Brown AND Caldwell

Prepared for
Guam Waterworks Authority

GUAM WATERWORKS AUTHORITY

Water Resources Master Plan Update

Volume 3: Wastewater System

Final | August 2018

Prepared by | Brown and Caldwell | Hagåtña Office | T 671.300.4220

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Water Resources Master Plan Update

Volume 3

Wastewater System

Final | August 2018

Prepared for

Guam Waterworks Authority, Mangilao, Guam

August 2018

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List of Abbreviations

ACH	aluminum chlorohydrate
ACP	asbestos cement pipe
AFB	Air Force Base
ARV	air release valve
AVV	air vacuum valve
AWWA	American Water Works Association
BC	Brown and Caldwell
BMP	best management practice
BOD_5	5-day biochemical oxygen demand
BWF	base wastewater flow
CAV	combination air valve
CCTV	closed-circuit television
CEPT	chemically enhanced primary treatment
CFR	Code of Federal Regulations
CIP	Capital Improvement Program
CIPP	cured-in-place pipe
CWA	Clean Water Act
d/D	depth to diameter
DMR	discharge monitoring report
DoD	Department of Defense
FOG	fats, oils, and grease
GAR	Guam Administrative Rules and Regulations
gpcd	gallons per capita per day
gpd	gallons per day
gpm	gallons per minute
EPA	Environmental Protection Agency
GIS	geographic information system
GPA	Guam Power Authority
GPS	global positioning system
GWA	Guam Waterworks Authority
GWI	groundwater infiltration
GWQS	Guam Water Quality Standards
H_2S	hydrogen sulfide
hp	horsepower
I/I	infiltration and inflow
lbs	pounds
LIDAR	Light Detection and Ranging
MACP	Manhole Assessment and Certification Program
mg/L	milligrams per liter
mgd	million gallons per day
N	nitrogen
NASSCO	National Association of Sanitary Sewer Companies
NEIC	National Enforcement Investigations Center
NGLA	Northern Guam Lens Aquifer
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Services
O&M	operation and maintenance
P	phosphorous
PACL	poly-aluminum chloride
PACP	Pipeline Assessment and Certification Program
PVC	polyvinyl chloride
RAS	return activated sludge
RDII	rainfall-derived infiltration and inflow
SSES	Sewer System Evaluation Study
SSO	sanitary sewer overflow
TRC	total residual chlorine
TSS	total suspended solids
USDA	United States Department of Agriculture
USEPA	United States Environmental Protection Agency
UV	ultraviolet
WAS	waste activated sludge
WRMP	Water Resources Master Plan
WRMPU	2016 Water Resources Master Plan Update
WSE	Wastewater System Evaluation
WWTP	wastewater treatment plant

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Section 1

Introduction

This volume is the third of three volumes for the Guam Waterworks Authority (GWA) Water Resources Master Plan Update (WRMPU). This volume describes the island's wastewater system facilities, an analysis of the wastewater system, and outlines recommendations for improvements to the wastewater system. This volume includes the following sections:

- **Section 2, Existing Wastewater System:** describes existing wastewater system facilities.
- **Section 3, Hydraulic Model Development:** describes the computer model of the wastewater collection system.
- **Section 4, Gravity Piping Evaluation:** describes an evaluation of the capacity and condition of the collection system's gravity piping.
- **Section 5, Force Main Evaluation:** describes an evaluation of the capacity and condition of the collection system's force main piping.
- **Section 6, Lift Station Evaluation:** describes an evaluation of the capacity and condition of the lift stations in the collection system.
- **Section 7, Wastewater Treatment Evaluation:** describes an evaluation of the capacity and condition of the wastewater treatment plants (WWTPs).
- **Section 8, Solids Management Plan:** describes a long-term solids management plan that addresses wastewater solids processing and disposal needs.
- **Section 9, Sewer System Evaluation Study Evaluation:** summarizes Sewer System Evaluation Study (SSES) work that has been completed for the collection system.
- **Section 10, General System Recommendations:** describes general recommendations for the wastewater system.
- **Section 11, Recommended Project Sheets:** contains detailed sheets for each recommended improvement project.

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Section 2

Existing Wastewater System

This section describes GWA's existing wastewater collection and treatment facilities.

2.1 Facilities

GWA provides wastewater service to much of the island's civilian population. As of January 2016, GWA had approximately 42,100 water customers, and of those customers, GWA provided wastewater service to approximately 26,000 customers. GWA also provides service to Andersen Air Force Base (AFB) and other military installations on the North end of the island. GWA's wastewater system facilities include the following:

- **Piping:** GWA's wastewater collection system consists of nearly 320 miles of piping, ranging in diameter from 3 to 48 inches.
- **Lift stations:** the wastewater collection system includes approximately 82 lift stations operated by GWA. Additional private lift stations also connect to GWA's system.
- **Treatment facilities:** GWA owns and operates seven WWTPs.
- **Sewer basins:** GWA's seven sewer basins flow to six of the WWTPs. The seventh WWTP, Pago Socio, serves a small area.

The facilities listed above are described in more detail in the following subsections. Figure 2-1 and Figure 2-2 show schematics of the collection system with general boundaries and flow paths. Figures 2-3 through 2-6 show the collection system piping, lift stations, WWTPs, and service areas.

2.1.1 Other Island Wastewater Systems

Other major wastewater systems on the island include the following:

- **Andersen AFB wastewater system:** Andersen AFB owns and operates its collection system, which serves the main base and Northwest Field. The main base and Northwest Field systems discharge into GWA's collection system at the two locations shown in Figure 2-1.
- **Naval Base Guam (Apra Harbor) wastewater system:** Naval Base Guam owns and operates a collection system and WWTP.
- **Other military areas:** several other military areas have collection systems which are maintained by the Navy or Air Force. Those small collection systems discharge into GWA's collection system as shown in Figure 2-1.
- **LeoPalace Resort Guam:** LeoPalace has a wastewater collection system and treatment plant. The waste sludge from the LeoPalace treatment plant is hauled by tanker truck to GWA's Northern District WWTP.

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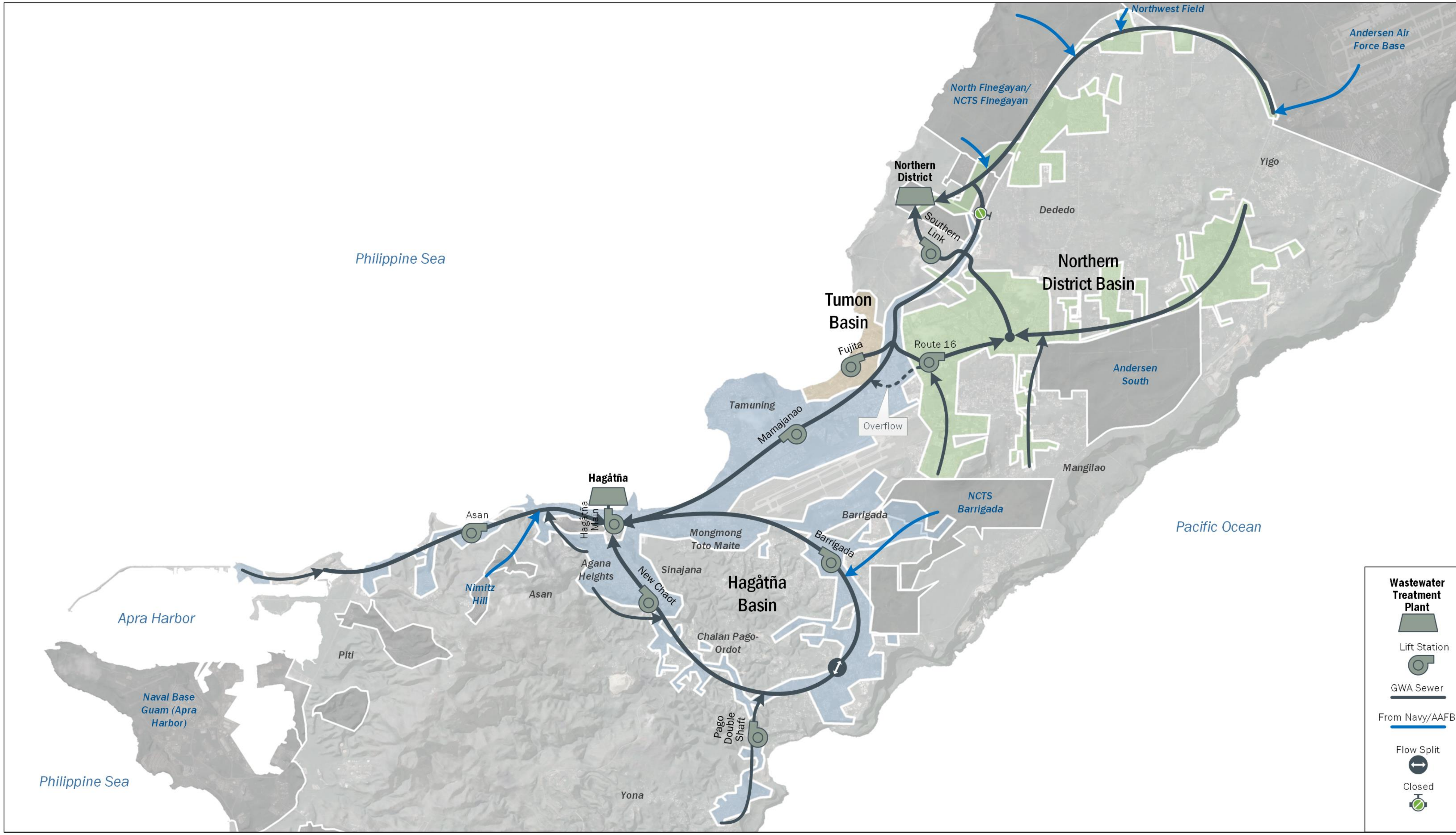

Figure 2-1. Schematic of Basins in Northern Guam

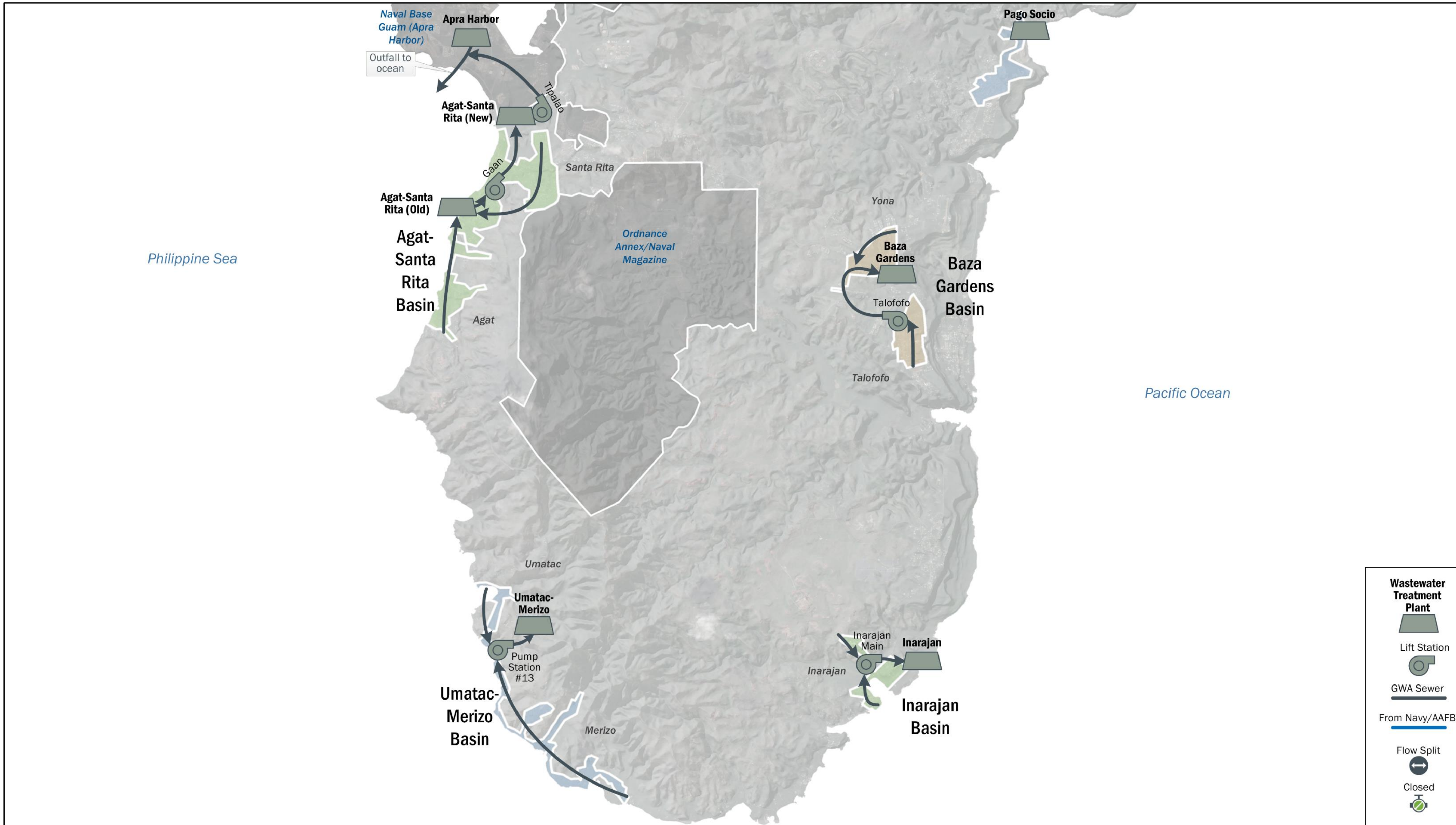

Figure 2-2. Schematic of Basins in Southern Guam

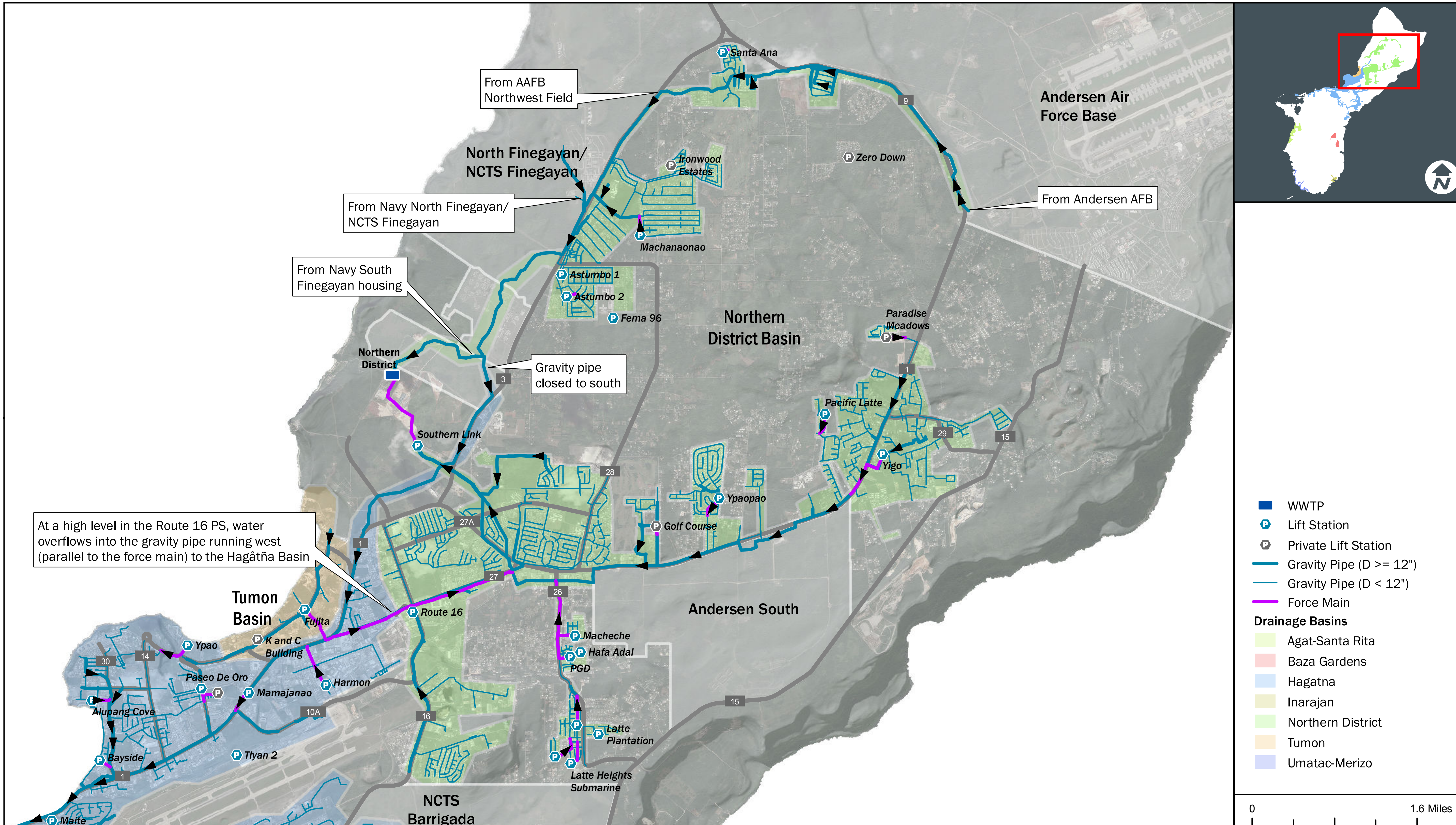

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Figure 2-3. Northern District and Tumon Basins

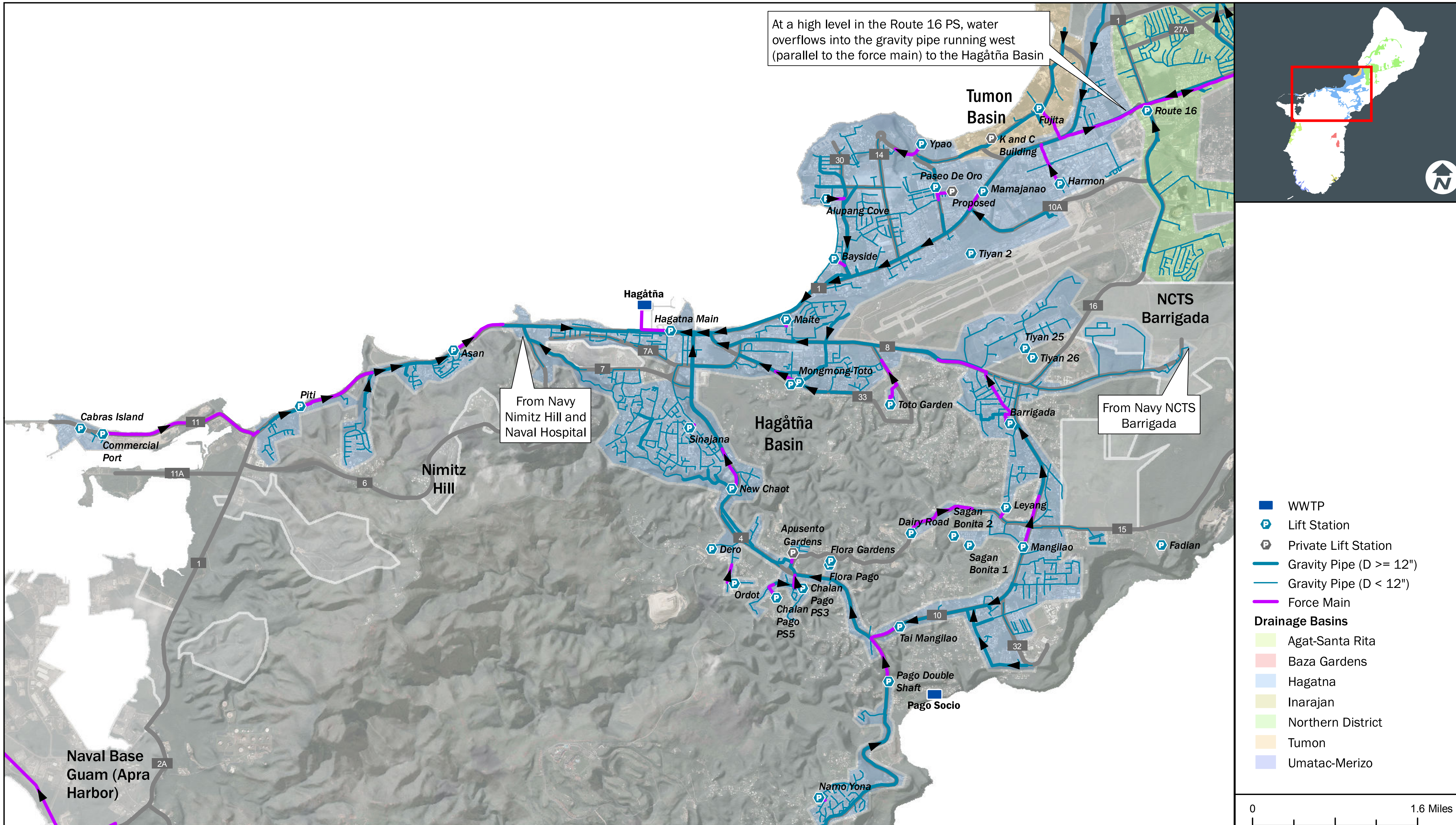

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Figure 2-4. Hagåtña Basin

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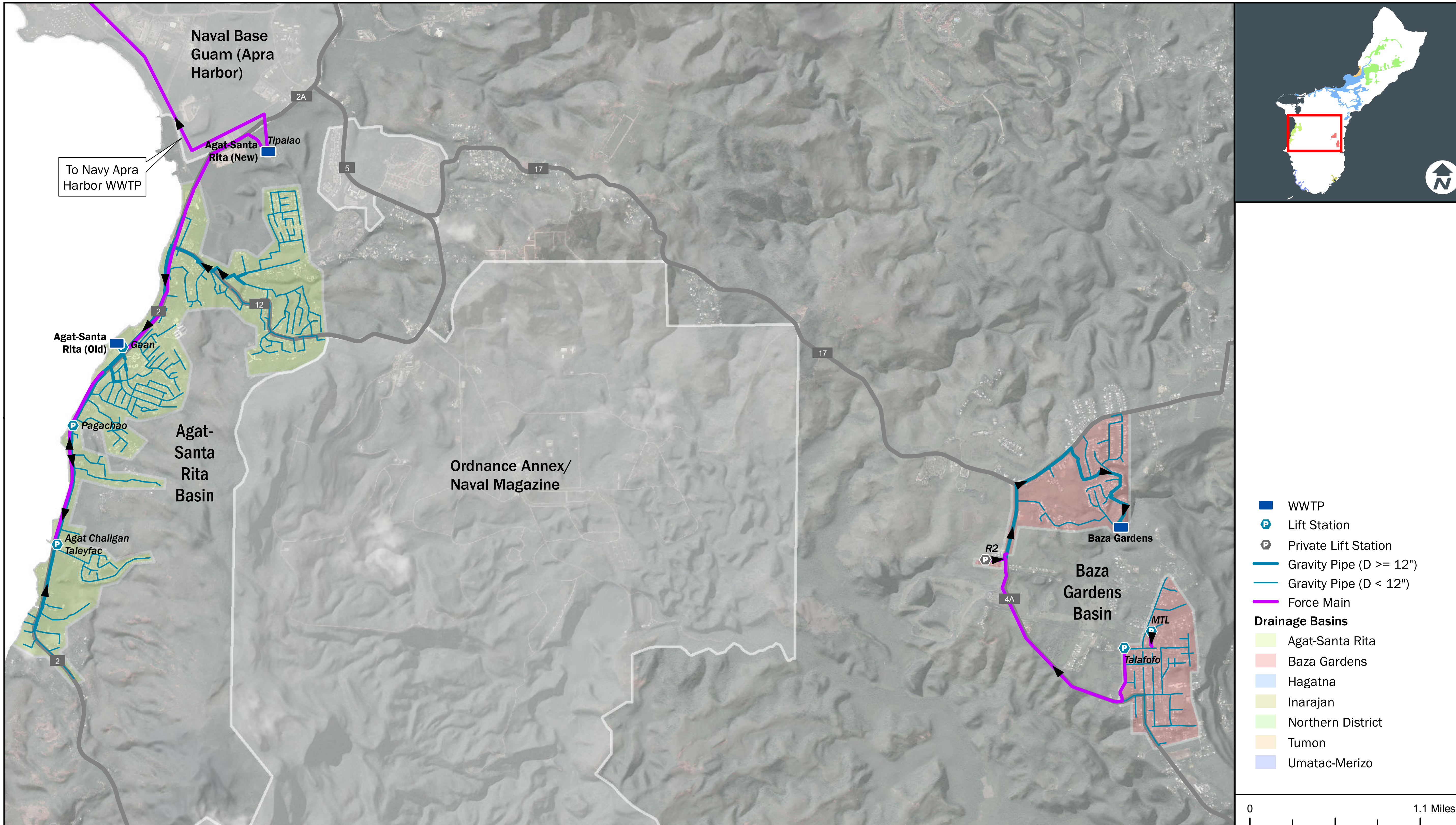

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Figure 2-5. Agat-Santa Rita and Baza Gardens Basins

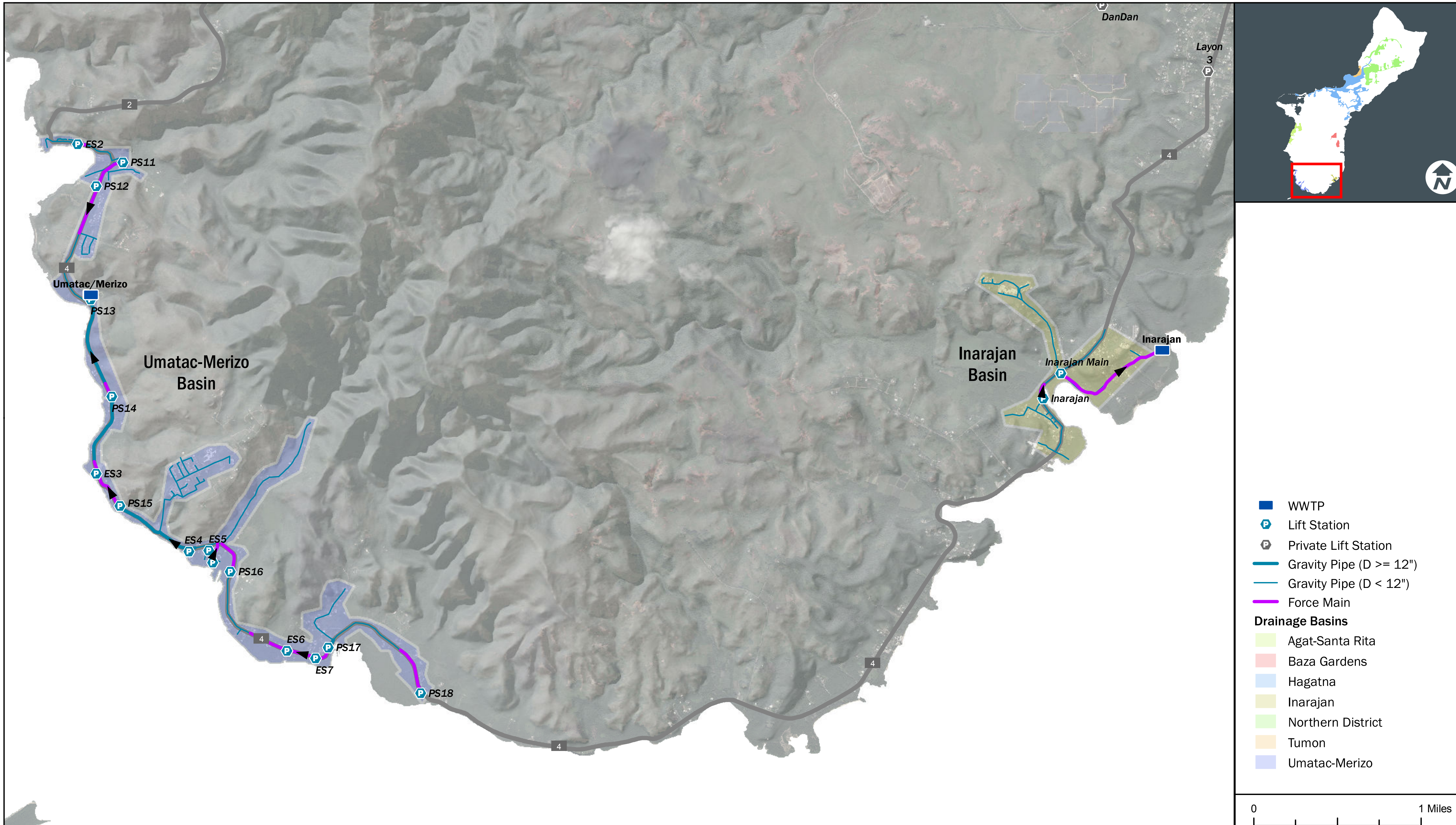

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Figure 2-6. Umatac-Merizo and Inarajan Basins

2.2 Wastewater Treatment

GWA owns and operates seven WWTPs. Table 2-1 summarizes the design capacity, treatment process, effluent disposal system, and service area of each WWTP. Additional information, including current configuration and planned changes, is presented in Section 7.

Table 2-1. Wastewater Treatment Facilities

WWTP	Design Capacity Average Daily Flow (mgd)	Type of Treatment, Process	Effluent Disposal System	Basins Served	Municipalities Served
Northern District	12.0 [a]	Chemically enhanced primary	Ocean outfall	Northern District	Dededo, Yigo, Andersen AFB, portions of Barrigada, Mangilao
				Tumon	Portions of Tamuning (including Tumon)
Hagåtña (Agana)	12.0	Chemically enhanced primary	Ocean outfall	Hagåtña	Agana, Agana Heights, Asan, Chalan Pago Ordot, Mongmong Toto Maite, Piti, Sinajana, portions of Barrigada, Mangilao, Tamuning, Yona
Agat–Santa Rita	0.75	Secondary: contact stabilization	Ocean outfall	Agat-Santa Rita	Agat, Santa Rita
Baza Gardens	0.60	Secondary: extended aeration	Togcha River	Baza Gardens	Talofofo, portions of Yona
Umatac–Merizo	0.39	Secondary: aerated lagoon/overland flow	Dry weather: evapotranspiration and percolation Wet weather: Toguan River	Umatac-Merizo	Umatac, Merizo
Inarajan	0.19	Secondary: aerated lagoon	Percolation	Inarajan	Inarajan
Pago Socio	0.025	Secondary: packaged aeration treatment system	Percolation	Serves a few homes	A very small area in Chalan Pago Ordot

a. The 2011 Court Order limits average daily flow to 6 mgd, but allows for conditional increases to 9 mgd.

Figure 2-7 shows the relative size of the design capacities of the WWTPs with respect to the total capacity provided by GWA facilities.

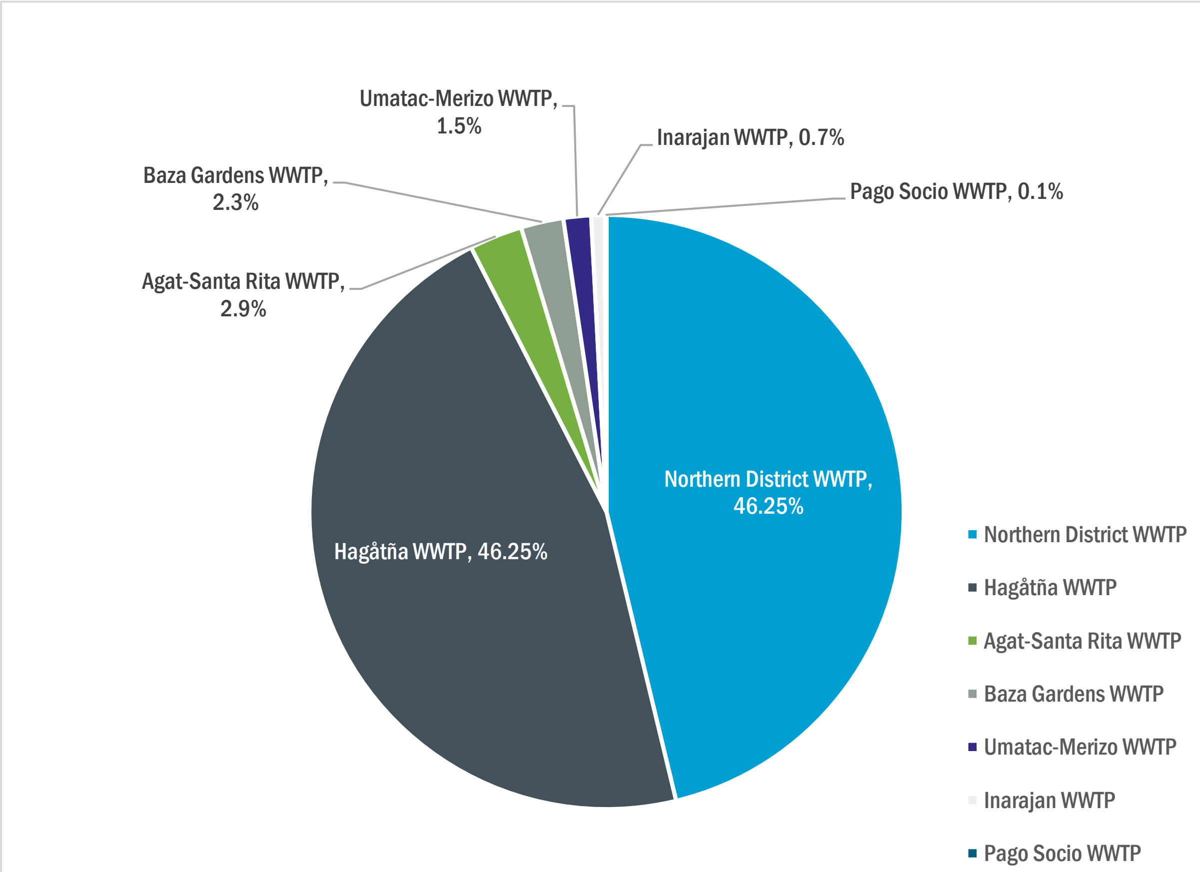

Figure 2-7. Relative Size of Design Capacities of GWA WWTPs

2.3 Collection System Piping

The collection system includes approximately 290 miles of gravity pipe with diameters ranging from 4 to 48 inches, and 27 miles of force main pipe with diameters ranging from 3 to 36 inches, for a total length of 317 miles of piping. As a comparison, the total length of modeled piping in the 2006 Water Resources Master Plan (WRMP) was 101 miles.

Tables 2-2 and 2-3 list the length of gravity piping by material and sewer basin. Table 2-4 lists the length of force main piping by sewer basin. Figures 2-3 through 2-6 show mapping of the collection system piping.

Table 2-2. Gravity Pipe Material Summary

Diameter (inches)	Length by Material (feet) [a]								Total (feet)	Percent of Length
	Asbestos Cement	Cast-in-Place Concrete	Cast Iron	Polyethylene	PVC	Reinforced Concrete	Vitrified Clay	Unknown		
4	-	-	-	-	248	-	-	604	*852*	*0%*
6	53,366	-	555	-	12,608	-	-	68,066	*134,595*	*9%*
8	227,631	2,651	12,565	6,368	345,651	1,066	2,470	316,822	*915,950*	*60%*
10	50,436	326	2,300	1,731	19,482	-	-	27,950	*102,225*	*7%*
12	25,337	1,352	725	2,122	30,293	-	110	18,899	*79,038*	*5%*
14	19,865	184	675	-	8,319	-	-	20,177	*49,309*	*3%*
15	5,469	153	-	582	18,053	-	1,044	2,522	*27,823*	*2%*
16	13,074	569	-	-	4,436	-	-	5,897	*23,975*	*2%*
18	10,625	-	227	798	24,229	751	-	11,714	*48,344*	*3%*
20	1,750	-	468	-	284	-	-	2,686	*5,188*	*0%*
21	-	-	-	-	552	-	-	-	*552*	*0%*
24	26,505	1,424	44	841	9,416	6,534	-	8,754	*53,518*	*3%*
27	9,369	-	-	999	-	-	-	205	*10,573*	*1%*
30	21,373	-	436	658	5,592	253	-	14,325	*42,637*	*3%*
36	723	853	-	1,002	2,515	-	-	15,659	*20,937*	*1%*
42	-	-	398	-	35	-	-	6,007	*6,440*	*0%*
48	-	-	-	786	7,451	-	-	254	*8,492*	*1%*
Total (feet)	**465,524**	**7,511**	**18,393**	**15,887**	**489,165**	**8,603**	**3,624**	**520,542**	**1,530,450**	**100%**
Total (miles)	**88.2**	**1.4**	**3.5**	**3.0**	**92.6**	**1.6**	**0.7**	**98.6**	**289.9**	
Percent of Length	**30%**	**0%**	**1%**	**1%**	**32%**	**1%**	**0%**	**34%**	**100%**	

PVC = polyvinyl chloride

a. Ductile iron (700 feet), polymer concrete (238 feet), and terracotta (261 feet) pipe are not broken out in the table but are included in the total length of piping.

Table 2-3. Gravity Pipe Basin Summary

Diameter (inches)	Length by Basin (feet)							Total (feet)	Percent of Length
	Agat-Santa Rita	Baza Gardens	Hagåtña	Inarajan	Northern District	Tumon	Umatac-Merizo		
4	-	-	-	-	852	-	-	*852*	*<1%*
6	13,739	-	68,758	-	50,167	1,184	746	*134,595*	*9%*
8	80,628	37,252	370,184	17,413	370,807	6,578	33,086	*915,950*	*60%*
10	8,705	7,047	49,867	729	24,198	6,515	5,164	*102,225*	*7%*
12	5,433	2,396	42,564	-	19,905	5,478	3,262	*79,038*	*5%*
14	2,533	-	31,708	-	15,068	-	-	*49,309*	*3%*
15	3,345	7,420	7,098	-	4,861	-	5,099	*27,823*	*2%*
16	809	147	20,295	-	2,725	-	-	*23,975*	*2%*
18	3,868	-	31,668	-	8,445	4,363	-	*48,344*	*3%*
20	4,151	-	1,038	-	-	-	-	*5,188*	*<1%*
21	-	-	552	-	-	-	-	*552*	*<1%*
24	-	-	32,234	-	20,377	907	-	*53,518*	*3%*
27	-	-	5,692	-	4,881	-	-	*10,573*	*1%*
30	-	-	11,261	-	31,376	-	-	*42,637*	*3%*
36	-	-	3,277	-	17,660	-	-	*20,937*	*1%*
42	-	-	-	-	6,440	-	-	*6,440*	*<1%*
48	-	-	-	-	8,492	-	-	*8,492*	*1%*
Total (feet)	**123,212**	**54,262**	**676,196**	**18,142**	**586,255**	**25,025**	**47,357**	**1,530,450**	**100%**
Total (miles)	**23.3**	**10.3**	**128.1**	**3.4**	**111.0**	**4.7**	**9.0**	**289.9**	
Percent of Length	**8%**	**4%**	**44%**	**1%**	**38%**	**2%**	**3%**	**100%**	

Table 2-4. Force Main Pipe Material Summary									
Diameter (inches)	Length (feet)								Percent of Force Main Length
	Asbestos Cement	Cast Iron	Ductile Iron	Polyethylene	PVC	Reinforced Concrete	Unknown	Total	
3	-	-	-	-	-	-	1,571	*1,571*	*1%*
4	-	302	-	-	3,098	-	4,564	*7,964*	*6%*
6	1,332	8,672	7,551	987	7,049	-	5,953	*31,544*	*22%*
7.3	-	-	-	-	1,741	-	-	*1,741*	*1%*
8	4,092	-	-	2,238	3,186	-	6,934	*16,450*	*12%*
9.1	4,336	-	-	-	-	-	-	*4,336*	*3%*
10	2,739	-	-	1,045	8,849	-	-	*12,633*	*9%*
12	-	2,993	-	-	-	-	1,424	*4,417*	*3%*
14	6,078	-	-	-	-	-	1,186	*7,264*	*5%*
16	-	-	6,352	3,077	21,201	-	-	*30,630*	*22%*
18	-	-	7,154	-	-	-	-	*7,154*	*5%*
20	-	-	-	-	2,319	-	-	*2,319*	*2%*
24	-	-	-	-	-	2,724	-	*2,724*	*2%*
30	-	-	-	-	-	-	5,741	*5,741*	*4%*
36	-	-	4,311	-	-	-	-	*4,311*	*3%*
Total (feet)	**18,577**	**11,967**	**25,368**	**7,347**	**47,443**	**2,724**	**27,373**	**140,799**	**100%**
Total (miles)	**3.5**	**2.3**	**4.8**	**1.4**	**9.0**	**0.5**	**5.2**	**26.7**	
Percent of Length	**13%**	**8%**	**18%**	**5%**	**34%**	**2%**	**19%**	**100%**	

2.4 Lift Stations

A list of 82 GWA-owned lift stations was compiled from GWA operations staff and geographic information system (GIS) data. GWA operations staff provided at least some information for the 64 lift stations listed in Table 2-5, including the lift station location and number of pumps. Pump information was available for pumps at 20 of the 64 lift stations in the table. Those 20 lift stations include 22.7 million gallons per day (mgd) out of the total 25.9 mgd (87 percent) from the average dry weather flow listed in the table. No information besides a general location was available for the other 18 lift stations, which are listed after the table. The lift stations are shown in Figures 2-3 through 2-6.

Table 2-5. Lift Stations

Lift Station	Number of Pumps at Lift Station	Design Flow (mgd) [a]	Average Dry Weather Flow (mgd) [b]	Information Availability [c]
Agat-Santa Rita Basin				
Agat Chaligan Taleyfac (also called Chaligan)	2	Unknown, unknown	0.17	
Gaan	4 (1 Dry weather, 3 wet weather)	1.9 (Dry weather) 7.8 (Wet weather)	1.86	From Agat Santa Rita Design
Pagachao	2	2.4, 1.9	0.0080	
Tipalao	3	2.9, 2.9, 2.9	0.87	
Baza Gardens Basin				
Main Trunk Line	1	1.3	0.0034	
Talofofo	2	0.7, 0.7	0.057	
Hagåtña Basin				
Alupang Cove	2	Unknown, unknown	0.24	Pump information not available
Asan	1	1.2	0.55	
Barrigada	2	Unknown, unknown	0.39	
Bayside	1	Unknown	0.082	Pump information not available
Casamiro	2	0.4, 1.3	0.0038	Pump information not available
Chalan Pago PS 3	2	Unknown, unknown	0.042	Pump information not available
Chalan Pago PS 5	2	1.3, Unknown	0.0065	Pump information not available
Commercial Port	3	2.2, 1.6, 2.6	0.016	Pump information not available
Dairy Road	2	0.5, 0.5	0.17	Pump information not available
Hagåtña Main	3	7.9, 7.9, 7.9	7.16	
Harmon	2	1.3	0.23	Pump information not available
Leyang	2	Unknown, 1	0.018	Pump information not available
Maite	2	Unknown, unknown	0.041	Pump information not available
Mamajanao	3	3, 3, 3	0.64	
Mangilao	2	1.6, unknown	0.23	Pump information not available
Mongmong-Toto	2	2.4, 2.4	0.18	Pump information not available

Table 2-5. Lift Stations				
Lift Station	Number of Pumps at Lift Station	Design Flow (mgd) [a]	Average Dry Weather Flow (mgd) [b]	Information Availability [c]
Namo Yona	2	Unknown, unknown	0.013	Pump information not available
New Chaot	3	5.5, 5.5, 5.5	1.20	
Ordot	2	0.1, unknown	0.021	Pump information not available
Pago Double Shaft	2	Unknown, unknown	0.21	
Paseo De Oro	2	1.2, 1.2	0.050	Pump information not available
Piti	1	0.9	0.35	Pump information not available
Sinajana	2	0.4, 0.4	0.027	Pump information not available
Tai Mangilao	3	0.7, 0.7, 0.7	0.59	Pump information not available
Toto Garden	2	0.2, 0.2	0.029	Pump information not available
Ypao	2	0.7, unknown	0.25	
Inarajan Basin				
Inarajan	2	0.5, 0.4	0.032	
Inarajan Main	2	0.9, 1.2	0.12	
Northern District Basin				
Astumbo No. 1	2	0.7, 0.6	0.037	Pump information not available
Astumbo No. 2	2	0.6, 0.6	0.015	Pump information not available
Latte Heights Double Tree	2	1.6, 1.6	0.073	Pump information not available
Latte Heights Submarine	2	0.6, 0.6	0.016	Pump information not available
Latte Plantation	2	0.6, 0.6	0.013	Pump information not available
Machanaonao	2	1.3, 1.3	0.027	Pump information not available
Macheche	2	0.6, 0.6	0.035	Pump information not available
Pacific Latte	2	0.9, 0.9	0.013	Pump information not available
PGD	2	0.9, 1.3	0.039	Pump information not available
Route 16	4	5, 5, 5, 5	2.68	
Santa Ana	2	0.6, 0.6	0.057	Pump information not available
Southern Link	4	15.8, 11.5, 15.8, unknown	4.65	
Sunrise Villa	2	0.3, 0.3	0.0038	Pump information not available
Yigo	3	2.3, 3.8, 3.8	0.57	
Ypaopao	3	1.6, 1.8, 1.8	0.092	Pump information not available
Tumon Basin				
Fujita	3	2.7, 2.7, 2.7	2.02	
Umatac-Merizo Basin				
Ejector Station No. 2	2	0.1, 0.1	0.012	Pump information not available
Ejector Station No. 3	2	Unknown, unknown	0	Pump information not available
Ejector Station No. 5	2	Unknown, unknown	0.0040	Pump information not available

Table 2-5. Lift Stations				
Lift Station	Number of Pumps at Lift Station	Design Flow (mgd) [a]	Average Dry Weather Flow (mgd) [b]	Information Availability [c]
Ejector Station No. 6	2	Unknown, unknown	0.0055	Pump information not available
Ejector Station No. 7	2	Unknown, unknown	0.0080	Pump information not available
Pump Station No. 11	2	1.1, 0.7	0.041	Pump information not available
Pump Station No. 12	2	0.5, 0.5	0.041	Pump information not available
Pump Station No. 13	2	1.6, 2	0.24	
Pump Station No. 14	2	0.7, 2.6	0.14	Pump information not available
Pump Station No. 15	2	1, 1	0.14	Pump information not available
Pump Station No. 16	2	0.6, 0.6	0.063	Pump information not available
Pump Station No. 17	2	0.4, 0.4	0.039	Pump information not available
Pump Station No. 18	2	0.5, 0.5	0.0059	Pump information not available
Reyes	2	0.4, 0.4	0.0092	Pump information not available

mgd = million gallons per day

a. Capacity of each pump in order of pump number as listed on a pump inventory supplied by GWA in 2016.

b. Average existing dry weather flow from the computer model. NA indicates lift stations that lacked upstream piping and associated flows.

c. At the time the computer model was constructed, data was not available for these lift stations. Therefore, the lift stations were modeled as "Ideal" lift stations, which means that all the flow that flows into the wet well is pumped out. This is described in more detail in Section 3.

The following GWA lift stations are not included in Table 2-5 because information was unavailable for the lift station pumps or due to how the lift station connects to the collection system.

- Cabras Island, Dero, Ejector Station No. 4, Fadian, FEMA 96, Flora Gardens, Flora Pago, Hafa Adai, Layon DanDan, Latte Pacific, Pump Station No. 19, Pump Station No. 20, Route 4, Sagan Bonita 1, Sagan Bonita 2, Tiyan 2, Tiyan 25, Tiyan 26

In addition to GWA-owned lift stations, the following eight private lift stations (listed by basin) pump into GWA's collection system throughout the island:

- Baza Gardens: R2
- Hagåtña: Apusento Gardens, Asnamo Yona
- Northern District: Golf Course, Ironwood Estates, Paradise Meadows, Zero Down
- Tumon: K and C Building

The Baza Gardens collection system includes four locations in Talofofo where piping from small areas is not connected to the rest of the collection system. A tanker truck collects wastewater from the four locations and discharges the collected flow to a manhole just upstream of the WWTP. A project is currently underway to construct four lift stations to serve the four locations and connect them to the collection system.

Section 3

Hydraulic Model Development

This section describes the development of the computer model used to evaluate GWA's collection system.

3.1 Model Facilities

A computer model of the wastewater collection system was built using Innovyze's InfoSWMM software. Models of each sewer basin were initially constructed and combined into a single model. Table 3-1 lists the source of information for each basin model.

Table 3-1. Summary of Model Sources			
Region	Basin	Year Basin Model Updated	Report Discussing Model Update
North	Northern District	2016	Northern District Wastewater Conveyance System Model Update (Brown and Caldwell [BC], 2016)
North	Tumon	2015	Tumon Wastewater Conveyance System Model Update (BC, 2015)
Central	Hagåtña	2016	Central District Wastewater Conveyance System Model Update (BC, 2016)
South	Agat-Santa Rita	2015	Agat-Santa Rita Wastewater Conveyance System Model Update (BC, 2015)
South	Baza Gardens	2016	Baza Gardens Wastewater Conveyance System Model Update (BC, 2016)
South	Inarajan	2016	Inarajan Wastewater Conveyance System Model Update (BC, 2016)
South	Umatac-Merizo	2016	Umatac Wastewater Conveyance System Model Update (BC, 2016)

3.1.1 Piping

The GWA GIS was the original source of the manhole and pipe locations for the model, which includes all active pipes and manholes in the GIS database. BC performed field investigations and interviewed GWA staff to update the piping in many areas throughout the collection system where the piping was incorrect. Missing and incorrect manhole rim elevations were interpolated from a 2007 Light Detection and Ranging (LIDAR) survey performed by the U.S. Army Corps of Engineers for the entire island. Missing and incorrect pipe inverts were estimated from neighboring pipe inverts. Appendix B discusses the use of GIS data in the model in more detail.

3.1.2 Lift Stations

Table 2-5 lists the 64 modeled lift stations. The last column of the table indicates the availability of pump information. For the 44 lift stations where pump information was unavailable, the pumps were modeled as ideal, which is a type of model pump where the pump's discharge flow rate equals the inflow rate. The pumps at the other 20 lift stations were modeled with pump curves.

Wet wells at the lift stations were modeled with the following sizes:

1. Actual wet well sizes were used in the model for the lift stations with known wet well sizes.

2. Wet wells at the rest of the lift stations were modeled using estimated wet well sizes. The actual dimensions for a wet well at a specific lift station (e.g. a 1-mgd lift station) were used for a similarly sized lift station with an unknown wet well size.

3.2 Model Flows

The following section describes the flows used in the collection system model.

3.2.1 Flow Metering

Between 2005 and 2015, temporary flow metering was performed by GWA; ADS Environmental Services; Engineering, Science, and Technology, Inc.; and Stanley Consultants. The flow metering was performed for previous studies. Temporary flow metering efforts, including timing and installation location, are summarized in Appendix A. The flow metering data was used to calibrate the computer model.

3.2.2 Wastewater Flow Components

Wastewater has three basic flow components: base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-derived infiltration and inflow (RDII). A typical representation of these wastewater components is shown in Figure 3-1. Each component is described below.

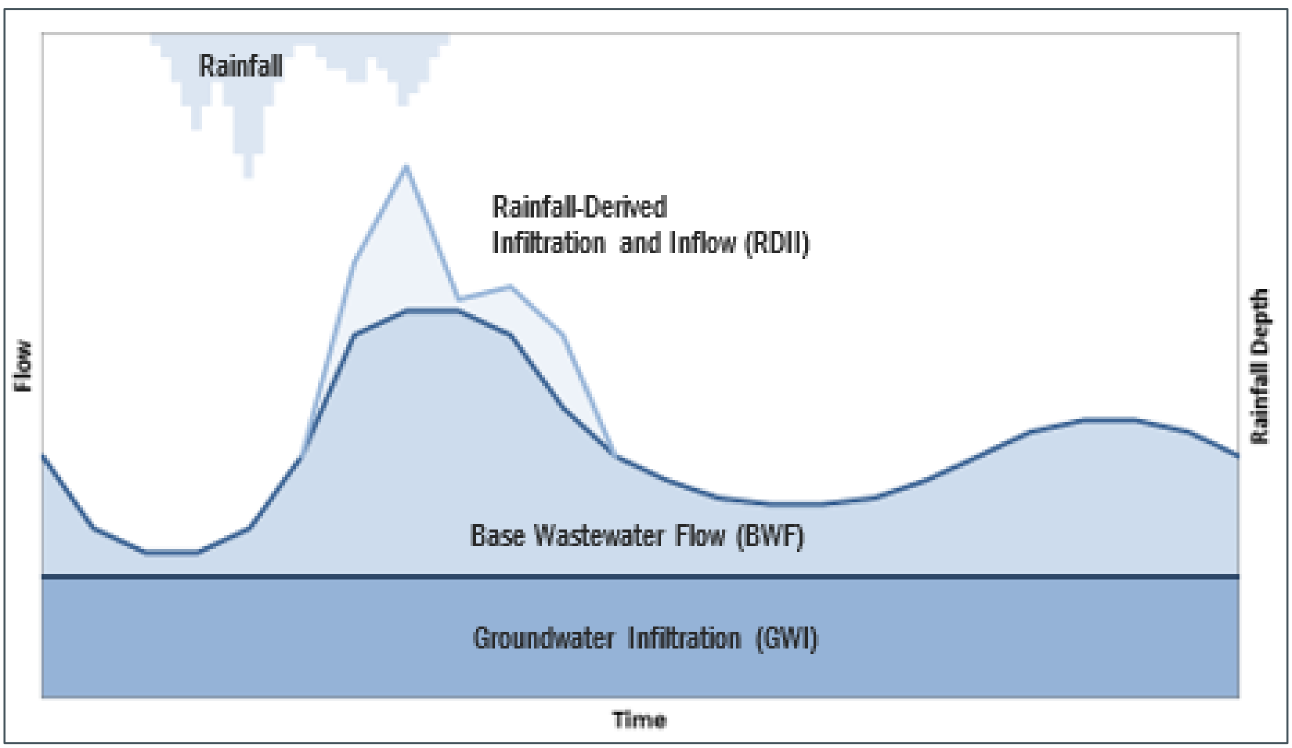

Figure 3-1. Wastewater Flow Components

Base Wastewater Flow

BWF is wastewater flow generated from residential, commercial, industrial, and public or institutional sources that discharges into the wastewater collection system. BWF may vary in magnitude throughout the day, but it generally follows a predictable diurnal pattern with peak flow occurring during the morning and evening hours. Predominantly commercial or industrial areas may have patterns that are different from residential areas, depending on the type of use. Peak flows may also be higher or lower on weekends than on weekdays, particularly in predominantly residential areas. BWF may be impacted by water use practices such as water conservation.

Average daily BWF was calculated from average water billing data. Water billing data was obtained from GWA for March 2015 through January 2016 and the average billed water use was calculated for each sewered customer. A percentage of the average water use for each customer was applied to the closest model manhole using the following steps:

1. An analysis was performed to find the closest collection system pipe to each customer. The customer was then assigned to the downstream manhole on that pipe. Customers more than 1,000 feet from a pipe were not assigned and were assumed to be on a septic or cesspool system.
2. Average billed water use for each customer was multiplied by a scaling factor until the total water use within a flow meter area matched the average flow through each temporary flow meter. Because not all water used by a customer returns to the sewer, scaling the billed water use accounted for the difference between water and sewer flows.
3. Total flow for each model manhole was calculated as the sum of the scaled water use for all customers assigned to that manhole.

Diurnal patterns, or hourly peaking factors, were created and calibrated to match the timing of peak BWF based on flow metering data. The diurnal patterns used in the model for each basin are shown in Appendix B. In the model, diurnal patterns are multiplied by the BWF applied at the model manholes.

Groundwater Infiltration

GWI is groundwater infiltration and inflow (I/I) that infiltrates into the wastewater system through joints and cracks in pipes and manholes. GWI varies by area depending on the condition of the pipes and manholes and their location with respect to the local groundwater table. GWI typically stays constant throughout a single day, but can vary seasonally. Near the coast, GWI can also be influenced by ocean tides.

GWI was calculated by finding the difference between total flow and BWF during dry-weather periods. In the model, the GWI calculated for each meter drainage area was spread out to the manholes in each drainage area based on the length of pipe flowing into each manhole.

Rainfall-Derived Infiltration and Inflow

RDII consists of stormwater entering the collection system as the direct inflow of stormwater runoff or rainfall-induced infiltration. Inflow occurs when stormwater flows directly into the collection system through connected catch basins, manhole covers, roof drains, or yard drains. Inflow usually occurs very rapidly during rain events and can become more severe if surface flooding occurs and manholes are submerged or are used to drain low-lying areas. Rainfall-induced infiltration is caused by stormwater percolating through the ground and entering pipes, manholes, and service laterals through cracks and defective joints. RDII may also include flow from basement drains or sump pumps. If these defects are combined with a high water table, RDII can last several days after the end of a rainfall event.

The magnitude of RDII is related to the intensity and duration of the rainfall, relative soil moisture at the time of the rainfall event (typically a function of the amount of rainfall prior to the event), condition of the pipes, and other factors such as soil type and topography. In most areas, peak flows during rainfall events are the highest flow rates that occur in the wastewater system. However, in areas where the pipes are relatively "tight" and I/I is minimal, peak wet weather flows may not be appreciably higher than peak dry weather flows.

In the model, RDII is classified into the following types of response:

- **Fast Response:** direct inflow due to rainwater draining into the collection system from surfaces that drain quickly, such as impermeable roads and roofs.
- **Medium response:** inflow similar to fast response, except that the precipitation takes longer to drain into the system, such as from fields.
- **Slow response:** slower runoff response that can last several days after the end of the rainfall. Slow response is a result of saturated soil and temporarily raised groundwater due to a rain event.

RDII rates were initially developed by reviewing flow metering data. Rates were then adjusted during model calibration by comparing model and meter flows during wet weather events and adjusting the model RDII response to achieve a good match between meter data and model results. The area contributing wet weather flow due to RDII was assumed to be a 200-foot buffer (i.e., 100 feet on each side) along the length of each pipe. In the model, RDII calculated for each meter drainage area was distributed to the manholes in each drainage area based on the length of pipe draining to each manhole.

Figure 3-2 shows an example of RDII in a flow meter in the Northern District collection system. The lines at the top of the graph depict rainfall. The solid line indicates the flow through the meter, including during the rainfall event. As a comparison, the dashed line shows average daily dry weather flows. The difference between the solid and dashed lines is the fast and medium response as the flow increases sharply during heavy rainfall. This meter did not indicate very much slow response, which would show as elevated flows even after the rainfall ended. This lack of slow response is expected due to the permeable limestone in northern Guam which has the potential for rapid infiltration of rainfall (see Volume 1, Section 5.2.1).

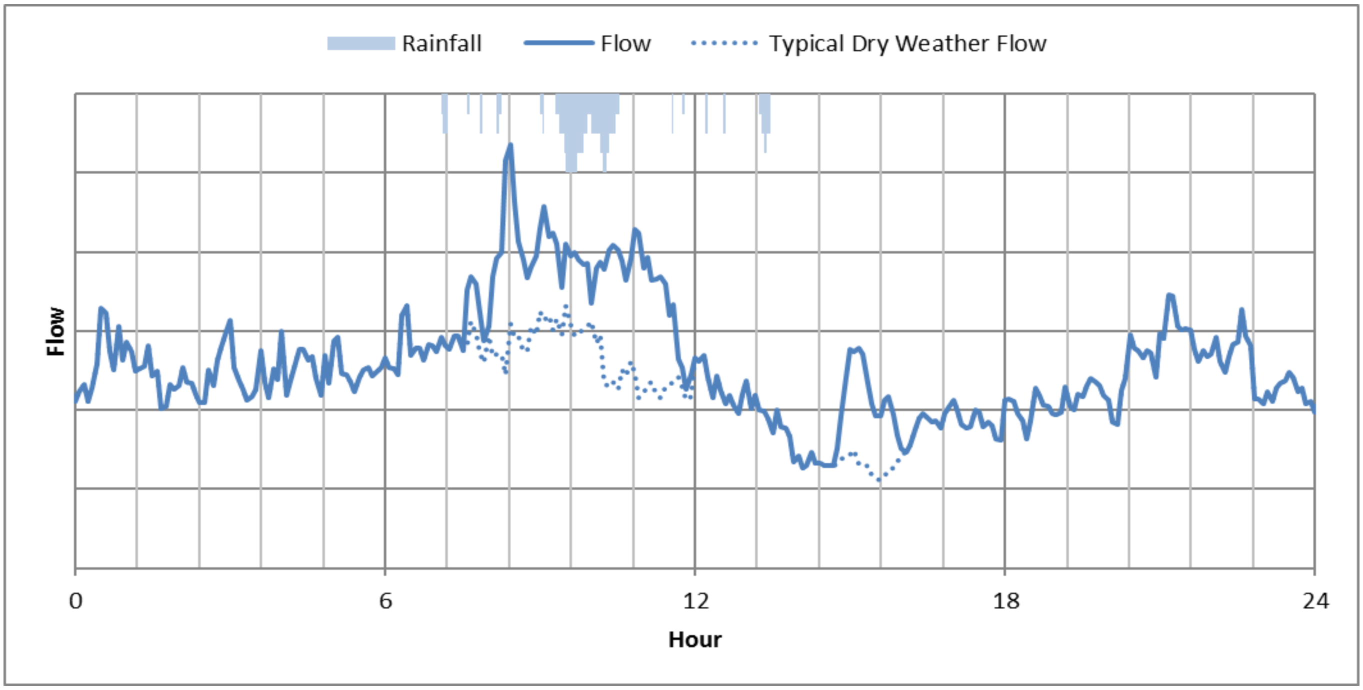

Figure 3-2. Example of RDII at a Flow Meter

3.2.3 Model Calibration

Calibration is the process of adjusting modeling input parameters to match model results with measured data or observed conditions within the system. The calibration included adjusting the following:

- **Dry weather flows:** dry weather flows were adjusted to match metered flows for dry weather periods.
- **Wet weather flows:** wet weather calibration included adjusting rainfall runoff parameters for each meter drainage area to obtain a better match between metered and modeled flows for wet weather periods.
- **Model piping and lift station configuration:** flow metering identified situations where model piping was not connected correctly or a lift station layout was not correct. In those situations, additional research was performed and the model piping and lift station layouts were corrected to reflect real-world conditions.
- **Model operations:** model pump operations were simulated by setting controls to turn pumps on and off according to wet well levels. Information obtained from GWA and information from the flow metering were used for the pump controls.

The reports listed in Table 3-1 describe the calibration of each basin model. Table 3-2 summarizes the overall calibration results for each basin.

Table 3-2. Summary of Model Calibration		
Region	**Basin**	**Calibration Summary**
North	Northern District	This basin model was adequately calibrated. Flow meter data was poor for three of the ten flow meters, with mass balance problems and poor quality flow data at those meters. However, there was sufficient quality data from the other seven meters to calibrate the model.
North	Tumon	Flow data for this basin was poor, especially during wet weather events. Flows decreased between upstream and downstream meters and pipe diameters were questionable where the meters were located (which affected the flow meter flow calculations). Overall, this basin model did not calibrate well.
Central	Hagåtña	Overall, this model did not calibrate well, especially during wet weather events. This was primarily due to the quality of the flow data. Another factor included difficulty in matching pump operations during storm events, especially for lift stations with limited available pump data.
South	Agat-Santa Rita	This basin model was adequately calibrated.
South	Baza Gardens	This basin model was adequately calibrated.
South	Inarajan	This basin model was calibrated based on flow data collected during the 2006 WRMP. Flow data appeared to be sufficient to calibrate this model.
South	Umatac-Merizo	This basin model was adequately calibrated.

3.2.4 Future Flows

Future 2035 flows were calculated using the population projections discussed in Volume 1, Section 4. The following values list the calculated population for 2015 and 2035:

- 2015 population = 164,882
- 2015–2035 non-military growth = 29,399
- 2015–2035 military growth (on-base troops and dependents) = 6,300
- 2035 total population = 164,882 (2015) + 6,300 (military growth) + 29,399 (non-military growth) = 200,581

The projected growth will not occur evenly throughout each municipality. Much of the non-military growth will occur in new developments. Military growth and planned new developments that GWA was tracking at the time of this report are shown in Volume 1, Table 4-18 and Figure 4-16.

Flows for future military growth were applied to appropriate locations in the model. Flows for non-military growth were added to the model using the following steps:

1. Existing average dry weather flow was calculated for each municipality by summing the existing base average wastewater flow inside of each municipality.
2. The increase in flow due to growth in each municipality was calculated by multiplying the existing flows by 17.6 percent (2015–2035 non-military growth of 29,399 on top of the existing population of 164,882).
3. The increase in flow in step 2 was allocated to each municipality using the following steps:
   a. Future development – flows for future developments were allocated directly to the appropriate manholes near the developments.

b. Existing customers – the remainder of the flow due to growth was spread among the customers within the municipality. The remainder of the flow was calculated as the overall increased flow for a municipality (Step 2) minus the growth already applied to future developments (Step 3a).

4. The new flows calculated in Step 3 were applied to the manhole closest to the new flow in the model.

3.2.5 Model Flow Summary

Table 3-3 lists the total average daily flows used in the model for existing and future conditions.

Table 3-3. Flow Summary

Region	Basin	2015–2016 Average Billing Data (mgd)	Existing Average Daily Flow (2015) (mgd)	Future Average Daily Flow (2035) (mgd)
North	Northern District	3.19	3.75	4.33
North	Tumon	2.00	1.99	2.82
Central	Hagåtña	4.64	7.59	8.92
South	Agat-Santa Rita	0.45	0.47	0.58
South	Baza Gardens	0.09	0.21	0.28
South	Inarajan	0.06	0.12	0.14
South	Umatac-Merizo	0.10	0.24	0.28
Total		**10.54**	**14.38**	**17.36**

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Section 4

Gravity Piping Evaluation

This section summarizes the capacity and condition of the gravity piping in GWA's wastewater system.

4.1 Condition Assessment

As part of the development of the 2006 WRMP, GWA conducted condition assessments at 303 manholes to prioritize inspection efforts and ongoing data collection. Other key industry standard information normally used for condition assessment, such as closed-circuit television (CCTV) reports, were not available at the time of the assessment. As a result, a prioritized data collection program for GWA's wastewater system was developed as part of the 2006 WRMP to support future condition assessment efforts. Since the completion of 2006 WRMP, GWA has been collecting data through CCTV inspections, manhole inspections, and field surveys.

The latest condition assessment data for gravity pipes and manholes was used in the capacity and condition analyses. The data included:

- **GIS data:** GIS data used in the analysis is summarized in Appendix D. The GIS data was used in the capacity and condition analyses.
- **CCTV and condition assessment reports:** approximately 2,200 CCTV videos and 800 condition assessment reports were available. Most CCTV videos were completed between 2012 and 2015. Most condition assessment reports were completed from 2014 through 2016. As CCTV data was reviewed, CCTV data and reports were collected from multiple computer and server locations. Some videos and reports were saved in more than one location, resulting in the creation of duplicate copies. Section 4.7.1 contains a recommendation regarding CCTV data storage management. The CCTV and condition data was used in the gravity pipe condition analysis.
- **Manhole inspection reports:** approximately 615 manholes were inspected in 2015. Figure 4-1 shows the locations of the manhole inspections and identifies the manholes that were flagged as needing repair during the inspections. Appendix D includes a list of inspected manholes and the results of the inspections. The manhole inspection data was used in developing recommendations for manholes.
- **Sanitary sewer overflow forms:** 982 sanitary sewer overflow (SSO) incident forms/reports were reviewed. The location for approximately 85 percent of the SSOs were identified in the GIS from location descriptions in the SSO forms. The location information was insufficient to locate the rest of the SSOs in the GIS. Volume 1, Section 8 (GIS) discusses recommendations for improved collection of data such as SSO locations. Figure 4-2 shows the locations of the SSO incidents. The SSO data helped in developing and prioritizing gravity pipe capacity recommendations. The following two types of forms were used:

— **Incident notification forms:** approximately 800 SSO incident notification forms were collected from Appendix CS-N in the 2013 U.S. Environmental Protection Agency (USEPA) National Enforcement Investigations Center (NEIC) wastewater report (USEPA NEIC, 2013). The forms document spill incidents reported between 2007 and 2013 and contain records of various sewer system operation and maintenance (O&M) incidents such as overflows/spills, surcharges, sewer line backup events, etc. Information recorded on the forms includes the date, location, incident description, and corrective action taken.

— **Incident reports:** GWA provided 182 SSO incident reports for 2013 through 2016.

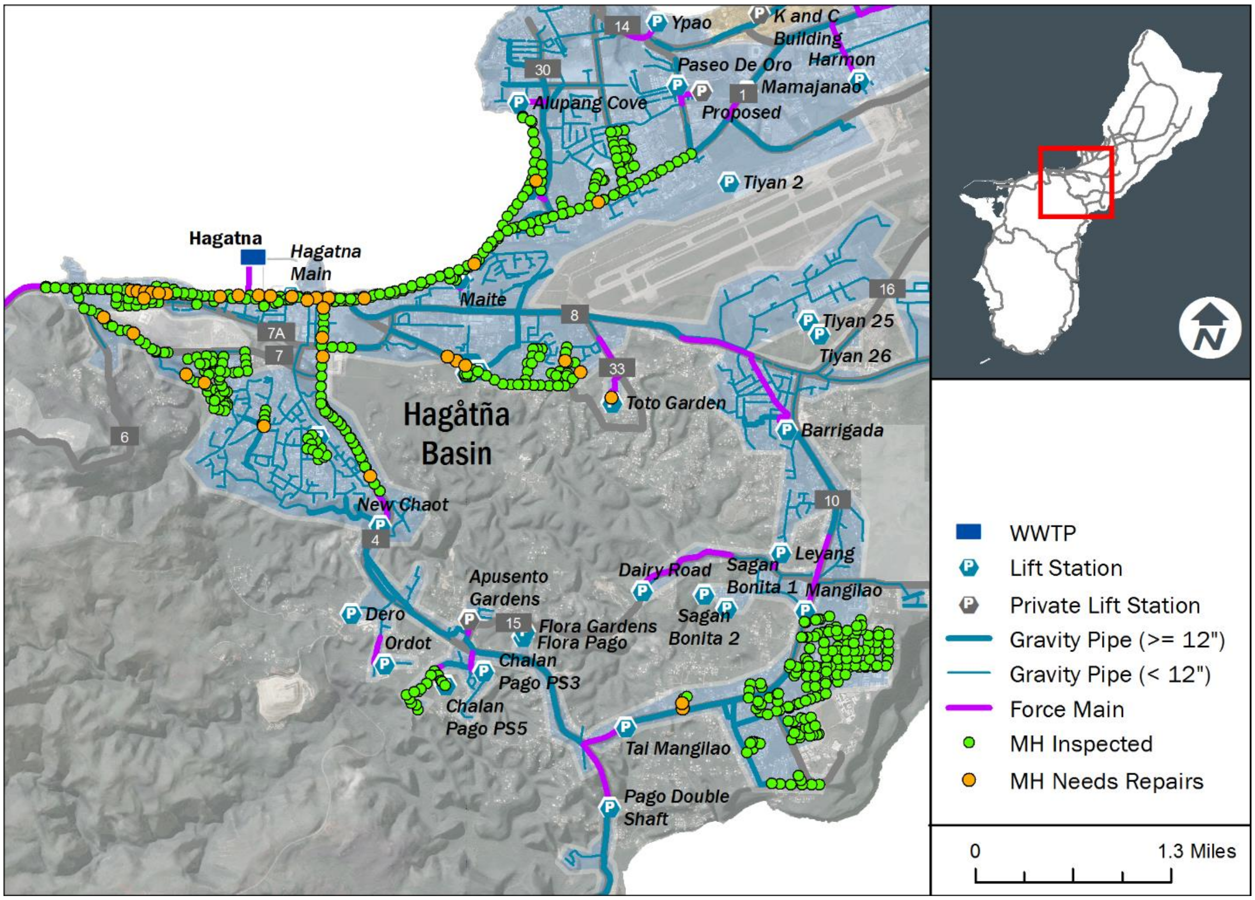

Figure 4-1. Priority Manhole Rehabilitation

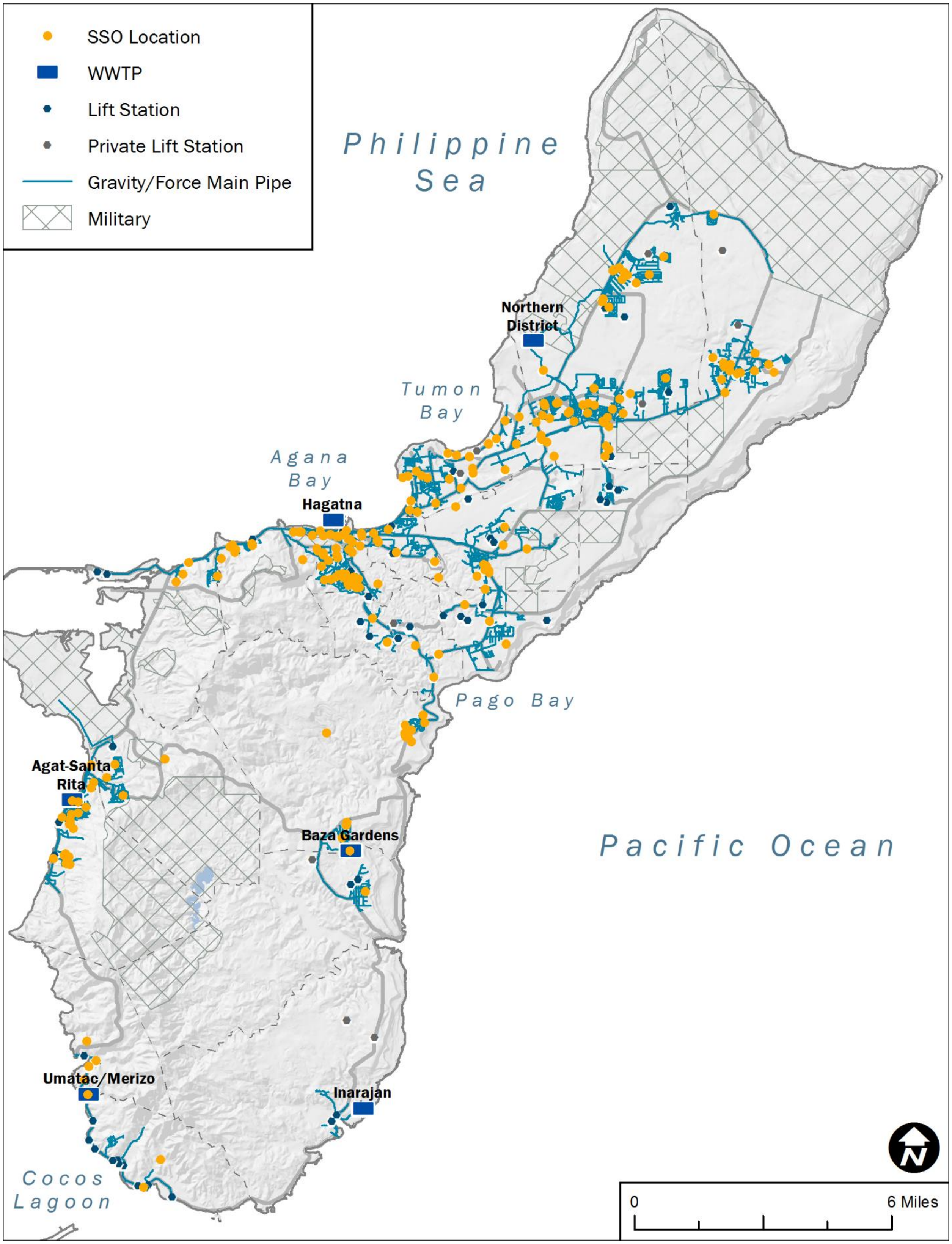

Figure 4-2. SSO Notifications Between 2007–2016

4.1.1 Manholes

GWA is starting to conduct manhole inspections using Manhole Assessment and Certification Program (MACP) standards when a pipe is inspected with CCTV. Section 4.7.2 lists recommendations for manholes based on the new MACP data and the existing condition data summarized above.

4.2 Gravity Piping Capacity Evaluation

The capacity of the gravity piping was analyzed and compared to existing and future flows using the following criteria (see Appendix C for additional details on the criteria):

- **Design storm:** a 2-year, 24-hour design storm was used to evaluate the collection system, identify deficiencies, and develop improvements. The model was evaluated using peak wet weather flows from the design storm.
- **Allowable depth to diameter ratio at peak flow:** a pipe was flagged as deficient if the depth to diameter (d/D) ratio was greater than 1 (if the water level reaches the top of the pipe). Pipe segments with d/D greater than 1 because of backups from downstream piping were not considered to have insufficient capacity.

Gravity piping was evaluated using the criteria at existing and future dry and wet weather flows. Table 4-1 summarizes the evaluation results for each flow scenario. Figures 4-3 through 4-6 show the locations of the piping identified as having insufficient capacity at future peak wet weather flows.

Table 4-1. Percentage of Total Pipe Length with Capacity Deficiency by Basin

Flow Scenario	Agat-Santa Rita	Baza Gardens	Hagåtña	Inarajan	Northern District	Umatac
Existing Dry Weather	-	-	-	-	-	-
Future Dry Weather	-	-	1%	-	1%	-
Existing Wet Weather	7%	11%	24%	-	7%	4%
Future Wet Weather	7%	11%	28%	-	13%	5%

Figure 4-4 and Table 4-1 shows that a large amount of the collection system in the Hagåtña basin was predicted to have capacity problems. As shown in Table 3-2, the Hagåtña basin did not calibrate well due to issues with flow metering data. Therefore, in the Hagåtña basin, improvement projects were only developed for a few areas with known capacity issues.

There is overlap between the identified capacity issues and the condition issues discussed later in this section. Therefore, recommendations for pipe improvements were developed by looking at capacity and condition issues together. Recommendations for pipe improvements are discussed in Section 4.7.4.

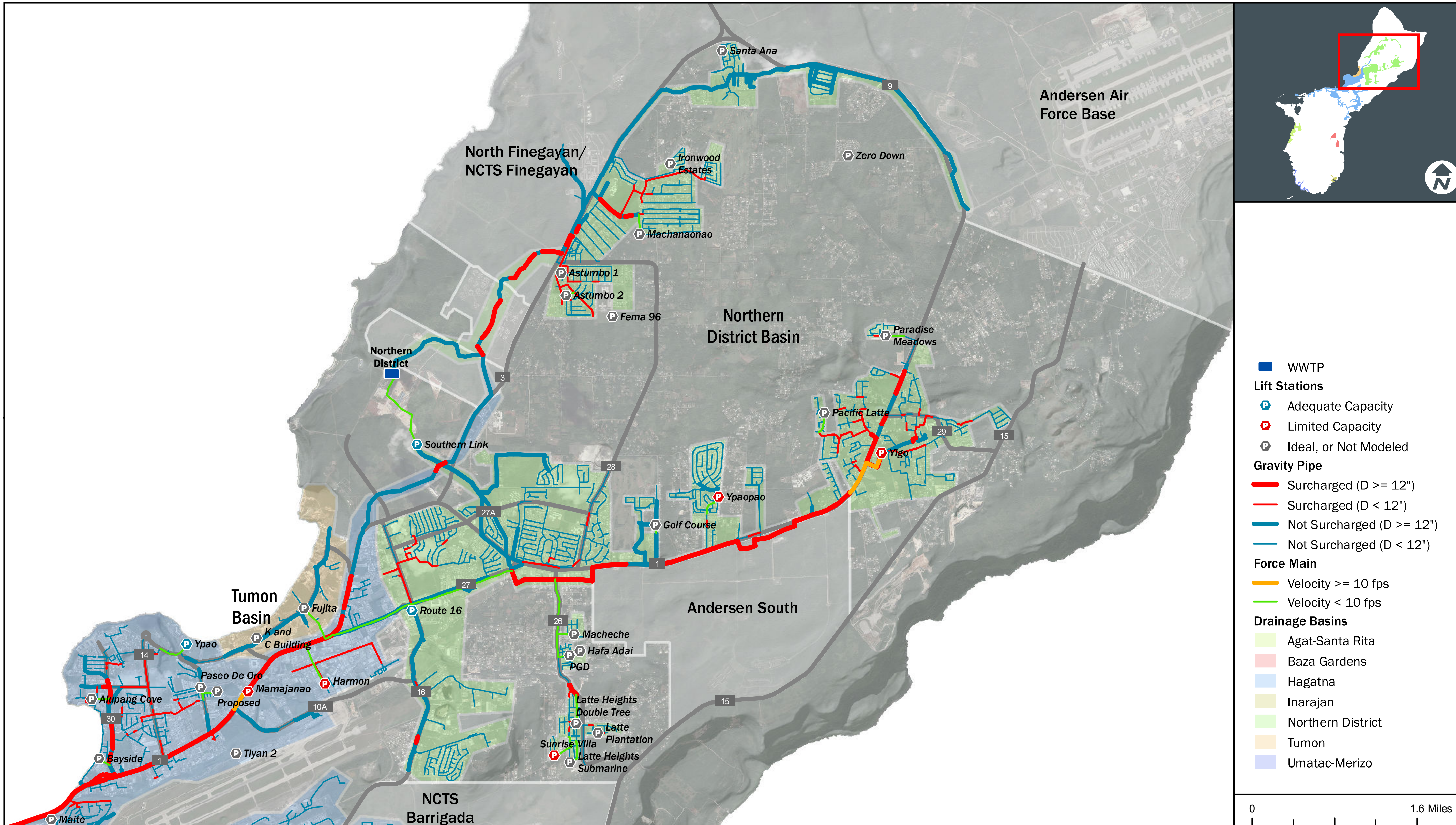

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Figure 4-3. Northern District and Tumon Basins Piping and Lift Station Deficiencies

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Figure 4-4. Hagåtña Basin Piping and Lift Station Deficiencies

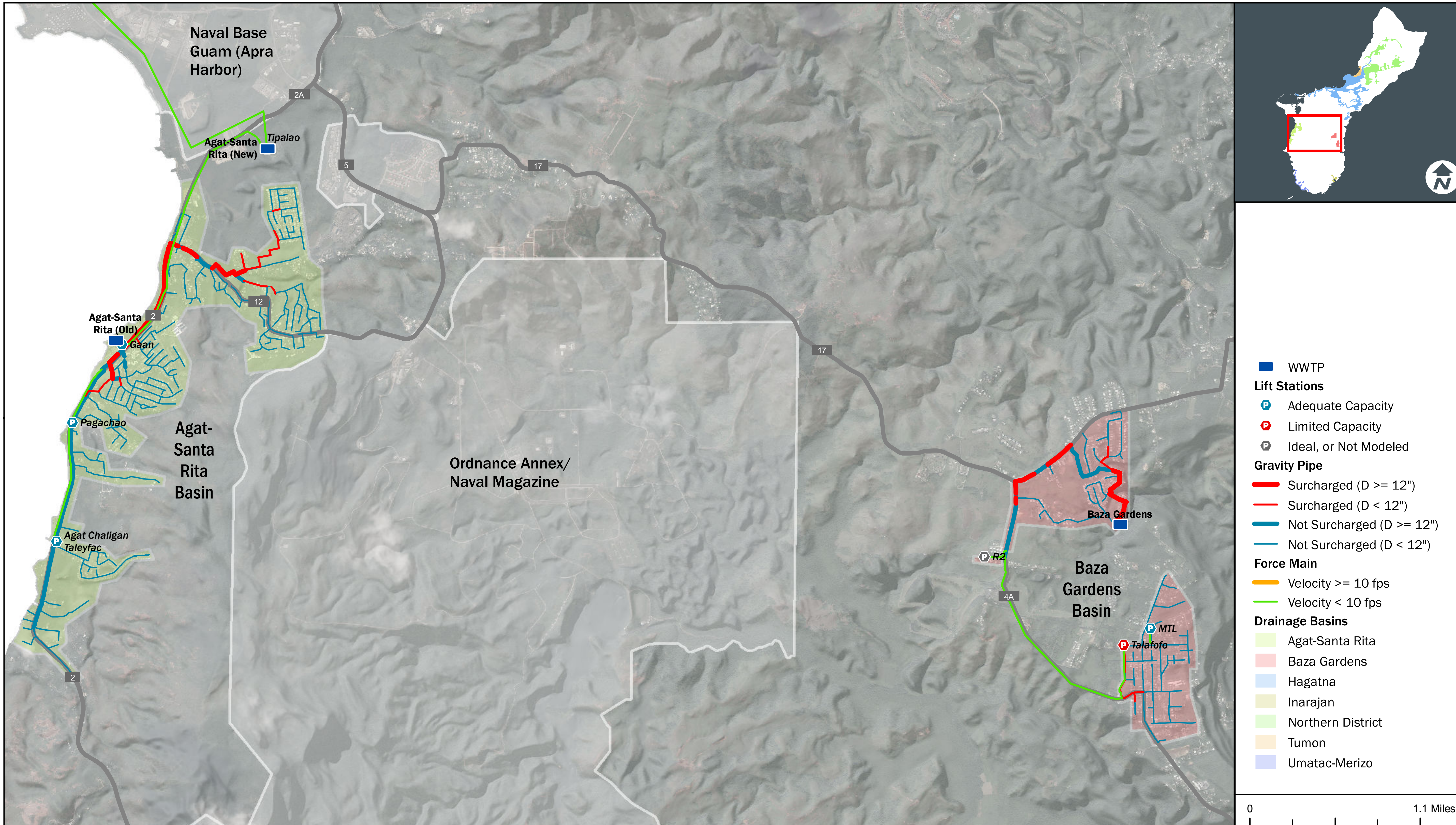

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Figure 4-5. Agat-Santa Rita and Baza Gardens Basins Piping and Lift Station Deficiencies

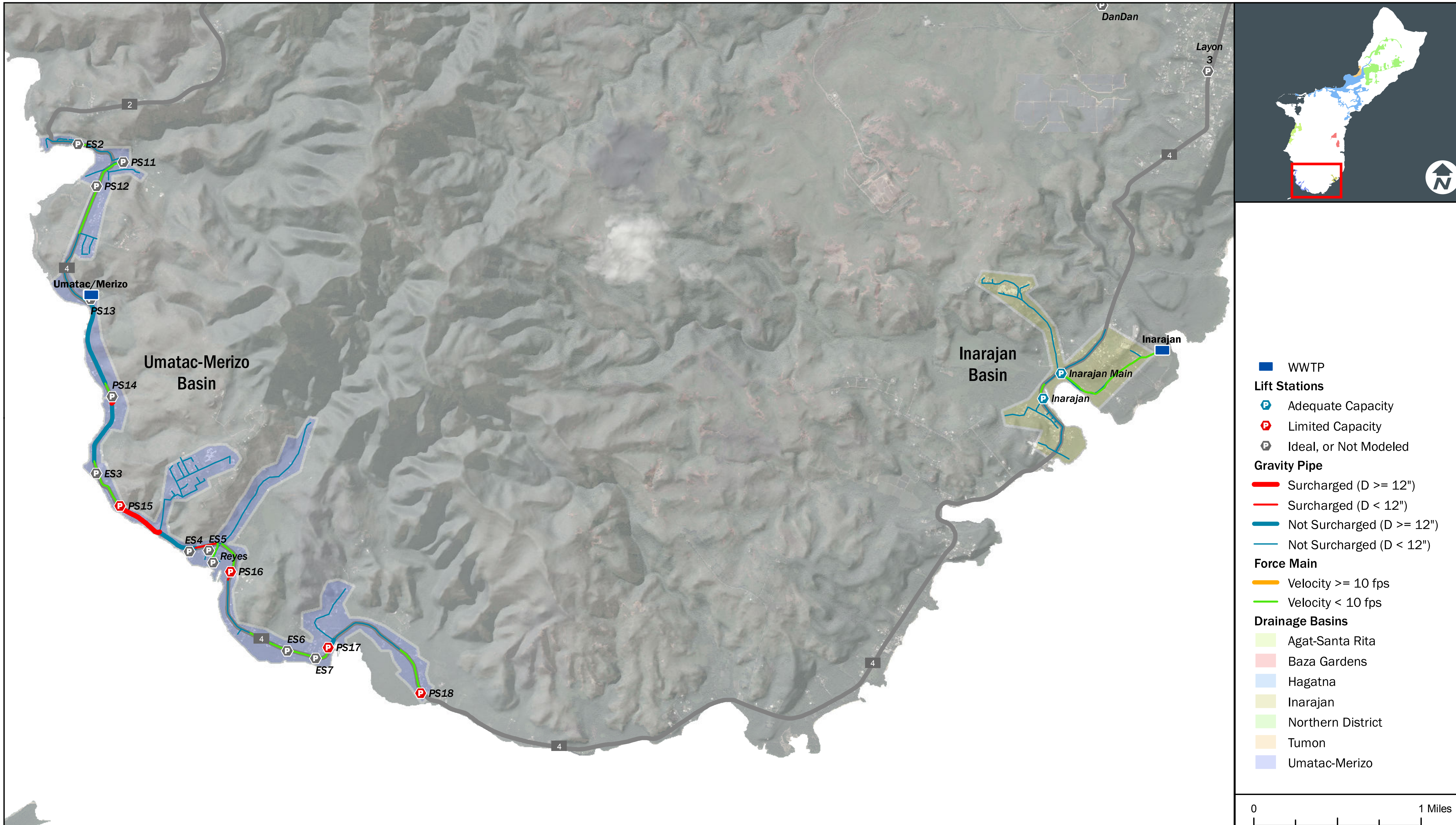

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Figure 4-6. Umatac-Merizo and Inarajan Basins Piping and Lift Station Deficiencies

4.2.1 Mamajanao Lift Station Evaluation

GWA reported capacity problems with piping downstream of the Mamajanao lift station. The Mamajanao lift station can currently run only one pump. If a second pump is turned on, piping downstream surcharges, and because the line is shallow, a manhole lid pops up and an SSO occurs. With only one pump running, the lift station wet well can overflow during peak flows. GWA recently cleaned piping downstream of the lift station, but there still appears to be a capacity issue in the downstream piping.

The model was used to calculate pipe sizes needed downstream of the lift station to eliminate capacity issues. Because the model did not calibrate well in the Hagåtña basin, piping was sized based on the capacity of the Mamajanao lift station and was not based on model generated flows. The upsized pipe sizes are listed in the recommendations in Table 4-13 at the end of this section and include approximately 4,200 feet of piping. Figure 4-7 shows a profile of the piping into and out of the Mamajanao lift station and the extents of the piping proposed for the gravity sewer upsizing to the Hagåtña WWTP.

Option to Reverse Flows

There is a potential option to address the Mamajanao capacity issues by reversing flows from the Mamajanao lift station to the northeast along Route 1 to the Route 16 lift station. This would reduce flow to the Hagåtña WWTP and add the flow to the Northern District WWTP. A force main was installed between the Mamajanao and Route 16 lift stations in the early 1990s. The route and elevation change of the force main are shown in Figure 4-7. The current condition of this pipeline is unknown. The Mamajanao lift station currently lifts flow about 40 feet in elevation, but pumping to the Route 16 lift station would require an additional 60 feet of elevation lift. The additional static head would likely require pump replacement at Mamajanao.

Advantages of reversing flow from Mamajanao include:

- Removes the flow from the collection system draining from Mamajanao to the Hagåtña WWTP which would free up capacity at the WWTP for other developments.
- Reduces the ultimate capacity needs of the Hagåtña WWTP which is beneficial due to the current limited space available at the current WWTP site.
- Adds the flow to the Northern District WWTP which is currently being upgraded to provide secondary treatment.

Disadvantages of reversing flow from Mamajanao include:

- The capability of the Route 16 lift station to overflow to Mamajanao will be eliminated or at least become more complicated to operate.
- There will be a higher power cost for the increase in static head.
- Improvements may be required at the Route 16 lift station to accommodate the increase in capacity.

Summary

An improvement project is proposed for increasing the gravity pipe size downstream of the Mamajanao lift station. A component of this project will be to complete a study to analyze the reverse flow alternative including inspection of the existing pipe condition and to evaluate any improvements required for the Route 16 lift station.

