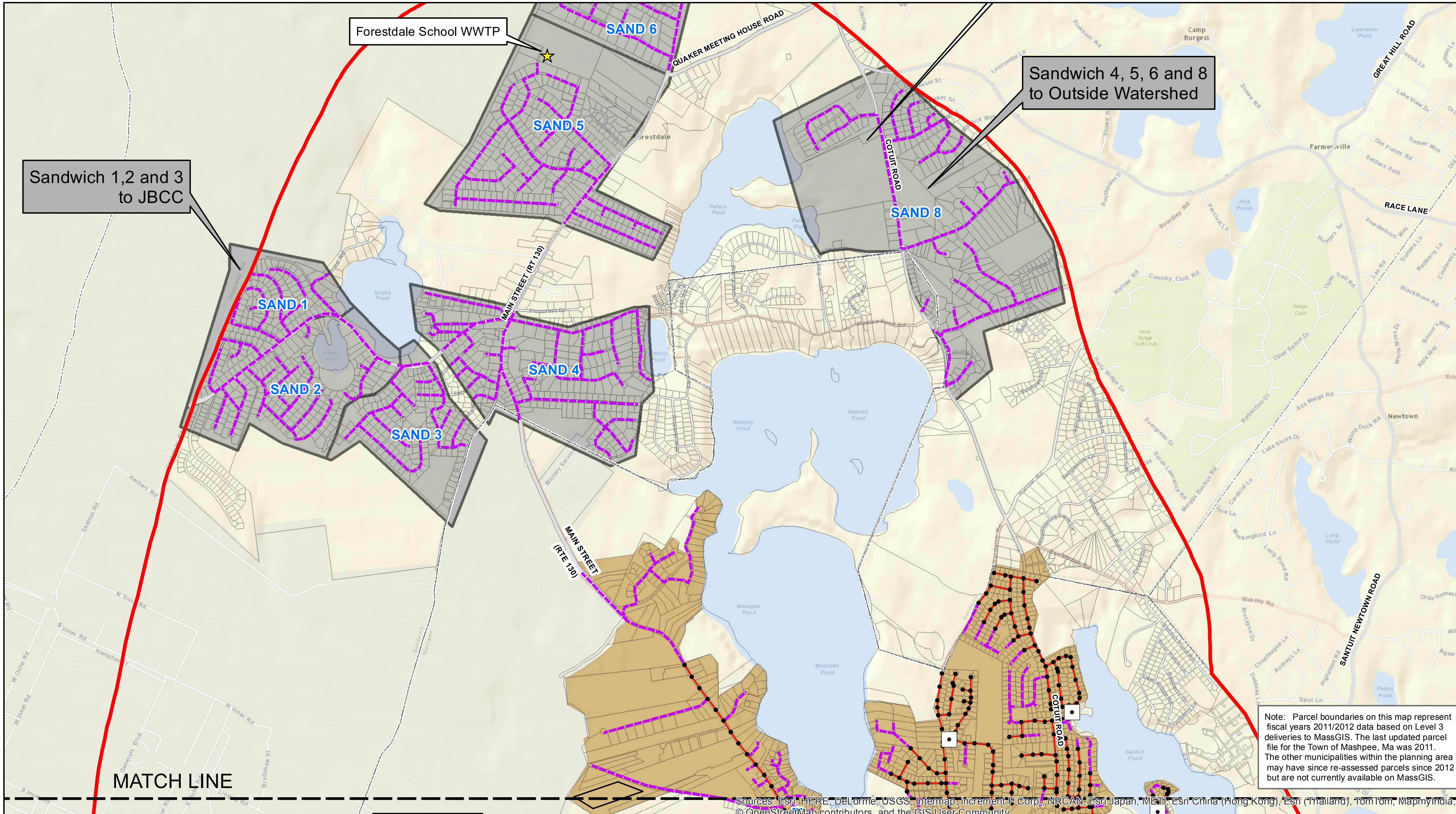
Data source: Data Custodian, Data Set Name/Title, Version/Date. Created by:jjobrien



Paper Size ANSI B

0 1,000 2,000 4,000 Feet

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

N

- MMR_Site
- I/A -Large Individual or Neighborhood Only
- Proposed Treatment/Discharge Site
- Planning Area Boundary
- Parcel Boundary
- Existing Private Sewer
- Proposed Private Sewer
- Proposed Gravity MH
- Proposed Gravity Sewer
- Proposed Low Pressure Sewer
- Proposed Pump Station
- Dual Force Main
- Single Force Main
- Proposed Force Main to PS or WWTF

Town of Mashpee Sewer Commission
Watershed Nitrogen Management Plan

Job Number	86-12001
Revision	A
Date	06 May 2015

PRELIMINARY INFRASTRUCTURE BY SUBAREA (NORTH PPA)

Figure 6-5

G:\86\12001\GIS\00074 Mashpee\2014\Final Report Figures\New folder\8612001F6-5_Sewer North.mxd

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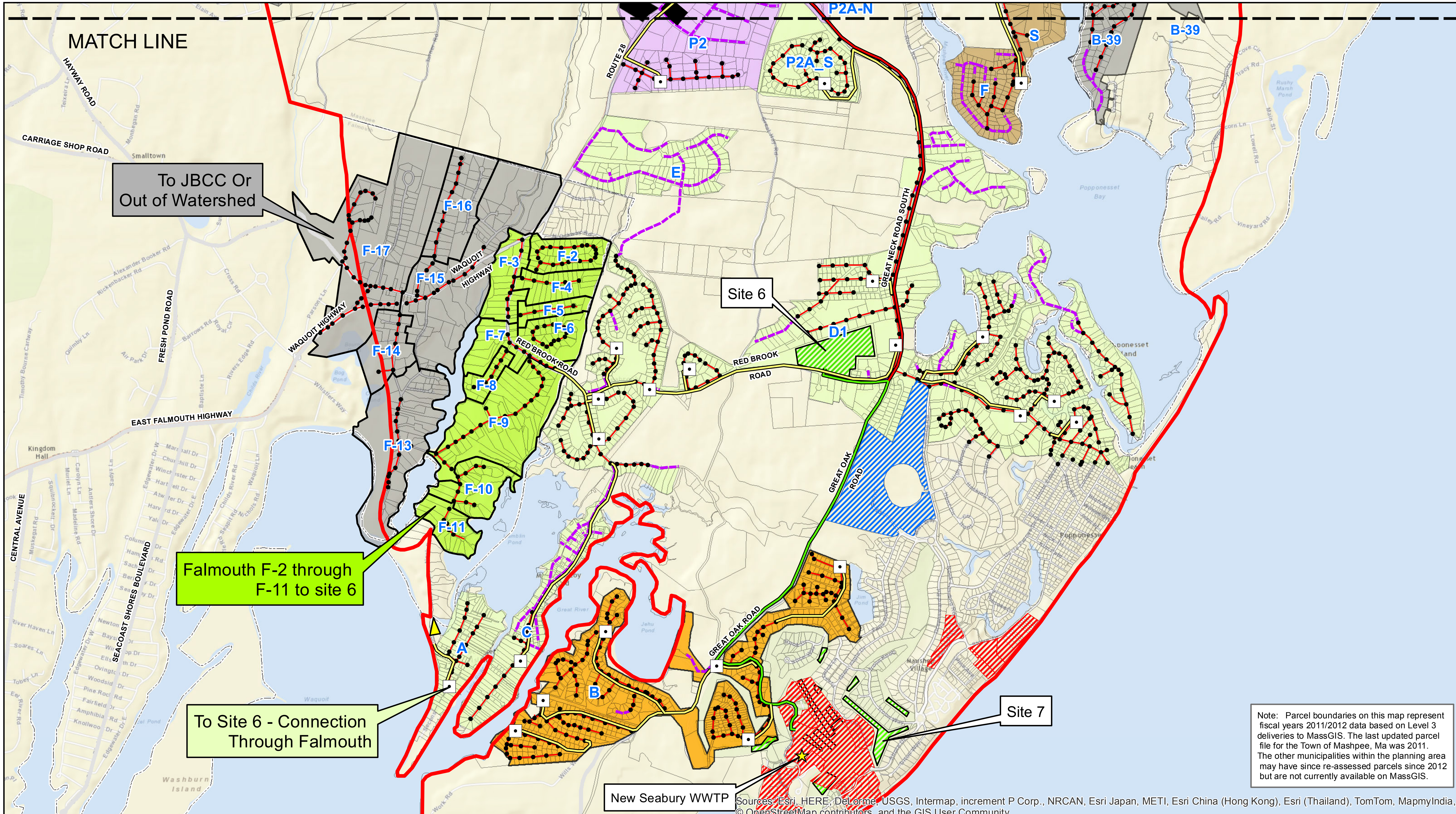
Data source: Data Custodian, Data Set Name/Title, Version/Date. Created by:jjobrien



Paper Size ANSI B

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

N

- MMR_Site
- I/A -Large Individual or Neighborhood Only
- Proposed Treatment/Discharge Site
- Planning Area Boundary
- Parcel Boundary
- Existing Private Sewer
- Proposed Private Sewer
- Proposed Gravity MH
- Proposed Gravity Sewer
- Proposed Low Pressure Sewer
- Proposed Pump Station
- Dual Force Main
- Single Force Main
- Proposed Force Main to PS or WWTF

Town of Mashpee Sewer Commission
Watershed Nitrogen Management Plan

Job Number	86-12001
Revision	A
Date	06 May 2015

PRELIMINARY INFRASTRUCTURE BY SUBAREA (SOUTH PPA)

Figure 6-6

G:\86\12001\GIS\00074 Mashpee\2014\Final Report Figures\New folder\8612001F6-6_Sewer South.mxd

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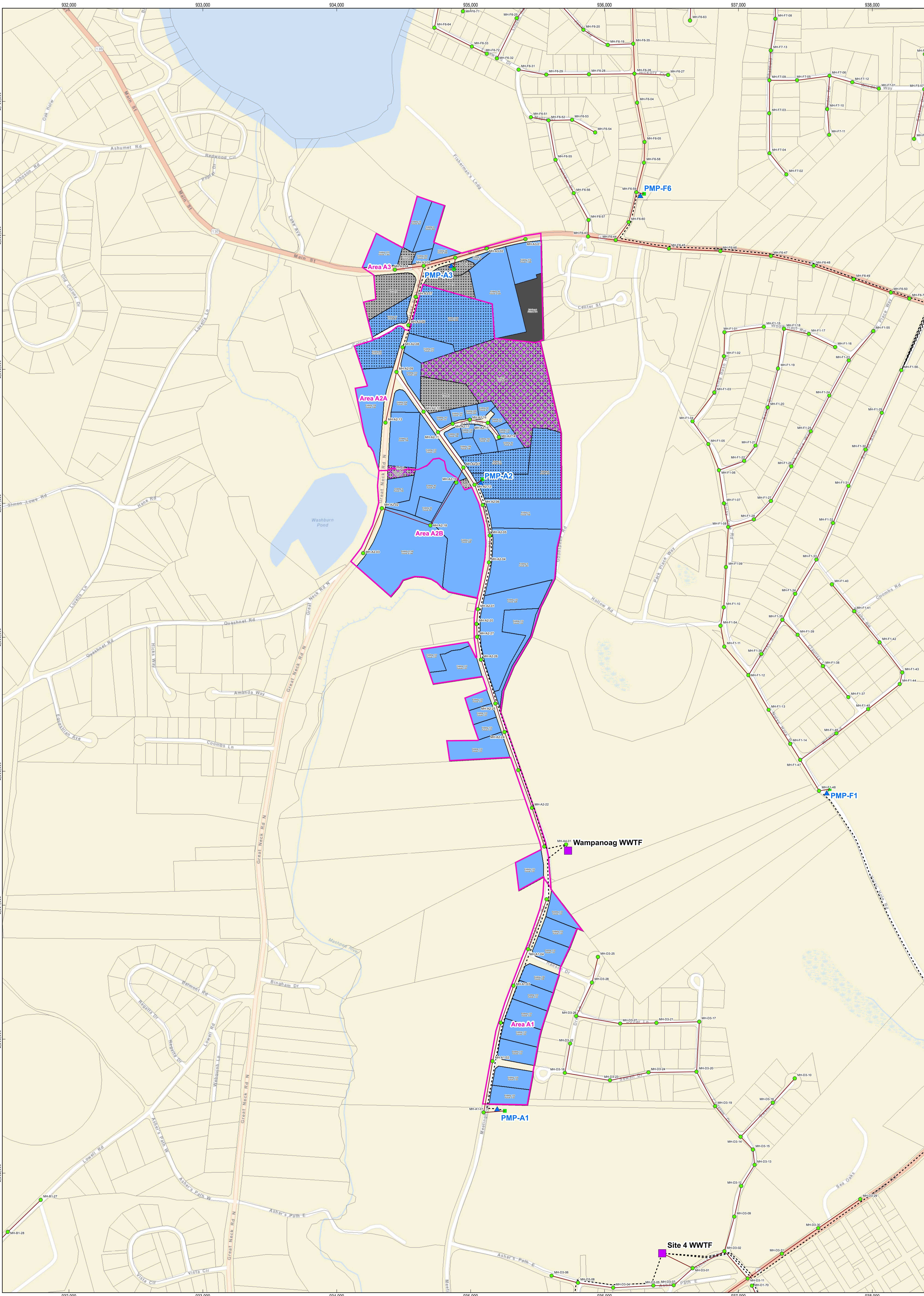
Data source: Data Custodian, Data Set Name/Title, Version/Date. Created by:jjobrien

Attachment 3

Study Areas A, B, and C



Legend

- Wet Well
- Pump
- WWTP

Manhole
- < 20 ft
- Deeper than 20 ft

- Conduit
- Pressure Pipe
- Low Pressure Pipe
- Study Areas
- Parcels
- Municipal Property
- Parcels Not Connected (No Anticipated Future Water Use)
- Existing I&A Parcels
- Parcels Requiring Grinder Pump
- Parcel Loading
- Parcels without Loading

Paper Size ANSI D
1 inch = 333 feet
0 125 250 500 750 1,000
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Model Loading - Area A

Job Number	11110939
Revision	A
Date	May 31, 2017

Figure 1

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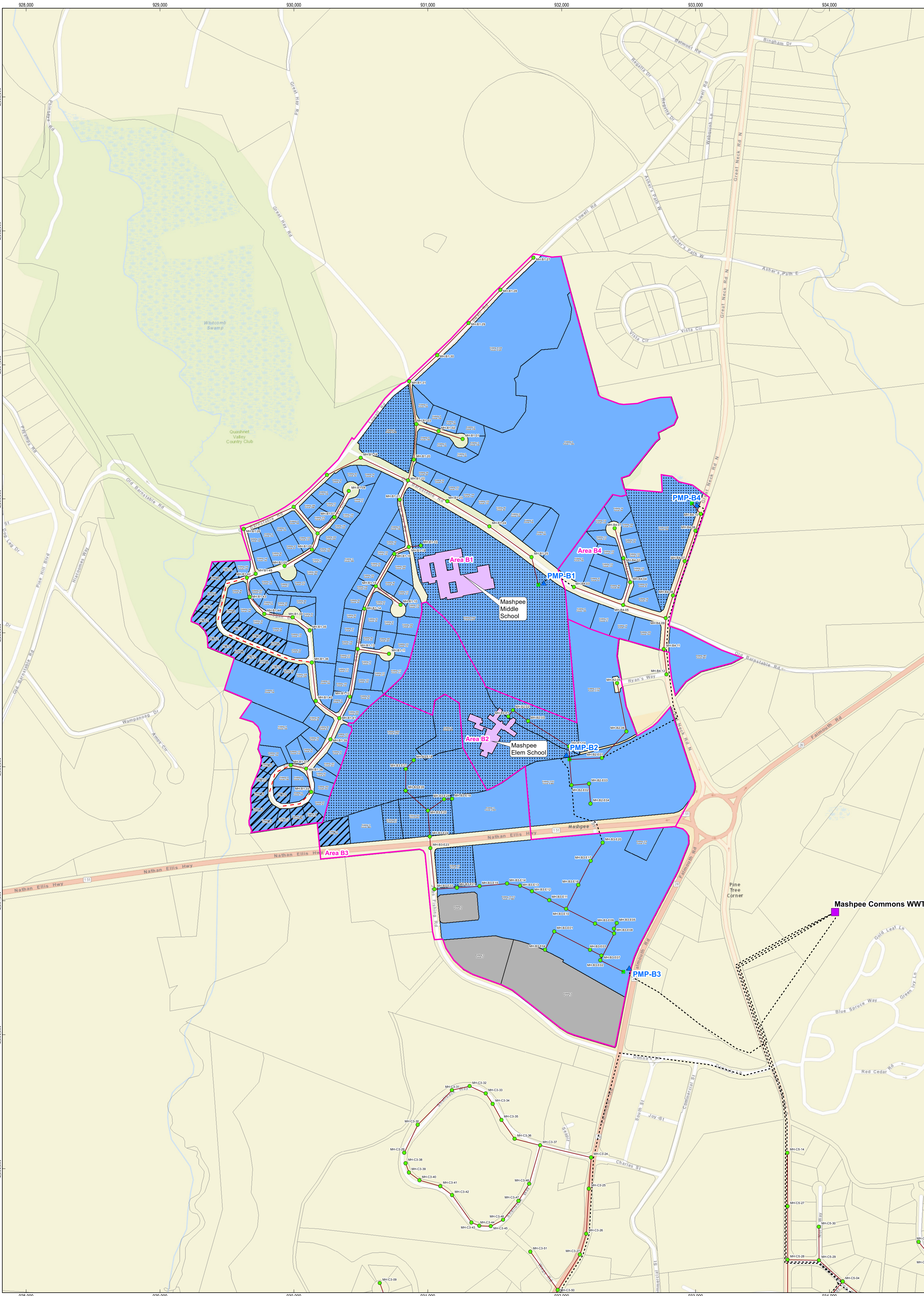

Legend

- Wet Well
- Pump
- WWTP
- **Manhole**
  - < 20 ft
  - Deeper than 20 ft
- Conduit
- Pressure Pipe
- Low Pressure Pipe
- Study Areas
- Parcels
- Municipal Property
- Parcels Not Connected (No Anticipated Future Water Use)
- Existing I&A Parcels
- Parcels Requiring Grinder Pump
- Parcel Loading
- Parcels without Loading