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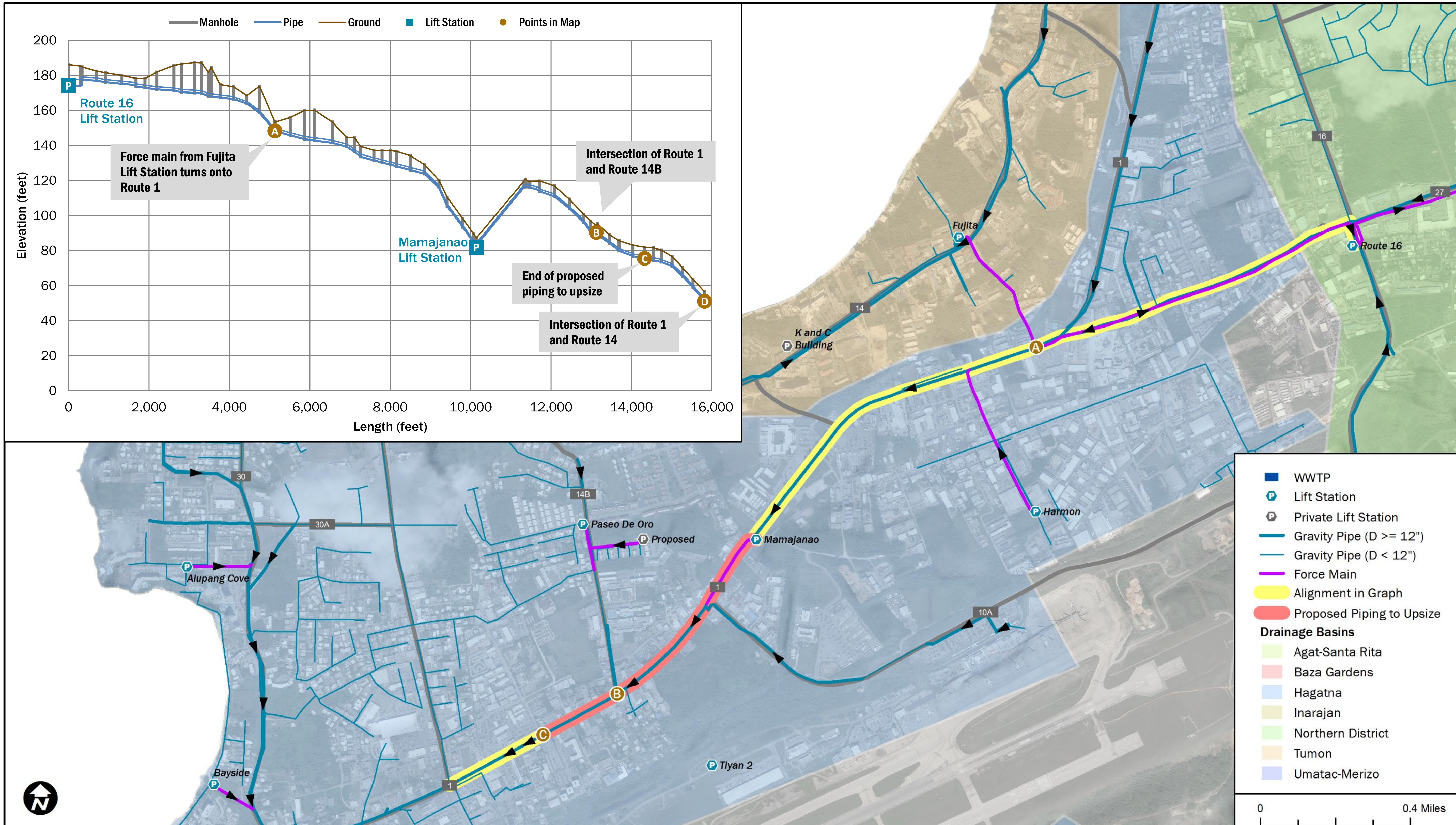

Figure 4-7. Profile of Piping to/from Mamajanao Lift Station

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4.3 Connect Septic/Cesspool Customers to Collection System

As outlined in Volume 1, Section 3, GWA has established a goal to construct 5,000 feet of new piping per year to connect houses on septic and cesspool systems to the GWA collection system and treatment system. The program will initially be focused on systems located over the Northern Guam Lens Aquifer (NGLA) due to the potential vulnerability of the groundwater supply in this area. Volume 1, Section 5 discusses options and priorities for connecting septic/cesspool customers to the collection system.

The relative coverage of adding 5,000 feet of piping per year into unsewered areas was evaluated by looking at areas in Northern Guam. Figure 4-8 illustrates a scenario where 100,000 feet of piping (5,000 feet of per year for 20 years) is added to the collection system. As a goal that was established for GWA in the 2006 WRMP and re-emphasized in the Levels of Service established for this WRMPU, it is recommended that GWA begin a project to connect customers with septic/cesspool systems to the GWA collection system. Section 4.7.3 discusses the recommended project.

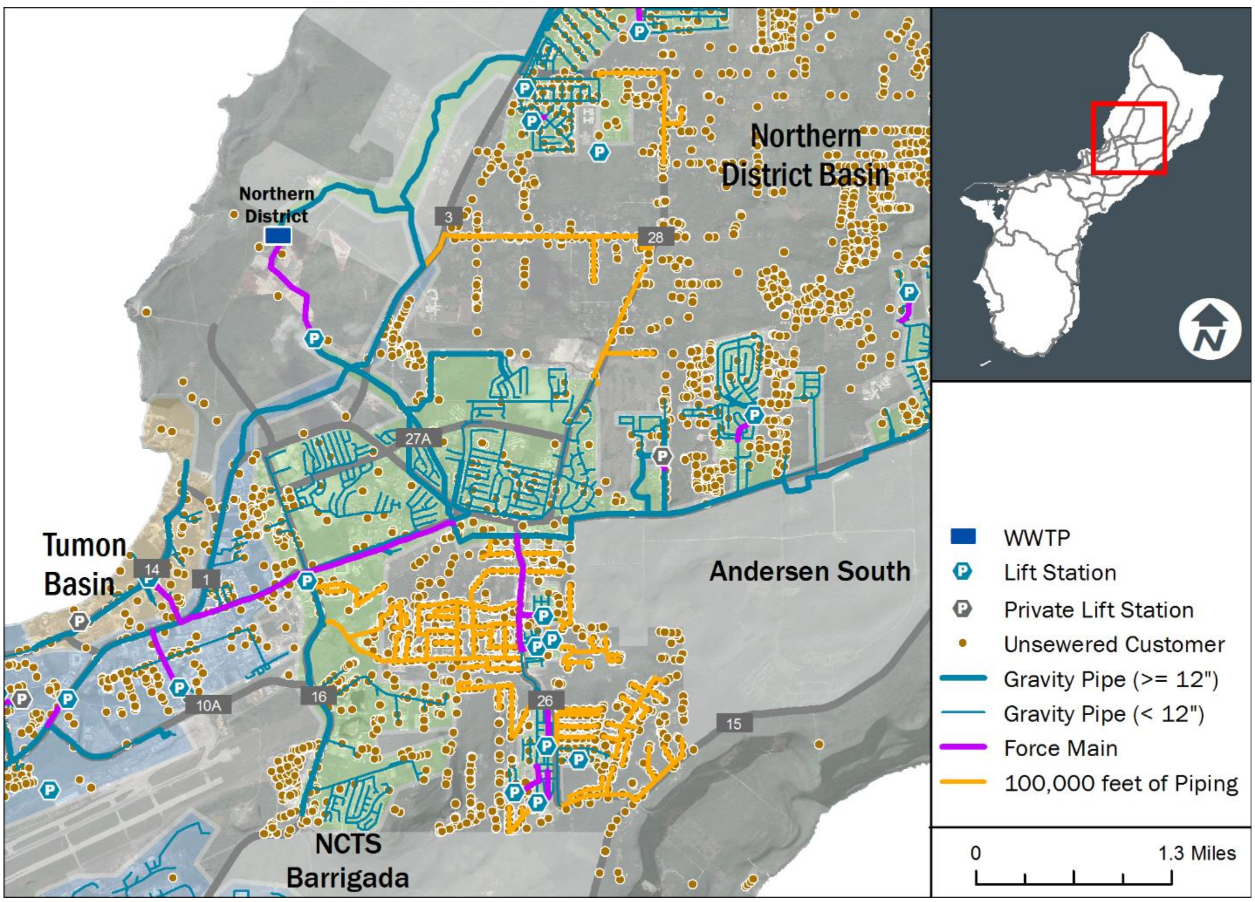

Figure 4-8. Sample Area Showing 100,000 Feet of Piping (5,000 Feet Per Year for 20 Years)

4.4 Gravity Piping Condition Assessment and Ranking

This section describes a risk-based approach used to prioritize the renewal (rehabilitation or replacement) of GWA's gravity piping. Similar to the water distribution system analysis outlined in Volume 2, Section 8, the wastewater system gravity pipeline renewal analysis described in this section included the following two steps:

1. Calculate total renewal needs per year.
2. Identify which pipes need to be renewed per year using a risk-based approach.

A 20-year planning horizon was used to calculate pipeline renewal needs. As part of the analysis, a renewal needs model was run for 65 years. A renewal needs model is typically run for a long period to observe how the model reacts in later years. The renewal needs model results were then put into the context of the 20-year planning horizon.

4.4.1 Calculation of Total Renewal Needs per Year

The first step in prioritizing the renewal of gravity piping was to calculate total renewal needs per year using a long-term outlook. This step is described below.

4.4.1.1 Installed Pipeline Inventory

Existing pipeline data from GWA's GIS was used as an input to the renewal analysis because age and material of existing piping significantly impacts future replacement needs. Table 4-2 lists the length of piping by material and decade installed. As a comparison, Table 2-3 lists the length of piping by basin and diameter.

Table 4-2. Length of Gravity Piping Installed by Decade

Years	Length of Piping (miles) [a]									Percent of Total
	Asbestos Cement	Cast-In-Place Concrete	Cast Iron	Polyethylene	PVC	Reinforced Concrete	Vitrified Clay	Unknown	Total	
1965–1969	22.2	-	0.9	-	0.6	-	-	5.0	*28.8*	*10%*
1970–1979	52.7	0.5	0.9	0.7	6.3	0.9	-	19.4	*81.3*	*28%*
1980–1989	12.9	1.0	1.7	1.4	43.7	0.8	0.7	47.5	*109.6*	*38%*
1990–1999	0.2	-	-	0.8	39.9	-	-	7.4	*48.6*	*17%*
2000–2009	0.2	-	-	0.1	2.0	-	-	1.3	*3.6*	*1%*
Unknown	-	-	-	-	-	-	-	18.1	*18.1*	*6%*
Total	**88.2**	**1.4**	**3.5**	**3.0**	**92.6**	**1.6**	**0.7**	**98.6**	**289.9**	**100%**
Percent of Length	**30%**	**<1%**	**1%**	**1%**	**32%**	**1%**	**<1%**	**34%**	**100%**	

a. Ductile iron (700 feet), polymer concrete (238 feet), and terracotta (261 feet) pipe are not listed in the table but are included in the total length of piping.

An assumed year was used for piping without an installation date in the GIS. The median installation date for all materials is approximately 1980, and because a large amount of piping was installed in the 1980s due to the island's high growth period, piping missing an installation date was assumed to be installed in 1980.

4.4.1.2 Service Life Curve Development

The renewal modeling calculations used estimated pipe service life values to develop service life curves, indicating how pipe assets will "survive" over time. The curves are similar to a human life expectancy curve with the majority of people surviving to middle age, some infant mortality, and the rest living to an old age. The curves were developed using a three-point method with the following three points:

1. The first point is the year at which 90 percent of the pipes within that group are expected to remain in service before they completely fail.
2. The second point is the year at which 50 percent of the pipes in that pipe category are expected to remain in service and the other 50 percent fail.
3. The third point is the year at which only 10 percent of the pipes remain in service.

A Hertz distribution function was used to randomly select pipe segments of each material type to model the failure of the complete set of pipes of each material type based on the length of time they have been in the ground. Through this process, the real-world random distribution of sewer line failure was estimated.

To develop the service life values for GWA, information was used from other utilities and the latest American Water Works Association (AWWA) guidance regarding water pipe service life (AWWA, 2012). Generally, a wastewater pipe made of the same material as a water pipe has a slightly shorter service life due to the more corrosive and abrasive wastewater flow. Table 4-3 lists the pipe service life values used in the analysis.

Table 4-3. Pipe Service Life Values

Material Description	Pipe Age at % of Service Life Remaining			AWWA Service Life (years)	Notes
	90%	50%	10%		
Asbestos Cement	40	60	80	80	
Cast Iron	55	70	85	120	
Other	55	70	85		
PVC	40	65	90	100	PVC water line manufacturers have stated that PVC pipe generally has a 100-year life span, but many systems are experiencing issues with PVC piping earlier than 100 years.
Unknown	50	65	80		

4.4.1.3 Long-Term Renewal Needs Results

The renewal needs model generated a year-by-year quantity of piping by material type that should be targeted for replacement between 2015 and 2080. Table 4-4 summarizes the length of piping to renew by decade. Figure 4-9 shows a graph of the same information.

Table 4-4. Length of Piping to Renew by Decade								
Years	Length of Piping (miles)							Percent of Total System to Replace per Year
	Asbestos Cement	Cast Iron	PVC	Other	Unknown	Total	Miles to Replace per Year	
2015–2019	5.6	0.1	2.9	0.2	1.6	10.4	2.1	0.7%
2020–2029	17.4	0.3	12.3	0.8	8.0	38.9	3.9	1.3%
2030–2039	22.4	0.9	19.4	1.8	18.8	63.3	6.3	2.1%
2040–2049	19.1	1.6	22.3	2.3	28.0	73.3	7.3	2.5%
2050–2059	11.4	1.7	20.1	1.6	24.3	59.1	5.9	2.0%
2060–2069	5.3	1.2	14.5	0.8	12.1	33.8	3.4	1.1%
2070–2080	2.3	0.6	9.3	0.3	4.4	16.8	1.5	0.5%
Total to Renew (2015 through 2080)	**83.5**	**6.3**	**100.8**	**7.8**	**97.2**	**295.6**	**4.5 (average per year)**	**1.4% (average per year)**
Total in System (from Table 4-2)	**94.1**	**6.6**	**111.6**	**8.0**	**100.0**	**320.38**	**-**	**-**
Percent to Renew (Total to Renew / Total in System)	**89%**	**95%**	**90%**	**97%**	**97%**	**92%**	**-**	**-**

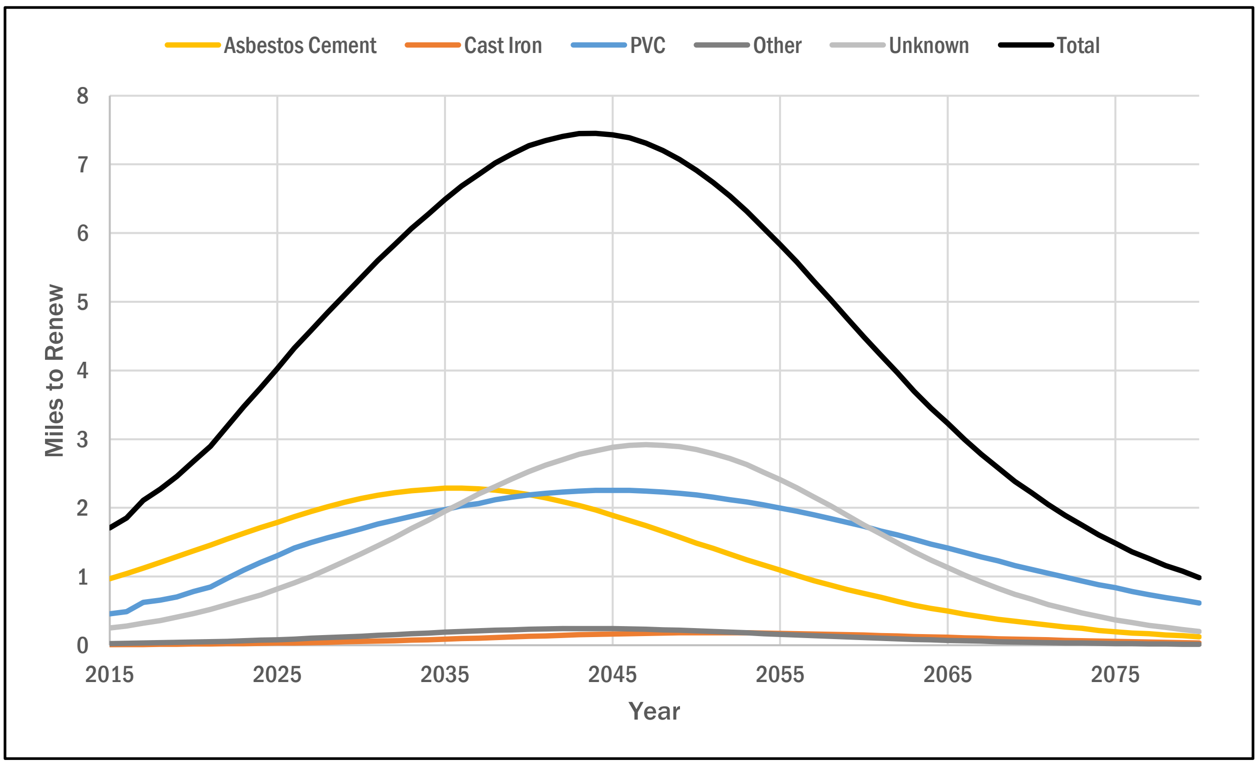

Figure 4-9. Pipeline Renewal Needs by Year

Table 4-4 shows that the projected renewal need over the next 65 years is an average of 4.5 miles per year, or about 1.4 percent of the total existing piping. This is slightly higher than the general rule of thumb within the industry of renewing a minimum of 1 percent per year. Renewal needs will vary by year, ranging from approximately 1.7 miles per year to 7.5 miles per year. The greater required number of miles to replace per year is due to the large amount of piping constructed in the 1980s and 1990s. The assumed year of construction of 1980, mentioned above, also impacts the calculations.

The goal of this analysis is to estimate the overall amount of piping that needs to be renewed to reach a steady state of pipe installation and retirement over time. In other words, this analysis estimated the amount of pipe that needs to be renewed on average every year. The analysis does not identify specific pipeline segments that may experience early failures and may need to be replaced before the end of their useful life. The analysis also does not include information regarding actual pipe condition, which is considered in the risk analysis detailed below.

4.4.2 Risk Calculations

The next step in calculating renewal needs was to estimate and prioritize which pipes require replacement each year using a risk-based methodology. Risk was calculated from the likelihood of failure and consequence of failure of each pipe. Each pipe was ranked to prioritize rehabilitation or replacement of the pipe compared to the piping in the rest of the system. The goal of this analysis was to identify areas of the system with the greatest potential impact in the event of a failure and focus asset management resources on the most critical assets to minimize risks of failure. The factors used to calculate likelihood and consequence of failure are discussed below.

Likelihood of Failure Risk Factors

Calculating likelihood of failure involves obtaining information about the pipeline's original design, material, installation, and operating parameters in conjunction with an assessment or estimate of its potential condition. Table 4-5 lists the likelihood of failure factors and Appendix D lists the scoring breakdown for each factor. Each factor was given a score ranging from 1 (good) to 5 (poor) and a weight (which allowed some factors to be given more importance than others). If CCTV condition data was available for a pipe segment, the score from the CCTV inspection was used as the entire likelihood of failure score. If a CCTV inspection score was not available for a pipe segment, the other factors listed in Table 4-5 were analyzed.

Table 4-5. Likelihood of Failure Factors

ID	Criteria	Factor Description	Process	Weight	
				If Condition Data Available for a Pipe	If Condition Data Not Available for a Pipe
P2	Soils	Ranked pipes for potential failure based on soil type or corrosivity of soil. Clay soils trap water, which can increase rate of corrosion. Pipelines within a clay based soil were ranked worst, within a loam or silty soil type were ranked medium, and within an urban area soil type were ranked best.	• Intersected pipes with soil data from the Natural Resources Conservation Services	-	0.3
P3	CCTV or other condition record data	Ranked sewer lines based on the condition determined by CCTV inspection (if available). CCTV defect scores were used to rank pipelines.	• Joined CCTV dataset to sewer pipes • Assigned score based on CCTV condition score	1.0	-

Table 4-5. Likelihood of Failure Factors					
ID	Criteria	Factor Description	Process	Weight	
				If Condition Data Available for a Pipe	If Condition Data Not Available for a Pipe
P4	Pipe installation or lining year	Ranked pipes based on installation or lining year. This factor is purely based on age with the assumption that older pipes are more likely to fail.	• Grouped pipes by the GIS install year field • Set pipes with unknown install year field to 1980 as discussed above	-	0.3
P5	Material	Ranked pipes for potential failure based on material. Different materials have different thicknesses and thus different expectations at which they will fail.	• Used pipe GIS material field	-	0.2
P11	Depth	Ranked pipes based on depth. Shallow pipes are more likely to fail due to vehicles passing over the pipe.	• The GIS depth field was used to determine pipe depth	-	0.2

Likelihood Factor P3 Condition Assessment Results

Condition scoring from CCTV data were used to calculate the likelihood of failure of each pipeline. The methodology for the calculations was adapted from the National Association of Sanitary Sewer Companies (NASSCO) Pipeline Assessment and Certification Program (PACP). Some of the data collected in the CCTV surveys was not collected in a PACP format due to issues that GWA had with the CCTV software. Those issues have since been fixed and future data will be PACP compliant. The CCTV data not in a PACP format was modified to fit a PACP format, as discussed in Appendix D.

Scoring was calculated using a method similar to the quick scoring method described in the PACP manual, which approximates the number of severe defects observed in a pipe (NASSCO version 6, 2016). The following steps were taken:

1. Each defect found along a pipe during the CCTV surveys was scored from 1 to 5 using PACP scores.
2. The highest scoring defect was found for each pipe.
3. The final score for each pipe was calculated as the highest scoring defect.

Only 14.5 miles of pipeline, or 6 percent of the system, had usable data available from this process. As additional CCTV data is collected, the likelihood of failure scores can be updated.

Consequence of Failure Risk Factors

Determining the consequence of failure involves assessing potential consequences if a pipe fails. Table 4-6 lists the consequence of failure factors and Appendix D lists the scoring breakdown for each factor. As with the likelihood of failure, each factor was given a score and weight.

Table 4-6. Consequence of Failure Factors				
ID	Criteria	Factor Description	Process	Weight
C1	Damage or disruption to sensitive locations	Pipes that could flood or disrupt priority facilities in the event of failure were given a higher consequence of failure. Priority facilities include hospitals, schools, police stations, fire stations, government buildings, and hotels.	• Data merged from multiple sources (including U.S. Geological Survey Place Names and Google Earth hotel locations) to develop sensitive locations list: Schools, Hospitals, Mayor's Office, Churches, and Hotels. • Distance calculated from sensitive locations to pipes.	0.25
C3	Damage or disruption to roadways	Pipes that will damage or flood important roads or highways in the event of failure were given a higher consequence of failure.	• Distance determined from pipes to major and minor streets.	0.2
C7	Service outage – number of customers	Ranked pipes based on the number of customers out of service due to a failure or flooding.	• Customer data was joined to the nearest sewer pipe and the number of customers was summed for each pipe.	0.15
C12	Flooding potential – flow	Quantified potential for economic damage and negative publicity in the event of pipe failure. This factor was used to estimate volume of water during a break.	• Used average flow from the hydraulic model for each pipe.	0.2
C16	Population density	Pipes serving areas with higher population densities will experience greater disruption in the event of failure and were given a higher consequence of failure.	• Calculated population density as persons per square mile using Guam Population by Municipality derived from U.S. Census Tracts 2010.	0.2

Risk Calculation

Scores were calculated for each pipe segment using the following steps:

1. Assign a score of 1 to 5 for each likelihood of failure factor to each pipe segment.
2. Calculate a total likelihood of failure factor for each pipe segment by summing the scores: $L1_{score}$ x $L1_{weight}$ + $L2_{score}$ x $L2_{weight}$ + … Ln_{score} x Ln_{weight}
3. Normalize all likelihood of failure scores so the scores range from 1 to 5. A higher score indicates a higher likelihood of failure.
4. Repeat steps 1 to 3 for consequence of failure.
5. Calculate total risk for each pipe segment: likelihood of failure score (1 to 5) x consequence of failure score (1 to 5).

4.4.3 Initial Ranking of Wastewater Lines for Inspection or Renewal

This section describes the overall results of the system-wide risk analysis. Table 4-7 summarizes the likelihood and consequence of failure score ranges. Likelihood of failure scores ranged from 1.0 to 5.0 and consequence of failure scores ranged from 1.0 to 4.7. Higher scores indicate a higher likelihood or consequence of failure.

Table 4-7. Failure Summary				
Score Range	Likelihood of Failure		Consequence of Failure	
	Miles	Percent of Total System	Miles	Percent of Total System
0–1	0.0	0.0%	0.0	0.0%
1–2	47.3	16.3%	46.5	16.0%
2–3	140.6	48.5%	178.4	61.6%
3–4	86.8	29.9%	58.3	20.1%
4–5	15.1	5.2%	6.6	2.3%
Total	**289.9**	**100%**	**289.9**	**100%**

Likelihood of failure and consequence of failure scores were broken into four categories: high priority, high likelihood, highly critical, and lower priority. These categories were established using a threshold score of 3 for likelihood and consequence of failure. Table 4-8 and Figure 4-10 summarize the results by risk category.

Table 4-8. Risk Summary			
Risk Category	Score Range	Miles	Percent of Total System
High Priority	Likelihood and consequence of failure are greater than or equal to 3.	32.5	11.2%
High Likelihood	Likelihood of failure is greater than or equal to 3 and consequence of failure is less than 3.	69.4	23.9%
Highly Critical	Likelihood of failure is less than 3 and consequence of failure is greater than or equal to 3.	32.5	11.2%
Lower Priority	Likelihood and consequence of failure are less than 3.	155.5	53.7%
Total		**289.9**	**100%**

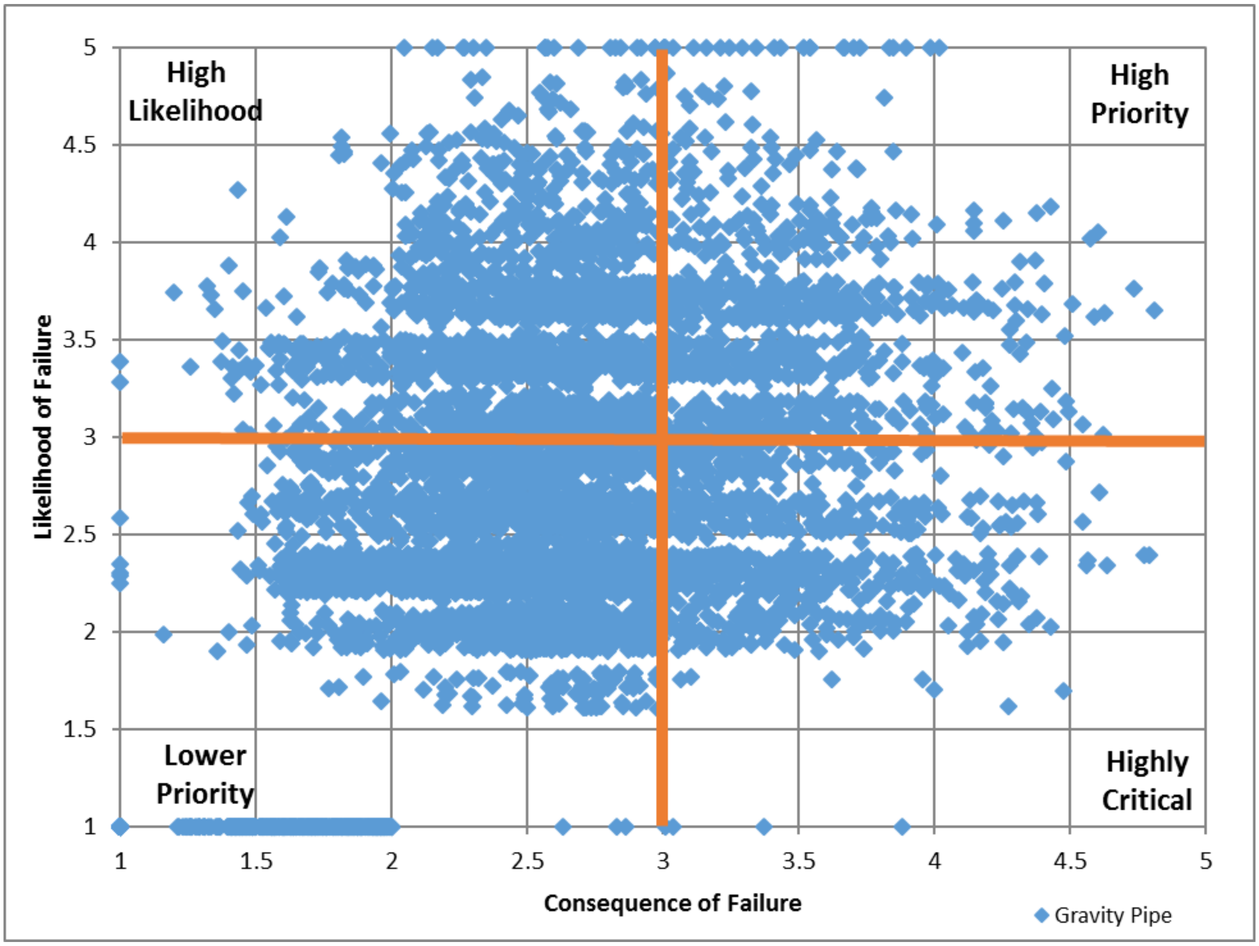

Figure 4-10. Likelihood and Consequence of Failure Results by Risk Category

Piping was grouped into the four categories to allow decisions to be made for renewal actions. Because high priority gravity pipe lines comprise approximately 11 percent of the total system, that piping could be renewed within 11 years at a 1 percent annual renewal rate. Figure 4-11 and Figure 4-12 provide system-wide maps showing the water lines color coded by the risk categories.

4.5 Pipeline Renewal Needs Analysis

The following section describes the rehabilitation or replacement needs of GWA's gravity pipelines. The rehabilitation and replacement needs were developed using the following approaches:

1. **Full replacement:** full replacement assumes that all piping within a project area will be replaced.
2. **Targeted rehabilitation and replacement:** targeted rehabilitation and replacement assumes that condition assessments will be performed on all pipes in a project. For a project, 20 percent of pipelines inspected by condition assessment will require rehabilitation (14 percent) or replacement (6 percent). These percentages are based on values observed by BC in similar projects.

The following assumptions were used to develop the cost estimates in this section:

- Costs for condition assessment work were based upon values observed by BC for similar projects on the mainland. An escalation factor was established by comparing costs for new pipeline replacement on the mainland to new pipeline costs on Guam used for this project (which are listed in Volume 1, Appendix D). This factor was applied to escalate condition assessment costs to expected costs on Guam.
- The cost estimates are for budgeting purposes only and may not represent the actual cost of conducting condition assessment, rehabilitation, and replacement activities in these areas. Unit costs for condition assessment, rehabilitation, replacement, and engineering costs are listed in Volume 1, Appendix D.
- All costs are in 2017 dollars.

4.5.1 Candidate Project Areas

Using the likelihood and consequence of failure results and risk categories, pipes were grouped into candidate project areas for condition assessment, rehabilitation, and replacement activities. Pipes from the high priority and high likelihood categories were grouped based on proximity to each other. Pipes from the lower ranking categories were included if located between higher priority pipes. Because there were scattered, individual pipes in the higher-ranking categories that were not close to other high priority pipes, these individual pipes were not included in projects at this time. These individual pipes should be considered for rehabilitation or replacement after the identified projects are completed.

Table 4-9 lists candidate project areas and pipes included within each area. Figures 4-13 and 4-16 illustrate the location of each proposed project area. These figures are presented later in this section and show the recommended capacity and condition projects together. Note that if targeted rehabilitation and replacement is used, all pipes within a project will have condition assessment performed, but only those that are found to be in poor condition will be rehabilitated or replaced.

Table 4-9. Candidate Projects for Wastewater Pipeline Rehabilitation and Replacement

Condition Project ID	Average Score Weighted by Length of Each Pipe		Percent of Project Length by Risk Category				Length of Pipe (miles)
	Likelihood	Consequence	High Priority	High Likelihood	Highly Critical	Lower Priority	
01	3.4	3.3	94%	6%	-	-	2.5
02	3.2	2.6	14%	72%	2%	12%	6.5
03	3.3	2.1	5%	76%	-	19%	5.8
04	3.2	3.1	66%	14%	8%	12%	3.1
05	3.1	2.1	3%	74%	-	24%	1.6
06	3.1	2.6	22%	52%	7%	19%	3.6
07	3.3	2.3	5%	66%	1%	29%	10.0
08	3.1	2.4	7%	58%	4%	31%	2.2
09	3.0	2.8	23%	34%	16%	27%	21.1
10	3.0	2.5	14%	33%	2%	51%	5.8
11	2.9	3.0	29%	30%	21%	20%	5.6
12	2.9	2.6	5%	50%	15%	30%	9.3

Table 4-9. Candidate Projects for Wastewater Pipeline Rehabilitation and Replacement							
Condition Project ID	Average Score Weighted by Length of Each Pipe		Percent of Project Length by Risk Category				Length of Pipe (miles)
	Likelihood	Consequence	High Priority	High Likelihood	Highly Critical	Lower Priority	
13	2.9	2.9	36%	17%	12%	35%	6.8
14	2.8	3.0	25%	27%	24%	24%	7.4
15	2.8	3.4	42%	10%	38%	10%	4.7
16	2.8	2.9	35%	7%	7%	51%	6.8
17	2.9	3.0	35%	17%	19%	28%	9.3
18	2.6	2.9	12%	13%	29%	46%	9.9
Total							**121.9**

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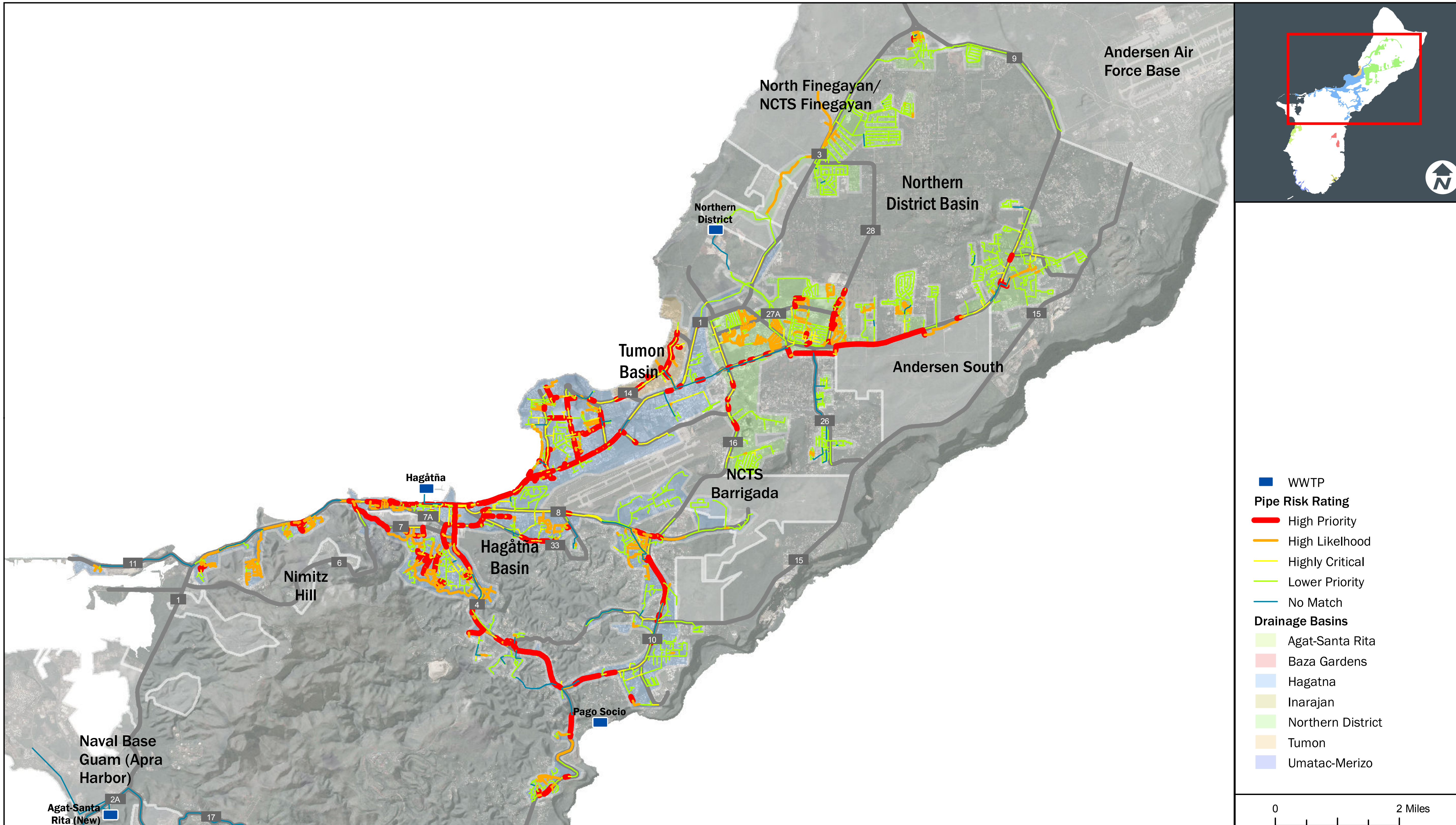

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Figure 4-11. Risk Category Summary (North)

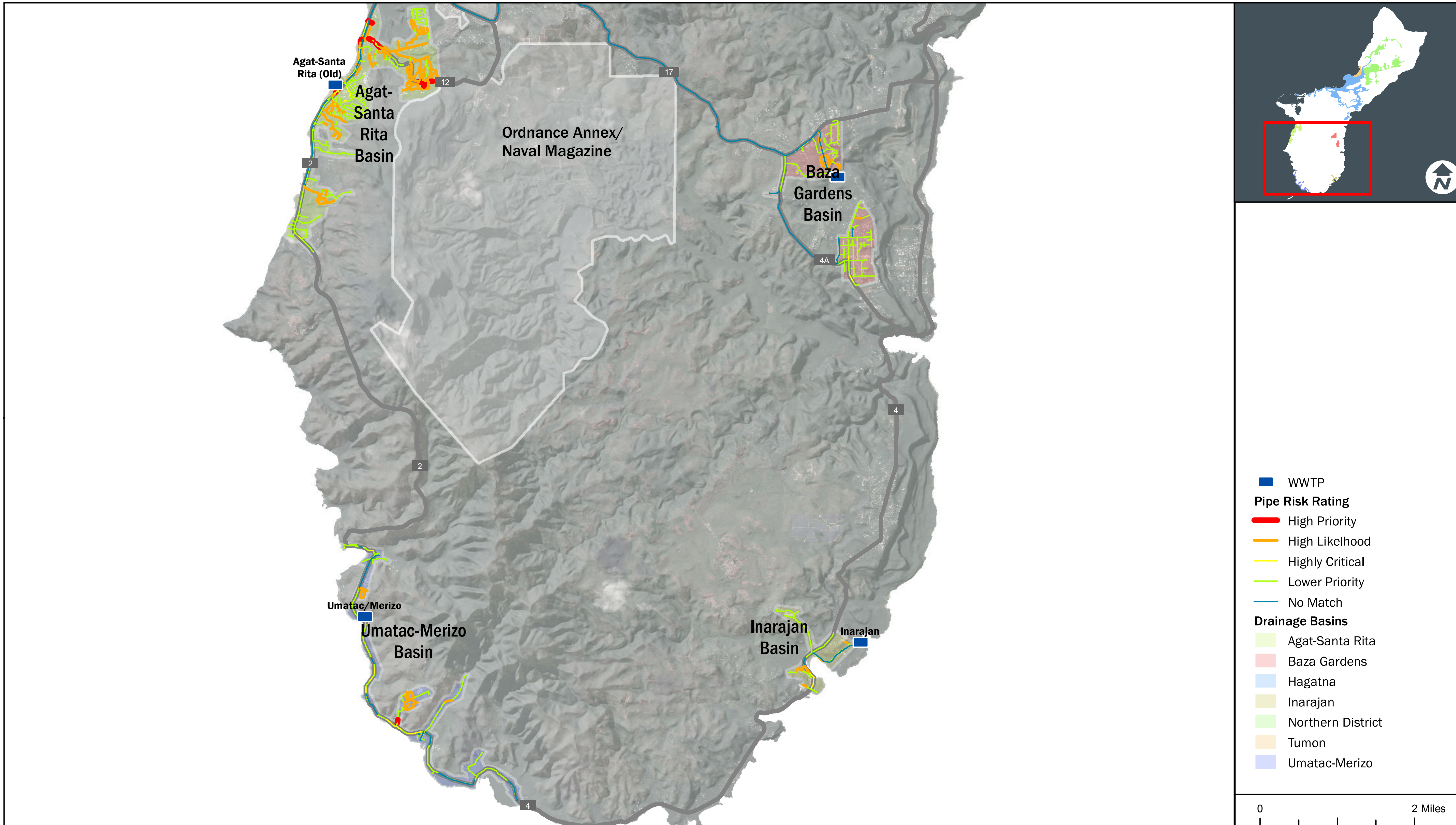

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Figure 4-12. Risk Category Summary (South)

4.5.2 Overall Renewal Recommendations

The following steps were used to develop renewal recommendations:

1. Calculate pipeline rehabilitation and replacement costs.
2. Identify scenarios for the planning timeframe using different amounts of sewer line replacement work based on long-term analysis results.
3. Develop proposed projects.

The following sections describe each step.

Step 1. Pipeline Rehabilitation and Replacement Costs

Table 4-10 lists total costs for pipeline rehabilitation and replacement by risk category. The table includes costs for full replacement and targeted rehabilitation and replacement. As discussed above, targeted replacement includes performing condition assessment on all pipes in a project and assumes that 20 percent of the pipes will require rehabilitation or replacement.

Table 4-10. Rehabilitation and Replacement Costs by Risk Category

Risk Category	Miles	Percent of Total System	Full Replacement (millions of dollars)	Targeted Rehabilitation and Replacement (millions of dollars)
High Priority	32.5	11.2%	$205.9	$34.8
High Likelihood	69.4	23.9%	$342.1	$50.6
Highly Critical	32.5	11.2%	$183.6	$30.3
Lower Priority	155.5	53.7%	$801.8	$119.8
Total	**289.9**	**100%**	**$1,533.5**	**$235.4**

Step 2. Gravity Pipeline Capital Needs Scenarios

Table 4-11 lists four scenarios that were developed which consider either full replacement or targeted rehabilitation and replacement.

The scenarios also vary based on assumed available funding per year. In GWA's current, 5-year capital improvement program (CIP) (2016–2020), the capital plan for gravity piping is included as CIP PW 09-06, Wastewater Collection System Replacement/Rehabilitation Program. The funding level for this project is $25,470,000 over three years, which equates to approximately $8.5 million per year.

Table 4-11. Gravity Pipeline Renewal Scenarios Cost and Timeframe Summary				
Number	Scenario	System Renewal Timeframe (Years)	Average Miles per Year	Average Annual Cost (millions of dollars)
1	Full replacement using average yearly renewal rate of 4.5 miles per year, as identified by the long-term analysis in Table 4-4 (289.9 miles total length/4.5 miles per year)	64 (full replacement of entire system)	4.5 (full replacement)	$23.9
2	Targeted rehabilitation and replacement using average yearly renewal rate of 4.5 miles per year, as identified by the long-term analysis in Table 4-4 (289.9 miles total length/4.5 miles per year). Assuming condition assessment finds that 20 percent of system needs rehabilitation or replacement, 22.5 miles would need to be inspected.	13 (targeted renewal for entire system)	4.5 (rehab or replacement) 22.5 (targeted replacement)	$18.3
3	Full replacement using current funding of $8.5 million per year ($1.53B from Table 4-10 for $8.5M per year)	180 (full replacement of entire system)	1.6 (full replacement)	$8.5
4	Targeted rehabilitation and replacement using current funding of $8.5 million per year ($235M from Table 4-10 for $8.5M per year). Assuming condition assessment finds that 20 percent of system needs rehabilitation or replacement, 2.1 miles would be rehabilitated or replaced.	28 (targeted renewal for entire system)	2.1 (rehab or replacement) 10.5 (targeted replacement)	$8.5

The first scenario almost triples GWA's current annual funding level by assuming that all pipes will be fully replaced. The second scenario uses targeted replacement and has an annual cost lower than the current funding level. The third scenario results in an unacceptable system renewal timeframe of 180 years. Finally, the fourth scenario is based on maintaining current funding levels and using a targeted rehabilitation and replacement approach. Based on a review of the four scenarios, the fourth scenario is recommended and was used for development of the proposed improvement project. Although scenario 2 meets the long-term recommendation of replacing 4.5 miles per year, scenario 4 is recommended so GWA can maintain the current funding of $8.5 million per year.

Step 3. Proposed Improvement Project

Table 4-12 lists planning level costs developed using unit cost assumptions for each candidate project area. The plan shown in the table was developed by applying the Scenario 4 assumptions of targeted rehabilitation and replacement (perform condition assessment on all pipes in a project and assume that 20 percent of the pipes will require rehabilitation or replacement) and annual funding of $8.5 million to the list of candidate projects presented in the previous sections. This plan optimizes use of GWA's resources by prioritizing the highest risk pipelines for renewal.

Table 4-12. Candidate Project Cost Estimates for Pipeline Rehabilitation and Replacement							
Condition Project ID	Length of Pipe (miles)	Cost				Annual Cost	Year
		Condition Assessment	Targeted Rehabilitation (Lining)	Targeted Replacement	Total		
01	2.5	$236,000	$2,692,000	$1,518,000	$4,446,000	$8,755,000	1
02	6.5	$604,000	$1,904,000	$1,801,000	$4,309,000		
03	5.8	$543,000	$1,927,000	$1,662,000	$4,132,000	$7,283,000	2
04	3.1	$293,000	$1,788,000	$1,070,000	$3,151,000		
05	1.6	$150,000	$420,000	$435,000	$1,005,000	$10,072,000	3
06	3.6	$336,000	$921,000	$972,000	$2,229,000		
07	10.0	$928,000	$3,088,000	$2,822,000	$6,838,000		
08	2.2	$204,000	$715,000	$622,000	$1,541,000	$8,800,500	4, 5
09	21.1	$1,960,000	$7,727,000	$6,373,000	$16,060,000		
10	5.8	$536,000	$2,227,000	$1,710,000	$4,473,000	$9,836,000	6
11	5.6	$520,000	$2,745,000	$2,098,000	$5,363,000		
12	9.3	$865,000	$2,591,000	$2,547,000	$6,003,000	$6,003,000	7
13	6.8	$632,000	$3,912,000	$2,740,000	$7,284,000	$7,284,000	8
14	7.4	$684,000	$2,735,000	$2,174,000	$5,593,000	$9,573,000	9
15	4.7	$441,000	$2,079,000	$1,460,000	$3,980,000		
16	6.8	$629,000	$3,516,000	$2,332,000	$6,477,000	$6,477,000	10
17	9.3	$862,000	$3,215,000	$2,677,000	$6,754,000	$6,754,000	11
18	9.9	$918,000	$3,585,000	$2,919,000	$7,422,000	$7,422,000	12
Total	**121.9**	**$11,341,000**	**$47,787,000**	**$37,932,000**	**$97,060,000**		

Following completion of the proposed projects, GWA should reassess the risk profile of the gravity piping system based on the condition assessment findings and rehabilitation and replacement performed.

4.6 Fats, Oils, and Grease

Fats, oils, and grease (FOG) are introduced into the sewer systems through food preparation, cooking, and clean-up. FOG solidifies in the sewer and hardens on the walls of pipes, thereby retarding or blocking flow. It also clings to components in pump stations and fouls machinery at the wastewater treatment plant. Buildup of FOG in the sewer lines and at treatment plants is the leading cause of sewer blockage and SSOs within GWA's wastewater collection system. FOG causes sewage overflows that pose a hazard to public health and to the environment.

4.6.1 Current Program Development

GWA is currently in the process of implementing a FOG control program. The FOG program is being developed to reduce the introduction of FOG into the sewer system and combat FOG problems throughout GWA's system. The program will consist of new regulations, public education, inspections, and enforcement. The FOG Program report is scheduled for completion in December 2017.

GWA has tracked and submitted quarterly SSO reports to USEPA since 2011. On average, FOG causes over 50% of SSOs on Guam. Correcting FOG-related problems is costly and can be avoided through proper management before problems arise.

FOG control is required by the National Pretreatment Program (40 Code of Federal Regulations [CFR] 403) and National Pollutant Discharge Elimination System (NPDES) Permit Program. Under the provisions of these regulations, GWA is responsible for implementing controls and educating the community to prevent FOG discharge from causing interference within GWA's collection system and WWTPs.

The objectives of the GWA FOG Program are as follows:

- Comply with the Clean Water Act (CWA)
- Minimize the amount of FOG in the sewer system
- Eliminate SSOs
- Improve the functionality of the wastewater collection and treatment facilities
- Reduce the maintenance costs of the sewer system
- Provide for effective FOG control devices and maintenance
- Educate commercial and residential customers about FOG abatement and best management practices (BMPs)

4.6.2 FOG Treatment and disposal

The implementation of the FOG program will improve the collection system operations by reducing the quantity of FOG that enters the system. The installation of improved grease traps, local collection systems, public education, and other FOG reduction measures included in the plan will increase the quantity of FOG collected by waste haulers and that requires proper disposal. Adequate and effective disposal systems are necessary for the FOG program to be complete.

There are currently limited FOG treatment facilities on Guam and none within the GWA treatment system. FOG is typically treated in anaerobic digesters as part of a WWTP sludge treatment system. However, because none of GWA's WWTPs include anaerobic digestion in the treatment process, an alternate disposal system is required.

GWA should embark on a follow-up study to evaluate the increased quantity of FOG that will be collected by haulers and received at local collection stations. The collected FOG will require treatment and disposal. Based on the projected quantities of FOG collected, the report will review current disposal options at private facilities and evaluate the potential to include FOG treatment at one of the GWA WWTPs.

4.7 Recommendations

Recommendations for GWA's gravity piping system are summarized in the following section.

4.7.1 CCTV Recommendations

The following actions are recommended for CCTV:

- As noted above, CCTV data and reports were stored on multiple computer and server locations. Some videos and reports were saved in more than one location and therefore duplicated copies were created. CCTV data should be organized and stored in a single centralized location on a GWA server. Volume 1, Section 8.2 discusses recommendations for organizing CCTV data.
- GWA will continue to collect valuable information during CCTV and manhole inspections. This risk analysis should be updated periodically (every five years at a minimum) using the latest data.

The analysis will include an update to risk scores, re-prioritization of the piping to be renewed, and an update to the GWA CIP. For example, piping planned for renewal may be superseded as more critical pipes needing renewal are identified.

4.7.2 Manhole Recommendations

The following actions are recommended for manholes:

- GWA should implement a manhole rehabilitation program. Results and issues from the MACP manhole inspections should be compiled.
- Issues identified during the 2015 manhole inspections described in Section 4.1 should also be combined with new issues as they are found.
- GWA (or a contractor) should fix issues as they are found, including raising manholes, cutting down brush, maintaining easements, and rehabilitating or replacing manholes. GWA should fix minor manhole issues as they are found.
- Major manhole issues should be grouped into projects and put out to bid to be fixed by a qualified contractor, as recommended in project MP-WW-MH-01 in Section 11.

4.7.3 Septic/Cesspool System Recommendations

The following actions are recommended for reducing customers with septic/cesspool systems:

- Implement actions for connecting houses on septic/cesspool systems to the collection system, as recommended in Section 4.3. The recommended project is summarized in Section 11 as Project MP-WW-Pipe-27.

4.7.4 Piping Improvement Projects

Figures 4-13 through 4-16 show the recommended improvements for collection system piping. The figures show the locations of recommended capacity and condition projects to show how the projects correlate. Mapping for each project is shown in the corresponding project sheet in Section 11. Table 4-13 summarizes the length of piping that would be upsized for each project. Upsized piping was sized to be the minimum pipe size needed to convey peak flows with the same slope as the existing piping. Pipe sizes should be refined during design by looking at the size of neighboring pipes, utility conflicts, etc.

Table 4-13 lists candidate projects for pipeline rehabilitation and replacement that were identified during the risk analysis. The overall project recommendation is summarized in Section 11, Project MP-WW-Pipe-01. The table also lists a priority ranking for each project. The following criteria was used to rank the projects:

- Capacity projects with existing deficiencies were ranked higher than those with only future deficiencies.
- Improvement projects with larger diameter piping were ranked higher.
- Improvement projects with piping near reported SSOs were ranked higher.
- Piping with overlapping capacity and condition improvements were ranked higher.
- Lower ranked improvement projects that are immediately downstream of higher ranked projects were given a higher rank to ensure projects take place in a downstream to upstream order.
- Several pipe rehabilitation projects are currently in progress in Baza Gardens, Agat, and Santa Rita. Several additional projects are in the design stage that will cover sections of piping in Agat, Asan, Hagåtña, and Dededo. These projects are not listed in Table 4-13.

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Table 4-13. Recommended Piping Projects											
Basin	Project Number	Project Name	Description	Original Diameters in inches [a]	Recommended Diameters in inches (length in feet) [a]	Total Length (feet) [a]	Percent of Length of Project with Issue due to			SSO Reported Along Alignment	Rank
							Existing Peak Flow	Future Peak Flow	Condition		
Hagåtña	MP-WW-Pipe-02	Barrigada Pump Station Pipe Rehabilitation/ Replacement	• Replace gravity piping along Route 10 from the Mangilao force main to the Barrigada pump station. • Section 8 discusses I/I issues along this pipeline that should be investigated before replacing this pipeline. • The original pipeline liner is blistering and separating from the pipe. The pipeline will probably need to be replaced.	14, 18	14 (1,570), 18 (747), 21 (1,684), 24 (416)	4,417	62%	62%	100%	Yes	8
Hagåtña	MP-WW-Pipe-03	Route 1 Piti Pipe Rehabilitation/ Replacement	• Rehabilitate or replace gravity piping in Piti along Marine Corp Drive. The project will evaluate the use of CIPP options and open cut and replace construction to complete the necessary repairs. • The original liner on this pipeline is in poor condition. The crowns of the pipes are deteriorated and GWA is concerned that jetting the pipe for routine maintenance could cause a failure.	8, 10, 14	8 (1,083), 10 (1,834), 14 (1,757)	4,673	-	-	100%	No	22
Northern District	MP-WW-Pipe-04	Southern Link Pump Station Pipe Rehabilitation/ Replacement	• Replace gravity piping just upstream of the Southern Link Pump Station. • Piping in this section partially collapsed and was fixed as an emergency repair. The pipe needs to be repaired for long-term operation and to eliminate the possibility of another collapse in the future.	48	48 (224)	224	-	-	100%	No	15
Hagåtña	MP-WW-Pipe-05	Agana Heights Replacement	• Replace gravity piping in Agana Heights. • This pipe has failed in the past and there is no vehicular access to the pipe alignment. An alternate pipeline alignment should be evaluated.	8, 10	10 (566), 12 (1,754)	2,320	100%	100%	100%	No	14
Northern District	MP-WW-Pipe-06	Northern District Route 1 Capacity Replacement – Phase 1	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	8, 24, 27, 30	10 (1,027), 30 (303), 36 (5,227), 42 (730)	7,289	82%	100%	66%	No	3
Northern District	MP-WW-Pipe-07	Northern District Route 1 Capacity Replacement – Phase 2	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	24, 27, 30	30 (3,869), 36 (3,177)	7,046	14%	100%	100%	Yes	12
Northern District	MP-WW-Pipe-08	Northern District Route 1 Capacity Replacement – Phase 3	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	18, 24	30 (2,468), 36 (2,310), 42 (435)	5,213	64%	100%	-	No	13
Northern District	MP-WW-Pipe-09	North Dededo Capacity Replacement – Phase 1	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	8, 10, 12, 18	10 (4,356), 12 (1,099), 15 (342), 18 (2,014), 24 (1,346)	9,157	39%	100%	-	Yes	9
Northern District	MP-WW-Pipe-10	North Dededo Capacity Replacement – Phase 2	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity. Some of the piping may or may not need upsizing depending on where the projected development in Chamorro Land Trust Tract 10125 discharges (see Volume 1, Section 5.4.5 for a description of the tract).	8, 12	10 (2,651), 12 (1,119), 15 (1,370), 18 (1,793), 21 (303), 24 (293), 30 (122), 36 (1,896)	9,548	48%	100%	-	Yes	10
Northern District	MP-WW-Pipe-11	Route 16 Capacity Replacement	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	6, 8, 10	8 (700), 10 (2,765), 12 (1,541), 15 (2,737)	7,743	78%	100%	100%	Yes	4
Northern District	MP-WW-Pipe-12	Barrigada Capacity Replacement	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	8, 14	10 (261), 18 (320)	581	100%	100%	-	No	16
Northern District	MP-WW-Pipe-13	Mangilao Capacity Replacement	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	6, 8, 12	8 (564), 10 (404), 12 (576), 15 (688)	2,232	2%	100%	-	No	24
Northern District	MP-WW-Pipe-14	Dededo Capacity Replacement	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	8	10 (1,741), 12 (1,747)	3,489	100%	100%	32%	No	20
Northern District	MP-WW-Pipe-16	Yigo Capacity Replacement	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	8, 10, 12, 18, 24	10 (7,280), 12 (2,783), 15 (5,799), 18 (607), 21 (398), 30 (2,172), 36 (520)	19,559	53%	100%	-	Yes	11
Hagåtña	MP-WW-Pipe-17	Mamajanao Capacity Replacement	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	Gravity: 14, 16 Force main: 14	Gravity: 16 (109), 18 (199), 21 (1,259), 24 (1,285), 30 (146) Force main: 24 (1,186)	4,184	69%	97%	72%	Yes	1

Table 4-13. Recommended Piping Projects

Basin	Project Number	Project Name	Description	Original Diameters in inches [a]	Recommended Diameters in inches (length in feet) [a]	Total Length (feet) [a]	Percent of Length of Project with Issue due to			SSO Reported Along Alignment	Rank
							Existing Peak Flow	Future Peak Flow	Condition		
			• The Mamajanao lift station can only currently run one pump. If a second pump is turned on, the piping downstream surcharges, and because the line is shallow, a manhole lid pops up and an SSO occurs. With only one pump running, the lift station wet well can overflow during peak flows. • This project is not required if the Mamajanao lift station is redirected to Northern District WWTP.								
Agat-Santa Rita	MP-WW-Pipe-18	Agat-Santa Rita Capacity Replacement – Phase 1	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	8, 10, 12, 14, 20	10 (437), 12 (1,270), 18 (955), 30 (149)	2,811	78%	100%	-	No	5
Agat-Santa Rita	MP-WW-Pipe-19	Agat-Santa Rita Capacity Replacement – Phase 2	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	16, 18, 20	21 (838), 24 (1,952)	2,790	90%	100%	100%	No	6
Agat-Santa Rita	MP-WW-Pipe-20	Agat-Santa Rita Capacity Replacement – Phase 3	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	8, 10, 12, 14	10 (3,260), 12 (752), 15 (769), 18 (292), 21 (765)	5,839	86%	100%	100%	Yes	7
Baza Gardens	MP-WW-Pipe-21	Baza Gardens Capacity Replacement – Phase 1	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	8, 12, 15	10 (346), 12 (520), 15 (482), 18 (1,524), 21 (837)	3,709	91%	100%	-	No	17
Baza Gardens	MP-WW-Pipe-22	Baza Gardens Capacity Replacement – Phase 2	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	15	18 (2,287)	2,287	100%	100%	-	No	18
Baza Gardens	MP-WW-Pipe-23	Baza Gardens Capacity Replacement – Phase 3	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	8, 10	10 (617), 12 (315), 15 (1,681)	2,612	48%	100%	-	No	21
Umatac-Merizo	MP-WW-Pipe-24	Umatac-Merizo Capacity Replacement	• Replace existing gravity piping with new larger diameter piping. The hydraulic model identified the piping as having insufficient capacity.	8, 10, 12, 15	10 (310), 12 (758), 15 (1,558), 18 (60)	2,686	98%	100%	-	No	19
Hagåtña	MP-WW-Pipe-25	Piping Near Bayside Lift Station	• A study is planned to replace the Bayside lift station. • Complete an additional study on the piping draining into the lift station. The study should recommend replacing piping along the beach that flows to the Bayside lift station to another location away from the beach. Piping from the south side of Route 1 is too low and cannot drain into Route 1 and drains to Bayside. Study this piping to see if it can be connected to Route 1. Then implement the recommendations from the study. • See Pump Station Upgrades and Erosion Evaluation – Initial Findings (BC, 2014) for information on the issues at the site.	To be studied	To be studied	To be studied	-	-	-	No	2
Agat-Santa Rita	MP-WW-Pipe-26	Finile Drive Rehabilitation - Agat	• This piping has been identified by field investigation to be in very poor condition and requires rehabilitation or replacement	8, 10	8 (357), 10 (1,182)	1,538	-	-	100%	No	23

a. All pipe diameters and lengths are for gravity piping unless specified otherwise.

CIPP = cured-in-place pipe

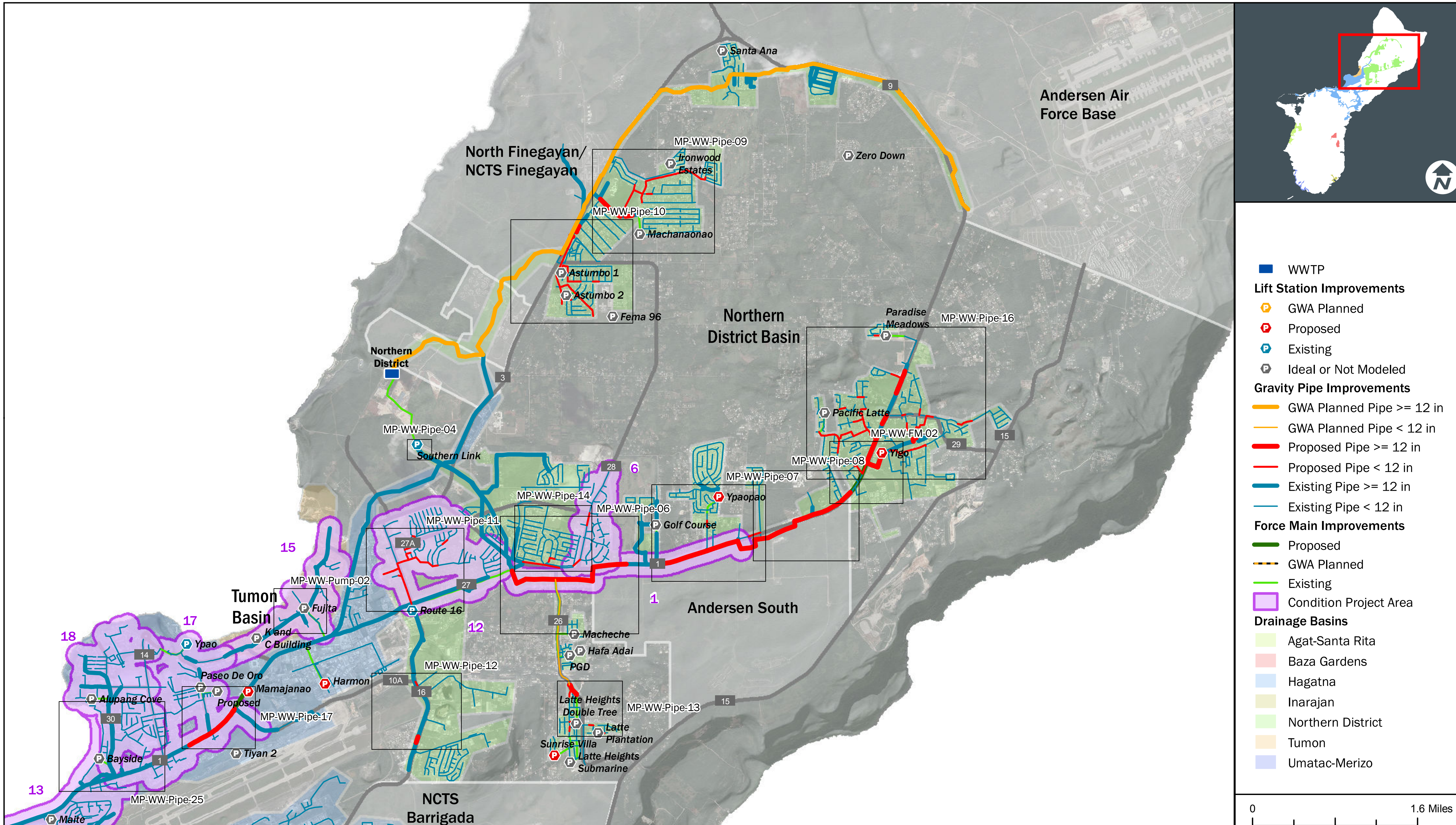

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Figure 4-13. Northern District and Tumon Basins Piping and Lift Station Improvements

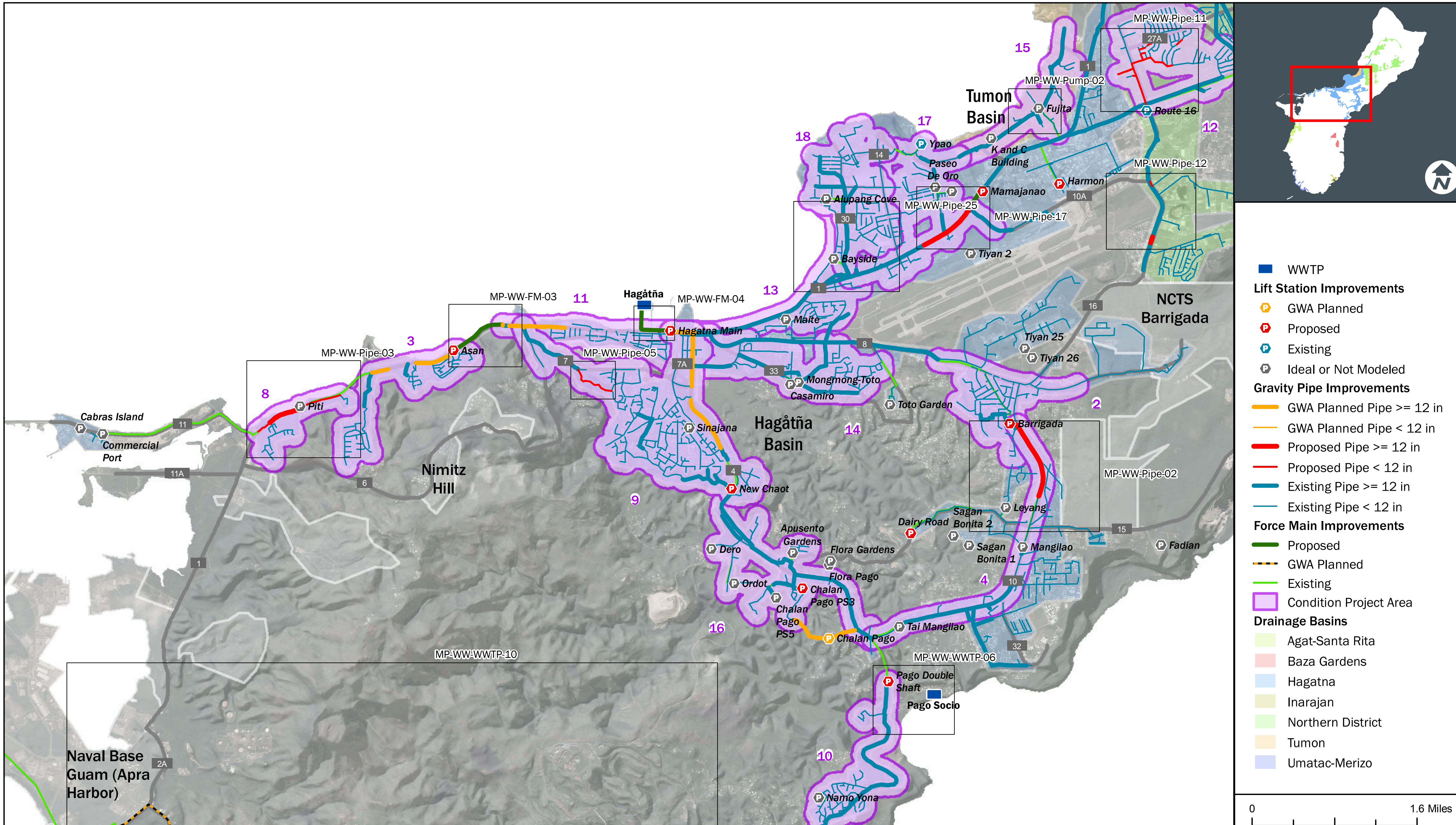

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Figure 4-14. Hagåtña Basin Piping and Lift Station Improvements

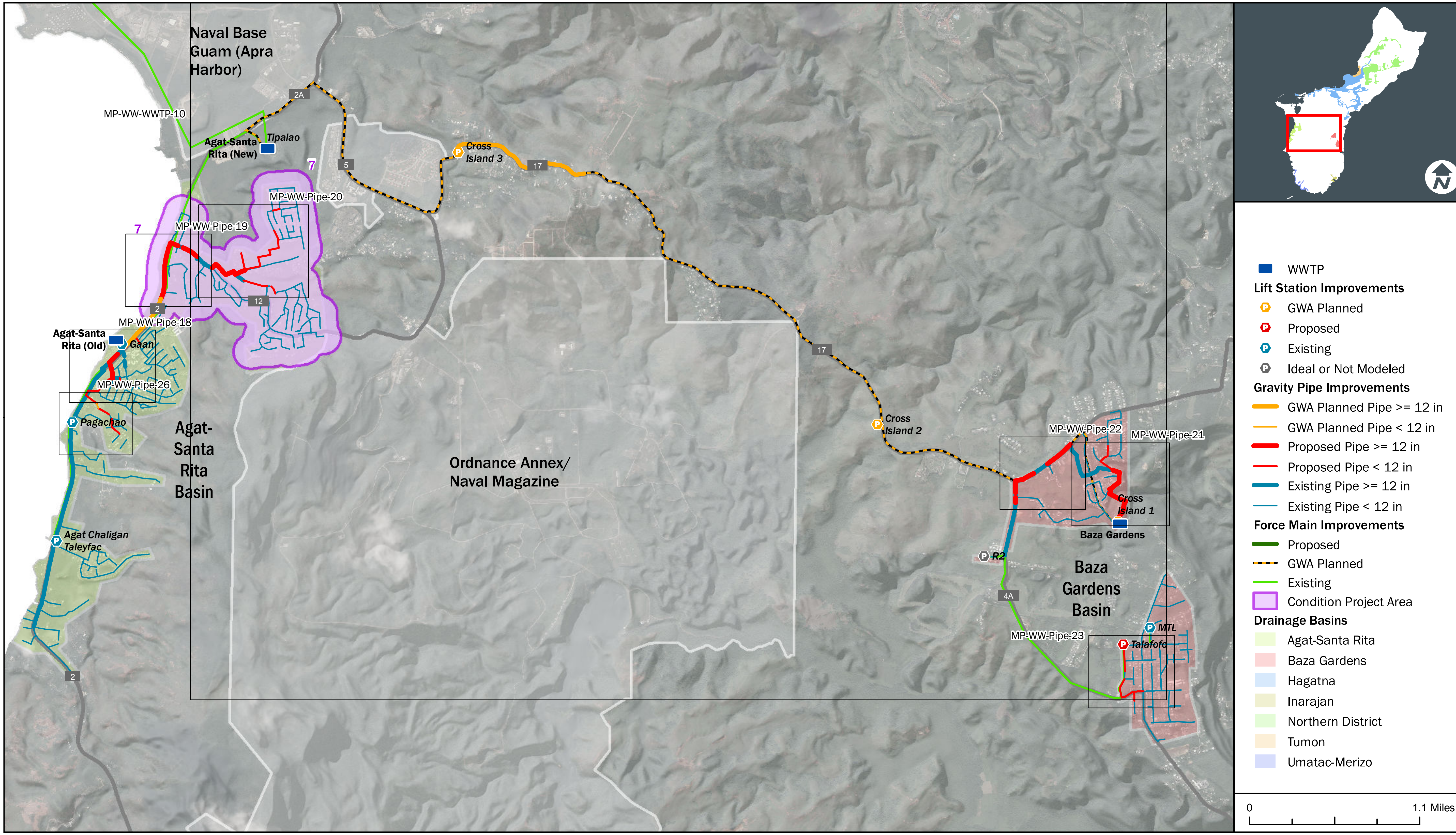

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Figure 4-15. Agat-Santa Rita and Baza Gardens Basins Piping and Lift Station Improvements

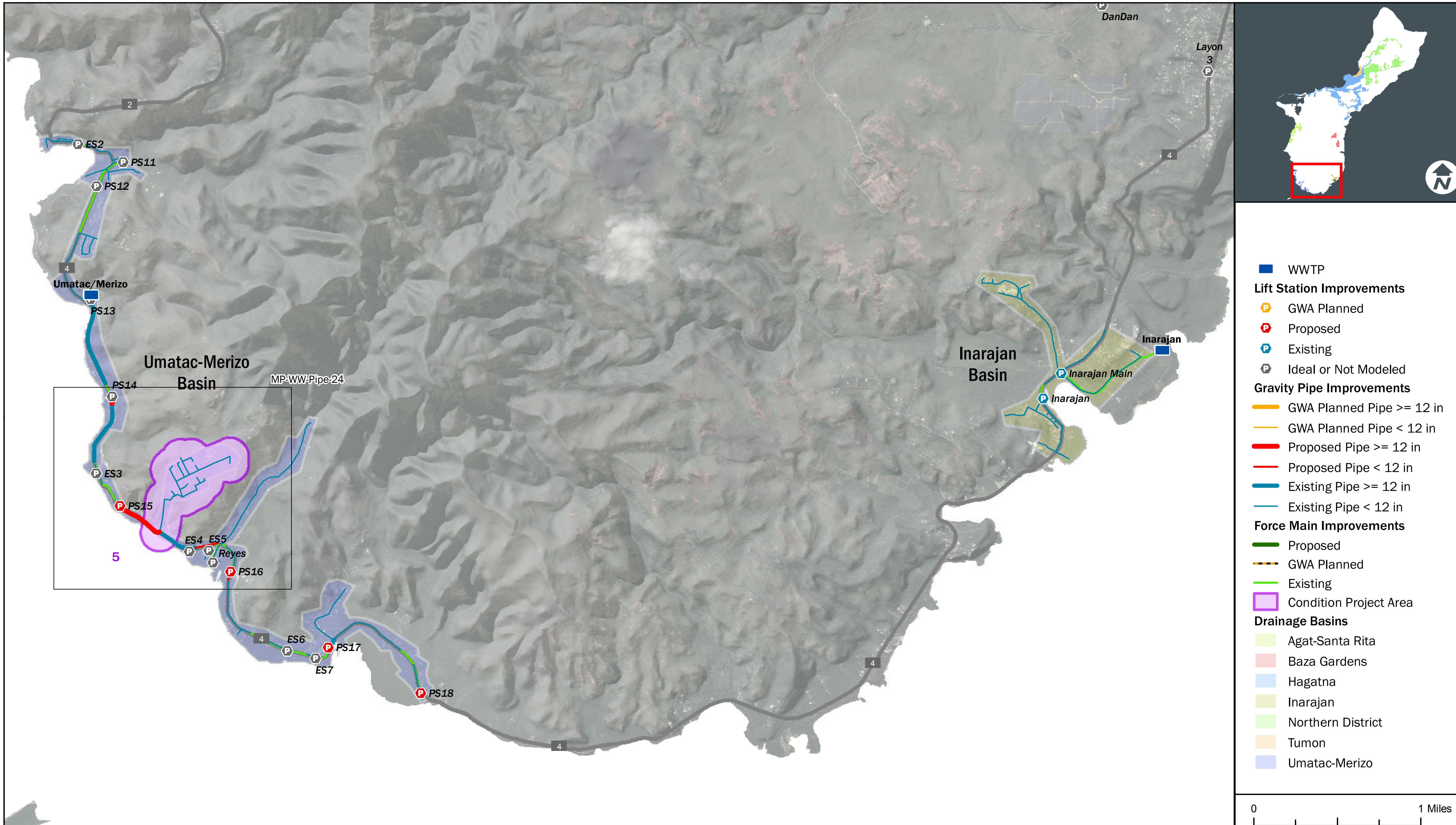

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Figure 4-16. Umatac-Merizo and Inarajan Basins Piping and Lift Station Improvements

Section 5

Force Main Evaluation

This section summarizes the capacity and condition of the force mains in GWA's wastewater system.

5.1 Force Main Capacity Evaluation

Force main piping capacity was evaluated under peak conditions for existing and future scenarios. Model results were reviewed to identify existing and future deficiencies. Capacity limitations at a lift station can be due to insufficient pump and/or force main capacity. Capacities of the pumps and force mains were analyzed simultaneously to identify if the pumps, force mains, or both the pumps and force mains need to be upsized.

Force mains were only analyzed for lift stations with sufficient information as listed in the last column in Table 2-5. Data was available for 19 of the lift stations listed in Table 2-5. The remaining force mains should be evaluated as additional data is gathered for the lift stations. In addition, four of the lift stations listed in Table 2-5 did not have dedicated force mains in the GIS piping (Ejector Station Nos. 3, 6, 7, and 13). These lift stations appear to pump into a common force main on Route 4.

5.1.1 Criteria

Capacity of the force main piping was analyzed and compared to existing and future flows using the following criteria (see Appendix C for additional details on the criteria):

- **Peak velocity:** pipes were considered to be at capacity at 10 feet per second.
- **Design storm:** a 2-year, 24-hour storm was used to evaluate the collection system, identify deficiencies, and develop improvements.
- **Lift station flow:** the lift stations were analyzed first and upsized in the model if necessary.
- **Force mains:** the force mains were analyzed with the lift stations pumping with the largest pump on standby.

5.1.2 Capacity Evaluation

Force main piping was evaluated using the criteria listed above. Table 5-1 lists the only force main identified as having insufficient capacity for existing or future peak flows. Figure 4-3 (in the gravity piping evaluation section) shows the location of the deficient force main. Improvement projects were developed to address deficiencies and are discussed at the end of this section.

Table 5-1. Force Mains with Insufficient Capacity

Source Lift Station	Basin	Diameter (inches)	Length (feet)	Notes
Yigo	Northern District	16	3,077	The force main is projected to have a capacity problem if the Yigo lift station is upgraded as discussed in Table 6-1.

As discussed in Section 4.2 for gravity piping, Figure 4-4 shows that a large amount of the collection system in the Hagåtña basin was predicted to have capacity problems. As described in Table 3-2, the Hagåtña basin did not calibrate well due to issues with the flow metering data. Therefore, even though Figure 4-4 shows issues with several force mains in the Hagåtña basin, the deficiencies are not listed in Table 5-1 and improvement projects were not developed for those force mains. Force mains in the Hagåtña basin that were identified with issues include force mains from the Asan, Barrigada, Hagåtña Main, and Mamajanao lift stations. The pipelines and their drainage areas need to be studied in more detail, as recommended in Section 9, before improvement projects can be developed.

5.2 Force Main Condition Assessment

Similar to gravity piping, a risk-based approach was used to prioritize force main piping renewal. This section describes the risk calculations and recommendations for force main piping renewal. Force mains that were analyzed include those with piping in the GIS.

5.2.1 Condition Assessment

No condition data was available for the force main piping. Interviews were held with GWA operations staff in February 2017 to discuss known issues with force mains. Table 5-2 lists issues reported by GWA operations staff. These issues were integrated into the risk calculations.

Table 5-2. Force Main Issues						
Force Main Lift Station	Basin	Diameter (inches)	Total Length (feet)	Material	Location of Force Main	Issue
Critical Issue						
Asan	Hagåtña	12	2,993	Cast Iron	Conveys flow along the coast on Route 1 in Asan.	The force main has been exposed due to erosion along the coastline. The pipe has required spot repairs in the past and a long-term solution is required.
Hagåtña Main	Hagåtña	24	2,724	Reinforced concrete	Conveys flow from Route 1 to the Hagåtña WWTP.	The pipe was previously repaired at a joint as emergency work and the overall condition of this section of pipe is questionable. This is the only line feeding the Hagåtña WWTP, and a failure at this location would be a significant problem.
Non-Critical Issue						
Fujita	Tumon	18	7,154	From GIS = ductile iron From GIS = cast iron	Conveys flow to the Route 16 lift station.	Rust has been observed on this force main at ARVs along Hamburger Road.
Pago Double Shaft	Hagåtña	8	2,474	Asbestos cement	Conveys flow from Chalan Pago-Ordot north along Route 4 until it discharges into a gravity pipe at the top of a hill.	There is no ARV at the top of the hill and recurring line breaks have occurred.
Pump Station No. 12	Umatac-Merizo	6	1,619	Unknown	Conveys flow south along Route 4 in Umatac.	This cast iron force main is corroded and has recurring breaks and pinholes.
Pump Station No. 17	Umatac-Merizo	6	2,840	From GIS = ductile iron From GIS = cast iron	Conveys flow along Route 4 in Merizo.	This force main is corroded and has recurring breaks and pinholes.

ARVs = air release valves

5.2.2 Risk Calculations

Table 5-3 lists the likelihood of failure factors and Table 5-4 lists the consequence of failure factors. The two force mains listed in Table 5-2 as having critical issues were not scored for likelihood of failure because they are recommended for immediate renewal later in Table 5-5.

Table 5-3. Likelihood of Failure Factors				
ID	Criteria	Factor Description	Score (see Appendix E for scoring breakdown)	Weight
L1	Age	Pipes with older installation dates are more likely to fail.	1 to 5	5
L2	Material	Some pipe materials are more likely to fail than others.	1 to 5	4
L3	Condition	Force mains with non-critical issues noted in Table 5-2 were given a score.	1 to 5	2

Table 5-4. Consequence of Failure Factors				
ID	Criteria	Factor Description	Score (see Appendix E for scoring breakdown)	Weight
C1	Diameter	Larger diameter pipes will have higher repair costs, may take longer to repair, and parts may be harder to obtain.	1 to 5	2
C2	Major roadways	Major roadways are ranked higher for flooding or repair disruption.	1 to 5	2
C3	Proximity to surface water	Flow from a failure is more likely to drain into a river or the ocean.	1 to 5	3
C4	Proximity to water well	Flow from a failure may pollute a potable water well.	1 to 5	3
C5	Serves important area	Pipes that serve the key economic areas of Tamuning (including Tumon) and Hagåtña will have a greater economic impact.	1 to 5	5
C6	Serves important facilities	Pipes that serve schools, hospitals, or the airport are ranked higher.	1 to 5	3
C7	Average flow	Pipes with higher flows will release more flow during a failure.	1 to 5	5

Scores were calculated for each force main using the following steps:

1. Assign a score of 1 to 5 for each likelihood of failure factor to each force main.
2. Calculate a total likelihood of failure factor for each force main by summing the scores: $L1_{score} \times L1_{weight} + L2_{score} \times L2_{weight} + \ldots Ln_{score} \times Ln_{weight}$
3. Normalize all likelihood of failure scores so the scores range from 1 to 5.
4. Repeat steps 1 to 3 for consequence of failure.
5. Calculate the total risk for each force main: likelihood of failure score (1 to 5) x consequence of failure score (1 to 5).
6. Normalize all risk scores so the highest score is 100.

Table 5-5 lists the force main piping prioritization for renewal based on the risk calculations. The pipes were sorted into the same four priorities used for gravity pipes:

1. High Priority – Likelihood of failure >= 3, Consequence of failure >= 3
2. High Likelihood – Likelihood of failure >= 3, Consequence of failure < 3
3. Highly Critical – Likelihood of failure < 3, Consequence of failure >= 3
4. Lower Priority – Likelihood of failure < 3, Consequence of failure < 3

Table 5-5. Force Main Renewal Prioritization

Force Main Lift Station	Basin	Diameter (inches)	Length (feet)	Material [a]	Installation Year	Failure Score (1 to 5)		Risk (1 to 100)	Full Replacement (1,000s of dollars) [b]	Targeted Rehabilitation/ Replacement (1,000s of dollars)
						Likelihood	Consequence			
Known Poor Condition										
Hagåtña Main	Hagåtña	24	2,724	Reinforced concrete	1965	Known issue	4.9		$7,399 [c]	$449
Asan	Hagåtña	12	2,993	Cast iron	1971	Known issue	2.8		$2,327	$347
High Priority (Likelihood >= 3, Consequence >= 3)										
Bayside	Hagåtña	6	646	ACP	1966	5.0	3.6	100	$411	$67
Pago Double Shaft	Hagåtña	8	2,474	ACP	1973	4.9	3.2	85	$1,682	$267
Mamajanao	Hagåtña	14	1,186	Unknown	1971	3.2	4.4	77	$925	$144
Barrigada	Hagåtña	14	6,078	ACP	1978	3.9	3.1	67	$4,742	$736
High Likelihood (Likelihood >= 3, Consequence < 3)										
Mangilao	Hagåtña	10	2,739	ACP	1974	4.5	2.8	68	$1,989	$301
Piti	Hagåtña	9.1	4,336	ACP	1971	4.5	2.6	64	$3,148	$476
Tai Mangilao	Hagåtña	8	1,618	ACP	Unknown	3.4	2.7	51	$1,100	$174
Pump Station No. 17	Umatac-Merizo	6	2,840	Ductile iron	1980	3.9	2.3	50	$1,807	$295
Paseo De Oro	Hagåtña	6	686	ACP	1967	5.0	1.8	49	$436	$71
Dairy Road	Hagåtña	6	3,616	Ductile iron	1983	3.1	2.5	42	$2,301	$376
Pump Station No. 16	Umatac-Merizo	6	1,095	Ductile Iron	1980	3.1	2.5	42	$697	$114
Maite	Hagåtña	4	393	Unknown	1971	3.2	1.7	29	$250	$41
Harmon	Hagåtña	6	2,260	Unknown	1972	3.2	1.5	26	$1,438	$235
Highly Critical (Likelihood < 3, Consequence >= 3)										

Table 5-5. Force Main Renewal Prioritization

Force Main Lift Station	Basin	Diameter (inches)	Length (feet)	Material [a]	Installation Year	Failure Score (1 to 5)		Risk (1 to 100)	Full Replacement (1,000s of dollars) [b]	Targeted Rehabilitation/ Replacement (1,000s of dollars)
						Likelihood	Consequence			
Fujita	Tumon	18	7,154	Ductile iron	1992	3.0	3.7	62	$6,365	$982
Route 16	Northern District	30	5,741	Unknown	1989	2.1	5.0	59	$7,768	$1,126
Yigo	Northern District	16	3,077	Polyethylene	1973	2.8	3.5	54	$2,559	$394
Chaligan	Agat-Santa Rita	16	6,352	Ductile iron	1995	2.6	3.1	44	$5,282	$813
Ypao	Hagåtña	7.3	1,741	PVC	Unknown	1.7	3.9	37	$1,184	$188
Lower Priority (Likelihood < 3, Consequence < 3)										
Inarajan Main	Inarajan	8	3,893	Unknown	1984	2.7	2.9	42	$2,646	$419
Southern Link	Northern District	36	4,311	Ductile iron	1992	2.6	2.9	41	$6,999	$980
Inarajan	Inarajan	4	505	Unknown	1984	2.7	2.5	36	$321	$53
Commercial Port	Hagåtña	6	8,672	Cast Iron	2001	2.5	2.5	33	$5,517	$902
Pump Station No. 12	Umatac-Merizo	6	1,619	Unknown	Unknown	3.0	2.0	32	$1,030	$168
Pagachao	Agat-Santa Rita	4	27	Unknown	Unknown	2.1	2.6	30	$17	$3
Ejector Station No. 2	Umatac-Merizo	4	225	PVC	1980	2.2	2.5	30	$143	$23
Sinajana	Hagåtña	4	302	Cast iron	Unknown	3.0	1.8	30	$192	$31
Mongmong-Toto	Hagåtña	8	1,334	Polyethylene	1972	2.8	1.9	29	$907	$144
Toto Garden	Hagåtña	4	2,748	Unknown	1988	2.1	2.5	29	$1,748	$286
Pump Station No. 14	Umatac-Merizo	8	466	PVC	1980	2.2	2.2	28	$317	$50
Pump Station No. 15	Umatac-Merizo	8	1,687	PVC	1980	2.2	2.2	28	$1,147	$182
New Chaot	Hagåtña	20	2,319	PVC	1989	1.7	2.9	28	$2,510	$371

Table 5-5. Force Main Renewal Prioritization

Force Main Lift Station	Basin	Diameter (inches)	Length (feet)	Material [a]	Installation Year	Failure Score (1 to 5)		Risk (1 to 100)	Full Replacement (1,000s of dollars) [b]	Targeted Rehabilitation/ Replacement (1,000s of dollars)
						Likelihood	Consequence			
Pump Station No. 11	Umatac-Merizo	6	1,249	Unknown	Unknown	2.1	2.3	27	$795	$130
Reyes	Umatac-Merizo	4	703	Unknown	1994	2.1	2.3	27	$447	$73
Gaan	Agat-Santa Rita	16	10,125	PVC	1995	1.7	2.9	27	$8,420	$1,295
Alupang Cove	Hagåtña	6	905	PVC	1991	1.7	2.8	26	$576	$94
Pump Station No. 18	Umatac-Merizo	6	1,575	PVC	1980	2.2	2.1	26	$1,002	$164
Ypaopao	Northern District	8	989	Unknown	Unknown	2.1	2.0	23	$672	$107
Ejector Station No. 5	Umatac-Merizo	4	188	Unknown	1980	2.7	1.5	22	$120	$20
Sunrise Villa	Northern District	3	1,571	Unknown	1981	2.7	1.5	22	$1,000	$163
Talofofo	Baza Gardens	10	8,849	PVC	1994	1.7	2.2	20	$6,424	$971
Macheche	Northern District	6	825	Unknown	Unknown	2.1	1.7	20	$525	$86
Latte Heights Submarine	Northern District	8	1,283	Unknown	Unknown	2.1	1.6	19	$872	$138
Machanaonao	Northern District	6	987	Polyethylene	1992	1.7	2.0	19	$628	$103
Tipalao	Agat-Santa Rita	16	11,076	PVC	1995	1.7	2.0	19	$9,211	$1,417
PGD	Northern District	6	4,569	PVC	Unknown	1.7	2.0	18	$2,907	$475
Santa Ana	Northern District	8	189	Unknown	Unknown	2.1	1.4	17	$128	$20
Casamiro	Hagåtña	8	263	Unknown	Unknown	2.1	1.4	17	$179	$28
Latte Heights Double Tree	Northern District	12	1,424	Unknown	Unknown	2.1	1.4	17	$1,107	$165
Namo Yona	Hagåtña	8	317	Unknown	Unknown	2.1	1.4	17	$215	$34
Astumbo No. 1	Northern District	8	109	PVC	1993	1.7	1.8	17	$74	$12

Table 5-5. Force Main Renewal Prioritization										
Force Main Lift Station	Basin	Diameter (inches)	Length (feet)	Material [a]	Installation Year	Failure Score (1 to 5)		Risk (1 to 100)	Full Replacement (1,000s of dollars) [b]	Targeted Rehabilitation/ Replacement (1,000s of dollars)
						Likelihood	Consequence			
Latte Plantation	Northern District	4	115	PVC	1982	2.2	1.3	16	$73	$12
Pacific Latte	Northern District	4	894	PVC	1986	2.2	1.3	16	$569	$93
Ordot	Hagåtña	4	1,291	PVC	1994	1.7	1.7	16	$821	$134
Chalan Pago PS 3	Hagåtña	10	1,045	Polyethylene	1992	1.7	1.6	15	$759	$115
Astumbo No. 2	Northern District	8	376	PVC	1993	1.7	1.4	14	$256	$41
Chalan Pago PS 5	Hagåtña	8	904	Polyethylene	1992	1.7	1.4	14	$615	$97
Main Trunk Line	Baza Gardens	4	573	PVC	1996	1.7	1.3	12	$365	$60
Leyang	Hagåtña	8	548	PVC	2004	1.2	1.6	10	$373	$59
Total			140,799						$116,435	$17,314

a. ACP = asbestos cement pipe

b. The replacement costs assume replacement due to condition at the same diameter. The costs may differ in other sections where the force mains are recommended for upsizing due to capacity.

c. The replacement cost is based on replacing the existing 24-inch with a new 42-inch pipeline. See the project description for project MP-WW-MP-04 in Section 11 for more details.

Force mains could undergo full replacement or targeted rehabilitation and replacement as described for gravity pipes in Section 4.5. Targeted rehabilitation and replacement of 5 miles of force main per year would yield:

- System renewal timeframe = 11 years for the entire system for be inspected and 20% of the system to be lined or replaced
- Average annual cost = $1,575,000 = $17,314,000 (from Table 5-5) divided by 11 years

5.2.3 Force Main Inspection

The force mains listed in Table 5-5 require additional inspection before rehabilitation or replacement is performed.

The following steps summarize a protocol for force main inspections. The steps include updates to the methodology described in the 2006 WRMP.