Paper Size ANSI D
1 inch = 333 feet

0 125 250 500 750 1,000
Feet

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

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Model Loading - Area B

Job Number	11110939
Revision	A
Date	May 31, 2017

Figure 2

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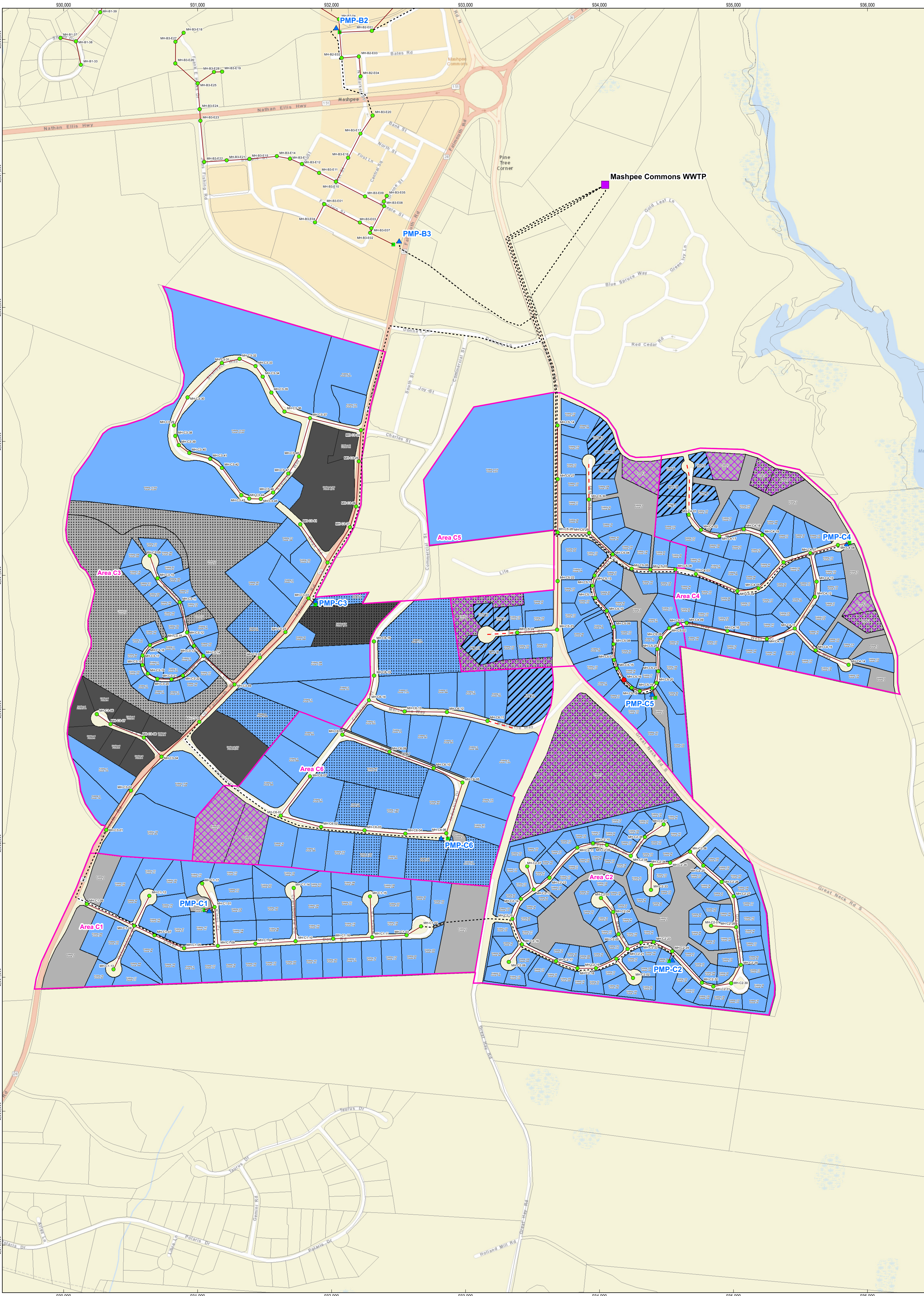

Legend

Wet Well	**Manhole**	Conduit	Study Areas	Parcels Not Connected (No Anticipated Future Water Use)	Parcel Loading
Pump	< 20 ft	Pressure Pipe	Parcels	Existing I&A Parcels	Parcels without Loading
WWTP	Deeper than 20 ft	Low Pressure Pipe	Municipal Property	Parcels Requiring Grinder Pump	

Paper Size ANSI D
1 inch = 333 feet

0 125 250 500 750 1,000
Feet

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

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Model Loading - Area C

Job Number	11110939
Revision	A
Date	May 31, 2017

Figure 3

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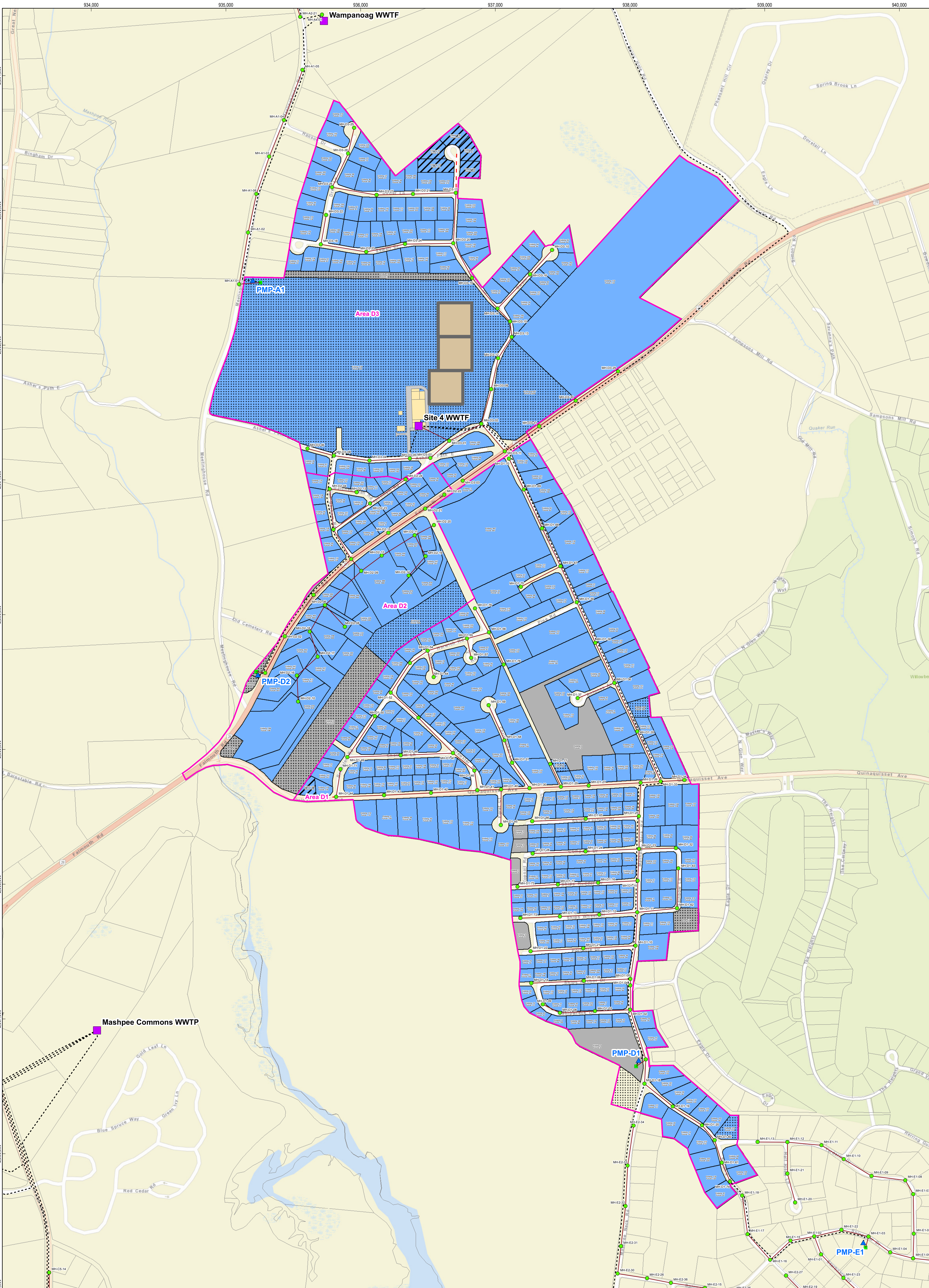
Attachment 4
Study Areas D, E, and F



Legend

Wet Well	**Manhole**	Conduit	Study Areas	Parcels Not Connected (No Anticipated Future Water Use)	Parcel Loading
Pump	< 20 ft	Pressure Pipe	Parcels	Existing I&A Parcels	Parcels without Loading
WWTP	Deeper than 20 ft	Low Pressure Pipe	Municipal Property	Parcels Requiring Grinder Pump	

Paper Size ANSI D
1 inch = 333 feet

0 125 250 500 750 1,000
Feet

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

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Model Loading - Area D

Job Number	11110939
Revision	A
Date	May 31, 2017

Figure 4

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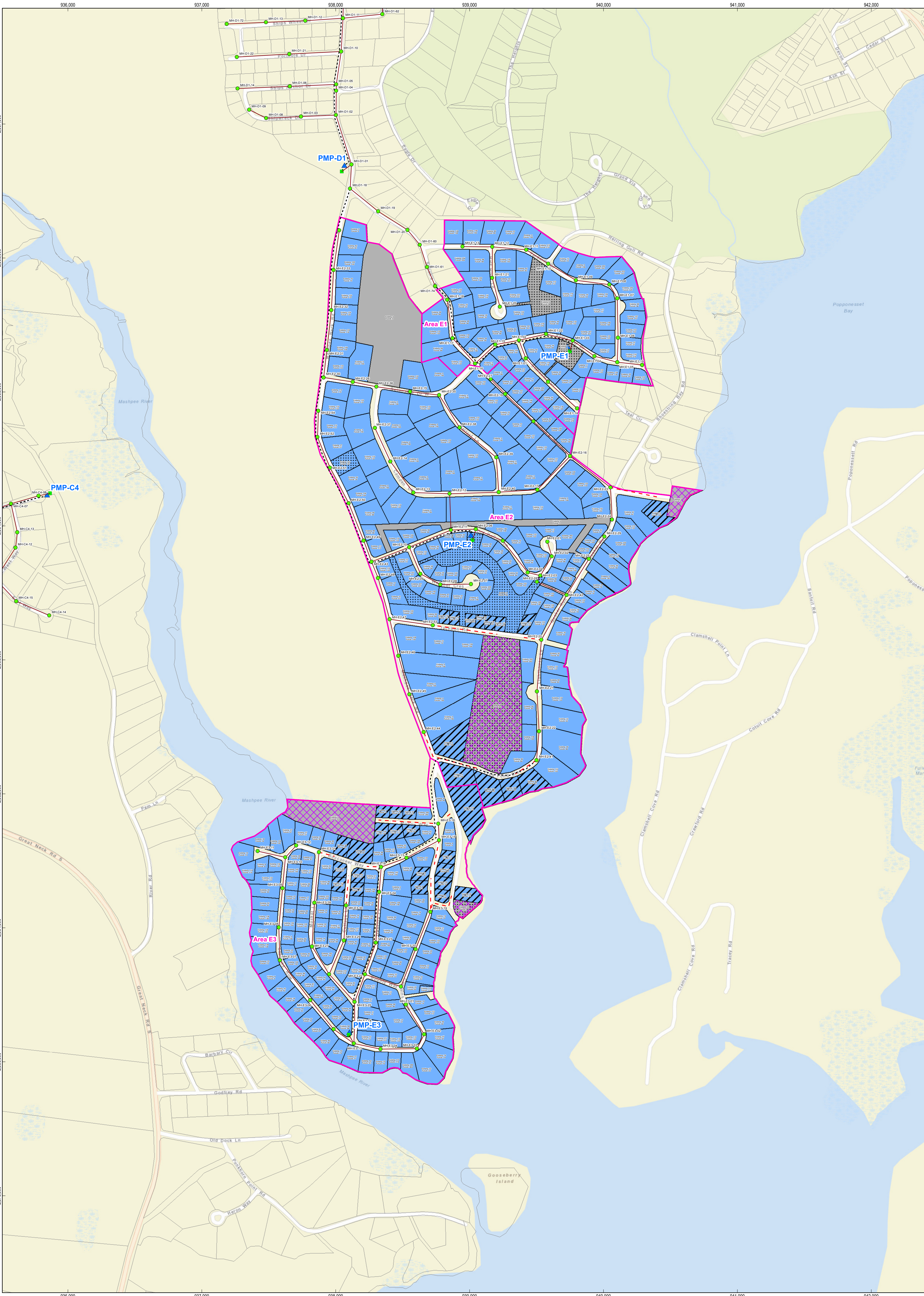


Legend

Wet Well	**Manhole**	Conduit	Study Areas	Parcels Not Connected (No Anticipated Future Water Use)	Parcel Loading
Pump	< 20 ft	Pressure Pipe	Parcels	Existing I&A Parcels	Parcels without Loading
WWTP	Deeper than 20 ft	Low Pressure Pipe	Municipal Property	Parcels Requiring Grinder Pump	

Paper Size ANSI D
1 inch = 333 feet
0 125 250 500 750 1,000
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

GHD **DRAFT**

Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Model Loading - Area E

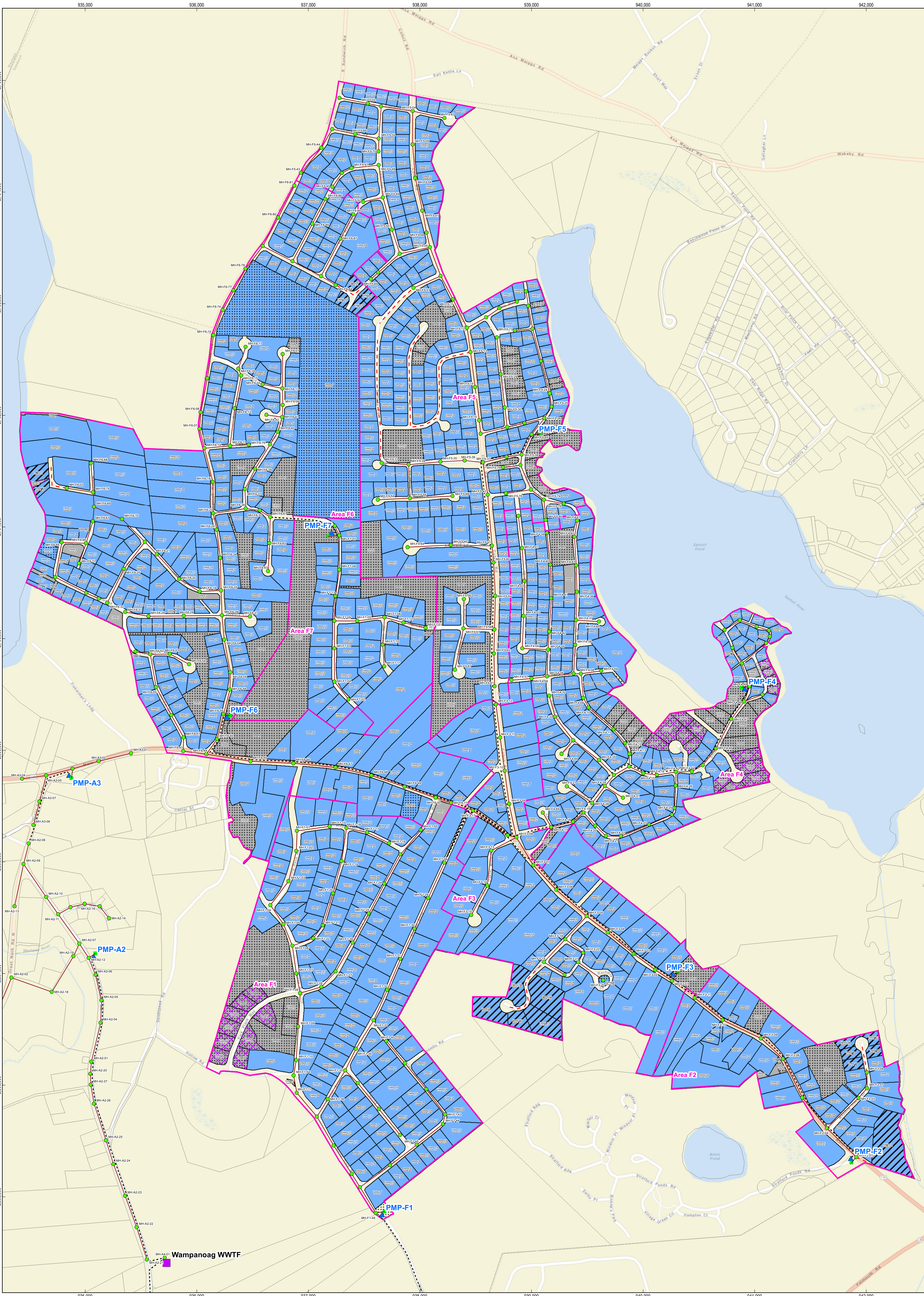
Job Number	11110939
Revision	A
Date	May 31, 2017

Figure 5

C:\ArcGIS\11110987 Mashpee\- Figures -\Model Loading - 22x34 - Areas ABCDEF.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

Created by:jsriling



Legend

Wet Well	Manhole	Conduit	Study Areas	Parcels Not Connected (No Anticipated Future Water Use)
Pump	< 20 ft	Pressure Pipe	Parcels	Existing I&A Parcels
WWTP	Deeper than 20 ft	Low Pressure Pipe	Municipal Property	Parcels Requiring Grinder Pump
			Parcel Loading	Parcels without Loading