1. **Conduct initial force main inspections:** in most cases, a comprehensive, direct inspection of the force mains (such as CCTV inspection or man entry in larger lines) would require putting the pipelines out of service. Exterior inspection of an entire pipeline would be impractical. To minimize excavation and system shut-down time, the following procedures are recommended for an initial force main condition inspection. Force mains found to be in the worst condition based on the initial inspections should be programmed for additional, more comprehensive interior and exterior pipe inspection. Initial inspection includes:
    a. **Perform reconnaissance/inspection of fittings:** this effort will provide a general condition assessment of the force main and most of its critical appurtenances without physically entering the pipe or exposing and potentially damaging buried sections of the force mains. The objective of this effort is to identify and catalog the type and location of each fitting and perform a visual condition assessment. Reconnaissance assessment survey procedures include the following:
        - Begin the assessment at the lift station.
        - Take a photograph of each fitting, assess if it is operable, estimate its condition (including the level of corrosion), take a global positioning system (GPS) location, and record the data on a field inspection form.
        - If visible, conduct a similar assessment of the exterior of the force main pipe.
        - Repeat the steps for each fitting.

    Various types of fittings may be found on the force main. Below is a description of some of these fittings.

    - **Air release valve:** air release valves (ARVs) manually or automatically vent trapped gases. Gases trapped at these locations increase the head against which the pump must operate, providing an opportunity for internal pipe corrosion and increasing the potential for high-pressure transients (water hammer) and cavitation in the pipeline. Trapped gases can also disrupt operation of the flow tubes. ARVs are typically located at the beginning of the force mains near the flow tubes and at intermediate high points where gas can accumulate.
    - **Air vacuum valve:** air vacuum valves (AVVs) are installed at high points in the force main to allow air to enter the system when it is draining. These valves will break a vacuum that can form in a force main and prevent the pipe from collapsing.
    - **Combination air valve:** combination air valves (CAVs) combine the function of an ARV and AVV into one unit.

- **Air bleeder:** air bleeders have the same function as an ARV except the valve is operated manually. Air bleeders may also be identified as manual ARVs.
- **Blow-off valve:** a blow-off valve is usually installed at low points in the force main system where debris can accumulate. This valve is used to drain wastewater and debris out of the force main. Debris trapped at these locations increases the head against which the pump must operate and provides an opportunity for corrosion at the invert of the pipe.
- **Gate valve:** a gate valve is usually installed on either side of a flow tube or ARV such that they can be isolated from wastewater flow.
- **Check valve:** a check valve is usually installed at the discharge end of each pump to provide a positive shutoff from force main pressure when the pump is not running. The valve also prevents the force main from draining back into the wet well when the pump is not running.
- **Cathodic protection systems:** cathodic protection systems are designed to protect metallic pipelines from galvanic corrosion.
- **Cathodic corrosion test site:** cathodic corrosion test sites are used to determine if the cathodic protection system is properly functioning.
- **Flow tube:** flow tubes are used to measure the flow rate in the force main with a Venturi meter mounted outside of the lift station.
- **Other items:** other items include pressure manholes and cleanouts. These items are installed to facilitate maintenance activities.

b. **Inspect force main discharge pipe and manhole:** the condition of the discharge end of the force main pipe and the condition of the discharge manhole itself will indicate the potential for interior corrosion in other areas of the force main. This information can be used in conjunction with results of liquid and gas sampling to identify the potential presence of hydrogen sulfide (H_2S) gas in the force main. If possible, a visual inspection of the discharge end of the force main pipe and the discharge manhole should be conducted. A manned entry inspection would likely require taking the upstream lift station offline. If this is not feasible, a surface-level visual inspection would still provide useful information. The inspection should try to identify/quantify the following:
   - Force main pipe material.
   - Force main discharge pipe corrosion condition.
   - Manhole corrosion condition (cover, rungs, walls, etc.).

c. **Conduct liquid sulfide sampling:** liquid sulfide sampling will quantify the presence and/or generation potential of sulfides and H_2S gas in the force main. High sulfide concentration in the force main increases the potential of sulfide-related corrosion in air pockets that may form at high points along the force main alignment. Wastewater grab samples should be collected at the lift station influent wet well and at the force main discharge point. Several samples should be collected at each location at various times of the day over a 2-day sample period. Samples should be collected while the force main is actively discharging and ideally at the beginning of the pumping cycle to catch flows that are likely to have the highest sulfide concentrations. Samples must be analyzed in the field within one minute of collection to minimize off-gassing of liquid sulfide to H_2S gas. Samples will be analyzed for total sulfide concentration using the LaMotte Pomeroy methylene blue titration technique. Wastewater temperature and pH measurements should be taken in conjunction with each grab sample collected for liquid sulfide analysis.

2. **Prioritize force mains needing additional inspection:** based on information gathered in Step 1, prioritize the force mains that need additional inspection.
3. **Select an appropriate inspection technology:** select an appropriate inspection technology for screening tests and subsequent condition assessment program. Selection criteria should be based on force main size, pipe material, availability of access points, type of data to be collected, technology availability in Guam, and cost considerations.
4. **Conduct screening tests:** conduct screening tests to obtain condition data that can be used to identify potential corrosion, leaks, air pockets, and damage. Perform internal/external testing of pipelines to obtain preliminary condition data that is used to identify areas of potential corrosion, leaks, air pockets, and damage (cracks). Areas that could be sites for external corrosion can be identified by soil testing or above-ground investigations such as electromagnetic conductivity surveys. Acoustic testing methods such as SmartBall (Pure Technologies), See Snake (PICA), or PipeDiver can be deployed for internal inspection.
5. **Assess pipe wall:** complete a more detailed condition assessment using one or more destructive and/or non-destructive testing methods to determine the actual condition of the force mains identified with potential problems in Step 4. Destructive testing techniques include collection of samples from the pipeline, and non-destructive pipe wall assessment techniques include acoustic, magnetic flux leakage, ultrasonic, remote field eddy current, etc.
6. **Prioritize future assessments and perform improvements:** use collected data to prioritize future condition assessments and conduct force main rehabilitation and replacement.

5.3 Recommendations

Table 5-6 summarizes recommended piping projects to address identified capacity and condition issues for force mains. Surge analysis was not performed for this project and should be performed as part of a force main design project.

Table 5-6. Force Main Piping Improvement Recommendations

Project Name	Project Number	Description	Reasons for Recommendation	Length of Piping		Drawing
				Diameter (inches)	Length (feet)	
Force Main Rehabilitation/ Replacement Program	MP-WW-FM-01	Implement an annual program to perform condition assessment and then rehabilitate and replace force main piping based on the results of the condition assessment. The force mains should be inspected according to the prioritization in Table 5-5. New piping should be sized to handle future planned peak wet weather flows.	The risk analysis conducted for the force mains, described in Section 5.2, shows that GWA must begin with a pipe renewal program to replace piping that will reach the end of its service life.	Varies	Varies	None
Replace Yigo Lift Station Force Main	MP-WW-FM-02	Replace the existing 16-inch force main from the Yigo lift station.	The force main is projected to have a capacity problem if the Yigo lift station is upgraded as discussed in Table 6-1.	Replace 16 with 30	3,077	See project sheet in Section 11

Table 5-6. Force Main Piping Improvement Recommendations						
Project Name	Project Number	Description	Reasons for Recommendation	Length of Piping		Drawing
				Diameter (inches)	Length (feet)	
Route 1 Asan Force Main Rehabilitation/ Replacement	MP-WW-FM-03	Replace the force main along Route 1.	The force main has been exposed in this location due to erosion along the coastline. The pipe has required spot repairs in the past and a long-term solution is required.	12	2,953	See project sheet in Section 11
Hagåtña WWTP Force Main Rehabilitation/ Replacement	MP-WW-FM-04	Study options for the repair or replacement of the force main between the Hagåtña Main pump station and the Hagåtña WWTP. Then replace sections, replace, or parallel the pipeline. The pipeline was originally constructed as a gravity pipeline, but was converted to a force main. The pipeline is too long to CCTV so a manhole may need to be constructed along the pipeline to complete condition assessment of the entire pipeline. Due to the complexity of this project, a new pipeline was not sized using the model and the existing 24-inch size was assumed for costing purposes.	The pipe was previously repaired at a joint as emergency work and the overall condition of this section of pipe is questionable. This is the only line feeding the Hagåtña WWTP, so a failure at this location would be a significant problem.	24	2,724	See project sheet in Section 11

Section 6

Lift Station Evaluation

This section summarizes the capacity and condition of the lift stations in GWA's wastewater system.

6.1 Lift Station Capacity Evaluation

Lift stations were evaluated under peak conditions for existing and future scenarios. Model results were reviewed to identify existing and future deficiencies. Capacity limitations at a lift station can be due to insufficient pump and/or force main capacity. Capacities of the pumps and force mains were analyzed simultaneously to identify if the pumps, force mains, or both the pumps and force mains need to be upsized.

Only lift stations with sufficient information, as listed in the last column in Table 2-5, could be analyzed. Pump data was available for 20 of the 64 lift stations in Table 2-5, which includes 87 percent of the system flow as discussed in Section 2.4. The remaining lift stations should be evaluated as additional data is gathered.

6.1.1 Criteria

Capacity of the lift stations was analyzed and compared to existing and future flows using the following criteria (see Appendix C for additional details on the criteria):

- **Redundancy/reliability:** each lift station should have a minimum of two pumps.
- **Minimum capacity (with largest pump on standby):** each lift station should be sized to handle the design storm.
- **Design storm:** a 2-year, 24-hour storm was used to evaluate the collection system, identify deficiencies, and develop improvements.

Operations staff use the following criteria, which should be noted when sizing new wet wells:

- In the past, GWA staff have calculated the capacity of a wet well as the entire wet well volume. It is recommended that when a new wet well is sized, the capacity should be calculated as only the volume available to the invert of the inlet line. This calculation will result in a larger wet well, which will reduce cycling of pumps.
- A wet well can be sized to allow backwater, but not SSOs, into the upstream piping.

6.1.2 Capacity Evaluation

Lift stations were evaluated in the model using the criteria listed above. During interviews with GWA operations staff, additional lift stations with insufficient capacity were identified. Table 6-1 lists the lift stations identified as having insufficient capacity by the model and operations staff.

Table 6-1. Lift Stations with Insufficient Capacity					
Basin	Lift Station	Model Capacity Issue	From GWA Operations Staff		Notes
			Pump Capacity Inadequate	Wet Well Too Small	
Hagåtña	Ypao		X	X	
Hagåtña	Piti		X		
Hagåtña	Tai Mangilao		X		
Hagåtña	Dairy Road			X	
Hagåtña	Harmon			X	
Hagåtña	Mongmong-Toto			X	
Inarajan	Inarajan		X		
Northern District	Latte Heights Submarine			X	
Northern District	Sunrise Villa			X	
Northern District	Yigo	X (Issue for future, peak wet weather flow)			This lift station is planned for improvements. The improvements should look at the capacity of the lift station.
Agat-Santa Rita	Gaan		X		This lift station is being upgraded as part of the Santa Rita WWTP project. Therefore, capacity issues are not discussed further for this lift station.
Umatac-Merizo	Ejector Station No. 2			X	
Umatac-Merizo	Ejector Station No. 3			X	

As discussed in Section 4.2 for gravity piping, Figure 4-4 shows that a large amount of the collection system in the Hagåtña basin was predicted to have capacity problems. As discussed in Table 3-2, the Hagåtña basin did not calibrate well due to issues with the flow metering data. The model predicted additional capacity issues for the lift stations in the Hagåtña basin, including the Asan, Barrigada, Hagåtña Main, Mamajanao, and Pago Double Shaft lift stations. Due to the calibration issues, the deficiencies are not listed in Table 6-1 and improvement projects were not developed for those lift stations. The lift stations and their drainage areas need to be studied in more detail as recommended in Section 9 before improvement projects can be developed.

The capacity issues identified in Table 6-1 were considered in the risk-based condition analysis in the following section.

6.2 Lift Station Condition Assessment

Similar to the gravity piping, a risk-based approach was used to prioritize lift station renewal. This section describes the risk calculations and recommendations for lift station renewal.

6.2.1 Condition Assessment

Each lift station was visited and visually assessed between 2013 and 2017. The assessments focused on the physical condition of the lift stations but no capacity testing was performed. The assessments were done during the following periods.

- **May 2013 assessment:** ten critical lift stations were assessed in the Northern District basin in May 2013. A report was generated, which listed deficiencies and recommendations for rehabilitation and replacement. Improvements are currently under construction for seven of the ten lift stations. Because identified deficiencies are currently being addressed, the following lift stations were not analyzed further:
  - Astumbo No. 1
  - Astumbo No. 2
  - Fujita
  - Hafa Adai
  - Macheche
  - Route 16
  - Santa Ana
  - Southern Link
  - Yigo
  - Ypaopao

  Of the ten lift stations, funding was not available as of 2017 to repair the Astumbo No. 2, Hafa Adai, and Yigo lift stations. And of the remaining seven lift stations, funding was not available to repair all of the deficiencies at the lift stations.
- **January 2017 assessment:** GWA staff visited and assessed 55 of the 64 lift stations listed in Table 2-5 in January 2017. Table 6-2 summarizes major issues identified at the lift stations. GWA staff did not assess the 10 lift stations listed above.
- **Other:**
  - Gaan lift station is being upgraded and was not analyzed further (being upgraded as part of the Santa Rita WWTP project)
  - GWA inherited an old lift station serving the Donut Hole area in Tiyan from the Navy. The lift station is in extremely poor condition and needs to be replaced. Project MP-WW-Pump-03 includes the design and construction of a replacement lift station.

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Table 6-2. Lift Station Condition Assessment

Station	Electrical				Health and Safety				Capacity		Building and Site				Station				Other				
	No Generator Present	Generator Needs Repair or Replacement	Control Systems, Alarms, or SCADA Problem	Other (including lighting)	Ventilation Needed	Railings Needed	Eye Wash Stations Needed	Gratings/ Hatches Needed	Backup Pump(s) Needed	Under Capacity	General Building or Site Issue (e.g. painting, rustproofing, spalling)	Water Supply Needed	Road, Fencing, or Other Site Access Problem	Crane/ Lift Needed	New Station Needed	Entire Station Rehabilitation Needed	Wet Well Too Small	Is or Was Ejector Station, Upgrade Needed	Comminutor, Screen, or Grit Removal Needed	Flow Meter Needed	Piping and/or Valve Issues (e.g. needs painting or rusty)	Other Equipment Corrosion	Maintenance Difficult
Agat-Santa Rita Basin																							
Agat Chaligan Taleyfac (Chaligan)			X	X	X	X	X		X		X										X	X	
Pagachao			X	X		X	X		X					X									
Tipalao									X														
Baza Gardens Basin																							
Main Trunk Line			X	X					X		X	X									X		
Talofofo				X	X		X		X		X		X								X	X	
Hagåtña Basin																							
Alupang Cove				X		X		X	X		X			X									
Asan									X		X												
Barrigada									X														
Bayside				X					X		X												
Casamiro									X														
Chalan Pago PS 3						X			X														
Chalan Pago PS 5									X														
Commercial Port			X	X					X		X									X	X	X	
Dairy Road									X								X						
Hagåtña Main				X	X				X		X		X						X		X		
Harmon			X	X			X	X	X		X		X	X			X			X	X		
Leyang				X		X			X														
Maite				X					X		X							X			X		
Mamajanao								X	X		X				X						X		X
Mangilao				X					X										X	X	X		
Mongmong-Toto					X				X		X						X	X			X	X	
Namo Yona	X			X	X			X	X		X							X		X	X		
New Chaot			X						X				X						X		X		
Ordot						X			X												X		
Pago Double Shaft		X	X	X					X										X	X			
Paseo De Oro			X	X	X		X	X	X		X		X	X					X		X		
Piti			X	X					X	X						X							
Sinajana			X				X		X		X												

Table 6-2. Lift Station Condition Assessment

Station	Electrical				Health and Safety				Capacity		Building and Site				Station				Other				
	No Generator Present	Generator Needs Repair or Replacement	Control Systems, Alarms, or SCADA Problem	Other (including lighting)	Ventilation Needed	Railings Needed	Eye Wash Stations Needed	Gratings/ Hatches Needed	Backup Pump(s) Needed	Under Capacity	General Building or Site Issue (e.g. painting, rustproofing, spalling)	Water Supply Needed	Road, Fencing, or Other Site Access Problem	Crane/ Lift Needed	New Station Needed	Entire Station Rehabilitation Needed	Wet Well Too Small	Is or Was Ejector Station, Upgrade Needed	Comminutor, Screen, or Grit Removal Needed	Flow Meter Needed	Piping and/or Valve Issues (e.g. needs painting or rusty)	Other Equipment Corrosion	Maintenance Difficult
Tai Mangilao					X				X	X									X	X			
Toto Garden		X				X		X	X														
Ypao			X	X	X		X		X	X	X			X	X		X					X	
Inarajan Basin																							
Inarajan						X			X	X	X			X							X		
Inarajan Main						X	X		X		X			X							X		
Northern District Basin																							
Latte Heights Double Tree			X	X	X	X			X		X			X					X	X	X		
Latte Heights Submarine			X	X			X		X		X		X	X			X				X	X	
Latte Plantation			X		X				X		X	X		X					X	X	X		
Machanaonao		X	X	X	X			X	X		X		X							X			
Pacific Latte		X	X	X			X		X		X		X	X						X	X	X	
PGD			X	X	X		X		X					X						X			
Sunrise Villa			X						X				X	X			X		X	X	X		
Umatac-Merizo Basin																							
Ejector Station No. 2				X					X		X						X	X			X		
Ejector Station No. 3				X					X		X						X	X			X		
Ejector Station No. 5				X					X		X							X			X		
Ejector Station No. 6				X					X		X							X			X		
Ejector Station No. 7				X					X		X							X					
Pump Station No. 11						X			X		X										X		
Pump Station No. 12						X	X		X		X								X		X		
Pump Station No. 13				X		X	X	X	X		X										X		
Pump Station No. 14				X			X		X		X										X		
Pump Station No. 15				X			X		X		X	X									X		
Pump Station No. 16				X			X		X		X		X						X		X		
Pump Station No. 17				X			X		X		X										X		
Pump Station No. 18				X		X	X		X		X										X		
Reyes	X			X					X		X	X									X		

6.2.2 Risk Calculations

Table 6-3 lists the likelihood of failure factors, which match the condition assessment factors listed in Table 6-2. Scores were assigned for each factor and each lift station listed in Table 6-2. A score of 5 was given for problems (pink shaded cells in Table 6-2) and 1 for no problems (unshaded cells in Table 6-2).

Table 6-3. Likelihood of Failure Factors

Category	Factor Description	Weight
Electrical	No generator present	4
Electrical	Generator needs repair or replacement	3
Electrical	Control systems, alarms, or SCADA problem	1
Electrical	Other (including lighting)	1
Health and Safety	Ventilation needed	4
Health and Safety	Railings needed	1
Health and Safety	Eye wash stations needed	1
Health and Safety	Gratings/hatches needed	1
Capacity	Backup pump(s) needed	3
Capacity	Under capacity	5
Building/Site	General building or site issue (e.g. painting, rustproofing, spalling)	1
Building/Site	Water supply needed	1
Building/Site	Road, fencing, or other site access problem	1
Building/Site	Crane/lift needed	1
Station	New station needed	5
Station	Entire station rehabilitation needed	5
Station	Wet well too small	3
Station	Is or was ejector station, upgrade needed	3
Other	Comminutor, screen, or grit removal needed	3
Other	Flow meter needed	1
Other	Piping and/or valve issues (e.g. needs painting or rusty)	2
Other	Other equipment corrosion	2
Other	Maintenance difficult	3

The lift stations used the same consequence of failure factors as the force mains for factors C3 through C7 in Table 5-4.

Scores were calculated for each lift station using the following steps:

1. Assign a score of 1 to 5 for each likelihood of failure factor to each lift station.
2. Calculate a total likelihood of failure factor for each lift station by summing the scores: $L1_{score} \times L1_{weight} + L2_{score} \times L2_{weight} + \ldots Ln_{score} \times Ln_{weight}$
3. Normalize all likelihood of failure scores so the scores range from 1 to 5.

4. Repeat steps 1 to 3 for consequence of failure.
5. Calculate the total risk for each lift station: likelihood of failure score (1 to 5) x consequence of failure score (1 to 5).
6. Normalize all risk scores so the highest score is 100.

Table 6-4 lists lift station priorities for renewal based on the risk calculations. The table includes the same 55 lift stations listed in Table 6-2. The lift stations were sorted into the same four priorities used for gravity pipes:

1. High Priority: Likelihood of failure >= 3, Consequence of failure >= 3
2. High Likelihood: Likelihood of failure >= 3, Consequence of failure < 3
3. Highly Critical: Likelihood of failure < 3, Consequence of failure >= 3
4. Lower Priority: Likelihood of failure < 3, Consequence of failure < 3

As mentioned above, funding was not available to repair three of the lift stations or to repair all of the deficiencies at the other seven lift stations identified under the Northern District Critical Pump Stations project. The repair project for the lift stations was still under development at the time of this report. Therefore, the ten lift stations are also listed in Table 6-4. The three projects deleted from the construction phase should be included in the next round of lift station upgrades. The remaining deficiencies at those lift stations should be considered when prioritizing lift station rehabilitation.

Table 6-4. Lift Station Rehabilitation Prioritization

Lift Station	WWTP Basin	Failure Score (1 to 5)		Risk Score (1 to 100)
		Likelihood	Consequence	
Lift Stations Planned for Repair				
Astumbo No. 1	Northern District	Not available, lift stations currently planned for or undergoing rehabilitation	Not available, lift stations currently planned for or undergoing rehabilitation	Not available, lift stations currently planned for or undergoing rehabilitation
Astumbo No. 2	Northern District			
Fujita	Tumon			
Hafa Adai (note that this lift station was not listed in the lift station summary in Table 2-5 because information was not available on the lift station)	Northern District			
Macheche	Northern District			
Route 16	Northern District			
Santa Ana	Northern District			
Southern Link	Northern District			
Yigo	Northern District			
Ypaopao	Northern District			
High Priority (Likelihood >= 3, Consequence >= 3)				
Ypao	Hagåtña	5.0	3.6	100
Hagåtña Main	Hagåtña	3.6	5.0	98
Mamajanao	Hagåtña	3.6	3.9	76

Table 6-4. Lift Station Rehabilitation Prioritization				
Lift Station	WWTP Basin	Failure Score (1 to 5)		Risk Score (1 to 100)
		Likelihood	Consequence	
High Likelihood (Likelihood >= 3, Consequence < 3)				
Agat Chaligan Taleyfac (also called Chaligan)	Agat-Santa Rita	3.7	2.5	50
Pago Double Shaft	Hagåtña	3.2	2.7	46
Tai Mangilao	Hagåtña	3.7	2.1	43
Machanaonao	Northern District	3.7	2.0	41
Piti	Hagåtña	3.6	2.0	39
Inarajan	Inarajan	3.3	2.0	36
Ejector Station No. 2	Umatac-Merizo	3.3	2.0	36
Pump Station No. 16	Umatac-Merizo	3.2	2.0	35
Commercial Port	Hagåtña	3.1	2.0	34
Ejector Station No. 3	Umatac-Merizo	3.3	1.8	33
Reyes	Umatac-Merizo	3.2	1.8	32
Talofofo	Baza Gardens	3.7	1.6	31
Harmon	Hagåtña	3.8	1.4	30
Latte Heights Submarine	Northern District	3.7	1.4	29
Namo Yona	Hagåtña	4.2	1.2	28
Sunrise Villa	Northern District	3.6	1.4	28
Paseo De Oro	Hagåtña	4.0	1.2	27
Mongmong-Toto	Hagåtña	3.9	1.2	26
Latte Heights Double Tree	Northern District	3.9	1.2	26
Latte Plantation	Northern District	3.8	1.2	26
Pacific Latte	Northern District	3.8	1.2	26
PGD	Northern District	3.2	1.4	25
Pump Station No. 12	Umatac-Merizo	3.1	1.4	24
Highly Critical (Likelihood < 3, Consequence >= 3)				
Bayside	Hagåtña	2.3	3.3	42
Lower Priority (Likelihood < 3, Consequence < 3)				
Alupang Cove	Hagåtña	2.7	2.9	43
New Chaot	Hagåtña	2.9	2.3	37
Mangilao	Hagåtña	2.9	2.2	36

Table 6-4. Lift Station Rehabilitation Prioritization				
Lift Station	WWTP Basin	Failure Score (1 to 5)		Risk Score (1 to 100)
		Likelihood	Consequence	
Inarajan Main	Inarajan	2.8	2.3	36
Asan	Hagåtña	2.2	2.8	34
Barrigada	Hagåtña	2.1	3.0	34
Pagachao	Agat-Santa Rita	2.7	2.1	32
Toto Garden	Hagåtña	2.7	2.0	30
Dairy Road	Hagåtña	2.5	2.0	27
Pump Station No. 17	Umatac-Merizo	2.7	1.8	27
Maite	Hagåtña	2.9	1.6	26
Ejector Station No. 6	Umatac-Merizo	2.9	1.6	26
Pump Station No. 11	Umatac-Merizo	2.6	1.8	26
Pump Station No. 13	Umatac-Merizo	2.9	1.6	26
Sinajana	Hagåtña	2.5	1.8	25
Ejector Station No. 7	Umatac-Merizo	2.8	1.6	25
Pump Station No. 15	Umatac-Merizo	2.8	1.6	25
Pump Station No. 18	Umatac-Merizo	2.8	1.6	25
Pump Station No. 14	Umatac-Merizo	2.7	1.6	24
Ordot	Hagåtña	2.6	1.6	23
Ejector Station No. 5	Umatac-Merizo	2.9	1.4	23
Main Trunk Line	Baza Gardens	2.8	1.2	19
Leyang	Hagåtña	2.3	1.4	18
Chalan Pago PS 3	Hagåtña	2.2	1.4	17
Tipalao	Agat-Santa Rita	2.1	1.2	14
Casamiro	Hagåtña	2.1	1.2	14
Chalan Pago PS 5	Hagåtña	2.1	1.2	14

Part of the renewal needs include replacing ejector pumps and constructing regular lift stations. GWA operations has reported that the ejectors do not work well because they constantly turn on due to condensation. Also, during peak flows, flow needs to be regulated because the air compressors are small, which also leads to compressor burnouts.

The notes in Table 6-5, collected from GWA operations staff should also be considered when rehabilitating or replacing lift stations. These notes are not comprehensive and do not include all issues found during condition assessment visits. However, operations staff highlighted these as key problems during interviews.

Table 6-5. GWA Operations Staff Lift Station Notes		
Lift Station	**WWTP Basin**	**Notes**
Dairy Road	Hagåtña	Need to upgrade wet well, prison population is going to increase.
Harmon	Hagåtña	There is a belly in the road in front of the lift station with two manholes. During rain, flow goes into the two manholes and into the lift station. The pumps constantly run during rain events due to high inflow so the pumps burn out.
Maite	Hagåtña	Ejector station, needs to be converted to a full lift station. The wet well is a shallow manhole, so the low volume causes the pumps to cycle excessively.
Mamajanao	Hagåtña	The lift station needs upgraded. The valves and wet well are hard to clean. The lift station was designed to pump northeast, but it currently pumps to the Hagåtña WWTP.
Piti	Hagåtña	The lift station is now receiving flow from Cabras Island and the port so the pumps need upsized. If the pumps are upsized, the electrical system would also need to be upgraded. There are erosion issues from the ocean at the back of the building. Level controls are old mercury switches and need upgraded.
Tai Mangilao	Hagåtña	Two of the pumps were downsized to save energy, but they are currently too small. The two smaller pumps should be sized to match the size of the larger third pump. An agitator or comminutor is needed.
Ypao	Hagåtña	A new lift station was constructed at the same site, but was not put into operation. The new lift station was constructed to pump to Mamajanao and then to Route 16, but there were issues with the force main (and Mamajanao does not pump to Route 16). The old lift station has safety issues and the new lift station is safer with a bigger wet well. However, the new lift station would likely need rehabilitation because it has not been used since it was constructed. Either the new lift station should be activated or the old lift station needs to be rehabilitated, including with larger pumps and a larger wet well. Options for this lift station should be studied.
Latte Heights Submarine	Northern District	The wet well is a manhole.
Latte Plantation	Northern District	The 2-inch force main should be upsized to at least 4 inches.
Route 16	Northern District	Grit removal is needed before the lift station. Because of the configuration of the lift station, grit removal is difficult and must be done by hand.
Sunrise Villa	Northern District	Expand wet well or re-route incoming line to center of wet well so there is enough room to put a basket in the wet well.
Pump Station No. 11	Umatac-Merizo	Need to replace isolation and check valves, piping, and risers.
Pump Station No. 12	Umatac-Merizo	Need to replace isolation and check valves, piping, and risers.
Pump Station No. 13	Umatac-Merizo	Need to replace isolation and check valves, piping, and risers.
Pump Station No. 14	Umatac-Merizo	There are electrical issues, the lift station gets service at 230 volts but is running at 460 volts.
Pump Station No. 15	Umatac-Merizo	There are electrical issues, the lift station gets service at 230 volts but is running at 460 volts.
Pump Station No. 16	Umatac-Merizo	There are electrical issues, the lift station gets service at 230 volts but is running at 460 volts.
Pump Station No. 17	Umatac-Merizo	There are electrical issues, the lift station gets service at 230 volts but is running at 460 volts.
Pump Station No. 18	Umatac-Merizo	There are electrical issues, the lift station gets service at 230 volts but is running at 460 volts.
Reyes	Umatac-Merizo	The lift station needs new piping and the control panel needs a shelter

6.3 Recommendations

The following items are recommended for lift stations:

- Lift stations should be rehabilitated and replaced based on the risk analysis described in this section and based on the priorities listed in Table 6-4. Lift stations should be grouped into projects and GWA should put the projects out to bid to be fixed by a qualified contractor. This project is referenced as project MP-WW-Pump-01 in Section 11. The notes in Table 6-5 should also be considered for the rehabilitation or replacement of the lift stations.
- A study was recently conducted for the Fujita lift station and force main. The study report, titled *Preliminary Planning/Engineering Report Fujita Pump Station Service Area Improvements* (CDM Smith, 2017), discusses issues and five options for the force main and lift station, such as a new parallel force main. GWA should review the report and select an option for implementation. A project to implement the recommendations is referenced as MP-WW-Pump-02 in Section 11.
- GWA should implement a lift station preventive maintenance program. One of the main issues with operations at lift stations relates to grease. Grease and rags clog the pumps and float sensors cannot operate due to thick grease, which can prevent pumps from turning on. Operations staff have reported floatables in lift stations up to 1 foot thick.

Section 7

Wastewater Treatment Evaluation

This section summarizes the current conditions, regulatory concerns, and proposed improvements to GWA's wastewater treatment systems.

A brief summary of the wastewater conveyance and collection systems is given in Section 7.1. The facilities evaluation discusses Umatac-Merizo WWTP in Section 7.2, Baza Gardens WWTP in Section 7.3, Agat-Santa Rita WWTP in Section 7.4, Inarajan WWTP in Section 7.5, Pago Socio WWTP in Section 7.6, Northern District WWTP in Section 7.7, Hagåtña WWTP in Section 7.8, and GWA's Solids Management Plan in Section 8. Each of the treatment plant evaluations describe the existing conditions, the regulatory requirements, the wastewater characteristics, and the recommended improvement projects at each facility.

7.1 Conveyance and Collection Systems

A network of gravity sewers, lift stations, and force mains collect and convey wastewater to each WWTP. Figure 2-2 shows the sewer basin collection areas for the five southern wastewater systems, while Figure 2-1 shows the wastewater collection areas for the two northern/central GWA WWTPs.

On the west coast of southern Guam, the Umatac-Merizo WWTP treats wastewater flows from the villages of Umatac and Merizo. The Agat-Santa Rita facility treats wastewater flows from the communities immediately south of the Apra Harbor Naval Base. For effluent disposal, the Agat-Santa Rita WWTP shares the ocean outfall with the Navy's Apra Harbor WWTP. On the east coast of southern Guam, wastewater from the Baza Gardens and Talofofo communities is currently treated at the Baza Gardens WWTP, and the Inarajan WWTP handles flows from the Inarajan village and leachate from the Layon Landfill. The Pago Socio WWTP serves a small community on the eastern coast.

The northern and central areas of Guam are serviced by the Northern District WWTP and Hagåtña WWTP. The 4.3 mgd Apra Harbor WWTP, also located in the northern Guam region, is owned and operated by the U.S. Navy and is not included in the current GWA master plan.

7.2 Umatac-Merizo WWTP

At the time of this report, the Umatac-Merizo WWTP is undergoing major modifications to meet the requirements of Paragraph 16 of the 2011 Court Order (United States of America, 2011). The court order requires GWA to identify and complete improvements necessary to meet the Umatac-Merizo WWTP NPDES permit by December 31, 2018. A design-build team is currently under contract to implement the improvement project and by the required completion date. Based on the current design criteria, the capacity of the new plant will be adequate through 2035 flows.

The following section describes the existing conditions, regulatory requirements, wastewater characteristics, and recommended improvement projects at Umatac-Merizo WWTP.

7.2.1 Existing Conditions

Existing conditions at Umatac-Merizo WWTP are described below.

7.2.1.1 Treatment and Disposal Processes

The Umatac-Merizo WWTP was originally constructed in 1981 and consists of a Parshall flume influent flow meter, influent pump station with basic basket screening, aerated lagoon, and overland flow system. The overland flow system acts as a polishing treatment system and provides for partial disposal of the WWTP effluent. During dry weather, most or all WWTP effluent is assimilated by the overland flow system. During wet weather, when the combination of WWTP effluent and precipitation exceeds the assimilative capacity of the overland flow terraces, there is discharge to the Toguan River. Figure 7-1 shows the Umatac-Merizo WWTP in February 2017.

Figure 7-1. Umatac-Merizo WWTP (February 2017)

The wastewater influent flows through a Parshall flume. The flume's level sensor is currently inoperative. GWA uses a FlowShark area velocity flow monitor to record influent flow rates. The influent is first screened through a screening basket that must be manually cleaned. Figure 7-2 shows the manual basket screen during a routine maintenance cleaning.

Figure 7-2. Umatac-Merizo WWTP Manual Basket Screen (January 2013)

Once screened, wastewater is pumped to the aerated lagoon via Pump Station No. 13, comprised of a concrete wet well with two submersible pumps. The station is in fair condition.

The aerated lagoon provides most of the biological treatment and is equipped with two aerators. Per available record drawings, each aerator has a 25 horsepower (hp) motor. An overflow pipe was installed to protect the lagoon berm integrity. The overflow leads to the Toguan River. The current NPDES permit does not recognize this emergency overflow pipe; therefore, any discharge through the lagoon overflow is in violation of the permit. Table 7-1 summarizes the existing lagoon's characteristics.

Table 7-1. Umatac-Merizo WWTP Aerated Lagoon Characteristics

Description		Value
Dimensions (length x width)	At top of berm	305 feet x 170 feet
	At normal water line	274 feet x 139 feet
	At bottom	224 feet x 89 feet
Side slope	H:V	3:1
Side slope protection		Concrete
Approximate normal water depth (feet)		8.3
Volume at normal water depth (million gallons)		1.8
Aerators	Type	Floating aspirator
	Number	2
	Motor size	25 hp

Note. Per 1996 Design Plans for PUAG Project No. S92-001 LOC and 2004 CPE Report.

The lagoon effluent is pumped via Pump Station No. 19 and the adjoining force main to the overland flow system. In the past several years, the lagoon has overflowed during wet weather, even when both pumps at Pump Station No. 19 were running at full capacity. The pumping capacity of the influent pump station exceeds the capacity of the lagoon effluent pump station and force main, causing lagoon overflows during high flow events.

The overland flow system provides polishing treatment to meet secondary treatment requirements and partial final disposal via percolation and evapotranspiration. The overland flow terraces were designed to allow the effluent to flow, percolate, and evaporate over approximately 8.8 terraced acres. The overland flow distribution system is generally configured to apply water to one of the two sides of the terraces. The north side has an area of 2.9 acres, which is approximately half of the south side area (5.9 acres). The grading of the overland system terraces has deteriorated, causing flows to short circuit, resulting in only a portion of each terrace available to treat the flow. Table 7-2 summarizes the characteristics of the overland flow system.

Table 7-2. Umatac-Merizo WWTP Overland Flow System

Parameter	North Side	South Side
Area (acres)	2.9	5.9
Number of terraces	4	6
Average terrace length (feet)	350	450
Average terrace width (feet)	100	100
Soil type	Akina-Badland complex, 7 to 15 percent slopes	
Permeability	1.5-5.0 cm/hour (depth 0-10 cm) 0.5-1.5 cm/hour (depth 10-61 cm)	
Vegetation	Grasses	

Effluent not removed by the overland flow system flows to a recirculation pond, from where it is pumped back (via Pump Station No. 20) to the top of the overland flow system. When the recirculation pond is full, it overflows to a channel that discharges to the Toguan River (“Discharge” in Figure 7-3). The facility’s NPDES permit authorizes the discharge. Although there is poor distribution through the overland flow terraces, the system generally does not discharge to the Toguan River during dry weather conditions. Figure 7-3 presents a system flow schematic of the existing plant.

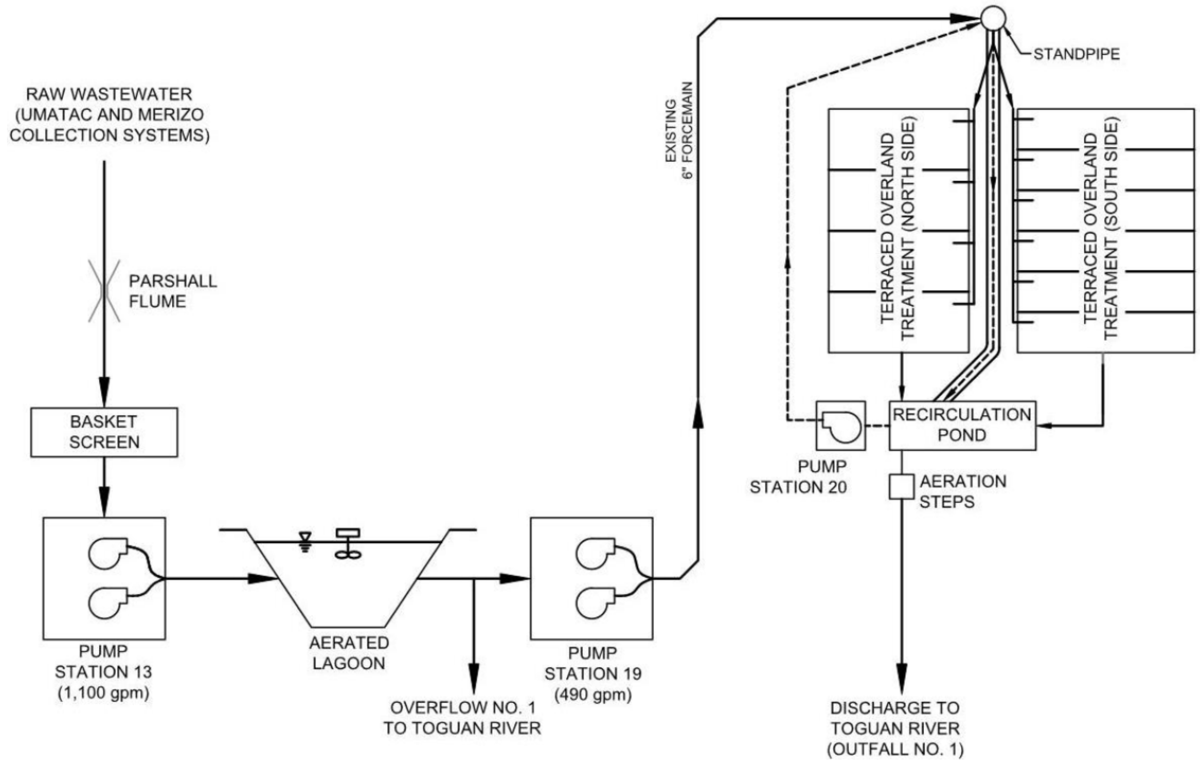

Figure 7-3. Umatac-Merizo WWTP Existing Flow Schematic

7.2.2 Regulatory Requirements

Regulatory requirements that apply to Umatac-Merizo WWTP are described below.

7.2.2.1 NPDES Requirements

The Umatac-Merizo WWTP operates under NPDES permit No. GU0020273 issued on August 19, 2015, and valid until August 31, 2020. The permit is intended to apply to discharges to the Toguan River via Outfall No. 1 shown in Figure 7-3. The permit is based on federal and Guam Water Quality Standards (GWQS). GWA is required to report and sample discharges to the Toguan River.

The NPDES permit assumes a system consisting of an aerated lagoon followed by constructed wetlands (design provided in 1996 by Winzler & Kelley, but not constructed) with a monthly average discharge of 0.391 mgd into the Toguan River which is considered a Category S-3 (low) surface water. Category S-3 water is defined in the GWQS as surface water primarily used for commercial, agricultural, and industrial activities. Table 7-3 summarizes the NPDES requirements for Umatac-Merizo WWTP, as presented in permit No. GU0020273.

Table 7-3. Umatac-Merizo WWTP NPDES Effluent Limitations

Parameter	Units	Average Monthly	Average Weekly	Maximum Daily
Flow	mgd	0.39	--	Monitoring only
Temperature	°C	--	--	Monitoring only
BOD_5	mg/L	30	45	--
	lbs/day	98	147	--
	%	Average monthly shall not be less than 85 percent removal		

Table 7-3. Umatac-Merizo WWTP NPDES Effluent Limitations				
Parameter	**Units**	**Average Monthly**	**Average Weekly**	**Maximum Daily**
TSS	mg/L	30	45	--
	lbs/day	98	147	--
	%	Average monthly shall not be less than 85 percent removal		
pH	Std. Units	--	--	--
Total chlorine residual	µg/L	6.1	--	12
Dissolved oxygen	mg/L	--	--	Monitoring only
Enterococcus	CFU/100mL	33	--	108
Oil and grease, total recoverable	mg/L	10	--	15
Ammonia (as N)	mg/L	Monitoring Only	--	Monitoring only
Ammonia impact ratio	Ratio	1.0	--	--
Nitrate-nitrogen (as N)	mg/L	0.5	--	--
Orthophosphate (PO_4-P)	mg/L	0.1	--	--
Chronic toxicity	Pass/Fail	--		
Priority pollutant scan	µg/L	--	--	Monitoring only

BOD_5 = 5-day Biochemical Oxygen Demand

TSS = Total Suspended Solids

7.2.2.2 Court Order and Additional Considerations

The Toguan River adjacent to the Umatac-Merizo WWTP is classified as category S-3 surface water in the GWQS. The WWTP was designed to comply with the secondary treatment standards in effect in the 1970s when the facility was initially designed. Secondary treatment standards regulate the amount of biodegradable organic material allowed in an effluent discharge, as measured by the 5-day biochemical oxygen demand (BOD_5) and total suspended solids (TSS) analytical parameters. The WWTP has generally achieved compliance with the secondary treatment standards.

The NPDES permit includes discharge limits for bacterial indicator organisms. The WWTP has never included a disinfection process; therefore, the facility has been unable to comply with the bacterial indicator organism requirements in the permit.

The NPDES permit includes discharge limits for nutrients in the form of nitrogen (N) and phosphorus. The WWTP was not designed to remove nutrients; therefore, the facility has been unable to consistently comply with the nutrient requirements in the permit.

GWA was required by the 2011 Court Order to design and complete WWTP improvements “that will achieve consistent compliance with the WWTP’s NPDES permit” (United States of America, 2011). GWA completed the required system evaluation and identified potential improvements at the Umatac-Merizo WWTP. GWA is pursuing site-specific water quality standards concurrently with the court-mandated WWTP improvement project to be completed by December 31, 2018.

7.2.3 Wastewater Characteristics

The characteristics of wastewater flow at Umatac-Merizo WWTP are described below.

7.2.3.1 Historical Flows and Loads

Monitoring data for the Umatac-Merizo WWTP is reported via Discharge Monitoring Reports (DMR). Reporting years for the Umatac-Merizo WWTP begin in October of the previous year and end in September. Unless otherwise noted, the historical flow characteristics described are based on the most recent 5-year reporting period from October 2011 through September 2016, representing the 2012–2016 reporting years.

7.2.3.1.1 Historical Flows

The average flow at the Umatac-Merizo WWTP for the previous five years was 0.347 mgd, with 2014 having the highest average flow of 0.539 mgd. Figure 7-4 shows the average monthly flow and peak day flow for each month during the last five reporting years.

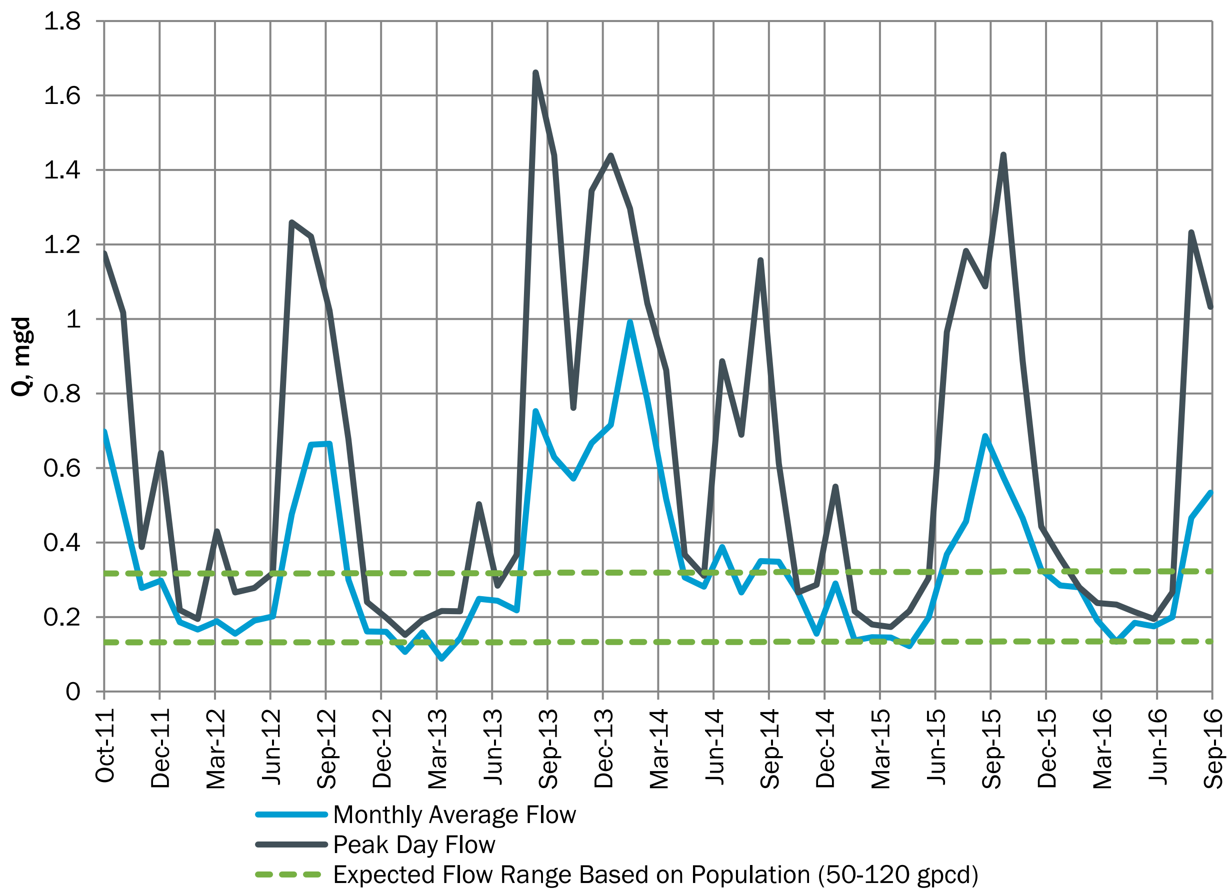

Figure 7-4. Umatac-Merizo WWTP Influent Flow, 2008–2012

Typical flow rates in the United States range between 50 gallons per capita per day (gpcd) in rural areas and 120 gpcd in typical urban areas (Tchobanoglous et al, 2003). Between 2012 and 2016, the combined population of Umatac and Merizo was estimated as 2,668 to 2,777 (See Volume 1, Section 4.4). The 0.347 mgd average flow to the plant for the same period represents a per capita flow of approximately 130 gallons per day (gpd), which is relatively high for a rural area. Groundwater infiltration into the collection system is a likely cause for the high per capita flow rates originating from these two rural villages.

Peak day peaking factors were calculated based on the recorded flows. The peak day peaking factor was calculated as the peak daily flow in a given month, divided by that month's average flow. Figure 7-5 shows the peak day peaking factors for the previous five reporting years.

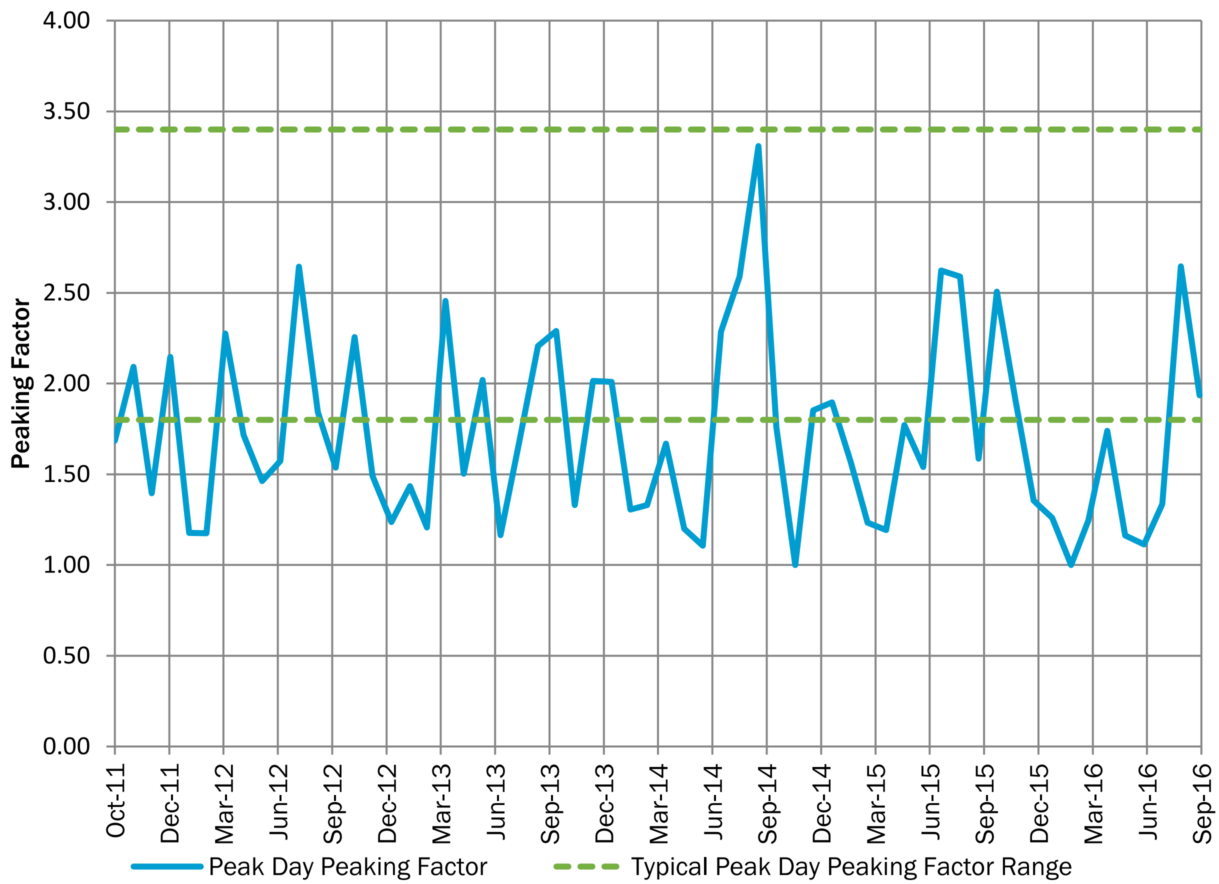

Figure 7-5. Umatac-Merizo WWTP Peaking Factors

The highest recorded peak day peaking factor was 3.31, recorded during severe wet weather conditions that occurred in September 2014. The maximum recorded peaking factor of 3.31 is within the range for typical small (less than 1.0 mgd), domestic WWTPs (Crites & Tchobanoglous, 1998), as shown as the dotted lines on the graph. The graph shows that stormwater inflow into the collection system during wet weather conditions does not appear to be excessive. Hydraulic modeling confirms the assessment.

7.2.3.1.2 Historical Loads

The DMRs for Umatac-Merizo WWTP include weekly influent sampling along with weekly effluent samples when a discharge to the Toguan River occurs. Based on the 2012 to 2016 data, the average influent BOD_5 was 41 milligrams per liter (mg/L) (99 pounds[lbs]/day), while the average TSS was 65 mg/L (151 lbs/day). Both values are very low compared to typical values for domestic wastewater. It should be noted that the average BOD_5 and TSS in 2006 (as reported in the WRMP) were reportedly higher at 216 mg/L and 70 mg/L, respectively. The reason for the discrepancy is not known, but could involve sampling technique variations.

Table 7-4 presents a comparison of the influent wastewater characteristics at Umatac-Merizo WWTP from 2012 to 2016, the calculated per capita mass based on influent flow and census population, and typical U.S. mainland values. As shown in the table, the per capita mass loads are well below typical U.S. mainland values.

Table 7-4. Comparison of Umatac-Merizo WWTP Influent Data with Typical Values

Year	Population [b]	Type of Value	BOD_5		TSS		Flow (mgd)
			mg/L	lbs/capita/day [a]	mg/L	lbs/capita/day [a]	
U.S. Typical Values			110–350	0.180–0.220	120–400	0.200–0.250	--
2012	2,668	Average	31	0.026	75	0.059	0.332
		Minimum	10	0.007	5	0.007	0.070
		Maximum	80	0.101	203	0.143	1.259
2013	2,687	Average	41	0.027	91	0.055	0.270
		Minimum	7	0.004	7	0.004	0.020
		Maximum	95	0.087	246	0.160	1.662
2014	2,705	Average	35	0.055	43	0.064	0.539
		Minimum	15	0.017	1	0.001	0.058
		Maximum	76	0.138	255	0.361	1.439
2015	2,723	Average	43	0.026	49	0.029	0.276
		Minimum	17	0.004	3	0.001	0.037
		Maximum	104	0.066	163	0.130	1.183
2016	2,777	Average	53	0.047	69	0.054	0.318
		Minimum	15	0.009	7	0.006	0.029
		Maximum	99	0.124	201	0.187	1.441

a. Tchobanoglous et al., 2003.

b. See Population Projections in Volume 1, Section 4.4.

Figure 7-6 shows the measured average influent BOD_5 concentration during the 2012–2016 reporting period, along with the typical BOD_5 concentration range for domestic wastewater. The graph shows that the influent BOD_5 concentration is also consistently low, confirming dilution by GWI.

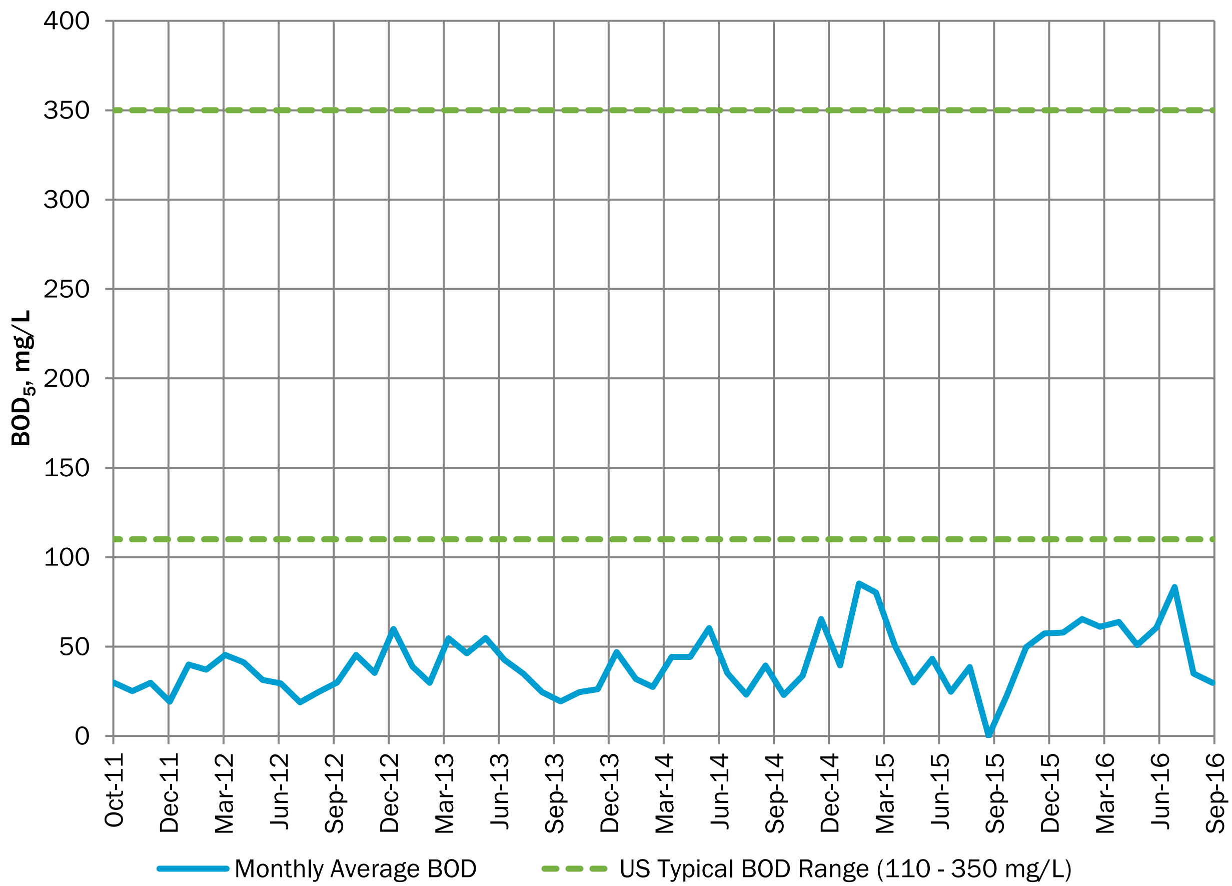

Figure 7-6. Umatac-Merizo WWTP Influent BOD_5 Concentration

Figure 7-7 shows the measured average influent TSS concentration during the 2012–2016 reporting period, along with the typical TSS concentration range for domestic wastewater. The graph shows that the influent TSS concentration is consistently low, indicating dilution by GWI.

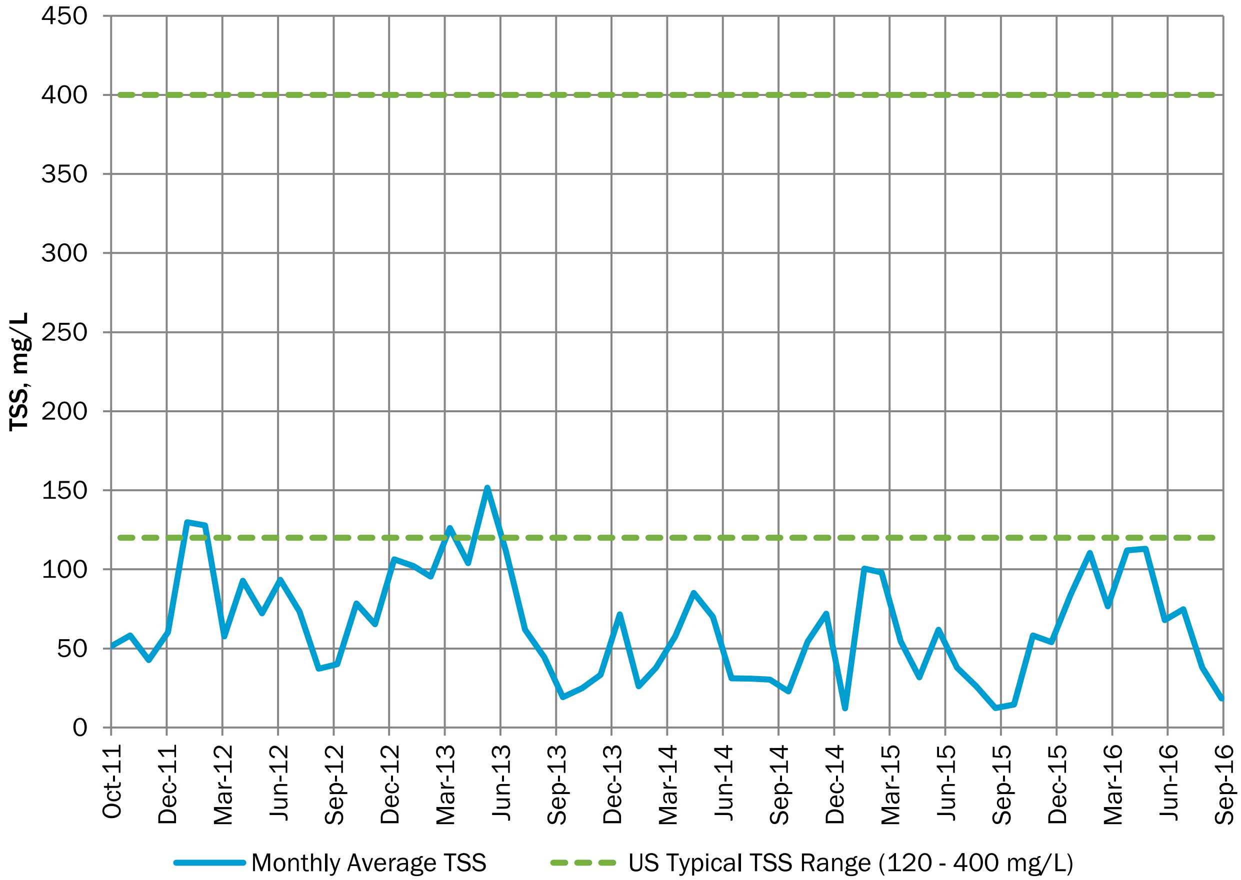

Figure 7-7. Umatac-Merizo WWTP Influent TSS Concentration

7.2.3.1.3 Historical Flow and Load Assessment

The average influent flow rates are significantly higher than would normally be expected for the size of the population. The influent strength (as measured by BOD_5 and TSS concentration) is significantly weaker than normal with TSS concentrations slightly higher than BOD_5. Based on the available data, it appears that significant GWI into the collection system is occurring. It should be noted that low concentrations of BOD_5 and TSS are experienced throughout Guam, including at Navy-operated facilities.

The peak day flow peaking factors appear to be generally within normal limits for a small WWTP, indicating stormwater inflow into the collection system is not excessive. Published curves for WWTPs with average dry weather flow capacities less than 1.0 mgd indicate typical peak day peaking factors between 1.8 and 3.4 (Crites & Tchobanoglous, 1998). The highest peak day peaking factor during the last five years at the WWTP was 3.31, which occurred during severe wet weather conditions in September 2014. An excessive stormwater inflow condition would have resulted in a higher peak day flow under those conditions.

7.2.3.2 Effluent Characteristics

During the same 5-year period, sampled effluent was recorded in the monthly DMRs at least once a month in 49 of the 60-month period. The most commonly exceeded parameter limits in the samples were E. coli/enterococci and biological nutrients (phosphorous [P] and N). The effluent rarely exceeded the NPDES permit limits of 30 mg/L for the average effluent BOD_5 or TSS. The average effluent BOD_5 during the 5-year period was 5 mg/L, and the average effluent TSS was 16 mg/L. Due to the low influent concentrations, the 85-percent removal requirement for both BOD_5 and TSS is rarely reached.

7.2.3.2.1 Existing WWTP Process Assessment

The treatment system is unable to meet the N and P requirements of the existing NPDES permit. Lagoon systems followed by overland flow can nitrify effluent, but a denitrification process would be required to remove nitrate to achieve the effluent limits. A lagoon system followed by overland flow can only remove negligible amounts of P.

The enterococcus requirement of the NPDES permit requires a disinfection process. The Umatac-Merizo WWTP has never been equipped with a disinfection process.

A water balance was prepared to assess the disposal capacity of the overland flow terrace system. During the wet season, the average precipitation exceeds the evapotranspiration rate of the vegetation; therefore, the only pathway for effluent disposal other than permitted discharge is via percolation. The wastewater system evaluation (WSE) completed in December 2013, estimated the combined wastewater and precipitation disposal potential to be 381,300–403,500 gpd when using both sides of the overland flow system, depending on the time of year. Table 7-5 summarizes the disposal potential assessment. Combined wastewater flow and precipitation more than the site's disposal potential will result in runoff and discharge to the Toguan River. GWA currently does not use both sides of the overland flow system at the same time, but typically achieves zero discharge to the Toguan River during dry weather conditions.

Table 7-5. Umatac-Merizo WWTP Overland Flow Disposal Potential

Month	Evapotranspiration (gpd)	Percolation (gpd)	Recirculation Pond Evaporation (gpd)	Total Disposal Potential (gpd)
January	45,100	339,400	1,200	385,700
February	51,900	339,400	1,400	392,700
March	57,000	339,400	1,500	397,900
April	62,400	339,400	1,700	403,500
May	59,700	339,400	1,600	400,700
June	55,200	339,400	1,500	396,100
July	46,500	339,400	1,300	387,200
August	40,800	339,400	1,100	381,300
September	41,100	339,400	1,100	381,600
October	41,400	339,400	1,100	381,900
November	43,300	339,400	1,200	383,900
December	44,300	339,400	1,200	384,900

7.2.3.3 Flow and Load Projections

Influent flow and load projections are presented below.

7.2.3.3.1 Flow Projections

Future average flows and maximum month flows at the Umatac-Merizo WWTP were estimated using the assumption that flows will increase at the same rate as population growth. This is a conservative approach because it ignores any future improvements to reduce I/I in the collection system. Figure 7-8 shows average and maximum month daily flow projections for the Umatac-Merizo WWTP based on the projected population growth.

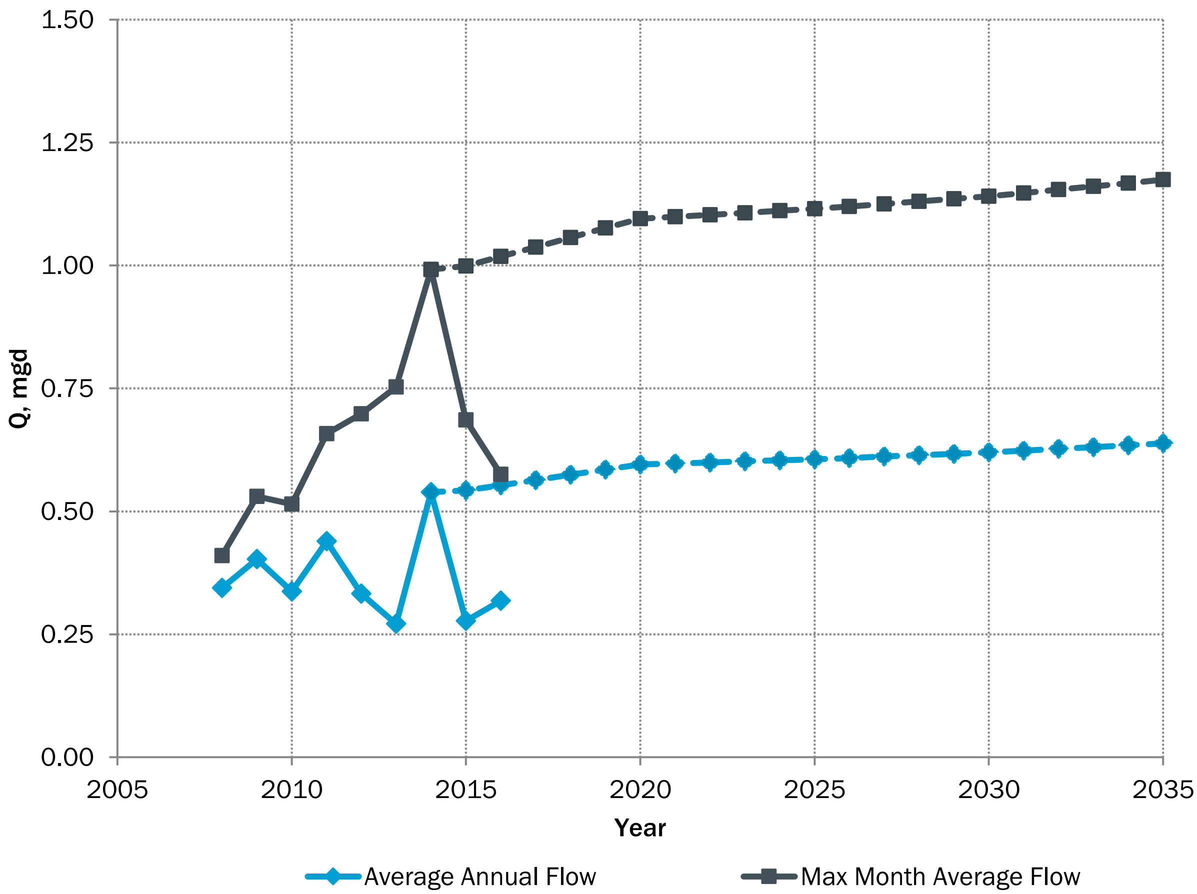

Figure 7-8. Umatac-Merizo WWTP Flow Projections

The SSES for the Umatac-Merizo collection system revealed pathways for extensive GWI into the collection system. The results of the SSES estimate that collection system improvements will lead to a significant reduction in I/I throughout the system. Hydraulic modeling was also performed to assess future peak wet weather flows. Taking into consideration the projected I/I reduction, the hydraulic modeling confirmed an average daily flow of approximately 0.6 mgd and determined a 2035 peak day flow at the Umatac-Merizo WWTP of 1.5 mgd, with a peak hourly flow of 1.7 mgd.

Table 7-6 summarizes flow projections for the 20-year planning period.

Table 7-6. Umatac-Merizo WWTP Planning Period Influent Flows	
Description	2035 [a]
Average annual dry weather flow (mgd)	0.6
Wet season average flow (mgd)	1.0
Peak day flow (mgd)	1.5
Peak hour flow (mgd)	1.7

a. From Basis of Design Report, Brown and Caldwell, 2016

7.2.3.3.2 Projected Loads

Based on the 2012–2016 data, both the average influent BOD_5 and TSS values were low compared to typical values for domestic wastewater flows. The historical samples present inconsistencies and include considerable dilution due to I/I. Efforts to reduce I/I will increase the influent concentrations; therefore, planning assumptions are not based on the diluted flows currently experienced at the WWTP. Future BOD_5 and TSS concentrations are calculated on a per capita basis. The Umatac-Merizo community has no significant commercial or industrial base; therefore, it was assumed that the typical per capita loads for domestic waste only in the United States are applicable to Guam. Values of 0.200 pounds of BOD_5 and 0.225 pounds of TSS per capita per day were used (Tchobanoglous, 2003). These loads were applied to the population projections and the resulting projected loads are presented in Figure 7-9, along with the average annual flow projections.

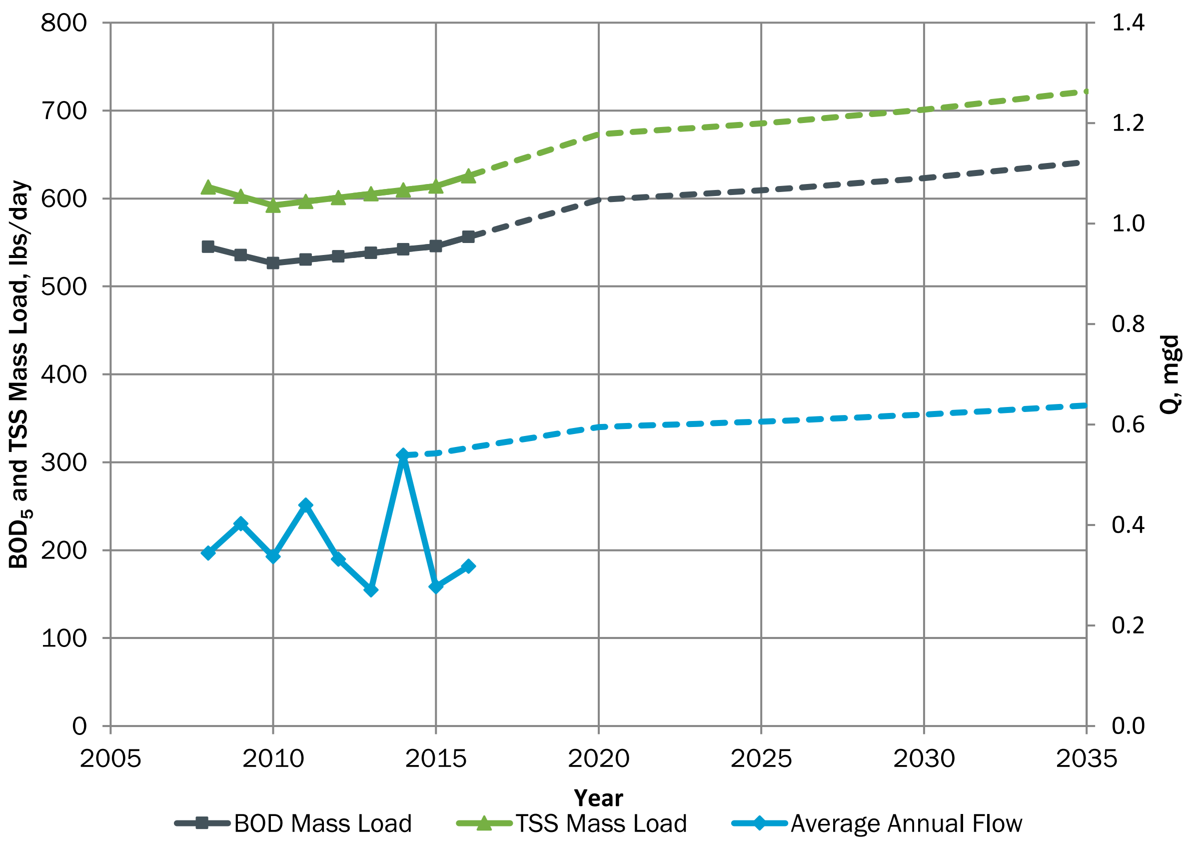

Figure 7-9. Umatac-Merizo WWTP Mass Load Projections

Since the highest concentrations would occur during a low-flow scenario, the projected average annual flows were used to calculate both BOD_5 and TSS concentrations (in mg/L). Table 7-7 compares Umatac-Merizo WWTP projected loads and concentration data with typical U.S. values.

Table 7-7. Umatac-Merizo WWTP Projected Flows and Loads

Description	BOD_5		TSS	
	mg/L	lbs/day	mg/L	lbs/day
U.S. Typical [a]	110–350	--	120–400	--
Umatac-Merizo 2035 [b]	210	642	236	722

a. Tchobanoglous et al., 2003.

b. Based on Umatac and Merizo projected population of 3,208 and typical values of 0.200 lbs/capita/day BOD_5 and 0.225 lbs/capita/day TSS.

7.2.4 Recommended Improvement Projects

Recommended improvement projects for Umatac-Merizo WWTP are described below.

7.2.4.1 2011 Court Order Projects

Since the 2006 WRMP, GWA has completed several projects in accordance with the 2011 Court order. No projects outside of the Court Order were completed for the Umatac-Merizo WWTP.

In December 2014, GWA submitted a WSE for the Umatac-Merizo WWTP, collection system, and conveyance system in accordance with Paragraph 15 of the 2011 Court Order. The WSE report and consequent supplement identified a plan to meet NPDES requirements at the plant through design improvements and water quality standards site-specific modifications.

GWA also tried to implement additional treatment and disposal improvements through an interim improvements project. The design included an additional force main to convey water from the lagoons to the overland treatment system and a disinfection system to prevent pathogens from entering the Toguan River when the plant discharges during wet weather. However, the interim design was not completed and the improvements will be incorporated into the complete WWTP redesign per the 2011 Court Order.

GWA has completed design-build documents for an improvement project at the Umatac-Merizo WWTP as imposed by the 2011 Court Order, paragraph 16 (United States of America, 2011). Paragraph 16 states that:

> *By December 31, 2018, GWA shall complete the improvements identified in the approved plan required by Paragraph 15 and achieve consistent compliance with the Umatac-Merizo WWTP's NPDES permit. GWA shall also meet the following interim compliance milestones:*
>
> *a. By June 30, 2016, GWA shall execute a design contract and issue a notice to proceed with the design.*
>
> *b. By June 30, 2017, GWA shall execute a construction contract and issue a notice to proceed with construction.*

A notice to proceed for the design-build contract was issued prior to the June 30, 2017 deadline. To meet the NPDES limits as required by the 2011 Court Order, the current contract includes the following modifications: a new headworks, rehabilitation of the influent pump station, improvements to the existing lagoon including new aerators and a baffle curtain, a new UV disinfection system, a new lagoon effluent pump station, new force main to the overland flow treatment system, improvements to the overland flow treatment system including a new distribution system, and a new storage tank, and new sampling equipment. The WWTP upgrade is scheduled to be complete by December 31, 2018.

Changes to the current GWQS will be pursued concurrently with construction of the treatment processes to establish site-specific requirements and ensure that the upgraded WWTP will be in compliance.

7.2.4.2 Recommended Improvement Project

The WWTP upgrade will be complete in 2018 and will require typical regular maintenance, but no additional improvement projects are expected in the near future.

In addition to adequate maintenance, a carefully planned replacement program is recommended for all major treatment process equipment. The planned treatment process equipment rehabilitation should occur every 15 years and include:

- Replacement or refurbishment of mechanical equipment and controls
- Rehabilitation of lagoon berms and site access roads
- Removal of lagoon sludge
- Replacement of lagoon baffle curtain
- Regrading of overland flow terraces and renovation of distribution system pipes and valves
- Inspection and repair of overland flow storage tank
- Replacement of all sampling equipment including flow meters and composite samplers
- Rehabilitation of backup generator and other electrical panels

The current flow measurement and monitoring project should continue after the WWTP has been upgraded. The effluent flow monitoring program is needed for NPDES compliance and would provide long-term assessment of the disposal system capacity and effluent quality to defend site-specific water quality standards. Data from an ongoing monitoring program can also help dictate the timeline for future expansions to the treatment system, not necessarily determined based on current population and economic projections.

7.3 Baza Gardens WWTP

At the time of this report, the Baza Gardens WWTP is undergoing major modifications to meet the requirements of Paragraph 15 of the 2011 Court Order (United States of America, 2011). The court order requires GWA to identify and complete improvements necessary to meet the Baza Gardens WWTP NPDES permit by April 30, 2018. Under the existing contract, only preliminary treatment will occur at the existing Baza Gardens WWTP location. The improvement project will construct a cross-island pipeline which will transfer Baza Gardens flows to the Agat-Santa Rita WWTP to complete treatment and disposal. Based on the current design criteria, the capacity of the cross-island pipeline will be adequate through 2035 flows.

The following section describes the existing conditions, regulatory requirements, wastewater characteristics, and recommended improvement projects at the Baza Gardens WWTP.

7.3.1 Existing Conditions

Constructed in 1975, the Baza Gardens WWTP is a packaged treatment unit manufactured by Smith & Loveless. The outer wall and floor of the packaged treatment unit are constructed of reinforced concrete, and the inner walls are steel. The plant uses a single process train utilizing an extended aeration activated sludge process to meet its design secondary treatment objective.

The wastewater influent enters the headworks and passes through an aerated grit chamber followed by a comminutor. If flow exceeds the comminutor capacity, a channel equipped with a manually-cleaned bar rack allows de-gritted wastewater to bypass the comminutor. Once the wastewater enters the aeration section, it is aerated and mixed with return activated sludge (RAS). Figure 7-10 shows the aeration section with the secondary clarifier in the middle of the structure. Mixed liquor from the aeration tank flows into the secondary clarifier, and then into the chlorine contact tank. Chlorination is currently not practiced at the WWTP.

Figure 7-10. Baza Gardens WWTP (June 2013)

Waste activated sludge (WAS) is stabilized in the aerobic digestion section before being pumped into a tanker truck and hauled to the Hagåtña WWTP for digestion and dewatering. Final dewatered cake disposal is at the Layon Landfill.

Wastewater effluent is ultimately discharged into the Togcha River, which follows a two-mile course before flowing into the Pacific Ocean.

Table 7-8 lists the Baza Gardens WWTP's main characteristics.

Table 7-8. Baza Gardens WWTP Characteristics		
Parameter		Value
WWTP design capacity		0.60 mgd
Overall tank dimensions	Inside diameter	98.5 feet
	Top rim elevation	204.0 feet mean sea level
	Side wall height	16.5 feet
Aeration section normal side water depth		15.25 feet
Aeration section volume (approximate)		580,000 gallons
Clarifier section surface area (approximate)		875 $feet^2$
Aerobic digester section volume (approximate)		130,000 gallons
Chlorine contact section volume (approximate)		17,500 gallons

The plant's flow meters were not operational during a portion of the 5-year reporting period used for this report, from October 2011 to September of 2016. A daily stick measurement was used to estimate daily flow until the influent flow meter was replaced in early 2012. The new influent flow meter is an Isco area velocity meter that is inserted in a pipe immediately downstream of the comminutor.

7.3.1.1 Existing WWTP Processes Assessment

GWA repaired structural deficiencies in the steel walls that separate individual sections of the WWTP. However, the WWTP processes (and their operation) are inadequate for the current permit requirements:

- The aeration section of the WWTP has inadequate mixing to maintain the activated sludge in suspension. As a result, sludge accumulates in the bottom of the aeration section and the system is operating more like a partial-mix aerated lagoon rather than a completely-mixed activated sludge process. GWA regularly removes sludge from the tank; nevertheless, the accumulation of sludge reduces the aeration section volume, decreasing the system's detention time.
- The currently recommended secondary clarifier overflow rates for extended aeration package plants are 600 gallons per square foot per day based on peak flow, and 300 gallons per square foot per day based on average flow (Crites and Tchobanoglous, 1998). Influent flows resulting in overflow rates higher than these can be expected to result in poor effluent quality. The plant currently experiences flows higher than 1.0 mgd (equivalent to an overflow rate of 1,150 gallons per square foot) several weeks a year. 1,150 gallons per square foot is almost twice the recommended design overflow rate.
- The WWTP includes a chlorine contact section, but chlorine is not added. A viable disinfection process is required to achieve compliance with the E. coli and fecal coliform limits in the NPDES permit.
- The existing WWTP process was designed to remove organic materials and suspended solids from the wastewater, as measured by BOD_5 and TSS. The treatment system was not designed to meet the N and P requirements in the existing NPDES permit.

In addition, the facility generally lacks the process redundancy needed to properly maintain the equipment.

7.3.2 Regulatory Requirements

Regulatory requirements that apply to the Baza Gardens WWTP are described below.

7.3.2.1 NPDES Permit

The facility currently discharges effluent to the Togcha River, and is regulated by an NPDES permit number GU0020095 issued on August 19, 2015, and valid until August 31, 2018.

The NPDES permit allows a monthly average discharge of 0.60 mgd into the Togcha River, which is considered a Category S-3 surface water. Category S-3 water is defined in the GWQS as surface water primarily used for commercial, agricultural, and industrial activities. Table 7-9 summarizes the NPDES requirements for the Baza Gardens WWTP, as presented in permit No. GU0020095.

Table 7-9. Baza Gardens WWTP NPDES Effluent Limitations

Parameter	Units	Average Monthly	Average Weekly	Maximum Daily	Monitoring Frequency	Monitoring Sample Type
Flow rate	mgd	0.60	--	--	Continuous	Metered
Temperature	°C	--	--	Monitoring only	Weekly	Discrete
Total chlorine residual	µg/L	9	--	19	Weekly	Discrete
pH	Standard units	Between 6.5 and 8.5 at all times			Weekly	Discrete
BOD_5	mg/L	30	45	--	Weekly	24-hr. composite
	lbs/day	150	225	--		
TSS	mg/L	30	45	--	Weekly	24-hr. composite
	lbs/day	150	225	--		
Enterococcus	CFU/ 100mL	33	--	108	Monthly	Discrete
Dissolved oxygen	mg/L	--	--	Monitoring only	Monthly	Discrete
Nitrate-N (NO_4-N)	mg/L	0.41	--	0.82	Quarterly	24-hr. composite
Ammonia-N	mg/L	Monitoring only	--	Monitoring only	Quarterly	24-hr. composite
Ammonia impact ratio	Ratio	1.0			Quarterly	24-hr. composite
Orthophosphate (PO_4-P)	mg/L	0.08	--	0.16	--	--
Oil and grease	mg/L	10	--	15	--	--
Chronic toxicity	Pass/fail	Pass			Once per permit term	24-hr. composite
Priority pollutant scan	µg/L	Monitoring only			Once per permit term	24-hr. composite

7.3.2.2 Court Order and Additional Considerations

The Baza Gardens WWTP currently discharges to the adjacent Togcha River, which is classified as a category S-3 (low) surface water in the GWQS. Table 7-10 lists the numeric water quality standards for nutrients in S-3 waters. The current NPDES permit considers these water quality standards within the discharge limits. The WWTP discharge does not have an authorized mixing zone; therefore, effluent must meet the water quality standards at the point of discharge.

Table 7-10. Numeric Water Quality Standards for Nutrients in S-3 Waters

Parameter	Water Quality Standard
Orthophosphate	0.10 mg/L
Nitrate – N	0.50 mg/L
Ammonia N – acute	2.01 mg/L
Ammonia N – chronic	0.37 mg/L

The Water Environment Research Foundation has studied WWTPs designed for advanced nutrient removal to meet low effluent limits (Parker, et al., 2011). The study identified WWTPs that could reliably achieve a monthly maximum effluent concentration of 3.0 mg/L total N or 0.1 mg/L total P, but no facility could meet both N and P limits simultaneously. Therefore, the water quality standards for discharge to the Togcha River without a mixing zone appear to be beyond the limits of current technology. The GWQS allows the use of mixing zones to achieve water quality standards in S-3 waters after a "thorough study to assess the consequences of the effluent on the environment, and approval of an Environmental Impact Statement". A mixing zone must be approved by the Guam Environmental Protection Agency (EPA) with concurrence of USEPA. A mixing zone must also have zone of passage around it for fish and wildlife. The small size of the Togcha River and intermittent nature of flow in the water body make it a poor candidate for achieving reliable dilution of effluent via a mixing zone while maintaining a zone of passage in the stream bed.

GWA was required by the 2011 Court Order to prepare a WSE identifying improvements "that will achieve consistent compliance with the WWTP's NPDES permit" (United States of America, 2011). GWA completed the evaluation and submitted it to USEPA by the court-ordered deadline of December 31, 2013. The evaluation identified transferring Baza Gardens wastewater to the Agat-Santa Rita WWTP as the best alternative to achieve compliance. GWA is currently implementing the cross-island force main project, including pump stations, and preliminary treatment at the existing Baza Gardens WWTP. The improvements project is expected to be completed by the court-ordered deadline of April 30, 2018.

7.3.3 Wastewater Characteristics

Characteristics of the wastewater flow at Baza Gardens WWTP are described below.

7.3.3.1 Historical Flows and Loads

Sampling data for the Baza Gardens WWTP is reported via DMRs. Reporting years for the Baza Gardens WWTP begin in October of the previous year and end in September. The historical flow characteristics described are based on the most recent 5-year reporting period from October 2011 through September 2016, representing the 2012–2016 reporting years.

Over the 5-year period beginning in October 2011, the average influent BOD_5 was 85 mg/L, with a monthly range of 23 to 207 mg/L. The average TSS for the same period was 183 mg/L, with a monthly range of 13 to 1,729 mg/L. Both average influent BOD_5 and TSS concentrations are low for typical domestic wastewater, potentially indicating considerable infiltration into the collection system. The average BOD_5 is significantly lower than TSS indicating the wastewater may also be impacted by factors other than dilution from I/I.

7.3.3.1.1 Historical Influent Flows

Until early 2012, influent flows at the WWTP were estimated by measuring the water level at a v-notch weir at the headworks. In early 2012, GWA installed an Isco area velocity meter in the pipeline immediately following the comminutor—the only available location. Based on the flows reported in the Baza Gardens DMRs, the annual average influent flow at the Baza Gardens WWTP for the 2012–2016 reporting years was 0.167, 0.066, 0.099, 0.225, and 0.225 mgd, respectively, with an overall average of 0.156 mgd.

The accuracy of the flow meter installed downstream of the comminutor was tested by comparing the flow data report in the DMR with a six-week collection system flow monitoring survey. Figure 7-9 compares the two flow measurements. During the first half of the 6-week study period, more than 13 inches of rain were recorded, and the flow survey clearly reflects the expected increase in flows. However, the flow meter at the WWTP appears to be nonfunctional, recording the same 0.083 mgd flow for the entire 3-week storm event. During the second half of the study period, rainfall was minimal and the two flows exhibited a similar pattern. However, flows recorded at the plant and reported in the DMR are approximately half of the flow survey values.

Turbulent hydraulic conditions downstream of the comminutor can create inaccurate flow measurements. Currently, the flow is not calibrated on a regular basis. Given the inconsistent measurements at the WWTP, hydraulic model results have been relied upon to estimate existing and future flows at the WWTP. Existing WWTP influent flows measured during the flow meter survey (performed for the hydraulic model) are compared to recorded values from the plant flow meter in Figure 7-11.

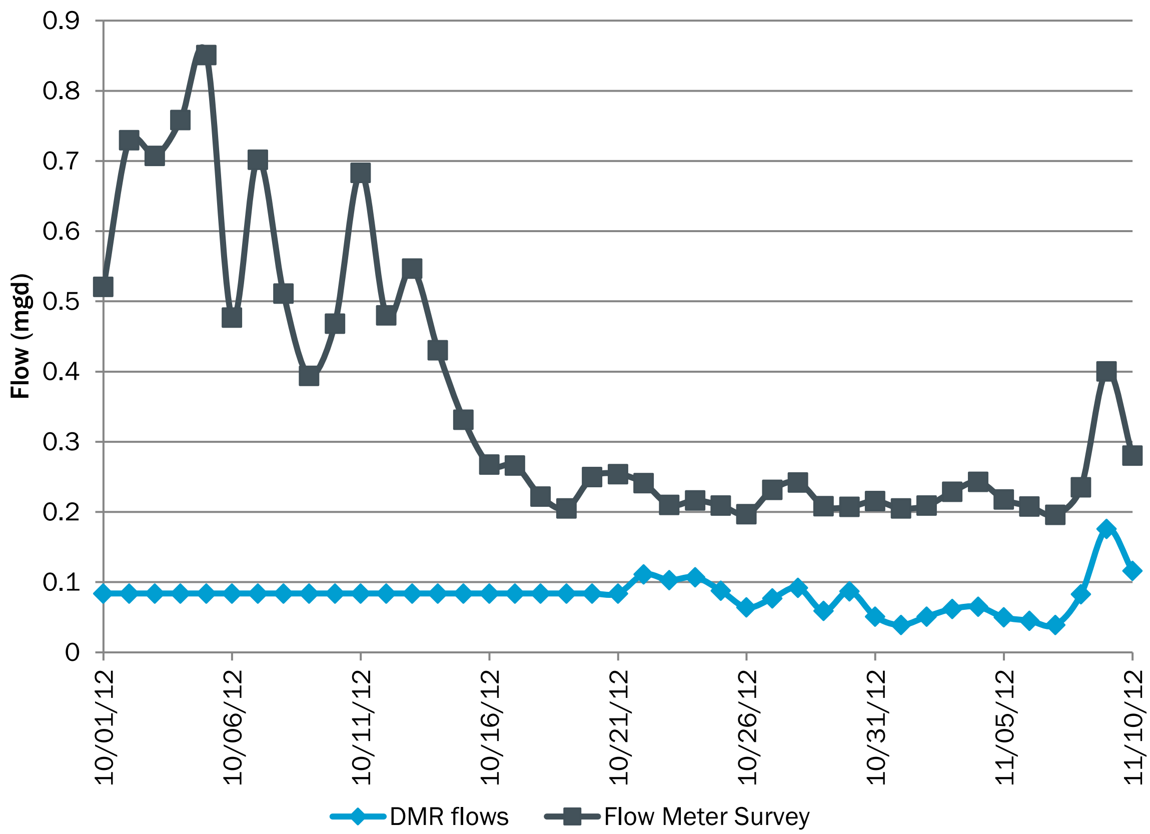

Figure 7-11. Baza Gardens WWTP Influent Flow Comparison

For small treatment plants, the ratio of peak day dry weather flow rates to average dry weather flow rates is between 1.8 and 2.8 (Crites and Tchobanoglous, 1998). Using a typical peaking factor value of 2.5, the peak day dry weather flow at the Baza Gardens WWTP is estimated to be 0.49 mgd. The peak dry weather flow value for the Baza Gardens WWTP is within the current design rate of 0.6 mgd.