Paper Size ANSI D
1 inch = 400 feet

0 125 250 500 750 1,000
Feet

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

N

GHD DRAFT

Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Model Loading - Area F

Job Number 11110939
Revision A
Date May 31, 2017

Figure 6

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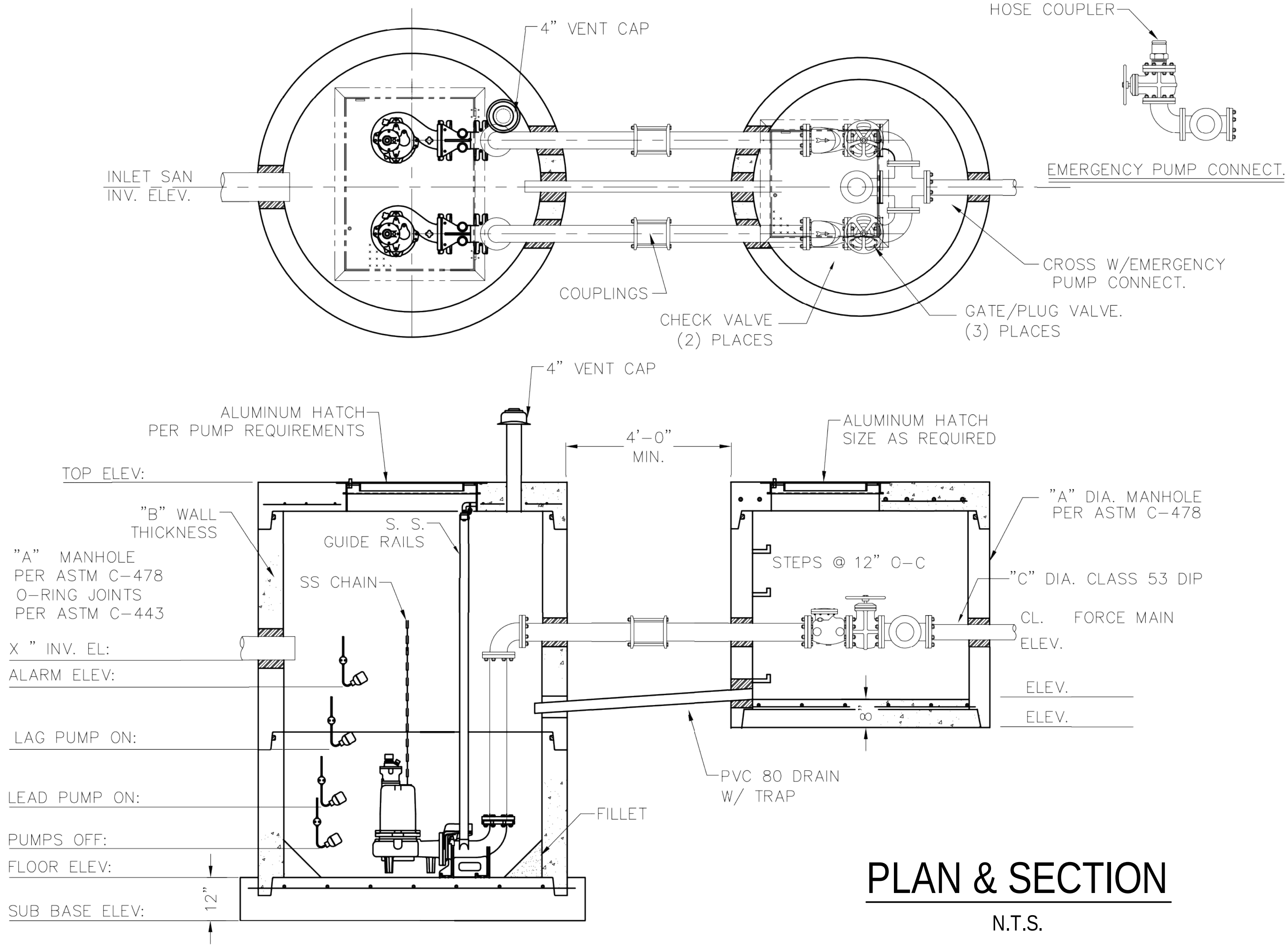
Attachment 5
Typical Pump Station Layouts



PLAN & SECTION

N.T.S.

GHD

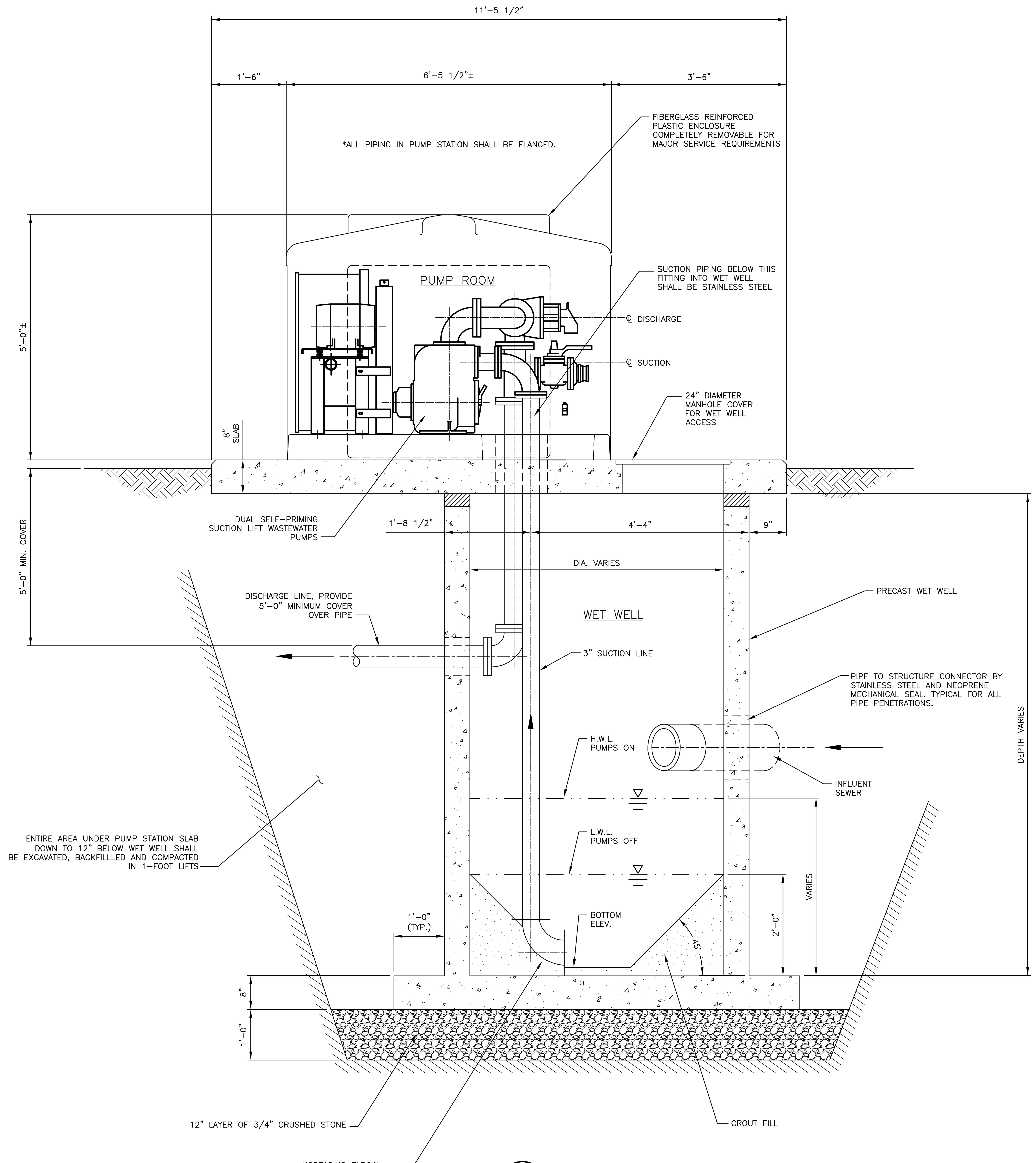
SUBMERSIBLE PUMP STATION

Job Number
Revision
Date FEB 17

Figure 01

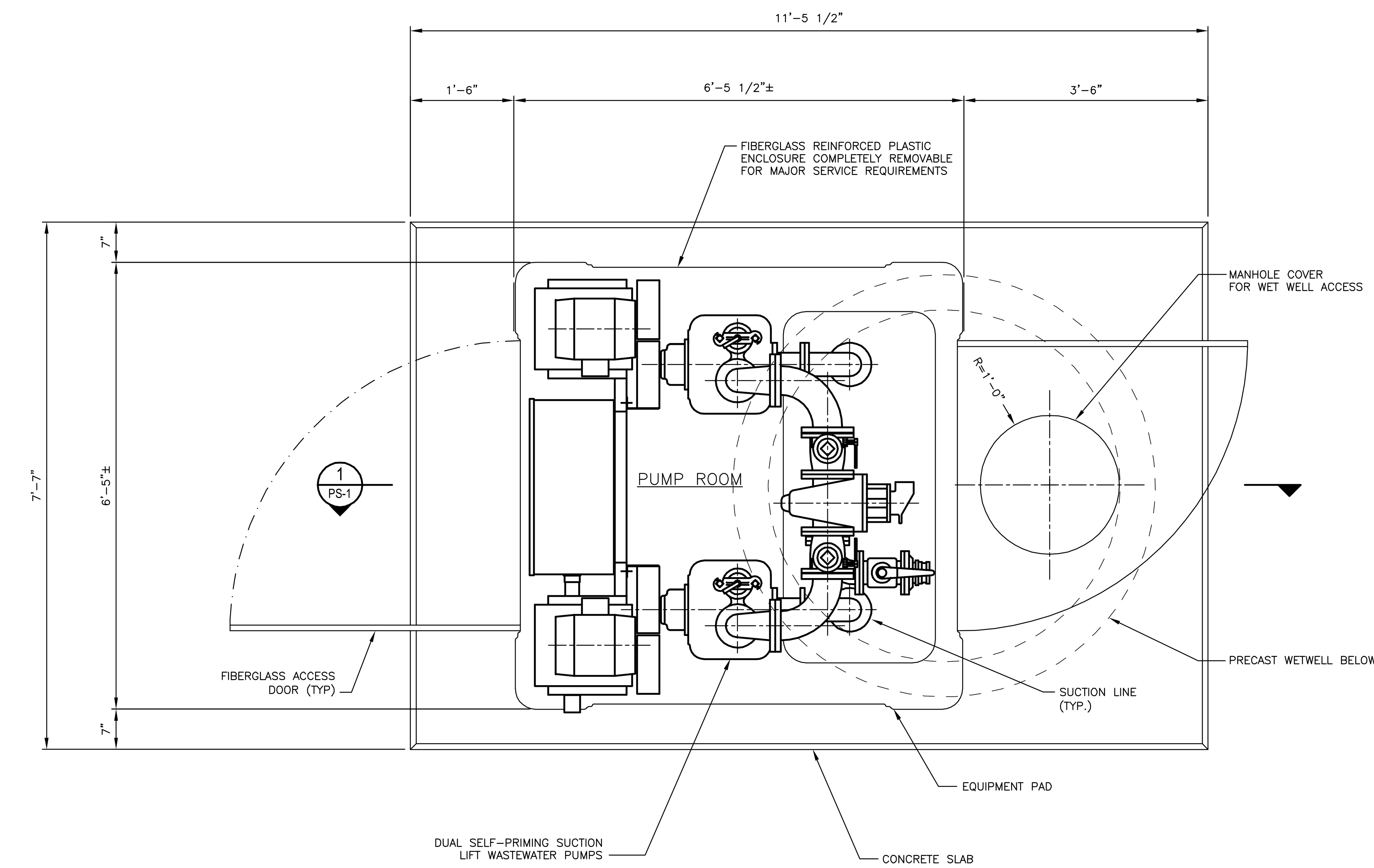
1545 Iyannough Road, Hyannis Massachusetts 02601 USA T 1 774 470 1630 F 1 774 470 1631 E hyamail@ghd.com W www.ghd.com

Plot Date: 14 February 2017 - 10:33 AM Cad File No: C:\data\SUBMERSIBLE PS.dwg



1 PS-1 SECTION

SCALE : 3/4"=1'-0"



PLAN - TYPICAL 6' X 6' ABOVE GROUND PUMP STATION

SCALE 3/4"=1'-0"

N

0 1'-0" 2'-0" 3'-0"

SCALE 3/4"=1'-0" AT ORIGINAL SIZE

NOTES:

1. MAX DEPTH OF WETWELL VARIES ON PUMP HEAD AND FLOW REQUIREMENTS. TYPICAL MAXIMUM DEPTHS RANGE FROM 15 TO 20 FEET BELOW GRADE.
2. ELECTRICAL PANELS ARE LOCATED OUTSIDE IN WEATHERPROOF ENCLOSURES.

NOT FOR CONSTRUCTION

No	Revision	Drawn	Job Manager	Project Director	Date
A	PRELIMNARY DESIGN				

Note: * indicates signatures on original issue of drawing or last revision of drawing

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.

DRAFT

GHD Inc.
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

Drawn	AEF	Designer	
Drafting Check		Design Check	
Approved (Project Director)			
Date			
Scale	AS SHOWN	This Drawing shall not be used for Construction unless Signed and Sealed For Construction	

Client **TOWN OF FALMOUTH, MASSACHUSETTS**

Project **WASTEWATER AND NUTRIENT MANAGEMENT SERVICES**

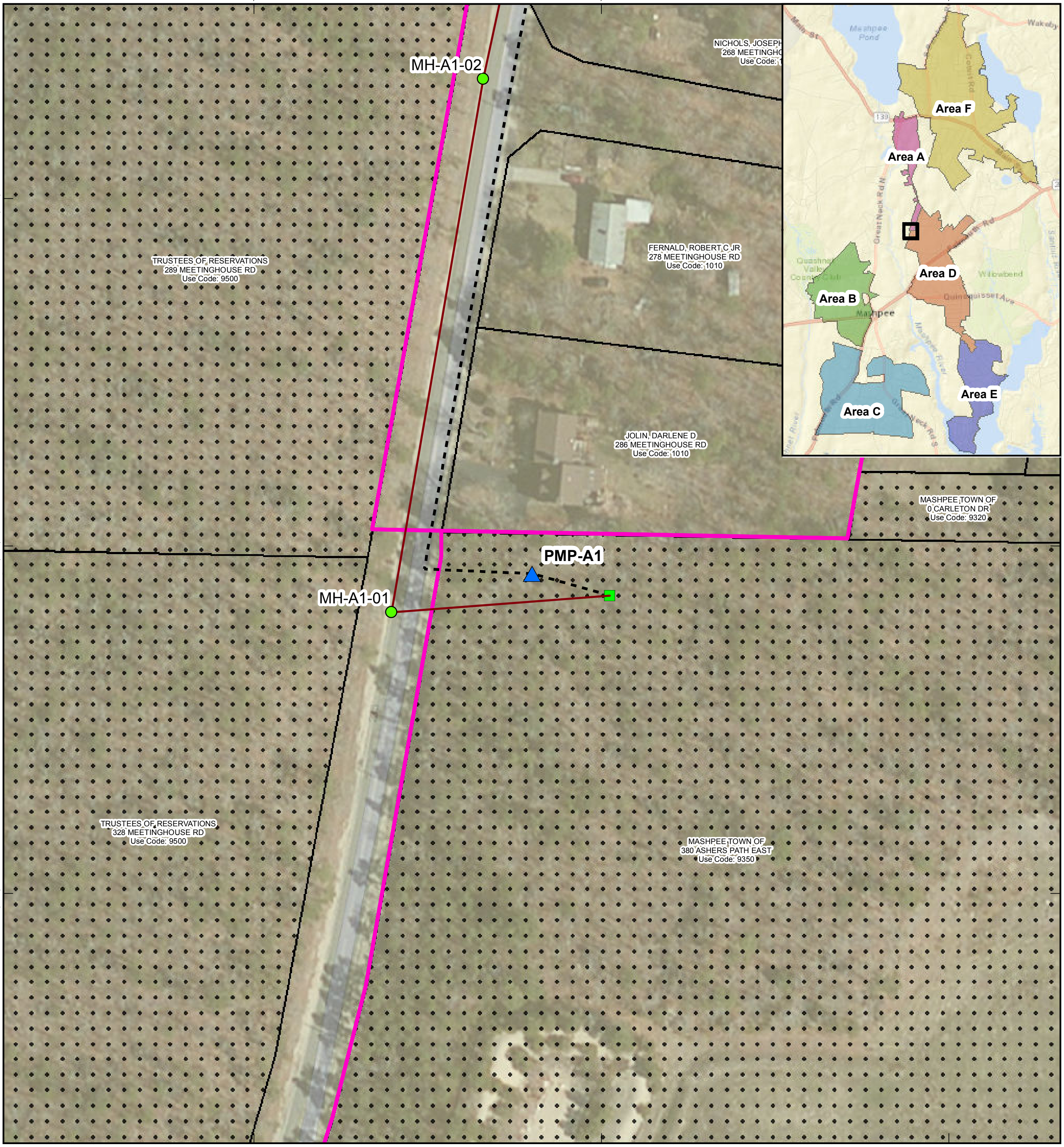
Title **6'x6' ABOVE GROUND PUMPING STATION**

Original Size Arch D **Drawing No: 86-15097-PS-01** Rev: A

Plot Date: 17 December 2012 - 2:23 PM Plotted by: Aaron Foscato Cad File No: N:\US\Hyannis\Projects\86\15097\CADD\Drawings\Architectural\86-15097-PS-01.dwg

Attachment 6

Proposed Pump Station Locations



Legend

Wet Well	Manhole	Sewer Sheds
Pump	Conduit	Parcels
WWTP	Pressure Pipe	Municipal Property