Typical average daily flow rates in the United States range between 50 gallons gpcd in rural areas and 120 gpcd in typical urban areas (Tchobanoglous et al, 2003). An estimated 1,130 households are connected to the Baza Gardens WWTP collection system, based on the available GIS and water meter data. The 2010 U.S. Census Bureau estimates the average number of individuals per household in Guam at 3.15 (Guam Statistical Yearbook, 2011). The average 0.196 mgd influent flow at the Baza Gardens WWTP represents an estimated per capita flow of 62 gallons. This value is within the above-cited typical rural area values.

7.3.3.1.2 Historical Influent Concentration

The DMRs for Baza Gardens WWTP include both influent and effluent weekly sampling. As mentioned earlier, the average influent BOD_5 concentration for the 5-year period of 2012–2016 was 85 mg/L, and the average TSS concentration was 183 mg/L. Both values are relatively low compared to typical values for domestic wastewater. The influent may be diluted due to I/I into the system, or the reason for the variations could involve sampling technique variations and inconsistencies in flow measurements.

Figure 7-12 shows the average influent BOD_5 concentration as reported in the WWTP's DMRs during the 2012–2016 reporting period, along with the typical BOD_5 concentration range for domestic wastewater (Tchobanoglous et al, 2003). The graph shows that the influent BOD_5 concentration is consistently low.

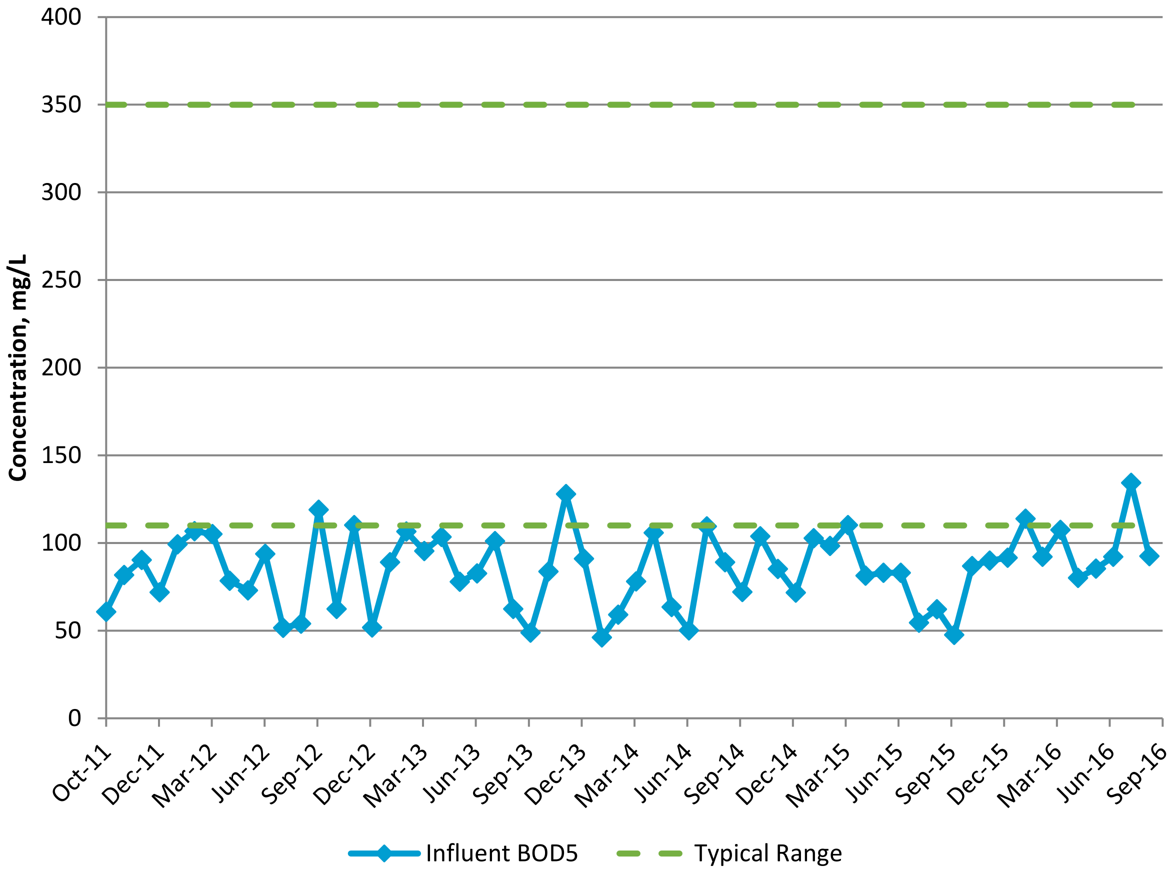

Figure 7-12. Baza Gardens WWTP Influent BOD_5 Concentration

Figure 7-13 shows the measured average influent TSS concentration during the 2012–2016 reporting period, along with the typical TSS concentration range for domestic wastewater (Tchobanoglous et al, 2003). The graph shows that the influent TSS concentration is often within normal values.

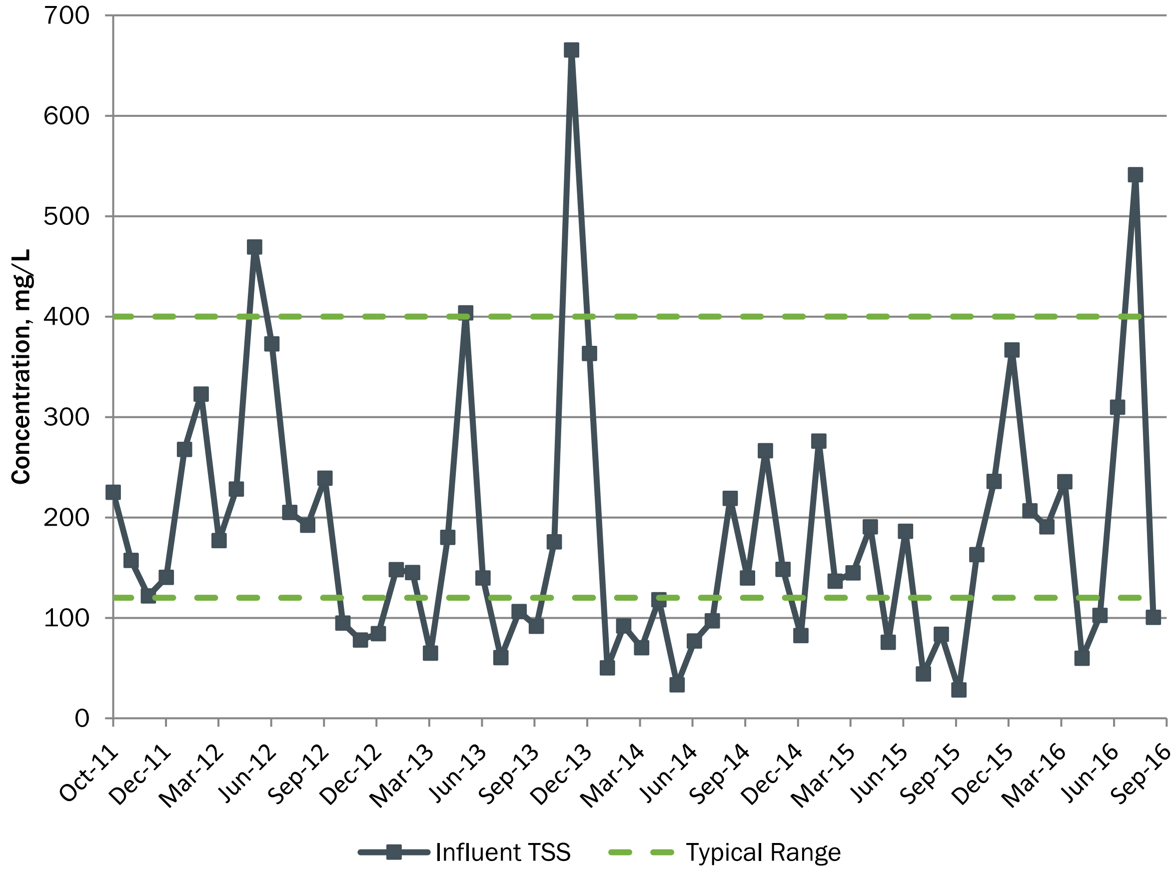

Figure 7-13. Baza Gardens WWTP Influent TSS Concentration

7.3.3.2 Effluent Characteristics

During the same 5-year period, sampled effluent as recorded in the monthly DMRs rarely met the discharge requirements. The most commonly exceeded parameter limits in the samples were E. coli and nutrients (P and N).

Figure 7-14 shows the effluent BOD_5 and TSS sampling results. The average effluent BOD_5 in the same period was 22 mg/L, while the average effluent TSS was 17 mg/L. Monthly BOD_5 limits (30 mg/L) were exceeded 10 times during the 2012–2016 reporting years, while TSS limits (30 mg/L) were exceeded four times.

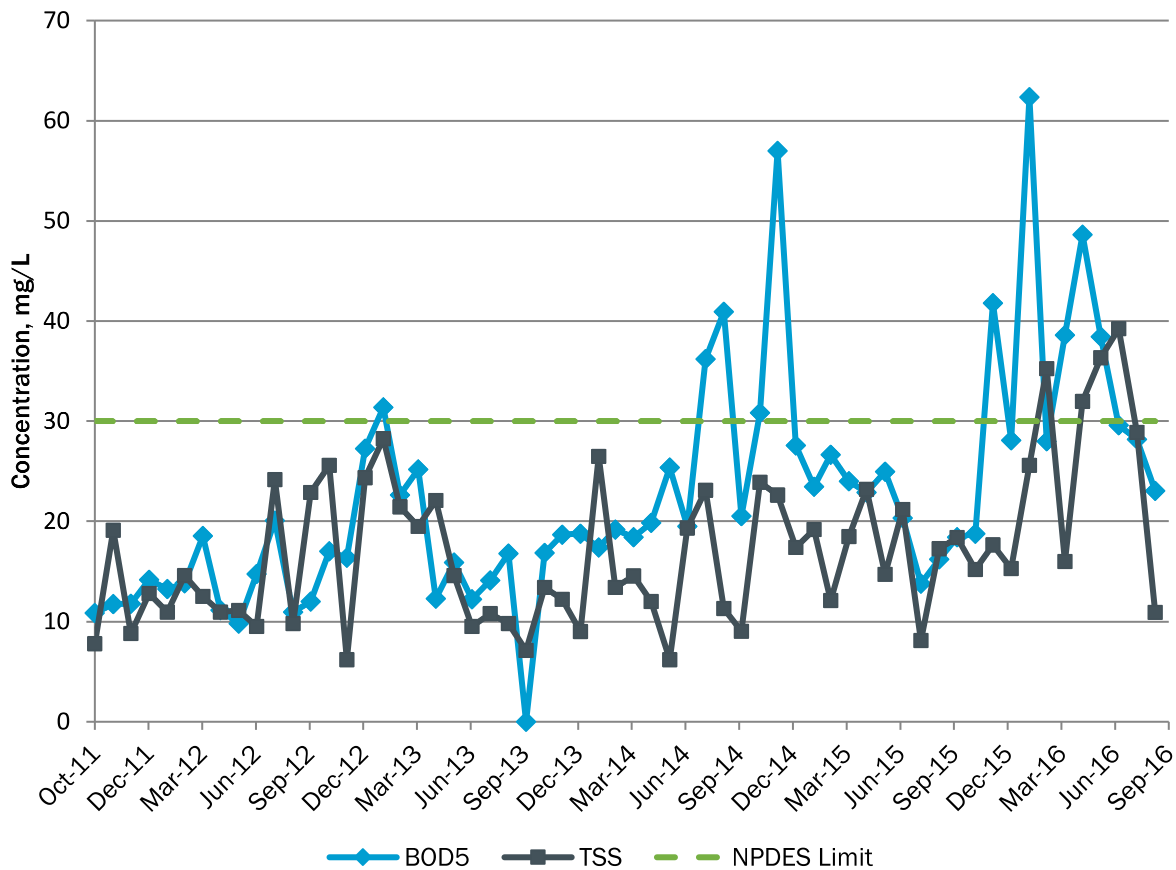

Figure 7-14. Baza Gardens WWTP Effluent BOD_5 and TSS Results

Figure 7-15 shows the monthly average effluent total N and ammonia weekly sampling results and Figure 7-16 shows the effluent nitrate and orthophosphate results.

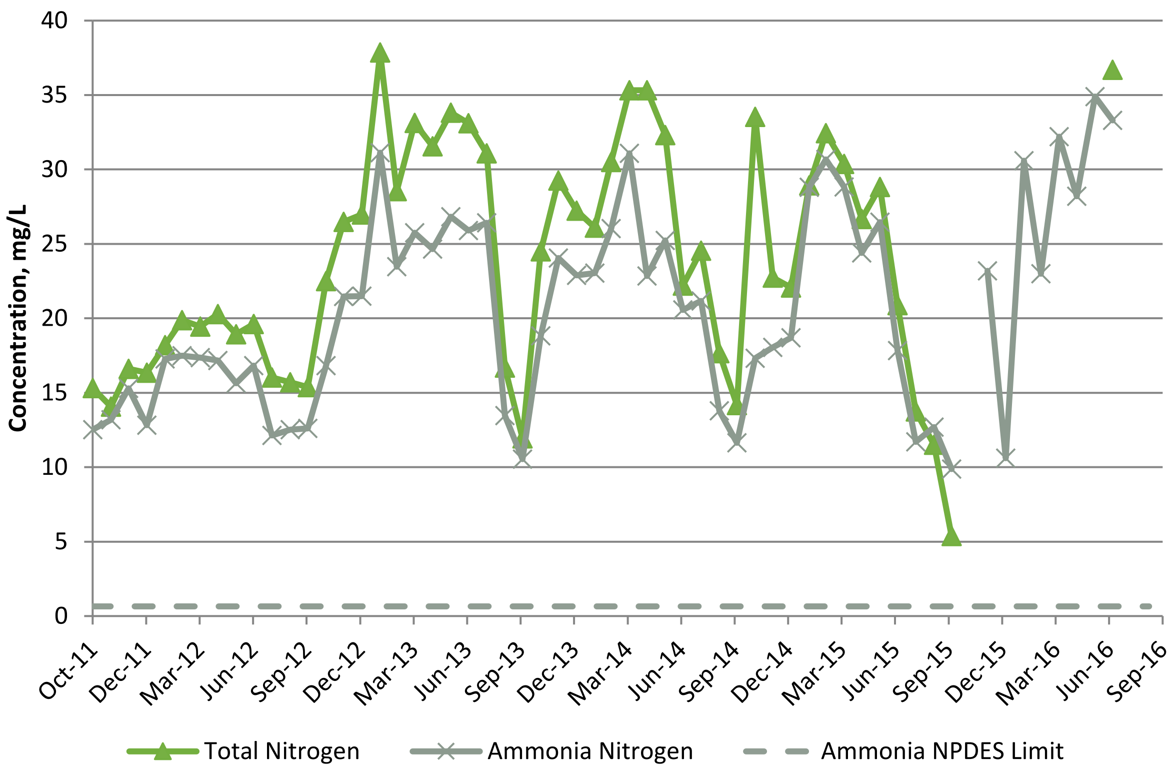

Figure 7-15. Baza Gardens WWTP Effluent Total N and Ammonia-N Results

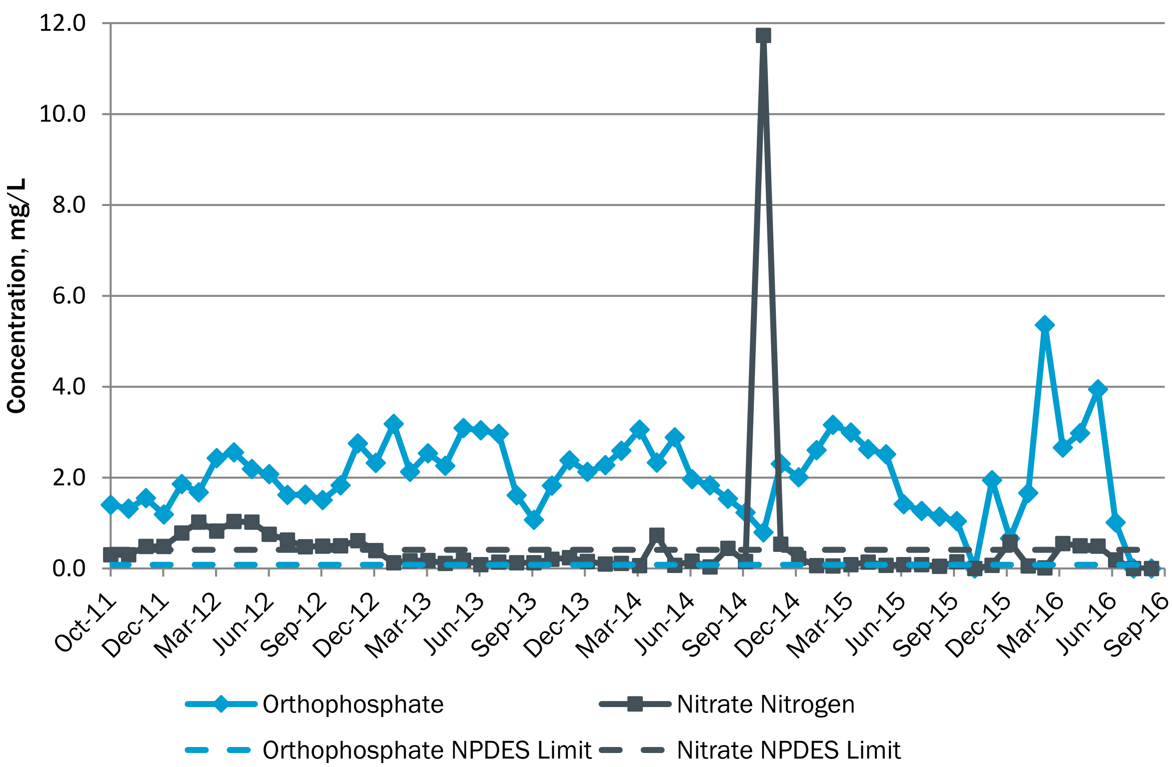

Figure 7-16. Baza Gardens WWTP Effluent Nitrate-N and Orthophosphate Results

7.3.3.3 Flow and Load Projections

Influent flow and load projections are presented below.

7.3.3.3.1 Projected Flows

Future flows at Baza Gardens WWTP were predicted using two methods. Future dry weather flow rates were estimated based on projected population growth, and peak day and peak hour wet weather flow rates were predicted using the hydraulic model developed for the Baza Gardens sub-basins. The 5-year, 24-hour storm event was used to estimate peak wet weather flows.

Dry Weather Flows

To predict future dry weather flows, it was assumed that average dry weather flows will increase at the same rate as population growth. This approach ignores any improvements to reduce I/I or increase water conservation, ensuring that the treatment system will be designed with ample capacity for future growth. The peak dry weather flows were calculated using the previously noted 2.5 typical peaking factor.

Figure 7-17 shows average dry weather flow and peak day dry weather flow projections for the Baza Gardens WWTP.

Collection system improvements and public education measures to reduce I/I could reduce wet weather flows to the WWTP. However, as previously discussed, no I/I reduction factors were incorporated into the derivation of future flow projections.

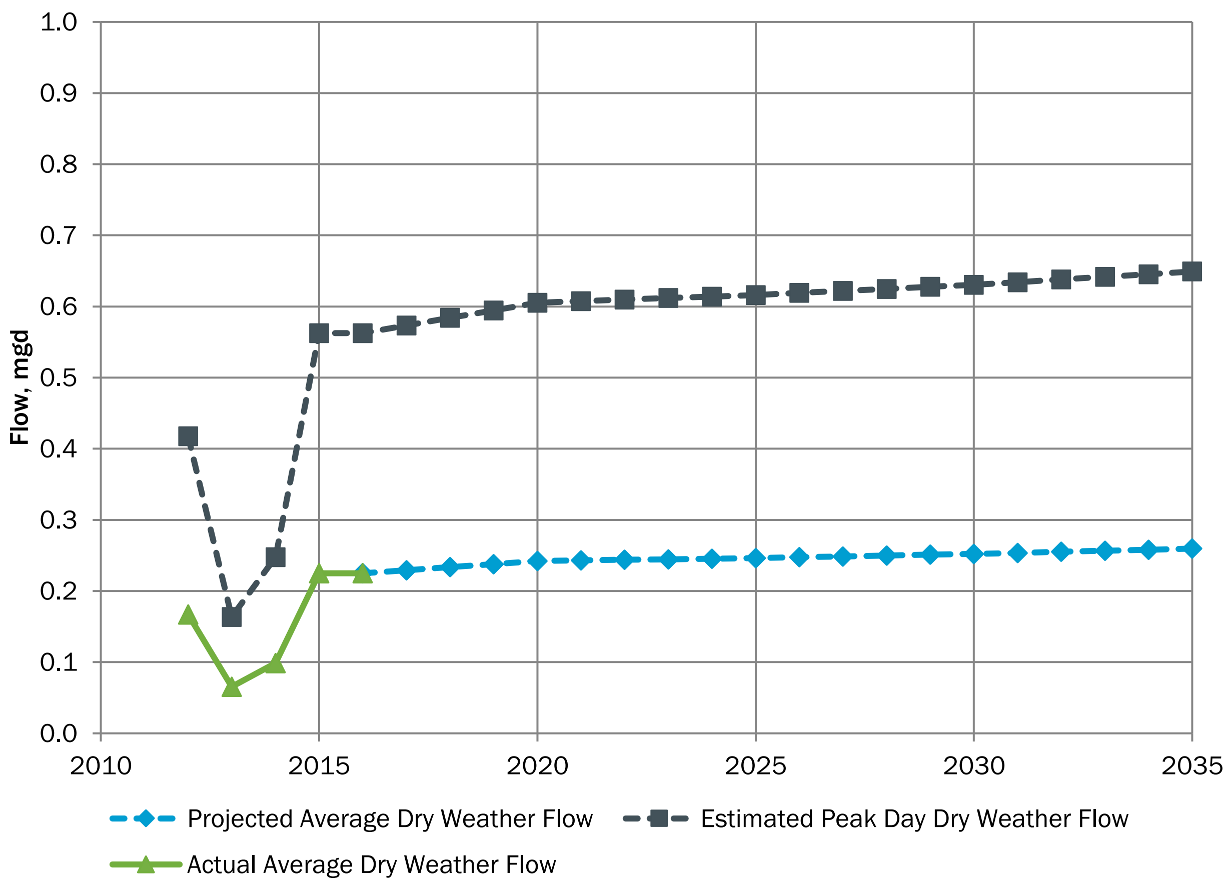

Figure 7-17. Baza Gardens WWTP Dry Weather Flow Projections

Table 7-11 summarizes the Baza Gardens WWTP dry weather flows for the planning period.

Table 7-11. Baza Gardens WWTP Planning Period Dry Weather Flows					
Description	Existing	2020	2025	2030	2035
Average Dry Weather Flow (mgd)	0.156 mgd	0.242	0.246	0.252	0.260
Peak Day Dry Weather Flow (mgd)	0.39 mgd	0.60	0.62	0.63	0.65

Peak Wet Weather Flows

The hydraulic model previously described was used to develop the peak day and peak hour wet weather influent flow projections. Figure 7-18 presents the peak day and peak hour wet weather flow projections for the Baza Gardens WWTP.

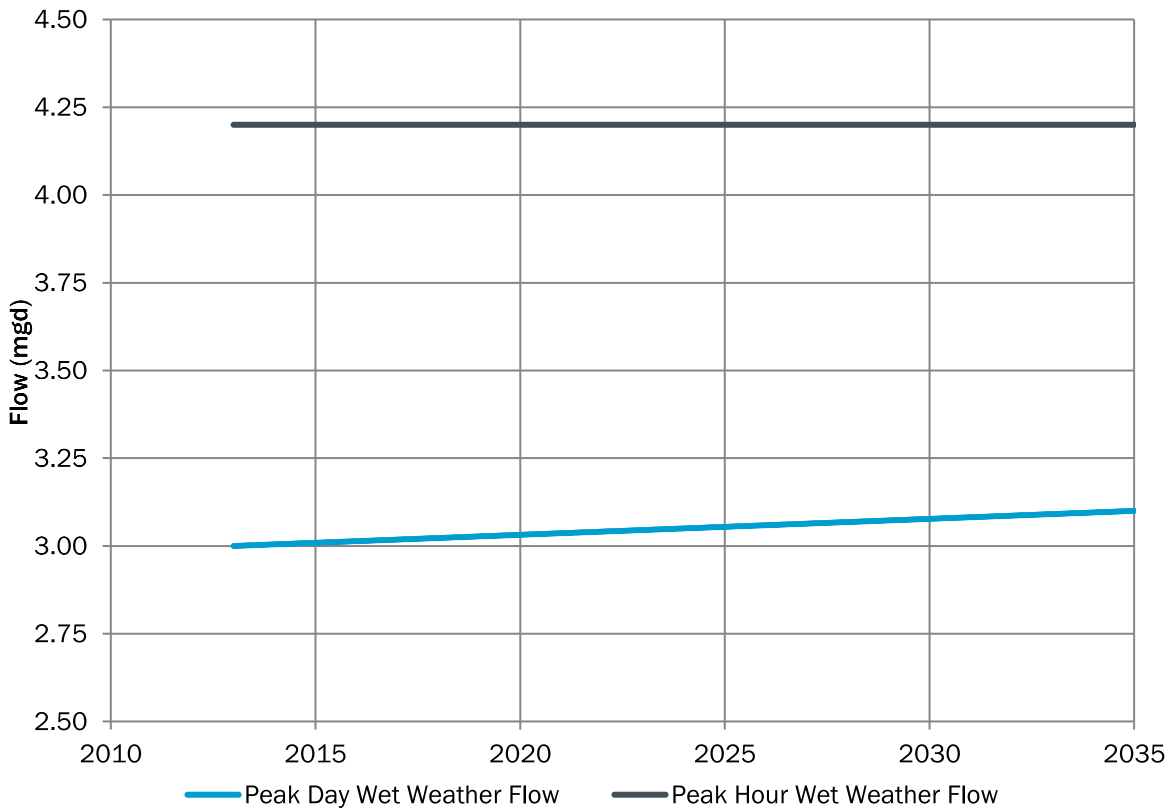

Figure 7-18. Baza Gardens WWTP Wet Weather Flow Projections

Table 7-12 summarizes the Baza Gardens WWTP wet weather flows for the planning period.

Table 7-12. Baza Gardens WWTP Planning Period Wet Weather Flows		
Description	Existing	2035
Peak day wet weather flow	3.0 mgd	3.1 mgd
Peak hour wet weather flow	4.2 mgd	4.2 mgd

Peak hour wet weather flow is not expected to increase during the planning period, as it is constricted by the capacity of the Talofofo pump station. The 5-year, 24-hour storm event will cause a longer peak flow in 2035, but the value of the peak flow is expected to remain the same.

7.3.3.3.2 Projected Loads

Based on the 2008–2016 data, both the average influent BOD_5 and TSS values were low compared to typical values for domestic wastewater flows. The future BOD_5 and TSS mass loads were estimated based on typical U.S. concentrations. For planning purposes, Baza Gardens WWTP average influent strength was assumed to be 200 mg/L BOD_5 and 250 mg/L TSS. These concentrations are higher than the average influent concentrations measured during the past six years, but using higher influent concentrations will ensure that adequate treatment capacity is provided. The chosen concentrations were then applied to the projected influent flow at the plant to estimate projected mass loading. Figure 7-19 presents the projected mass loads for both BOD_5 and TSS through the year 2035.

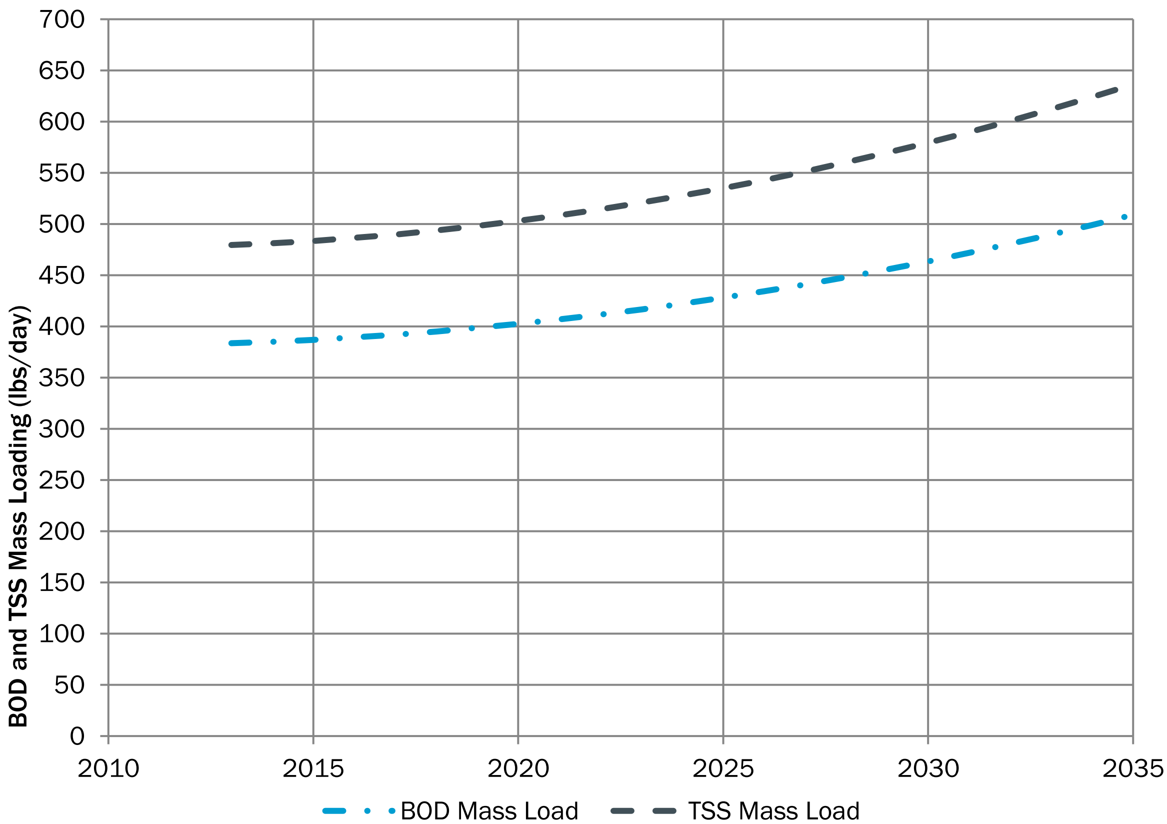

Figure 7-19. Baza Gardens WWTP Mass Load Projections

7.3.3.3.3 Planning Criteria

Table 7-13 summarizes the projected flows and influent concentrations that will be used for planning purposes to accommodate the wastewater needs of the Baza Gardens community through 2035.

Table 7-13. Baza Gardens WWTP Year 2035 Planning Criteria	
Parameter	**Value**
Average dry weather flow	0.31 mgd
Peak day dry weather flow	0.58 mgd
Peak day wet weather flow	3.1 mgd
Peak hour wet weather flow	4.2 mgd
Average dry weather influent BOD_5 concentration	200 mg/L
Average BOD_5 load	509 lbs/day
Average dry weather influent TSS concentration	250 mg/L
Average TSS load	636 lbs/day

The new Agat-Santa Rita WWTP will receive the Baza Gardens effluent after the cross-island pipeline project is completed. The above values were used in the basis of design for the new Agat-Santa Rita WWTP to ensure adequate treatment was provided.

7.3.4 Recommended Improvement Projects

Recommended improvement projects for Baza Gardens WWTP are described below.

7.3.4.1 2011 Court Order Projects

Since the 2006 WRMP, GWA has completed several projects in accordance with the 2011 Court Order (United States of America, 2011). No projects outside of the Court Order were completed for the Baza Gardens WWTP.

Initially, GWA completed a Court Order project that repaired structural deficiencies at the Baza Gardens WWTP.

In April 2014, GWA submitted to the USEPA a WSE for the Baza Gardens WWTP, collection system, and conveyance system in accordance with Paragraph 13 of the 2011 Court Order. The WSE report and consequent supplement identified a plan to avoid having to meet the NPDES requirements at the plant by pumping Baza Gardens wastewater to the new Agat-Santa Rita WWTP. This project, referred to as the Cross-Island project, is currently under construction and will bring the Baza Gardens WWTP in compliance with the 2011 Court Order, paragraph 14 which states that:

> *By April 30, 2018, GWA shall complete the improvements identified in the approved plan required by Paragraph 13 and achieve consistent compliance with the Baza Gardens WWTP's NPDES permit, ensure that solids generated by the WWTP are adequately stabilized and dewatered at the Baza Gardens WWTP, and comply with the sludge and biosolids requirements in 40 C.F.R. Part 503. GWA shall also meet the following interim compliance milestones:*
>
> *a. By October 31, 2015, GWA shall execute a design contract and issue a notice to proceed with the design.*
>
> *b. By October 31, 2016, GWA shall execute a construction contract and issue a notice to proceed with construction.*

The Cross-Island project will transform the Baza Garden WWTP into an equalization basin and pump station system equipped with preliminary treatment. Preliminary treatment will remove rags and grit from the wastewater and ensure easier maintenance for the entire system.

No other improvement projects were completed since the 2006 WRMP.

7.3.4.2 Recommended Improvement Projects

The redesigned Baza Gardens wastewater system will require regular maintenance of the pump stations and preliminary treatment, but no improvement projects are expected in the near future.

In addition to adequate maintenance, a carefully planned replacement program is recommended for all major equipment. The planned rehabilitation should occur every 15 years depending on the equipment and should include:

- Replacement or refurbishment of mechanical equipment and controls
- Renovations of the electrical system
- Replacement of force main valves and appurtenances, as necessary
- Rehabilitation of pump station buildings (or enclosures) and back-up generator buildings, when necessary
- Rehabilitation of back-up generators

It is recommended that a flow measurement and monitoring project be implemented to provide long-term assessment of the system capacity and to help dictate the timeline for future expansions.

7.4 Agat-Santa Rita WWTP

The following section describes the existing conditions, regulatory requirements, wastewater characteristics, and recommended improvement projects at the Agat-Santa Rita WWTP.

7.4.1 Existing Conditions

Existing conditions at Agat-Santa Rita WWTP are described below.

7.4.1.1 Existing Agat-Santa Rita WWTP

Constructed in 1972, the old Agat-Santa Rita WWTP was a packaged plant designed to provide secondary treatment through a single train contact stabilization process. Wastewater first enters the headworks, which is comprised of an influent pump station and manual bar screen. Wastewater then enters the treatment tank, which is comprised of contact aeration basins, a clarifier, and an aerobic digester. The effluent pump station conveys treated effluent to the ocean outfall into Tipalao Bay. Agat-Santa Rita effluent combines with the Navy's Apra Harbor flow prior to discharge.

To avoid wastewater backup in the collection system during high flow events, operators bypass the headworks. According to the WSE completed by EA Engineering, Science and Technology, Inc. (dated December 2013, revised September 2014), flow through the bypass pump is not measured and actual flows were only obtained during a temporary flow study at the plant.

For a brief description of the new Agat-Santa Rita Plant currently under construction, see Section7.4.4. Figure 7-20 shows the Agat-Santa Rita WWTP in April 2010.

Figure 7-20. Existing Agat-Santa Rita WWTP (April 2010)

7.4.1.2 New Agat-Santa Rita WWTP

Completion of the new Agat-Santa Rita is pending, with portions of the new WWTP in operation in early 2017. The new plant is located near the existing Tipalao pump station site, as shown in Figure 7-21.

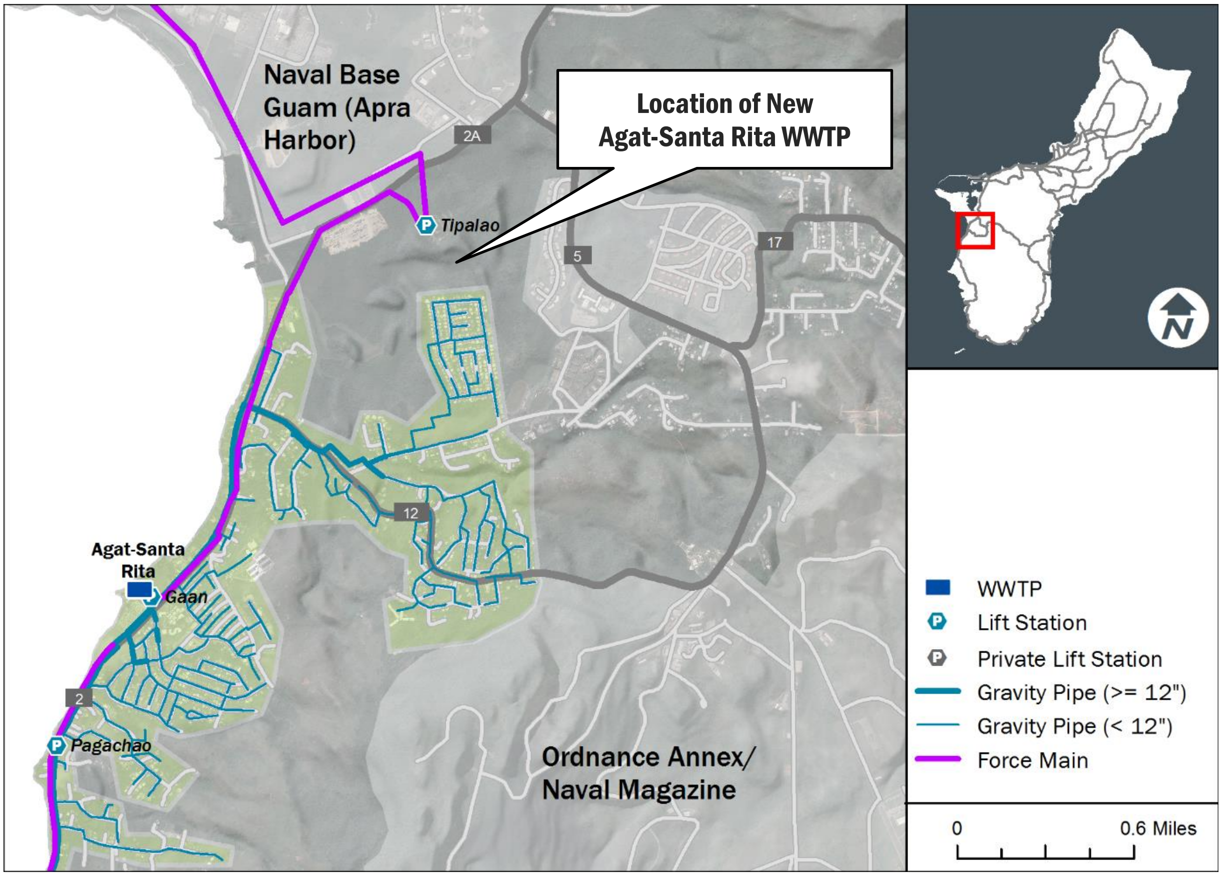

Figure 7-21. New Agat-Santa Rita WWTP Location

New treatment processes include: headworks, an equalization tank, oxidation ditches, secondary clarifiers, and an aerobic digestion process for solids treatment. An ultraviolet (UV) system will be used for disinfection prior to discharging the effluent into Tipalao Bay. The ocean outfall will continue to be shared with the Navy's Apra harbor facility. A summary of the design criteria for the new WWTP is presented in Section 7.4.3.3. A diagram of the new wastewater treatment processes is shown in Figure 7-22. The plant will be fully operational in early 2018.

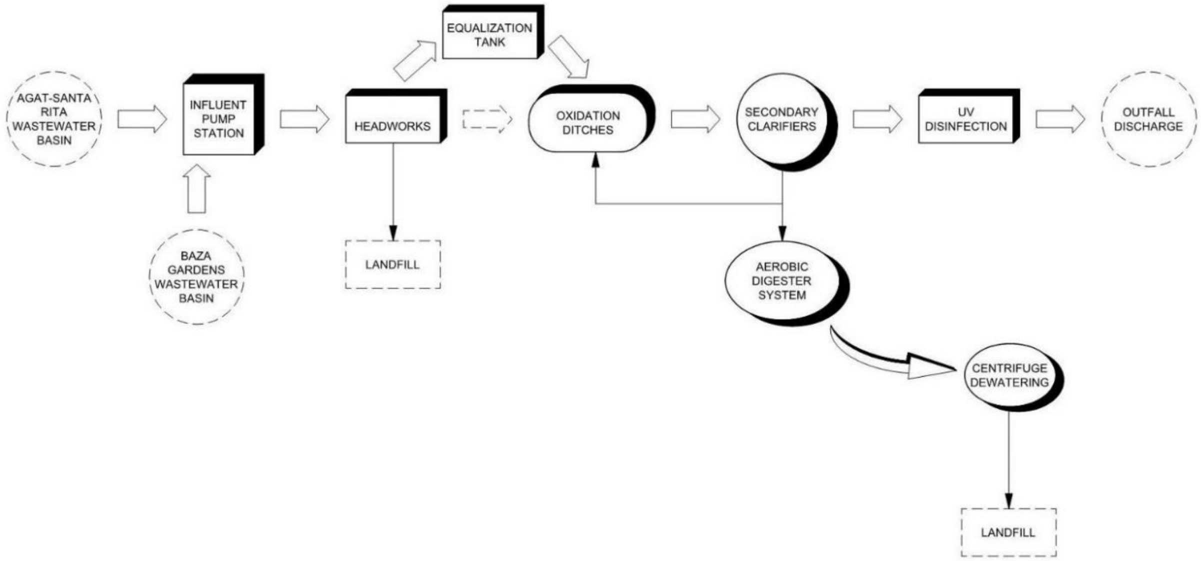

Figure 7-22. New Agat-Santa Rita WWTP Treatment Process Diagram

7.4.2 Regulatory Requirements

Regulatory requirements that apply to Agat-Santa Rita WWTP are described below.

7.4.2.1 NPDES Permit

The facility discharges effluent to Tipalao Bay, and is regulated by an NPDES permit. The most recent permit, No. GU0020222, was issued in November 2017 for the new Agat-Santa Rita WWTP and is valid from January 1, 2018 to December 31, 2022. The new plant is expected to complete construction in early 2018.

The old NPDES permit expired in 2015. The old Agat-Santa Rita WWTP was allowed to operate under the expired permit during the design and construction of the new plant and development of the new permit. The expired permit allowed a monthly average discharge of 0.75 mgd into Tipalao Bay (the Philippine Sea). Table 7-14 summarizes NPDES requirements for the old Agat-Santa Rita WWTP, as presented in the old permit No. GU0020222.

Table 7-14. Agat-Santa Rita WWTP NPDES Effluent Limitations

Parameter	Units	Average Monthly	Average Weekly	Maximum Daily	Instantaneous Minimum	Instantaneous Maximum
Flow Rate	mgd	0.75	--	--	--	--
BOD_5	mg/L	30	45	--	--	--
	lbs/day	188	282	--	--	--
pH	Standard units	--	--	--	6.5	8.5
TSS	mg/L	30	45	--	--	--
	lbs/day	188	282	--	--	--
Fecal coliform	CFU/ 100mL	200	400	--	--	--
Enterococci	CFU/ 100mL	35	--	104	--	--
Total chlorine residual	µg/L	7.5	--	12.3	--	--

Table 7-14. Agat-Santa Rita WWTP NPDES Effluent Limitations						
Parameter	Units	Average Monthly	Average Weekly	Maximum Daily	Instantaneous Minimum	Instantaneous Maximum
	lbs/day	0.05	--	0.08	--	--
Copper	µg/L	2.2	--	4.8	--	--
	lbs/day	0.014	--	0.03	--	--
Nickel	µg/L	8.2	--	13	--	--
	lbs/day	0.051	--	0.081	--	--
Zinc	µg/L	45.8	--	95.0	--	--
	lbs/day	0.29	--	0.59	--	--
Aluminum	µg/L	120	--	200	--	--
	lbs/day	0.75	--	1.25	--	--
Heavy metals	mg/L or µg/L	--	--	Monitoring only	--	--
Pesticides	mg/L or µg/L	--	--	Monitoring only	--	--
4,4-DDE	mg/L or µg/L	--	--	Monitoring only	--	--
4,4-DDD	mg/L or µg/L	--	--	Monitoring only	--	--
Chlordane	µg/L	0.182	--	0.320	--	--
	lbs/day	1.14 x 10-3	--	2.00 x 10-3	--	--
Dieldrin	mg/L or µg/L	--	--	Monitoring only	--	--
Oil and grease	mg/L	10	--	15	--	--
	lbs/day	63	--	94	--	--
Whole effluent toxicity	TUC	67	--	134	--	--
Ammonia	mg/L	--	--	Monitoring only	--	--
Priority pollutant toxic scan	mg/L or µg/L	--	--	Monitoring only	--	--

7.4.2.2 Court Order and Additional Considerations

GWA was required by the 2011 Court Order to prepare a wastewater systems evaluation identifying improvements "that will achieve consistent compliance with the WWTP's NPDES permit" (United States of America, 2011). GWA completed the evaluation and submitted it to USEPA by the court-ordered December 31, 2013 deadline. The evaluation identified designing and constructing a new Agat-Santa Rita WWTP. The new treatment plant is partially in operation and should be complete in early 2018.

Also in December 2013, GWA completed the WSE for the Baza Gardens WWTP and determined that the best course of action for complying with the court order will be to transfer flows from Baza Gardens to the proposed Agat-Santa Rita WWTP. The Baza Gardens Cross-Island project is currently under construction, and the new Agat-Santa Rita WWTP has been designed with sufficient capacity for the additional Baza Gardens wastewater flows.

7.4.3 Wastewater Characteristics

Characteristics of the wastewater flow at the old Agat-Santa Rita WWTP are described below.

7.4.3.1 Historical Flows and Loads

Based on EA's WSE, reported average flow at the Agat-Santa Rita WWTP, according to the 2012 Fiscal Year Discharge Monitoring Reports, was 1.23 mgd. From the same year's data, the average BOD_5 and TSS in the influent wastewater were 57 mg/L and 79 mg/L, respectively. Both BOD_5 and TSS were significantly lower than the U.S. typical low-strength concentrations of 110 mg/L BOD_5 and 120 mg/L TSS (Tchobanoglous, 2003).

The low influent concentrations suggest a high level of dilution due to I/I.

7.4.3.2 Effluent Characteristics

During the same fiscal year, sampled effluent, as recorded in the monthly DMRs, exhibited a low removal rate for both BOD_5 (64 percent removal rate) and TSS (only 32.7 percent). The existing treatment processes cannot adequately treat the amount of flow and flow characteristics currently reaching the Agat-Santa Rita WWTP. Additional effluent NPDES requirements were also exceeded during the period assessed in the WSE.

7.4.3.3 Flow and Load Projections (New WWTP Design Criteria)

The Agat-Santa Rita WSE projected flows and flow characteristics for the Agat-Santa Rita wastewater basin for the next 20 years, the typical planning cycle of a WWTP. Projected flows were based on projected population growth and a 50 percent decrease in I/I due to interim I/I mitigation measures implemented throughout the wastewater collection and conveyance system. The overall design flow for the new plant also included the estimated flows conveyed from Baza Gardens via the Cross-Island Pipeline project.

Table 7-15 summarizes the flow and load characteristics used as the design criteria for Agat-Santa Rita WWTP (as presented in the 2013 WSE).

Table 7-15. Agat-Santa Rita WWTP Year 2035 Planning Criteria

Parameter	Value
Average dry weather flow (Agat-Santa Rita basin only)	1.3 mgd
Average dry weather flow (Agat-Santa Rita + Baza Gardens basins)	1.6 mgd
Peak day wet weather flow (Agat-Santa Rita basin only)	7.8 mgd
Peak day wet weather flow (Agat-Santa Rita + Baza Gardens basins)	9.2 mgd
Average influent BOD_5 concentration (Agat-Santa Rita basin only)	114 mg/L
Average influent TSS concentration	158 mg/L

7.4.4 Recommended Improvement Projects

Recommended improvement projects for the Agat-Santa Rita WWTP are described below.

7.4.4.1 2011 Court Order Projects

Since the 2006 WRMP, GWA has completed several projects in accordance with the 2011 Court Order (United States of America, 2011). No projects outside of the Court Order were completed for the Agat-Santa Rita WWTP.

In December 2013, GWA submitted to the EPA a WSE for the Agat-Santa Rita WWTP, collection system, and conveyance system in accordance with Paragraph 10 of the 2011 Court Order. The WSE report identified improvements needed to meet NPDES requirements at the plant. The improvements are part of the new Agat-Santa Rita WWTP, which is currently under construction. The new plant will bring the Agat-Santa Rita WWTP in compliance with the 2011 Court order, paragraph 11 which states that:

> *By December 31, 2016, GWA shall complete the improvements identified in the approved plan required by Paragraph 10 and achieve consistent compliance with the Agat-Santa Rita WWTP's NPDES permit, eliminate bypasses at the WWTP, ensure that solids generated by the WWTP are adequately stabilized and dewatered at the Agat-Santa Rita WWTP, and comply with the sludge and biosolids requirements in 40 C.F.R. Part 503.*

The new Agat-Santa Rita WWTP is being constructed relatively near the existing WWTP. The new plant is partially complete and currently treating Agat wastewater flows, and the remainder of the facility will be completed in early 2018. When complete, the new plant will include: a new headworks equipped with screens, flow meter, grit removal system, and odor control; a flow equalization system including a 2-MG tank and a pumping system; oxidation ditches; secondary clarifiers; a UV disinfection system; aerobic digesters; gravity belt thickener; centrifuge system; and all appurtenant systems needed for adequate facility operation. The capacity of the new plant is 1.6 mgd, which accounts for Agat-Santa Rita and Baza Gardens projected wastewater flows. The Baza Gardens flows will reach the Agat-Santa Rita WWTP via the Cross-Island project currently under construction.

No other improvement projects were completed since the 2006 WRMP.

7.4.4.2 Recommended Improvement Projects

The new WWTP will require regular maintenance, but no improvement projects are expected in the near future. A WWTP rehabilitation project is recommended after 15 years of operation to include:

- Replacement or refurbishment of mechanical equipment and controls
- Inspection and repair of structures
- Rehabilitation of electrical equipment and control systems
- Rehabilitation of backup generator

7.5 Inarajan WWTP

The following section describes the existing conditions, regulatory requirements, wastewater characteristics, and recommended improvement projects at the Inarajan WWTP.

7.5.1 Existing Conditions

The Inarajan WWTP receives influent from a portion of Inarajan village in the southern region of Guam and leachate from Layon Landfill. The WWTP was built in 1989 as a secondary wastewater treatment facility with a design capacity of 190,000 gpd. Treatment is achieved through an aerated lagoon system, with three of the four existing lagoons currently in operation. The effluent disposal method at the plant is percolation via three basic soil aquifer treatment basins. Figure 7-23 shows the Inarajan WWTP lagoons.

Figure 7-23. Inarajan WWTP Lagoons

The influent pump station sends wastewater to the existing lagoons through a 5-inch force main. The lagoons are aerated with floating mechanical aerators and designed to be operated in series. Only three of the four lagoons are currently in operation. From the lagoons, effluent is conveyed to the percolation basins. Dosing chambers were designed to alternate flow between each of the percolation ponds, and a V-notch weir was implemented to help measure flows out of the lagoons.

The WWTP was designed to allow stabilized solids from the lagoon to be transferred to a decant well for thickening prior to being pumped to the drying beds. The top layer of the decant well water goes back to the lagoons. Dried sludge is raked and transported by trucks to the landfill.

GWA upgraded the WWTP when the Layon Landfill began discharging leachate. The upgrades included new aerators, valves, and electrical system improvements. Figure 7-24 presents a schematic process train flow diagram for the Inarajan WWTP.

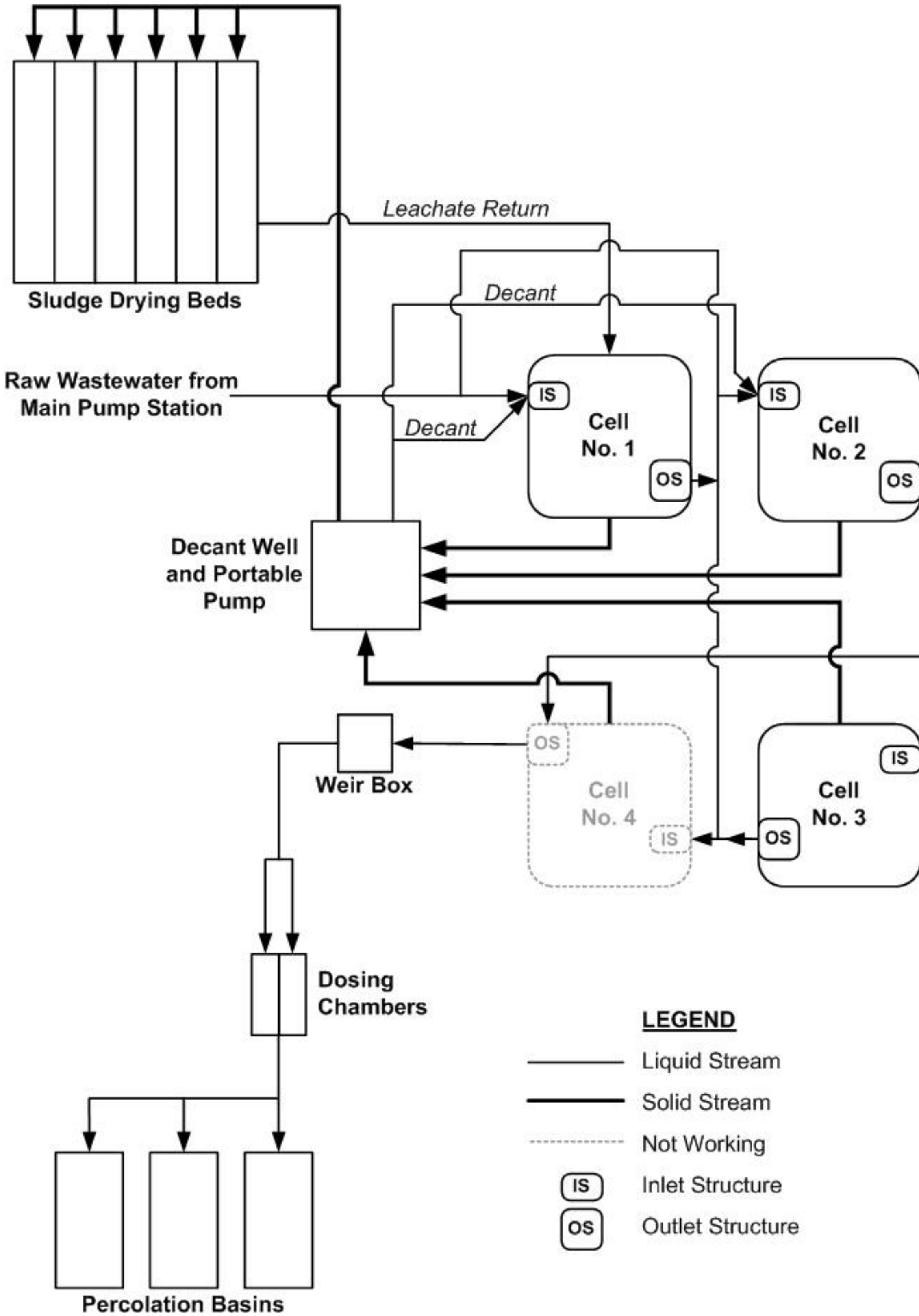

Figure 7-24. Inarajan WWTP Existing WWTP Schematic

7.5.2 Regulatory Requirements

The Inarajan WWTP operates as a zero-discharge facility and does not require an NPDES permit.

7.5.3 Wastewater Flow and Characteristics

GWA does not prepare DMRs for the WWTP because there is no NPDES permit. Neither wastewater characteristics or flow data are available for the plant.

In 2006, estimated wastewater flow to the plant was 70,000 gpd, with an estimated per capita flow of 80 gpd (WRMP, 2006). At an estimated per capita flow of 80 gpd, a third of the Inarajan Village population was estimated to be connected to the WWTP. There is no known development in Inarajan to suggest that sewer connections will increase significantly in the near future. Assuming the number of residents connected to the Inarajan WWTP will grow at a similar rate to the population projections presented in Volume 1, Section 4, influent wastewater flow at the Inarajan WWTP in 2050 will approach 80,000 gpd.

The WWTP also receives leachate from the Layon Landfill. Projected leachate flow for the approximate 40-year life of the landfill was estimated by A-Mehr, Inc. in 2008. Average annual flow for the entire life of the landfill was projected at 11,987 gpd, with projected peak day flow at 39,321 gpd.
Table 7-16 summarizes total flows to the Inarajan WWTP including residential wastewater and Layon landfill leachate. Figure 7-25 shows projected leachate flows and the plant's total average dry weather flow.

Table 7-16. Inarajan WWTP Projected Average Influent Flows

Year	Inarajan Total Population	Inarajan Population Connected to the WWTP [b]	Residential Wastewater Flow (gpd) [d]	Layon Landfill Average Leachate (gpd) [e]	Total Flow (gpd)
2006	2,585 [a]	875	70,000 [c]		70,000
2010	2,273	758	60,613		60,613
2011	2,289	775	61,989	2,768	64,757
2012	2,305	780	62,417	1,383	63,800
2013	2,320	786	62,845	702	63,547
2014	2,336	791	63,272	954	64,226
2015	2,352	796	63,700	6,595	70,295
2020	2,584	875	69,984	7,210	77,194
2025	2,630	890	71,230	11,987	83,217
2030	2,692	911	72,909	11,987	84,896
2035	2,771	938	75,048	11,987	87,035
2040	2,841	962	76,944	11,987	88,931
2045	2,893	979	78,353	11,987	90,340
2050	2,936	994	79,517	11,987	91,504

a. Estimated population based on U.S. Census population of 3,052 in 2000 and 2,273 in 2010.

b. An estimated 33 percent of the Inarajan Village population was connected to the sewer in 2006, based on 2006 WRMP estimated flow of 70,000 gpd at 80 gpd per capita, and an estimated Inarajan total population of 2,585.

c. From 2006 WRMP.

d. Assuming 80 gpd per capita.

e. From 2008 A-Mehr report on Layon Landfill.

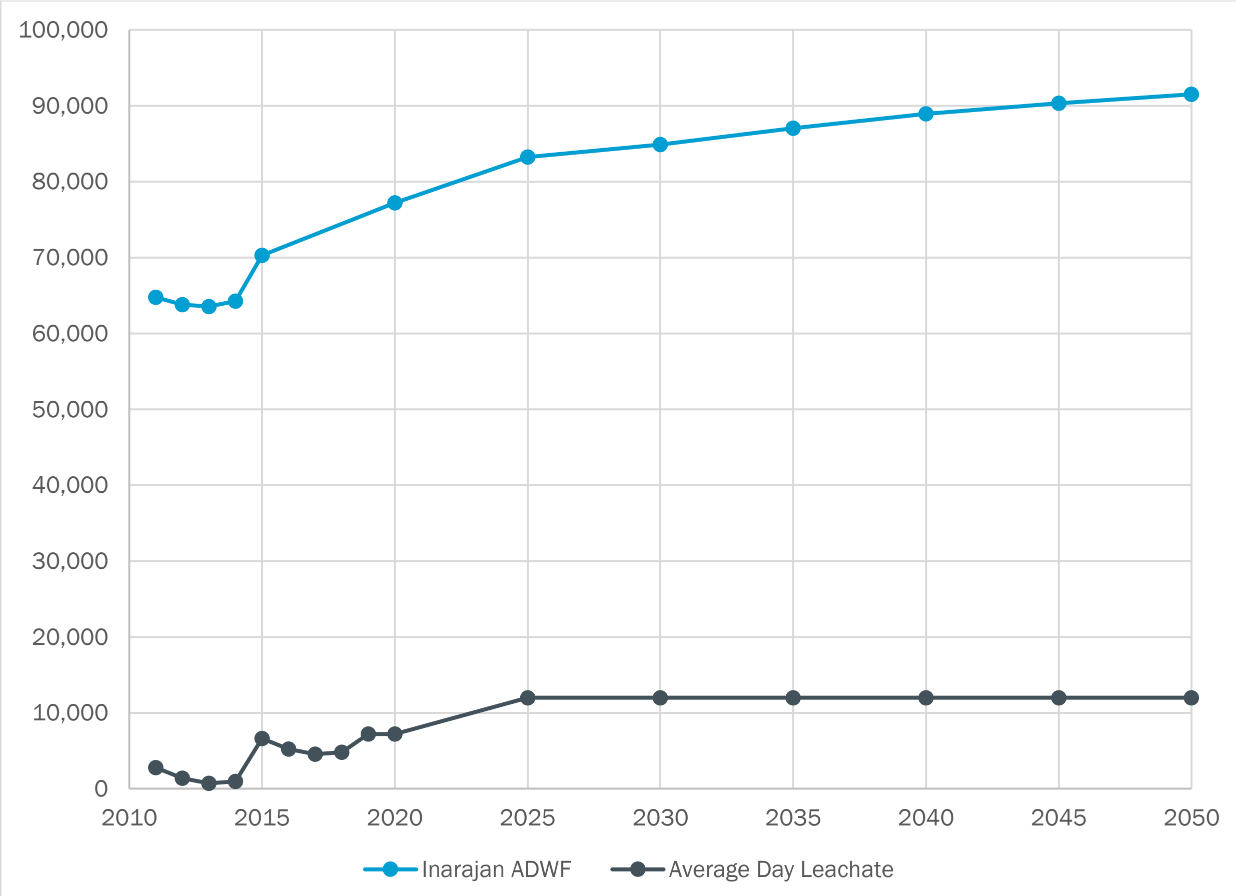

Figure 7-25. Inarajan WWTP Projected Average Dry Weather Flow

By 2050, the average day flow is estimated to remain under 100,000 gpd, operating at less than 50 percent of the WWTP's capacity of 190,000 gpd. The plant does not have a flow meter installed and residential flows were estimated based on population projections. A flow meter should be installed and a capacity analysis should be performed on the WWTP in the future.

7.5.4 Recommended Improvement Projects

The Inarajan WWTP was not mentioned in the 2011 Court Order, which triggered improvements at the majority of GWA's WWTPs. However, as noted for the other WWTPs, routine improvement projects are necessary to maintain the plant in sound operating condition.

A renovation project is recommended at the Inarajan WWTP. The project should include:

- Rehabilitation of concrete structures
- Installation of new electrical and control systems
- Installation of new floating mechanical aerators
- Rehabilitation or replacement of valves and pipe appurtenances
- Installation of a new headworks with automatic screens and influent flow meter
- Implementation of sludge removal

It is also recommended that a flow measurement and monitoring program be implemented to provide long-term assessment of the system capacity and to help dictate the timeline for future expansions.

7.6 Pago-Socio WWTP

The following section describes the existing conditions, regulatory requirements, wastewater characteristics, and recommended improvement projects at the Pago-Socio WWTP.

7.6.1 Existing Conditions

The Pago-Socio WWTP is a packaged aerated treatment system with a series of subsurface percolation pits. The plant was built by a developer as part of the wastewater service for a 16-home community and was later dedicated to GWA for operation and maintenance. The aeration system is not currently operational and a significant number of additional homes have been connected to the treatment plant since the WWTP was first brought online. Figure 7-26 shows the Pago Socio WWTP blower housing.

Figure 7-26. Pago Socio WWTP Blower Housing (2012)

7.6.2 Regulatory Requirements

The Pago-Socio WWTP is a zero-discharge facility and does not require an NPDES permit.

7.6.3 Wastewater Characteristics

Without NPDES permit requirements, no flow or wastewater quality data have been collected for the plant.

7.6.4 Recommended Improvement Projects

GWA plans to convert the existing Pago-Socio WWTP into a pump station. Wastewater flows would be conveyed into an existing nearby wastewater line and subsequently to the Hagåtña WWTP for ultimate treatment and disposal. A 2014 study, *Pago Socio Wastewater Transfer Study*, discussed five alternative routes to connect to the piping on Route 4 (BC, 2014). The alternatives include a combination of force main and gravity piping ranging from 2,660 to 3,600 feet. The new pump station should include a back-up generator. In addition, GWA should develop an adequate maintenance plan and equipment replacement program to ensure long-term operation of the pump station. See Project MP-WW-WWTP-06 in Section 11 for more information on the proposed conversion.

Conversion of the WWTP to a pump station at the Pago-Socio WWTP site should consider area population projections and provide sufficient capacity for the expected 20–30-year growth of the community serviced. The pump station and force main design should also consider the capacity of the receiving WWTP and be coordinated with any Hagåtña WWTP improvements and expansion projects.

7.7 Northern District WWTP

The following section describes the existing conditions, regulatory requirements, wastewater characteristics, and recommended improvement projects at the Northern District WWTP.

7.7.1 Existing Conditions

The Northern District WWTP is in Dededo on the northwestern coast of Guam. The treatment plant was originally commissioned in 1979 and designed as a primary treatment facility. In 2012, the plant was upgraded to a chemically enhanced primary treatment (CEPT) facility. Treated wastewater is discharged to the Philippine Sea near Tanguisson Point via an ocean outfall. The coastal waters off Tanguisson Point are considered “Category M-2 Good” marine waters in the GWQS.

Existing conditions at the Northern District WWTP are further described below. Figure 7-27 shows the Northern District WWTP in February 2016.

Figure 7-27. Northern District WWTP (February 2016)

The facility collects and treats wastewater from the regions of Dededo, Latte Heights, Perez Acres, Ypaopao, Marianas Terrace; the Yigo collection system; portions of Tumon; and other unincorporated subdivisions throughout the Yigo and Dededo municipalities. The service area also includes U.S. military facilities (Air Force and Navy) within the areas of Dededo and Harmon Annex, and Anderson AFB. The Northern District WWTP currently serves a population of approximately 76,000 people.

7.7.1.1 Liquid Treatment

Liquid treatment processes at the Northern District WWTP are described below.

Original Facility

The original treatment plant constructed in 1979 consisted of an influent comminutor, manual bypass bar screen, pre-aeration tank, two aerated grit chambers, two primary clarifiers, effluent flow meter, and a chlorine contact tank.

Table 7-17 summarizes the original WWTP capacity. Comparison of the flow capacity shown with recent data shows that the WWTP currently operates at approximately 47 percent of its original design average flow capacity.

Table 7-17. Original Northern District WWTP Design Capacity	
Description	Value
Average flow capacity	12.0 mgd
Peak hour flow capacity	27.0 mgd
Peak hour peaking factor	2.25

The plant currently experiences a higher peak day peaking factor than the original design peak hour peaking factor. Peak hour peaking factors are always higher than peak day peaking factors. This indicates that I/I into the aging collection system is likely causing higher peak hour wet weather flow surges than contemplated in the original design criteria.

Chemically Enhanced Primary Treatment Modifications

A modification project to convert the facility to a CEPT process to improve primary treatment performance was completed in December 2012. Table 7-18 summarizes the flow capacity and influent characteristics that were used to design the CEPT modifications.

Table 7-18. Flow Capacity and Influent Characteristics for CEPT Design	
Description	Value
Average flow capacity	9.0 mgd
Peak (hour) flow capacity	20.25 mgd
Peak hour peaking factor	2.25
Influent BOD_5 concentration	205 mg/L
Average influent BOD_5 mass load	15,400 lbs/day
Influent TSS concentration	229 mg/L
Average influent TSS mass load	17,200 lbs/day

The treatment plant currently includes the following components for the liquid treatment process:

- Headworks building, incorporating a single automatic screen and manual bar rack.
- Flocculation chambers, one with mechanical aerators and one back-up basin with coarse bubble diffused aeration mixing.
- Two primary clarifiers.
- Primary sludge and scum transfer pumps.
- Chemical storage and feed systems.
- Non-potable water booster pump system.
- Chlorine contact tank (not in use).

Wastewater flows into the headworks from a 42-inch gravity main serving the northern area including Anderson AFB, and a 27-inch force main from the Southern Link pump station that collects wastewater from the southern portion of the service area. The two flows are combined at a junction box located just upstream of the headworks building. Flow from the junction box enters the headworks building through a single 42-inch influent pipe.

Inside the headworks building, wastewater can pass through the manual bar rack or the automatic screen. The automatic screen removes solids greater than ¼ inch from the liquid stream, which are then washed and dewatered before being hauled to the landfill for disposal. The manual bar rack has 1-inch openings and is used when the automatic screen is out of service.

Screened wastewater flows through a Parshall flume for flow measurement. The Parshall flume is currently operational, but in the past when it was out of service, GWA has used an area velocity meter inserted into the 42-inch influent pipe.

The current process flow diagram for the Northern District WWTP is shown in Figure 7-28, and the site plan is shown in Figure 7-29.

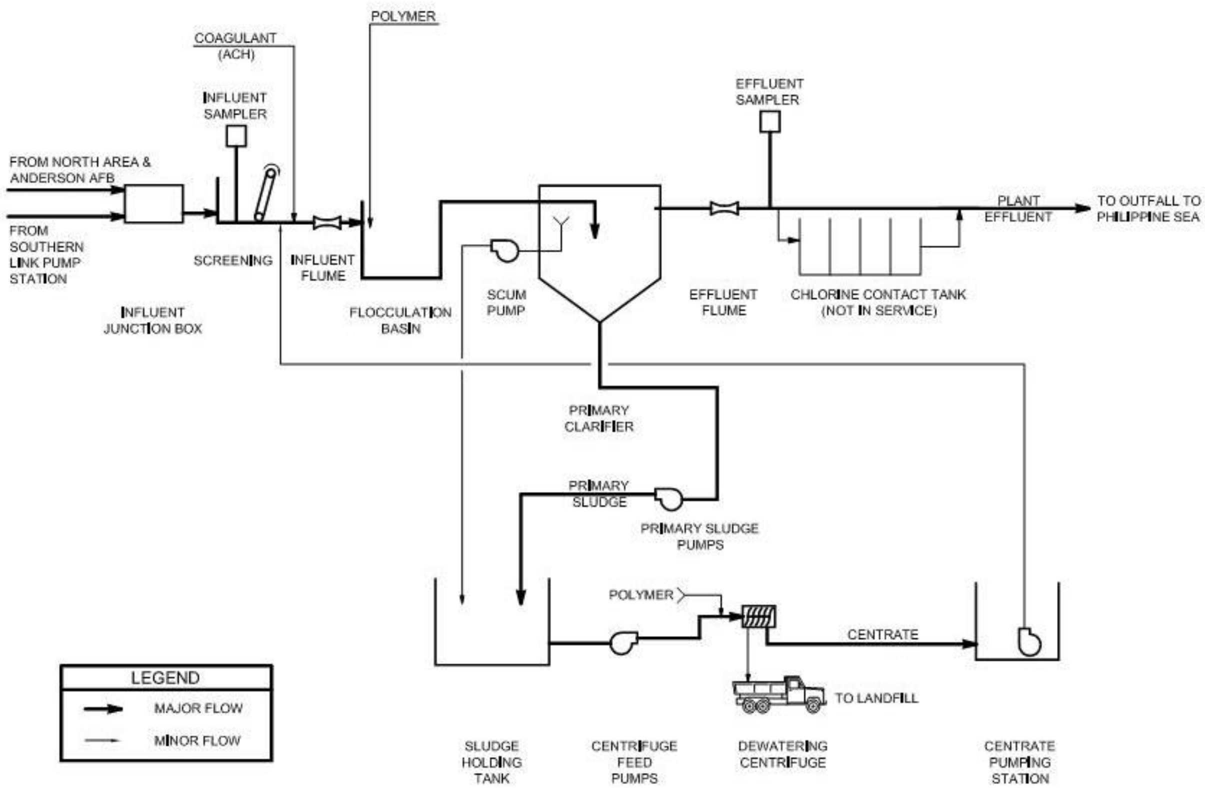

Figure 7-28. Northern District WWTP Existing Treatment Process Flow Diagram

KEY NOTES

1. HEADWORKS BUILDING (BAR SCREENS)
2. FLOCCULATION BASIN
3. PRIMARY CLARIFIER
4. PRIMARY SLUDGE & SCUM PUMP STATION
5. CHLORINE DISINFECTION TANKS (NOT IN SERVICE)
6. SLUDGE DRYING BEDS
7. SEPTAGE RECEIVING STATION
8. PLANT WATER SYSTEM
9. SLUDGE HOLDING TANK
10. SLUDGE DEWATERING BUILDING
11. DIGESTERS (NOT IN SERVICE)
12. ADMINISTRATION BUILDING & LABORATORY
13. GPA GENERATOR
14. CHEMICAL STORAGE / FEED BUILDING

Figure 7-29. Northern District WWTP Existing Site Plan

Aluminum chlorohydrate (ACH) or poly-aluminum chloride (PACL) coagulant is added at the Parshall flume to begin the CEPT process. The chemical addition location allows the flume’s hydraulic jump to complete the initial chemical mixing. After ACH addition, flow passes into the flocculation basin, which is converted from one of the original aerated grit chambers. Polymer flocculant can be added to the flow upstream and downstream of the flocculation basin. The second of the original grit chamber basins functions as a backup to the flocculation basin. The back-up basin includes aeration to provide mixing and maintain solids in suspension, but it is not properly designed or equipped.

Flow from the flocculation basins enters a box that splits the flow to the two primary clarifiers. With relatively low average flow into the treatment plant, operators currently attempt to use only one clarifier at a time to keep the second as a standby. Effluent from the primary clarifiers flows through a 42-inch pipe to the effluent Parshall flume and chlorine contact chamber.

The WWTP was originally designed to use chlorine for effluent disinfection; however, the chlorine equipment has since been removed due to the lack of a dechlorination system. With the chlorine contact chamber offline, flow is bypassed by diverting flow from the chlorine contact tank influent channel directly to the outfall.

Treated wastewater flows through a 48-inch pipeline to the ocean outfall. The pipeline reduces to 30 inches as it descends the steep bluff to the shoreline.

Table 7-19 summarizes the condition of the liquid treatment elements.

Table 7-19. Northern District WWTP Liquid Treatment Elements Condition Assessment

Location	Equipment/Structure	Condition/Issues
Headworks	Building	• Overall marginal condition • Some corrosion of influent channels, open roof added for automatic screen installation • Does not comply with National Fire Protection Association 820 code requirements
	Screen inlet gates	• Overall poor condition • Gates are manual slide gates that are difficult to operate and dangerous for operators to access
	Automatic screen	• Overall poor condition • Screen has not been in operation for an extended period and there have been operational, maintenance, and spare parts issues with the screen since the original installation • No standby automatic screen is currently provided, only a manual raked bar rack • Access for maintenance is poor, as the current screen must be completely removed from the channel through the roof opening to determine the current reason for failure
	Influent flume	• Overall poor condition • Flow inlet conditions to the flume are poor, which affects flow measurement accuracy
	Preaeration tanks	• Overall acceptable condition • Concrete appears to be in fair condition and could be refurbished • Tanks currently function as polymer mixing zones for CEPT that may not be required in the upgrade
	Grit/flocculation basins	• Tank condition is acceptable but could use refurbishment • Tanks may not be suitable for a new grit system installation
	Grit/flocculation equipment	• Original grit handling equipment has been removed • Flocculation equipment is in acceptable condition, but is currently out of service while waiting for spare parts
Primary Clarifiers	Clarifier tanks	• Overall acceptable condition • Concrete tanks appear to be in good condition and could continue to be used • See Section 5 for discussion of design deficiencies
	Clarifier mechanism	• Overall good condition • Clarifier mechanisms were replaced in 2012 and are currently in good condition with no mechanical operational issues reported

Table 7-19. Northern District WWTP Liquid Treatment Elements Condition Assessment		
Location	Equipment/Structure	Condition/Issues
	Sludge and scum pumps	• Overall good condition • New pumps were installed in 2012 upgrade and are currently functional
Chlorine Disinfection	Contact tanks	• Overall good condition • Tanks can be refurbished or modified for use • Currently bypassed
	Chlorination equipment	• None present, all chlorination equipment has been removed

Septage Receiving Station

A septage receiving station was constructed on site in 2013 to accept delivery of septage waste. Prior to this installation, septage waste was dumped into an upstream manhole. The septage receiving station consists of an unloading area for the trucks, concrete storage tank, submersible propeller mixer, and septage transfer pumps. Waste is delivered from the truck through a manually cleaned bar rack and into an uncovered septage holding tank, allowing operators to see and control incoming septage. From the holding tank, septage is pumped using one of two progressing cavity pumps into the plant’s influent channel upstream of the screens. The pumps allow the septage to be transferred to the plant at a lower constant flowrate that is better for process control, as operators know the quantity and rate of septage being introduced into the flow stream and can adjust chemical feed rates accordingly. When the septage receiving station is out of service for any reason, septage is discharged into a manhole at the Southern Link Pump Station.

7.7.1.2 Solids Processing

Solids processing at the Northern District WWTP is described below.

Original Facility Solids Processing

Sludge generated at the WWTP was originally anaerobically digested and dewatered before hauling to the Ordot Dump for disposal.

Current Solids Processing

The original anaerobic digesters have been out of service for years and the sludge pumps, mixers, heaters, and other digester equipment have all been removed. Currently, sludge generated from the CEPT process is not stabilized, and is dewatered on site before being transported to the Layon Landfill for disposal. The only requirement for sludge disposal to the landfill is that it meets the paint filter test as defined in 40CFR258.28, which is readily achievable with centrifuge dewatering.

The current solids processing system design was shown schematically in Figure 7-26, and currently includes the following components:

- Primary sludge and scum transfer pumps
- Sludge holding tank
- Sludge dewatering feed pumps
- Sludge dewatering centrifuges

Sludge and scum are collected from the primary clarifiers and pumped to the sludge holding tank where it is stored temporarily prior to dewatering. Sludge and scum are each pumped by dedicated progressing cavity pumps with macerators installed upstream of the pumps. The pumps were installed in the original primary sludge pump station building in 2013 as part of the conversion to the CEPT process.

Primary sludge and scum are stored in an above-ground sludge storage tank for ultimate feed to the dewatering centrifuges. The sludge holding tank is a bolted steel tank that is mixed by a single submersible propeller-type mixer. GWA operators report that the mixer does not provide adequate mixing energy to keep the solids in suspension. Sludge is then pumped from the holding tank to the centrifuges as required to accommodate the sludge volume generated and the operating time for the centrifuges.

Two centrifuges provide sludge dewatering. The centrifuges are installed in the original dewatering building, which was upgraded as part of the 2013 project. The feed sludge is conditioned with polymer prior to dewatering. The polymer system is installed on the upper level of the original anaerobic digester pump room and is in good condition. The centrifuge dewatering is very effective, with dry solids content exceeding 30 percent. The plant is currently hauling three roll-off containers of sludge on Mondays and two containers on Tuesday through Saturday.

The existing anaerobic digester tanks are used for emergency sludge storage. The facility also has sludge drying beds that are used for dewatering Vactor truck pumpings and for emergency sludge dewatering and storage.

Solids Production

Figure 7-30 shows Northern District WWTP solids production from January 2013 through December 2014.

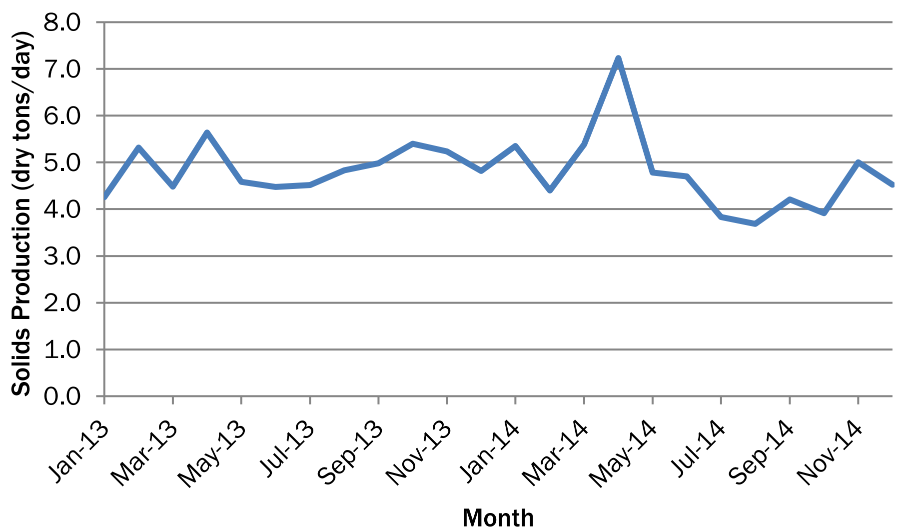

Figure 7-30. Northern District WWTP Solids Production from January 2013 through December 2014

Table 7-20 summarizes solids production and disposal at the WWTP.

Table 7-20. Northern District WWTP Solids Production from Jan 2013 through Dec 2014

Description	Value
Typical dewatered cake solids content	32%
Average disposal	458 wet tons per month 146 dry tons per month
Peak month disposal (April 2014)	700 wet tons 217 dry tons
Average solids production rate	4.8 dry tons per day

Table 7-21 summarizes the condition of the solids treatment elements.

Table 7-21. Northern District WWTP Solids Treatment Elements Condition Assessment

Location	Equipment/Structure	Condition/Issues
Sludge Holding Tank	Tank	• Overall good condition • Tank is bolted steel, installed as part of the 2012 upgrade • Tank volume may be too small for future sludge requirements • No redundancy provided
	Submersible sludge mixer	• Overall condition is unknown – mixer is submerged and was not removed from service for inspection but is operational • Condition assumed to be good, since the mixer was installed as part of the 2012 upgrade • No standby unit in place, but being submersible equipment, a shelf spare could be purchased for redundancy • Mixer subjected to high ragging wrapped around the impeller blades • GWA reports mixer is too small to provide adequate mixing performance
Sludge Dewatering	Building	• Overall condition is good • Building was refurbished and the superstructure added as part of the 2012 upgrade • Size of the building is likely to be not be adequate if larger centrifuges are required • Space for a third centrifuge is available
	Centrifuge feed pumps	• Overall condition is good • Pumps are operational and show no major signs of deterioration
	Dewatering centrifuges	• Overall condition is good • Polymer system condition is good • One unit was out of service at the time of the visit for a gearbox repair • Capacity and performance for upgraded sludge treatment system will need to be evaluated for continued use in the planned upgrade
Anaerobic Digesters	Concrete tanks	• Exterior condition is good • Interior condition could not be inspected, and could require rehabilitation • There are likely significant volumes of grit in the tanks that will need to be cleaned out
	Floating covers	• Overall condition is poor • Covers are tilted in the tanks and seals are no longer functional • Covers have reached the end of their useful service lives and will require replacement if anaerobic digestion is part of the upgrade

Table 7-21. Northern District WWTP Solids Treatment Elements Condition Assessment		
Location	Equipment/Structure	Condition/Issues
	Equipment	• No equipment present – all original digester equipment (external to the tanks) has been removed • Any piping and equipment internal to the tanks will need to be removed if the tanks are used in the upgrade
Sludge Drying Beds	Sludge drying beds	• Overall condition is acceptable • Guam climate (high precipitation) limits usefulness of the system

7.7.2 Regulatory Requirements

Regulatory requirements that apply to the Northern District WWTP are described below.

7.7.2.1 NPDES Permit

The Northern District WWTP operates under NPDES permit No. GU0020141, and is classified as a Wastewater Treatment Class III facility. The plant was historically not able to meet primary treatment standards, and the 2011 Court Order required that GWA construct interim primary treatment improvements and achieve consistent compliance with interim treatment limits of 85 mg/L BOD_5 and 50 mg/L TSS. Interim improvements implemented upgraded the plant to the CEPT process.

The CWA requires that publicly owned treatment facilities provide secondary treatment to wastewater that is discharged to the United States. Section 301(h) of the CWA allows an exception to this general requirement if the discharger demonstrates to the satisfaction of the EPA and with the concurrence of the state, that certain requirements are met. For years, the Northern District WWTP operated under a variance that allowed discharge of primary treated wastewater to the Philippine Sea. GWA requested a continuation of the variance and negotiated with USEPA over several years until it was ultimately denied in September 2009. The current NPDES permit issued in April 2013 requires secondary treatment. Therefore, the WWTP is out of compliance with the current NPDES permit requirements, which are summarized in Table 7-22.

Secondary treatment is required to meet the BOD_5 and TSS limits, and a disinfection process is required to meet the enterococcus limit. A dechlorination process will be required to comply with the total chlorine residual requirement if chlorine is used for disinfection.

Table 7-22. Northern District WWTP Effluent Limits				
Parameter	Maximum Allowable Discharge Limits			
	Concentration and Loading			
	Average Monthly	Average Weekly	Maximum Daily	Units
Flow rate	12	[a]	[a]	mgd
BOD_5	30	45	—	mg/L
	3002	6760	—	lbs/day
	The average monthly percent removal shall not be less than 85 percent. [b]			%
TSS	30	45	—	mg/L
	3002	6760	—	lbs/day
	The average monthly percent removal shall not be less than 85 percent. [b]			%
pH (hydrogen ion)	Within 6.5 and 8.5 at all times			pH units
Settleable solids	1	—	2	mL/L
Oil and grease, total recoverable	10	—	15	mg/L
Enterococcus[c]	35 [c]	—	104	CFU/100mL
Chlorine, total residual (TRC)	1.5	—	2.46	mg/L
Temperature	[a]	—	[a]	°C
Ammonia	[a]	—	[a]	mg/L
Chronic toxicity	[a]	—	[a]	Pass/Fail
Priority pollutant scan	[a]	—	[a]	—

a. No effluent limits are set at this time, but monitoring and reporting is required.

b. Both the influent and the effluent shall be monitored for BOD_5 and TSS. The arithmetic mean of the concentrations of effluent samples collected in a calendar month shall not exceed 15 percent of the arithmetic mean of the influent samples collected in the same calendar month (e.g., must achieve 85 percent removal rates).

c. Average monthly Enterococcus effluent monitoring shall be reported as a 30-day geometric mean. Maximum daily Enterococcus effluent monitoring shall be reported as the highest instantaneous maximum (the maximum of any single sample shall not exceed 104 CFU/100mL).

The existing Northern District WWTP effluent monitoring schedule is presented in Table 7-23. An effluent flow meter is required for continuous flow rate monitoring. A refrigerated automatic composite sampler is required for the weekly and annual 24-hour composite sampling requirements. All other sampling requirements are discrete (grab) samples.

Table 7-23. Northern District WWTP Effluent Monitoring Requirements		
Parameter	Monitoring Requirements	
	Frequency	Sample Type
Flow rate	Continuous	Metered
BOD_5	Weekly	24-hour composite
TSS	Weekly	24-hour composite
pH (hydrogen ion)	Weekly	Discrete
Settleable solids	Weekly	Discrete
Oil and grease, total recoverable	Weekly	Discrete
Enterococcus	Weekly	Discrete
Chlorine, total residual (TRC)	Weekly	Discrete
Temperature	Weekly	Discrete
Ammonia	Yearly [b]	24-hour composite
Chronic toxicity [a]	Yearly [b]	24-hour composite
Priority pollutant scan	Yearly [b]	24-hour composite
Ambient monitoring	Quarterly	Discrete

a. The permittee shall attempt to ensure a total holding time from collection of the last portion of the composite sample until arrival at the laboratory of not more than 36 hours. EPA has granted an extension to the Permittee for the holding time due to logistical issues. The extended holding time shall not exceed 72 hours.

b. Yearly monitoring shall be completed by January 31 each year.

7.7.2.2 Outfall

The WWTP outfall discharges approximately 1,900 feet from shoreline into the Philippine Sea at a nominal depth of 140 feet below mean sea level. The new submarine outfall is an extension of an existing outfall and was completed and placed in operation in January 2009. A 400-foot-long multiport diffuser with 40 ports was originally planned to be included at the end of the outfall, but due to design and construction issues was not installed. The USEPA's acceptance of the outfall and its relative dilution capability were based on the dilution that would be achieved from the mixing zone created by the diffuser. Therefore, a diffuser is required to be installed on the outfall to comply with USEPA requirements. The existing pipeline from the WWTP to the submarine outfall has an estimated hydraulic capacity of 40 mgd.

7.7.3 Wastewater Characteristics

Characteristics of wastewater flow at the Northern District WWTP are described below.

7.7.3.1 Historical Flows and Loads

Northern District WWTP historical flows and loads are described below.

Average and Peak Day Flows

Figure 7-31 shows influent flow data from the facility for the 3-year period from October 2012 through September 2015. The data shows that average flows were relatively constant over the 3-year period. Peak flow events appear to be of short duration caused by storm events.

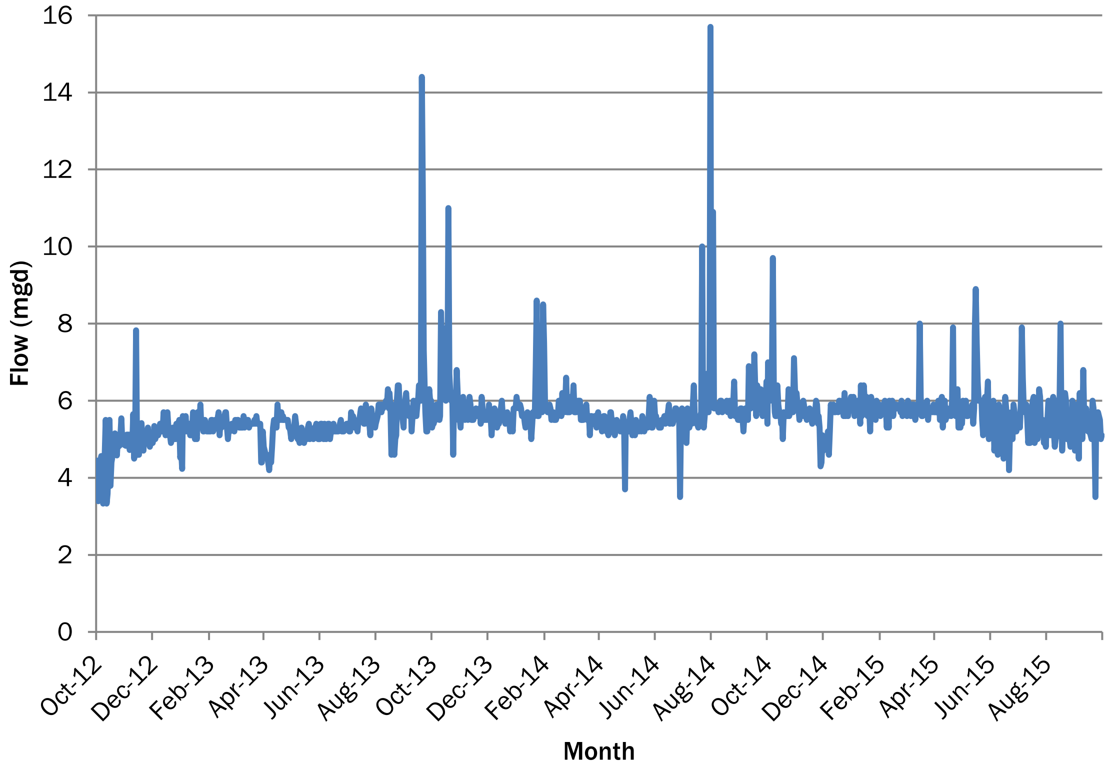

Figure 7-31. Northern District WWTP Influent Flow, October 2012 through September 2015

Figure 7-32 presents the same three years of data arranged by calendar month to provide a seasonal analysis. The figure shows that there is not a significant variation in average flows between the wet season (July through November) and the dry season, nor is there a significant increase in flow during peak tourist season (January through May).