Paper Size ANSI A

1 inch = 100 feet

0 15 30 60 90 120

Feet

Map Projection: Lambert Conformal Conic

Horizontal Datum: North American 1983

Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

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Town of Mashpee, MA

Existing WWTF & Collection System Evaluation

Job Number	11110939
Revision	A
Date	Aug 24, 2017

Pump Station Parcel Map

Pump A1

Figure 1

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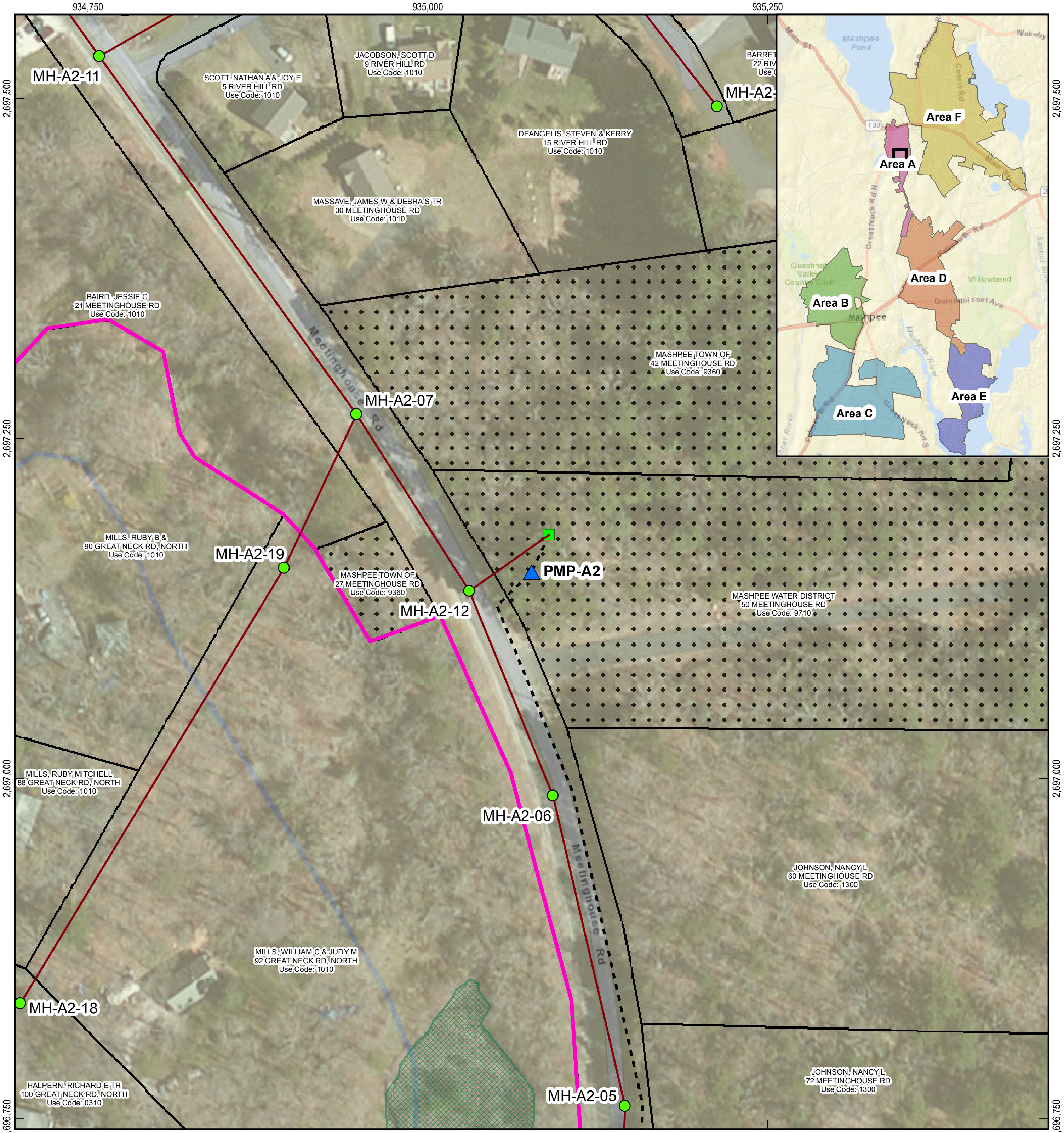

Legend

- Wet Well
- Manhole
- Sewer Sheds
- Pump
- Conduit
- Parcels
- WWTP
- Pressure Pipe
- Municipal Property

NWI Wetland Areas

- Freshwater Forested/Shrub Wetland

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Job Number	11110939
Revision	A
Date	Aug 24, 2017

Pump Station Parcel Map
Pump A2

Figure 2

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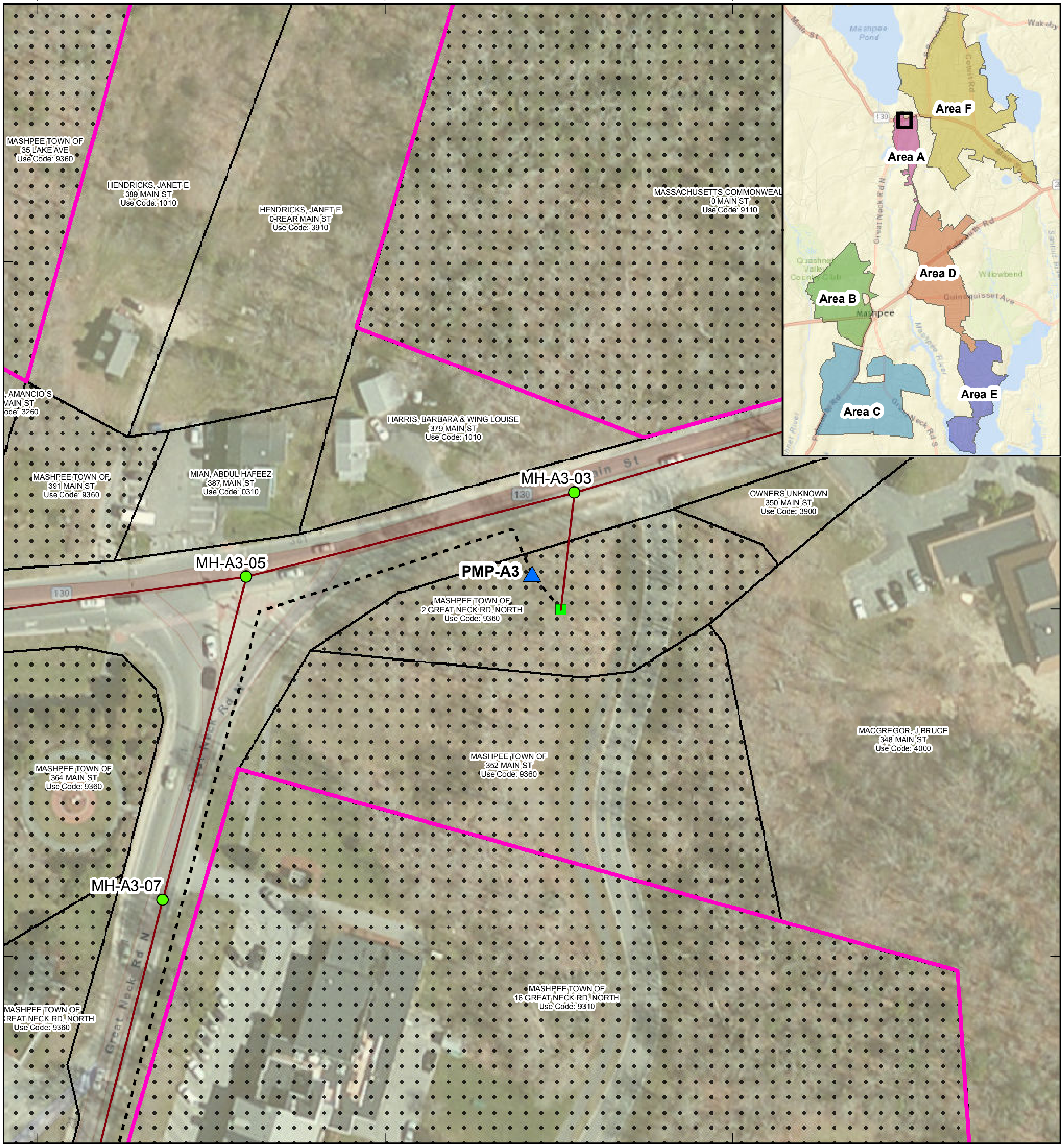

Legend

- Wet Well
- Manhole
- Sewer Sheds
- Pump
- Conduit
- Parcels
- WWTP
- Pressure Pipe
- Municipal Property

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Job Number 11110939
Revision A
Date Aug 24, 2017

Pump Station Parcel Map
Pump A3

Figure 3

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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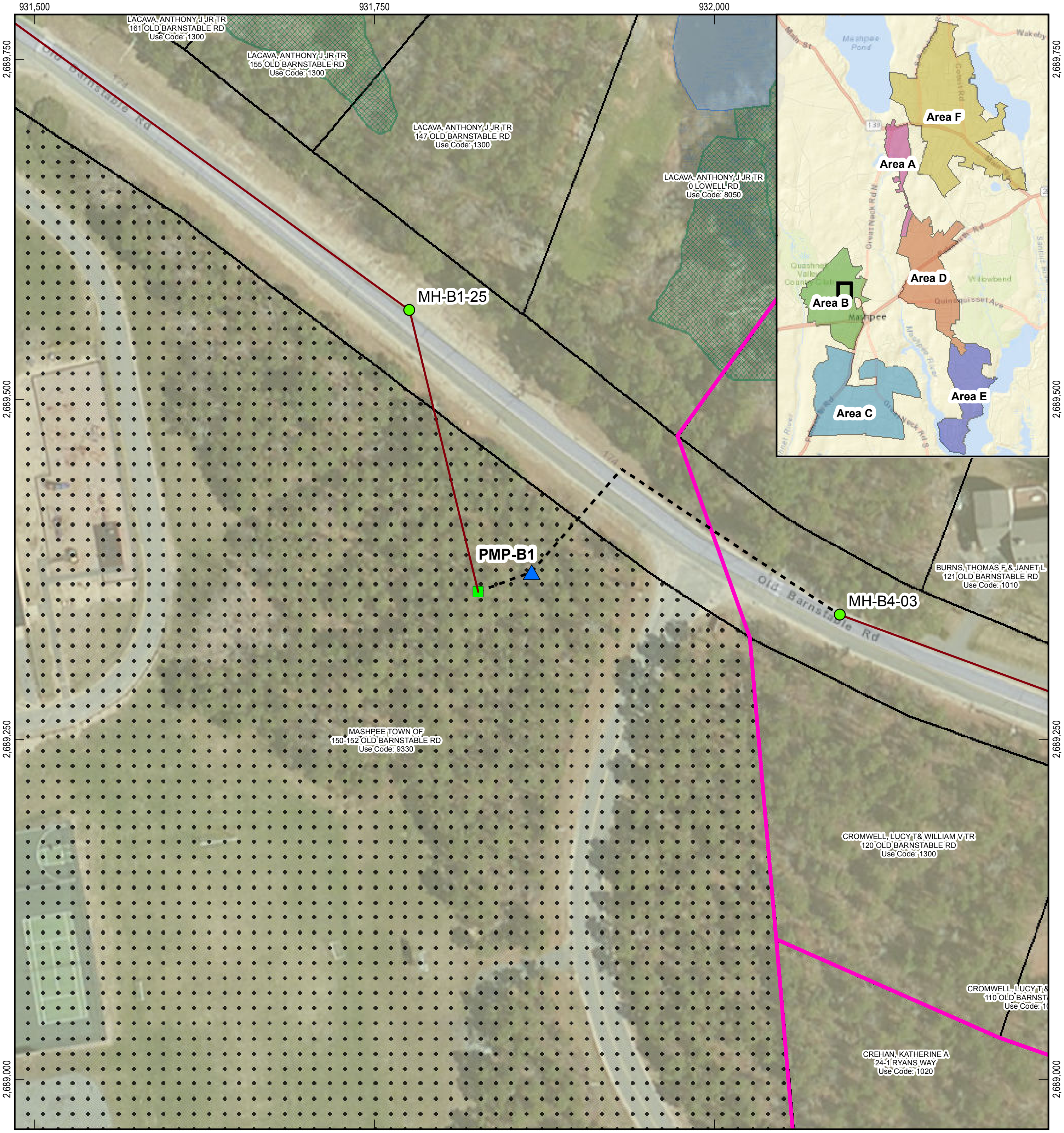

Legend

- Wet Well
- Manhole
- Sewer Sheds
- Pump
- Conduit
- Parcels
- WWTP
- Pressure Pipe
- Municipal Property

NWI Wetland Areas

- Freshwater Forested/Shrub Wetland
- Freshwater Pond

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Job Number 11110939
Revision A
Date Aug 24, 2017

Pump Station Parcel Map
Pump B1

Figure 4

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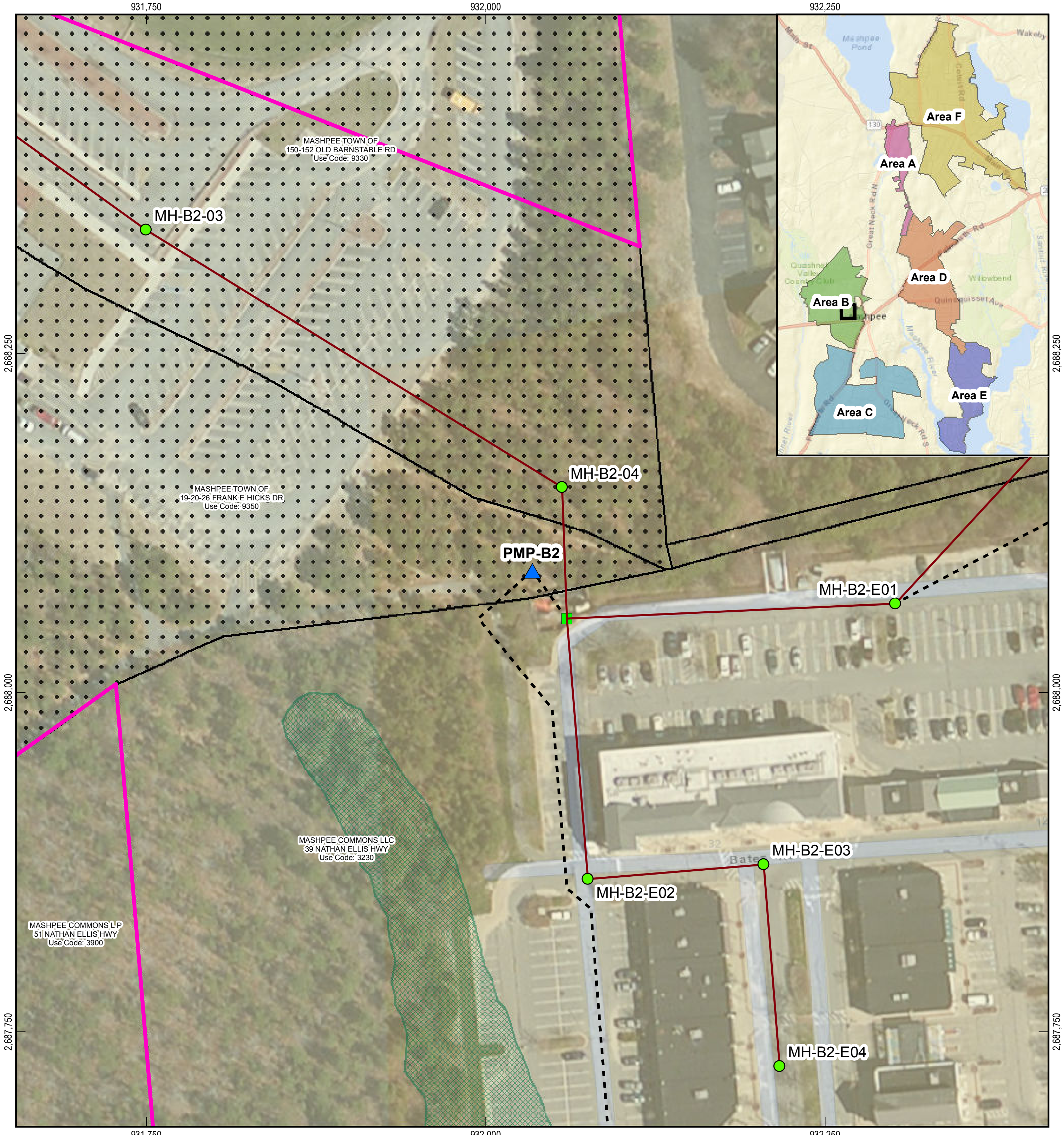

Legend

- Wet Well
- Manhole
- Sewer Sheds
- Pump
- Conduit
- Parcels
- WWTP
- Pressure Pipe
- Municipal Property

NWI Wetland Areas

- Freshwater Forested/Shrub Wetland

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Job Number	11110939
Revision	A
Date	Aug 24, 2017

Pump Station Parcel Map
Pump B2 (Modified)

Figure 5

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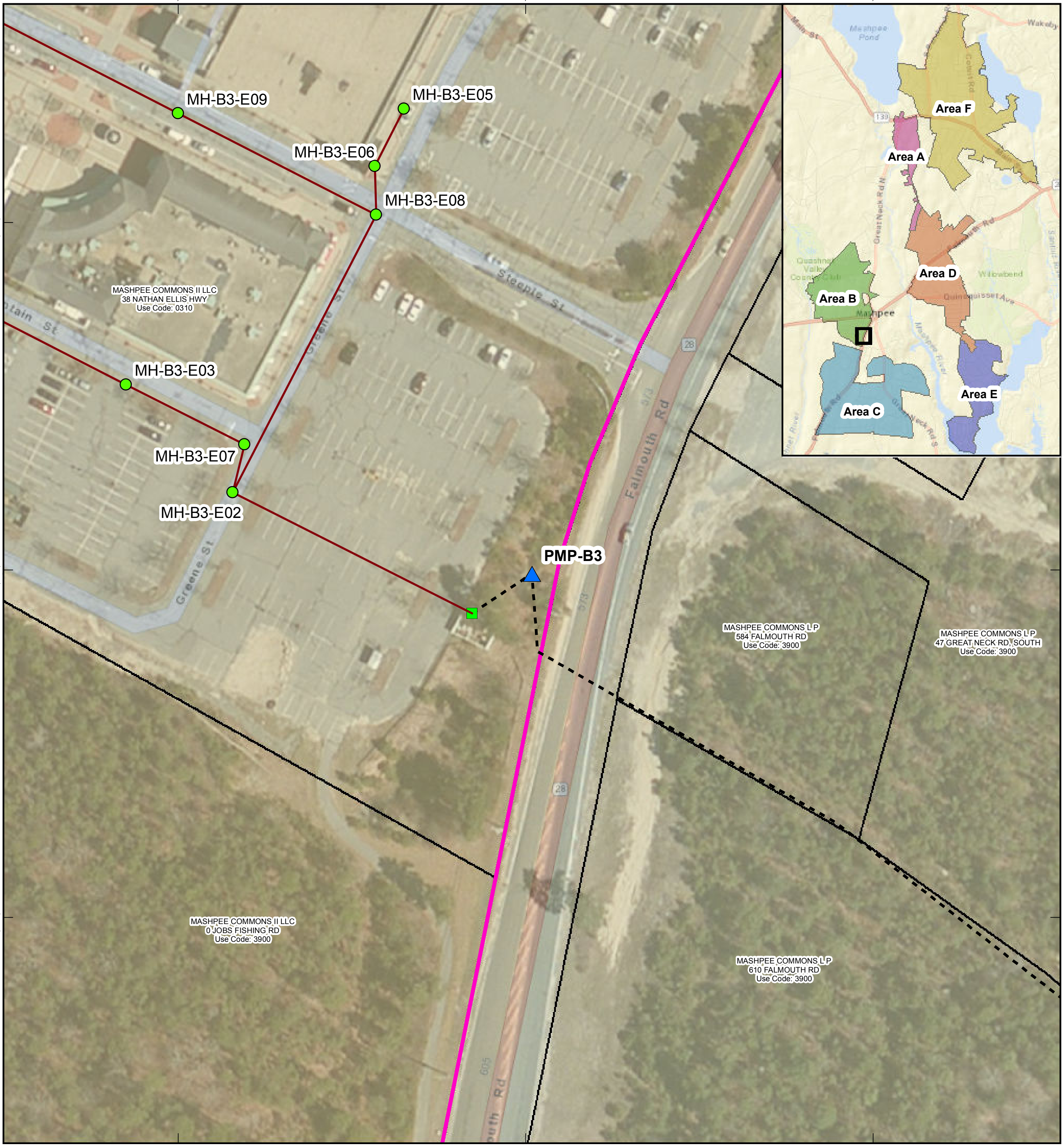