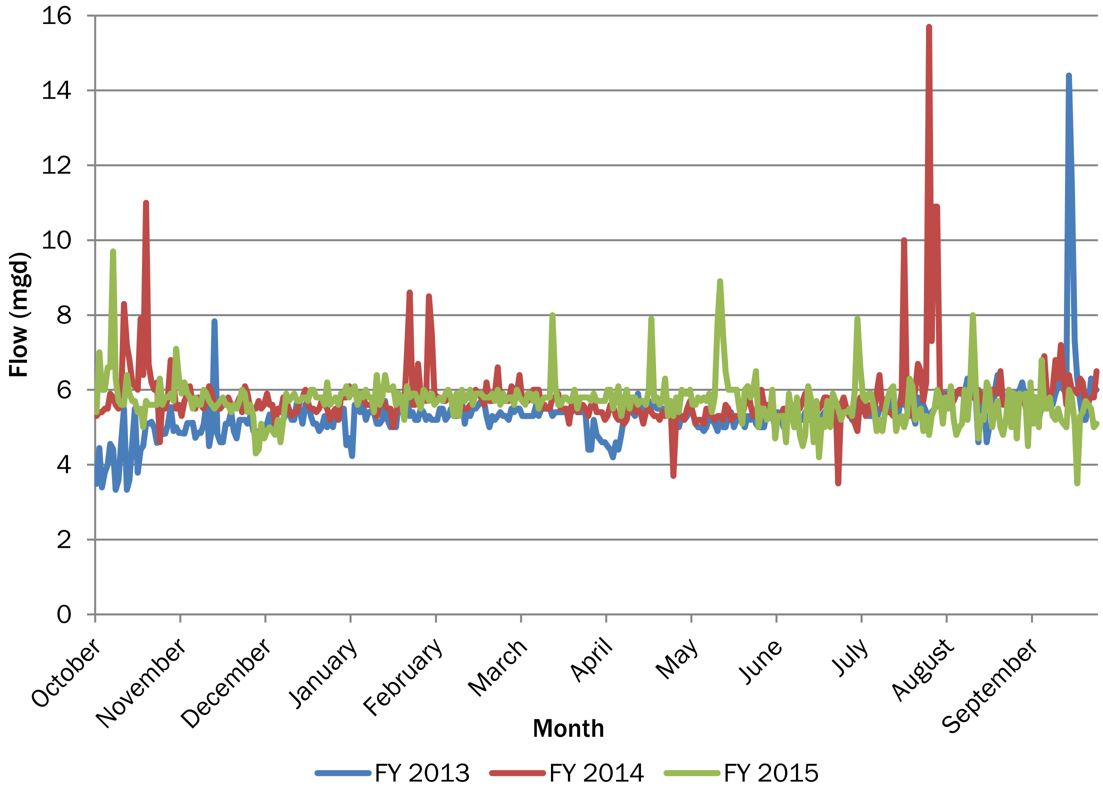

Figure 7-32. Northern District WWTP Influent Flow Seasonal Analysis

Table 7-24 lists existing flows based on the three years of data and corresponding calculated peaking factors.

Table 7-24. Northern District WWTP Existing Influent Flows		
Description	**Value**	**Peaking Factor**
Average flow	5.6 mgd	1.0
Peak day wet weather flow	15.7 mgd	2.8

7.7.3.1.1 Influent Characteristics

GWA collects influent composite samples weekly and tests them for BOD_5 and TSS. Figure 7-33 presents the influent BOD_5 and TSS data collected during the period from October 2012 through September 2015.

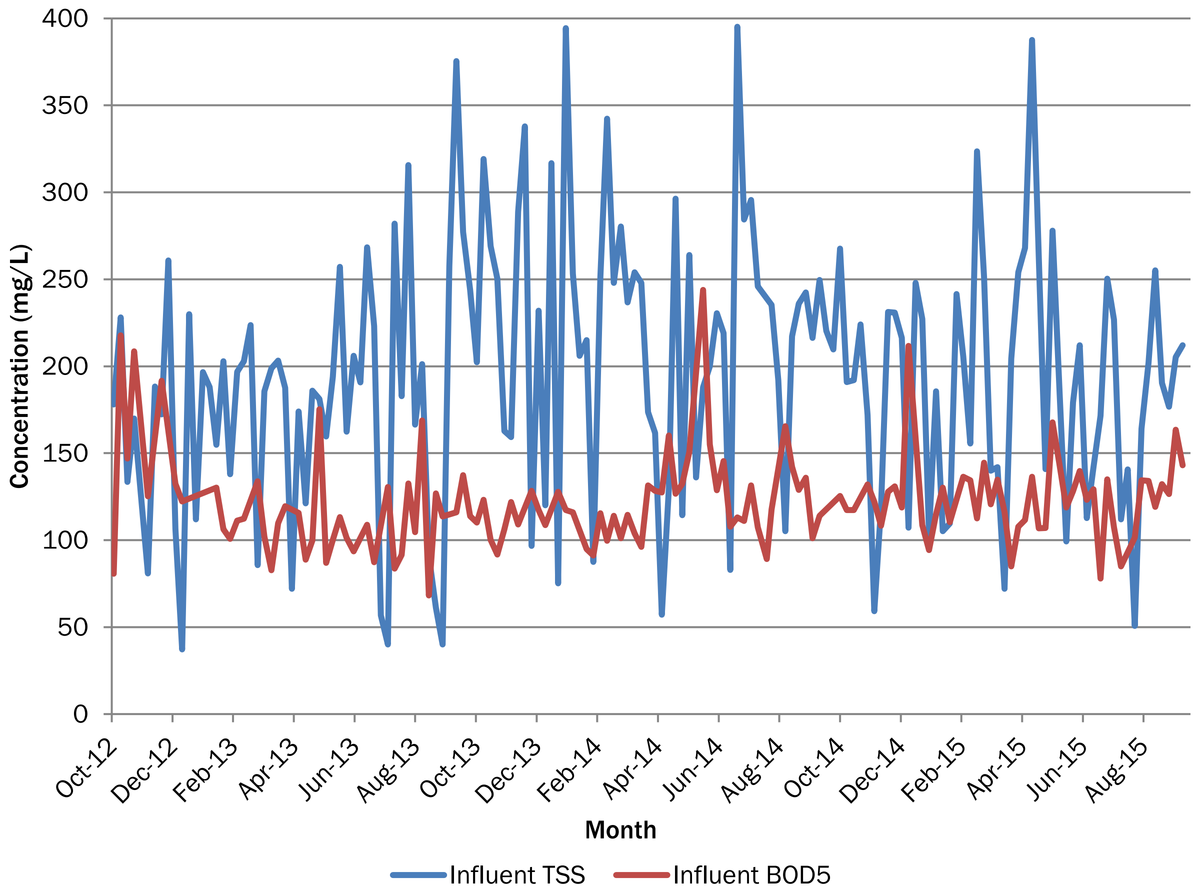

Figure 7-33. Northern District WWTP Influent Characteristics, October 2012 through September 2015

Table 7-25 summarizes influent characteristics over the 3-year period.

Table 7-25. Influent Characteristics, October 2012 through September 2015

Description	BOD_5	TSS
Average	122 mg/L	196 mg/L
Maximum	244 mg/L	395 mg/L
Minimum	68 mg/L	37 mg/L

7.7.3.1.2 Septage Quantities

The Northern District WWTP accepts septage and other liquid wastes from liquid waste haulers. The septage and other liquid waste enters the influent flow upstream of the influent sampler and influent flow meter, such that the flows and characteristics described above include the received hauled waste. Figure 7-34 shows the amount of hauled liquid waste received at the WWTP from October 2013 through September 2015.

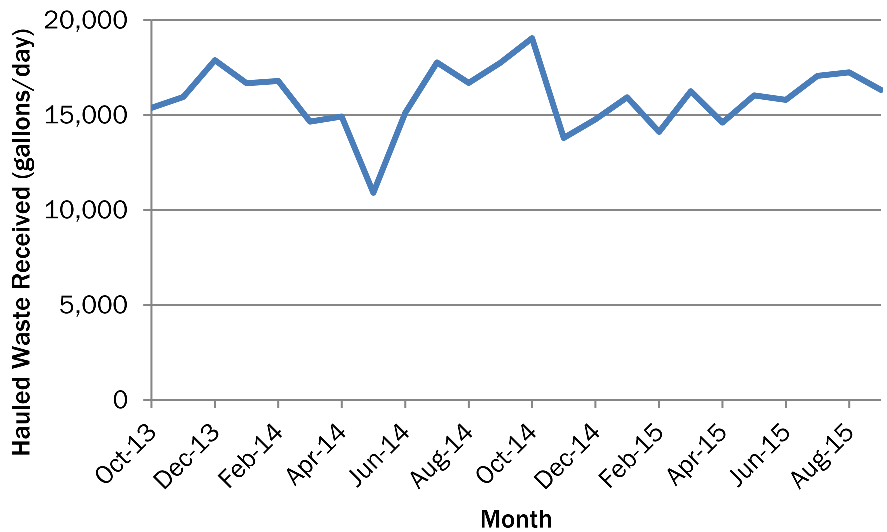

Figure 7-34. Northern District WWTP Hauled Liquid Waste Received from October 2013 through September 2015

Typical septage contains approximately 7,000 mg/L of BOD_5 and 15,000 mg/L of TSS. Table 7-26 summarizes septage flows and estimated loads received at the WWTP, based on typical septage characteristics.

Table 7-26. Northern District WWTP Septage Received, October 2013 through September 2015

Description	Value
Average number of truck loads received	7 per day
Average volume received	15,900 gpd
Average BOD_5 mass load (estimated)	900 lbs/day
Average TSS mass load (estimated)	2,000 lbs/day
Peak month volume	19,000 gpd

7.7.3.2 Future Flows and Loads

Northern District WWTP future flows and loads are described below.

7.7.3.2.1 Future Flows

An analysis of Guam population projections is included in Volume 1, Section 4. Figure 7-35 presents projected wastewater flows at the Northern District WWTP based on (Northern Basin municipalities) population growth including the expected increase in population, military buildup, Tourism 2020 plan, and potential additional sewer connections as defined in Volume 1, Section 4 and in the Northern District WWTP Facility Plan (BC, May 2017).

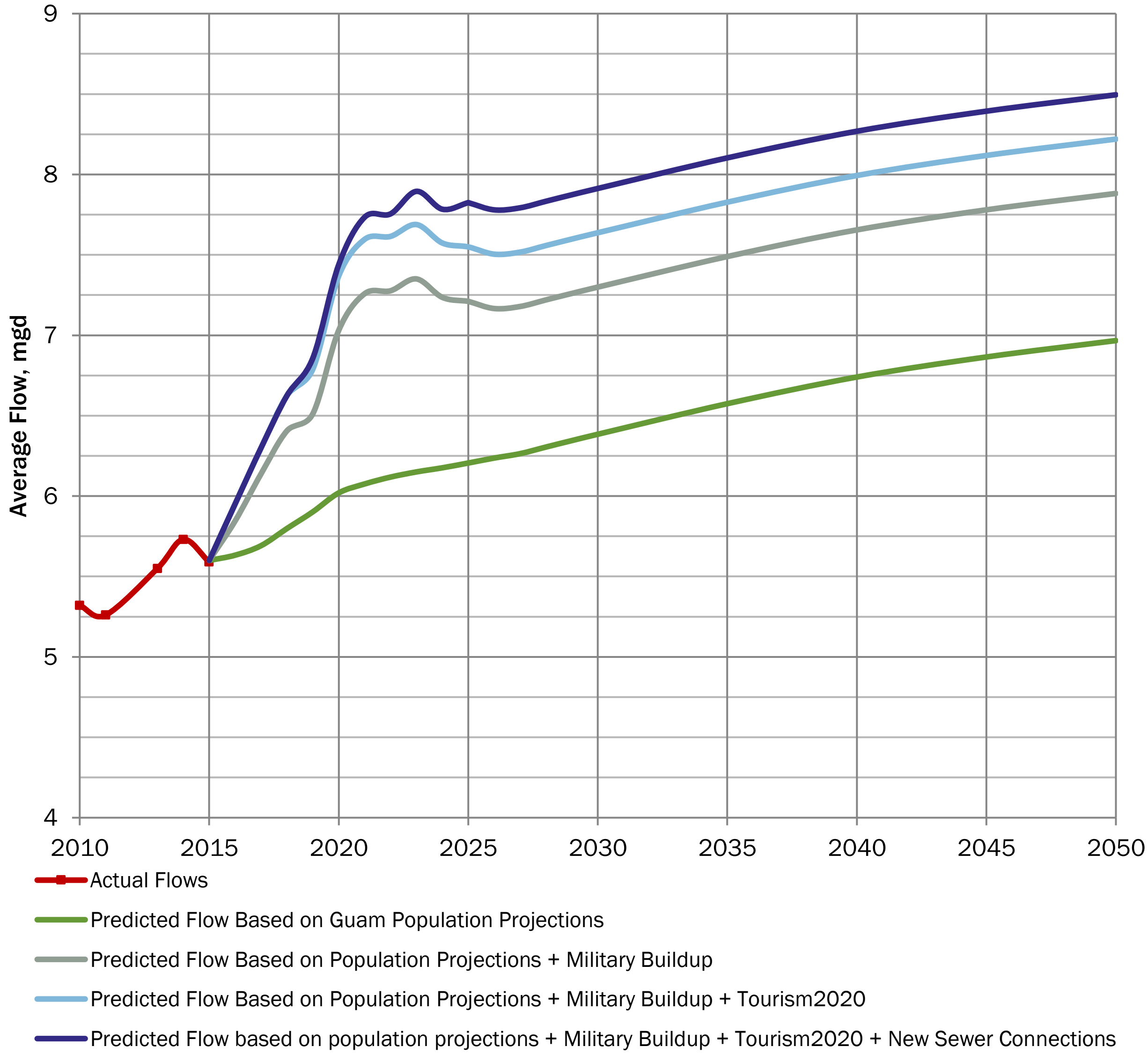

Figure 7-35. Northern District WWTP Projected Flows

Table 7-27 summarizes projected wastewater flows, which will be used as design criteria for the Northern District WWTP secondary treatment improvement project (Northern District WWTP Facility Plan, BC, May 2017).

Table 7-27. Northern District WWTP Flow Projection Summary

Year	Average Flow (mgd)	Peak Month Flow [a] (mgd)	Peak Day Flow [b] (mgd)	Peak Hour Flow [c] (mgd)
2016	6.0	6.5	16.6	20.2
2025	7.8	8.6	21.9	26.6
2035	8.1	8.9	22.7	27.5
2050	8.5	9.4	23.8	28.9
2065	9.0	9.8	25.1	30.4

a. Peak month peaking factor = 1.1

b. Peak day peaking factor = 2.8

c. Peak hour peaking factor = 3.4

7.7.3.2.2 Future Influent Characteristics and Loads

Northern District WWTP future influent characteristics and loads are described below.

Potential Impacts to Wastewater Characteristics

Future flows are expected to increase based on a variety of factors. However, the characteristics of future wastewater flows are expected to remain relatively similar to current conditions.

In the Northern District wastewater basin, residential and commercial users will increase as the overall population increases. The overall economic distribution is not expected to change significantly. While there may be additional commercial users specifically catering to the military buildup personnel and their families, there will also be an increase of residential wastewater connections. For planning purposes, it is reasonable to assume that wastewater characteristics will not change significantly.

One economic impact that may be noticed in the composition of the wastewater is the expected increase in tourism. Additional hotel rooms will incur an increase in laundry services, an industry that can significantly change the chemical configuration of wastewater. However, the total number of hotel rooms is expected to rise from 8,705 to 10,091 between 2015 and 2020, for a total of increase of 15.9 percent. This increase is comparable to the 14 percent population growth in the Northern District municipalities over the same 5-year period (see Table 3-1). Therefore, the projected tourism growth is not expected to significantly affect the wastewater characteristics of the Northern District WWTP influent flow.

Bringing online the majority of the existing septic tank users already adjacent to the wastewater collection will directly impact the amount of wastewater generated by certain neighborhoods and indirectly affect the composition of the flows at the plant. The plant currently accepts an average of 15,000 gpd of septage, which adds mass to the influent without significantly increasing flow. Connecting additional septic tank users to the wastewater collection system could lead to a small reduction in septage received at Northern District WWTP from liquid waste haulers while increasing flow. However, the net impact of connecting properties currently served by onsite wastewater systems is considered negligible for planning purposes.

Current Characteristics and Mass Loads

Table 7-28 summarizes current influent characteristics and mass loads.

Table 7-28. Northern District WWTP Existing Influent Mass Loadings		
Description	Average Concentration (mg/L)	Mass Loading [a] (lbs/day)
BOD_5	122 [b]	5,900
TSS	196 [b]	9,500
Total N	30 [c]	1500
Total P	6 [c]	300

a. Based on an average flow of 5.8 mgd.

b. Average of data collected from October 2012 through September 2015.

c. Estimate based on typical raw wastewater values.

Future Characteristics and Mass Loads

For planning purposes, future influent BOD_5 and TSS concentrations were estimated to be equal to the current average concentration plus one standard deviation. Table 7-29 summarizes the future wastewater characteristics that will be used for planning purposes.

Table 7-29 Northern District WWTP Wastewater Characteristics for Planning Purposes	
Parameter	Concentration
BOD_5	150 mg/L
TSS	275 mg/L
Total N[a]	30 mg/L
Total P[a]	6 mg/L

b. Influent data not available, typical values used.

Table 7-30 presents the future influent mass loadings that will be used for planning purposes.

Table 7-30. Northern District WWTP Mass Load Projections				
Parameter	Year			
	2025	2035	2050	2065
Average flow (mgd)	7.8	8.1	8.5	9.0
BOD_5 mass load (lbs/day)	9,800	10,100	10,600	11,300
TSS mass load (lbs/day)	17,900	18,600	19,500	20,600
Total N mass load (lbs/day)	2,000	2,000	2,100	2,300
Total phosphorus mass load (lbs/day)	500	600	600	600

7.7.3.3 Effluent Quality

Northern District WWTP effluent quality is described below.

7.7.3.3.1 Pre-CEPT Effluent Quality

Figure 7-36 shows a typical year of effluent quality data collected prior to implementation of the CEPT process.

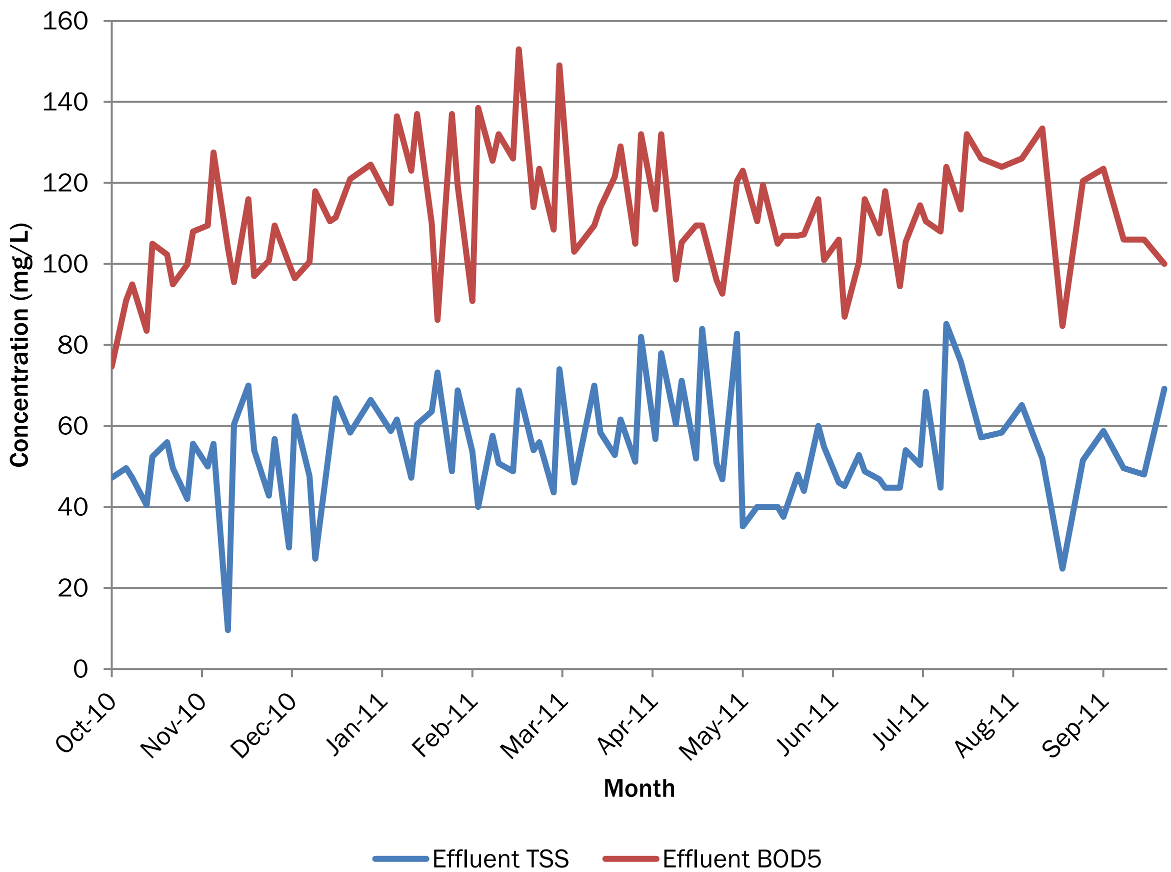

Figure 7-36. Typical Pre-CEPT Effluent Characteristics

Table 7-31 summarizes the data.

Table 7-31. Northern District WWTP Effluent Characteristics, October 2010 through September 2011

Description	BOD_5	TSS
Average (mg/L)	112	55
Maximum (mg/L)	153	85
Minimum (mg/L)	75	10
Standard deviation	15	13
Average removal rate	8%	72%

7.7.3.3.2 Post-CEPT Effluent Quality

GWA collects and tests daily effluent samples and tests for BOD_5 and TSS. Figure 7-37 presents the effluent quality data collected from January 2013 through September 2015. The CEPT process was in operation during this time.

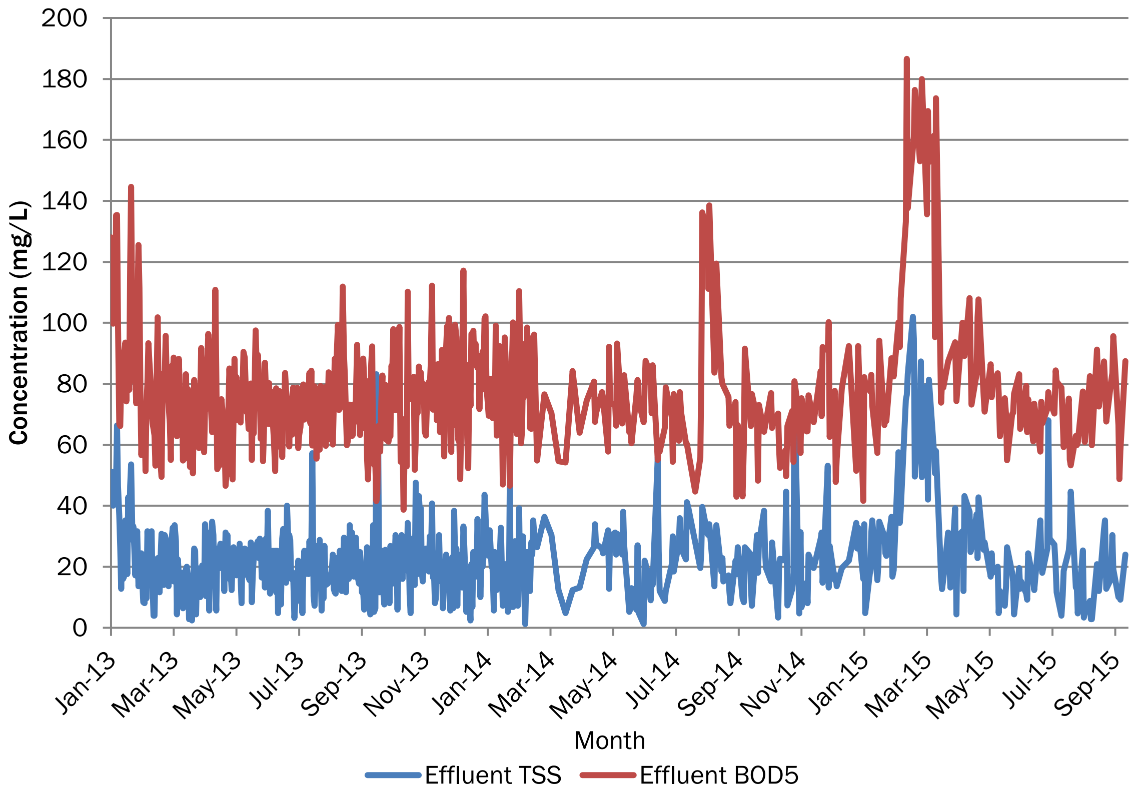

Figure 7-37. Northern District WWTP Effluent Characteristics, January 2013 through September 2015

Table 7-32 summarizes effluent characteristics.

Table 7-32. Northern District WWTP Effluent Characteristics, January 2013 through September 2015

Description	BOD_5	TSS
Average (mg/L)	77	22
Maximum (mg/L)	187	102
Minimum (mg/L)	39	1
Standard deviation	20	14
Average removal rate	37%	89%

The WWTP provides primary treatment and the effluent characteristics are a direct representation of the influent wastewater characteristics and the efficacy of the existing primary clarifiers. Table 7-33 compares BOD_5 and TSS concentrations and removal rates before and after implementation of the CEPT process, and typical removal rates for primary clarifiers. As shown in the table, primary clarifier performance prior to implementation of CEPT was poor for BOD_5 removal. The table shows that implementation of CEPT has increased BOD_5 removal to that found in typical primary clarifiers without CEPT, and TSS removal is significantly greater than typical values.

Table 7-33. Comparison of Northern District WWTP Primary Clarifier Performance

Description	Before CEPT		With CEPT	
	BOD_5	TSS	BOD_5	TSS
Average influent concentration [a]	122 mg/L	196 mg/L	122 mg/L	196 mg/L
Average effluent concentration	112 mg/L	55 mg/L	77 mg/L	22 mg/L
Average removal rate	8%	72%	37%	89%
Typical primary clarifiers [b]	25–40%	50–70%	--	--

a. Values listed in Table 7-22 were used for this evaluation.

b. Source: Metcalf & Eddy, Wastewater Engineering, Treatment and Reuse, 4th edition, 2003.

7.7.4 Recommended Improvements

Recommended improvement projects for Northern District WWTP are described below.

7.7.4.1 Secondary Treatment Project

A Facility Plan report was completed for the Northern District WWTP, in May 2017. Figure 7-38 provides a simplified schematic diagram of the Facility Plan's recommended alternative for secondary treatment upgrades at the Northern District WWTP. Figure 7-39 shows a preliminary site plan for the recommended project.

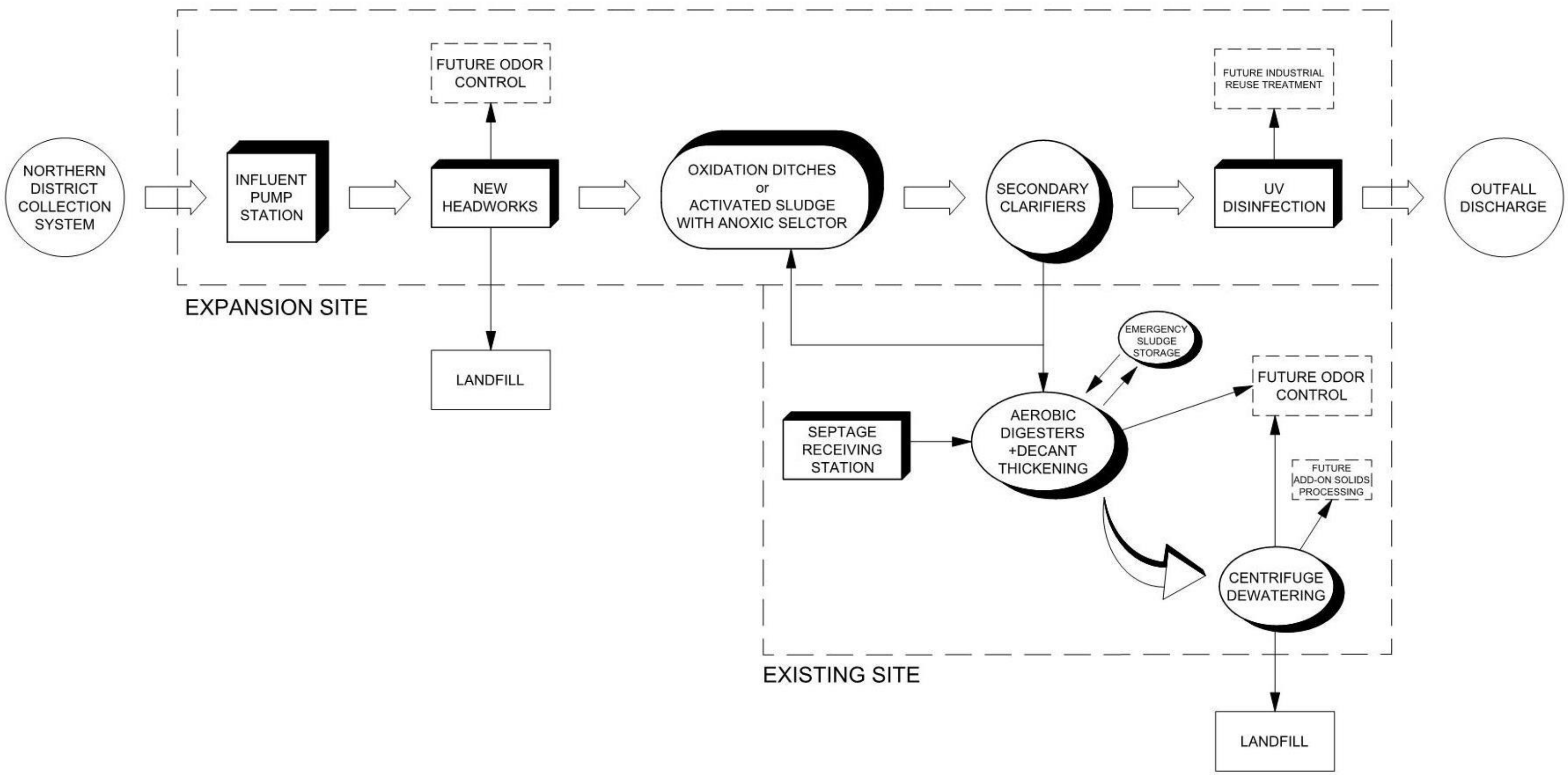

Figure 7-38. Northern District WWTP Facility Plan Recommended Improvements – Schematic Diagram

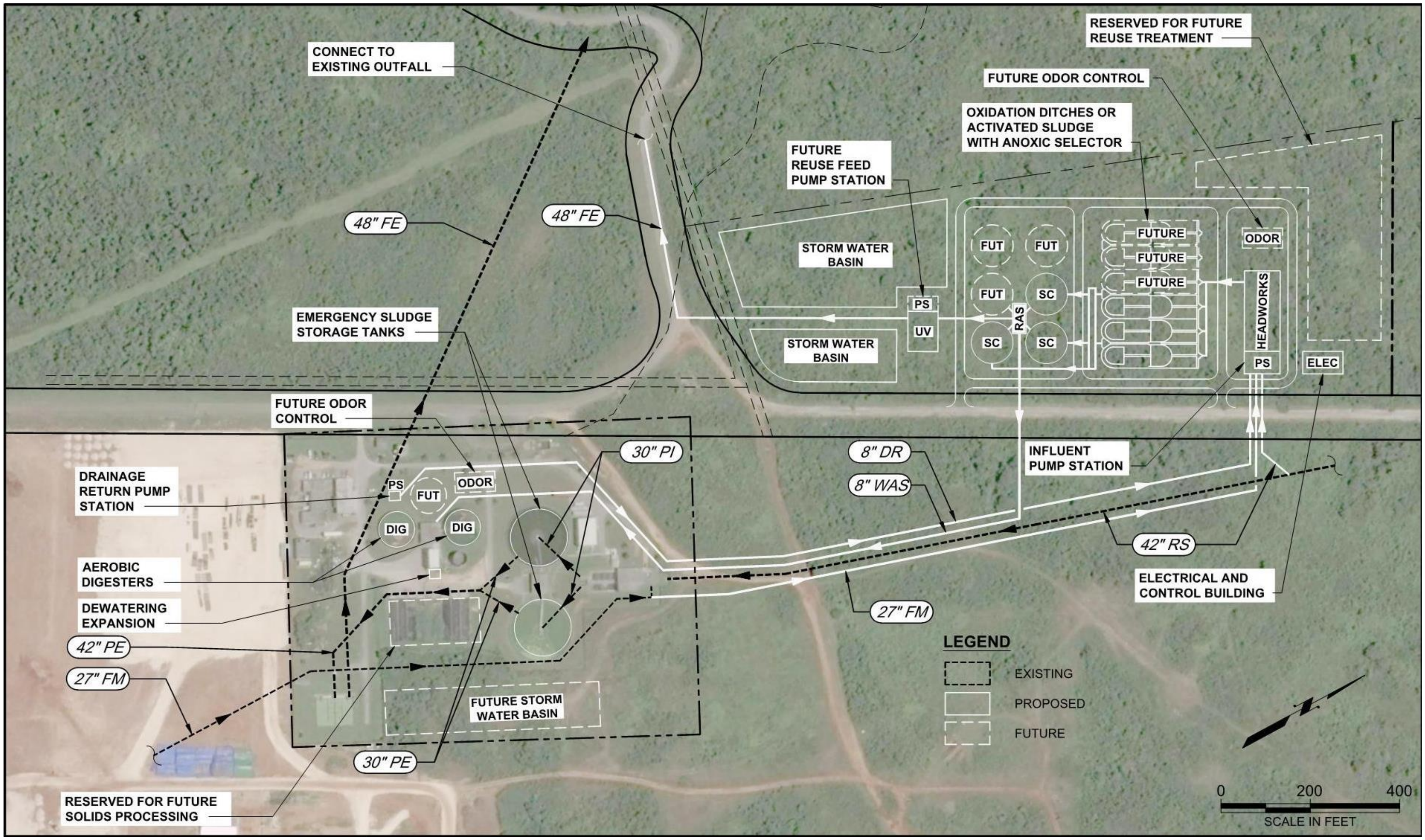

Figure 7-39. Northern District WWTP Facility Plan Recommended Improvements – Site Plan

The recommended alternative includes the following improvements at Northern District WWTP to meet the existing NPDES permit's secondary treatment requirements:

- Reroute the influent sewer to the expansion site.
- Extend the Southern Link force main to the expansion site.
- Install a new influent pump station with magnetic influent flow meter at the expansion site.
- Install a new headworks with composite sampler, step screens, manually-cleaned bar rack in a bypass channel, and HeadCell grit removal.
- Include provisions for future headworks odor control, if needed.
- Upgrade the electrical and control building.
- Install oxidation ditches or activated sludge with anoxic selector. The final choice will be made after an influent characterization study, process evaluation, and life-cycle cost estimates of the two options.
- Install circular secondary clarifiers with self-cleaning algae sweeps.
- Upgrade RAS/WAS pump station and piping.
- Implement UV disinfection.
- Upgrade the utility water pump station.
- Discharge to the existing outfall with diffuser installed.
- Upgrade aerobic digestion with decant thickening of primary sludge and waste-activated sludge.
- Route existing septage receiving station directly to aerobic digesters.
- Convert existing primary clarifiers into emergency sludge storage tanks.
- Expand centrifuge dewatering.
- Upgrade drainage return pump station and pipeline.
- Include provisions for future odor control at aerobic digesters and centrifuge dewatering, if needed.
- Retain existing drying beds for Vactor truck use.
- Include provisions for landfill disposal of grit, screenings, and dewatered sludge.
- Reserve space for future add-on solids processing and industrial reuse treatment.

7.7.4.2 Recommended Improvement Projects

The capital cost of the Northern District WWTP Secondary Treatment Upgrade project will be paid by the U.S. Department of Defense (DoD) as part of a larger effort to relocate U.S. Marines from Okinawa, Japan to Guam. The DoD budget is fixed, and GWA will strive to design a facility that can be constructed with the available budget. If the project cost ultimately exceeds the available DoD budget, GWA will need to provide supplemental funding to complete the project. The CIP includes a project with budget to address this contingency as well as potential costs associated with repairs and improvements to the interceptor pipeline. If GWA is successful in implementing the project within the available DoD funding, the contingency funds will be available for other needs.

Once the new WWTP is complete, it will require regular maintenance, but no major improvement projects are expected in the near future. A WWTP rehabilitation project is recommended after 15 years of operation to include:

- Replacement or refurbishment of mechanical equipment and controls
- Inspection and repair of structures
- Rehabilitation of electrical equipment and control systems

In addition, a visual inspection of the ocean outfall pipe and diffuser should be performed every 5 years. The water quality monitoring program only examines the discharge, without a visual examination of the complete length of the outfall pipe.

7.8 Hagåtña WWTP

The following section describes the existing conditions, regulatory requirements, wastewater characteristics, and recommended improvement projects at the Hagåtña WWTP.

7.8.1 Existing Conditions

The Hagåtña WWTP is located on a 500-by-700-foot man-made island in Agana Bay. The WWTP is owned and operated by GWA.

The treatment plant was originally commissioned in 1979 and designed as a primary treatment facility. The plant was upgraded to a CEPT facility in 2014. Treated wastewater is discharged to the Philippine Sea via an ocean outfall. The coastal waters off Agana Bay are considered "Category M-2 Good" marine waters in the GWQS. Figure 7-40 shows the Hagåtña WWTP in October 2016.

Figure 7-40. View at Hagåtña WWTP (October 2016)

The facility collects and treats wastewater from the central region of Guam, which includes the villages of Agana, Agana Heights, Asan, Piti, Tamuning, Mongmong-Toto, Sinajana, Chalan Pago-Ordot, Yona, Mangilao, a portion of Barrigada, and Tumon, as shown in Figure 2-1. The Hagåtña WWTP currently serves a population of approximately 82,600 people.

7.8.1.1 Liquid Treatment

Hagåtña WWTP liquid treatment is described below.

Original Facility

The original 1979 treatment plant consisted of three rectangular primary clarifiers used to remove suspended solids from the raw sewage and four aerobic digesters for stabilizing the solids removed by the primary clarifiers.

Table 7-34 summarizes the WWTP's original capacity. Comparison of the original flow capacity with the CEPT design shows that the WWTP currently operates at approximately 52 percent of its original design average daily flow capacity.

Table 7-34. Hagåtña WWTP Original Design Capacity

Description	Value
Average daily flow capacity	12.0 mgd
Peak hour flow capacity	21.0 mgd
Peak hour peaking factor	1.75

CEPT Modifications

A modification project to convert the facility to the CEPT process and improve primary treatment performance was completed in March 2014. The Process Design Report (Veolia, 2012) based the design criteria for the CEPT project on 2011 operation data. Table 7-35 summarizes the flow capacity and influent flow characteristics that were used to design the CEPT modifications.

Table 7-35. Hagåtña WWTP Flow Capacity and Influent Characteristics for CEPT Design [a]

Description	Value
Average flow capacity	7.0 mgd
Peak (hour) flow capacity	12.0 mgd
Peak hour peaking factor	1.71
Influent BOD_5 concentration	180 mg/L
Influent TSS concentration	202 mg/L
Temperature	68–86 °F
pH	6.0–9.0
Alkalinity	250 mg/L as $CaCO_3$
Soluble BOD_5: total BOD_5 ratio	≤ 40%
Settleable suspended solids: TSS ratio	≥ 65%
Oil and grease	≤ 25 mg/L

a. Agana Wastewater Treatment Plant Primary Treatment Upgrade, Process Design Report, Final, August 2012

The 2013–2014 CEPT upgrade included, but was not limited to, the following improvements:

- Headworks
  - New diversion box
  - New weir gate
  - New 24-inch magnetic flow meter
  - New automatic step screen
- New flocculation basin
  - Four-cell basin with variable speed mixers, which replaced one of the four original aerobic digesters
- Primary clarifiers
  - Replacement chain and flight in clarifier No. 2
  - Upgraded scum removal system with automatic actuators
  - Two replacement scum chopper pumps
- Effluent pump system
  - Existing effluent pump station repurposed into new chemical handling building
  - Effluent pump system relocated to effluent distribution box No. 2
  - New piping between effluent distribution boxes No. 1 and No. 2
  - Raised height for effluent distribution box No. 1
- New chemical handling building
  - Building repurposed from old effluent pump station
  - New coagulant mixing tanks
  - Two new coagulant day tanks
  - New coagulant metering pumps
  - Four new 40-foot containers for storage of coagulant and settling aid chemicals
- Centrifuges/sludge dewatering
  - Variable speed drives and magnetic flow meters added to existing centrifuge feed pumps
  - Replacement polymer activation/feed systems
  - Modified centrate piping to allow discharge to digesters, headworks, or flocculation basin
- Instrumentation
  - New controls for newly installed equipment
  - Integration into central programmable logic controller system

Existing Facility

The Hagåtña WWTP has operated as an enhanced (via CEPT) primary treatment facility for three years. The current process flow diagram for the plant is shown in Figure 7-41, and the site plan is shown in Figure 7-42. Wastewater flows into the headworks through a 36-inch ductile iron pipe force main from the Hagåtña main wastewater pump station. A diversion box allows influent to flow through to the automatic fine screen or go into the bypass channel to the manual bar screen.

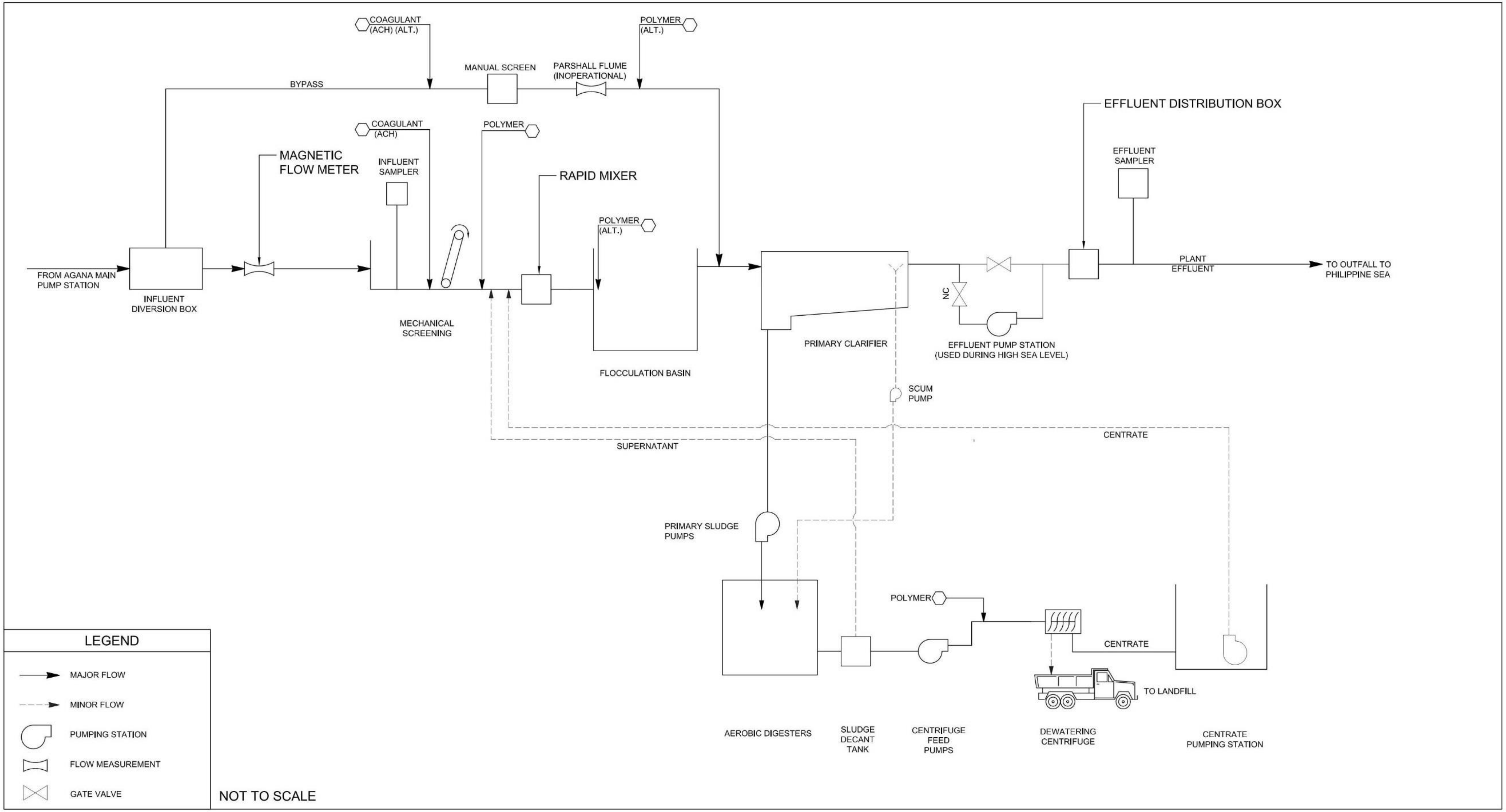

Figure 7-41. Hagåtña WWTP Existing Treatment Process Flow Diagram

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KEY NOTES

1. OPERATIONS BUILDING
2. GENERATOR BUILDING
3. MAINTENANCE BUILDING
4. INFLUENT CHANNEL
5. FLOCCULATION BASIN
6. EFFLUENT PUMPS
7. AEROBIC DIGESTERS
8. PUMP GALLERY
9. CHEMICAL STORAGE BUILDING
10. PRIMARY CLARIFIERS
11. DEWATERING BUILDING
12. EFFLUENT CHANNEL

Figure 7-42. Hagåtña WWTP Existing Site Plan

During typical operations, wastewater passes through a 24-inch magnetic flow meter prior to flowing through the automatic screen. The automatic screen removes solids greater than ¼ inch from the liquid stream, which are then washed and dewatered before being hauled to the landfill for disposal. A refrigerated automatic composite sampler is provided for collection of influent samples. The influent flow meter signals the automatic sampler to allow flow-paced samples, and drives flow-paced dosing of CEPT chemicals.

During bypass, flow goes to the manual bar rack and bypasses the influent sampler, influent magnetic flow meter, rapid mix, and flocculation basin. The manual bar rack has 1-inch openings and is used when the automatic screen is out of service.

ACH or PACL coagulant is added after screening for the CEPT process. The chemical addition point is located upstream of a rapid mixer. Polymer is added immediately downstream of the rapid mixer. After chemical addition, flow passes into the flocculation basin. The four-cell flocculation basin was converted from an aerobic digester, with slow mixers for floc agglomeration. Flow from the flocculation basins enters the primary clarifier distribution channel. Flocculated solids settle in the rectangular primary clarifiers and are removed with chain and flight collectors. Primary clarifier effluent flows to the 42-inch outfall for disposal. The effluent pump station is used to convey treated effluent during high flow and/or high tide events.

The existing liquid treatment facilities and pertinent design criteria are listed in Table 7-36, and the liquid treatment condition assessment is summarized in Table 7-37.

Table 7-36. Hagåtña WWTP Existing Liquid Treatment Processes Design Criteria

Description	Value
Headworks	
Automatic screen	
Number of units	1
Screen opening size	¼ inch
Manual bar rack	
Number of units	1
Screen opening size	1 inch
Influent flow meter	
Type	Magnetic
Primary clarifiers	
Number of units	3
Width, each	34 feet
Length, each	120 feet
Surface area, each	4,080 feet^2
Total surface area	12,240 feet^2
Side water depth	10 feet
Liquid volume, each	40,800 feet^3
Total liquid volume	122,400 feet^3

Table 7-37. Hagåtña WWTP Liquid Treatment Elements Condition Assessment		
Location	**Equipment/Structure**	**Condition/Issues**
General	Grit removal	• There is no grit removal system at the WWTP. • An interim grit removal system was installed at the Hagåtña wastewater pump station. Plant operators reported that grit removal was working normally.
	Disinfection	• Chlorine disinfection is currently not included in the treatment process due to safety concerns regarding the storage of chlorine gas and environmental concerns of chlorine exposure to ocean wildlife.
Headworks	Screen inlet gates	• Inlet gates are currently in acceptable condition.
	Automatic screen	• Automatic screen is often out of operation for extended periods of time due to mechanical issues and difficulty with procuring replacement parts.
	Manual screen	• Use of the manual screen bypasses influent sampler, flocculation chamber, flow meter, and mechanical screen. • Manual screen has seen extended use due to failure of the automatic screen. • No overflow to bypass channel occurs.
	Influent sampler	• The influent sampler is currently in acceptable condition.
	Flow meter	• The influent magnetic flow meter is currently in acceptable condition.
	Rapid mix	• ACH dosage computer was not working properly. ACH dosage was being estimated by weight.
	Flocculation basin	• The flocculation basin is currently in acceptable condition.
	Flocculation equipment	• Flocculation equipment is currently in acceptable condition.
Primary Clarifiers (chemically enhanced)	Clarifier tanks	• Covered concrete tanks appear to be in good condition. • ACH is currently being used in large quantities to cause bulk settling of particles when the rapid mix and flocculation basin are offline. • Polymer addition is causing sludge blanket to rise.
	Clarifier mechanisms	• Center rake was replaced in 2015, and the other rakes are approximately 8 years old. • Raking of scum required about twice per day.
	Sludge and scum pumps	• Pumps are in overall good condition. • Two pumps are currently offline. • New pumps were installed as part of the 2012 upgrade and are currently functional.
Effluent Pump Station	Pump station	• Effluent pump station is only used during periods of high flows and/or high tides. • Pump station is currently in acceptable condition.
Outfall	Effluent sampler	• Effluent sampler is currently in acceptable condition.
	Ocean outfall	• The ocean outfall is believed to be in acceptable condition.

7.8.1.2 Solids Processing

Sludge generated at the WWTP is aerobically digested and dewatered before hauling to Layon Landfill for disposal. The only requirement for sludge disposal to the landfill is that it meets the paint filter test as defined in 40 CFR 258.28, which is readily achievable with centrifuge dewatering.

The current solids processing system design shown schematically in Figure 7-41 includes the following components:

- Aerobic digesters
- Digested sludge thickener (decant tank)
- Thickened sludge/centrifuge feed pumps
- Dewatering centrifuges

Three covered aerobic digesters, each with a single mechanical surface aerator, digest sludge. A sludge thickening tank provides sludge thickening prior to pumping to the dewatering process. The digesters and thickener were constructed in an in-line configuration with reinforced concrete, share common walls, and are covered with a reinforced concrete deck. The digested sludge thickener is located between digesters 2 and 3 (AECOM, 2012).

Two centrifuge units (one duty, one standby) located on a mezzanine in the dewatering building dewater digested solids. Sludge is conveyed from the thickening tank to the centrifuges through two progressive-cavity pumps (one duty, one standby) located in the pump gallery. Magnetic flow meters and variable frequency drives have been retrofitted onto the centrifuge feed pumps to facilitate adjustment of the sludge feed rate (AECOM, 2012).

Polymer is injected upstream of the centrifuges to enhance sludge dewatering. Two polymer feed units (one duty, one standby) are in the dewatering building. Each polymer feed system consists of a control panel, polymer metering pump, and mixing unit. Feed water for the polymer unit is supplied by booster pumps located in the pump gallery (AECOM, 2012).

Dewatered solids are deposited in a dumpster bin located below each centrifuge. Centrate, diverted sludge at centrifuge start-up and shut-down, and cooling water are returned to the influent channel. Centrifuge dewatering is effective, with dry solids content at approximately 28 percent. The plant is currently hauling three roll-off containers of sludge on Mondays and two containers on Tuesday through Saturday.

The existing solids processing facilities and pertinent design criteria are listed in Table 7-38, and a condition assessment of the solids treatment is summarized in Table 7-39.

Table 7-38. Hagåtña WWTP Existing Solids Processing Facilities Design Criteria

Description	Value
Aerobic Digesters	
Number of units	3
Dimensions, each	36 feet x 36 feet
Side water depth	14 feet
Volume, each	18,144 $feet^3$
Total volume	54,432 $feet^3$ 407,200 gallons
Aeration	mechanical surface aerators
Sludge Thickening Tank	
Number of units	1
Dimensions	36 feet x 10 feet
Side water depth	11.5 feet
Volume	4,140 $feet^3$ 31,000 gallons
Centrifuge Dewatering	
Number of units	2 (one duty, one standby)
Feed rate, each	70 gpm

gpm: gallons per minutes

Table 7-39. Hagåtña WWTP Solids Treatment Elements Condition Assessment

Location	Equipment/Structure	Condition/Issues
Aerobic Digesters	Concrete tanks	• Exterior condition is good. • Operating temperature is approximately 135 °F. • Dissolved oxygen probe cannot be used at current operating temperature.
	Equipment	• Mixers were installed in 2007. • Intermittent mixer motor problems occur.
Sludge Dewatering Station	Building	• Overall condition is good.
	Centrifuge feed pumps	• Overall condition is good. • Pumping of solids from the clarifiers to the digesters occurs concurrently with pumping of sludge from the digesters to the centrifuges. • Pumps are operational and show no major signs of deterioration.
	Dewatering centrifuges	• Overall condition is acceptable. • Dewatered sludge contains approximately 28 percent solids. • One unit is often out of service due to power surge and has been intermittently offline. • Centrifuge units installed in 2007.

Table 7-39. Hagåtña WWTP Solids Treatment Elements Condition Assessment		
Location	Equipment/Structure	Condition/Issues
Centrate Pumping Station	Pumping station	• Overall condition is acceptable.

7.8.2 Regulatory Requirements

Regulatory requirements that apply to the Hagåtña WWTP are described below.

7.8.2.1 NPDES Permit

The Hagåtña WWTP operates under NPDES permit No. GU0020087 and is classified as a Wastewater Treatment Class III facility.

The plant was historically not able to meet primary treatment standards, and the 2011 Court Order required that GWA upgrade the primary treatment to achieve consistent compliance with the WWTP's NPDES permit. At the time, the NPDES limits for BOD_5 and TSS were 80 mg/L and 60 mg/L respectively. Interim improvements implemented upgraded the plant to the CEPT process in 2012.

The CWA requires that publicly owned treatment facilities provide secondary treatment to wastewater that is discharged to the United States. Section 301(h) of the CWA allows an exception to this general requirement if the discharger demonstrates to the satisfaction of the EPA and with the concurrence of the state, that certain requirements are met. For years, the Hagåtña WWTP operated under a 301(h) variance that allowed discharge of primary treated wastewater to Agana Bay in the Philippine Sea. GWA requested a continuation of the variance and negotiated with USEPA over several years until it was ultimately denied in September 2009. The current NPDES permit issued in April 2013 requires secondary treatment, but does not contain a compliance schedule. Table 7-40 summarizes the latest issued permit.

Table 7-40. Hagåtña WWTP Effluent Limits				
Parameter	Maximum Allowable Discharge Limits			
	Concentration and Loading			
	Average Monthly	Average Weekly	Maximum Daily	Units
Flow rate	12	[a]	[a]	mgd
BOD_5	30	45	—	mg/L
	3002	4506	—	lbs/day
	The average monthly percent removal shall not be less than 85 percent. [b]			%
TSS	30	45	—	mg/L
	3002	4506	—	lbs/day
	The average monthly percent removal shall not be less than 85 percent. [b]			%
pH (hydrogen ion)	Within 6.5 and 8.5 at all times			pH units
Settleable solids	1	—	2	mL/L
Oil and grease, total recoverable	10	—	15	mg/L
Enterococcus [c]	35 [c]	—	104 [c]	CFU/100mL
Chlorine, total residual (TRC)	0.75	—	1.23	mg/L
Temperature	[a]	—	[a]	°C
Ammonia	[a]	—	[a]	mg/L
Chronic toxicity	[a]	—	[a]	Pass/Fail
Priority pollutant scan	[a]	—	(1)	—

a. No effluent limits are set at this time, but monitoring and reporting is required.

b. Both the influent and the effluent shall be monitored for BOD5 and TSS. The arithmetic mean of the concentrations of effluent samples collected in a calendar month shall not exceed 15 percent of the arithmetic mean of the influent samples collected in the same calendar month (e.g., must achieve 85 percent removal rates).

c. Average monthly Enterococcus effluent monitoring shall be reported as a 30-day geometric mean. Maximum daily Enterococcus effluent monitoring shall be reported as the highest instantaneous maximum (the maximum of any single sample shall not exceed 104 CFU/100mL).

The existing HWWTP effluent monitoring schedule is presented in Table 7-41. An effluent flow meter is required for continuous flow rate monitoring. A refrigerated automatic composite sampler is required for the weekly and annual 24-hour composite sampling requirements. All other sampling requirements are discrete (grab) samples.

Table 7-41. Hagåtña WWTP Effluent Monitoring Requirements

Parameter	Monitoring Requirements	
	Frequency	Sample Type
Flow rate	Continuous	Metered
BOD_5	Weekly	24-hour composite
TSS	Weekly	24-hour composite
pH (hydrogen ion)	Weekly	Discrete
Settleable solids	Weekly	Discrete
Oil and grease, total recoverable	Weekly	Discrete
Enterococcus [c]	Weekly	Discrete
Chlorine, total residual (TRC)	Weekly	Discrete
Temperature	Weekly	Discrete
Ammonia	Yearly [b]	24-hour composite
Chronic toxicity [a]	Yearly [b]	24-hour composite
Priority pollutant scan	Yearly [b]	24-hour composite
Ambient monitoring	Quarterly	Discrete

a. The permittee shall attempt to ensure a total holding time from collection of the last portion of the composite sample until arrival at the laboratory of not more than 36 hours. EPA has granted an extension to the Permittee for the holding time due to logistical issues. The extended holding time shall not exceed 72 hours.

b. Yearly monitoring shall be completed by January 31 each year.

7.8.2.2 Outfall

The WWTP outfall discharges approximately 2,120 feet from shoreline into the Philippine Sea at a nominal depth of 275 feet below mean sea level. The submarine outfall was completed and placed in operation during December 2008.

7.8.3 Wastewater Characteristics

Characteristics of Hagåtña WWTP wastewater flow are described below.

7.8.3.1 Historical Flows and Loads

Hagåtña WWTP historical flows and loads are described below.

7.8.3.1.1 Historical Flows

The daily recorded influent flow for the 2010–2015 reporting years is presented in Figure 7-43. Monthly and annual averages for the same period are presented in Figure 7-44.

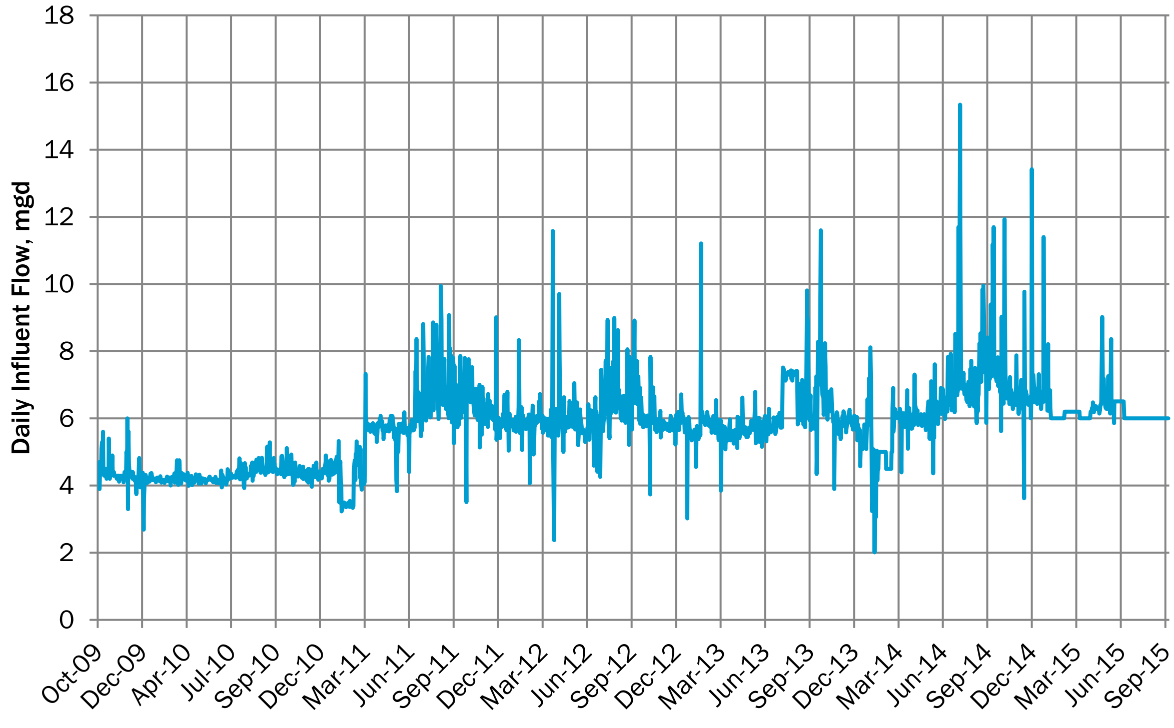

Figure 7-43. Hagåtña WWTP Daily Influent Flows

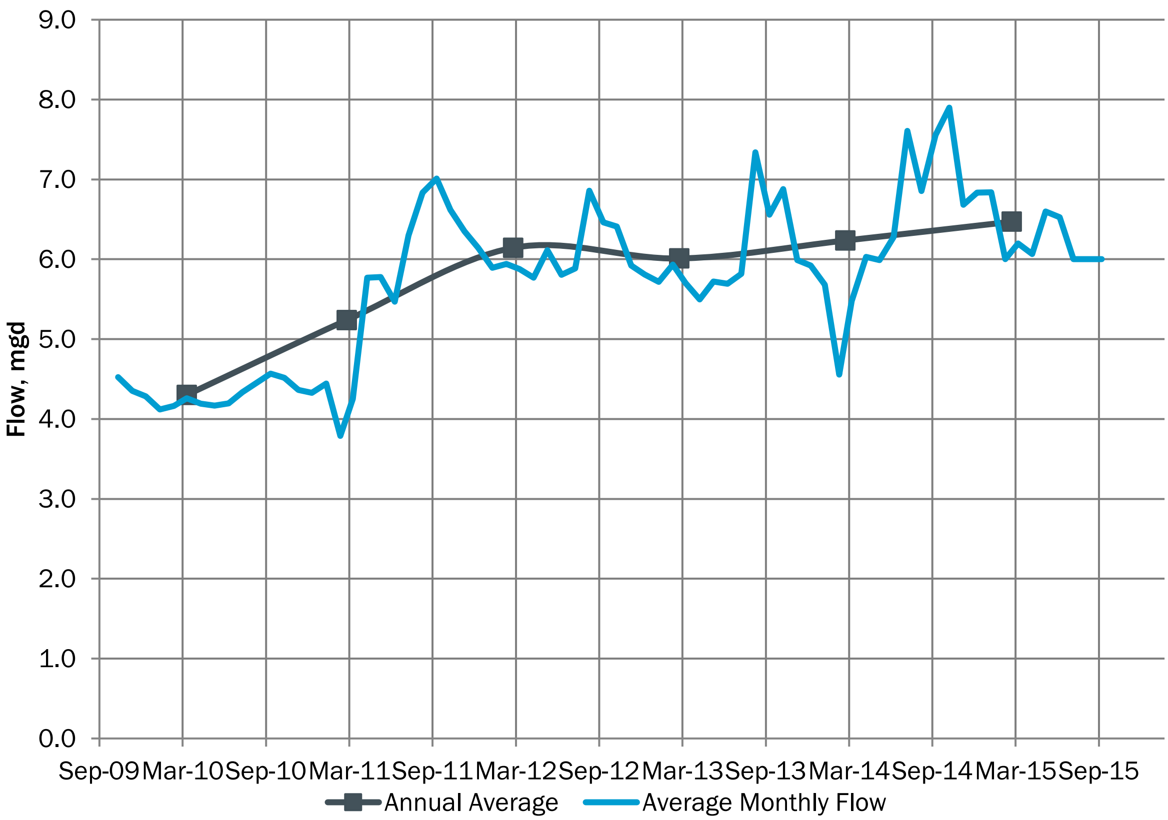

Figure 7-44. Hagåtña WWTP Average Influent Flows

The data shown in Figure 7-43 and Figure 7-44 is summarized in Table 7-42.

Table 7-42. Hagåtña WWTP Influent Flow, October 2009 through September 2015		
Description	Value	Peaking Factor
Average flow	5.72 mgd	1.0
Peak month average flow	7.9 mgd	1.4
Peak day wet weather flow	15.34 mgd	2.7

7.8.3.1.2 Historical Influent Characteristics

BOD_5 and TSS reported by GWA in DMRs for the Hagåtña WWTP for the period of October 2009 through September 2015 are shown in Figure 7-45.

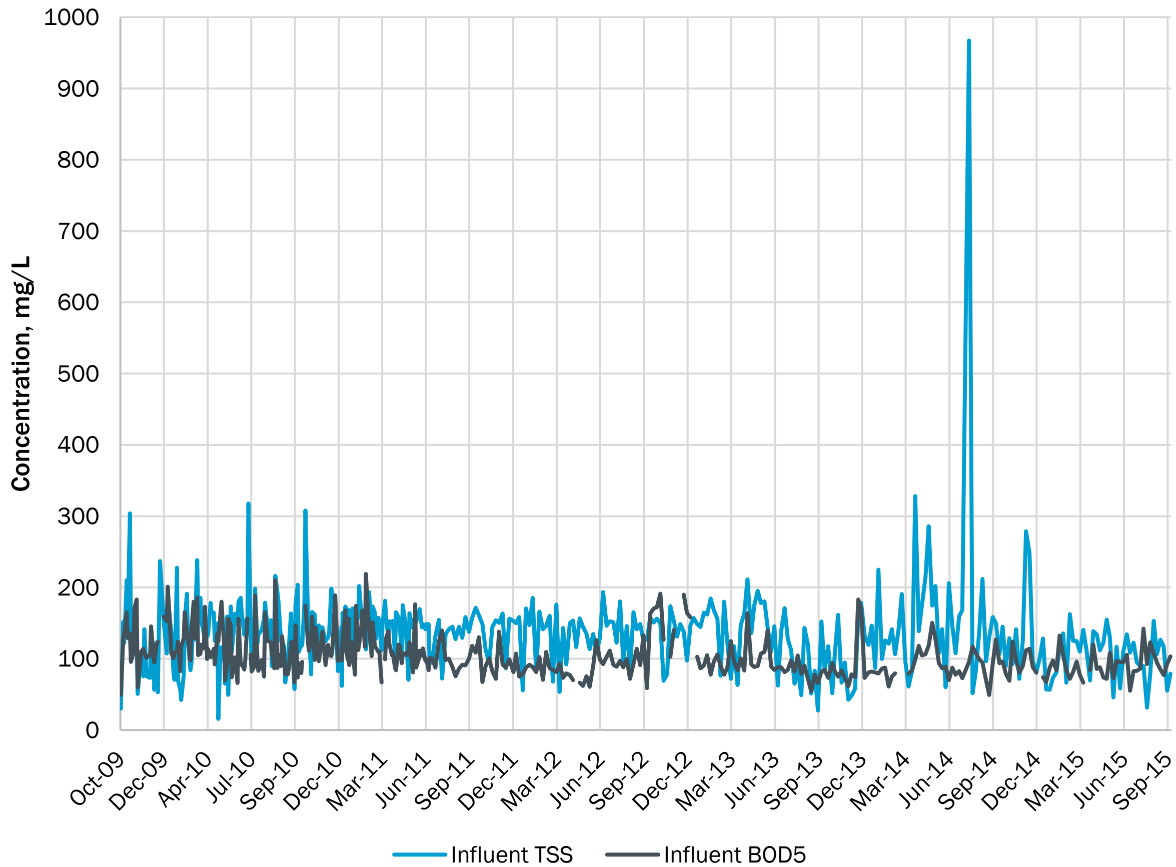

Figure 7-45. Hagåtña WWTP Influent BOD_5 and TSS Concentrations, October 2009 through September 2015

The data shown in the figure is summarized in Table 7-43.

Table 7-43. Hagåtña WWTP Influent Characteristics, October 2009 through September 2015		
Description	BOD_5	TSS
Average	105 mg/L	136 mg/L
Maximum	219 mg/L	967 mg/L
Minimum	49 mg/L	16 mg/L
Standard Deviation	30 mg/L	62 mg/L

The TSS maximum value of 967 was recorded on August 6, 2014, with no other day recording a TSS concentration higher than 328 mg/L in the six reporting years from October 2009 to September 2015.

7.8.3.2 Future Flows and Loads

Hagåtña WWTP future flows and loads are described below.

7.8.3.2.1 Flow Projections

Future flow projections are described below.

Population-Based Flow Projections

An analysis of Guam population projections is included in Volume 1, Section 4. Figure 7-46 presents projected wastewater flows at the Hagåtña WWTP based population growth in the Hagåtña Basin municipalities including the expected increase in population, military buildup, Tourism 2020 plan, and potential additional sewer connections as defined in Volume 1, Section 4.

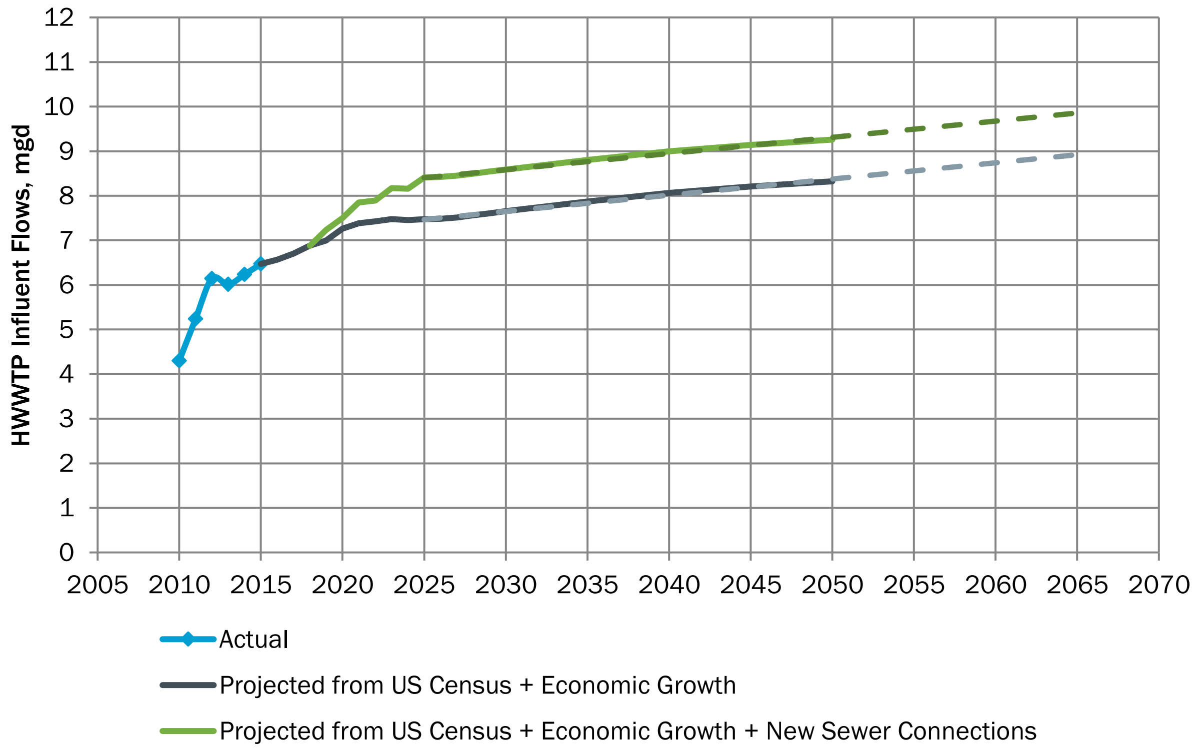

Figure 7-46. Hagåtña WWTP Average Flow Based on Population Projections and Onsite Conversions

Table 7-44 summarizes projected wastewater flows, which will also be used in the Facility Plan report for the Hagåtña WWTP (currently under development).

Table 7-44. Hagåtña WWTP Flow Projection Summary

Year	Average Flow (mgd)	Peak Month Flow [a] (mgd)	Peak Day Flow [b] (mgd)	Peak Hour Flow [c] (mgd)
2016	6.6	9.2	17.7	24.9
2025	8.4	11.8	22.7	32.0
2035	8.8	12.3	23.8	33.5
2050	9.3	13.0	25.0	35.2
2065	9.9	13.8	26.6	37.5

a. Peak month peaking factor = 1.4

b. Peak day peaking factor = 2.7

c. Peak hour peaking factor = 3.8

7.8.3.2.2 Future Influent Characteristics and Loads

Hagåtña WWTP future influent characteristics and loads are described below.

Potential Impacts to Wastewater Characteristics

Future flows are expected to increase based on a variety of factors; however, the characteristics of future wastewater flows are expected to remain relatively similar to current conditions.

In the Hagåtña wastewater basin, residential and commercial users will increase as the overall population increases. The overall economic distribution is not expected to change significantly. While there may be additional commercial users specifically catering to the military buildup personnel and their families, there will also be an increase of residential wastewater connections. For planning purposes, it is reasonable to assume that the characteristics will not change significantly.

Bringing online the majority of the existing septic tank users already adjacent to the wastewater collection will directly impact the amount of wastewater generated by certain neighborhoods and indirectly effect the composition of the flows at the Hagåtña WWTP. Septage from the Hagåtña basin is taken to the Northern District WWTP. Connecting additional septic tank users to the wastewater collection system should not drastically affect the composition of the wastewater at the Hagåtña WWTP. The net impact on wastewater characteristics due to connecting properties currently served by onsite wastewater systems is considered negligible for planning purposes.

Current Characteristics and Mass Loads

Table 7-45 summarizes current influent characteristics and mass loads.

Table 7-45. Hagåtña WWTP Existing Influent Mass Loadings

Description	Average Concentration (mg/L)	Mass Loading [a] (lbs/day)
BOD_5	105 [b]	5,700
TSS	136 [b]	7,350
Total N	30 [c]	1,650
Total P	6 [c]	350

a. Based on an average flow of 6.47 mgd.

b. Average of data collected from October 2009 through September 2015.

c. Estimate based on typical raw wastewater values.

Future Characteristics and Mass Loads

For planning purposes, future influent BOD_5 and TSS concentrations were estimated to be equal to the current average concentration plus one standard deviation. Table 7-46 summarizes the future wastewater characteristics that will be used for planning purposes.

Table 7-46. Hagåtña WWTP Wastewater Characteristics for Planning Purposes

Parameter	Concentration
BOD_5	140 mg/L
TSS	200 mg/L
Total N*	30 mg/L
Total P*	6 mg/L

** Influent data not available, typical values used.*

Table 7-47 presents the future influent mass loadings that will be used for planning purposes.

Table 7-47. Hagåtña WWTP Mass Load Projections

Parameter	Year			
	2025	2035	2050	2065
Average flow (mgd)	8.4	8.8	9.3	9.9
BOD_5 mass load (lbs/day)	9,800	10,300	10,900	11,600
TSS mass load (lbs/day)	14,000	14,700	15,500	16,500
Total N mass load (lbs/day)	2,100	2,200	2,350	2,500
Total P mass load (lbs/day)	420	440	465	495

7.8.3.3 Effluent Quality

GWA collects and tests daily effluent samples for BOD_5 and TSS. Section 7.8.3.3.1 presents pre-CEPT effluent quality at the Hagåtña WWTP, and Section 7.8.3.3.2 summarizes post-CEPT values.

7.8.3.3.1 Pre-CEPT Effluent Quality

Figure 7-47 shows a typical year of effluent quality data collected prior to implementation of the CEPT process.

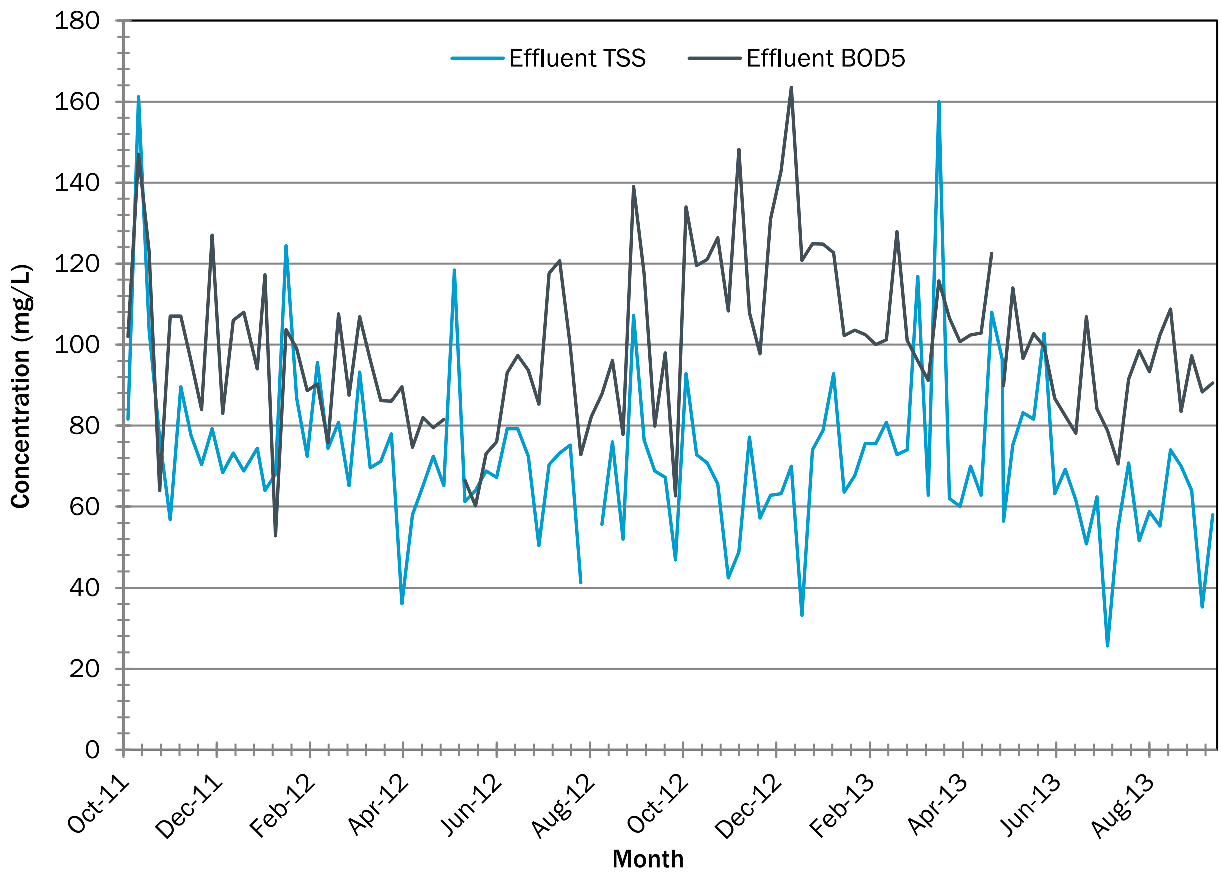

Figure 7-47. Hagåtña WWTP Typical Pre-CEPT Effluent Characteristics

Table 7-48 summarizes pre-CEPT effluent characteristics.

Table 7-48. Hagåtña WWTP Pre-CEPT Effluent Characteristics, October 2011 through September 2013

Description	BOD_5	TSS
Average	100 mg/L	72 mg/L
Maximum	164 mg/L	161 mg/L
Minimum	53 mg/L	26 mg/L
Standard deviation	20 mg/L	21 mg/L
Average removal rate [a]	-3 percent	40 percent

a. Influent averaged 98 mg/L BOD5 and 134 mg/L TSS during this time.

7.8.3.3.2 Post-CEPT Effluent Quality

Figure 7-48 presents the effluent quality data collected from May 2014 through April 2016. By this time, commissioning for the CEPT process was complete and it was being used as part of normal operations.

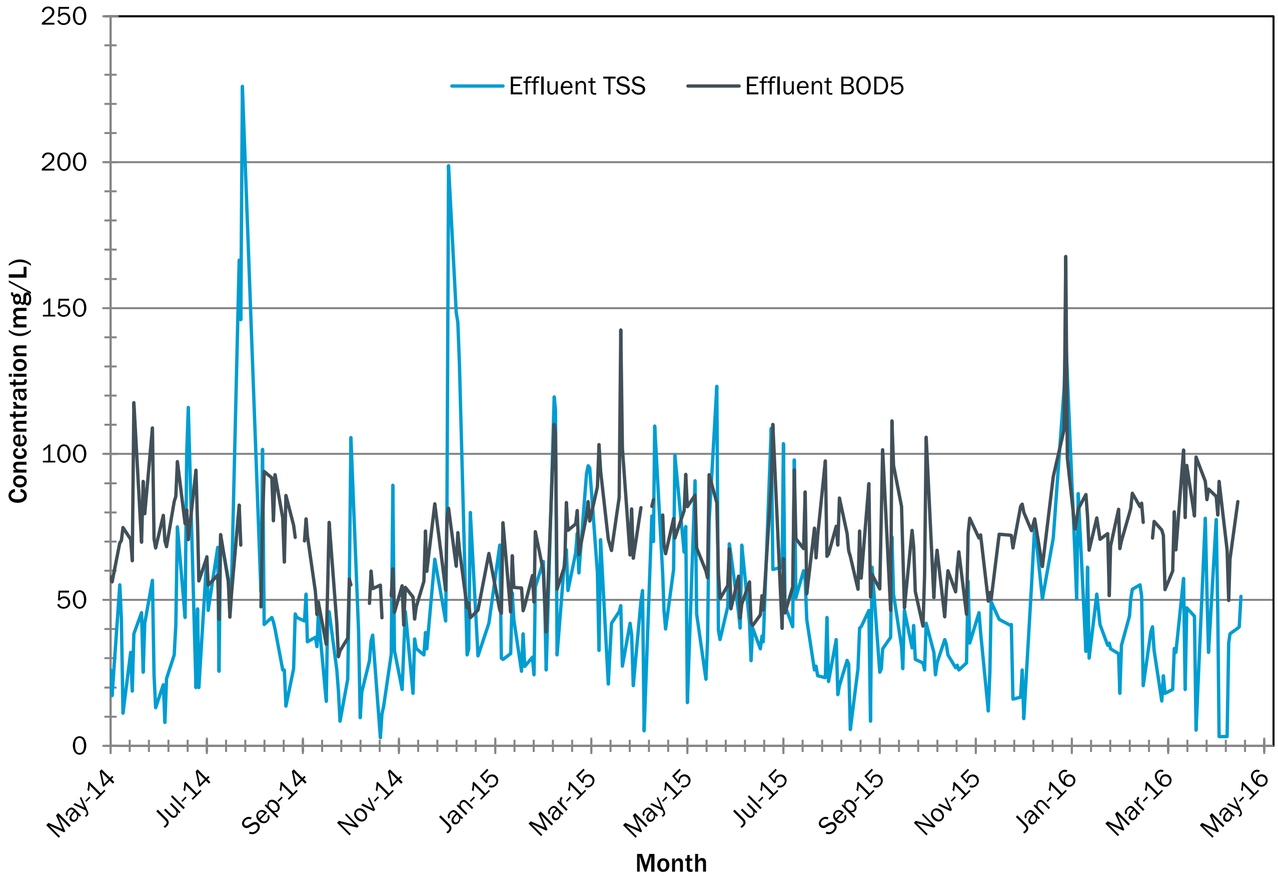

Figure 7-48. Hagåtña WWTP Post-CEPT Effluent Characteristics, May 2014 through April 2016

Table 7-49 summarizes the post-CEPT effluent characteristics.

Table 7-49. Hagåtña WWTP Post-CEPT Effluent Characteristics, May 2014 through April 2016

Description	BOD_5	TSS
Average	69 mg/L	47 mg/L
Maximum	168 mg/L	226 mg/L
Minimum	31 mg/L	3 mg/L
Standard deviation	18 mg/L	31 mg/L
Average removal rate [a]	27%	58%

a. Influent averaged 96 mg/L BOD_5 and 122 mg/L TSS during this time.

The Hagåtña WWTP provides primary treatment and the effluent characteristics are a direct representation of the efficiency of the existing primary clarifiers. Table 7-50 compares BOD_5 and TSS concentrations and removal rates before and after implementation of the CEPT process, and typical removal rates for primary clarifiers. The table shows that implementation of CEPT has increased BOD_5 removal from approximately zero to the removal found in typical primary clarifiers without CEPT. With CEPT, TSS removal has increased from lower than normal to within typical removal values.

Table 7-50. Hagåtña WWTP Primary Clarifier Performance Comparison

Description	Before CEPT Implementation		After CEPT Implementation	
	BOD_5	TSS	BOD_5	TSS
Average influent concentration	98 mg/L	134 mg/L	96 mg/L	122 mg/L
Average effluent concentration	100 mg/L	72 mg/L	69 mg/L	47 mg/L
Average removal rate	-3%	40%	27%	58%
Typical primary clarifiers [a]	25–40%	50-70%	--	--

a. Source: Metcalf & Eddy, Wastewater Engineering, Treatment and Reuse, 4th edition, 2003.

7.8.4 Recommended Improvements

Recommended improvement projects for the Hagåtña WWTP are described below.

7.8.4.1 Secondary Treatment

A facility plan is currently underway to evaluate options for upgrading the Hagåtña WWTP to full secondary treatment to comply with NPDES permit requirements. The plan will evaluate options to provide a complete secondary treatment plant at the current location as well as looking at options to relocate the entire treatment plant to an alternate location or to move the sludge treatment processes to another location. The secondary treatment upgrade is necessary, but will be a large financial burden to GWA so timing of the upgrade Is important. A secondary expansion project is included in the CIP planning with a budget based on a complete treatment plant upgrade at the current location. The scheduling for the HWWTP upgrade proposed in this WRMPU is for illustrative purposes only given the 20-year Master Plan forecast horizon.

7.8.4.2 Recommended Improvement Projects

Even without implementing secondary treatment processes, the Hagåtña WWTP requires regular maintenance. A WWTP rehabilitation project is recommended by the year 2027 (15 years after the completion of the CEPT improvements). The rehabilitation project should include (but is not limited to):

- Replacement or refurbishment of mechanical equipment and controls
- Inspection and repair of structures
- Rehabilitation of electrical equipment and control systems

In addition, a visual inspection of the ocean outfall pipe and diffuser should be performed every five years. The water quality monitoring program only examines the discharge, without a visual examination of the complete length of the outfall pipe.

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Section 8
Solids Management Plan

The long-term solids management plan presented in this section addresses wastewater solids processing and disposal needs for WWTPs owned and operated by GWA. Although solids treatment has been discussed for each facility, island-wide solids management planning is necessary to establish guidance and goals that support a comprehensive solids management program. The plan includes an evaluation of alternative solids management methods and approaches, and recommendations for improvements and future needs for a comprehensive wastewater solids management program.

8.1 Terminology

Throughout this document, the terms “wastewater solids”, “sewage sludge”, and “biosolids” are used. “Wastewater solids” and “sewage sludge” are both used to describe the residual material from the wastewater treatment process. Modern WWTPs remove waste from wastewater so that the clean effluent can be safely discharged to waterways or be used for beneficial purposes. The wastewater treatment process concentrates the waste materials present in sewage, creating wastewater solids or sewage sludge as a residual material.

The term “biosolids” is used to describe wastewater solids or sewage sludge that have undergone sufficient treatment to allow the material to be put to beneficial use. Wastewater solids that comply with USEPA standards for beneficial use can be called “biosolids”. USEPA standards include limitations on heavy metal and pathogen content, and requirements for vector attraction reduction. All “biosolids” are derived from wastewater solids; however, not all wastewater solids can be called “biosolids”. Wastewater solids currently generated at GWA’s WWTPs do not meet USEPA standards for recycling and cannot be called “biosolids”.

Biosolids are further differentiated by their pathogen content. “Class A” biosolids comply with the USEPA’s most stringent pathogen requirements, and are virtually pathogen free. “Class B” biosolids have higher pathogen content, but are just as safe to recycle if proper management practices are followed.

8.2 Wastewater Solids Overview

8.2.1 Current Wastewater Processing Systems Summary

GWA currently owns and operates seven WWTPs on the island of Guam: Northern District WWTP, Hagåtña WWTP, Agat-Santa Rita WWTP, Umatac-Merizo WWTP, Inarajan WWTP, Baza Gardens WWTP, and Pago Socio WWTP.

One other major WWTP is located on the island— the Apra Harbor WWTP. This secondary treatment facility is owned and operated by the U.S. Navy and has a designed average daily flow of 4.3 mgd, with a designed peak flow of 6 mgd. The Apra Harbor WWTP was not included in the Solids Management Plan developed for the 2016 WRMP update because GWA does not have responsibility for disposal of the Apra Harbor WWTP wastewater solids.

Table 8-1 summarizes solids treatment processes at each GWA wastewater facility. The average dry weather flow capacity is provided to provide perspective of the relative size of the facilities.

Table 8-1. Summary of GWA WWTP Solids Treatment Processes		
Facility	Design Average Dry Weather Flow Capacity	Solids Treatment Process Summary
Northern District WWTP	9 mgd	To be upgraded with full secondary treatment and will include aerobic digestion, and centrifuge dewatering.
Hagåtña WWTP	7 mgd	Currently a CEPT facility with aerobic digesters and centrifuge dewatering.
Agat-Santa Rita WWTP	1.6 mgd	Recently constructed facility includes aerobic digesters and dewatering centrifuges.
Umatac-Merizo WWTP	0.6 mgd	Aerated lagoon system: solids are stored in the bottom of the lagoon. Periodic solids removal is necessary (every 10 to 20 years).
Inarajan WWTP	0.19 mgd	Aerated lagoon system: solids are stored in the bottom of the lagoon. Periodic solids removal is necessary (every 10 to 20 years).
Pago Socio WWTP	25,000 gpd	Solids removed by pumping and transporting to another WWTP for processing.

8.2.2 Past Solids Management

Until 2007, wastewater solids were applied to land on farms on the island. However, solids were not treated, tested, applied, or reported in accordance with USEPA regulations. Upon receiving a Notice of Violation from the EPA in 2007, GWA discontinued the practice and began disposing of wastewater solids at the Ordot Dump. When the dump was closed, solids disposal was shifted to the Layon Landfill.

8.2.3 Current Solids Management

Grit, screenings, and dewatered sludge from the three largest WWTPs (Northern District WWTP, Hagåtña WWTP, and the upgraded Agat-Santa Rita WWTP) are currently disposed at the Layon Landfill.

Solids are allowed to accumulate in the two aerated lagoon systems (Umatac-Merizo WWTP and Inarajan WWTP) until removal is required. Solids removed from the lagoons are also disposed at the landfill.

8.2.4 Solids Projections

Only solids projections from the Northern District, Hagåtña, and Agat-Santa Rita WWTPs are considered for the current analysis. Sludge removed from Umatac-Merizo and Inarajan WWTPs will be infrequent and will represent only a very small fraction of the wastewater solids generated by the three larger GWA treatment plants.

Table 8-2 summarizes existing and projected future solids generation rates.

Table 8-2. Existing and Future Solids Projections				
Facility	Existing Average Dry Weather Flow	Projected 2035 Average Dry Weather Flow	Existing Solids Production	Projected Solids Production [b]
Northern District WWTP	6 mgd (5.6)	9 mgd	4.8 dry tons/day [c]	Solids: 30 wet tons/day (6 dry tons/day) Screenings and grit: 3 tons/day
Hagåtña WWTP	6 mgd (5.7)	9 mgd [a]	4 dry tons/day	Solids: 16 wet tons/day (5 dry tons/day) Screenings and grit: 3 tons/day
Agat-Santa Rita WWTP	0.75 mgd (old plant permit)	1.6 mgd (new plant)	0.3 dry tons/day [d]	Solids: 5 wet tons/day (1 dry ton/day) Screenings and grit: 0.6 tons/day

a. Hagåtña WWTP CEPT design flow is 7 mgd. Estimates assume rehabilitation/expansion of WWTP to increase to 9 mgd before 2035.

b. Estimates based on 30 percent solids for Hagåtña WWTP (primary treatment), and 20 percent solids for Northern District WWTP and Agat-Santa Rita WWTP (secondary treatment).

c. Northern District WWTP Facility Plan (BC, 2017).

d. Agat-Santa Rita WSE (EA Engineering, Science, and Technology, 2013).

8.3 Regulatory and Public Framework

This section addresses regulatory, policy, and public perception issues associated with solids management.

8.3.1 Regulatory Considerations

A number of regulatory considerations are associated with solids management. These considerations are presented below to provide a regulatory context to the solids planning efforts described in this report. Solids use and disposal is regulated at the federal and territorial levels, as described below.

8.3.1.1 USEPA

Solids from GWA WWTPs could be processed into biosolids, allowing for a variety of uses and disposal options. USEPA regulates biosolids use under the 40 CFR 503, which addresses land application, surface disposal, and incineration of biosolids. The 40 CFR 503 regulations are self-implementing and include monitoring, certification, and reporting requirements. Although a permit application must be submitted, USEPA Region 9 does not typically issue permits. Agencies are required to send an annual report to USEPA Region 9 summarizing and certifying their compliance with the rule.

The 40 CFR 503 regulations establish metal concentration limitations, pathogen density reduction requirements, vector attraction reduction requirements, and site management practices for land application of biosolids. Land application refers to the beneficial use of biosolids for their nutrient and organic matter content. Biosolids land application rates cannot exceed the fertilizer (N) needs of the vegetation that will be grown. The metal concentration limitations are based on a risk assessment prepared by USEPA. The pathogen density and vector attraction reduction requirements are based on past successful experience. Biosolids are classified as either “Class B” or “Class A” with respect to pathogen density. Class B biosolids have significantly reduced pathogen densities (as compared to raw sludge), but require application site management to ensure protection of public health and the environment. Class A biosolids have further reduced pathogen densities and do not require application site management to ensure protection of public health and the environment. Biosolids that meet the pollutant concentration, Class A pathogen, and vector attraction reduction requirements in 40 CFR 503 are typically called “Exceptional Quality Biosolids”, and can be sold or given away in bulk or bags without additional regulation by USEPA.

The 40 CFR 503 regulations also establish requirements for surface disposal of biosolids. Surface disposal includes monofills, surface impoundments, lagoons used for final disposal as opposed to treatment, waste piles, dedicated disposal sites, and dedicated beneficial use sites. In general, surface disposal of biosolids refers to application at high rates—in excess of crop nutrient requirements, if a crop is grown—as a management practice. The regulation establishes metal concentration limitations, pathogen density reduction requirements, vector attraction reduction requirements, and site management practices.

Incineration refers to combustion of sewage sludge or biosolids at high temperatures in an enclosed device. The 40 CFR 503 regulations establish metals concentration limits, total hydrocarbon emission limits, and management practices. Use or disposal of nonhazardous incinerator ash is not covered by 40 CFR 503 but other Federal regulations (40 CFR 257 and 40 CFR 258) cover these practices.

Disposal of solids is covered by 40 CFR 257, and landfill cover material requirements are established in 40 CFR 258.21, which allows states and/or owner/operators to use alternative cover materials when they do not present a threat to human health and the environment.

8.3.1.2 Criteria for Municipal Solid Waste Landfills: 40 CFR 258

Disposal of sewage sludge (or biosolids) to municipal solid waste landfills is regulated under 40 CFR 258. Sewage sludge that is disposed in municipal solid waste landfills cannot be classified as hazardous waste as defined by Federal rules. In addition, sewage sludge cannot contain free liquids and must pass the “Paint Filter Test” (EPA test method 9095B). Testing is often required to demonstrate that the sludge does not qualify as hazardous waste prior to initiation of disposal activities. Sludge or biosolids that are dewatered by centrifuge generally pass the paint filter test.

The Layon Landfill has the potential to continue as the primary disposal site for GWA’s wastewater sludge.

8.3.1.3 Local Regulation

Guam EPA regulations are presented in the Guam Administrative Rules and Regulations (GAR), Title 22 Guam EPA. Division IV of Title 22 regulates solid waste, including disposal of sludge and landfill cover materials. The water quality standards adopted by Guam EPA are also included in Chapter 5 of Title 22 of the GAR.

In addition, wastewater solids disposal should consider Air Pollution Standards and Regulations (GAR, Title 22, Chapter 1) as they pertain to dust control (for land disposal) or emissions (from incinerations systems).

8.3.2 Policy and Public Framework Considerations

In the continental United States, there are additional policies established by non-governmental organizations that should be considered for biosolids management planning purposes. Farm Bureaus may advocate against acceptance of wastewater (bio)solids for crops either for human or animal consumption. While solids treatment on Guam does not currently meet Class A biosolids regulations, future solids processing should consider potential policy changes throughout Guam.

The general public's reaction to the overall disposal or reuse of wastewater solids is an important aspect of solids management planning. The location of solids processing systems needs to consider the typical reaction from the public, varying from curiosity and fascination to suspicion and revulsion.

8.4 Solids Markets and Disposition

This section describes potential outlets for wastewater solids products, whether for beneficial use or disposal. Characteristics of the products that could potentially be produced and potential markets for the products are discussed.

8.4.1 Solids Products

Wastewater solids products can take a number of different forms, as described below.

8.4.1.1 Dewatered Cake

Dewatered cake represents the most basic and common form of biosolids products. Dewatered cake is produced using mechanical dewatering technologies, such as belt filter presses or centrifuges. Dewatered cake products typically consist of 85–70 percent moisture (15–30 percent solids) and have a gelatinous, bread dough consistency. The color, odor, and pathogen density characteristics of dewatered cake products are a function of the processes used to treat the biosolids prior to dewatering. Dewatered cake products can be produced with pathogen densities that achieve Class A standards.

Currently, both Northern District WWTP and Hagåtña WWTP dewatering centrifuges produce dewatered cake at approximately 30 percent solids content. The new Agat-Santa Rita WWTP will also produce dewatered cake from centrifuges.

8.4.1.2 Soil Amendments

Dewatered cake biosolids can be mixed with various other materials and processed to create soil amendments (such as compost) or topsoil replacement products. Potential feedstock materials that can be used include green waste, wood chips, sawdust, sand, lime, cement kiln dust, wood ash, and others. Soil amendment products are generally treated to Class A pathogen density standards. The soil amendment class of products usually has a pleasant, earthy odor and pleasing overall appearance to the general public.

8.4.1.3 Dried Products and Fertilizers

Dewatered cake biosolids can be dried to form fertilizer products. Drying methods include solar drying and thermal drying. This class of products can take a wide variety of forms. Solar dried biosolids typically contain less than 40 percent moisture and can have a dusty, soil-like appearance. Solar dried products may meet Class A pathogen density standards. Solar drying is usually land-intensive; therefore, it may not be a practical option for GWA's limited treatment plant lots. In addition, the length and intensity of Guam's wet season may only allow for seasonal solar drying during dry months.

Thermally dried biosolids products generally contain less than ten percent moisture. The product appearance is a function of the drying technology used, and can range from uniform spherical pellets with little dust to angular, non-uniform, dusty products. Thermally-dried biosolids products generally have a slightly stronger, more pungent odor than soil amendment products, but fewer odors than dewatered cake. The overall appearance of thermally-dried products is generally acceptable to the general public. Uniform, spherical products with low dust content are generally preferred over angular, non-uniform, dusty products.

8.4.2 Land Application Market for Class B Biosolids

Potential land application markets for Class B biosolids are described below.

8.4.2.1 Agricultural Land Application

Dewatered biosolids (Class A or Class B) can be spread on farm fields and used as soil amendment and fertilizer. The practice is used extensively on the U.S. mainland. Agencies often contract with service providers who secure users and permits, haul and spread dewatered cake on the farmers' fields, and provide all required monitoring and reporting. The farmer plants, grows, and harvests the crop after the biosolids are applied, and generally pays nothing for the biosolids. The applied biosolids provide macro and micro-nutrients for the crop, and increase moisture retention in the soil. Typically, biosolids are applied to land that is used to grow animal feed crops. Figure 8-1 shows a biosolids spreading operation in California.

Figure 8-1. Land Application of Dewatered Cake

Agricultural land application is a seasonal market. Land application activities are generally not possible (and may be prohibited by local regulations) during the wet season. Farm fields are usually too wet during this time of the year to allow access to the heavy equipment needed to spread biosolids without damaging the soil structure.

Land application rates are a function of the nutrient content of the biosolids and fertilizer needs of the crop. Typical land application rates are 2–5 dry tons per acre (per year). For planning purposes, assuming GWA produces approximately 12 dry tons per day of biosolids (Section 8.2) and the biosolids are applied at a rate of 5 dry tons per acre, GWA would need access to 876 acres of application area.

An Agriculture Census prepared by the U.S. Department of Agriculture in 2009 indicates that a total of 1,000 acres of Guam is farmed, mostly to grow food crops. Animal feed crops are not listed as typical Guam crops. Furthermore, most farms are small (less than 10 acres), making it more difficult to assemble a sufficient land base to provide a reliable program. There does not appear to be sufficient farm land growing appropriate crops on Guam to pursue land application of dewatered cake as a biosolids management option.

8.4.2.2 Dedicated Land Application Sites

Land application on land owned by wastewater agencies appears to be more sustainable than land application on private property. Many small wastewater agencies apply their biosolids to property they own that is adjacent to or near the WWTP of origin.

Dedicated land application sites are generally accepted by the local community, provided that they remain a good neighbor with respect to odors, dust, and other nuisance conditions.

8.4.3 Land Application Market for Improved Biosolids Products

Biosolids can be processed to create products with improved characteristics when compared with the typical Class B dewatered cake. Improved products can range from Class A dewatered cake to heat-dried pellets, compost, or other soil amendments. The aesthetic qualities of this broad category of “improved products” vary widely, as will marketability of the products for agricultural land application.

8.4.3.1 Class A Dewatered Cake

Upgrading treatment to produce Class A dewatered cake reduces the pathogen density in the biosolids, but does not improve the aesthetic qualities of the product. Class A dewatered cake is a product that does not require regulation to protect human health and the environment. However, local regulations have been enacted in some communities that limit (or ban) use of Class A biosolids in agriculture. Therefore, the market for Class A dewatered cake is somewhat similar to the market for Class B dewatered cake, although with less regulation.

Neighbors of land application sites cannot distinguish between Class A and Class B dewatered cake products, because they look and smell the same. Therefore, production of a Class A dewatered cake product does not guarantee that the receiving community will accept the use of the product.

8.4.3.2 Dried Biosolids – Pellets and Granules

Heat drying facilities can produce pellets or granules, which can be more easily transported and used. Fertilizer pellets can be used in bulk for agricultural purposes to grow animal feed and other crops. The pellets are similar in size and shape to conventional granular fertilizer materials, and conventional spreading equipment can be used.

The market potential for heat dried pellets in agriculture generally appears to be greater than for Class A dewatered cake due to the improved aesthetic qualities of the product. The product appearance and use resembles fertilizer rather than manure, and the pellets can be produced to be similar in size and shape to conventional fertilizer materials. The product contains minimal moisture, which minimizes truck traffic. Conventional spreading equipment is used to apply the product, and neighbors of sites where the product is used are less likely to react negatively.

The target market for the product is likely to be similar to dewatered cake biosolids, as marginal soils are used to grow animal feed crops. In addition, it may be possible to find product users who grow higher-value crops due to the improved product aesthetics. Product revenue is expected to be minimal due to the low cost of competing conventional fertilizing materials, but use of the product will likely prove to be more acceptable to the receiving communities than dewatered cake.

8.4.3.3 Compost and Other Soil Amendments

Soil amendments are generally only used in agriculture to correct soil problems. The market for compost and other soil amendment products derived from biosolids in agriculture is expected to be limited due to the availability of competing products. The primary market for compost and other soil amendments is typically for landscaping purposes.

8.4.4 Landscaping

Improved Class A biosolids products can be used for commercial and residential landscaping. The most common products used for landscaping are compost and heat-dried pellets. The product quality must be excellent with no foreign material (e.g., plastics) present and no objectionable odor characteristics. Heat-dried products with uniform pellet size and low levels of dust are preferred from a marketing perspective. Improved Class A products can be distributed in bulk or bags.

The U.S. Department of Agriculture's organic food regulations do not allow use of biosolids products to produce organic foods. As a result, some home gardeners will choose to not use biosolids products in their yards.

8.4.5 Landfill Disposal

GWA currently disposes its dewatered sludge at the Layon Landfill. Dewatered solids are hauled to the landfill via truck and disposed with other municipal solid waste. The landfill has a double liner and leachate collection system to protect groundwater. Continued disposal at the landfill is a viable alternative.

High quality dewatered sludge has been used as alternative daily cover at landfills in California. It may be possible to use a well-digested dewatered sludge as daily cover material at Layon Landfill, pending regulatory approvals.

8.4.6 Overall Market and Product Assessment

Table 8-3 presents a simplified assessment of the current markets for solids products in Guam, and of future market potential.

Table 8-3. Biosolids Market Assessment

Market	Current Market Assessment	Future Market Potential
Agricultural land Application of Class A or Class B dewatered cake	Poor. A large area is required, and not enough large farms are available on Guam. Application is highly seasonal. Animal feed crops are not typically grown on Guam.	Poor. Market potential may improve as island communities trend toward a more sustainable future and may attempt to produce more food locally. However, biosolids products are not allowed for organic food production, and use of dewatered cake biosolids for conventional food production is problematic due to public perception issues.
Dedicated land application	Poor. A large area is required. A short dry season limits time that biosolids can be land applied without damaging site soils. There is a limited market for animal feed crops on Guam.	Poor. Market potential is not expected to improve.

Table 8-3. Biosolids Market Assessment		
Market	Current Market Assessment	Future Market Potential
Landscaping	Limited. Some small-scale use of improved biosolids products is possible, but not enough to serve as a primary solids management system.	Limited. Overall market is not expected to improve with time.
Landfill disposal	Good. This is the current solids management method on Guam.	Good, but availability may become limited as landfill reaches capacity. There is good potential for use as alternative daily cover material, which may require additional processing.

Table 8-4 presents the compatibility of various biosolids products with future markets. A product that is compatible with multiple markets presents lower risk to GWA than a product that is compatible with only a few markets. The table shows that heat-dried pellets and compost (including compost-like soil amendments) have the greatest compatibility with multiple markets.

Table 8-4. Biosolids Product and Market Compatibility				
Products	Agricultural Land Application	Dedicated Land Application	Landscaping	Landfill
Dewatered sludge				✓
Class B dewatered cake	✓	✓		✓
Class A dewatered cake	✓	✓		✓
Compost		✓	✓	✓
Dried products	✓	✓	✓	✓
Construction products				✓
Ash				✓

8.5 Processing Technologies

This section describes and evaluates a wide range of technologies that are available in the field of wastewater sludge processing. The discussion focuses on technologies that could potentially be added to augment thickening, digestion, and dewatering processes that are already implemented or will be implemented at GWA WWTPs. The processing technologies are discussed within the following categories:

- Non-digestion stabilization technologies
- Drying technologies

The technologies discussed and evaluated here include those that are commonly used in the industry (in North America or Europe).

Screening criteria are established later in this section. These criteria are applied to the processes described to develop technology recommendations.

8.5.1 Non-Digestion Stabilization Technologies

A number of non-digestion processing technologies are used to stabilize sludge, ranging from alkaline processes to composting, and including many thermal processes. Some of these processes produce a specific product (such as a fuel product or construction aggregate). Other processes are more general in terms of the final product and its end use or disposition.

8.5.1.1 Alkaline Stabilization (PSRP or Class B process)

Alkaline stabilization consists of adding sufficient quantities of quicklime (CaO) or other alkaline materials to sludge to raise the pH of the mixture above 12 for two hours or more. The high pH significantly reduces or eliminates biological activity and destroys pathogens. Biological activity in the mixture can resume if the pH of the mixture is allowed to decline over time; therefore, alkaline-stabilized biosolids cannot be stored for long periods of time. Raising the pH of sludge releases ammonia; therefore, air collection and odor control equipment is frequently required for an alkaline stabilization process.

8.5.1.2 Alkaline Treatment (Class A process)

Several proprietary processes are available that use a combination of heat and high pH to create Class A soil amendment products. Quicklime, cement kiln dust, or other alkaline materials are mixed with dewatered biosolids in heated or insulated reactors. The high heat of the chemical reaction (or supplemental heat addition) destroys pathogens. Raising the pH of biosolids releases ammonia and sometimes other odorous compounds; therefore, air collection and odor control equipment is frequently required for an alkaline stabilization process. The appearance of the finished product varies with each proprietary process and some products are significantly more aesthetically agreeable than others.

8.5.1.3 Composting – Unconfined

Composting is the controlled aerobic decomposition of organic matter to produce a humus-like material. Thermophilic temperatures are achieved through auto-heating during the composting process, destroying pathogens. Bulking agents are mixed with dewatered cake to increase the porosity of the mixture and add carbon. Typical bulking agents include wood chips or sawdust. In some co-composting operations, the bulking agent is municipal green waste. Unconfined composting is accomplished outside of an enclosed building or vessel.

The lowest-cost composting technique is normally the use of mixed windrows; however, this technique has high odor emissions. Open windrow composting should therefore not be considered unless a very remote site can be located and odor transport potential carefully evaluated.

8.5.1.4 Composting – Confined

Confined composting is composting within an enclosed building or vessel. The advantage of confined composting is that odors can be controlled. There are many different arrangements for confined composting—as simple as aerated static pile composting within a building, to systems using automated, mechanical mixing and transport during the process.

One proven process is the agitated bed system manufactured by a number of companies. The composting occurs within concrete bays that measure approximately 10 feet wide by 6 feet deep by 200 feet long. Automated machinery periodically mixes and moves the composting mixture. Feed materials are introduced at one end of the bay and the finished compost is removed at the other end. The system is enclosed within a building so that foul air can be collected and treated in a biofilter.

Key challenges for a local composting facility include finding an appropriate site and ensuring that odors are adequately controlled. Shredded green waste could serve as the bulking agent. The market for a compost product will need to be carefully evaluated prior to implementing a project.

8.5.1.5 Thermal Processing with Energy Recovery

The most direct method of exploiting the energy value of biosolids is thermal processing with energy recovery (incineration). This process consists of complete combustion of biosolids in fluidized bed or multiple hearth incinerators. Exhaust from the combustion reaction passes through heat exchangers to recover energy. Usually, the energy recovered is directed back to the combustion process to reduce or eliminate supplemental fuel requirements. Supplemental fuel requirements are very low if raw sludge is dewatered to approximately 30–32 percent solids. For digested sludge, higher solids content would be required to avoid supplemental fuel needs. Air pollution control devices, such as wet scrubbers, dry and wet electrostatic precipitators, fabric filters, and afterburners, are used to reduce emissions to acceptable levels.

8.5.2 Drying Technologies

Drying can involve a number of options, and solids content of dried biosolids can range from 40 or 50 percent for partially dried, and to more than 95 percent solids for fully-dried material.

8.5.2.1 Air/Solar Drying – Open Systems

Biosolids can be dewatered and dried using open system drying beds. Drying bed area requirements are a function of the mass of water that must be removed and climatic characteristics of the site. Covers to limit rainfall on the bed can be used in areas of higher precipitation. Regulatory agencies typically require that newly constructed drying beds are lined to prevent groundwater contamination by nutrients or salts in the biosolids. Asphalt concrete pavement and other materials have been used successfully for this purpose. Paved beds work well as they allow mechanical equipment to work on the beds.

Biosolids must be stabilized prior to air drying to limit odor emissions. In urban areas, uncovered/uncontained drying beds are usually limited in size and sometimes dewatering precedes drying beds to limit the area required.

Open system drying beds are not a practical solution for Guam due to the high humidity and long rainy season.

8.5.2.2 Air/Solar Drying – Within Structure

Recent innovations in air/solar dewatering/drying operations involve handling biosolids within a greenhouse or hot-house structure equipped with forced-air ventilation and automatic mechanical mixing. Humidity and air temperature are monitored within the greenhouse and ventilation fans are energized as needed to maintain suitable drying conditions. Mechanical mixing systems vary in type and complexity. Treatment of the discharged airstream is required for sites with close neighbors. One small system has been successfully implemented at the Town of Discovery Bay in Northern California.

8.5.2.3 Heat Drying – Graded Pellet Product

Heat drying technologies use thermal energy to evaporate almost all moisture from biosolids to create a Class A product. A wide variety of dryer technologies are available. For master planning purposes, technologies can be divided into processes that create graded pellet products similar to commercial fertilizer products, and those that create ungraded products.

Most of the heat drying technologies that create graded pellet products are “direct” dryers. In direct dryers, moisture removal is achieved predominantly by convective heat transfer. A hot air/gas mixture is generated by a fuel-burning furnace, which exhausts hot gases directly into the drying vessel. The hot gases come into direct contact with the dewatered sludge, causing the water to evaporate. Direct drum dryers are capable of making a high-quality biosolids product consisting of uniform, hard, spherical pellets similar in appearance (with the exception of color and odor) to commercial inorganic fertilizer products. Most of the largest thermal drying operations in the U.S. direct drum dryers to create biosolids products.

A process schematic of a direct thermal drum dryer system is shown in Figure 8-2. Dewatered biosolids are first mixed with dried biosolids pellets upstream of the drying drum to control the moisture content of the mixture within the dryer. This first step in the drying process accomplishes two important functions. First, it provides a means of reincorporating “fines” and undersized particles that are separated from the product in the screening step following the dryer. Second, the physical form of the biosolids is altered so it does not stick to the internal parts of the drying drum. This preliminary mixing step is critical to producing a pellet product from the dryer. The triple-pass drying drum rotates as hot air and sludge particles pass through. Biosolids particles exiting the drum are screened to separate product of the desired particle size for cooling and temporary storage while awaiting distribution to market outlets. Oversized particles are crushed and returned to the head-end of the process, along with undersized particles and fines. Dryer off-gases are treated with a condenser prior to recycling back to the furnace or discharging to the atmosphere following treatment in a regenerative thermal oxidizer. Recycling a large portion of the process air serves to decrease the volume of air requiring treatment prior to discharge and to increase the thermal efficiency of the process.

Heat-dried biosolids products must be stored properly or they can catch fire. If a pile of heat-dried biosolids absorbs moisture, it can autoheat and combust; therefore, proper design of product storage facilities is vital. Product storage silos are generally equipped with temperature sensors and inert gas blanketing to reduce fire potential.

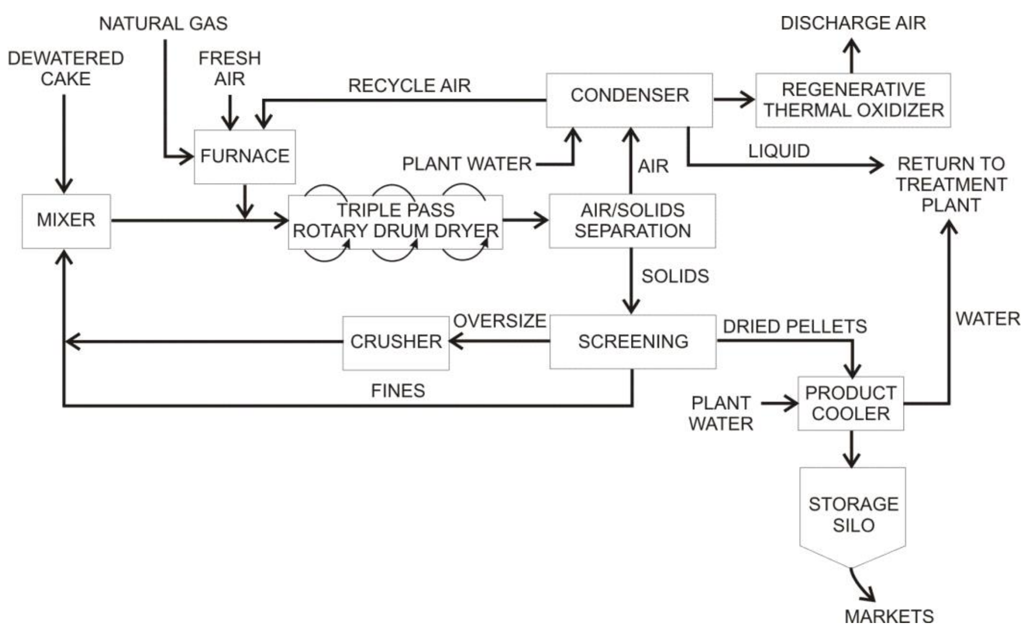

Figure 8-2. Direct Thermal Drum Dryer Producing Graded Pellet Product

8.5.2.4 Heat Drying – Ungraded Product

Heat drying technologies are also available that produce ungraded products containing wider variation in particle sizes and shapes. In general, ungraded heat dried biosolids products contain higher percentages of fines, creating dustier products. Ungraded product particles tend to be more angular in appearance and less like commercial fertilizer than graded products. Ungraded products will be more difficult to market as fertilizer than graded products because of these differences. Most of the ungraded products are produced using indirect drying technology.

Indirect dryers achieve moisture removal predominantly by conductive heat transfer, and the biosolids are kept separate from the primary heated drying medium (typically oil or steam). The drying medium is heated in a boiler or heat exchanger by the hot combustion gases from a fuel-burning furnace. An indirect dryer consists of a stationary vessel with an internal agitator and stirring assembly. The dewatered biosolids cake enters the stationary vessel of the indirect dryer and is continuously agitated and stirred during the drying cycle. Heat is transferred from the drying medium to the sludge by circulating the medium through the stirring mechanisms, augers, shafts, disks, dryer casing, or other equipment that comes into contact with the sludge.

A process diagram of a typical indirect thermal drying system is shown in Figure 8-3. Dewatered biosolids are introduced to the drying chamber, which is heated with hot oil or steam. Moisture evaporates from the biosolids as they move through the machine. Dried biosolids exit the dryer, and are cooled prior to temporary storage in a silo while awaiting distribution to market outlets. Vapor from the dryer passes through a condenser prior to treatment in a biofilter or other odor control process and discharge to the atmosphere. The volume of air that must be treated is significantly smaller than the direct drying systems because the furnace air does not come into contact with the drying biosolids.

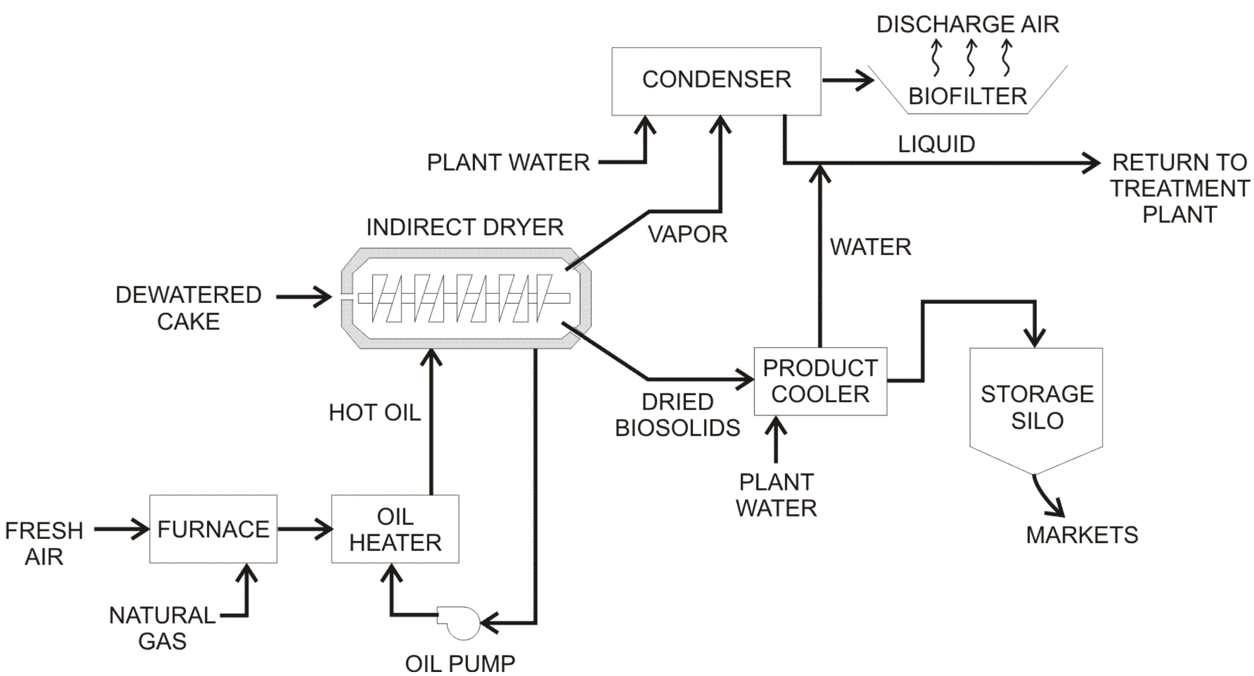

Figure 8-3. Indirect Thermal Dryer Producing Ungraded Product

Unlike direct dryers, indirect drying systems generally do not include product screening and recycle. The product storage silo must include temperature sensors and provisions for inert gas blanketing for fire prevention purposes. Indirect dryers may be operated on a continuous or batch basis, depending on the manufacturer.

8.5.3 Screening Criteria

The technologies described above are screened here to ascertain their suitability for implementation by GWA. The screening criteria are described below.

8.5.3.1 Experience at Similar Size Facilities

Biosolids and sludge processes often present significant materials handling challenges for equipment manufacturers. Experience has shown that design and operational problems are often encountered when equipment size is scaled-up to meet the needs of larger WWTPs. For these reasons, it is prudent for wastewater agencies to carefully consider whether technologies have had proven success at similarly sized facilities prior to investment of significant quantities of public funds.

8.5.3.2 Area Requirements

Some technologies require significantly more land area than others.

8.5.3.3 Odor Risk

Processing technologies produce varying degrees and types of odors, depending on the physical and chemical nature of the reactions involved. The likelihood/extent of odor produced and complexity of the odor control systems that will be required to mitigate odor risks needs to be considered.

8.5.3.4 O&M Complexity

Some technologies require higher skill levels to operate and maintain than others. The O&M complexity of technologies must be considered because qualified staff must be hired, trained, and retained throughout the life of a project. Finding and retaining skilled operators is difficult on Guam; therefore, operation of a complex system may require contracting to off-island companies at an additional cost. Therefore, process complexity is an important issue for GWA.

8.5.3.5 Worker Health and Safety

Some technologies present greater worker health and safety challenges than others due to chemical handling needs, high pressures, high temperatures, radiation, or equipment inertia. The complexity of maintaining a safe and healthy work environment must be considered.

8.5.3.6 Product Marketability

Market considerations were discussed in detail in a previous section of this report. Marketability of the products produced by technologies must be carefully considered, including regulatory compliance, product aesthetics, and market diversification potential.

8.5.3.7 Implementation Risks

Some technologies present greater implementation risks (such as permitting, overcoming negative public perceptions, etc.) than others.

8.5.3.8 Island Factors

As a remote tropical island, Guam has certain constraints, such as operational difficulties due to climate and isolation, that are unfamiliar to agencies in the mainland U.S. The long rainy season and humid conditions prevent the use of some technologies and limit product markets. Availability of spare parts and servicing of chosen technologies needs to be carefully considered.

8.5.4 Screening to Identify Viable Processes

Table 8-5 presents the result of the technology screening. Each technology was considered with respect to the screening criteria described above. The determination was then made whether:

- The technology has good potential for near-term application on Guam.
- The technology, with development and refinement, has potential for future use on Guam.
- The technology is not suitable for Guam.

Table 8-5. Technology Screening

Category	Technology	Screening Evaluation and Assessment	Recommended Option for GWA Facilities?
Non-Digestion Stabilization	Alkaline stabilization (PSRP)	Requires importation of bulk lime at significant cost. Significant odor concerns. Product is not marketable.	No
	Alkaline treatment (Class A)	Requires importation of bulk cement kiln dust or lime at significant cost. Significant odor issues. Poor product market.	No
	Composting – unconfined	Odors would be too high, even with digested feedstock. Unconfined composting considered infeasible.	No
	Composting – confined	Product market is limited; therefore, only small-scale operation is considered feasible. Extensive odor control would be required.	Future
	Thermal processing with energy recovery	Destruction of organics and pathogens. Concerns from air quality perspective, and major investment required. Ash is the final product, usually disposed. Continues to be a successful process at approximately fifty U.S. WWTPs. Public perception may be difficult to overcome. Expensive to implement. Inappropriate for a remote island like Guam due to process complexity and small volume of sludge produced.	No
Drying	Air/solar drying – open systems	Not feasible due to Guam climate.	No
	Air/solar drying – within structure	New, mechanical greenhouse-type systems. Odor must be highly controlled. High humidity on Guam would require large structure for given volume of solids.	No
	Heat drying – graded pellet product	Digested feedstock required. Very high degree of odor control needed. Experience is increasing in North America, and considerable experience in Europe at required scale. Safety is an issue – particularly fire/explosion. Class A product. Inappropriate for a remote island like Guam due to process complexity and high cost of imported energy.	No
	Heat drying – ungraded product	Digested feedstock required. Would only work with highly controlled systems and advances in dust control and safety. Waste heat from a future Guam Power Authority power plant located adjacent to Northern District WWTP could potentially be used.	Future

8.6 Solids Management Recommendations

The recommendations for GWA Solids Management are presented in Tables 8-3 through 8-5 are summarized below:

Primary Solids Management Approach. The recommended approach is to continue disposal of dewatered sludge at the Layon landfill. Landfill disposal is simple and reliable, and does not require capital investment in facilities to produce marketable biosolids products, nor does it require investment of human resources into operating a recycling program. GWA faces considerable challenges implementing system-wide court-ordered improvements and secondary treatment upgrades at the two largest WWTPs; therefore, attempting to add an optional biosolids recycling program to GWA's priority list is not advised. The CIP does not include any projects to implement biosolids recycling.

Secondary Solids Management. In the future, GWA could choose to recycle a portion of GWA's solids by converting the dewatered sludge to a Class A biosolids product with improved characteristics. We recommend that GWA continue to discuss opportunities with other agencies and consider jointly participating in projects if opportunities arise. Two potential opportunities are discussed below.

Future Composting. A portion of GWA's dewatered solids could potentially be composted using green waste as a bulking agent. This would serve to divert both green waste and dewatered sludge from the landfill, increasing capacity. An enclosed composting system is recommended for a future composting facility due to the long rainy season on Guam and for odor control. A Class A biosolids product with improved characteristics would be produced and marketed primarily for landscaping purposes. Debris removal from the sludge would be recommended prior to composting if implemented. The local compost market should be evaluated in more detail as part of the process of sizing the composting facility to avoid constructing too much production capacity.

Future Indirect Drying. The Guam Power Authority (GPA) is proposing to construct a new power generation facility adjacent to the Northern District WWTP. Waste heat from the GPA facility could potentially be used to dry dewatered solids to reduce landfill tip fee expenses or create a fuel for a future waste-to-energy facility (if developed). Another potential market for the dried solids would be use as alternative daily cover at the Layon Landfill. If the solids are dried to greater than 90 percent solids content, the product will qualify as Class A biosolids and could potentially be marketed for landscaping or land application purposes. The capacity of the heat dryer would be a function of the amount of available heat from the GPA facility. Odor control would be required for the drying process. GWA should continue to discuss opportunities with GPA as the power generation facility concept is developed.

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Section 9
SSES Evaluation

A Sewer System Evaluation Study (SSES) is a comprehensive and systematic approach that evaluates defects in sanitary sewer pipes and manholes to identify the scope and nature of I/I in a sewer collection system. An SSES is generally conducted in three phases: I/I analysis, field investigation to identify and prioritize the defects, and development and execution of rehabilitation projects to reduce the total amount of I/I in the system to the extent feasible. This section summarizes SSES work that has been done on Guam, findings from the SSES work, and findings from the latest modeling efforts.

9.1 2011 Court Order

In November 2011, a court order was issued by USEPA requiring GWA to conduct I/I analyses and an SSES program for most sanitary sewer basins on Guam (United States of America, 2011).

I/I analyses were conducted by collecting wastewater flow and rainfall data to determine the dry weather and wet weather wastewater flow rates generated within each sewer basin. For the areas identified to have excessive I/I, GWA was then required to perform an SSES to identify the source of the I/I. These investigative measures included manhole inspections to identify sources of I/I into manholes, smoke testing to identify sources of inflow not correlated to manholes, dye testing to identify cross connections in laterals and manholes, and CCTV to identify defects in pipes. The goal of the I/I analyses and the SSES program was to identify the issues that are potentially contributing to recurring wet weather SSOs, overloading of WWTPs, and/or bypasses at the WWTPs.

Other key court order reporting requirements pertinent to the I/I analysis and SSES work are as follows:

- Report summary of all SSO occurrences to USEPA on a quarterly basis summarizing the location, cause, and estimated volume of each SSO.
- Clean each gravity sewer main at least once every five years and clean at least 55 unique miles of gravity sewer main in a calendar year.
- Implement a hotspot cleaning program with more frequent sewer cleanings to address sewer areas with recurring blockages.
- Implement a CCTV sewer inspection program to include inspection and assessment of 40 percent of the gravity sewer within two years and all gravity sewers within five years.

9.2 Sewer System

Table 9-1 lists each sewer basin served by GWA and the status of the I/I analysis and SSES work. Most basins were analyzed in response to the 2011 court order. GWA has also done I/I analysis and SSES work for other basins not included in the court order. The basins are described in more detail in Section 2.

Table 9-1. Sewer Basins				
Region	**Basin**	**WWTP**	**Municipalities Served**	**Status in Court Order**
North	Northern District	Northern District	Dededo, Yigo, Andersen AFB, portions of Barrigada, Mangilao	Not included in the court order, but analyzed by GWA
North	Tumon	Northern District	Tumon and portions of Tamuning	Not included in the court order, but analyzed by GWA
Central	Hagåtña	Hagåtña	Agana, Agana Heights, Asan, Chalan Pago Ordot, Mongmong Toto Maite, Piti, Sinajana, portions of Barrigada, Mangilao, Tamuning, Yona	Included in the court order
South	Agat-Santa Rita	Agat-Santa Rita (effluent pumped to the outfall line of the Naval Base Guam Apra Harbor WWTP)	Agat, Santa Rita	Included in the court order
South	Baza Gardens	Baza Gardens	Talofofo, portions of Yona	Included in the court order
South	Inarajan	Inarajan	Inarajan	Not included in the court order and not studied by GWA
South	Umatac-Merizo	Umatac-Merizo	Umatac, Merizo	Included in the court order

Flow meters were installed in each basin to assist in identifying excessive I/I. The flow metering is described in more detail in Section 3.

9.3 Studies

This section describes the I/I analysis and SSES work that has been done for each sewer basin and the work that remains to be done.

9.3.1 Southern Basins

The southern sewer basins include the villages listed in Table 9-1. Fifteen flow meters were installed as part of the flow monitoring program to identify I/I locations in the southern basins. See Section 3 for the location of the meters.

Hydraulic Model

In August 2011, GWA engaged Veolia Water Guam to assess the sewer systems in the southern basins. Veolia subcontracted with MWH to develop a hydraulic model in InfoWorks for three sewer basins: Agat-Santa Rita, Baza Gardens, and Umatac-Merizo. The Inarajan sewer basin was not evaluated as part of this effort. As part of the field studies for the assessment, ADS Australia was subcontracted to install and operate 15 flow monitoring sites, three pump loggers, and five rain gages. The data was collected from the end of September to mid-November 2012. However, the model was not calibrated using the flow data and no runoff model was selected (MWH, 2013). Therefore, although the model was helpful to understand the network layout of the system, it did not fully depict the condition of I/I in the southern basins.

Infiltration and Inflow Study

In February 2013, Veolia Water Guam completed an I/I and SSES report using the flow and rainfall survey data collected by MWH for the same basins studied by MWH: Agat-Santa Rita, Baza Gardens, and Umatac-Merizo. The analysis was based on comparing the observed dry and wet weather flow to the acceptable limits set by USEPA of 120 gpcd for dry weather flow and 275 gpcd for wet weather flow. The number of housing units and the population data from the 2010 U.S. Census were compared with the number of buildings in the GIS and to the flows obtained from the MWH report. Although Veolia Water's scope also included obtaining and assessing groundwater data for the period of the flow survey, the data was not available, so tidal influences on flows were evaluated instead for piping near the coast (Veolia Water, 2013). For the flow meters with pipe invert elevations less than the highest tide elevation, tidal influences were evaluated to inspect the quality of the dry weather flow data. Flows in all 15 meters exceeded the USEPA wet weather flow limit. However, six meters, including five in Baza Gardens and one in Agat-Santa Rita, met the USEPA dry weather flow requirement.

Sewer System Evaluation Study

In April 2013, following the MWH and Veolia Water analyses, GWA contracted work for inspection of sewer lines in Guam to BC and Underground Services, Inc. (USi). The project included developing a fieldwork plan, conducting smoke testing in 107,000 linear feet of sewer line, reviewing existing CCTV inspections, 585 manhole inspections, and dye testing in the Agat-Santa Rita and Umatac-Merizo basins. Based on these analyses, GWA developed the following rehabilitation capital improvement projects:

- Agat-Santa Rita SSES Based Sewer Rehabilitation
- Umatac-Merizo SSES Based Sewer Rehabilitation
- Baza Gardens – Talofofo SSES Based Sewer Rehabilitation
- Route 2 Agat – War-in-the-Pacific Sewer Rehabilitation

Summary of Remaining Work

The SSES and I/I work has been completed for the southern basins. The remaining work in the southern basins includes construction of the capital improvement projects listed above.

The following issues were noted by GWA operations staff in the Umatac-Merizo basin and should be investigated during additional SSES work.

- Between Pump Station No. 17 and Pump Station No. 18, operations staff have noticed open cleanouts at some houses (see Figure 9-1). There appears to be an excessive amount of I/I in this area, which causes Pump Station No. 17 to continuously pump.
- Near Pump Station No. 17, operations staff have noticed flooding (see Figure 9-1). The flooding has been along the main pipeline running along Route 4 and on the 8-inch piping coming from the northwest.

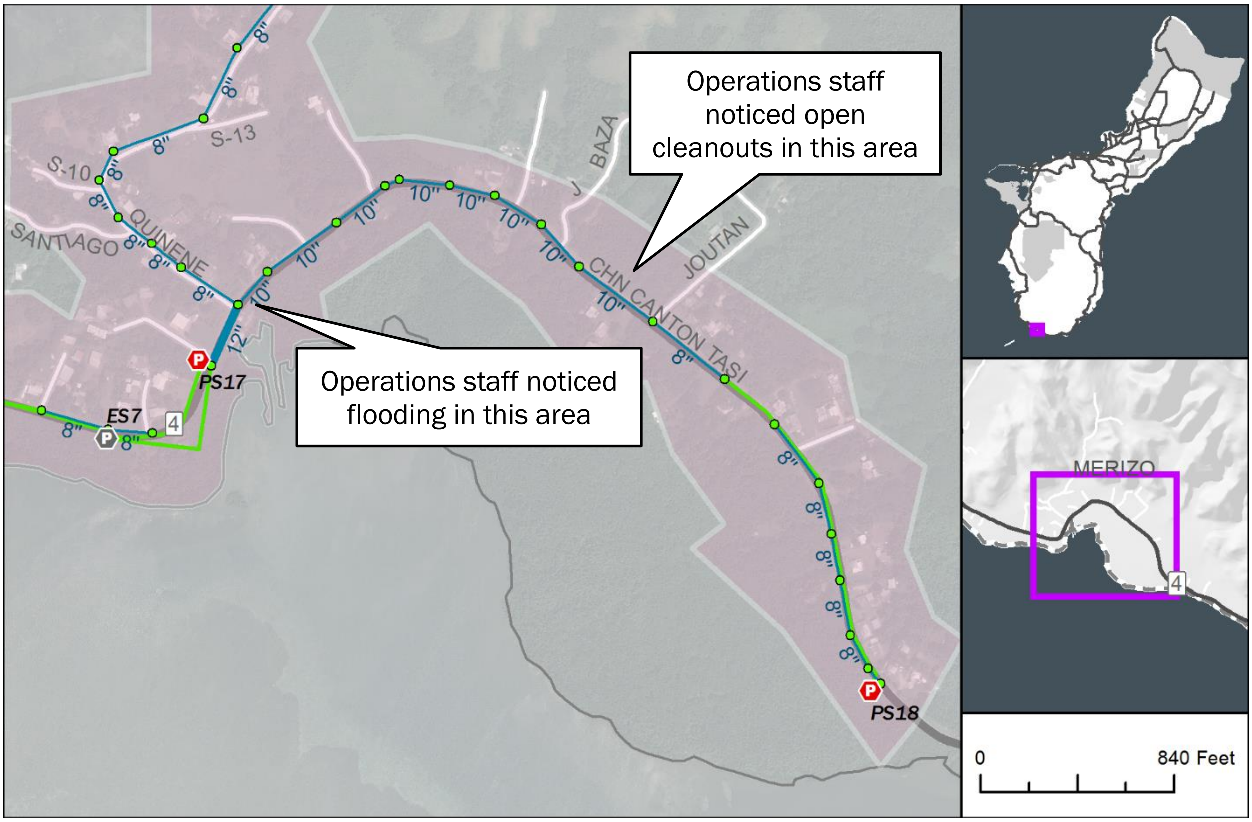

Figure 9-1. I/I Issues in Umatac-Merizo

9.3.2 Central/Hagåtña Basins

The central sewer basins include the villages listed in Table 9-1. Thirty-one flow meters were installed to identify excessive I/I in the Central/Hagåtña basin. Section 3 lists the location of 14 of the flow meters that were used for the model calibration.

Infiltration and Inflow Study

LYON Associates, Inc. issued an I/I report in March 2014 based on flow data collected from November 2013 to February 2014 for 31 flow meters and five rain gages. A prioritized list of potential high I/I locations was developed based on the wet weather I/I values. I/I was calculated as I/I per inch-diameter-mile of sewer piping for each flow meter basin. However, additional analysis by BC of the rainfall and flow data indicated that many flow meters installed in 2013 and 2014 suffered velocity sensor failure or sewer surcharges that affected the quality of the data (BC, 2014). Resulting analyses indicated that the priority listing should be modified according to normalized I/I rates.

Subsequently, a supplemental report was issued by BC to further validate and reprioritize locations where SSES should be used to determine likely sources of excessive I/I. The report was titled *Supplementary I/I Analysis for the Central Basin Sewer System* (BC, 2013). Typically, I/I rates are normalized and ranked on a per-foot of sewer piping, per capita, and per acre basis. However, because there are large areas that are currently unsewered, the per capita per acre normalization was not done. Instead, I/I was normalized on a per feet basis by taking the excess I/I and dividing it by the total length of pipe contributing to excess I/I. The supplemental report was used to support the program goal of maximizing the use of limited funds to fully prioritize SSES work in the Hagåtña basin. The recommendations for SSES work were refined based on locations with higher than average normalized I/I rates.

Implementation Plan and Schedule

After the completion of the I/I studies conducted in the Hagåtña basin, BC completed a report titled *Implementation Plan and Schedule – SSES Based System Improvements for Agana (Central) Sewer Basin Collection System* to document the progress of SSES work in the Hagåtña basin. The report included a summary of findings from the SSES work in Piti village. The report also formulated an implementation plan for completing the remaining SSES work in the Hagåtña basin. The report recommended that GWA continue smoke testing and manhole inspection using a revised priority listing in three phases: high and medium priority locations in the first phase, low priority locations in the second phase, with the third phase focused on locations for which additional flow monitoring data is required. Several locations did not have conclusive evidence that excessive I/I existed because the flow meter data quality was suspect. The priority levels were based on the sites that would yield the greatest benefit in reducing I/I. The highest priority sites mostly included sewer lines along the coast that are susceptible to seawater infiltration into manholes.

Sewer System Evaluation Study

In November 2014, GWA executed a contract with HDR to perform a comprehensive SSES in the Hagåtña basin based on the re-prioritized lists. A draft report was submitted by HDR in May 2017 that summarized the data and findings that were collected up to the date of the report. The project has included 496 manhole inspections, about 107,000 linear feet of smoke testing, dye testing, and review of over 75,000 linear feet of CCTV data (HDR, 2017). The draft report listed the following findings:

- A direct stormwater connection was found in Tamuning that is estimated to contribute about 0.8 mgd to the wastewater collection system during a 2-year storm.
- The investigation identified the following for rehabilitation:
  - 42 manholes
  - Targeted repairs for 7,200 feet of gravity pipe
  - CIPP for 23,775 feet of gravity pipe
  - Spot repair and CIPP for 5,170 feet of gravity pipe
  - Complete replacement of 1,080 feet of gravity pipe

Summary of Remaining Work

After a final report for the Hagåtña basin is delivered, the rehabilitation and replacement work recommended in the report needs to be integrated with the improvement plans developed in this WRMPU. The integration would include prioritizing the work using the risk-based analysis.

The following issues were noted by GWA operations staff in the Hagåtña basin and should be investigated during additional SSES work. Additional SSES work may include identifying and locating illegal or improper stormwater connections to the sewer system.

- At the Barrigada Pump Station (shown in Figure 9-2), operations staff have estimated that flows have risen from about 20,000 gallons per hour to over 100,000 gallons per hour during heavy rain.
- As noted in Figure 9-2, operations staff have seen significant I/I issues. The I/I appears to be due to direct connections to stormwater drainage and from homes below the grade of Route 10. Stormwater puddles in the yards of those homes, so homeowners have opened cleanouts to drain their yards. Operations staff have observed a manhole cover come off near Wendy's due to high sewer flows during heavy rainfall.
- Near the Harmon Pump Station (shown in Figure 9-3), operations staff believe that there are some direct stormwater connections from nearby warehouses. The minimal dry weather flows increase greatly during storms.

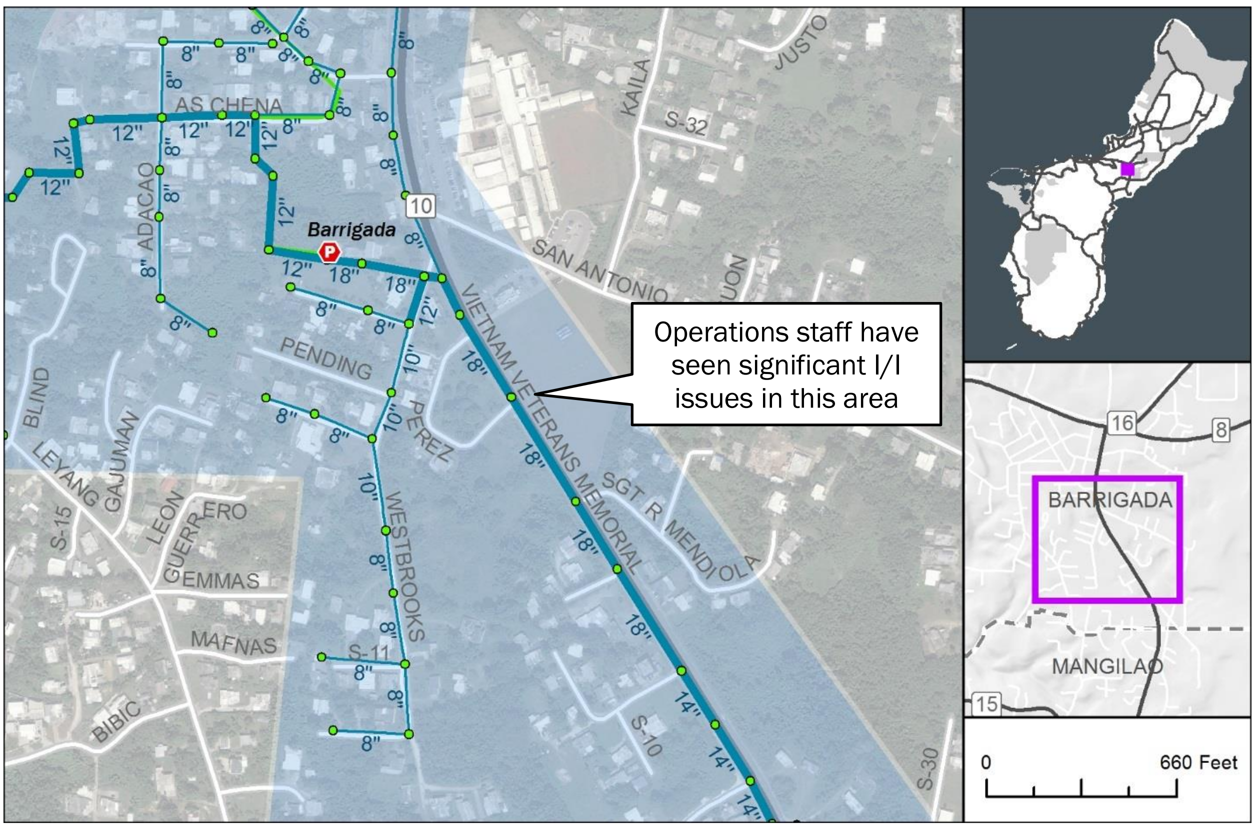

Figure 9-2. I/I Issues Near the Barrigada Pump Station

Figure 9-3. I/I Issues Near the Harmon Pump Station

9.3.3 Northern Basins

The northern sewer basins include the villages listed in Table 9-1. Twenty flow meters were installed in the northern basins with ten in the Tumon basin.

Infiltration Inflow Study

In April 2015, Stanley Consultants and EA Engineering, Science, and Technology completed I/I studies for the Northern District and Tumon basins. Analysis for Tumon and the Northern District was based on flow data collected as part of the temporary flow and rainfall monitoring program in the northern area of Guam. Priority areas were identified and recommended for further SSES efforts to identify potential sources of excess I/I. These priorities were determined based on flow monitoring results, inputs from GWA, topographic information, and flow meters with highest peaking factors.

In Tumon, several areas were selected for smoke testing and in the Northern District collection system, CCTV was selected as the method of inspection. Details on the priority areas can be found in the *Northern District Sewer System Evaluation Study* (Stanley, 2015).

Sewer System Evaluation Study

GWA retained the services of Stanley Consultants to develop and implement further I/I studies and SSES. The project approach involved first obtaining and evaluating flow monitoring data collected over ten weeks. Flow meters were placed in manholes to isolate and identify flows in various areas. Several I/I point sources were identified through the completed investigations and analysis. Further investigation of sump pump and roof drain investigations were recommended, particularly in Tumon, where the large resorts were suspected to be contributing stormwater to the sanitary sewer.

Summary of Remaining Work

Further investigation of sump pump and roof drain connections to the sanitary sewer system is recommended, as the large resorts may be contributing stormwater to the sanitary sewer system through sump pumps and roof drains and large volumes of I/I may go unnoticed. By the year 2020, GWA plans to line the main interceptor trunk sewer using CIPP from Anderson AFB to the Northern District WWTP to improve capacity and reduce I/I. The lining will be done through a grant from the DoD Office of Economic Adjustment. Parts of the Tumon sewer system are currently being analyzed and repaired to reduce hot spots and I/I. Evaluations are taking place to improve the capacity of the Fujita Pump Station and force main. Flow monitoring should continue in Tumon and the Northern District sewer basins to monitor progress in reducing I/I and to evaluate future projects that may be necessary.

9.4 I/I Estimate

I/I estimates were generated using the calibrated computer model and the design storm discussed in Appendix C. The total flow volume due to RDII during the design storm was calculated for the area draining to each flow meter used in the model calibration. The flow volume per length of gravity pipe was calculated for each flow meter. Figure 9-4 summarizes the average I/I per foot of gravity pipe per inch of rainfall for each flow meter used to calibrate the model. Because some areas were not metered, I/I estimates were not calculated for those areas.

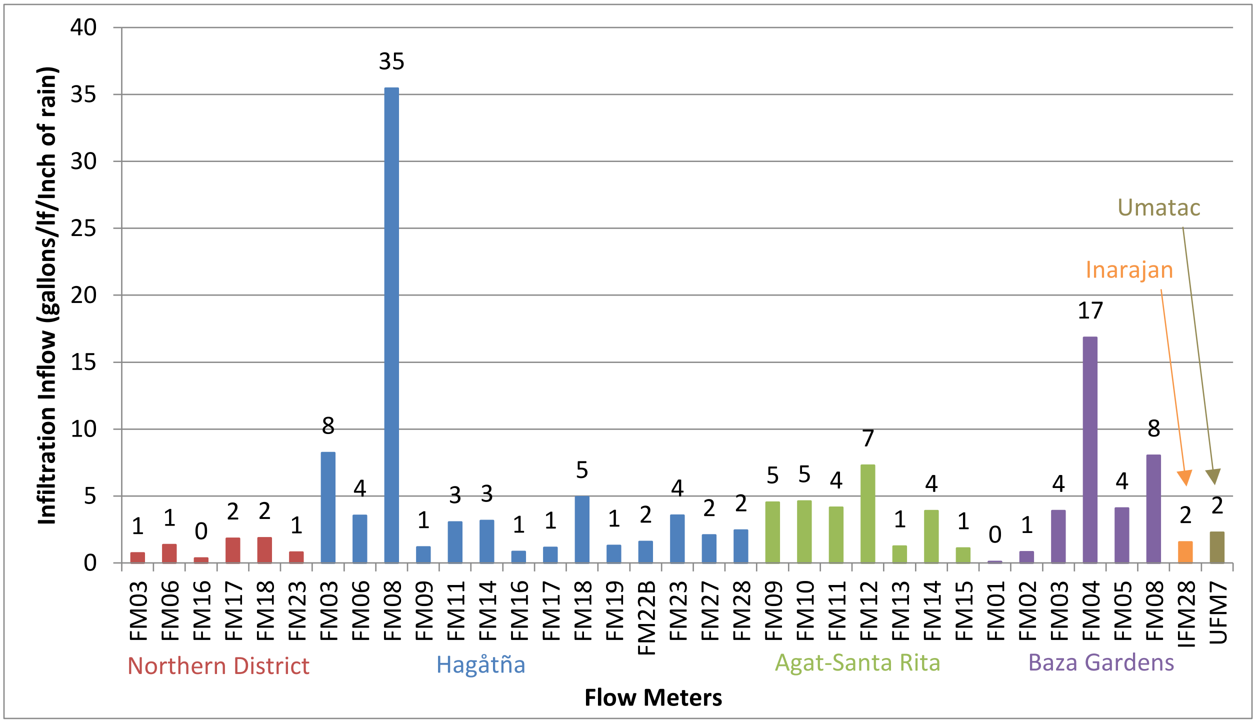

Figure 9-4. Average I/I per Foot of Gravity Pipe for Each Flow Meter

Table 9-2 summarizes the average I/I per foot of gravity pipe for each basin. Figures 9-5 through 9-7 show the flow meter locations in each sewer basin and the average I/I per foot of gravity pipe per inch of rainfall for pipes draining to those meters. These I/I values can be used to prioritize future SSES work by targeting the areas with the highest I/I.

Table 9-2. Average I/I per Basin Drainage Area	
Basin	Average I/I (gallons per linear foot per inch of rainfall)
Agat-Santa Rita	3.42
Baza Gardens	3.94
Hagåtña	3.44
Inarajan	1.55
Northern District	1.12
Umatac	2.27

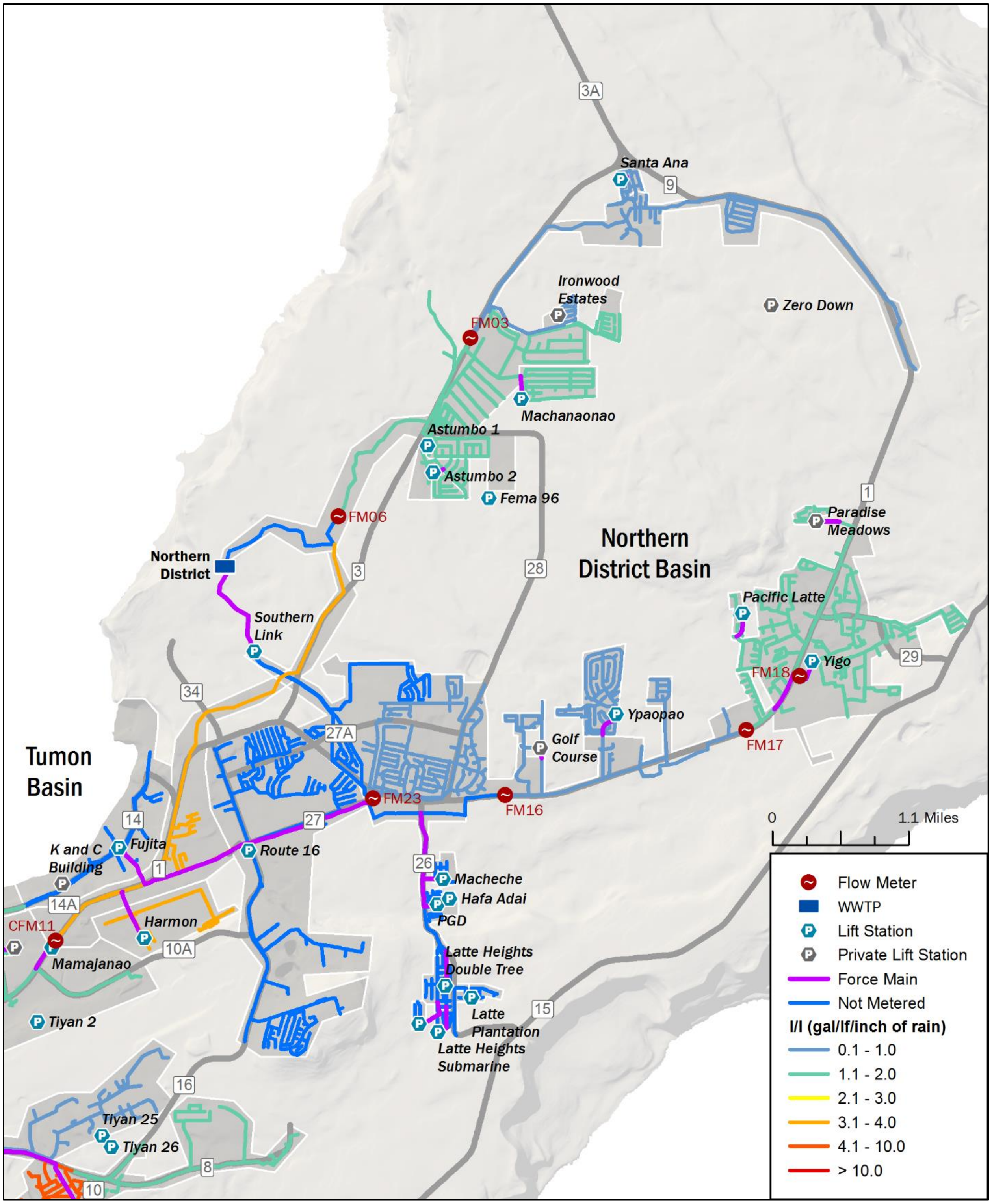

Figure 9-5. Average I/I per Foot of Gravity Pipe per Inch of Rain for North Area Flow Meters

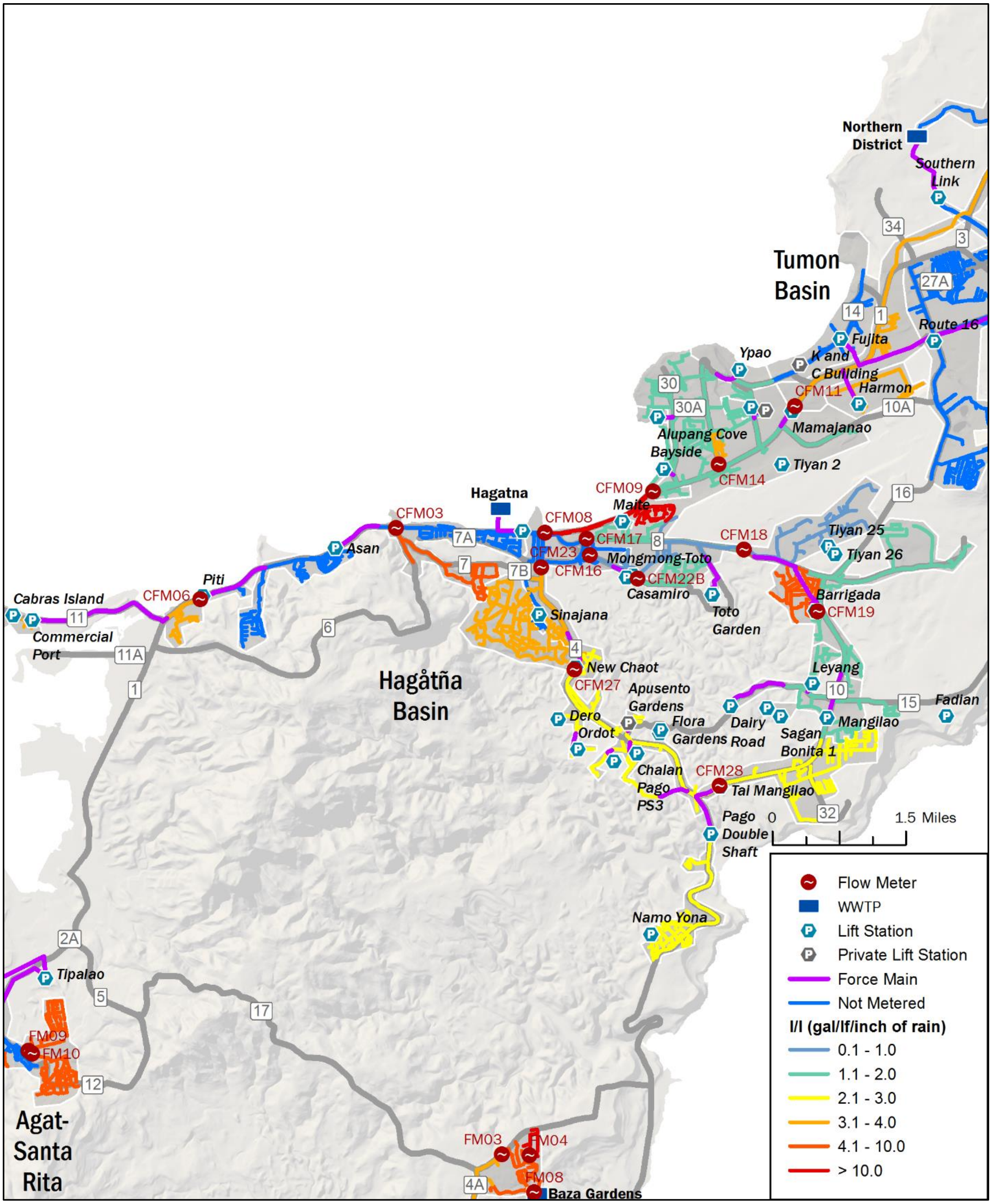

Figure 9-6. Average I/I per Foot of Gravity Pipe per Inch of Rain for Central Area Flow Meters

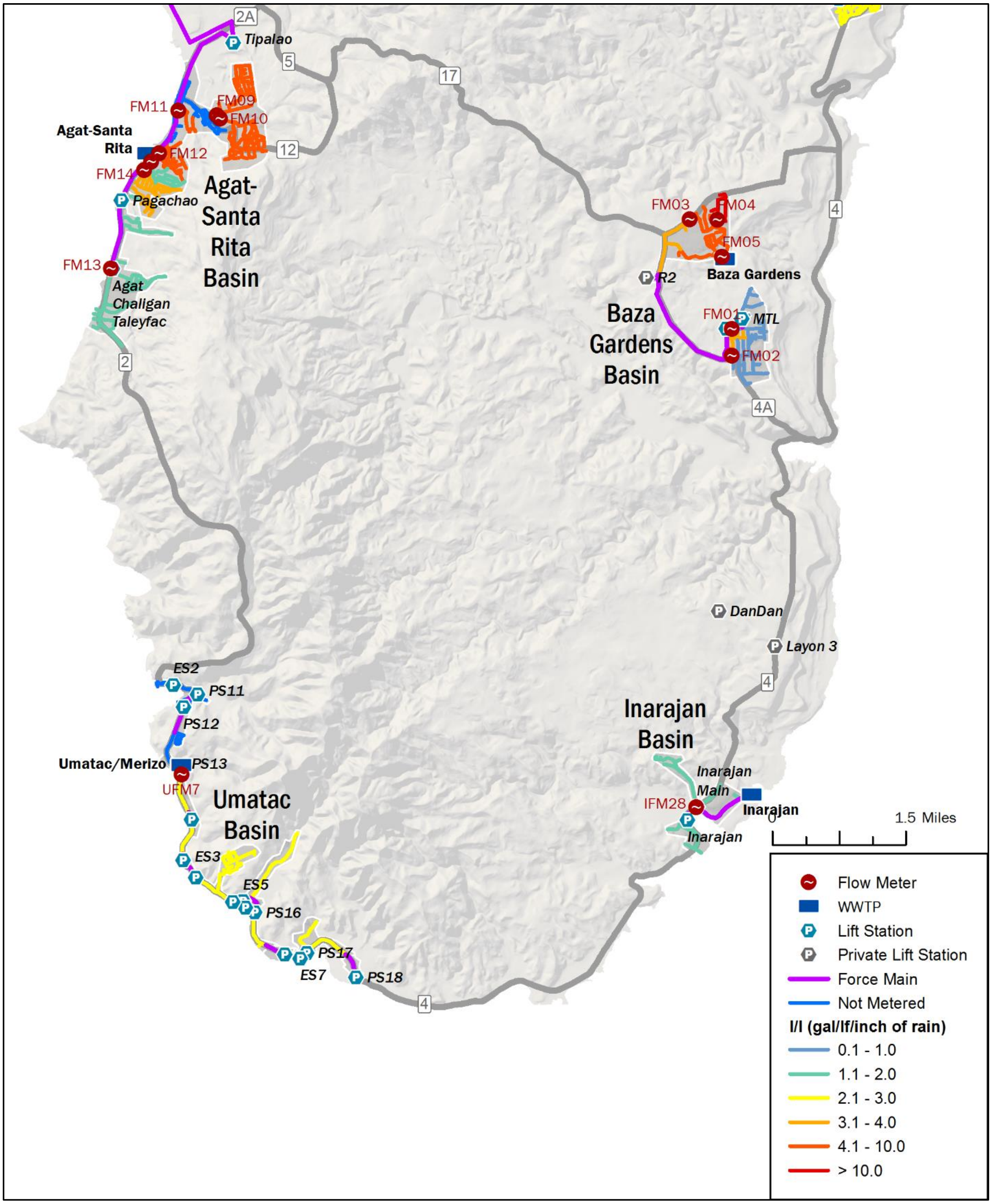

Figure 9-7. Average I/I per Foot of Gravity Pipe per Inch of Rain for South Area Flow Meters

9.5 Recommendations

The remaining work summarized for each basin should be reviewed and completed, including:

- **Southern basins:** SSES and I/I work has been completed for the Southern basins. The issues noted in Figure 9-1 should be investigated.
- **Central/Hagåtña Basins:** the 2017 draft report for the Hagåtña basin recommended rehabilitation and replacement of manholes and pipes. This work should be added to the CIP after the final report is delivered. The additional issues noted in Figure 9-2 and Figure 9-3 should be investigated.
- **Northern Basins:** further investigation of sump pump and roof drain connections to the sanitary sewer system is recommended, as the large resorts may be contributing stormwater to the sanitary sewer system through sump pumps and roof drains and large volumes of I/I may go unnoticed. Investigations should continue to reduce hotspots and I/I in the Tumon area. Flow monitoring should continue in Tumon and the Northern District sewer basins.

It is recommended that a project be developed to continue with I/I and SSES assessments. This project will cover I/I and SSES assessment work as necessary to determine the high probable locations where I/I is occurring. This project will review areas defined in the hydraulic model or based on operating anomalies where high I/I is suspected, but the root cause has not been determined adequately to define a repair, rehabilitation, or expansion project. This project may include installation of temporary flow meters, smoke testing, CCTV, surveying, or other investigation techniques. This project can also include minor repairs necessary to decrease I/I. This can include but is not limited to manhole inspection, manhole mapping, raising manholes, manhole seals, repairs to covers and frames, and other manhole defects. It can also provide limited gravity sewer inspection and limited gravity sewer repair and rehabilitation. Project MP-WW-Misc-02 in Section 11 discusses this ongoing I/I and SSES project.

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Section 10

General System Recommendations

This section summarizes general system recommendations and relative project rankings for GWA's wastewater system. Recommendations for specific components of the wastewater system are provided at the end of Sections 4 through 9.

10.1 2006 WRMP Recommended Projects

Recommended projects in the 2006 WRMP were analyzed and incorporated into this updated plan as appropriate. Some of the 2006 projects have been completed, some are still required, and others are no longer needed. Projects that are still required are incorporated into the recommendations listed in this section. Volume 1 summarizes the status of each 2006 WRMP project.

10.2 Projects Summary

Potential improvement projects were developed for the wastewater system and costs were assigned to each project. Table 10-1 lists all proposed improvement projects with estimated planning costs. Each project was assigned a unique project number, grouped by the system component. Detailed descriptions of each proposed project are included in Section 11. The cost estimates in this section and Section 11 are for budgeting purposes only and presented in 2017 dollars. Some projects are recurring projects that will be executed multiple times before 2037. Volume 1, Appendix D contains additional information for the cost estimates.

Table 10-1. Wastewater System Improvements Projects with Estimated Costs

Report Project Number	Report Project Name	Recurring Project [a]	Total Cost [b]
Gravity Pipeline Projects			
MP-WW-Pipe-01	Gravity Pipe Rehabilitation/Replacement Program	Annual	$25,962,000
MP-WW-Pipe-02	Barrigada Pump Station Pipe Rehabilitation/Replacement	No	$5,425,000
MP-WW-Pipe-03	Route 1 Piti Pipe Rehabilitation/Replacement	No	$4,478,000
MP-WW-Pipe-04	Southern Link Pump Station Pipe Rehabilitation/Replacement	No	$711,000
MP-WW-Pipe-05	Agana Heights Pipe Replacement	No	$3,228,000
MP-WW-Pipe-06	Northern District Route 1 Capacity Replacement - Phase 1	No	$15,431,000
MP-WW-Pipe-07	Northern District Route 1 Capacity Replacement - Phase 2	No	$14,579,000
MP-WW-Pipe-08	Northern District Route 1 Capacity Replacement - Phase 3	No	$11,128,000
MP-WW-Pipe-09	North Dededo Capacity Replacement - Phase 1	No	$9,803,000
MP-WW-Pipe-10	North Dededo Capacity Replacement - Phase 2	No	$12,443,000
MP-WW-Pipe-11	Route 16 Capacity Replacement	No	$7,539,000
MP-WW-Pipe-12	Barrigada Capacity Replacement	No	$609,000
MP-WW-Pipe-13	Mangilao Capacity Replacement	No	$2,142,000

Table 10-1. Wastewater System Improvements Projects with Estimated Costs			
Report Project Number	Report Project Name	Recurring Project [a]	Total Cost [b]
MP-WW-Pipe-14	Dededo Capacity Replacement	No	$3,313,000
MP-WW-Pipe-16	Yigo Capacity Replacement	No	$22,089,000
MP-WW-Pipe-17	Mamajanao Capacity Replacement	No	$5,570,000
MP-WW-Pipe-18	Agat-Santa Rita Capacity Replacement - Phase 1	No	$3,012,000
MP-WW-Pipe-19	Agat-Santa Rita Capacity Replacement - Phase 2	No	$4,093,000
MP-WW-Pipe-20	Agat-Santa Rita Capacity Replacement - Phase 3	No	$5,940,000
MP-WW-Pipe-21	Baza Gardens Capacity Replacement - Phase 1	No	$4,213,000
MP-WW-Pipe-22	Baza Gardens Capacity Replacement - Phase 2	No	$2,612,000
MP-WW-Pipe-23	Baza Gardens Capacity Replacement - Phase 3	No	$0 (planned for after 2037)
MP-WW-Pipe-24	Umatac-Merizo Capacity Replacement	No	$2,730,000
MP-WW-Pipe-25	Piping Near Bayside Lift Station	No	$250,000
MP-WW-Pipe-26	Finile Drive Rehabilitation - Agat	No	$830,000
MP-WW-Pipe-27	Septic/Cesspool System Reduction Program	Annual	$78,967,000
MP-WW-MH-01	Manhole Rehabilitation Program	Every 2 Years	$3,150,000
Force Main Projects			
MP-WW-FM-01	Force Main Rehabilitation/Replacement Program	Every 3 years	$9,468,000
MP-WW-FM-02	Replace Yigo Lift Station Force Main	No	$3,332,000
MP-WW-FM-03	Route 1 Asan Force Main Rehabilitation/Replacement	No	$2,298,000
MP-WW-FM-04	Hagåtña WWTP Force Main Rehabilitation/Replacement	No	$7,400,000
Lift Station Projects			
MP-WW-Pump-01	Lift Station Rehabilitation/Replacement Program	Every 2 Years	$49,896,000
MP-WW-Pump-02	Tumon Basin - Fujita Lift Station Analysis	No	$16,940,000
MP-WW-Pump-03	Replacement of Former Navy Pump Station (Donut Hole)	No	$1,320,000
WWTP Projects			
MP-WW-WWTP-01	Hagåtña WWTP Primary Treatment Repair/Rehabilitation Program	No	$24,000,000
MP-WW-WWTP-02	Hagåtña WWTP Secondary Treatment Upgrade	No	$208,000,000
MP-WW-WWTP-03	Inarajan WWTP Repair/Rehabilitation Program	No	$2,000,000
MP-WW-WWTP-04	Pago Socio WWTP Pump Station Conversion	No	$3,138,000
MP-WW-WWTP-05	Umatac-Merizo WWTP Repair/Rehabilitation Program	No	$4,500,000
MP-WW-WWTP-06	Agat-Santa Rita WWTP Repair/Rehabilitation Program	No	$13,500,000
MP-WW-WWTP-07	Baza Gardens Cross Island Pipeline - Preliminary Treatment Equipment Repair and Rehabilitation Program	No	$2,500,000
MP-WW-WWTP-08	Northern District WWTP Completion	No	$17,000,000
MP-WW-WWTP-09	Ocean Outfall Inspection Program	Every 5 Years	$600,000
Other Wastewater Projects			
MP-WW-Misc-01A	Update Wastewater Collection System Model (Major Update)	No	$500,000

Table 10-1. Wastewater System Improvements Projects with Estimated Costs			
Report Project Number	Report Project Name	Recurring Project [a]	Total Cost [b]
MP-WW-Misc-01B	Update Wastewater Collection System Model (Continued)	Every 2 Years for 8 years	$800,000
MP-WW-Misc-02	I/I and SSES Assessments	Every 3 Years	$2,400,000
MP-WW-Misc-03	Miscellaneous Wastewater Improvements	Every 2 Years	$7,128,000
MP-WW-Misc-04	Fats, Oils, and Grease Study	No	$150,000

a. Annual costs (without a number of years in parenthesis) are annual costs for the entire 20-year planning period.

b. Costs are the total projected for the 20-year planning period in 2017 dollars.

The following projects were existing wastewater system projects that were being designed or constructed at the time of this report, and are not included in the project rankings or project summary sheets because they are in progress.

Pipeline rehabilitation projects under design:

- Route 1, Asan to Hagåtña Sewer Line Improvements
- Route 2 Sewer Line Improvements
- Route 4 Sewer Line Improvements
- Tumon Hot Spots Sewer Line Improvements
- Tamuning Hot Spots Sewer Line Improvements

Pipeline rehabilitation projects under construction:

- Macheche Road Sewer Line Improvements
- Southern SSES Phase II Sewer Line Improvements

Lift station projects under design:

- Talofofo Area New Pump Stations
- Bayside Pump Station Improvements

Lift station projects under Construction:

- Critical Northern Area Sewer Pump Stations

WWTP Upgrades under design:

- Northern District WWTP

WWTP Upgrades under construction:

- Agat-Santa Rita WWTP
- Baza Gardens WWTP Conversion to Pump Station
- Umatac-Merizo WWTP

10.3 Project Rankings

During development of the wastewater system improvement projects, a workshop was held with key GWA representatives to discuss the projects and develop a non-financial ranking system to prioritize implementation. The project rankings also provide a general sequence for which projects should be scheduled in the future financial plan. Each project was ranked with a score from 1 (lowest importance) to 3 (highest importance) for each of nine categories used in the rankings. Volume 1, Section 1 describes the rankings in more detail. Based on the project ranking system and overall financial analysis, selected projects to pursue in the 20-year Master Plan time frame are included in Volume 1, Section 11.

The ranked wastewater projects are listed in Table 10-2.

Report Project Number	Report Project Name	Score out of 100	Health and Safety	Regulatory or Mandated	Reliability and Redundancy	Capacity	Operation, Maintenance, and Rehabilitation	Environmental Impact and Resource Use	Revenue and Expenditures	Customer Service and Stakeholder Confidence	Economic Development
Gravity Pipeline Projects											
MP-WW-Pipe-01	Gravity Pipe Rehabilitation/Replacement Program	90	1.7	2	3	1.7	3	2.7	1.7	2	1
MP-WW-Pipe-02	Barrigada Pump Station Pipe Rehabilitation/Replacement	85	1.7	2	2.7	1.7	2.7	2.3	1.7	1.7	1
MP-WW-Pipe-03	Route 1 Piti Pipe Rehabilitation/Replacement	92	1.7	2	3	1.7	3	3	1.7	2	1.3
MP-WW-Pipe-04	Southern Link Pump Station Pipe Rehabilitation/Replacement	91	1.7	2	3	1.7	3	2.7	1.7	2	1.3
MP-WW-Pipe-05	Agana Heights Pipe Replacement	86	1.7	2	2.7	1.7	3	2.3	1.7	1.7	1
MP-WW-Pipe-06	Northern District Route 1 Capacity Replacement - Phase 1	76	1.3	1	2.3	3	2	2.3	2	2	2
MP-WW-Pipe-07	Northern District Route 1 Capacity Replacement - Phase 2	71	1	1	2	3	2	2	2	2	2
MP-WW-Pipe-08	Northern District Route 1 Capacity Replacement - Phase 3	72	1	1	2.3	3	2	2	2	2	2
MP-WW-Pipe-09	North Dededo Capacity Replacement - Phase 1	71	1	1	2	3	2	2	2	2	2
MP-WW-Pipe-10	North Dededo Capacity Replacement - Phase 2	71	1	1	2	3	2	2	2	2	2.3
MP-WW-Pipe-11	Route 16 Capacity Replacement	76	1.3	1	2.3	3	2	2.3	2	2	2
MP-WW-Pipe-12	Barrigada Capacity Replacement	76	1.3	1	2.3	3	2	2.3	2	2	2
MP-WW-Pipe-13	Mangilao Capacity Replacement	69	1	1	2	2.7	2	2	2	2	2
MP-WW-Pipe-14	Dededo Capacity Replacement	72	1	1	2.3	3	2	2	2	2	2
MP-WW-Pipe-16	Yigo Capacity Replacement	72	1	1	2.3	3	2	2	2	2	2
MP-WW-Pipe-17	Mamajanao Capacity Replacement	85	1.7	1.3	2.3	3	2.7	2.3	2	2	2
MP-WW-Pipe-18	Agat-Santa Rita Capacity Replacement - Phase 1	72	1	1	2.3	3	2	2	2	2	2
MP-WW-Pipe-19	Agat-Santa Rita Capacity Replacement - Phase 2	72	1	1	2.3	3	2	2	2	2	2
MP-WW-Pipe-20	Agat-Santa Rita Capacity Replacement - Phase 3	73	1	1	2.3	3	2	2.3	2	2	2
MP-WW-Pipe-21	Baza Gardens Capacity Replacement - Phase 1	75	1.3	1	2.3	3	2	2	2	2	2
MP-WW-Pipe-22	Baza Gardens Capacity Replacement - Phase 2	72	1	1	2.3	3	2	2	2	2	2
MP-WW-Pipe-23	Baza Gardens Capacity Replacement - Phase 3	71	1	1	2	3	2	2	2	2	2
MP-WW-Pipe-24	Umatac-Merizo Capacity Replacement	83	2.7	2	1	1.7	1	2.7	2.3	2	2
MP-WW-Pipe-25	Piping Near Bayside Lift Station	85	1.3	1.7	2.7	2.7	2.3	2.7	1.7	2	2
MP-WW-Pipe-26	Finile Drive Rehabilitation - Agat	82	1.7	1	2.3	3	2.3	2.7	2	2	2
MP-WW-Pipe-27	Septic/Cesspool System Reduction Program	87	2.7	2	1	1.7	1	3	3	2	2
MP-WW-MH-01	Manhole Rehabilitation Program	76	1.7	2	2	1.3	2	2	1.7	1.7	1
Force Main Projects											
MP-WW-FM-01	Force Main Rehabilitation/Replacement Program	87	1.7	1.7	3	1.7	3	2.7	1.7	1.7	1.3
MP-WW-FM-02	Replace Yigo Lift Station Force Main	73	1.3	1.3	2.7	2	2.3	2.3	1	1.7	1.3
MP-WW-FM-03	Route 1 Asan Force Main Rehabilitation/Replacement	89	2	2	2.7	1.3	3	3	1	2.3	1
MP-WW-FM-04	Hagåtña WWTP Force Main Rehabilitation/Replacement	100	2.7	2	3	1.3	3	3	1.7	2.3	1.3
Lift Station Projects											
MP-WW-Pump-01	Lift Station Rehabilitation/Replacement Program	93	2.3	2.3	2.7	1.7	2.7	2.7	1	1.7	1.7

Table 10-2. Wastewater System Improvements Projects Prioritization (Non-Economic)

Table 10-2. Wastewater System Improvements Projects Prioritization (Non-Economic)											
Report Project Number	Report Project Name	Score out of 100	Health and Safety	Regulatory or Mandated	Reliability and Redundancy	Capacity	Operation, Maintenance, and Rehabilitation	Environmental Impact and Resource Use	Revenue and Expenditures	Customer Service and Stakeholder Confidence	Economic Development
MP-WW-Pump-02	Tumon Basin - Fujita Lift Station Analysis	93	2	1.7	3	2	3	2.7	1.3	2	3
MP-WW-Pump-03	Replacement of Former Navy Pump Station (Donut Hole)	80	2	1	3	2	2.5	2	1	2	2
WWTP Projects											
MP-WW-WWTP-01	Hagåtña WWTP Primary Treatment Repair/Rehabilitation Program	77	2	1	3	1	3	1.7	1.7	1.3	1.7
MP-WW-WWTP-02	Hagåtña WWTP Secondary Treatment Upgrade	76	1.7	2	2	1.7	2	2	1	1.7	1.7
MP-WW-WWTP-03	Inarajan WWTP Repair/Rehabilitation Program	75	2	1	3	1	3	1.7	1	1.3	1.7
MP-WW-WWTP-04	Pago Socio WWTP Pump Station Conversion	76	1.7	1	3	1.7	3	2	1	1.3	1.7
MP-WW-WWTP-05	Umatac-Merizo WWTP Repair/Rehabilitation Program	71	2	1	3	1	3	1	1	1.3	1
MP-WW-WWTP-06	Agat-Santa Rita WWTP Repair/Rehabilitation Program	74	2	1	3	1	3	1.7	1	1.3	1
MP-WW-WWTP-07	Baza Gardens Cross Island Pipeline - Preliminary Treatment Equipment Repair and Rehabilitation Program	70	2	1	3	1	3	1	1	1	1
MP-WW-WWTP-08	Northern District WWTP Completion	66	1	2	2	2	1.3	1	1	2.3	2
MP-WW-WWTP-09	Ocean Outfall Inspection Program	56	1	1	2	1.3	1.3	2.3	1	1.3	1
Other Wastewater Projects											
MP-WW-Misc-01A	Update Wastewater Collection System Model (Major Update)	74	1.3	1.3	2	2	2	2	2.3	2	2.7
MP-WW-Misc-01B	Update Wastewater Collection System Model (Continued)	72	1.5	1.3	2	2	1.5	2	2	2	2.5
MP-WW-Misc-02	I/I and SSES Assessments	86	2	2	2	2.3	2.3	2	2	2.3	1.3
MP-WW-Misc-03	Miscellaneous Wastewater Improvements	82	2	1.7	2	2	3	2	1.3	2	1.3
MP-WW-Misc-04	Fats, Oils, and Grease Study	85	2	1.7	2.3	2	3	2	1.7	2	1.3

Section 11

Recommended Project Sheets

This section contains a project sheet for the proposed improvement projects developed for GWA's wastewater system (listed in Table 10-1).

The proposed projects are subject to change and are based on information available at the time of this report. Projects will generally include an engineering study, field verification, detailed design and construction services to refine exact project scope. Engineering staff will lead the design for new or rehabilitated facilities with assistance from operations staff. The project schedules shown are based on the recommended CIP program included in Volume 1 Section 11.

11.1 Gravity Pipeline Projects

The following legend applies to the figures shown in the pipeline project sheets in this section.