Legend

Wet Well
Manhole
Sewer Sheds
Pump
Conduit
Parcels
WWTP
Pressure Pipe
Municipal Property

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Job Number	11110939
Revision	A
Date	Aug 24, 2017

Pump Station Parcel Map
Pump B3 (Modified)

Figure 6

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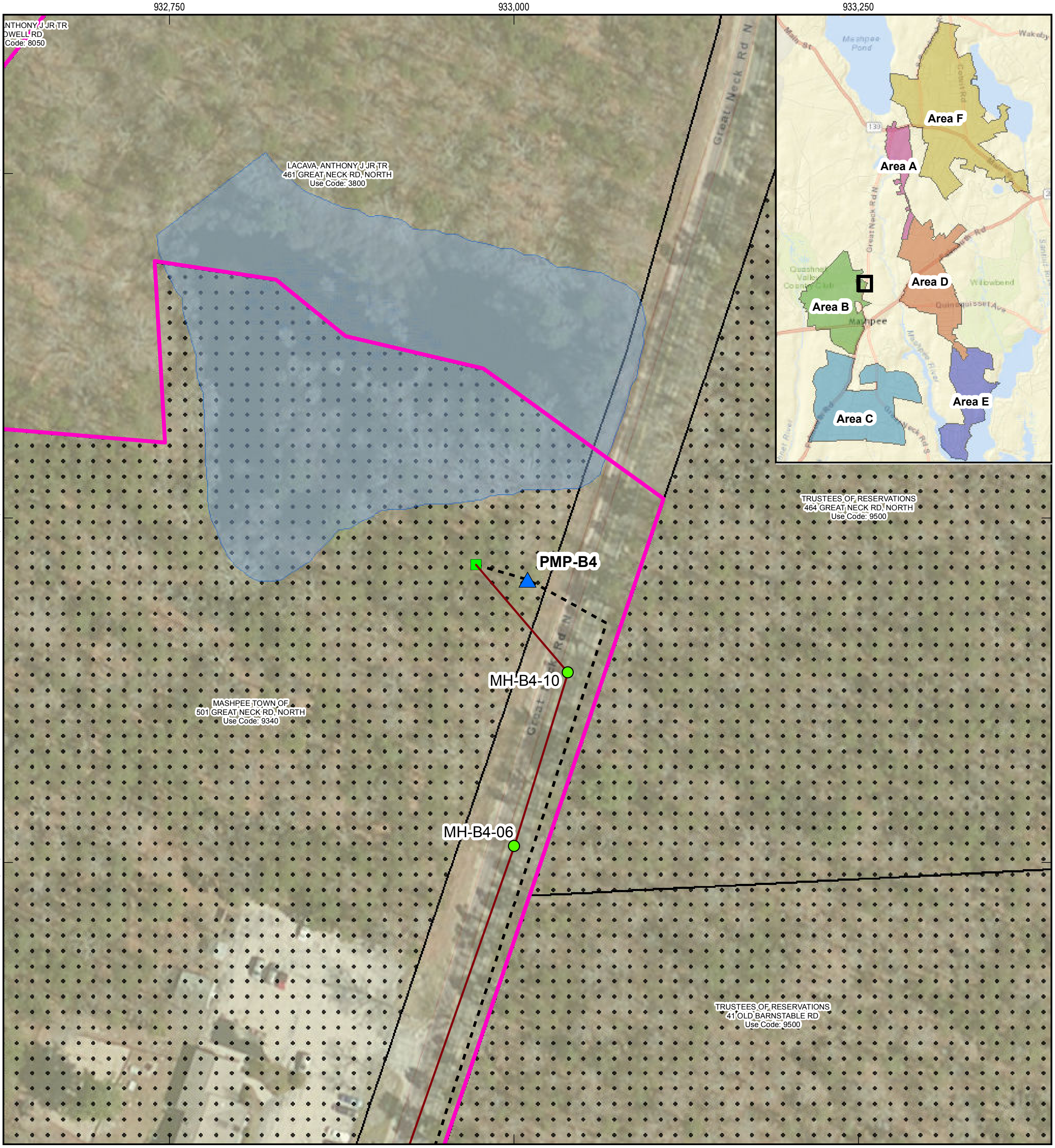
Created by:jsriling



Legend

Wet Well
Manhole
Sewer Sheds
Pump
Conduit
Parcels
WWTP
Pressure Pipe
Municipal Property

NWI Wetland Areas

Freshwater Pond

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Job Number	11110939
Revision	A
Date	Aug 24, 2017

Pump Station Parcel Map
Pump B4

Figure 7

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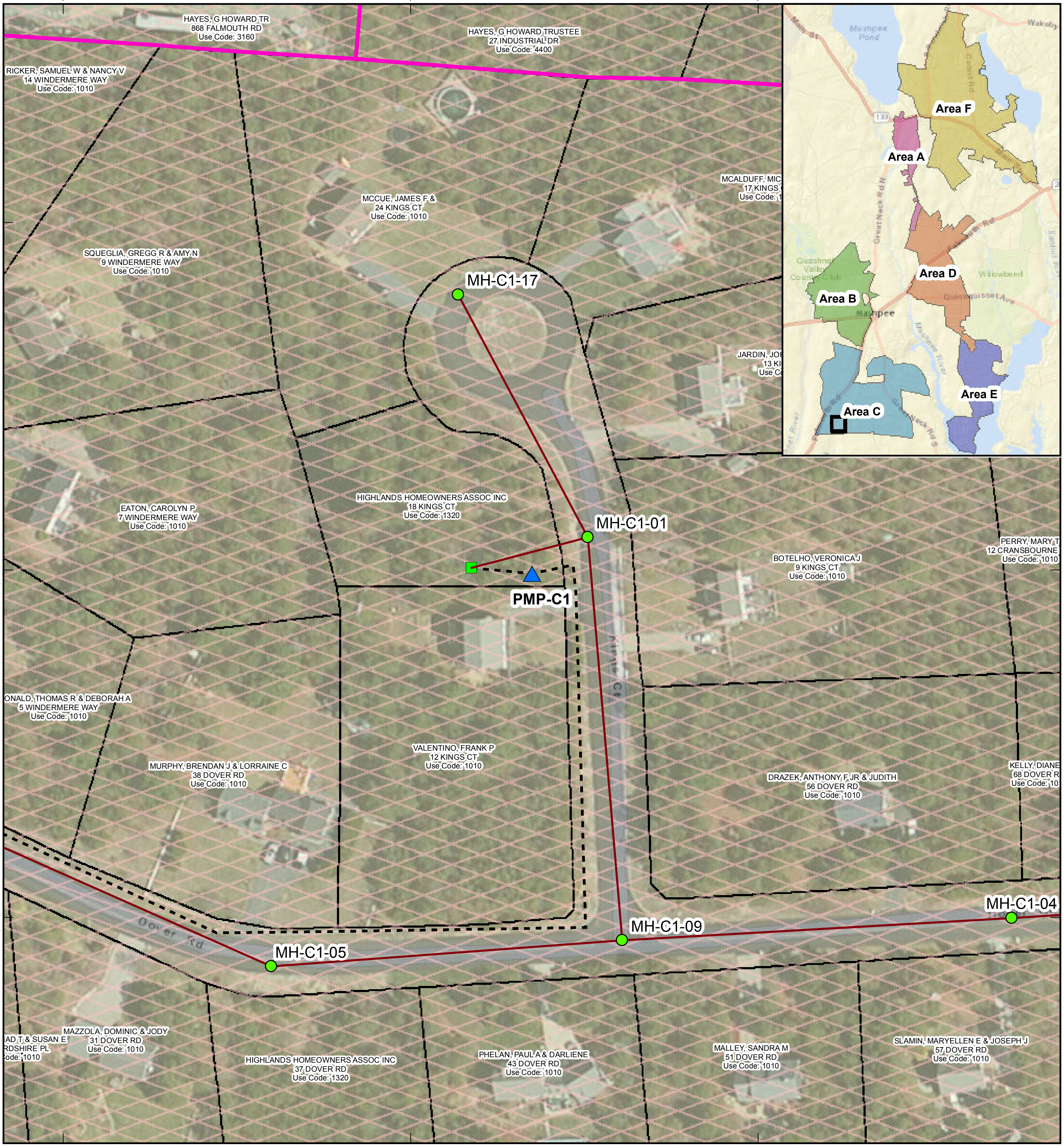

Legend

- Wet Well
- Manhole
- Sewer Sheds
- DEP Approved Zone II
- Pump
- Conduit
- Parcels
- WWTP
- Pressure Pipe
- Municipal Property

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Job Number	11110939
Revision	A
Date	Aug 24, 2017

Pump Station Parcel Map
Pump C1

Figure 8

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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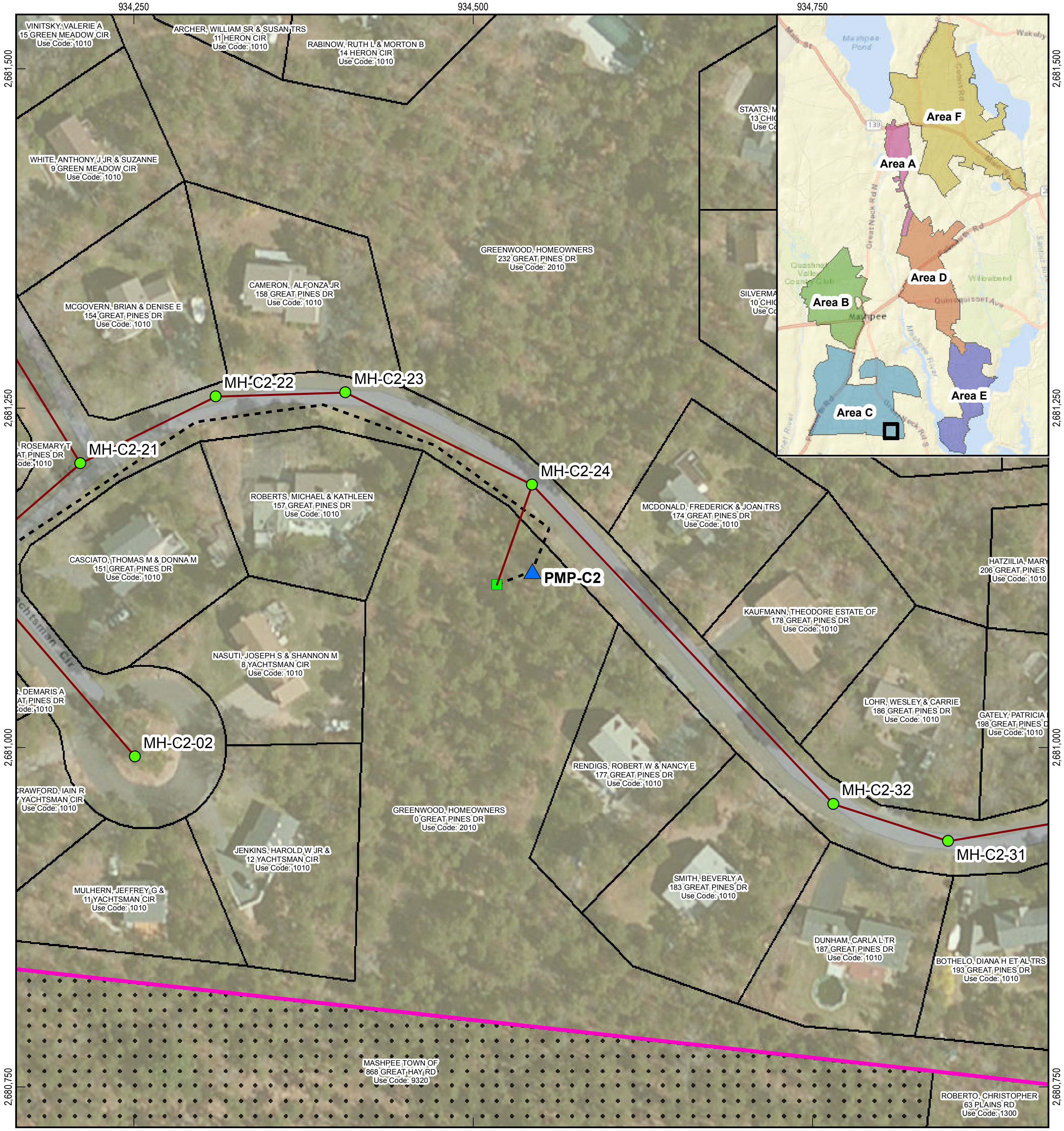

Legend

Wet Well	Manhole	Sewer Sheds
Pump	Conduit	Parcels
WWTP	Pressure Pipe	Municipal Property

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Job Number 11110939
Revision A
Date Aug 24, 2017

Pump Station Parcel Map
Pump C2

Figure 9

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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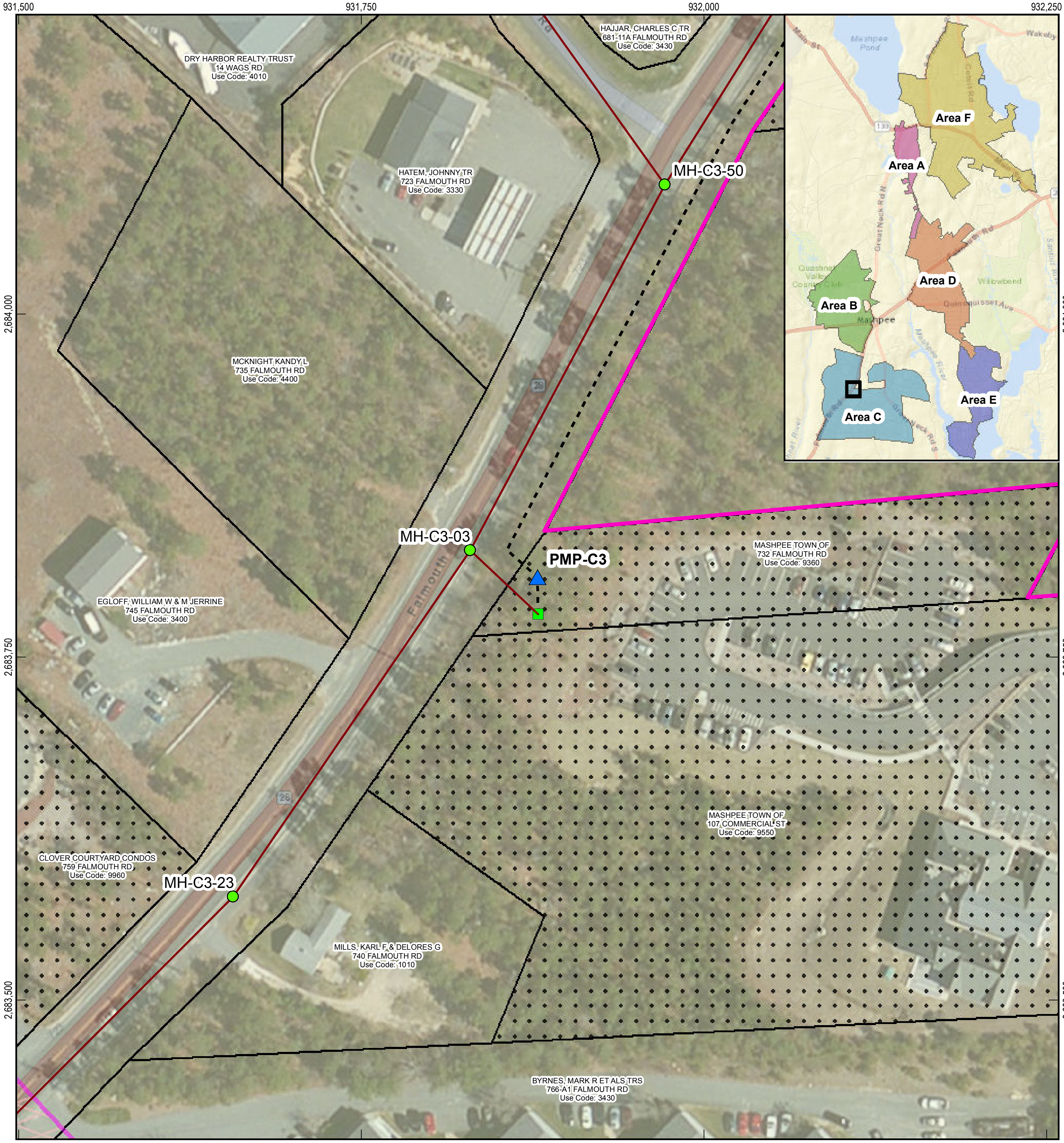

Legend

- Wet Well
- Manhole
- Sewer Sheds
- DEP Approved Zone II
- Pump
- Conduit
- Parcels
- WWTP
- Pressure Pipe
- Municipal Property

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Job Number	11110939
Revision	A
Date	Aug 24, 2017

Pump Station Parcel Map
Pump C3

Figure 10

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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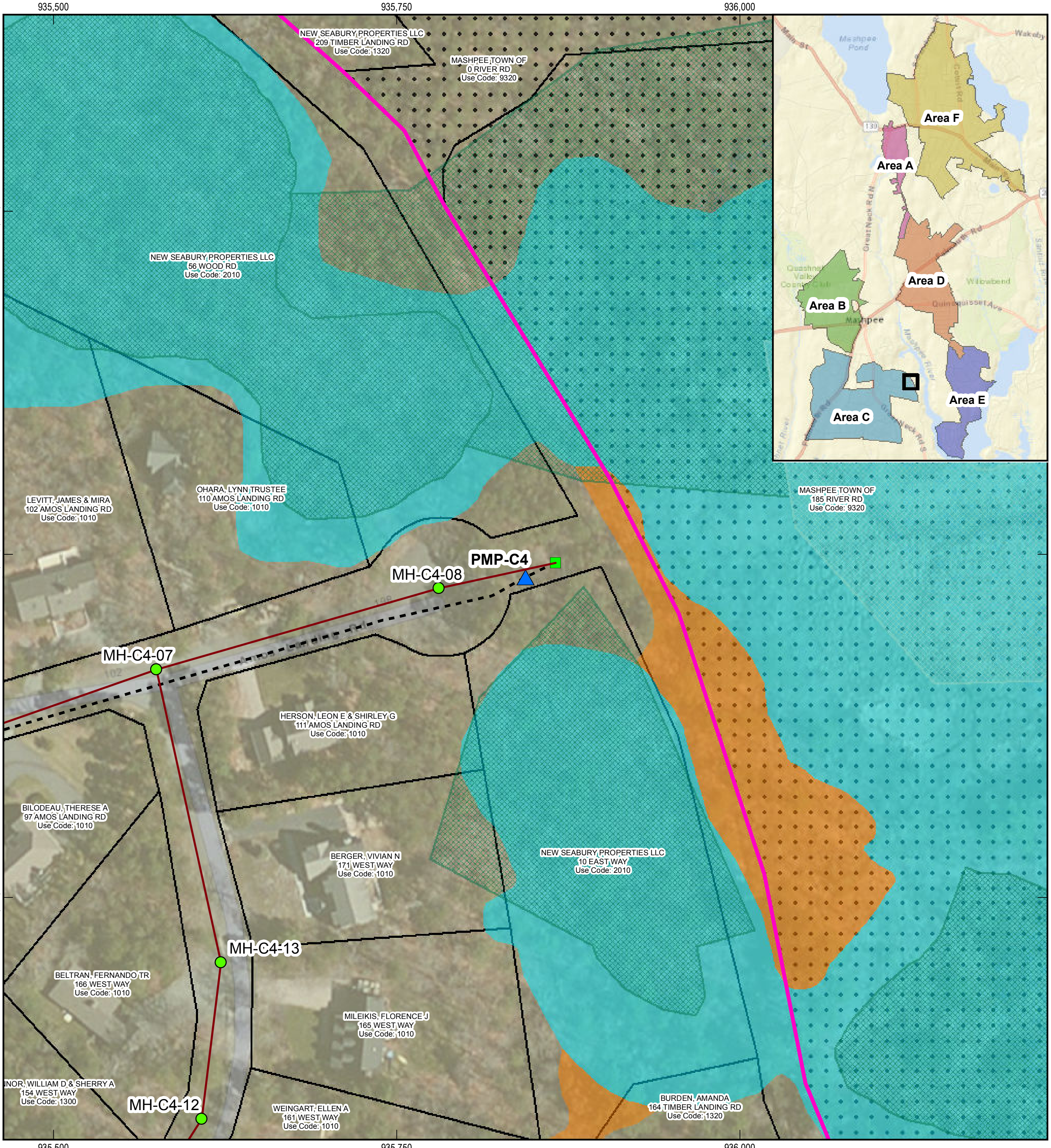
Created by:jsriling



Legend

Wet Well	Manhole	Sewer Sheds
Pump	Conduit	Parcels
WWTP	Pressure Pipe	Municipal Property

NWI Wetland Areas

- Estuarine and Marine Wetland
- Freshwater Forested/Shrub Wetland

FEMA National Flood Hazard Layer

- AE: 1% Annual Chance of Flooding, with BFE
- X: 0.2% Annual Chance of Flooding

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Job Number	11110939
Revision	A
Date	Aug 24, 2017

Pump Station Parcel Map
Pump C4

Figure 11

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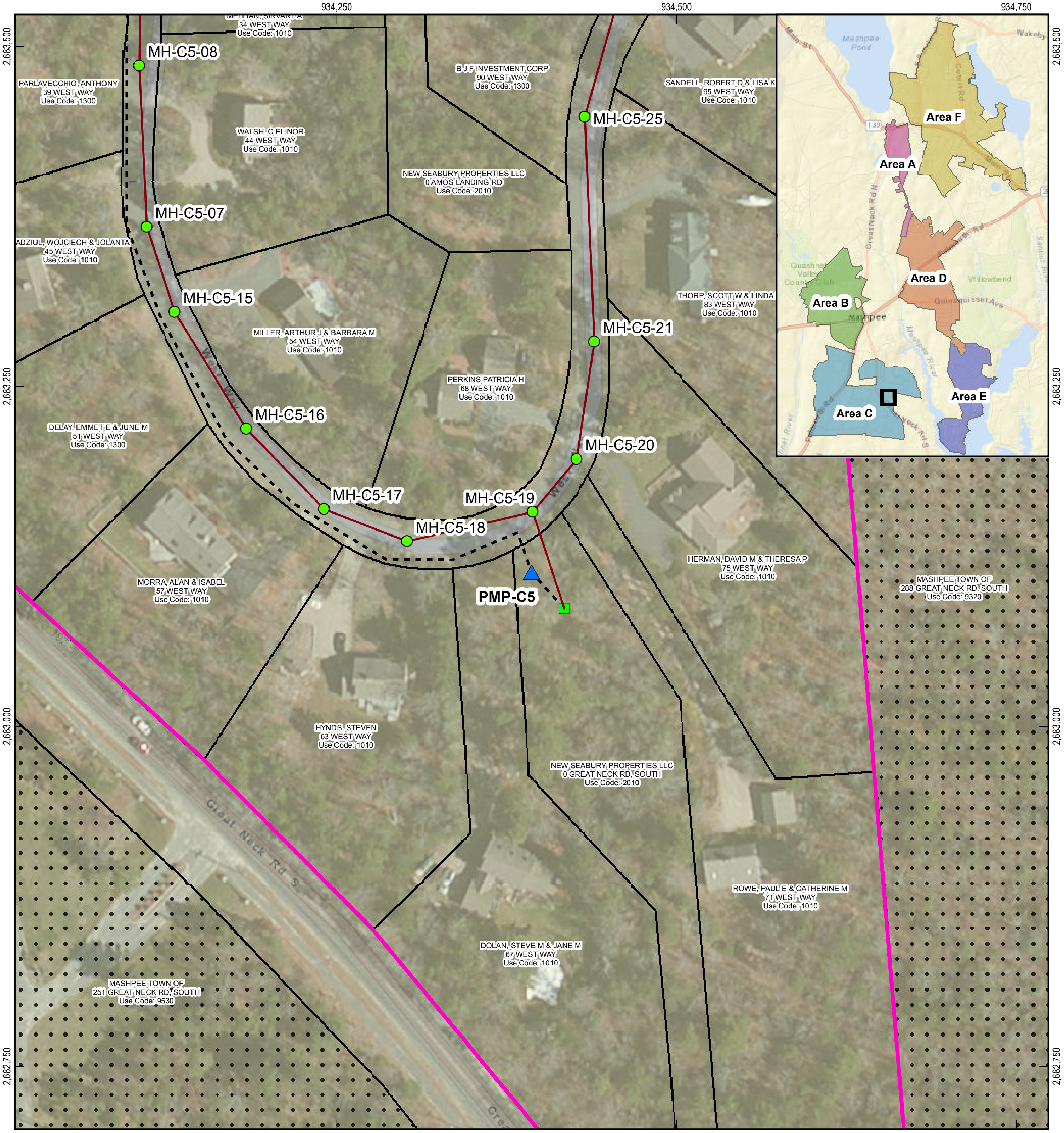
Created by:jsriling



Legend

- Wet Well
- Manhole
- Sewer Sheds
- Pump
- Conduit
- Parcels
- WWTP
- Pressure Pipe
- Municipal Property

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Job Number 11110939
Revision A
Date Aug 24, 2017

Pump Station Parcel Map
Pump C5

Figure 12

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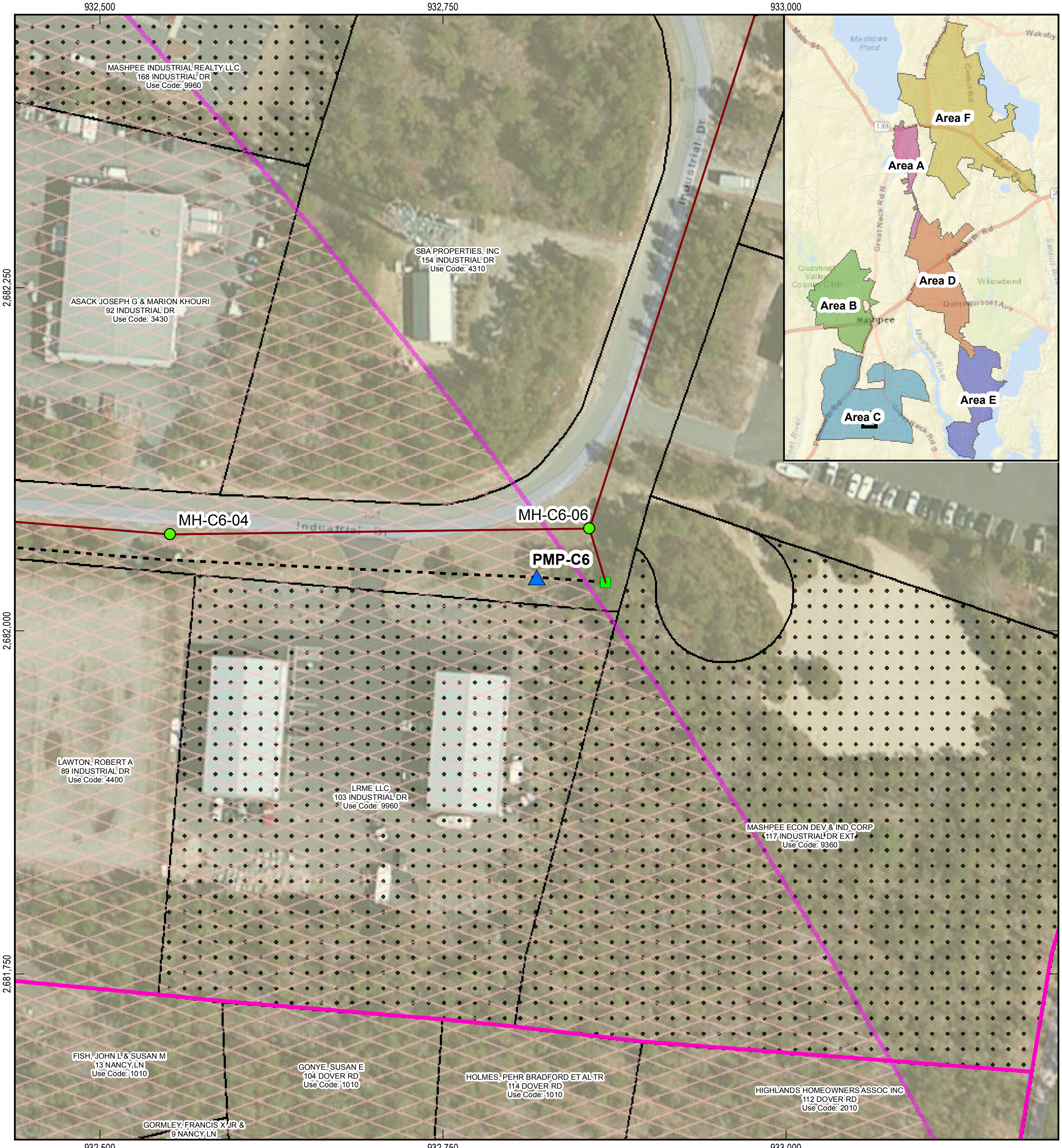
Created by:jsriling



Legend

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Job Number 11110939
Revision A
Date Aug 24, 2017

Pump Station Parcel Map
Pump C6

Figure 13

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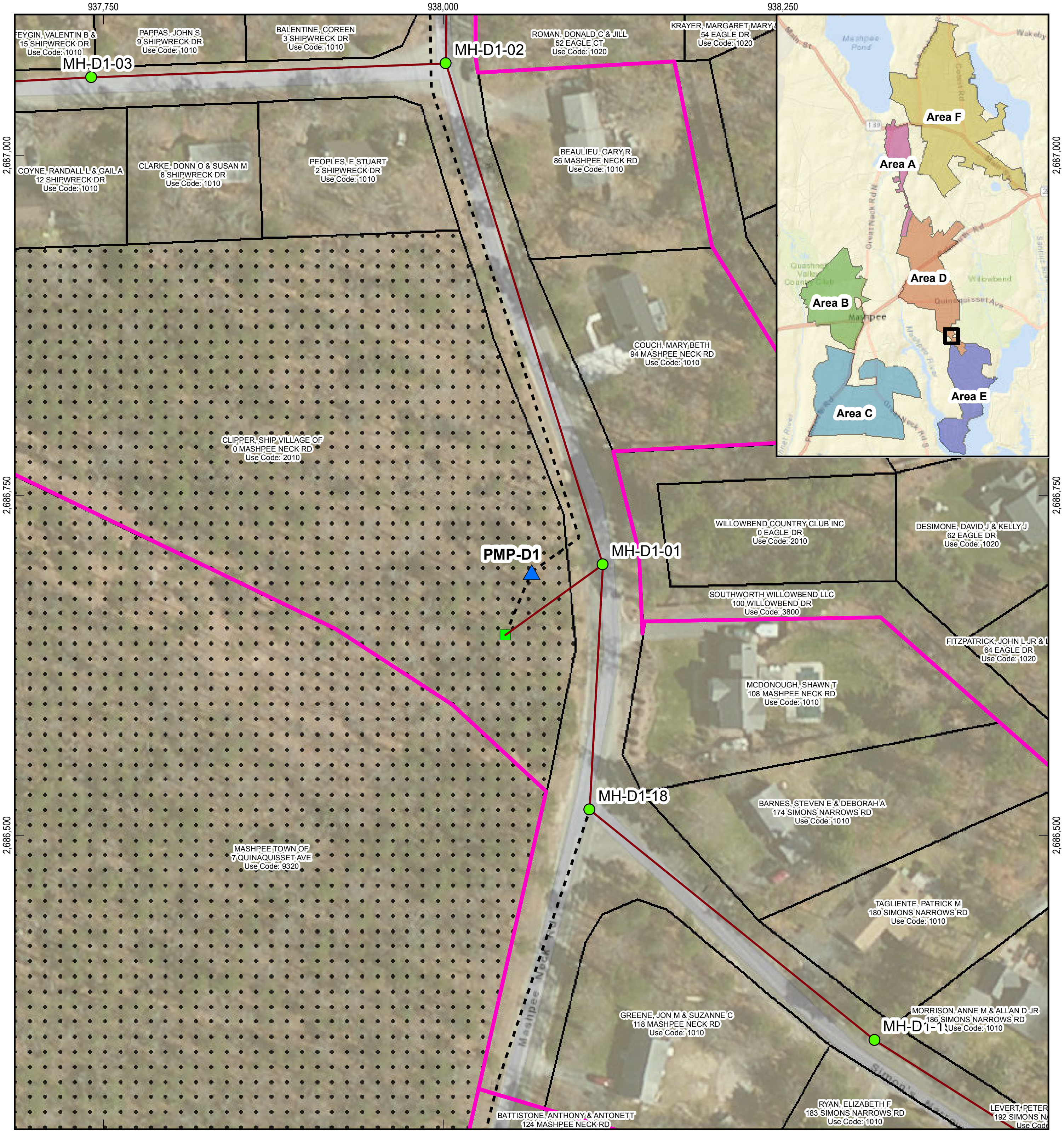
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Created by:jsriling



Legend

Wet Well	Manhole	Sewer Sheds
Pump	Conduit	Parcels
WWTP	Pressure Pipe	Municipal Property

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

N

GHD

Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Job Number 11110939
Revision A
Date Aug 24, 2017

Pump Station Parcel Map
Pump D1

Figure 14

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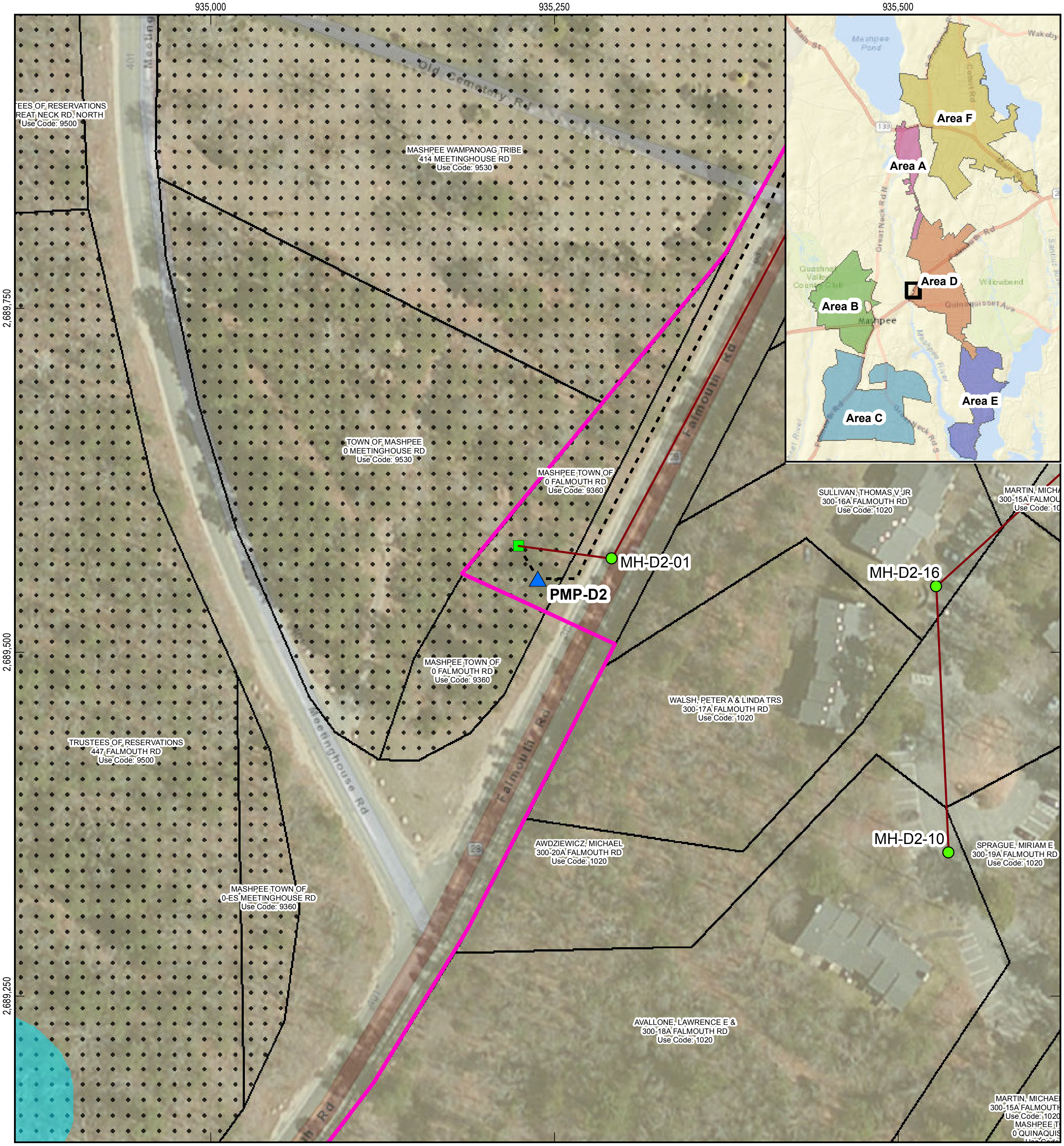
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
Created by:jsriling