Gravity Pipe Improvements

- Proposed Pipe >= 12 in
- Proposed Pipe < 12 in
- GWA Planned Pipe >= 12 in
- GWA Planned Pipe < 12 in
- Existing Pipe >= 12 in
- Existing Pipe < 12 in
- Condition Project

Force Main Improvements

- Proposed
- GWA Planned
- Existing

Lift Station Improvements

- Proposed
- GWA Planned
- Existing
- Ideal or Not Modeled

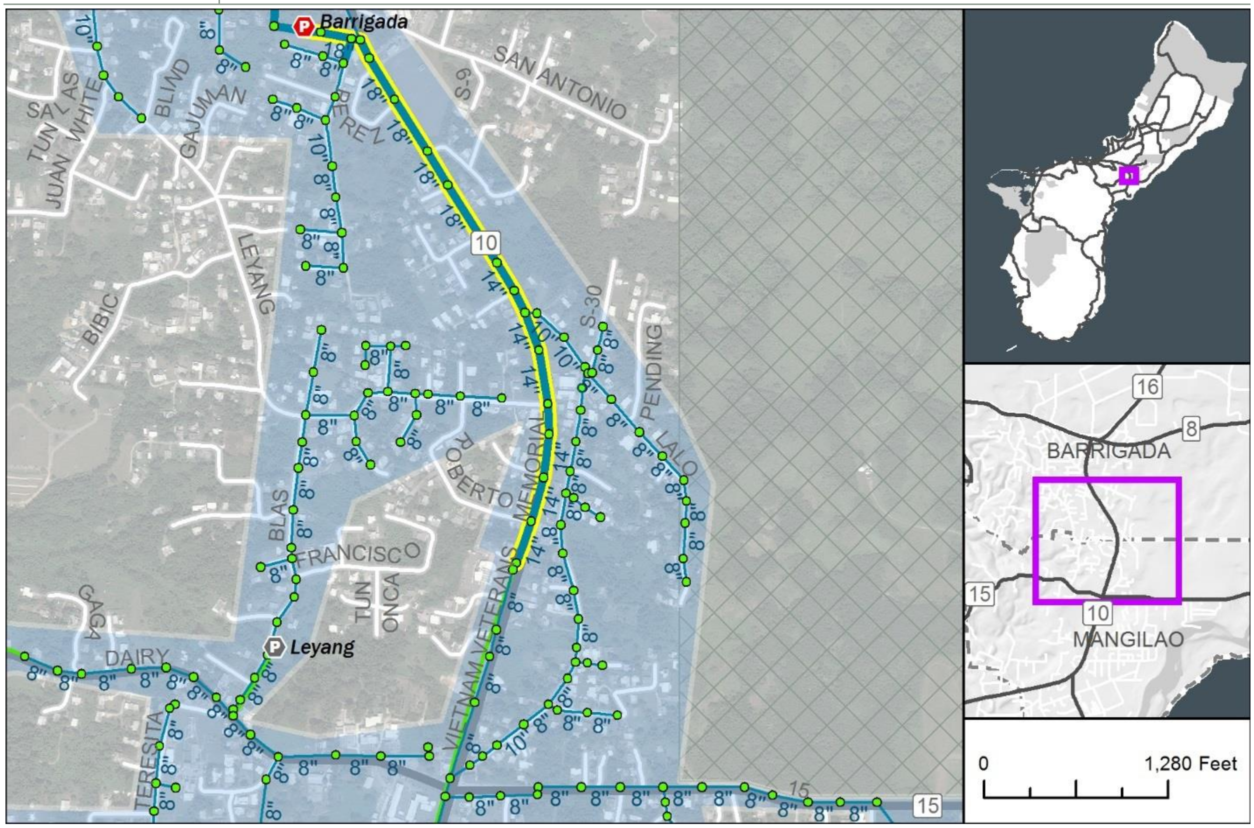
Other Facilities

- WWTP
- Manhole
- DoD Property

Gravity Pipe Rehabilitation/Replacement Program			
Project Number	MP-WW-Pipe-01	**Basin**	All
Description	Implementation of an annual program to inspect, rehabilitate, and replace gravity piping based on the condition assessment risk analysis. New piping should be sized to handle future planned peak wet weather flows.		
Justification	The risk analysis of the piping in Section 4 showed that GWA must continue with a pipe renewal program to replace piping that will reach the end of its service life. Continuation of previous project WW 09-06.		
Proposed Schedule	Annual, Begin: 2020		
Cost Estimate	$8.5M (Annually) This is the total funding allocated per year for gravity pipe rehabilitation/replacement. This project amount is reduced by the amount allocated to specific projects each year and the residual is available for new projects.		
Reference Documents	WRMPU Volume 3, Section 4		

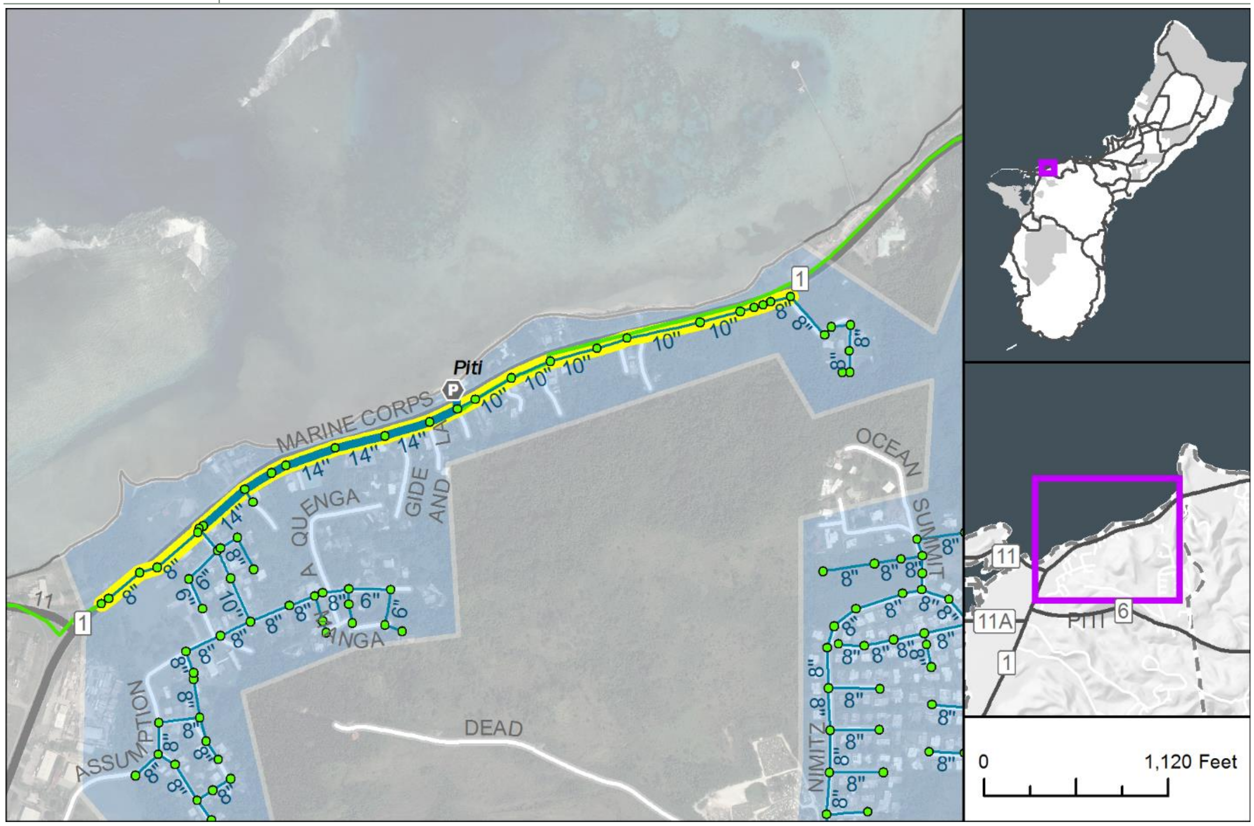
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Barrigada Pump Station Pipe Rehabilitation/Replacement			
Project Number	MP-WW-Pipe-02	**Basin**	Hagåtña
Description	Replace approximately 4,420 feet of gravity piping along Route 10 from the Mangilao force main to the Barrigada pump station (shown below in yellow). Section 8 discusses I/I issues along this pipeline that should be investigated before replacing this pipeline.		
Justification	The original pipeline liner is blistering and separating from the pipe. The pipeline will probably need to be replaced.		
Proposed Schedule	Begin Design: 2022		
Cost Estimate	$5.42M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Route 1 Piti Pipe Rehabilitation/Replacement			
Project Number	MP-WW-Pipe-03	**Basin**	Hagåtña
Description	Rehabilitate or replace approximately 4,675 feet of gravity piping in Piti along Marine Corp Drive (shown below in yellow). The project will evaluate the use of CIPP options and open cut and replace construction to complete the necessary repairs.		
Justification	The original liner on this pipeline is in poor condition. The crowns of the pipes are deteriorated and GWA is concerned that jetting the pipe for routine maintenance could cause a failure.		
Proposed Schedule	Begin Design: 2018		
Cost Estimate	$4.48M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		

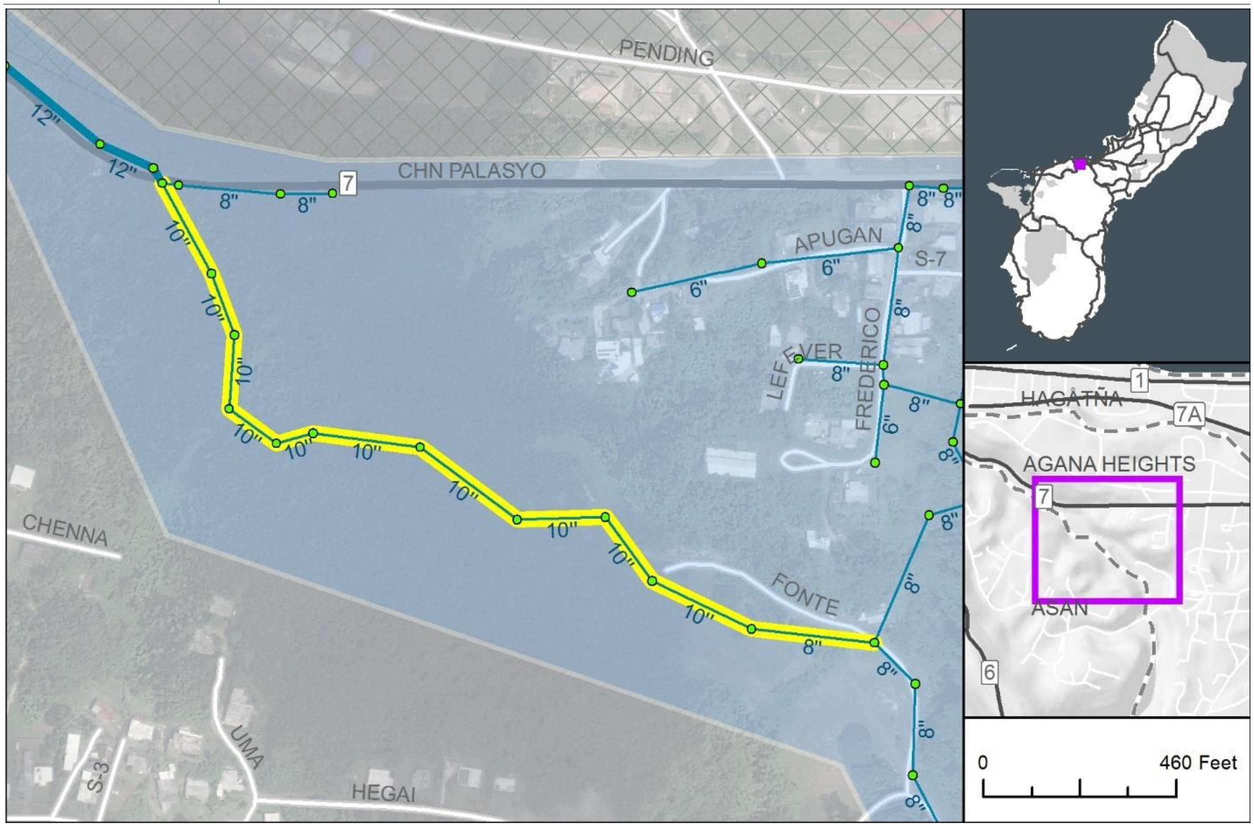


This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Southern Link Pump Station Pipe Rehabilitation/Replacement			
Project Number	MP-WW-Pipe-04	**Basin**	Northern District
Description	Replace approximately 225 feet of 48-inch gravity piping just upstream of the Southern Link pump station (shown in yellow below).		
Justification	Piping in this section partially collapsed and was fixed as an emergency repair. The pipe needs to be repaired for long-term operation and to eliminate the possibility of another collapse in the future.		
Proposed Schedule	2018		
Cost Estimate	$0.71M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		

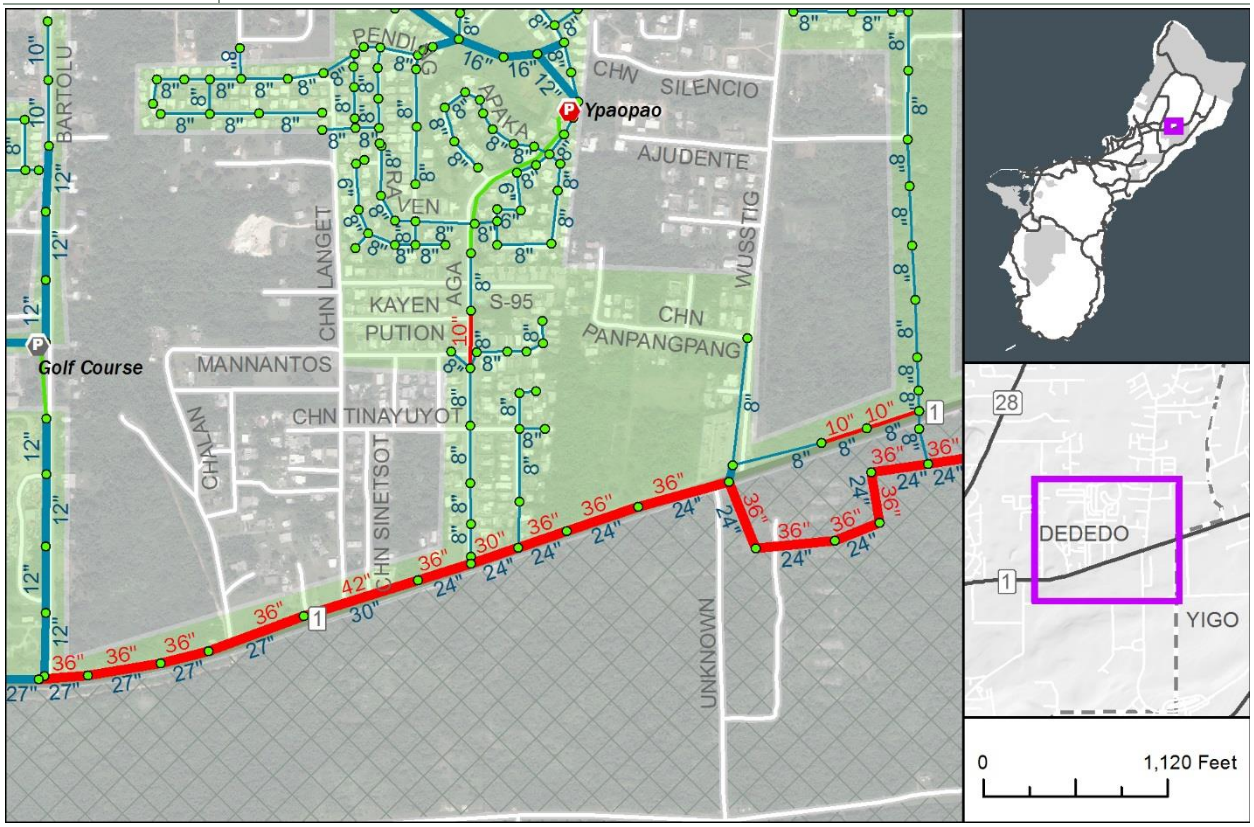
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Agana Heights Pipe Replacement			
Project Number	MP-WW-Pipe-05	Basin	Hagåtña
Description	Replace approximately 2,320 feet of 8 to 10-inch gravity piping in Agana Heights (shown in yellow below).		
Justification	This pipe has failed in the past and there is no vehicular access to the pipe alignment. An alternate pipeline alignment should be evaluated.		
Proposed Schedule	Begin Design: 2022		
Cost Estimate	$3.23M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



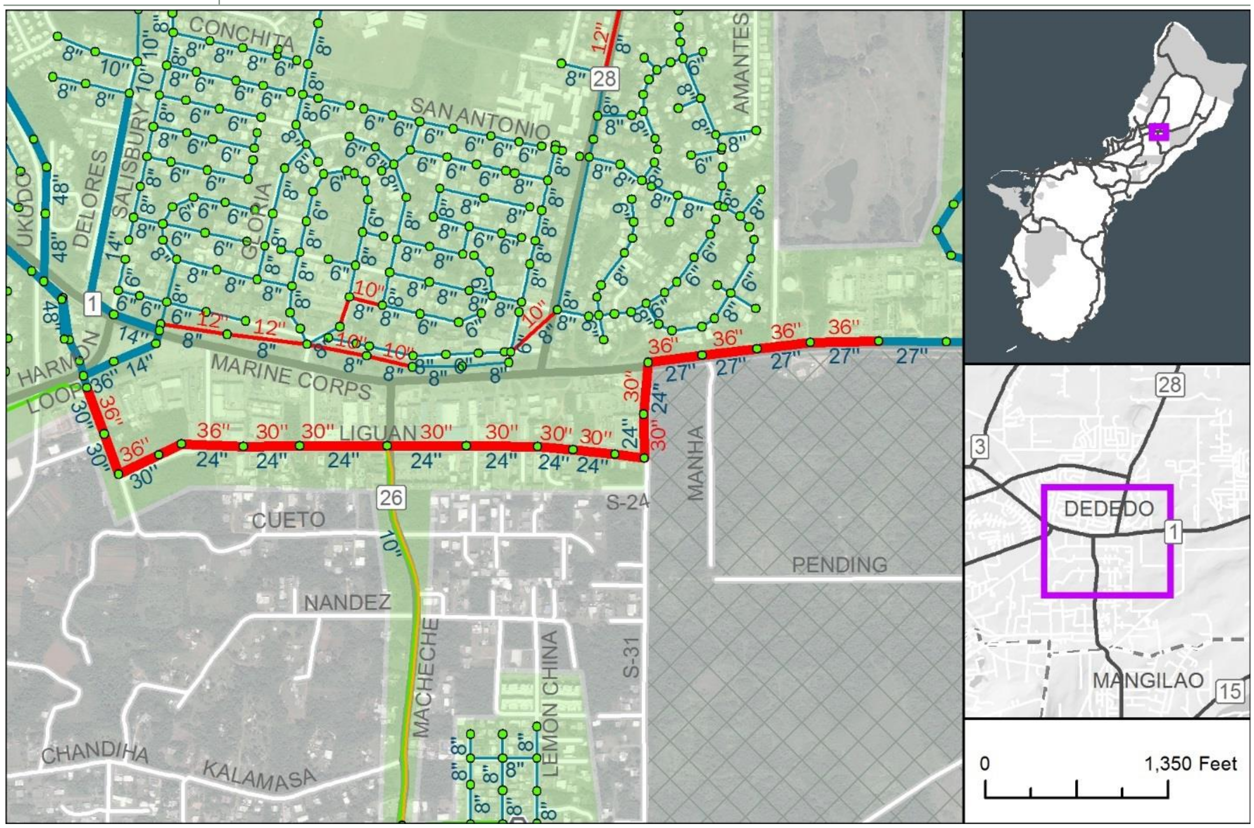
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Northern District Route 1 Capacity Replacement – Phase 1			
Project Number	MP-WW-Pipe-06	**Basin**	Northern District
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. Eighty-two percent of the pipeline length does not have sufficient capacity for existing peak wet weather flows, and the remainder does not have capacity for future peak wet weather flows.		
Proposed Schedule	Begin Design: 2024		
Cost Estimate	$15.43M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



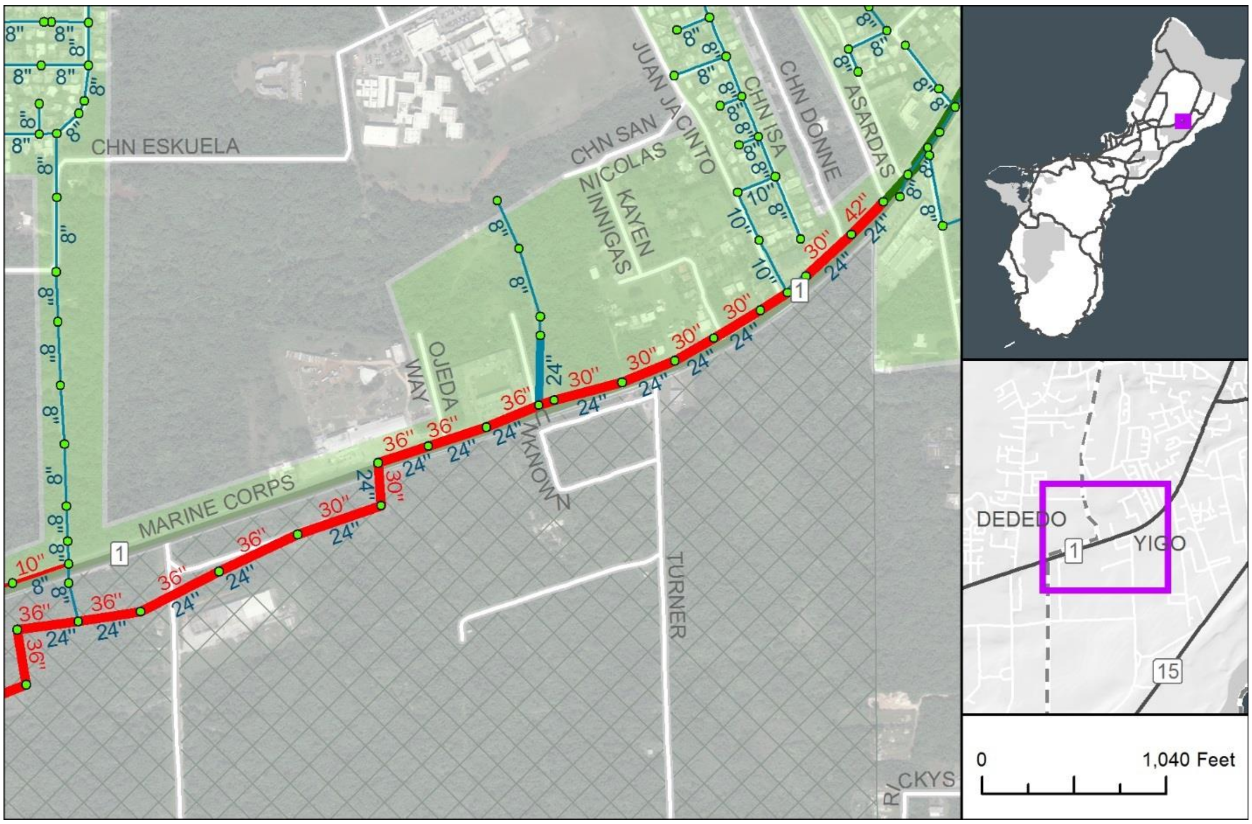
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Northern District Route 1 Capacity Replacement – Phase 2			
Project Number	MP-WW-Pipe-07	**Basin**	Northern District
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. Fourteen percent of the pipeline length does not have sufficient capacity for existing peak wet weather flows, and the remainder does not have capacity for future peak wet weather flows.		
Proposed Schedule	Begin Design: 2034		
Cost Estimate	$14.58M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



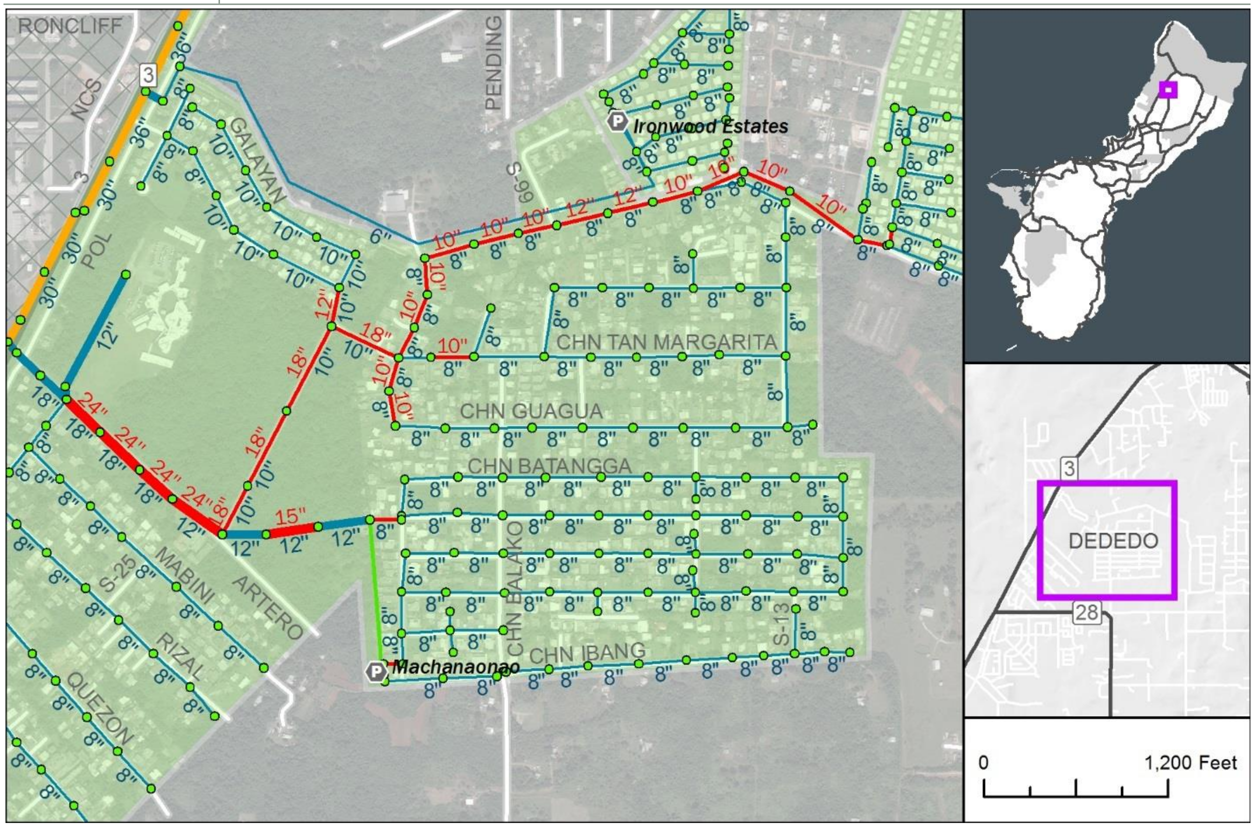
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Northern District Route 1 Capacity Replacement – Phase 3			
Project Number	MP-WW-Pipe-08	Basin	Northern District
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. Sixty-four percent of the pipeline length does not have sufficient capacity for existing peak wet weather flows, and the remainder does not have capacity for future peak wet weather flows.		
Proposed Schedule	Begin Design: 2036		
Cost Estimate	$11.13M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



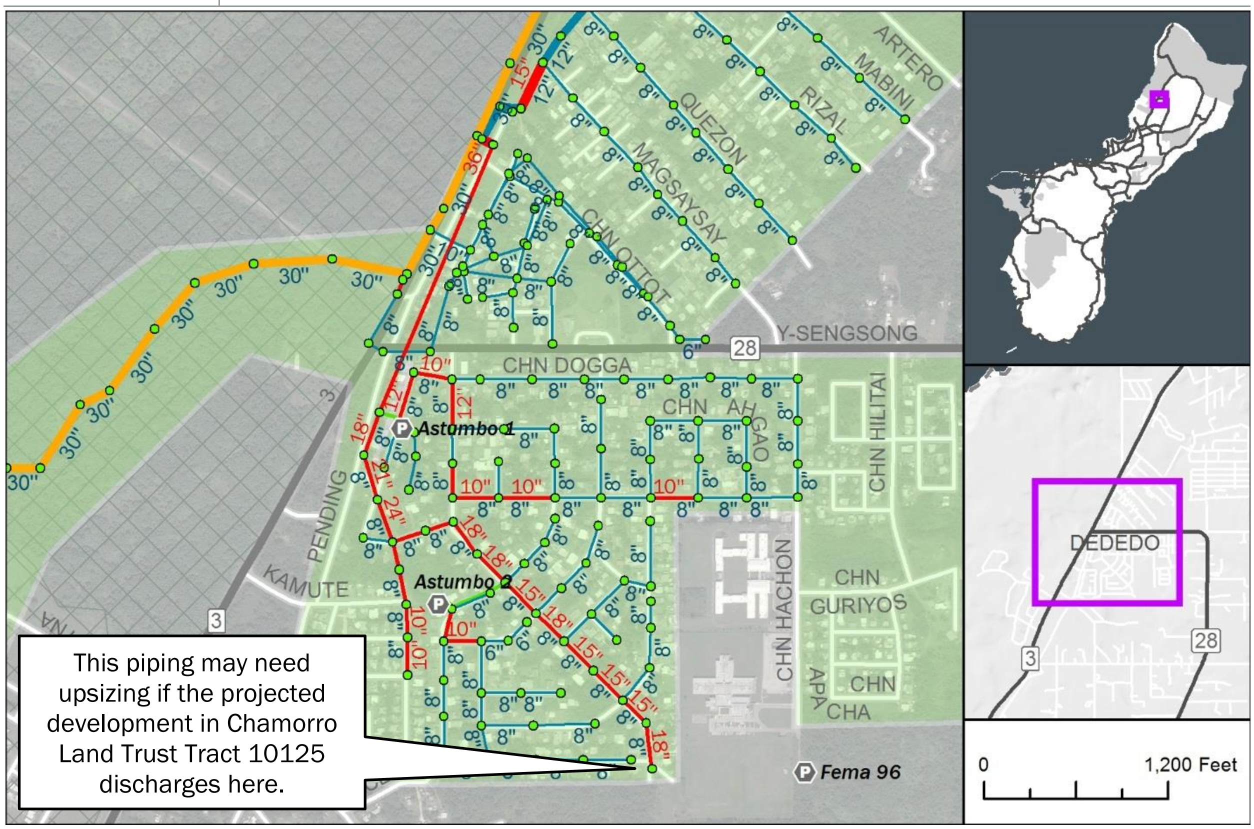
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

North Dededo Capacity Replacement – Phase 1			
Project Number	MP-WW-Pipe-09	**Basin**	Northern District
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. Thirty-nine percent of the pipeline length does not have sufficient capacity for existing peak wet weather flows, and the remainder does not have capacity for future peak wet weather flows.		
Proposed Schedule	Begin Design: 2028		
Cost Estimate	$9.80M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



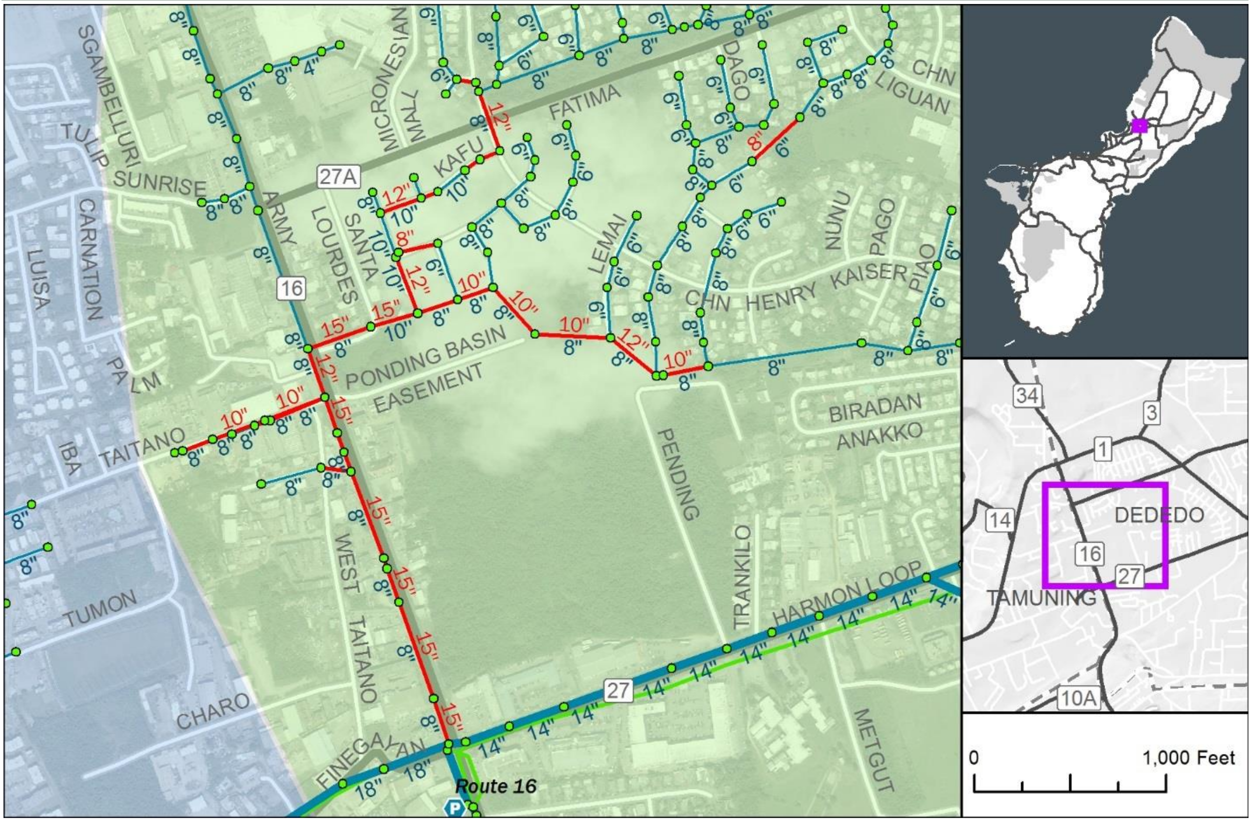
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

North Dededo Capacity Replacement – Phase 2			
Project Number	MP-WW-Pipe-10	Basin	Northern District
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping. Some of the piping shown in the figure below may or may not need upsizing depending on where the projected development in Chamorro Land Trust Tract 10125 discharges (see Volume 1, Section 4.5 for a description of the tract).		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. Forty-eight percent of the pipeline length does not have sufficient capacity for existing peak wet weather flows, and the remainder does not have capacity for future peak wet weather flows.		
Proposed Schedule	Begin Design: 2030		
Cost Estimate	$12.44M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Route 16 Capacity Replacement			
Project Number	MP-WW-Pipe-11	Basin	Northern District
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. 78 percent of the piping by length does not have sufficient capacity for existing peak, wet weather flows and the remainder does not have capacity for future peak, wet weather flows.		
Proposed Schedule	Begin Design: 2024		
Cost Estimate	$7.54M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		

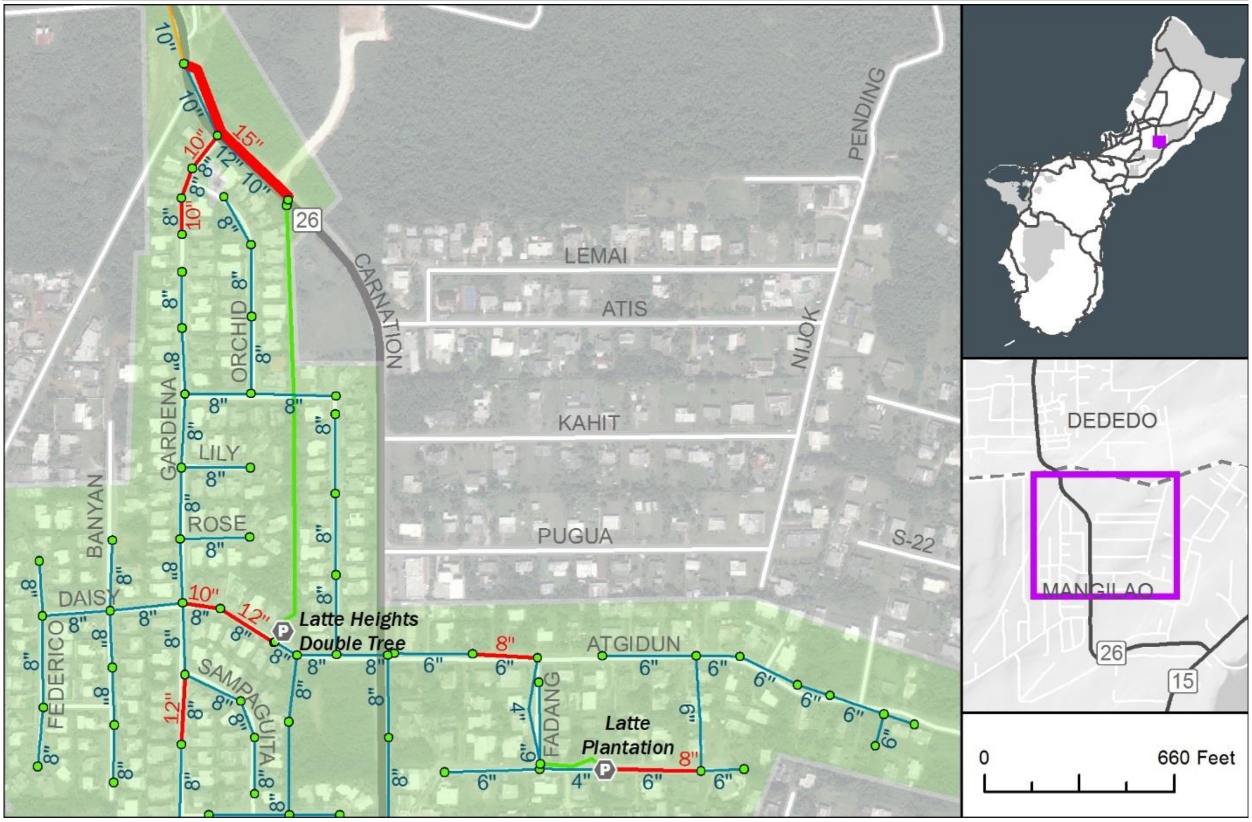


This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Barrigada Capacity Replacement			
Project Number	MP-WW-Pipe-12	Basin	Northern District
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. The entire pipeline length does not have sufficient capacity for existing peak wet weather flows.		
Proposed Schedule	Begin Design: 2024		
Cost Estimate	$0.61M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		

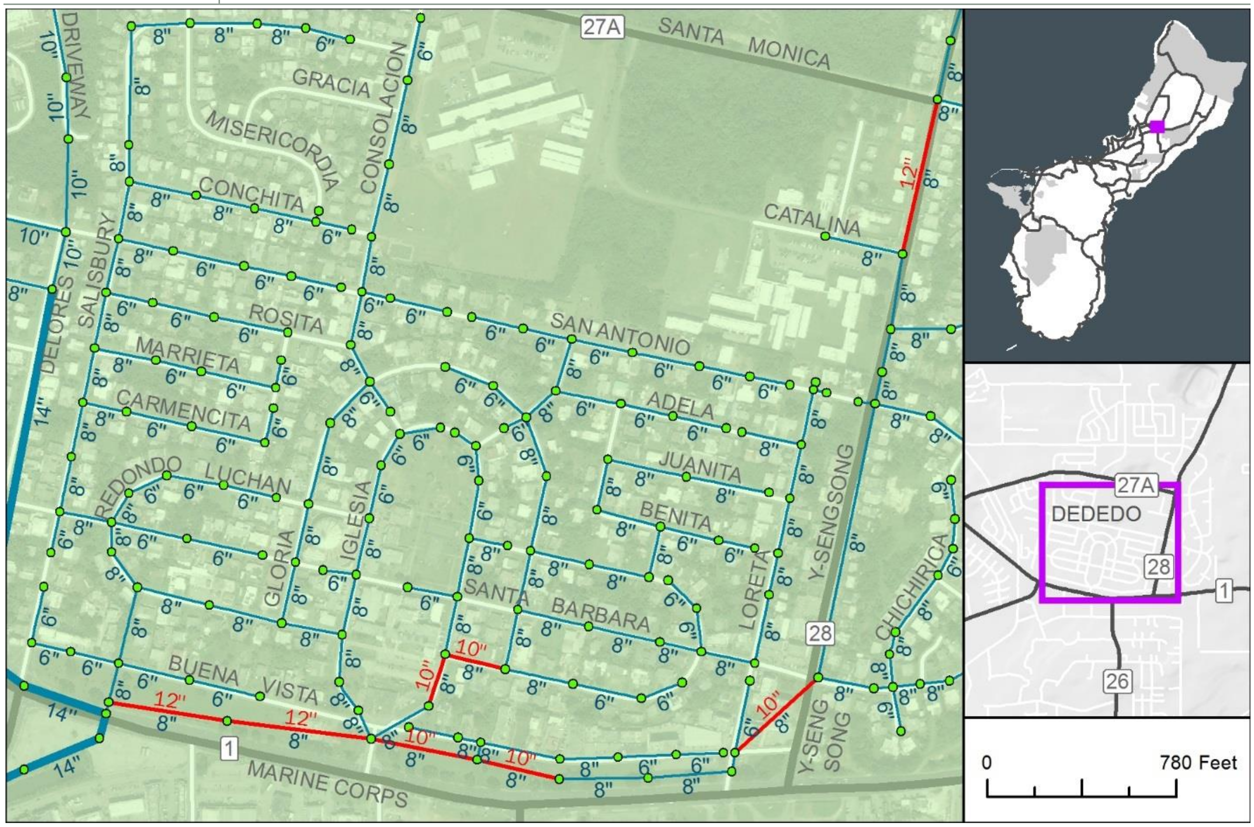
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Mangilao Capacity Replacement			
Project Number	MP-WW-Pipe-13	**Basin**	Northern District
Description	Replace the existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. Two percent of the pipeline length does not have sufficient capacity for existing peak wet weather flows, and the remainder does not have capacity for future peak wet weather flows.		
Proposed Schedule	Begin Design: 2035		
Cost Estimate	$2.14M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



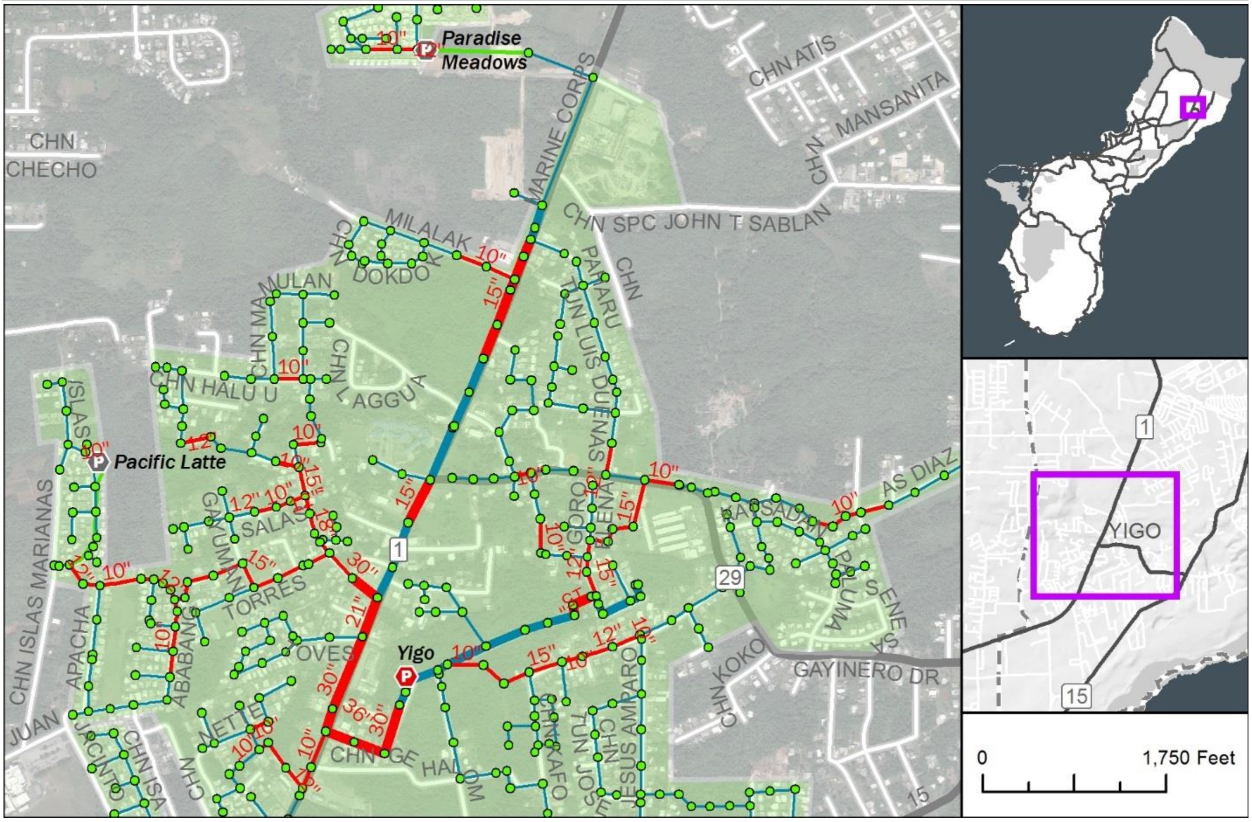
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Dededo Capacity Replacement			
Project Number	MP-WW-Pipe-14	**Basin**	Northern District
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. The entire pipeline length does not have sufficient capacity for existing peak wet weather flows.		
Proposed Schedule	Begin Design: 2028		
Cost Estimate	$3.31M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



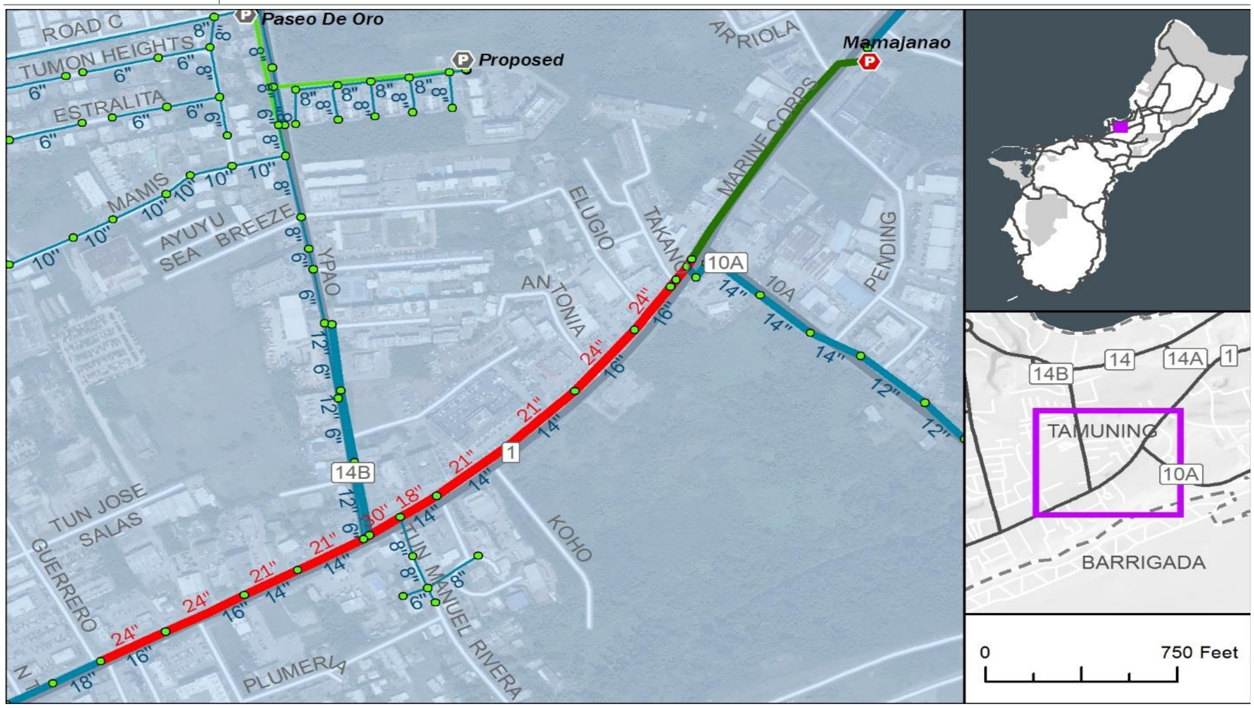
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Yigo Capacity Replacement			
Project Number	MP-WW-Pipe-16	**Basin**	Northern District
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. Fifty-three percent of the pipeline length does not have sufficient capacity for existing peak wet weather flows, and the remainder does not have capacity for future peak wet weather flows.		
Proposed Schedule	Begin Design: 2031		
Cost Estimate	$22.09M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Mamajanao Capacity Replacement			
Project Number	MP-WW-Pipe-17	**Basin**	Hagåtña
Description	Enlargement of piping downstream of the Mamajanao lift station or alternate lift station force main to Route 16 PS.		
Justification	The Mamajanao lift station can only currently run one pump. If a second pump is turned on, the piping downstream surcharges, and because the line is shallow, a manhole lid pops up and an SSO occurs. With only one pump running, the lift station wet well can overflow during peak flows. This project may also be used to evaluate redirecting flow from Mamajanao to the Northern District WWTP and for required pump station and pipeline improvements as shown in Figure 4-7. The evaluation would review the condition of the existing pipeline to the Route 16 pump station and the capacity of the Route 16 PS and force main to the Northern District WWTP.		
Proposed Schedule	Begin Design: 2020		
Cost Estimate	$5.57M		
Reference Documents	WRMPU Volume 3, Section 4		

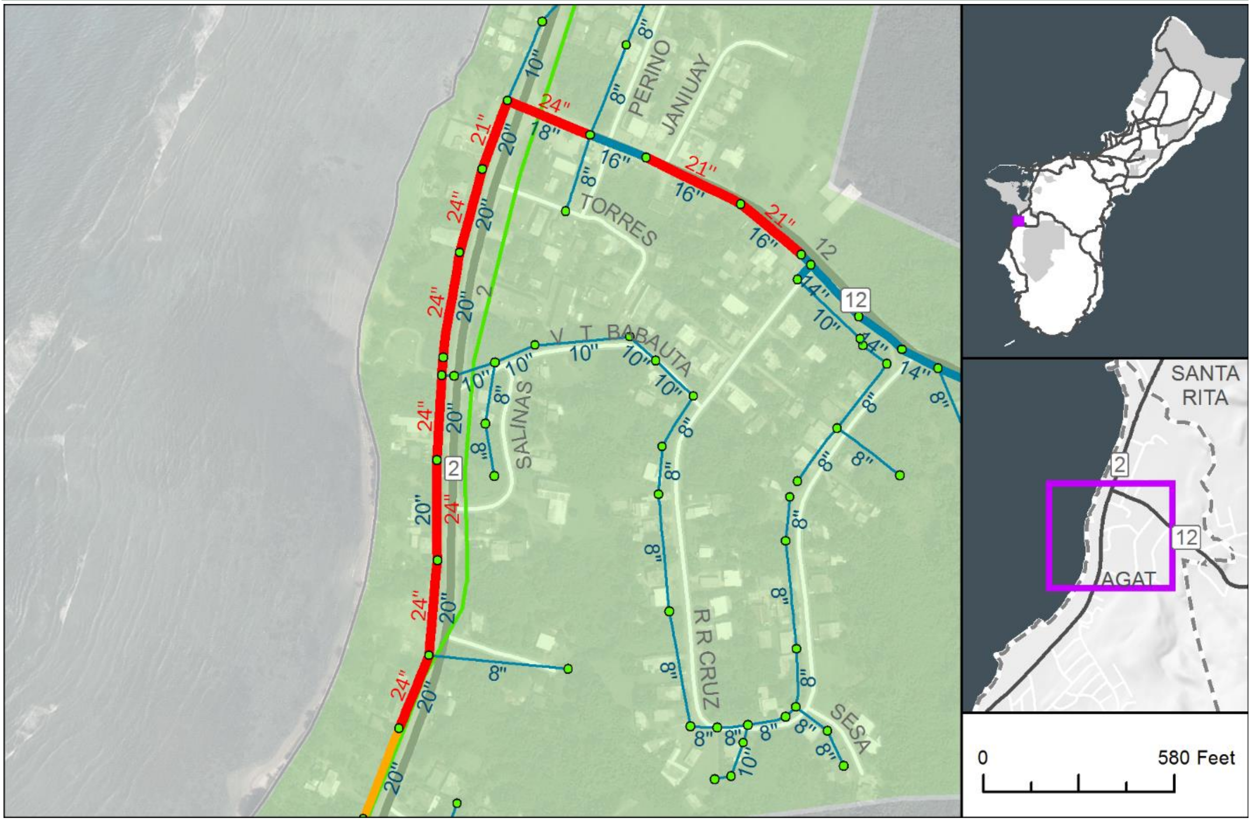


This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Agat-Santa Rita Capacity Replacement – Phase 1			
Project Number	MP-WW-Pipe-18	**Basin**	Agat-Santa Rita
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. Seventy-eight percent of the pipeline length does not have sufficient capacity for existing peak wet weather flows, and the remainder does not have capacity for future peak wet weather flows.		
Proposed Schedule	Begin Design: 2026		
Cost Estimate	$3.01M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		

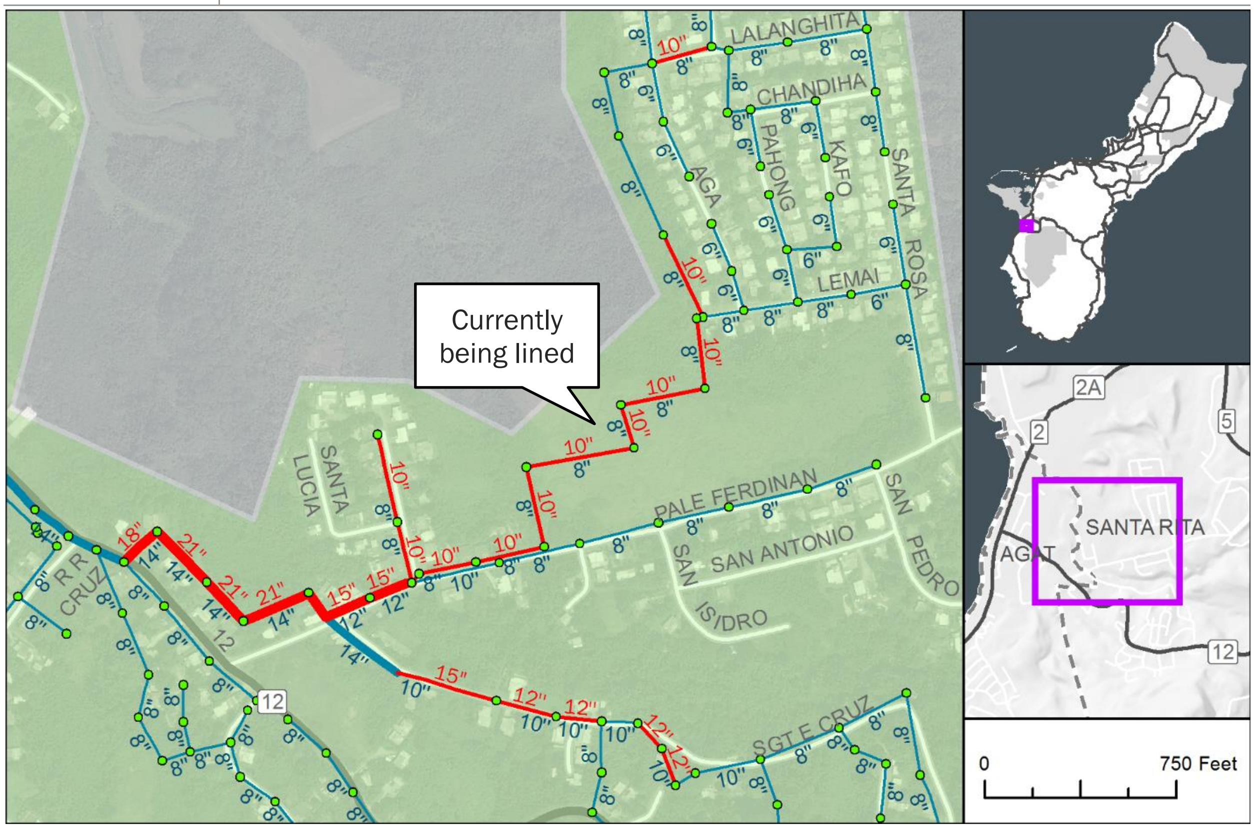
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Agat-Santa Rita Capacity Replacement – Phase 2			
Project Number	MP-WW-Pipe-19	**Basin**	Agat-Santa Rita
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. Ninety percent of the pipeline length does not have sufficient capacity for existing peak wet weather flows, and the remainder does not have capacity for future peak wet weather flows.		
Proposed Schedule	Begin Design: 2027		
Cost Estimate	$4.09M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



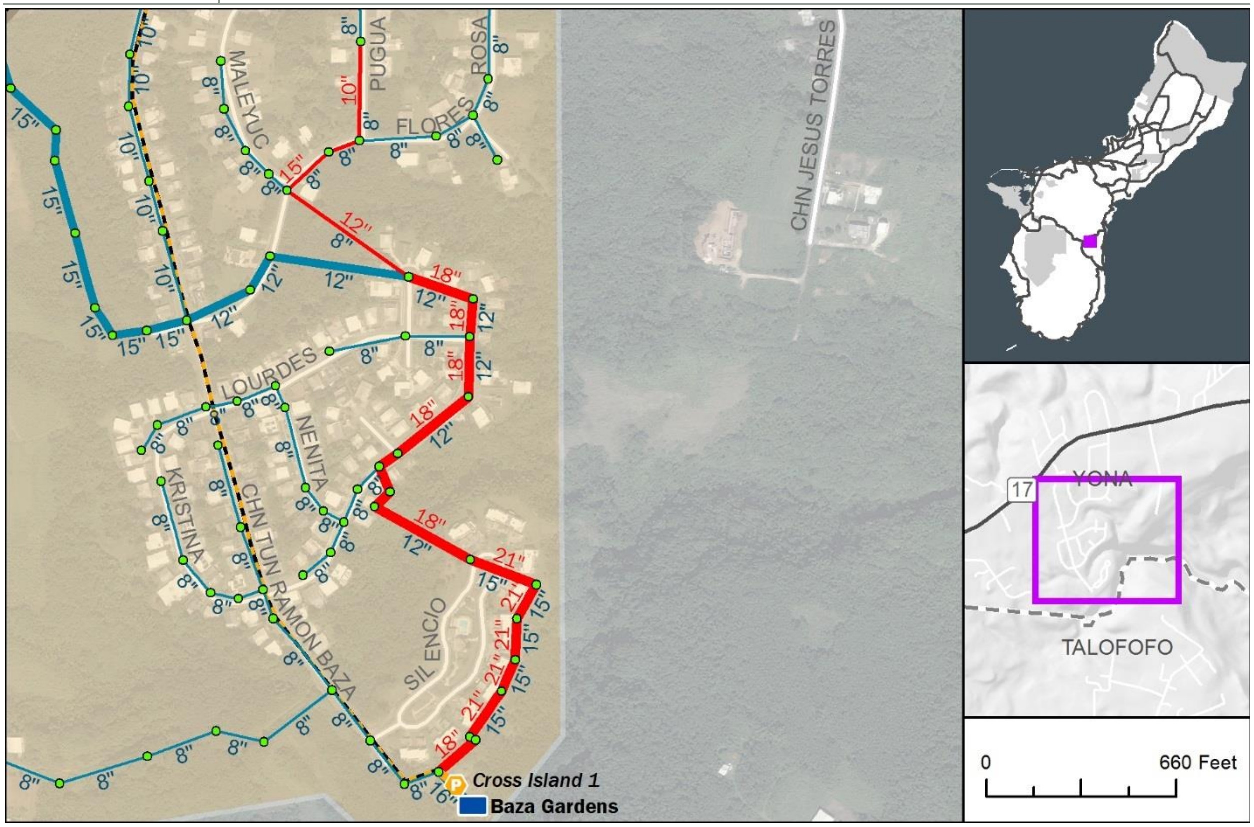
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Agat-Santa Rita Capacity Replacement – Phase 3			
Project Number	MP-WW-Pipe-20	**Basin**	Agat-Santa Rita
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping. A section of piping between Lemai Street and Pale Ferdinan is currently being lined and runs through a wetland. This section of piping should be realigned in the future.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. Eighty-six percent of the pipeline length does not have sufficient capacity for existing peak wet weather flows, and the remainder does not have capacity for future peak wet weather flows.		
Proposed Schedule	Begin Design: 2027		
Cost Estimate	$5.94M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



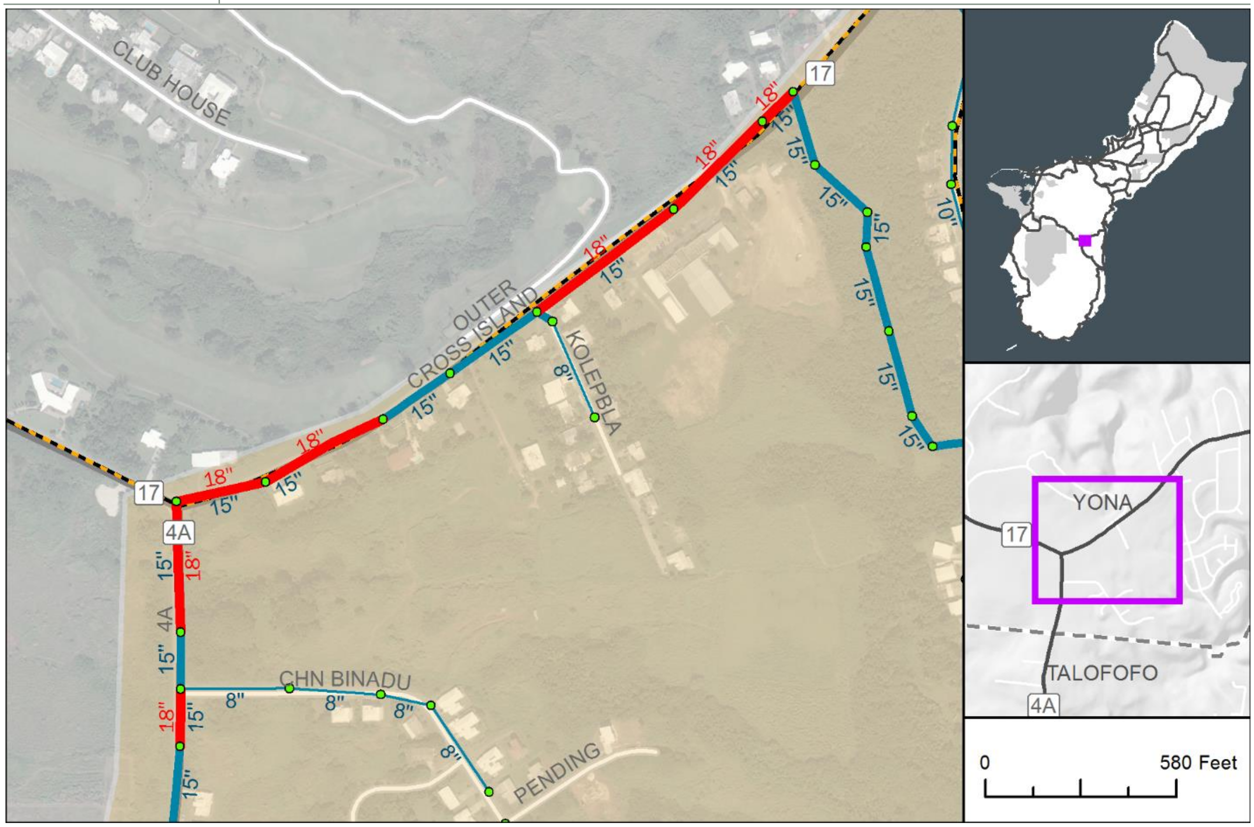
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Baza Gardens Capacity Replacement – Phase 1			
Project Number	MP-WW-Pipe-21	Basin	Baza Gardens
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. Ninety-one percent of the pipeline length does not have sufficient capacity for existing peak wet weather flows, and the remainder does not have capacity for future peak wet weather flows.		
Proposed Schedule	Begin Design: 2024		
Cost Estimate	$4.21M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



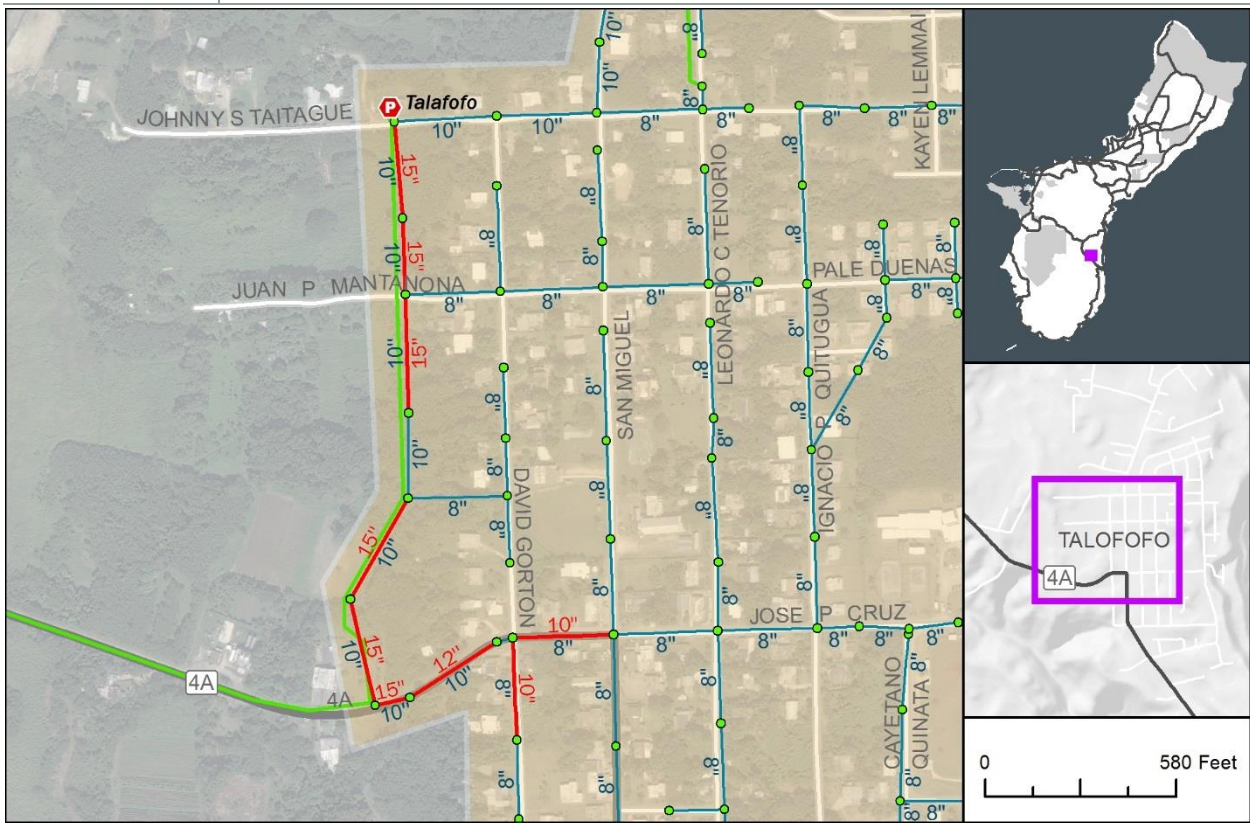
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Baza Gardens Capacity Replacement – Phase 2			
Project Number	MP-WW-Pipe-22	Basin	Baza Gardens
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. The entire pipeline length does not have sufficient capacity for existing peak wet weather flows.		
Proposed Schedule	Begin Design: 2033		
Cost Estimate	$2.61M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



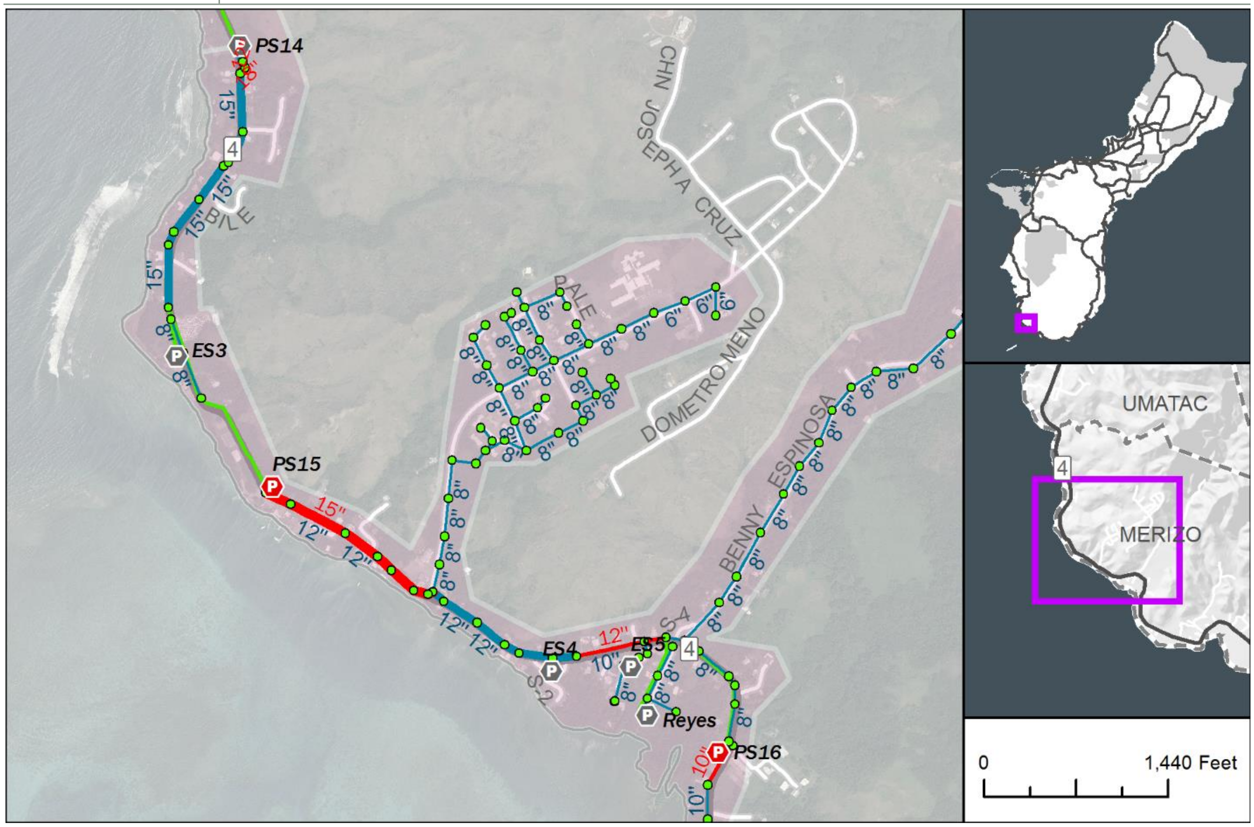
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Baza Gardens Capacity Replacement – Phase 3			
Project Number	MP-WW-Pipe-23	Basin	Baza Gardens
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. Forty-eight percent of the pipeline length does not have sufficient capacity for existing peak wet weather flows, and the remainder does not have capacity for future peak wet weather flows.		
Proposed Schedule	Begin Design: Project is not currently scheduled in 20-year Plan		
Cost Estimate	$2.64M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



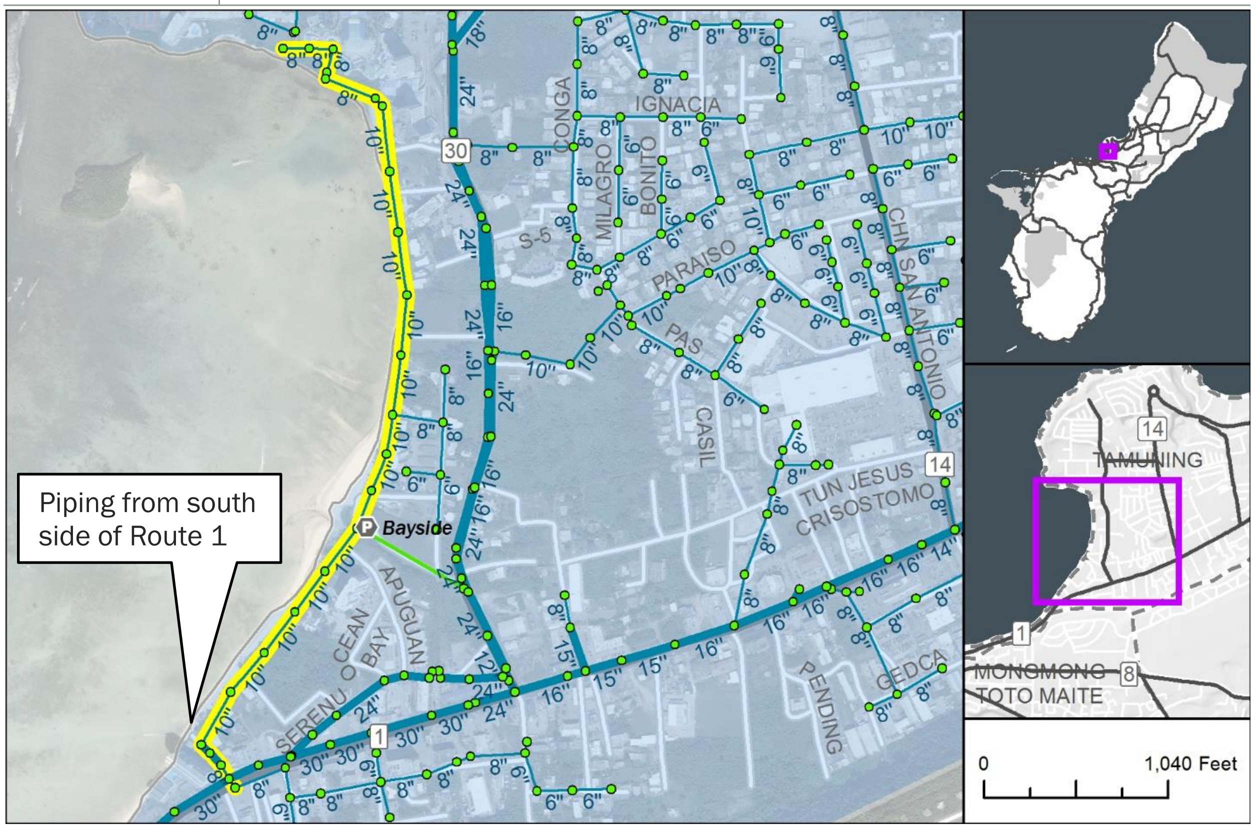
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Umatac-Merizo Capacity Replacement			
Project Number	MP-WW-Pipe-24	**Basin**	Umatac-Merizo
Description	Replace existing gravity piping (shown in red below) with new larger diameter piping.		
Justification	The hydraulic model identified the piping shown in red below as having insufficient capacity. Ninety-eight percent of the pipeline length does not have sufficient capacity for existing peak wet weather flows, and the remainder does not have capacity for future peak wet weather flows.		
Proposed Schedule	Begin Design: 2024		
Cost Estimate	$2.73M		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



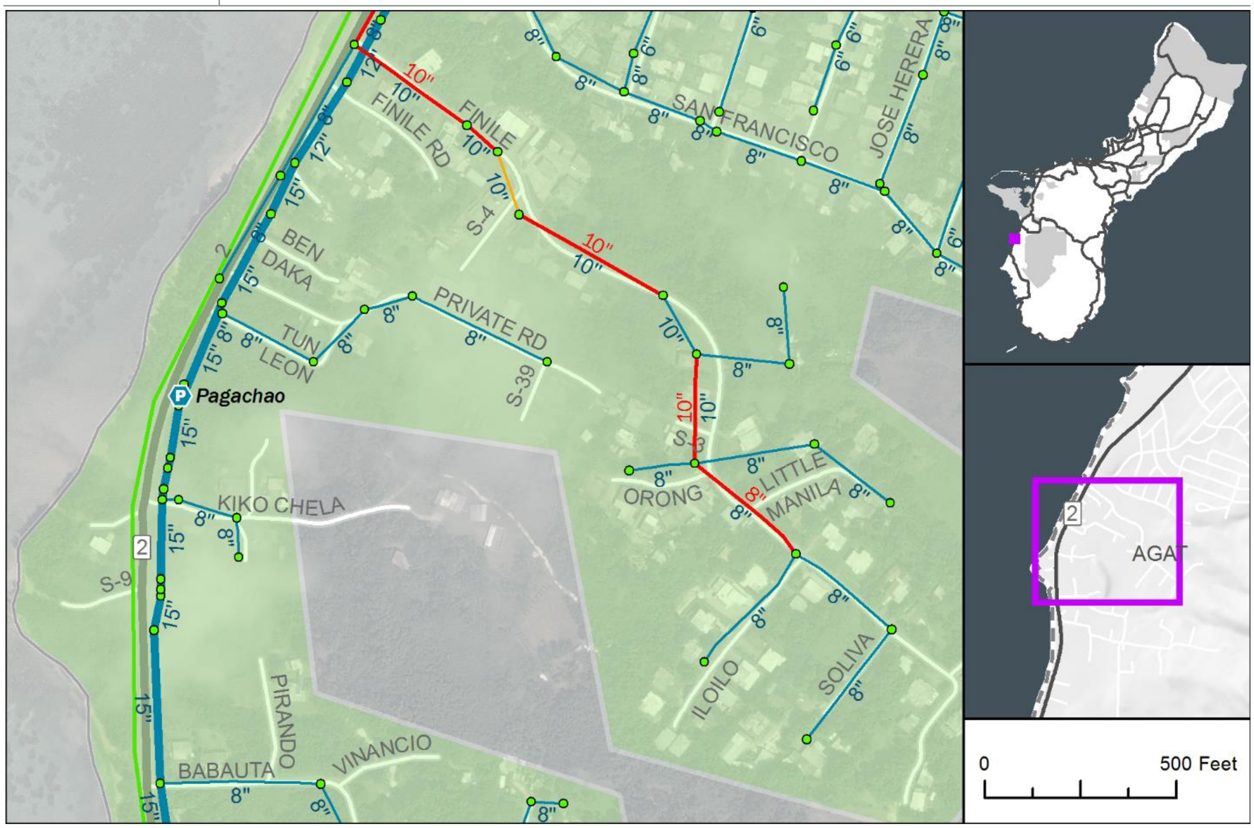
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Piping Near Bayside Lift Station			
Project Number	MP-WW-Pipe-25	**Basin**	Hagåtña
Description	Completion of a study on the piping draining into the Bayside lift station (shown below in yellow), in addition to the study planned on lift station replacement. The study should recommend replacing piping along the beach that flows to the Bayside lift station to another location away from the beach. Piping from the south side of Route 1 (see callout below) is too low and cannot drain into Route 1 and drains to Bayside. Study should determine if this piping can be connected to Route 1. Recommendations from the study should then be implemented.		
Justification	See *Pump Station Upgrades and Erosion Evaluation – Initial Findings* technical memorandum (BC, 2014) for information on the issues at the site.		
Proposed Schedule	2023		
Cost Estimate	$250,000		
Reference Documents	WRMPU Volume 3, Section 4, Table 4-13		



This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

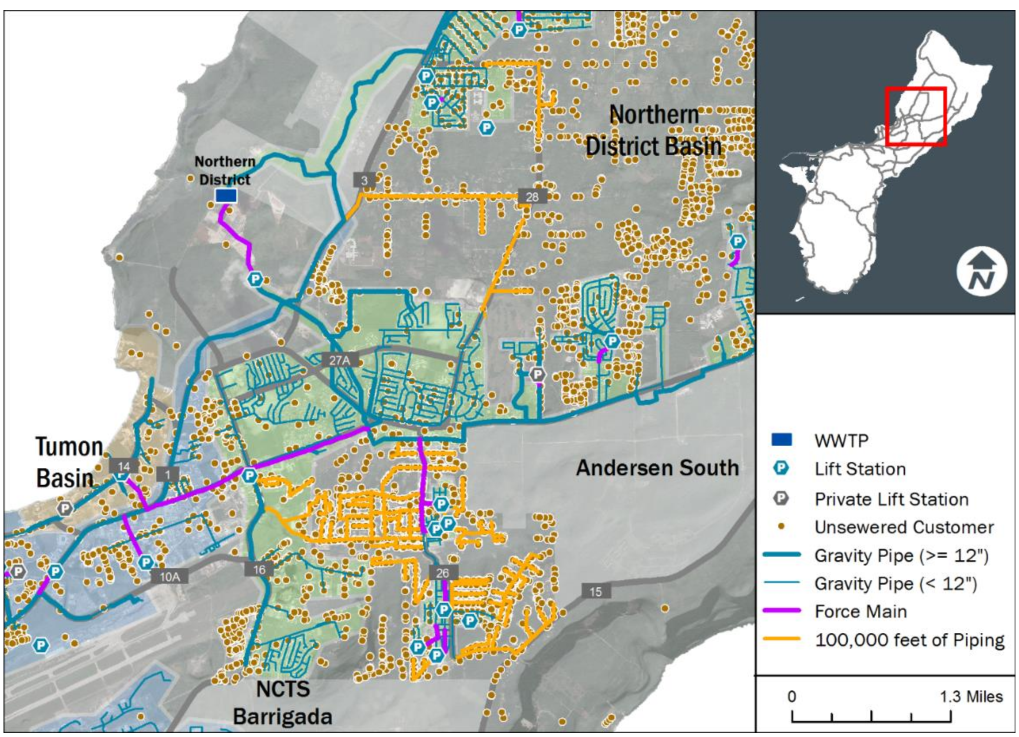
Finile Drive Rehabilitation - Agat			
Project Number	MP-WW-Pipe-26	**Basin**	Agat
Description	Rehabilitate or replace approximately 1,540 feet of gravity piping in Agat along Finile Drive (shown below in red and orange). The project will evaluate the use of CIPP options and open cut and replace construction to complete the necessary repairs.		
Justification	This piping has been identified by field investigation to be in very poor condition and requires rehabilitation or replacement.		
Proposed Schedule	Begin Design: 2019		
Cost Estimate	$830,000		
Reference Documents	Southern SSES Rehabilitation – Phase 1 (Agat-Santa Rita-Umatac-Merizo) (BC, 2016)		



This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Septic/Cesspool System Reduction Program			
Project Number	MP-WW-Pipe-27	**Basin**	All
Description	Connection of unsewered properties to eliminate septic systems and cesspools over the NGLA. Program components to include: • Construction of new sewer lines. • Connection of properties within wellhead protection areas and within 200 feet of an existing or newly constructed collection line. This includes evaluation of options to address the affordability and potential financing to home owners to help lessen the potential impact of the connection costs. • An on-site disposal system reduction strategy report which includes: ○ Prioritization of the connection of unsewered properties according to criteria outlined in the 2016 WRMPU Volume 1. ○ A plan to reduce or eliminate the construction of new septic systems over the NGLA. ○ A plan to connect existing septic/cesspool properties currently located within 200 feet of a sewer main and/or within wellhead protection zones. ○ A 20-year plan to connect existing septic/cesspool properties in conjunction with construction of new sewer lines at the rate of 5000 feet per year.		
Justification	To reduce potential for contaminants to enter the NGLA. This is a continuation of project CIP WW 17-01		
Proposed Schedule	• 2022: commission and complete report • 2023–2037: pipeline construction and connection of properties within 200 feet of new lines, 5000 feet/year • 2019–2023: connection of properties within 200 feet of existing lines and within wellhead protection areas		
Cost Estimate	$5.24M (Annually)		
Reference Documents	WRMPU Volume 3, Section 4.4 and WRMPU Volume 1, Section 5.2.5		

The map below is from Figure 4-8 and shows a sample area of 100,000 feet (5,000 feet of piping constructed each year for 20 years) to connect septic customers.



This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Manhole Rehabilitation Program			
Project Number	MP-WW-MH-01	**Basin**	All
Description	As part of a manhole rehabilitation and replacement program, group manhole issues into projects and put the projects out to bid to be fixed by a qualified contractor.		
Justification	Manhole replacement and rehabilitation is important to reduce and prevent I&I. A number of manholes are damaged and cracked, and excess water enters through cracks. Manholes should also be repaired or replaced before they fail, which can lead to more severe and costly problems.		
Proposed Schedule	Begin: 2020 (Contract every 2 years)		
Cost Estimate	$350,000		
Reference Documents	WRMPU Volume 3, Section 4.6		

This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

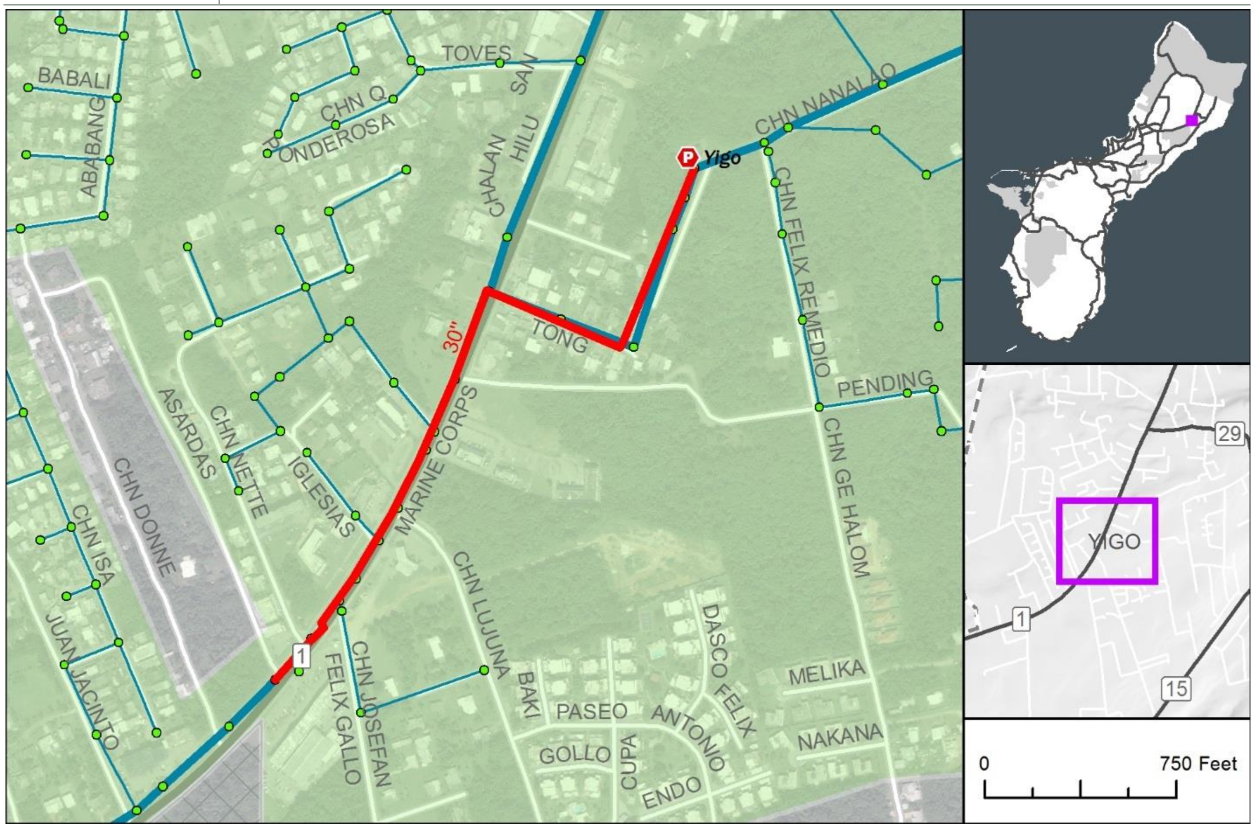
11.2 Force Main Projects

The following pages summarize the recommended force main projects.

Force Main Rehabilitation/Replacement Program			
Project Number	MP-WW-FM-01	**Basin**	All
Description	Implement an annual program to perform condition assessment and then rehabilitate and replace force main piping based on the results of the condition assessment. The force mains should be inspected according to the prioritization in Table 5-5. New piping should be sized to handle future planned peak wet weather flows.		
Justification	The risk analysis conducted for the force mains, described in Section 5.2, shows that GWA must begin with a pipe renewal program to replace piping that will reach the end of its service life.		
Proposed Schedule	Begin Assessments: 2021 (every 3 years)		
Cost Estimate	$1.58M (total for design and construction every 3 years)		
Reference Documents	WRMPU Volume 3, Section 5.2		

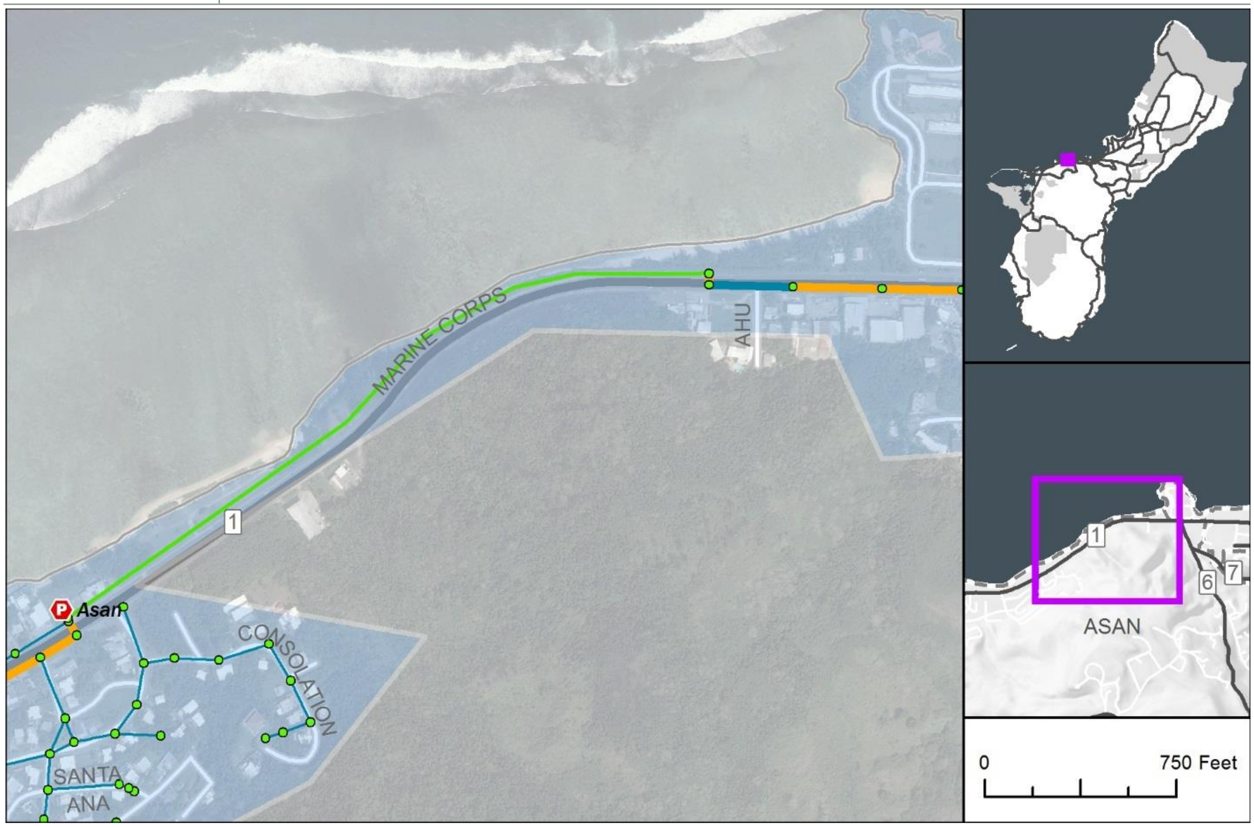
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Replace Yigo Lift Station Force Main			
Project Number	MP-WW-FM-02	**Basin**	Northern District
Description	Replace the existing 16-inch force main from the Yigo lift station.		
Justification	The force main is projected to have a capacity problem if the Yigo lift station is upgraded as discussed in Table 6-1.		
Proposed Schedule	Begin Design: 2021		
Cost Estimate	$3.33M		
Reference Documents	WRMPU Volume 3, Section 5.1		



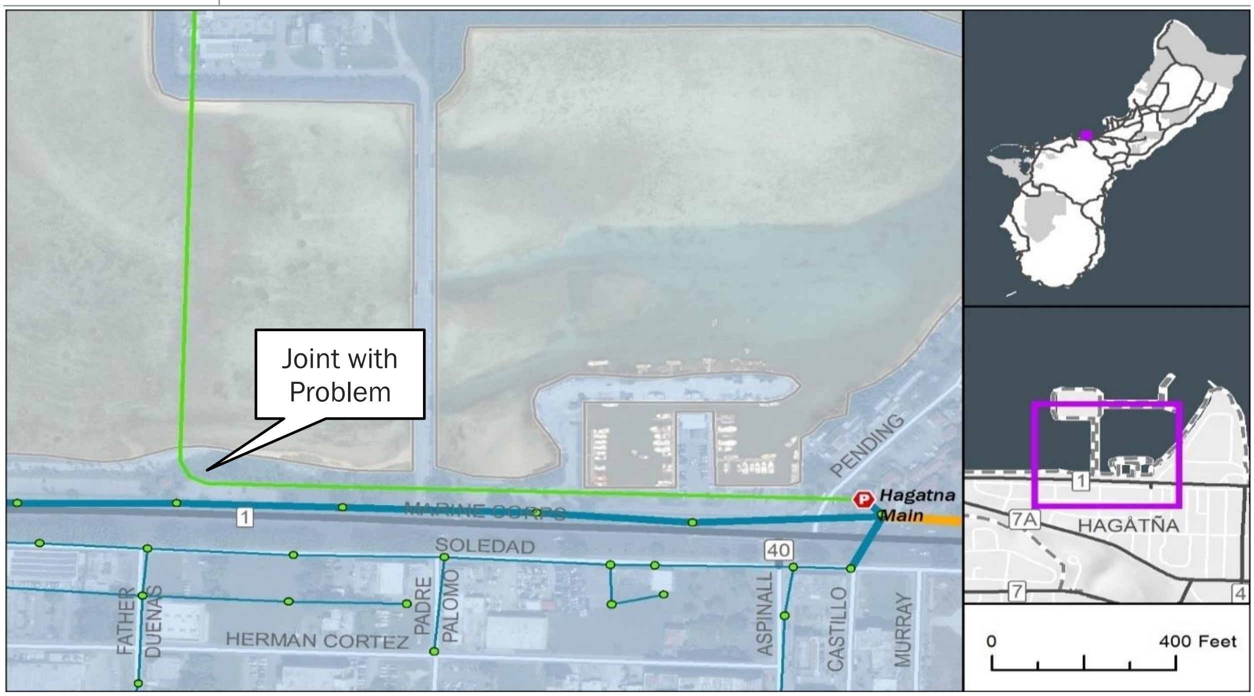
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Route 1 Asan Force Main Rehabilitation/Replacement			
Project Number	MP-WW-FM-03	**Basin**	Hagåtña
Description	Replace the force main along Route 1 (shown below in green).		
Justification	The force main has been exposed in this location due to erosion along the coastline. The pipe has required spot repairs in the past and a long-term solution is required.		
Proposed Schedule	Begin Design: 2018		
Cost Estimate	$2.30M		
Reference Documents	WRMPU Volume 3, Section 5.2		



This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Hagåtña WWTP Force Main Rehabilitation/Replacement			
Project Number	MP-WW-FM-04	Basin	Hagåtña
Description	Study options for the repair or replacement of the force main between the Hagåtña Main pump station and the Hagåtña WWTP. Then replace sections, replace, or parallel the pipeline. The pipeline was originally constructed as a gravity pipeline, but was converted to a force main. The pipeline is too long to CCTV so a manhole may need to be constructed along the pipeline to complete condition assessment of the entire pipeline. The model, which did not calibrate well in this area, predicted that the pipeline would need to be upsized from 24 to 42-inches. This project was costed as a 42-inch replacement, which gives a more conservative cost estimate due to the complexity of replacing this pipeline that runs under the ocean.		
Justification	The pipe was previously repaired at a joint as emergency work and the overall condition of this section of pipe is questionable. This is the only line feeding the Hagåtña WWTP, so a failure at this location would be a significant problem.		
Proposed Schedule	Begin Design: 2019		
Cost Estimate	$7.40M		
Reference Documents	WRMPU Volume 3, Section 5.2		



This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

11.3 Lift Station Projects

The following pages summarize recommended lift station projects.

Lift Station Rehabilitation/Replacement Program			
Project Number	MP-WW-Pump-01	**Basin**	All
Description	Rehabilitate and replace lift stations based on the capacity and condition assessment risk analysis. Lift stations should be grouped into projects and GWA should put the projects out to bid to be fixed by a qualified contractor. The projects should include a contract every two years with lift stations selected based on current information for each project. This project includes adding minor features such as pump hoists, odor control, grit removal, etc. at existing pump stations. The first group of projects should include adding grit removal before the Route 16 lift station as a high priority rehabilitation project.		
Justification	The model identified capacity issues and GWA operations found deficiencies at lift stations during condition assessment site visits. Continuation of CIP Project WW 09-01.		
Proposed Schedule	Begin Design: 2020 (rehabilitation of 10 lift stations every 2 years)		
Cost Estimate	$5.54M (assuming rehabilitation of 10 lift stations every 2 years)		
Reference Documents	See lift station prioritization list in WRMPU Volume 3, Section 6.2		

This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Tumon Basin - Fujita Lift Station Analysis			
Project Number	MP-WW-Pump-02	**Basin**	Hagåtña
Description	A study was recently done for the Fujita lift station and force main. The study report, titled *Preliminary Planning/Engineering Report, Fujita Pump Station Service Area Improvements* (CDM Smith, 2017), discusses issues and five options for the force main and lift station, such as a new parallel force main. GWA should review the report and select an option for implementation. A Tumon Sewer Basin Investigation study is also underway that will provide additional information on the work required in this area.		
Justification	The following is from the 2017 study: The Fujita Pump Station serves the Tumon Bay area including the major hotel area in lower Tumon. All the flow from Fujita is pumped to the Route 16 Pump Station through a single force main. Due to the lack of redundancy, the existing force main cannot be isolated or removed from service for long periods to perform repairs, maintenance or condition assessment. Failure of the force main could lead to service disruption and the main tourist area on the island would suffer a major impact that could affect the overall economic health of the Island. Also, based on the location of the Fujita Pump Station in the collection system, incoming flows to Fujita cannot easily be diverted to allow the pump station to be taken out of service for major repair activities.		
Proposed Schedule	Begin Design: 2019		
Cost Estimate	From 2017 study: $6.9M to $16.9M depending on alternative		
Reference Documents	WRMPU Volume 3, Section 6.3 *Preliminary Planning/Engineering Report Fujita Pump Station Service Area Improvements* (CDM Smith 2017)		

This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Replacement of Former Navy Pump Station (Donut Hole)			
Project Number	MP-WW-Pump-03	**Basin**	Hagåtña
Description	GWA inherited a previous Navy lift station in Tiyan that includes a bypass overflow structure and pipeline to Route 1. The lift station is in extremely poor conditions and needs to be replaced.		
Justification	The pump station condition is poor and an upgrade is required to provide safe and reliable operation of this pump station. A photo of the pump station is included below.		
Proposed Schedule	Begin Design: 2019		
Cost Estimate	$1.32M		
Reference Documents	WRMPU Volume 3, Section 6.2.1		

This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

11.4 WWTP Projects

The following pages summarize recommended WWTP projects.