Legend

- Wet Well
- Manhole
- Sewer Sheds
- Pump
- Conduit
- Parcels
- WWTP
- Pressure Pipe
- Municipal Property

FEMA National Flood Hazard Layer

- AE: 1% Annual Chance of Flooding, with BFE

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

Job Number	11110939
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Pump Station Parcel Map
Pump D2

Figure 15

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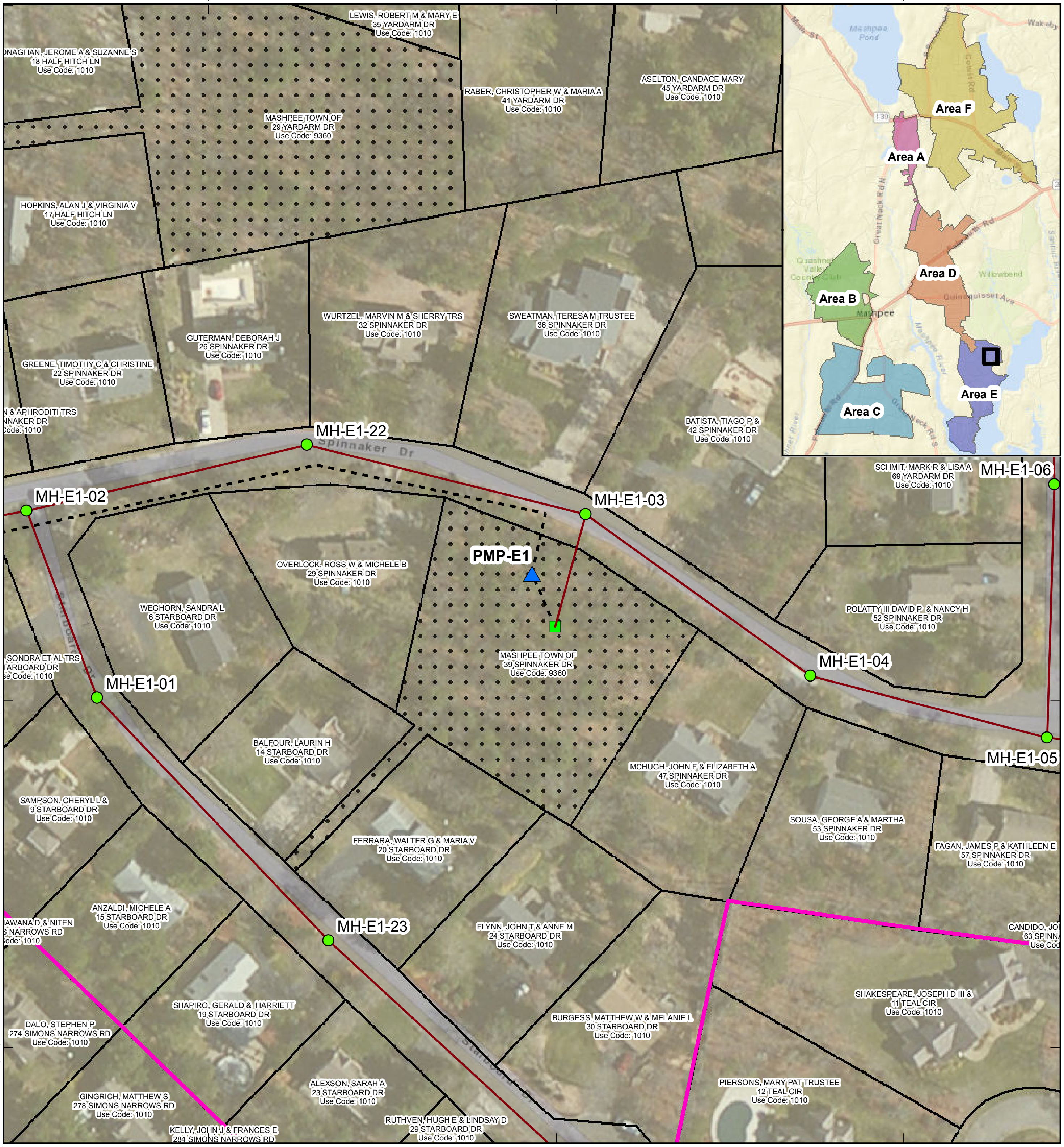

Legend

- Wet Well
- Manhole
- Sewer Sheds
- Pump
- Conduit
- Parcels
- WWTP
- Pressure Pipe
- Municipal Property

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

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Pump Station Parcel Map
Pump E1

Figure 16

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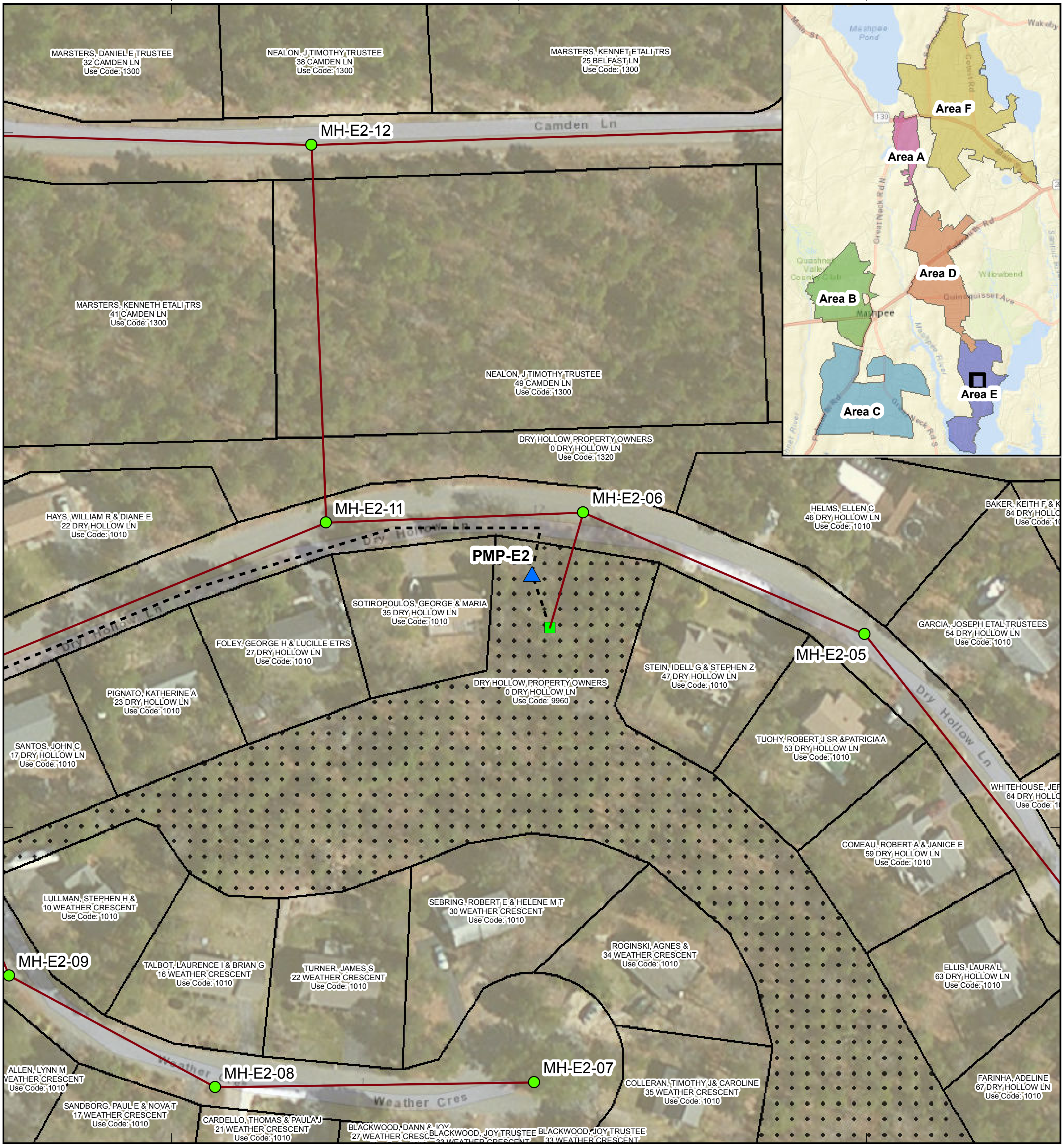
Created by:jsriling



Legend

Wet Well	Manhole	Sewer Sheds
Pump	Conduit	Parcels
WWTP	Pressure Pipe	Municipal Property

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

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Town of Mashpee, MA
Existing WWTF & Collection System Evaluation

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Pump Station Parcel Map
Pump E2

Figure 17

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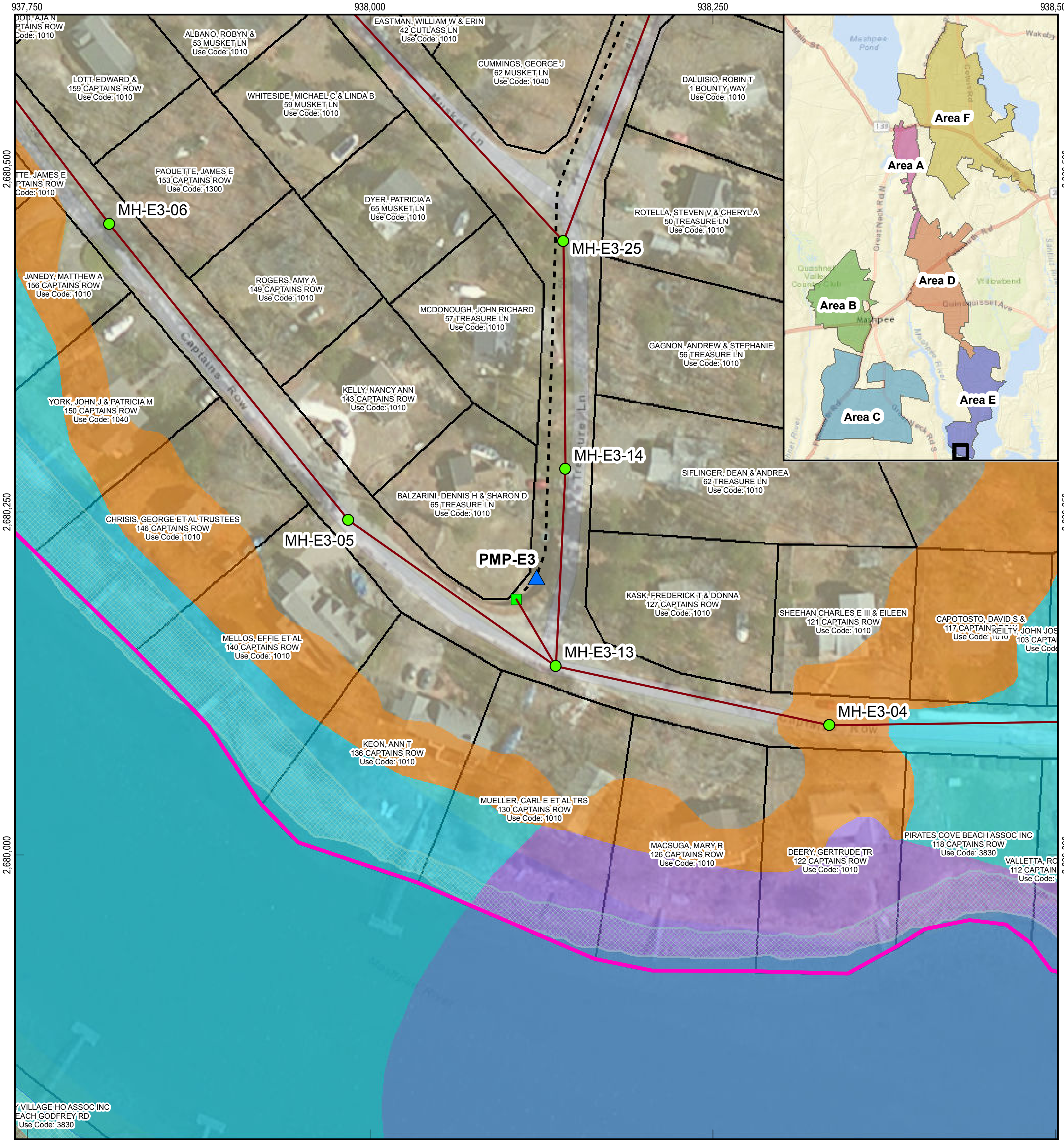

Legend

- Wet Well
- Pump
- WWTP
- Manhole
- Conduit
- Pressure Pipe
- Sewer Sheds
- Parcels
- Municipal Property

NWI Wetland Areas
- Estuarine and Marine Wetland
- Estuarine and Marine Deepwater

FEMA National Flood Hazard Layer
- AE: 1% Annual Chance of Flooding, with BFE
- VE: High Risk Coastal Area
- X: 0.2% Annual Chance of Flooding

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

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Pump Station Parcel Map
Pump E3

Figure 18

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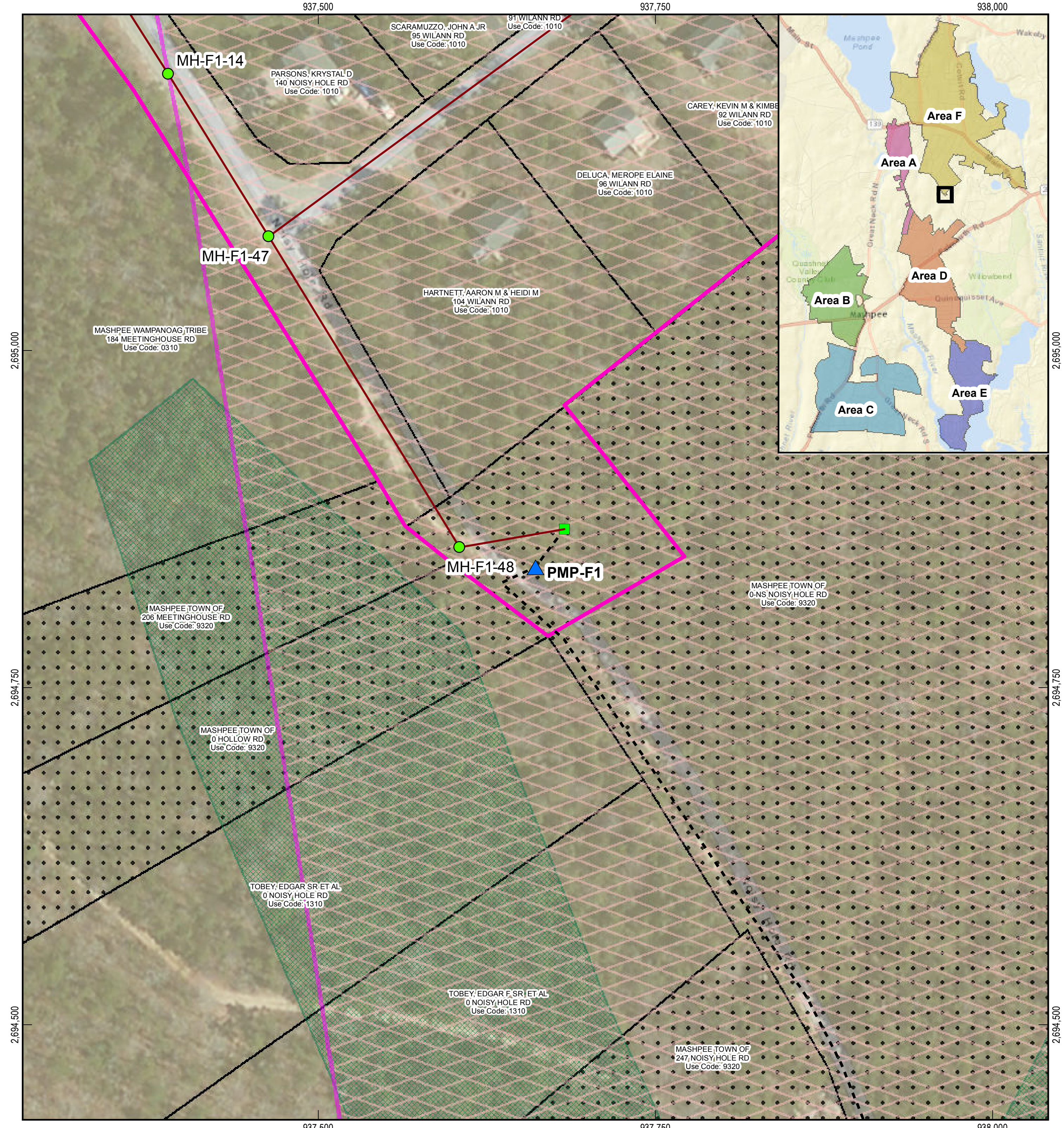

Legend

- Wet Well
- Manhole
- Sewer Sheds
- DEP Approved Zone II
- Pump
- Conduit
- Parcels
- WWTP
- Pressure Pipe
- Municipal Property

NWI Wetland Areas

- Freshwater Forested/Shrub Wetland

Paper Size ANSI A
1 inch = 100 feet

0 15 30 60 90 120
Feet

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

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Pump Station Parcel Map
Pump F1

Figure 19

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Legend

- Wet Well
- Manhole
- Sewer Sheds
- Pump
- Conduit
- Parcels
- WWTP
- Pressure Pipe
- Municipal Property

NWI Wetland Areas

- Freshwater Forested/Shrub Wetland
- Freshwater Pond

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

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Pump Station Parcel Map
Pump F2

Figure 20

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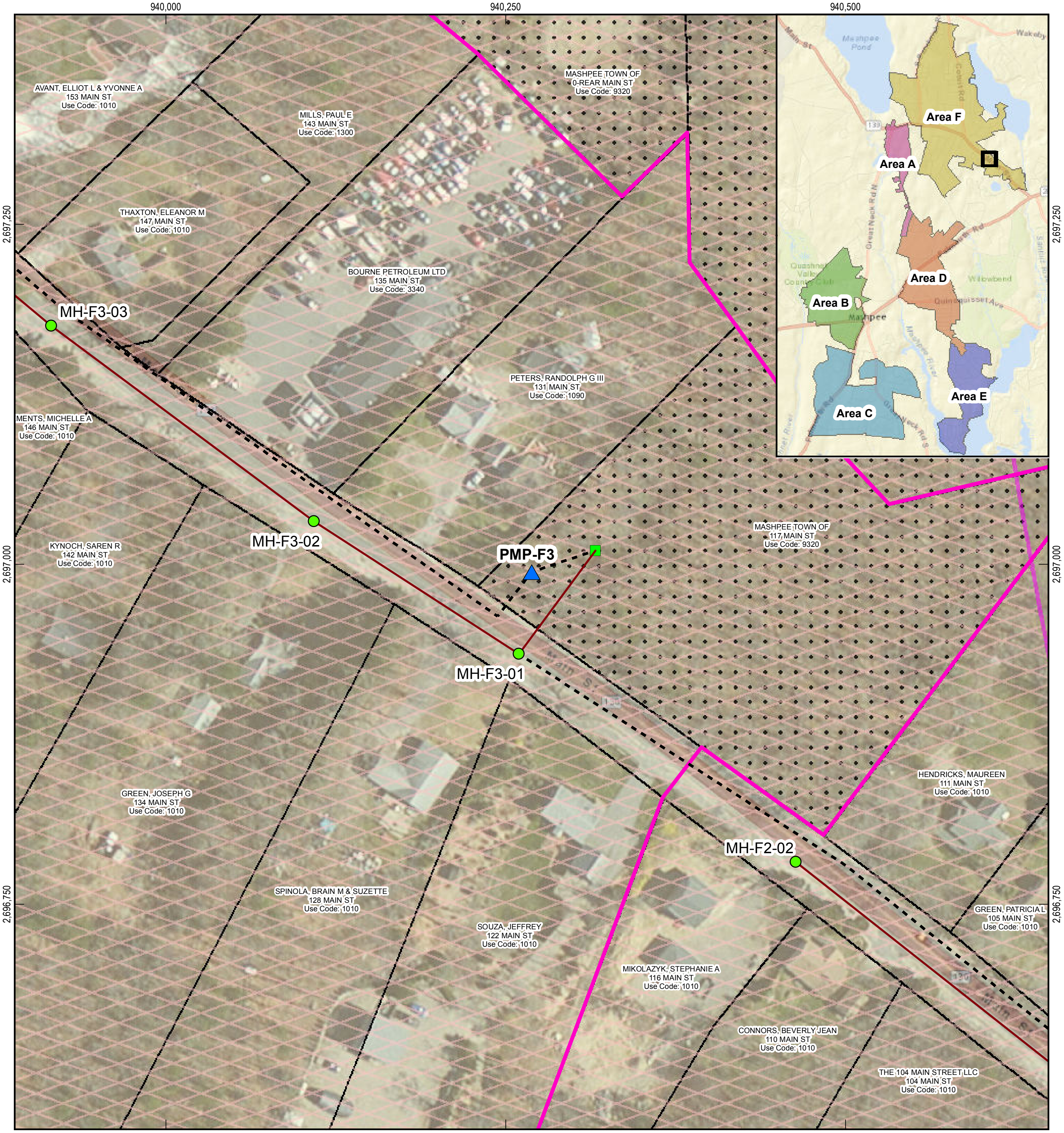
Created by:jsriling



Legend

Wet Well
Manhole
Sewer Sheds
DEP Approved Zone II
NWI Wetland Areas
Pump
Conduit
Parcels
Freshwater Forested/Shrub Wetland
WWTP
Pressure Pipe
Municipal Property

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

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Pump Station Parcel Map
Pump F3

Figure 21

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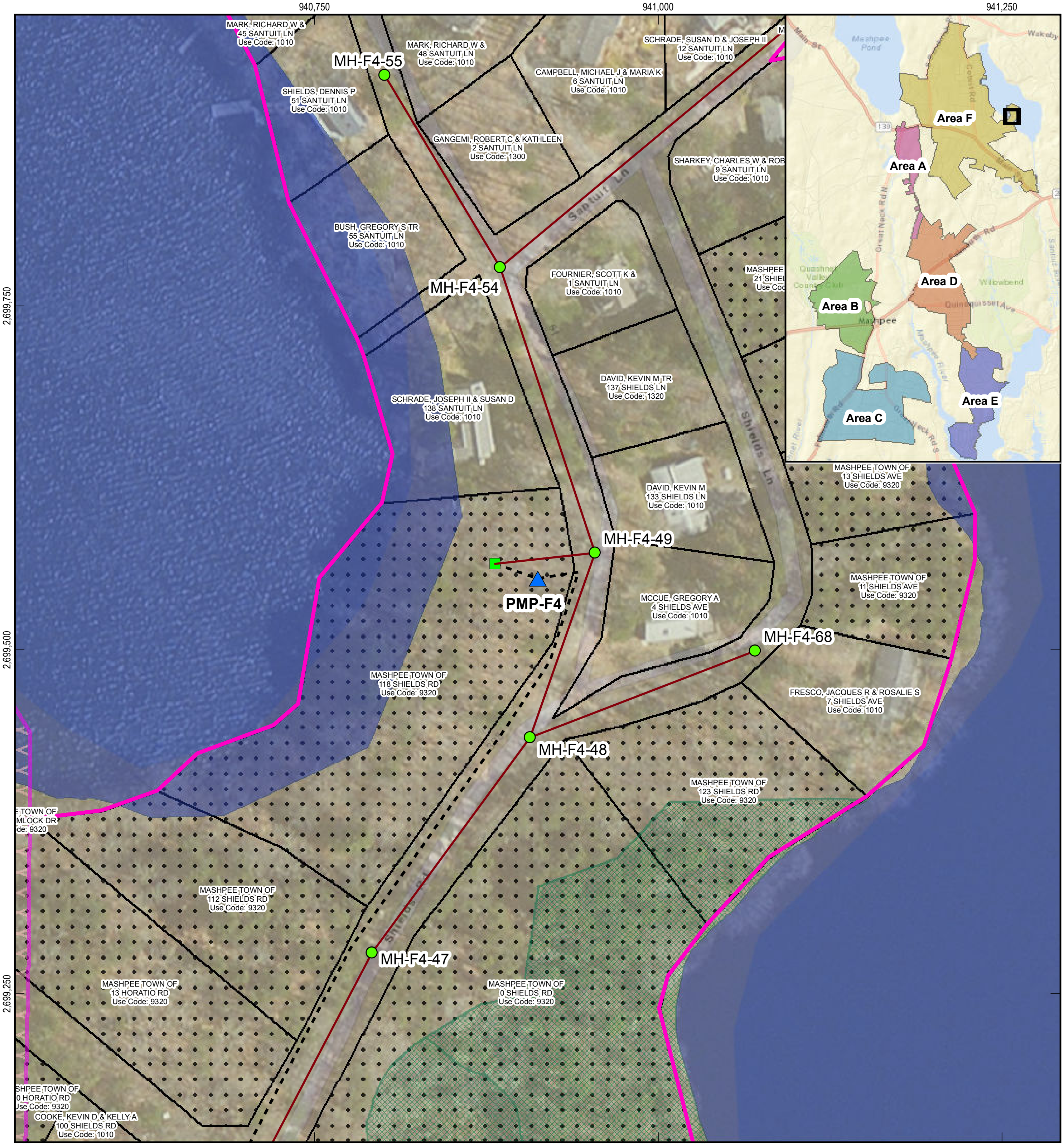
Created by:jsriling



Legend

Wet Well	Manhole	Sewer Sheds	DEP Approved Zone II	**NWI Wetland Areas**		
Pump	Conduit	Parcels		Freshwater Forested/Shrub Wetland		
WWTP	Pressure Pipe	Municipal Property		Lake		

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

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Pump Station Parcel Map
Pump F4

Figure 22

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Legend

- Wet Well
- Manhole
- Sewer Sheds
- DEP Approved Zone II
- Pump
- Conduit
- Parcels
- WWTP
- Pressure Pipe
- Municipal Property

NWI Wetland Areas

- Freshwater Forested/Shrub Wetland
- Lake

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

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Pump Station Parcel Map
Pump F5

Figure 23

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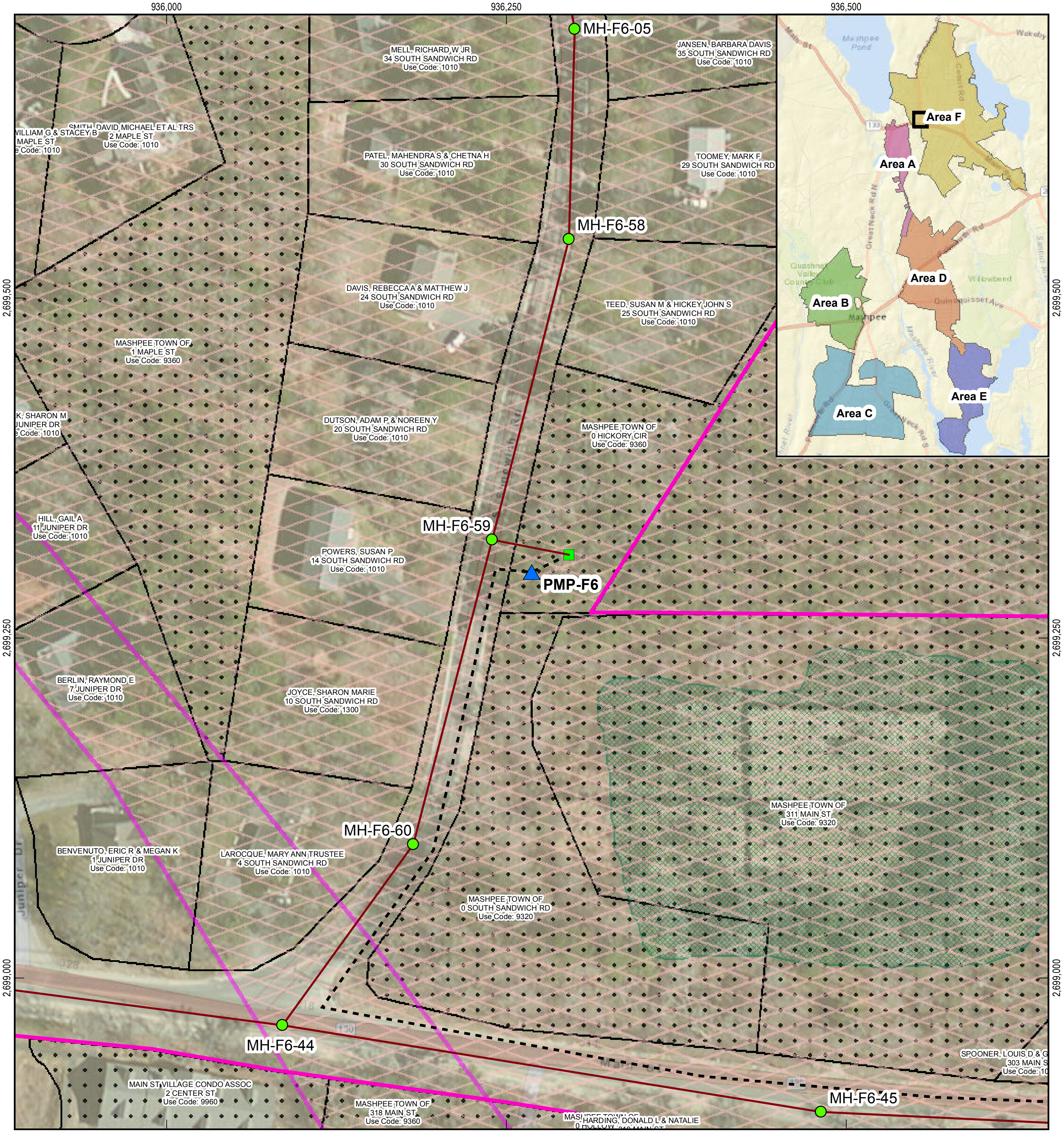

Legend

Wet Well	Manhole	Sewer Sheds	DEP Approved Zone II
Pump	Conduit	Parcels	**NWI Wetland Areas**
WWTP	Pressure Pipe	Municipal Property	Freshwater Forested/Shrub Wetland

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

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Pump Station Parcel Map
Pump F6

Figure 24

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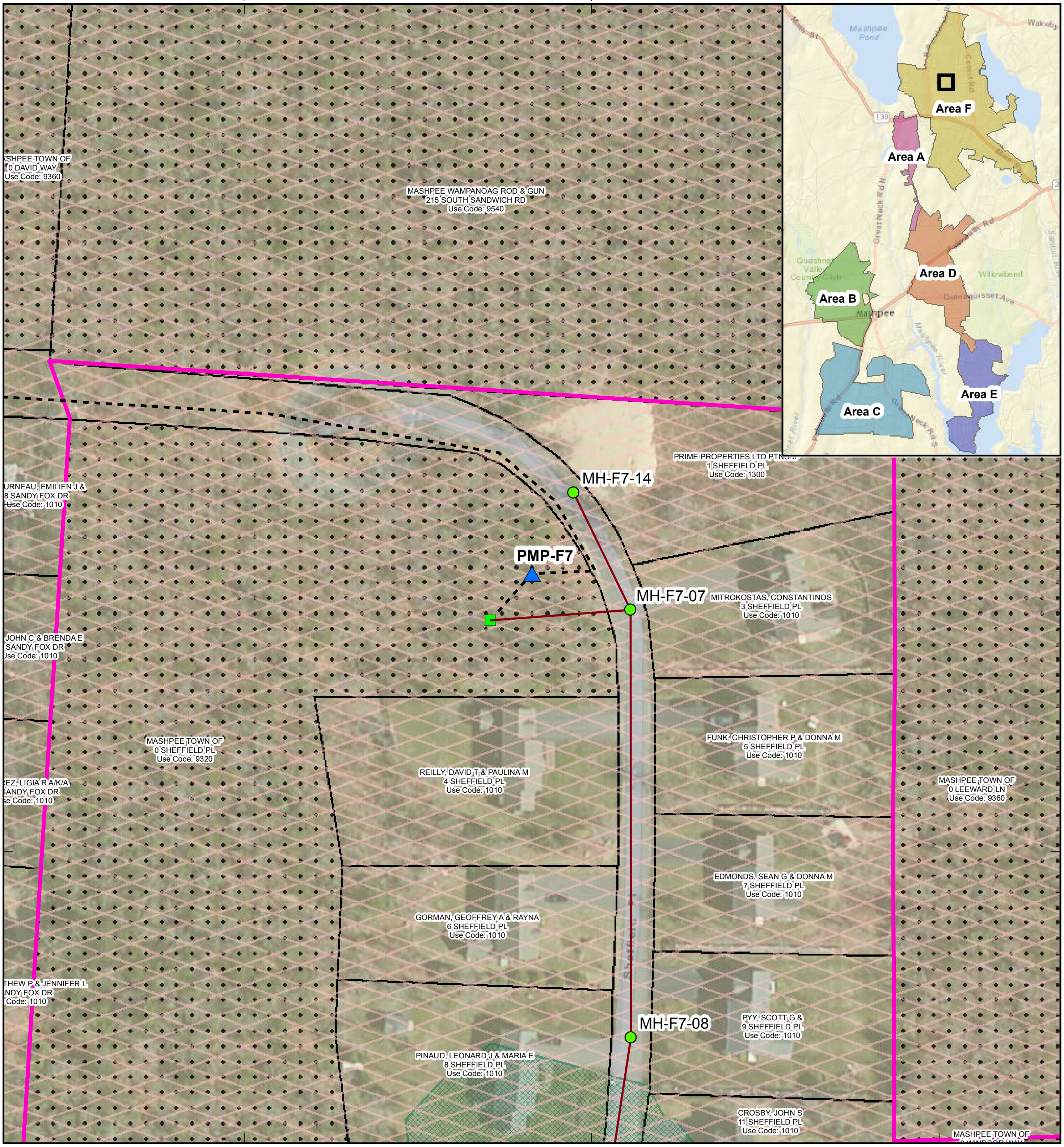

Legend

Wet Well
Manhole
Sewer Sheds
DEP Approved Zone II
NWI Wetland Areas
Pump
Conduit
Parcels
Freshwater Forested/Shrub Wetland
WWTP
Pressure Pipe
Municipal Property

Paper Size ANSI A
1 inch = 100 feet
0 15 30 60 90 120
Feet
Map Projection: Lambert Conformal Conic
Horizontal Datum: North American 1983
Grid: NAD 1983 StatePlane Massachusetts Mainland FIPS 2001 Feet

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Pump Station Parcel Map
Pump F7

Figure 25

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Document Status

Revision	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Draft A	Anastasia Rudenko	J. Jefferson Gregg	JJG			
Final	Anastasia Rudenko	J. Jefferson Gregg	JJG	J. Jefferson Gregg	JJG	9/6/17

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