Hagåtña WWTP Primary Treatment Repair/Rehabilitation Program			
Project Number	MP-WW-WWTP-01	**Basin**	Hagåtña
Description	Repair and replace worn out equipment and structures every 15 years. Expand CEPT capacity to 9.0 mgd.		
Justification	• Mechanical, electrical, and control systems wear out and require periodic replacement. • Structures may require rehabilitation measures to preserve structural integrity. • Ferrous metals experience accelerated corrosion due to exposure to sea salt. • A repair/rehabilitation project is recommended every 10 to 15 years. • First repair/rehabilitation project is scheduled for 15 years after completion of the CEPT improvements. • Expansion of CEPT capacity will be required to accommodate increasing flow. • Disinfection facilities may be required in advance of the Secondary Treatment Upgrade		
Proposed Schedule	Begin Design: 2026		
Cost Estimate	$24M		
Reference Documents	WRMPU Volume 3, Section 9		

This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

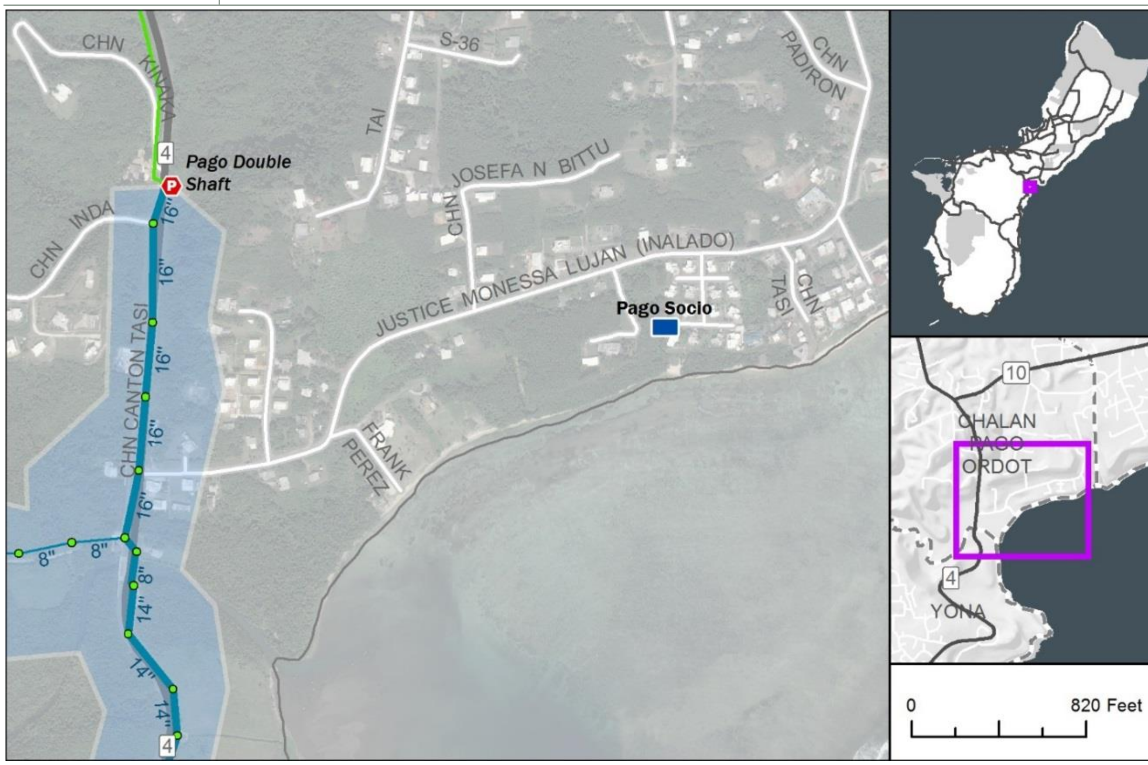
Hagåtña WWTP Secondary Treatment Upgrade			
Project Number	MP-WW-WWTP-02	**Basin**	Hagåtña
Description	Upgrade WWTP to provide full secondary treatment. Upgrades to include: • Upgrades to Hagåtña pump station to increase pump head. • New headworks, to include automatic screens, grit removal, and other elements. • New secondary treatment processes. • New disinfection system. • Effluent pumping modifications. • Solids treatment modifications. • Other improvements as required.		
Justification	Required to meet NPDES permit discharge requirements.		
Proposed Schedule	Begin Design: 2037 The scheduling for the HWWTP upgrade proposed is for illustrative purposes only given the 20-year Master Plan forecast horizon. The specific timing of this capital project will be determined based on Guam's financial capability to finance this project in the future.		
Cost Estimate	$208M		
Reference Documents	Hagåtña WWTP Facility Plan (in development)		

This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Inarajan WWTP Repair/Rehabilitation Program			
Project Number	MP-WW-WWTP-03	**Basin**	Inarajan
Description	Repair and replace worn out equipment and structures every 15 years.		
Justification	• Mechanical, electrical, and control systems wear out and require periodic replacement. • Structures may require rehabilitation measures to preserve structural integrity. • Ferrous metals experience accelerated corrosion due to exposure to sea salt. • A repair/rehabilitation project is recommended every 10 to 15 years. • First repair/rehabilitation project is scheduled for 15 years after completion of the last improvement project.		
Proposed Schedule	Begin Design: 2026		
Cost Estimate	$2M		
Reference Documents	WRMPU Volume 3, Section 9		

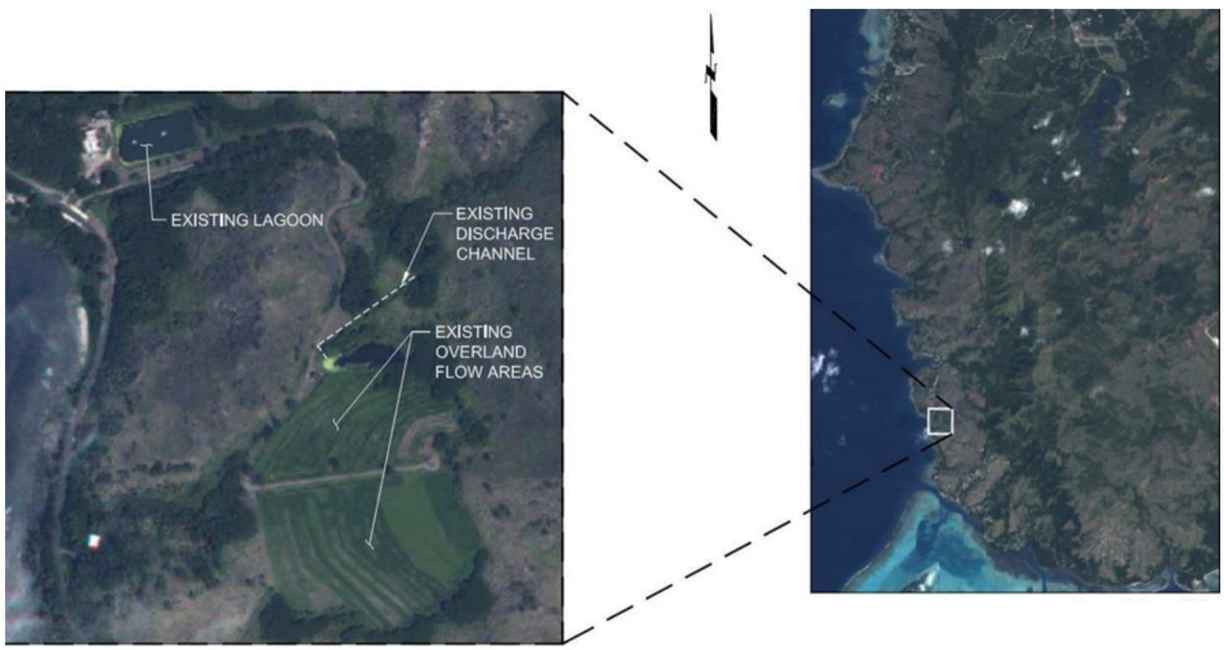
This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Pago Socio WWTP Pump Station Conversion			
Project Number	MP-WW-WWTP-04	**Basin**	Pago Socio/Hagåtña
Description	Convert the Pago Socio WWTP to a lift station and construct gravity and force main piping to transfer wastewater collected at Pago Socio to a sewer line on Route 4. A 2014 study, *Pago Socio Wastewater Transfer Study*, discussed five alternative routes to connect to the piping on Route 4 (BC, 2014). The alternatives include a combination of force main and gravity piping ranging from 2,660 to 3,600 feet.		
Justification	• Reduce O&M • Increase reliability		
Proposed Schedule	Prior to 2025, Planned for Design in 2021		
Cost Estimate	$2.46M to $3.14M depending on selected alternative		
Reference Documents	WRMPU Volume 3, Section 9 *Pago Socio Wastewater Transfer Study* (BC, August 2014)		



This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

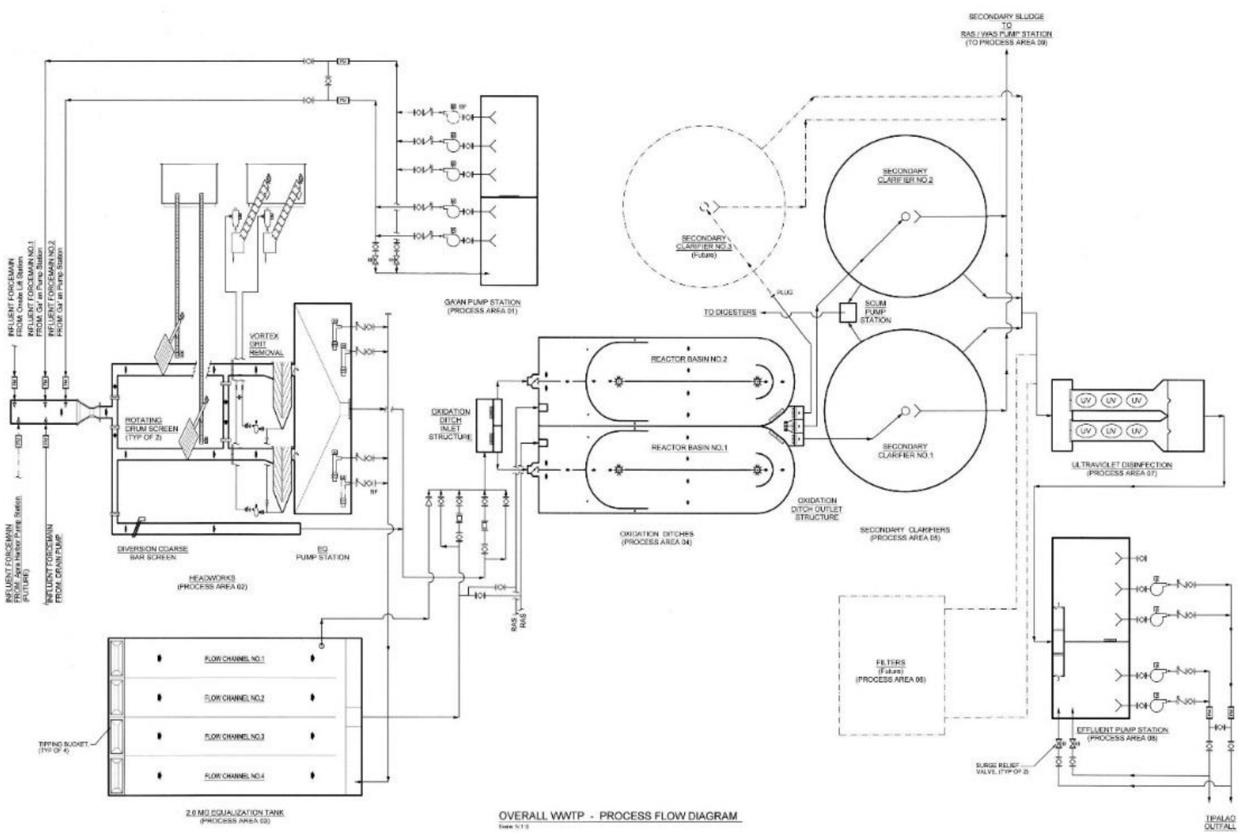
Umatac-Merizo WWTP Repair/Rehabilitation Program			
Project Number	MP-WW-WWTP-05	Basin	Umatac-Merizo
Description	Repair and replace worn out equipment and structures every 15 years.		
Justification	• Mechanical, electrical, and control systems wear out and require periodic replacement. • Structures may require rehabilitation measures to preserve structural integrity. • Ferrous metals experience accelerated corrosion due to exposure to sea salt. • A repair/rehabilitation project is recommended every 10 to 15 years. • First repair/rehabilitation project is scheduled for 15 years after completion of the last improvement project.		
Proposed Schedule	Begin Design: 2033		
Cost Estimate	$4.5M		
Reference Documents	WRMPU Volume 3, Section 9		



This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Agat-Santa Rita WWTP Repair/Rehabilitation Program			
Project Number	MP-WW-WWTP-06	**Basin**	Agat-Santa Rita
Description	Repair and replace worn out equipment and structures every 15 years.		
Justification	• Mechanical, electrical, and control systems wear out and require periodic replacement. • Structures may require rehabilitation measures to preserve structural integrity. • Ferrous metals experience accelerated corrosion due to exposure to sea salt. • A repair/rehabilitation project is recommended every 10 to 15 years. • First repair/rehabilitation project is scheduled for 15 years after completion of the last improvement project.		
Proposed Schedule	Begin Design: 2031		
Cost Estimate	$13.5M		
Reference Documents	WRMPU Volume 3, Section 9		

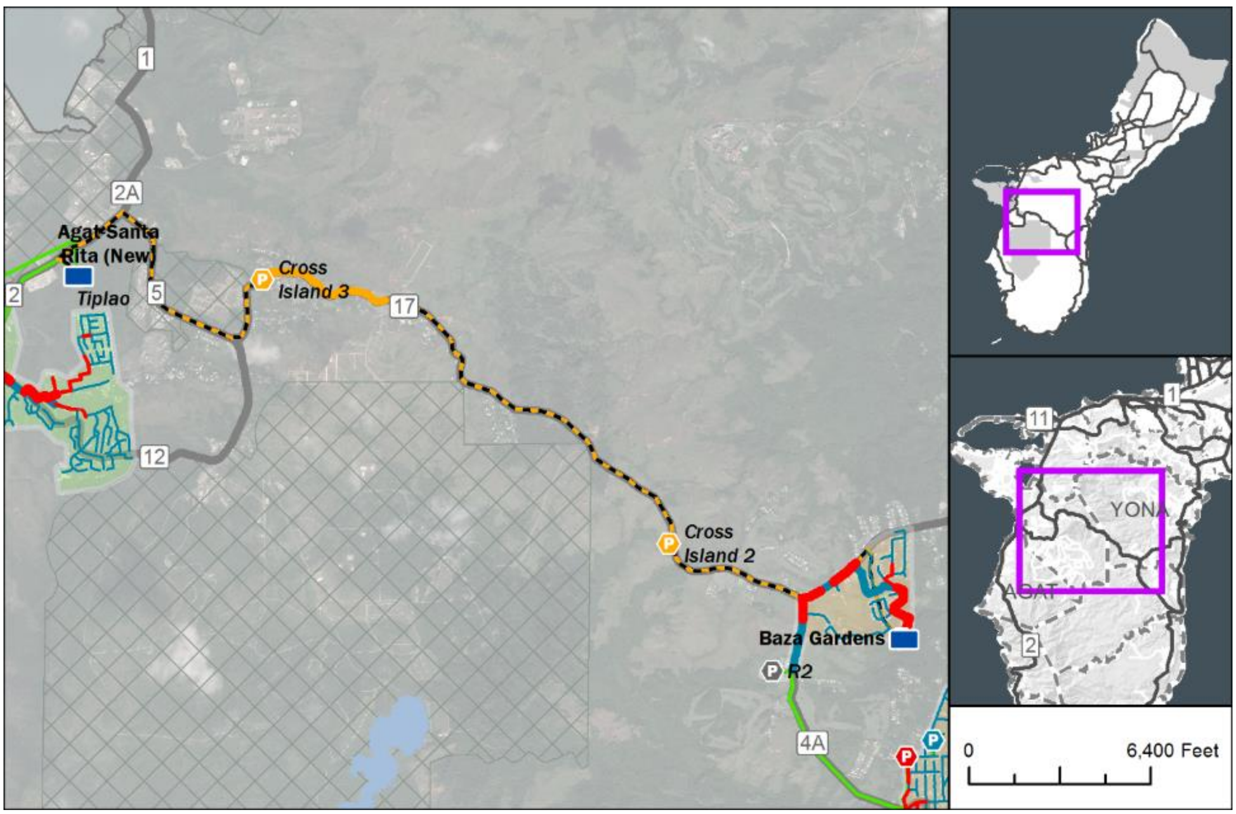
New Agat-Santa Rita Treatment Plant – Process Flow Diagram



This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

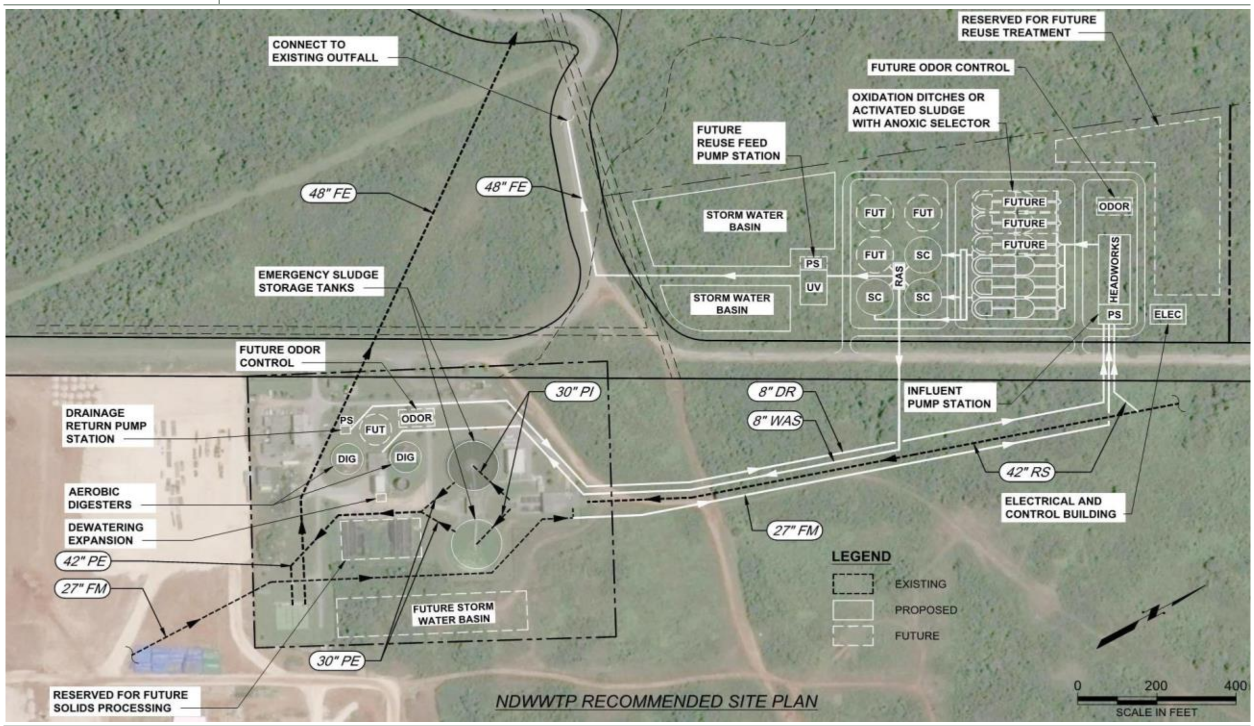
Baza Gardens Cross Island Pipeline – Preliminary Treatment Equipment Repair and Rehabilitation Program			
Project Number	MP-WW-WWTP-07	**Basin**	Baza Gardens, Agat-Santa Rita
Description	Repair and replace worn out equipment and structures every 15 years. This project includes the preliminary treatment (screens, grit removal, etc.) equipment installed at the former Baza Gardens WWTP site.		
Justification	• Mechanical, electrical, and control systems wear out and require periodic replacement. • Structures may require rehabilitation measures to preserve structural integrity. • Ferrous metals experience accelerated corrosion due to exposure to sea salt. • A repair/rehabilitation project is recommended every 10 to 15 years. • First repair/rehabilitation project is scheduled for 15 years after completion of the last improvement project.		
Proposed Schedule	Begin Design: 2032		
Cost Estimate	$2.5M		
Reference Documents	WRMPU Volume 3, Section 9		

Baza Gardens Cross Island Pipeline – Preliminary Pipeline Alignment



This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Northern District WWTP Completion			
Project Number	MP-WW-WWTP-08	**Basin**	Northern District
Description	The scope for the Northern District WWTP and trunk sewer projects has expanded since the original OEA/DoD funding amounts were determined. Due to property issues at the existing site, the liquid treatment process has been moved to an adjacent site requiring an additional influent pump station, headworks, piping and site development that was not anticipated. Modifications to the trunk sewer have also been necessary. This project provides funding to construct the additional facilities necessary for the added WWTP site and modifications to the Trunk Sewer.		
Justification	• Fixed DoD capital budget may be exceeded due to project scope changes. • New expansion site incurs additional infrastructure needs not contemplated by DoD when the budget was established.		
Proposed Schedule	2021–2022		
Cost Estimate	$17M		
Reference Documents	Northern District WWTP Facility Plan		



This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Ocean Outfall Inspection Program			
Project Number	MP-WW-WWTP-09	**Basin**	Northern District and Hagåtña
Description	Visual inspection of ocean outfalls at both Northern District and Hagåtña WWTP		
Justification	• Corrosion and deterioration of the outfalls can go undetected and potentially affect water quality • Water quality is monitored at the discharge point (ocean surface) without inspection of the entire length of the outfall pipe • Cleaning may be required		
Proposed Schedule	Begin: 2020 (every 5 years)		
Cost Estimate	$150,000 (every 5 years)		
Reference Documents	WRMPU Volume 3, Sections 7.7.4 and 7.8.4		

This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

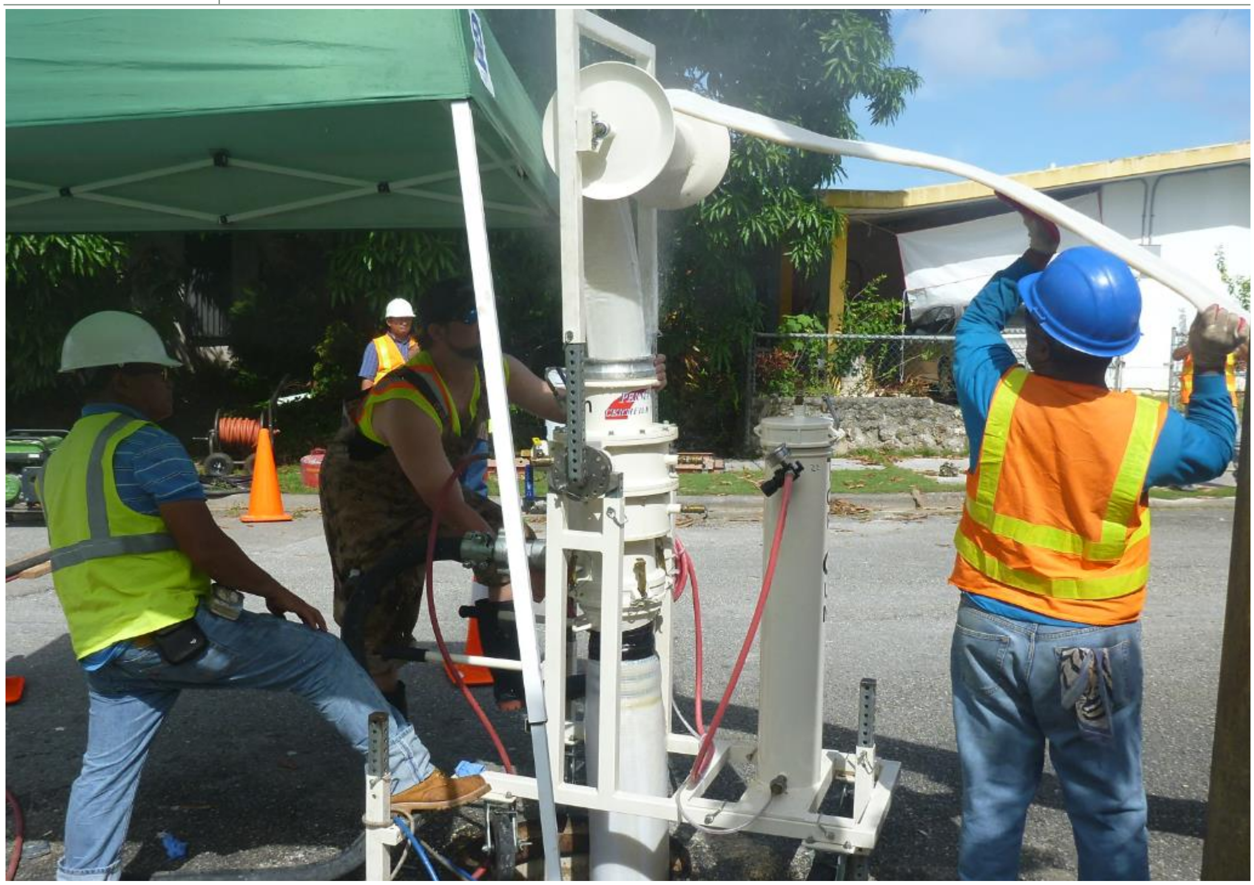
11.5 Other Wastewater Projects

The following pages summarize other recommended wastewater projects.

Update Wastewater Collection System Model			
Project Number	MP-WW-Misc-01A and B	**Basin**	All
Description	The wastewater collection system hydraulic model should be regularly updated. Periodic flow metering should be performed to collect data to calibrate the model. Model piping should be updated to the latest GIS and the model should be periodically calibrated to the latest flow metering data. Plan for 1 major update then biannual revisions.		
Justification	Incomplete data was used for portions of the computer model (such as for the Hagåtña basin as explained in Section 3.2.3). The model should be updated to the latest and most accurate data to allow for accurate modeling and sizing of system improvements.		
Proposed Schedule	2018 (Major Update); 2020 (Continued Updates) (every 2 years thereafter)		
Cost Estimate	$500,000 (Major Update), $200,000 (every 2 years thereafter)		
Reference Documents	WRMPU Volume 3, Section 3		

This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

I/I and SSES Assessments			
Project Number	MP-WW-Misc-02	Basin	All
Description	This project will cover I/I and SSES assessment work as necessary to determine the high probability locations where I/I is occurring. This project will review areas defined in the hydraulic model or based on operating anomalies where high I/I is suspected, but the root cause has not been determined adequately to define a repair, rehabilitation, or expansion project. This project may include installation of temporary flow meters, smoke testing, dye testing, CCTV, surveying, or other investigation techniques. The project could also include the implementation of longer-term flow metering to track changes in I/I as repairs are made. This project can also include minor repairs necessary to decrease I/I. This can include but is not limited to manhole inspection, manhole mapping, raising manholes, manhole seals, repairs to covers and frames, and other manhole defects. It can also provide limited gravity sewer inspection and limited gravity sewer repair and rehabilitation. This project will likely be performed by an outside consulting firm.		
Justification	This project will complete the preliminary assessments to define the scope of future larger rehabilitation and repair projects. The project may also identify areas where unauthorized connections for stormwater systems are occurring.		
Proposed Schedule	Begin: 2020, continue with one project every 3 years		
Cost Estimate	$400,000		
Reference Documents	WRMPU Volume 3, Section 9.5		



This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Miscellaneous Wastewater Improvements			
Project Number	MP-WW-Misc-03	**Basin**	All
Description	This project addresses any miscellaneous projects not covered in other CIP projects. Typical items covered under this project include addressing issues at hotspots (GWA is currently tracking 45 hotspots) and other collection system problems that cause overflows or excessive maintenance, identifying and correcting areas where stormwater systems are connected to the sewers, identifying and correcting where illegal connections exist, etc. This project also includes a potential on-call contract to complete inexpensive repairs that GWA does not have the resources to address in a timely manner.		
Justification	GWA's system is aging and there are various deficiencies throughout the system because components of the system were constructed without proper documentation, sometimes with improper materials, and without construction oversight. This project will allow GWA to define and correct the defects, illegal connections, and other issues on a routine basis to improve the overall system.		
Proposed Schedule	Begin: 2019, continue with projects every 2 years		
Cost Estimate	$1.19M (every 2 years)		
Reference Documents	WRMPU Volume 3, Section 9.5		

This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

Fats, Oils, and Grease Study			
Project Number	MP-WW-Misc-04	**Basin**	All
Description	This project will study the disposal options for Fats, Oils and Grease (FOG) collected during grease trap cleaning, sewer line cleaning and other sources. FOG is typically treated in anaerobic digesters, but no treatment plants on Guam currently include anaerobic digestion as part of the treatment process. Other disposal options are needed.		
Justification	Disposal of FOG is problematic and it may prevent illegal dumping of FOG into the wastewater treatment system if a proper disposal location can be provided. This would allow GWA to track the collection and disposal of FOG throughout the island. The quantity of FOG being collected will increase as GWA/Guam EPA increase inspections and begins enforcement of the planned Source Control Program.		
Proposed Schedule	Begin Study: 2019		
Cost Estimate	$150,000		
Reference Documents	WRMPU Volume 3, Section 4.6		

This proposed project is subject to change. Projects will generally include an engineering study, detailed design, and field verification to refine the exact project scope and budget. Costs are presented in 2017 dollars and do not account for increases due to inflation and escalation. See Volume 1, Appendix D for cost estimate assumptions.

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Section 12
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Appendix A

Temporary Flow Metering

This appendix summarizes the temporary flow metering used for hydraulic model calibration. Temporary flow metering was performed for past studies by GWA, ADS Environmental Services; EA Engineering, Science, and Technology; and Stanley Consultants. Table A-1 summarizes the flow metering periods.

Table A-1. Flow Meter Summary

Meter	Period of Record	Installed By
Agat-Santa Rita (7 meters)		
FM09, FM10, FM11, FM12, FM13, FM14, FM15	9/30/2012–11/10/2012	ADS
Baza Gardens (6 meters)		
FM01, FM02, FM03, FM04, FM05, FM08	9/30/2012–11/10/2012	ADS
Hagåtña (14 meters)		
FM03, FM17	1/14/2014-2/10/2014	GWA
FM06	11/21/2013-2/7/2014	GWA
FM08	1/16/2014-2/7/2014	GWA
FM09, FM14	1/14/2014-2/7/2014	GWA
FM11	1/9/2014-2/7/2014	GWA
FM16	11/19/2013-2/10/2014	GWA
FM18, FM23, FM28	1/13/2014-2/10/2014	GWA
FM19	11/18/2013-2/6/2014	GWA
FM22B	11/20/2013-2/10/2014	GWA
FM27	11/21/2013-2/6/2014	GWA
Inarajan (1 meter)		
FM28	8/1/2005-8/31/2005	Unknown
Northern District (10 meters)		
FM01	1/23/2014–1/12/2015	GWA
FM02	11/6/2013–1/12/2015	GWA
FM03	11/6/2013–12/5/2014	GWA
FM06	11/5/2013–1/12/2015	GWA
FM07, FM17	11/6/2013–1/15/2015	GWA
FM16	11/7/2013–12/31/2014	GWA
FM18	11/6/2013–1/13/2015	GWA
FM23	11/21/2013–1/15/2015	GWA
FM26	11/21/2013–10/22/2014	GWA

Table A-1. Flow Meter Summary		
Meter	Period of Record	Installed By
Tumon (10 meters)		
FM-1	11/15/2014–1/29/2015	EA and Stanley
FM-2, FM-3, FM-6	11/17/2014–11/29/2015	EA and Stanley
FM-4, FM-7, FM-8	11/19/2014–1/29/2015	EA and Stanley
FM-5, FM-9	11/18/2014–1/29/2015	EA and Stanley
FM-10	12/6/2014–1/29/2015	EA and Stanley
Umatac-Merizo (3 meters)		
FM06, FM07, PL01 (Pump Logger)	9/30/2012–11/10/2012	ADS

Figures A-1 through A-3 show the locations of the flow meters.

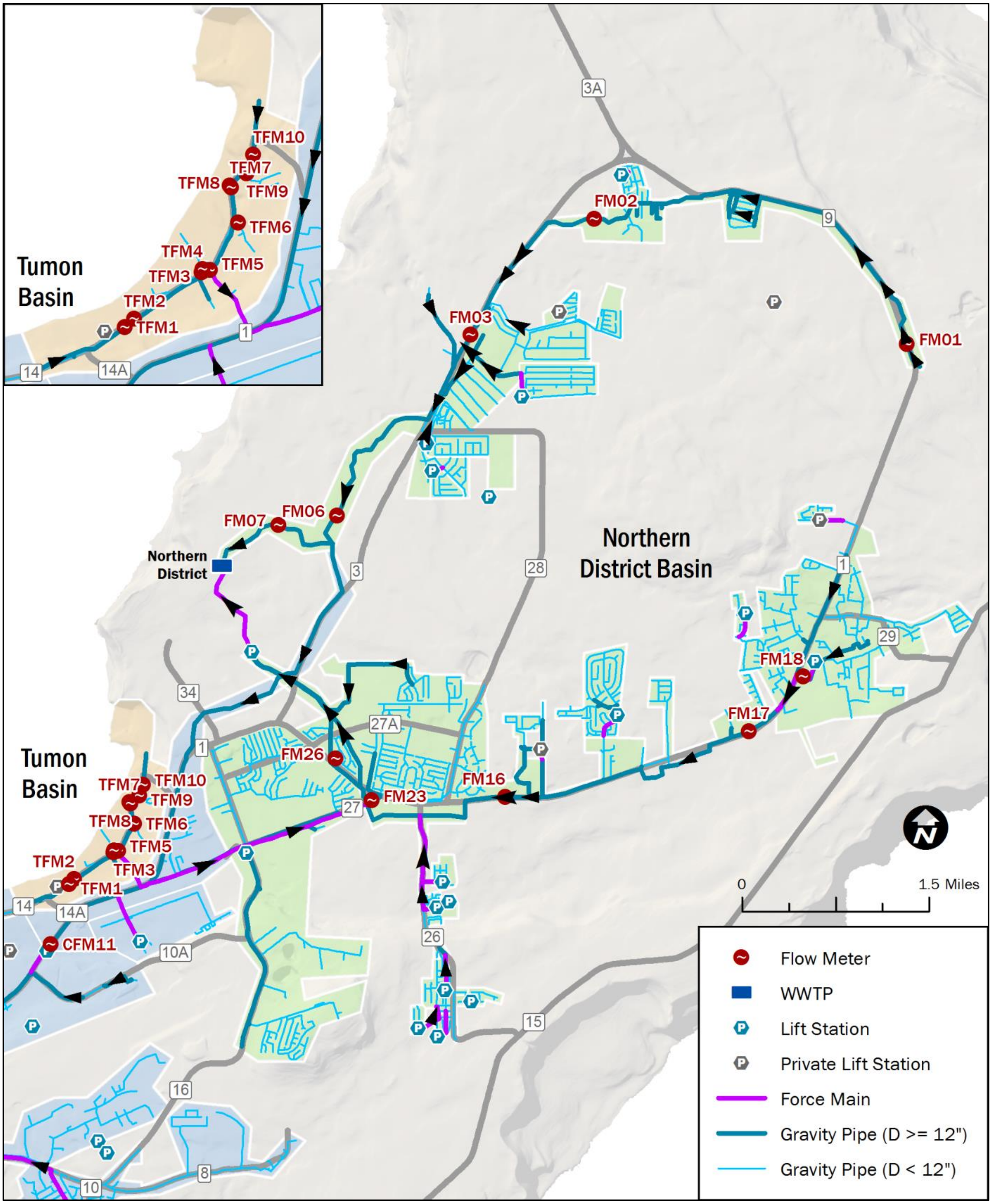

Figure A-1. Northern District and Tumon Flow Meter Locations

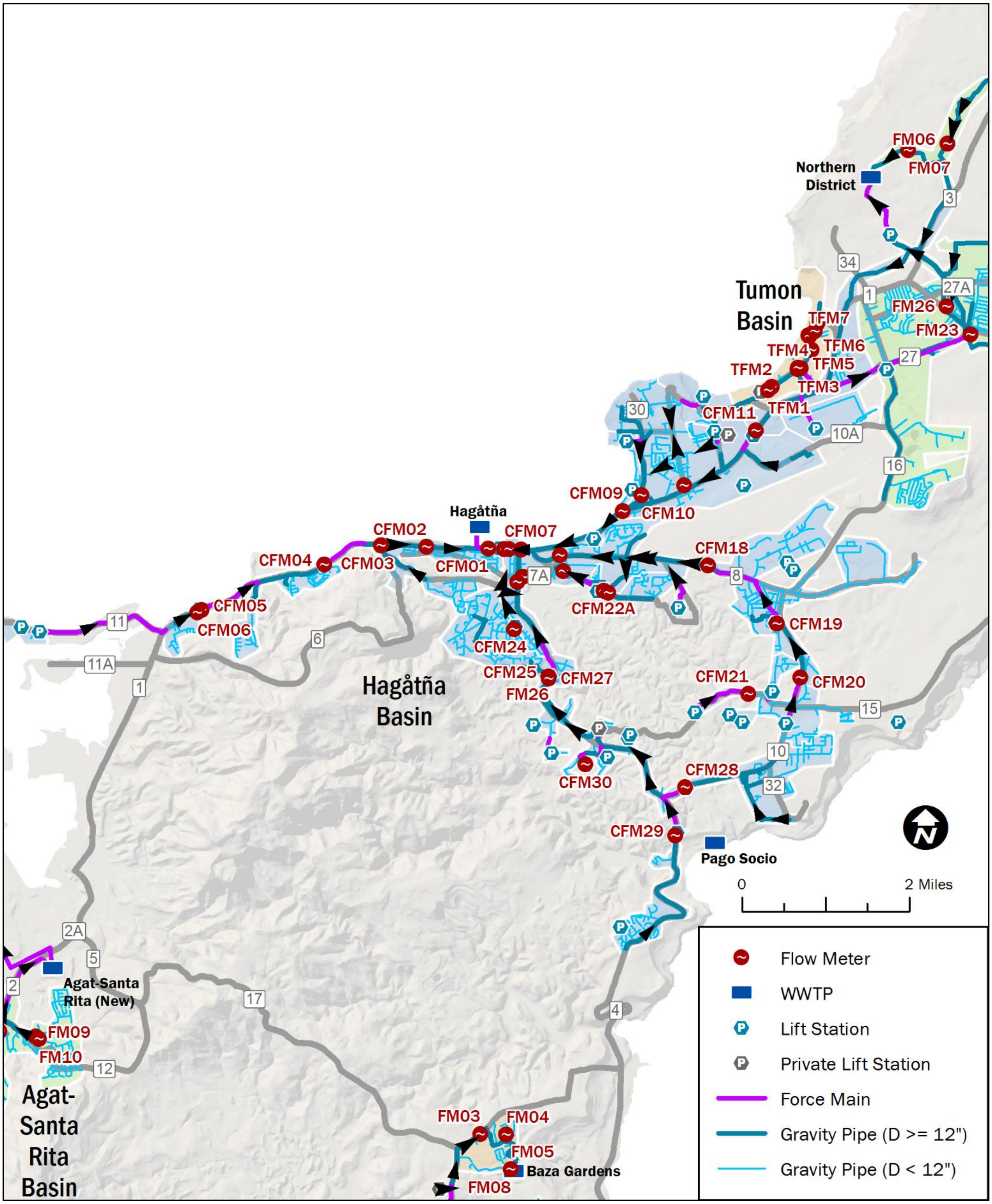

Figure A-2. Hagåtña Flow Meter Locations

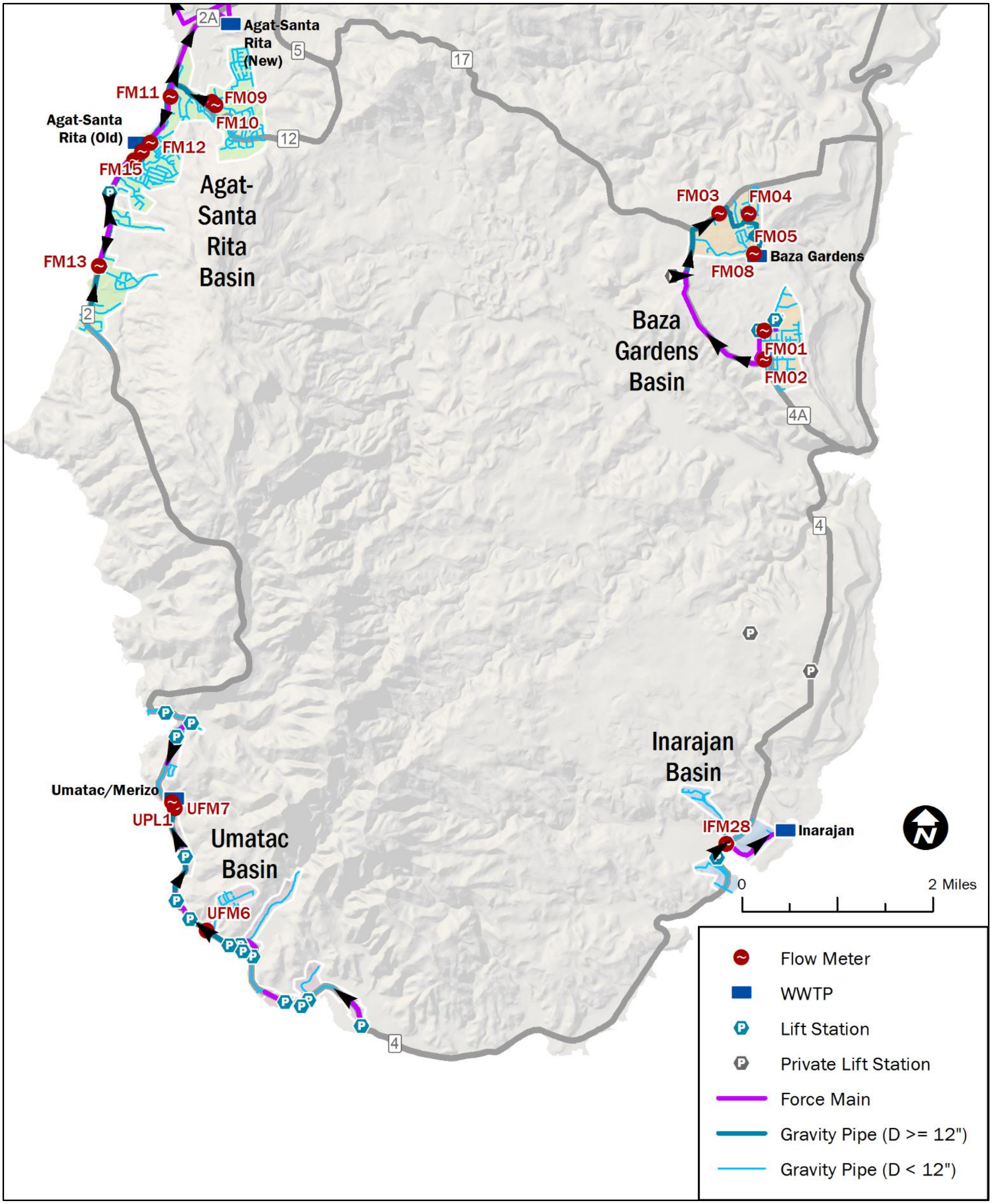

Figure A-3. Southern Basin Flow Meter Locations

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Appendix B
Model Development Notes

This appendix contains additional details regarding the development of the wastewater model described in Section 3.

Model GIS

This section summarizes the procedures and data used to update the collection system GIS database used for the sewer model and provides information on the limitations of the GIS data.

Required Data

The following information was required to build the sewer model:

1. Full connectivity of the entire system (all pipes must drain properly to a system outfall)
2. Manhole coordinates, rim elevations, and depths
3. Pipe diameter and invert elevations

The GIS database was missing some of this information, which was filled in as summarized below.

GIS Update

BC first interviewed GWA staff about connectivity issues throughout the island. In most cases, GWA staff sketched the system connectivity from their general knowledge of the system. In a few cases, field visits were performed by GWA or BC staff. Field visits included opening manholes to review connectivity and to measure the depth of the manholes.

The next step was to use as-built drawings to fill in information gaps at the lift stations including wet well dimensions, inlet and outlet pipe diameters and elevations, and number of pumps. Due to large amounts of missing information in the GIS lift station data, BC focused this effort on the key lift stations routing flow through major lines in the conveyance system.

The last step in the process was to use interpolation to fill in gaps for manhole rim elevations, manhole and pipe invert elevations, and pipe diameters. Missing manhole rim elevations were interpolated from the 2007 LIDAR data. To verify the quality of this methodology, interpolated LIDAR rim elevations were compared to GIS rim elevations in locations where the original GIS database provided rim elevation values. Discrepancies between the two datasets were generally within one foot of each other, but were up to ten feet in locations of steep ground slopes. Therefore, rim elevations at manholes with interpolated values should be treated with care and surveyed before being used for detailed design. Linear interpolation between known inverts was used to fill in missing invert elevations. In cases where this would cause the pipe to be aboveground, the inverts were set five feet below the ground surface elevation. Pipes with negative slopes were manually corrected. Pipe diameters were interpolated from neighboring pipes if possible. For the areas where no pipe diameters were available, missing pipe diameters were assumed to be 8 inches.

GIS Attributes

The source of the information for the manhole and pipe attributes was recorded during the GIS update process in a copy of the GIS database. The fields listed in Table B-1 were added to the copy of the GIS database to record the source of the manhole and pipe attributes. While using the sewer model, this source information will allow the user to understand the accuracy of the model in specific locations. For example, in locations where much of the elevation information was interpolated, additional field work may be required to measure elevations and inverts to improve the accuracy of the model results.

Table B-1. Data Source Attributes in Listed in GIS Database

GIS Layer	GIS Field	Value	Description
Manholes	SOURCE	GIS	Manhole present in original GWA GIS
		GWA	Manhole added per GWA update/correction
	RIM	GIS	Rim elevation in original GWA GIS
		LIDAR	Rim elevation interpolated from LIDAR
	INVERT	GIS	Invert elevation in original GWA GIS
		INTERPOLATED	Invert elevation interpolated from neighbors or edited manually
Pipes	SOURCE	GIS	Conduit present in original GWA GIS
		GWA	Conduit added per GWA update/correction
	DIAMETER	GIS	Diameter in original GWA GIS
		INTERPOLATED	Diameter interpolated from neighbors or edited manually
	INVERT	GIS	Invert elevation in original GWA GIS
		INTERPOLATED	Invert elevation interpolated from neighbors or edited manually

Diurnal Patterns

The diurnal patterns used in the model for each basin are shown in Figures B-1 through B-7. A separate diurnal pattern was calculated for the area draining to each flow meter. For example, the “FM09/FM10” curve in Figure B-1 was applied to manholes upstream of flow meters FM09 and FM10. For most of the flow meters, a single pattern was created for the entire week. However, separate weekend patterns were created for meters with wastewater flow patterns that were different between weekdays and weekends. The figures show these weekend patterns.

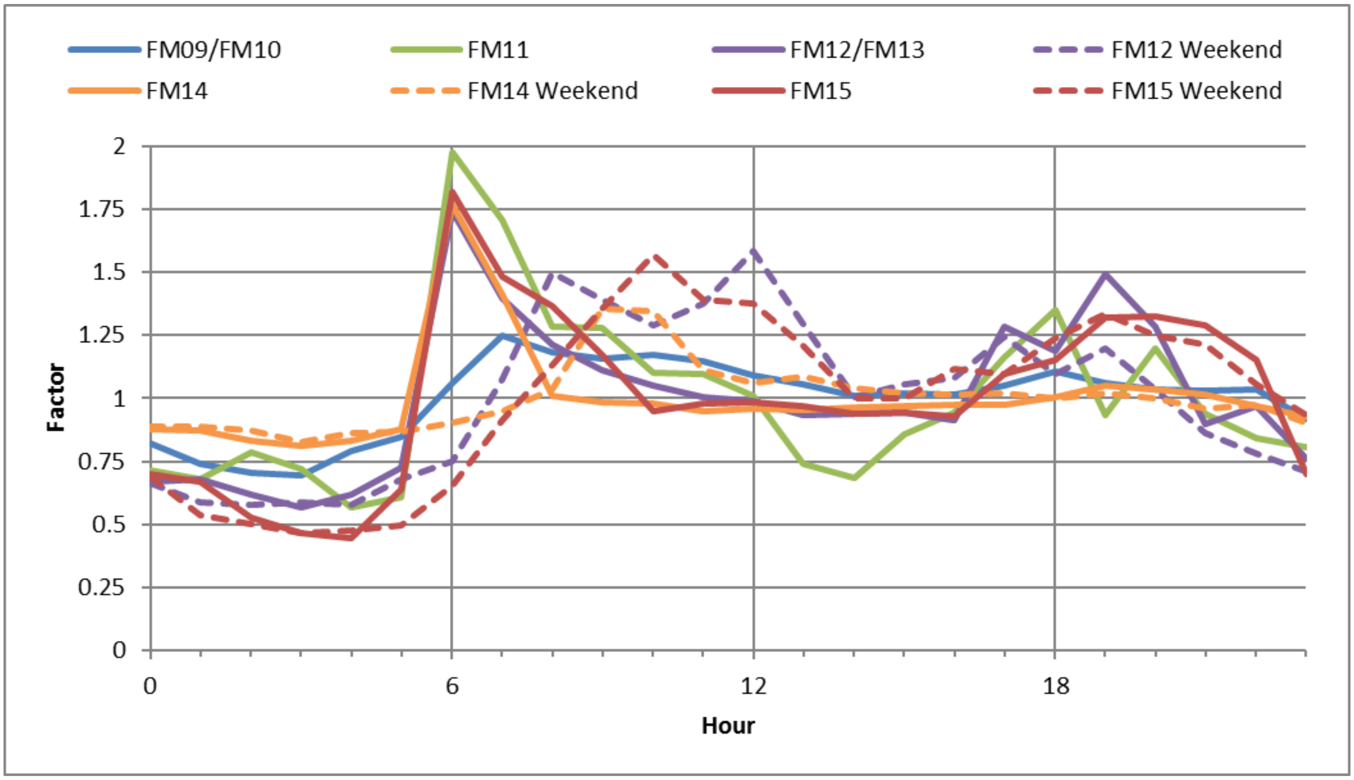

Figure B-1. Agat-Santa Rita Diurnal Patterns

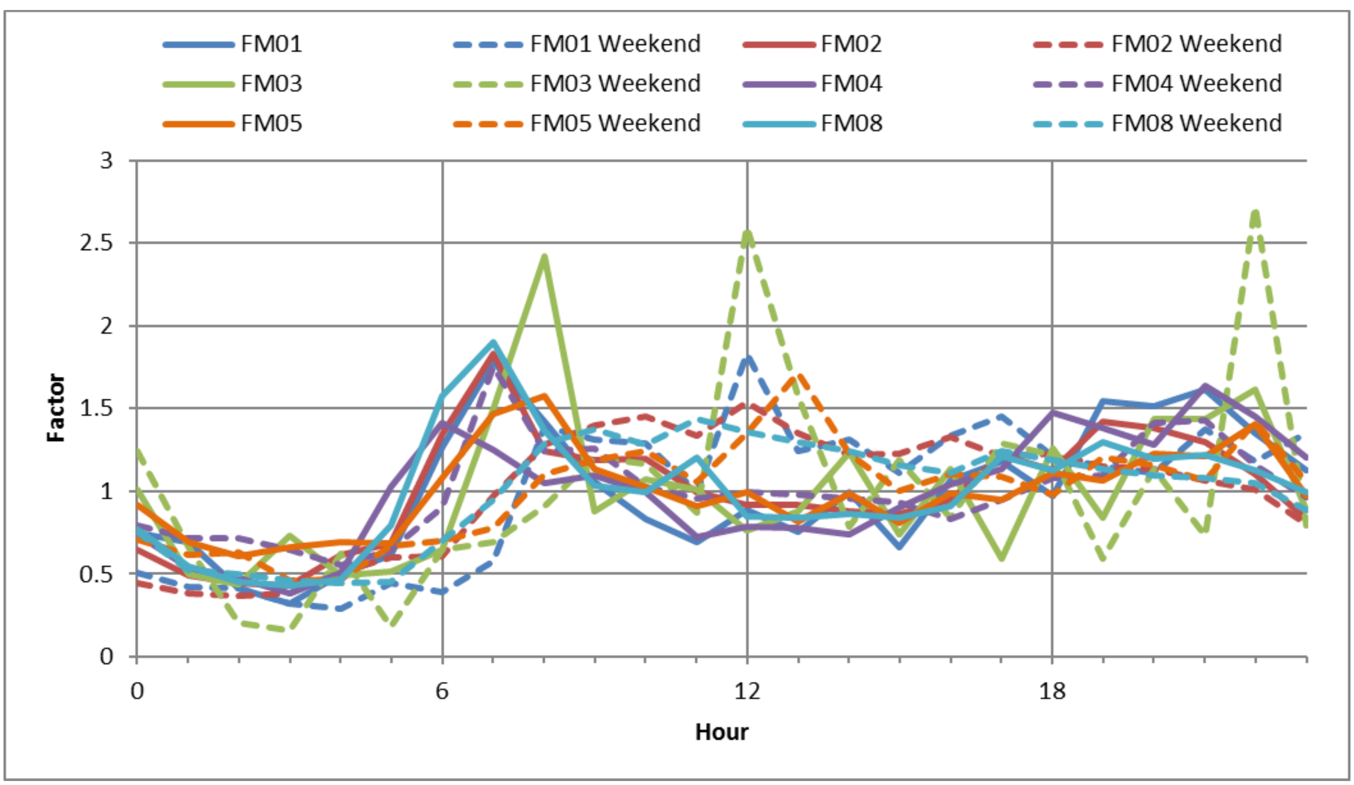

Figure B-2. Baza Gardens Diurnal Patterns

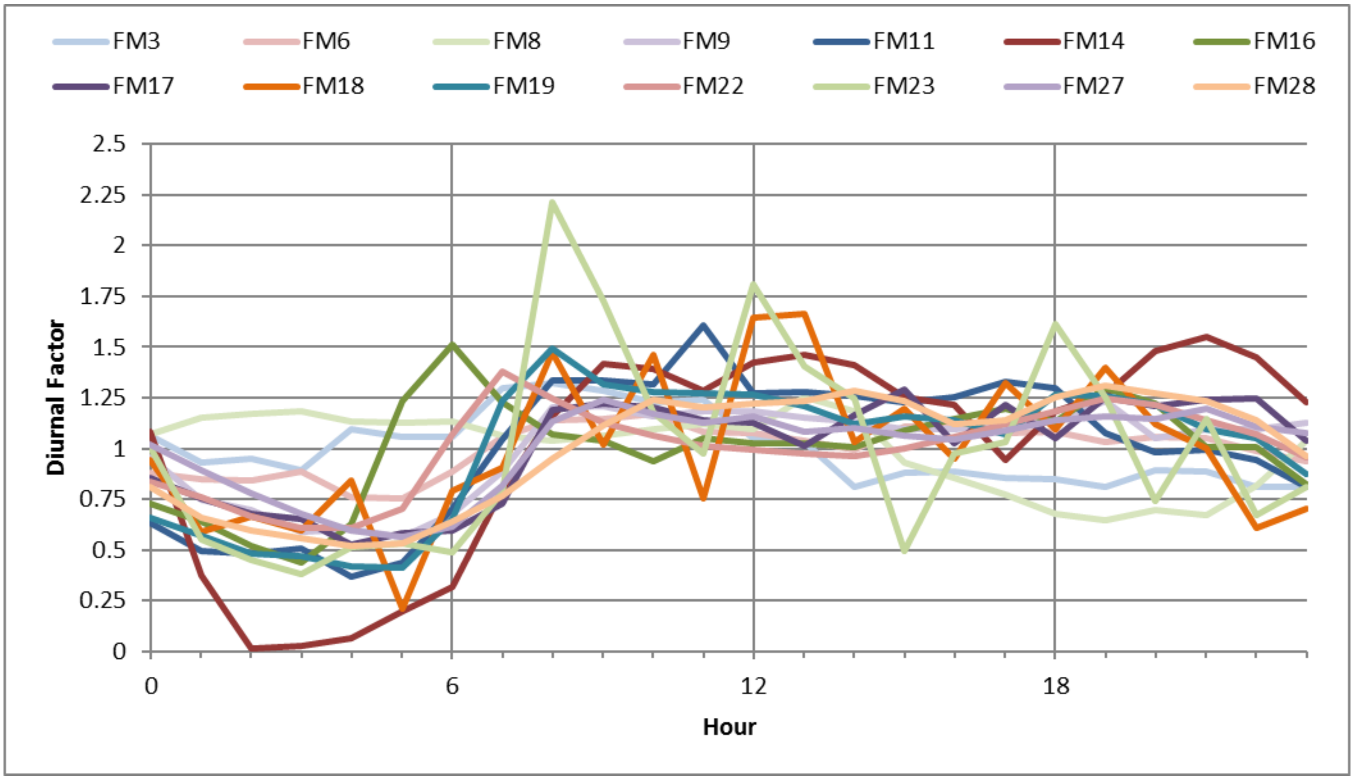

Figure B-3. Hagåtña Diurnal Patterns

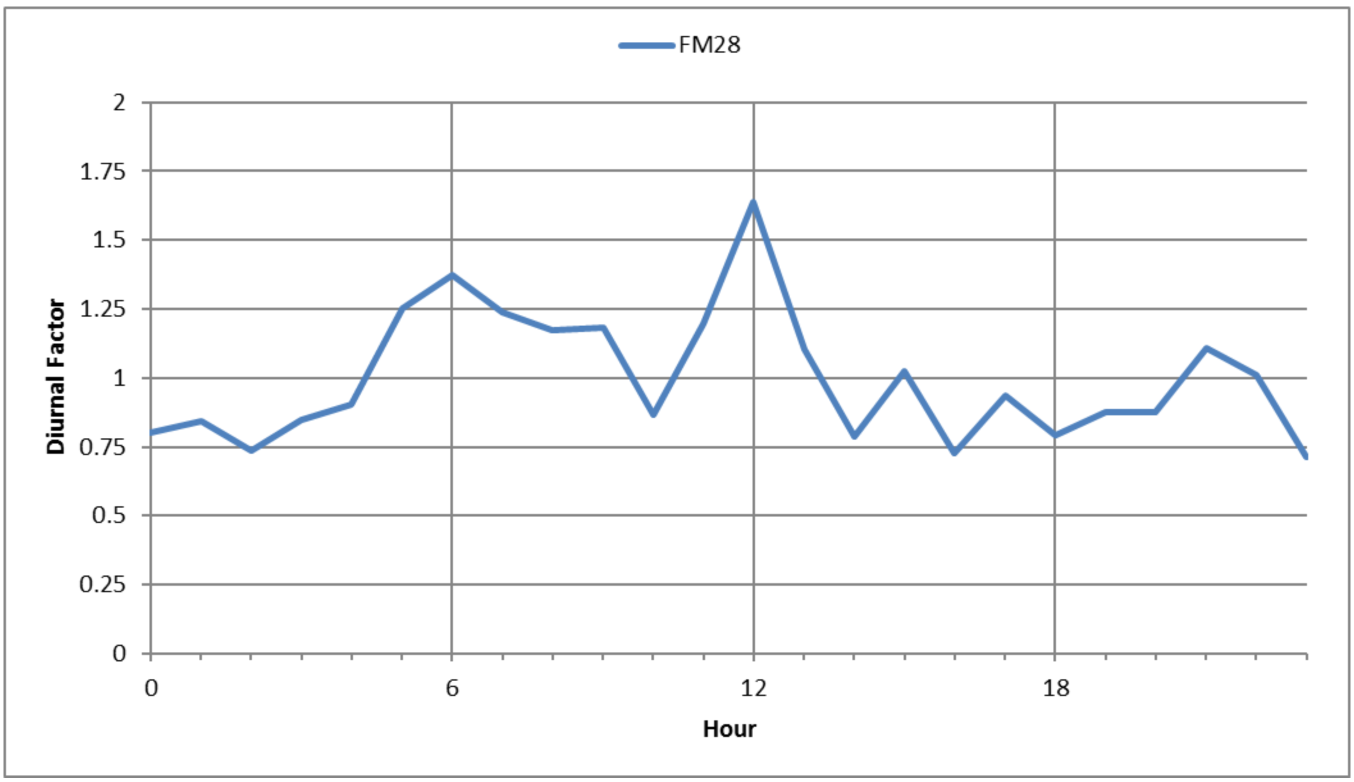

Figure B-4. Inarajan Diurnal Pattern

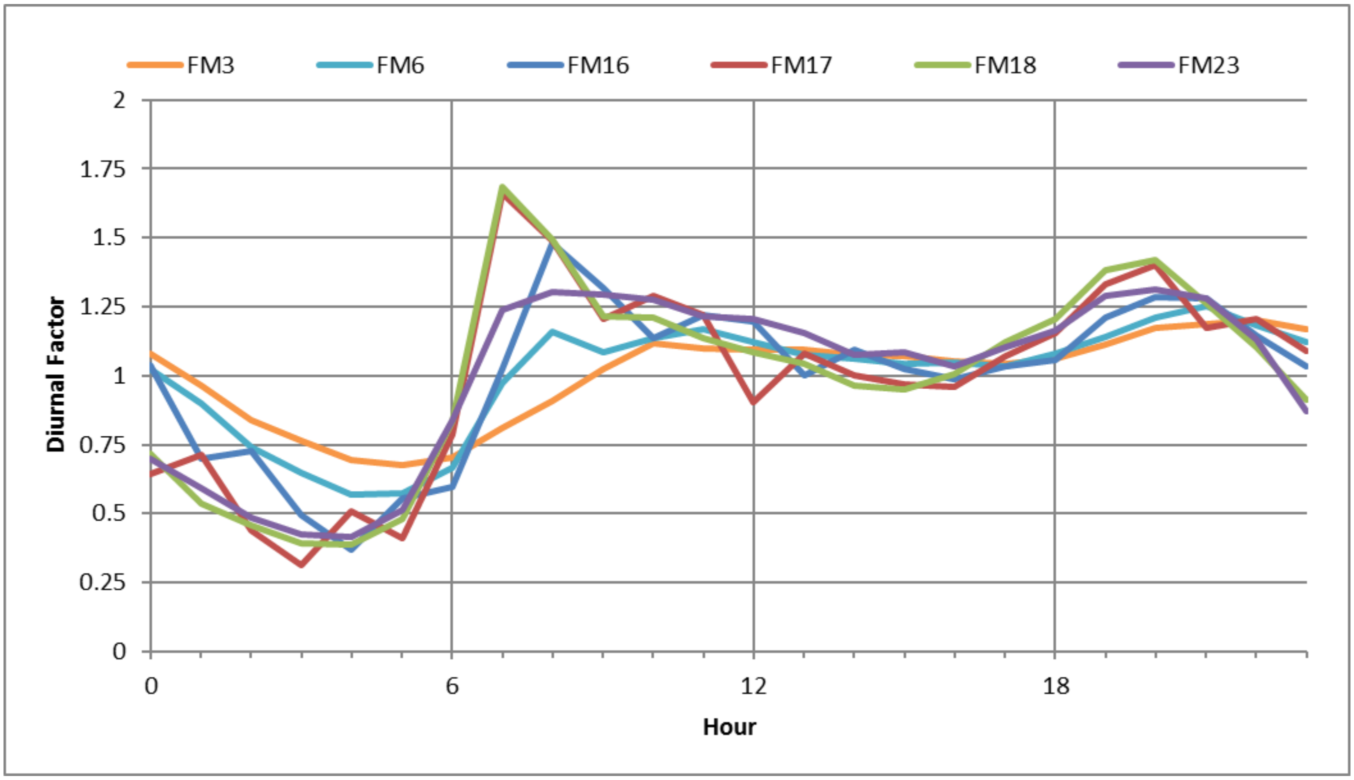

Figure B-5. Northern District Diurnal Patterns

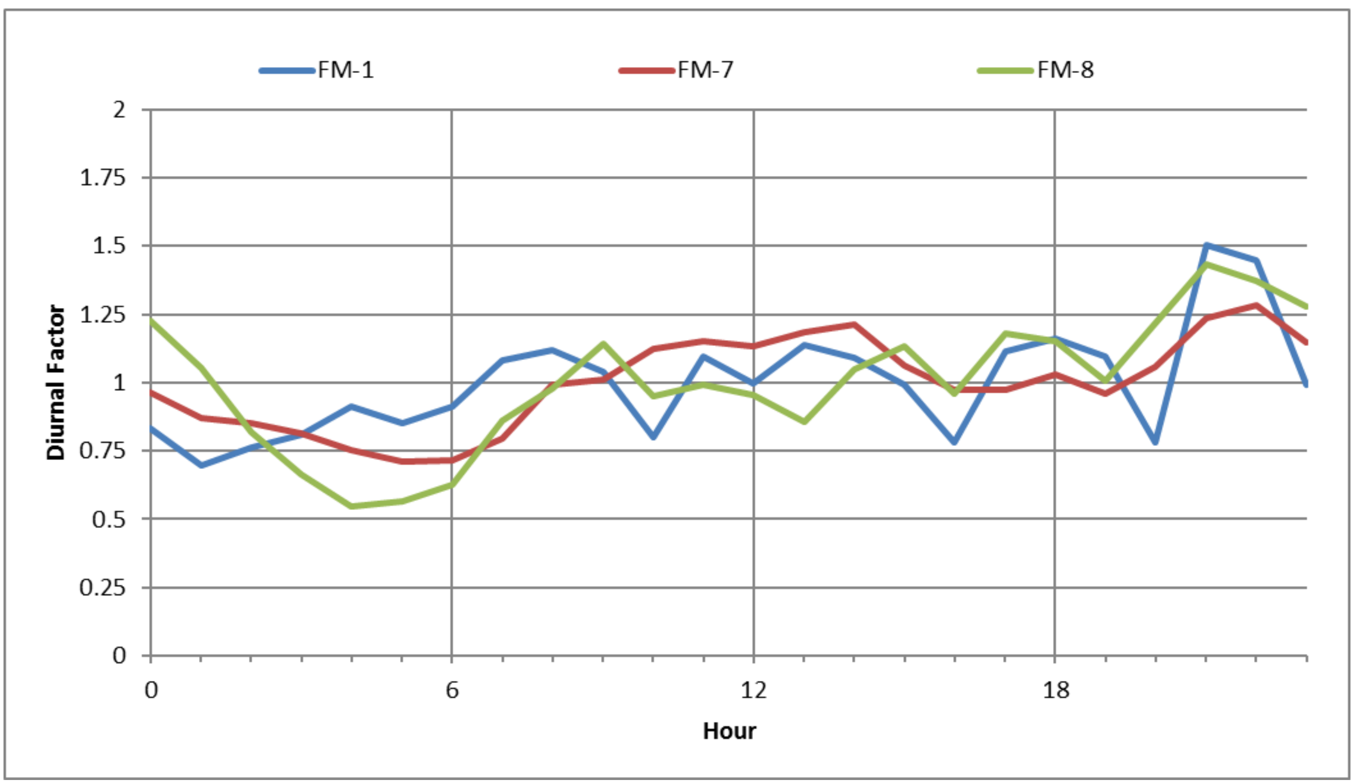

Figure B-6. Tumon Diurnal Patterns

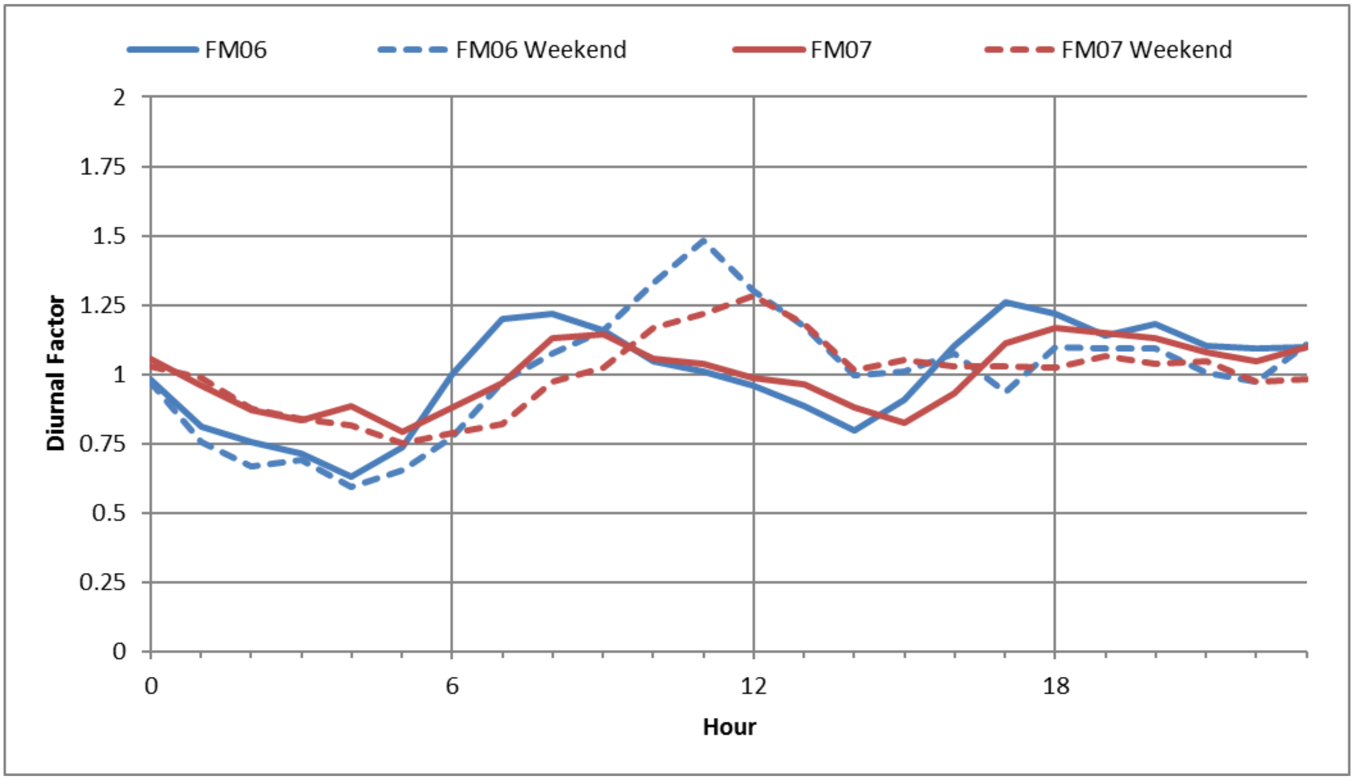

Figure B-7. Umatac-Merizo Diurnal Patterns

Appendix C

Wastewater Collection System Criteria

This appendix outlines criteria used for evaluation and design of the wastewater collection system. The evaluation criteria were used in identifying deficiencies during the analysis of the collection system. The design criteria were used in developing recommendations to address the identified deficiencies. The criteria in this appendix were based on criteria used for the 2006 WRMP, criteria used for other utilities, and feedback from GWA.

Collection System Piping Criteria

Figure C-1 shows the key dimensions used in defining the criteria for gravity pipelines.

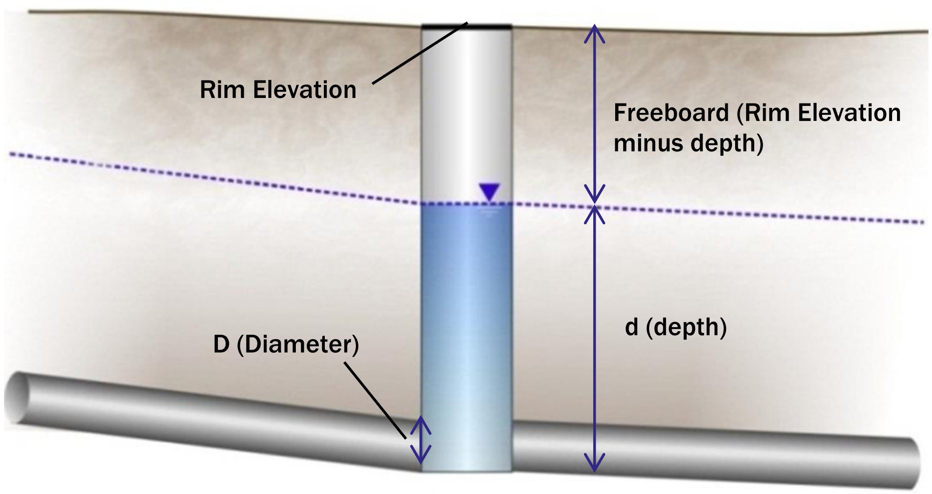

Figure C-1. Dimensions for Gravity Pipelines Used in Criteria

Depth to diameter (d/D) is calculated as the depth divided by the diameter. For a d/D greater than 1, the pipe is surcharged (the water level is at or above the top of the pipe) and the depth is defined by the elevation to which water would rise in a manhole, as shown in Figure C-1.

Table C-1 lists the criteria used for collection system piping.

Table C-1. Collection System Piping Criteria		
Item	**Description**	**Value**
Gravity pipe capacity	Allowable d/D at peak flow (evaluating system)	A pipe was flagged as deficient if the d/D was greater than 1 (if the water level reaches the top of the pipe). The following were considered when analyzing d/D: 1. SSOs – Priority was given to capacity issues that included SSOs. SSO reports compiled by GWA were reviewed and compared to model results to identify the highest priority capacity issues. 2. Pipe segments with d/D > 1 because of backups from downstream piping were not considered to have insufficient capacity.
	Allowable d/D at peak flow (designing new piping)	Recommended replacement piping was sized so the peak flow would be within the pipe (d/D < 1)
Force main capacity	Peak velocity (evaluating system)	10 feet/second
	Peak velocity (designing new piping)	5 feet/second
	Lift station flow	The lift stations were first analyzed and upsized in the model if necessary. The force mains were analyzed with the lift stations pumping with the largest pump on standby. Force mains were not analyzed for lift stations where the lift station capacity was unknown due to insufficient data.

Piping was initially flagged as deficient if the d/D was greater than 1 and the freeboard was less than 6 feet. The original idea was that some surcharging (pipes with water level above the top of the pipe) could be allowed if the water level did not get too close to the surface. However, analyzing for freeboard was found to be impractical. For example, Figure C-2 shows a pipeline with four pipe segments. Using the 6-foot freeboard criterion, Pipes 2 and 3 would be flagged as deficient. However, even though Pipe 4 would not be flagged as deficient with the 6-foot freeboard criteria, the pipe would also need fixed because it backs flow up into Pipe 3. In other situations, using the 6-foot freeboard criterion was even more complicated. Therefore, the idea of using freeboard in identifying deficiencies was abandoned and the criteria in Table C-1 were used.

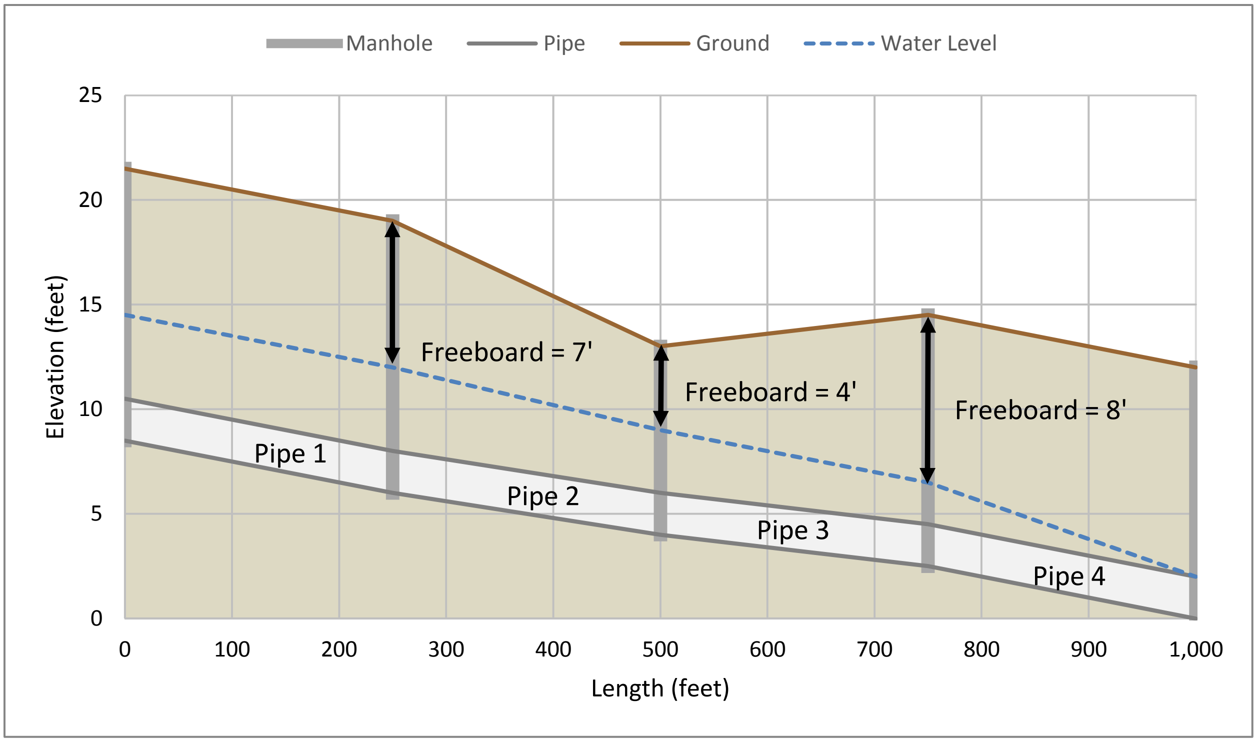

Figure C-2. Example Freeboard

Lift Station Criteria

The criteria in Table C-2 were used for lift stations.

Table C-2. Lift Station Criteria	
Item	**Value**
Minimum Capacity (with largest pump on standby)	Each lift station was analyzed to see if it could convey peak flows with the largest pump on standby.
Redundancy / Reliability	Each lift station should have a minimum of 2 pumps.
Number and Size of Pumps	Dry weather and peak wet weather flows may vary greatly, so the size and number of pumps were analyzed to ensure the flows are pumped efficiently for the varying flows with the largest pump on standby.

Design Storm

A design storm was used in the evaluation of the collection system. A design storm is a synthetic storm created for a specific storm frequency, rainfall distribution, storm duration, and total rainfall depth. A design storm is added to the calibrated model to generate peak wet weather flows throughout the collection system. The following sources were used to develop design storms:

- National Oceanic and Atmospheric Administration (NOAA) Atlas 14 – NOAA studied historical rainfall data for selected Pacific Islands to calculate total rainfall depths and rainfall distribution patterns for specific storm durations. The data was published in the NOAA Atlas 14 (NOAA 2011).

- United States Department of Agriculture (USDA) and Natural Resources Conservation Services (NRCS) 2008 report – USDA and NRCS published a report that includes design rainfall distributions for selected Pacific Islands including Guam (USDA, 2008). The study included rainfall data from four different stations in Guam with 14 to 27 years of record and elevations ranging from 10 to 110 feet above sea level.

The following sections describe each aspect of a design storm event.

Storm Frequency

A 2-year storm was used to evaluate the collection system and identify deficiencies. Improvements developed for the deficiencies were sized to handle the 2-year storm.

The system was initially analyzed using a 5-year storm. However, a 5-year storm identified many more deficiencies than are practical for GWA to fix. The rainfall depths for the 5-year storm are also relatively high (discussed below), with depths reflecting a typhoon event. Therefore, the criteria listed above were used for the storm frequency.

Rainfall Distribution and Duration

Rainfall distribution refers to the change in rainfall intensity during a storm, which can vary greatly during a storm. Many storms have a large burst of rainfall that occurs during a short period during a storm. The 2008 USDA/NRCS study provided rainfall distributions for 2 and 5-year rainfall frequencies for the 4 rain gauges. The report only provided distributions for 24-hour durations, therefore other durations (e.g. 6 or 12 hour) were not considered for the design storm. The distributions for the 4 rain gauges were compared and they primarily varied in the timing of rainfall before the peak intensities, but they all had very similar peak intensities. After reviewing the rainfall distributions, the distributions from the Piti rain gauge were used for the entire island. The Piti rain gauge had the longest period of available rainfall data, it is centrally located in Guam, and the other rainfall distributions had unusual patterns in the rainfall intensity leading up to the peak intensity. The rainfall distributions for the 2-year and 5-year events are shown in Figure C-3.

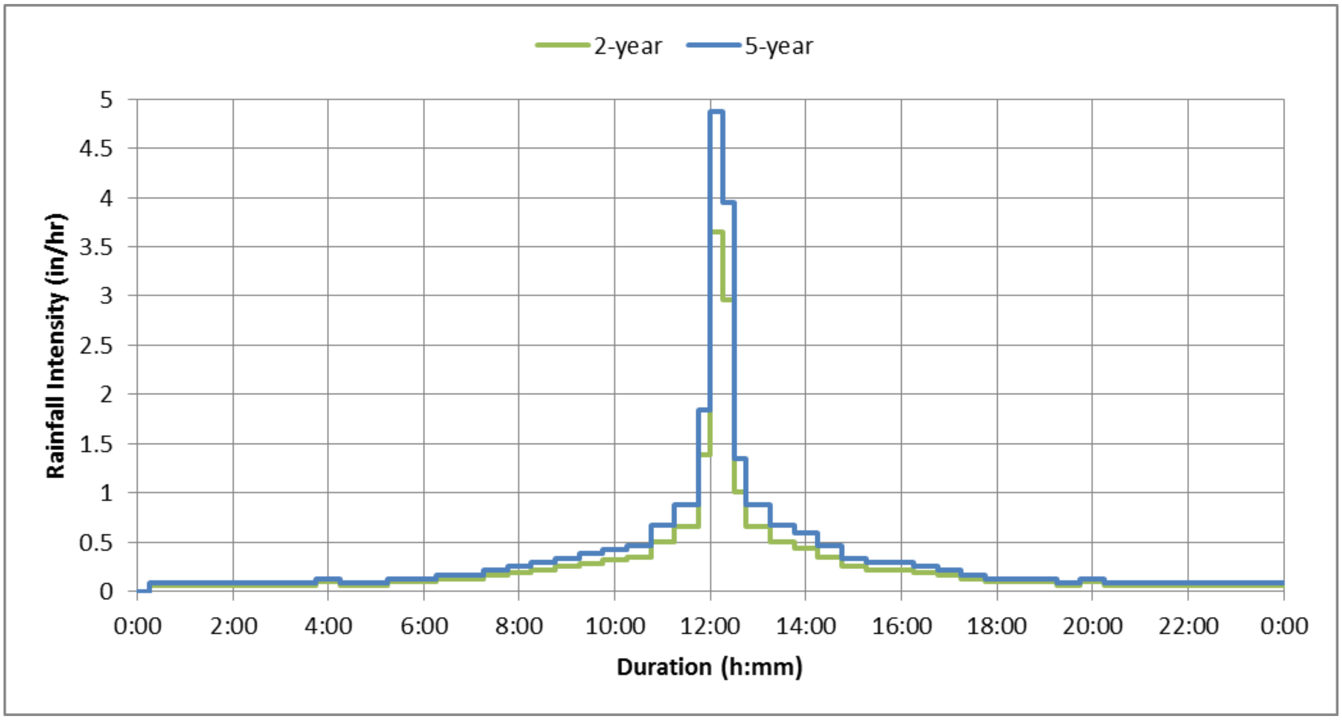

Figure C-3. Rainfall distribution for 2-year (green) and 5-year (blue) events

The selected rainfall distributions were compared to observed rainfall data and to rainfall distributions in the NOAA Atlas 14 (NOAA 2011). The NOAA distributions underestimate peak rainfall intensities when compared to observed rainfall. Therefore, the USDA/NRCS rainfall distributions shown in Figure C-3 were used.

Rainfall Depth

Rainfall depths were downloaded from the NOAA Atlas 14 website for 2-year and 5-year storms. Depths were only downloaded for storms with a 24-hour duration because rainfall distributions are only available for 24-hour durations as described above. The rainfall depths were downloaded for several locations throughout Guam because rainfall depth varies across the island. Table C-3 shows the storm depths used for the design storms.

Table C-3. NOAA 24-Hour Rainfall Depths

Basin	24-Hour Rainfall Depth (inches)	
	2-year Storm	5-year Storm
Northern District	6.5	8.7
Tumon	6.2	8.4
Hagåtña	6.5	8.7
Agat-Santa Rita	5.1	7.3
Umatac-Merizo	6.0	8.2
Baza Gardens	7.3	9.4
Inarajan	5.4	7.6

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Appendix D

Pipe Condition Assessment Data

This appendix contains background data used in the condition assessment of the wastewater facilities.

Data Used in Wastewater System Analysis

To support the condition assessment of the sewer system pipe network, the following information was gathered:

- GWA GIS data included the following datasets:
  - Sewer pipe data
  - Municipal/Village boundaries
  - Customer meter accounts and critical customers
  - Digital orthophotography of the service area
- Federal data included the following datasets:
  - TIGER roads
  - Census population
  - Place locations (hospitals, schools, churches)
- NOAA included the following datasets:
  - LIDAR elevation data
  - Landcover data
- Google Earth included the following datasets:
  - Hotel locations
  - Locations of potential critical customers

The following gaps in the available data were identified during the analysis:

- Unknown Installation Dates – Approximately 26% of the sewer pipes do not have an installation date noted in the GIS database. The date of installation of a sewer pipe is an important piece of information for determining a pipe's remaining service life. During the analysis performed and documented in this report an assumption was made to apply the median age of the system, identified as 1980, to pipes with an unknown installation date within the GWA GIS database.

Likelihood of Failure Factors

Table D-1 lists the scoring breakdown for the likelihood of failure factors.

Table D-1. Likelihood of Failure Factors	
Input Value	Score
P2, Soils	
Unknown, Agfayan-Akina association, Agfayan-Rock outcrop complex, Akina-Atate silty clays,	1
Akina-Agfayan association, Akina-Atate association, Akina-Badland association, Akina-Badland complex, Akina-Urban land complex, Guam cobbly clay loam, Guam-Saipan complex, Guam-Urban land complex, Guam-Yigo complex, Inarajan sandy clay loam, Pulantat-Urban land complex, Ritidian-Rock outcrop complex, Rock outcrop-Ritidian complex, Togcha-Ylig complex, Urban land-Ustorthents complex	3
Akina silty clay, Akina-Atate silty clays, Shioya loamy sand, Togcha-Akina silty clays	4
Agfayan clay, Chacha clay, Chacha variant clay, Inarajan clay, Inarajan variant mucky clay, Pulantat clay, Pulantat-Chacha clays, Pulantat-Kagman clays, Sasalaguan clay, Ylig clay	5
P3, CCTV or other Condition Record Data	
CCTV Score = 0 to 1.9	1
CCTV Score = 1.9 to 2.9	2
CCTV Score = 2.9 to 3.9	3
CCTV Score = 3.9 to 4.9	4
CCTV Score = 4.9 to 5.2	5
P4, Pipe Installation or Lining Year	
<= 1970	5
1971 through 1975	4
1976 through 1990	3
1991 through 2000	2
>= 2001	1
P5, Material	
CIP, CIPC	1
Null, DIP, PCP, PEP, RCP, TCP, VCP	3
ACP, PCV, UNK	5
P11, Depth	
Unknown or between 5 and 15 feet	1
Depth < 5 feet	3
Depth > 15 feet	5

CCTV Scoring

Figure D-1 shows the CCTV scores of 1 to 5 assigned to the gravity piping.

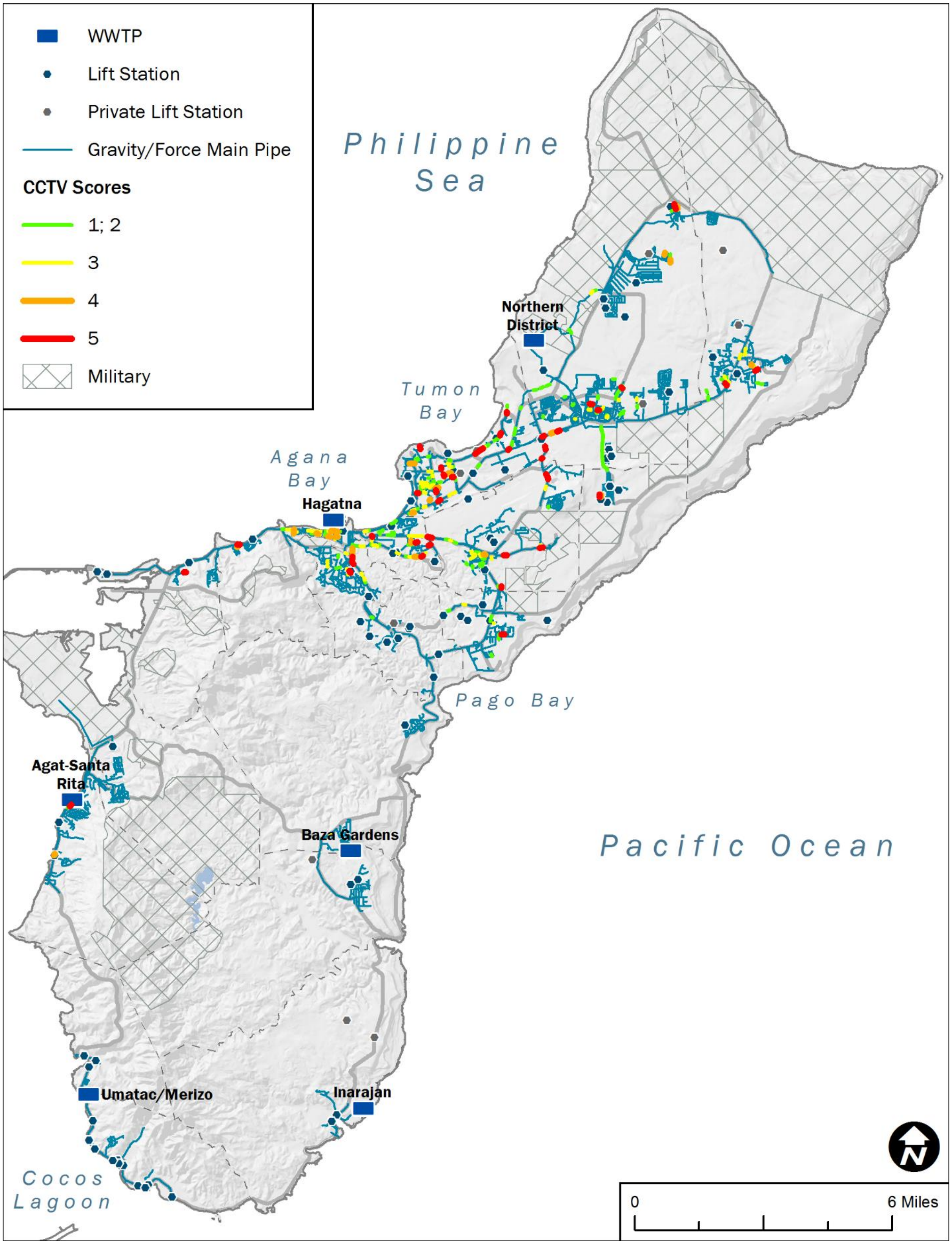

Figure D-1. CCTV Scores

As discussed in Section 4, some of the data collected in the CCTV surveys was not collected in a PACP format due to issues GWA had with the CCTV software. Where possible, the CCTV data that was not in a PACP format was modified to fit a PACP format. The non-PACP data was stored in spreadsheets. About 800 spreadsheets were reviewed. The comments in the spreadsheets were summarized and grouped into the comments and modifiers listed in Table D-2. The comments and modifiers were then aligned with the PACP codes and scores listed in the table.

Table D-2. Conversion of GWA non-PACP Codes to PACP Codes

GWA Codes		Estimated PACP Codes	
Comment	Modifier(s)	Equivalent Code(s)	Grades Based on Structure Condition
blocked trans	unable to proceed transporter tracks falling in to lateral connection unable to continue from upstream manhole. manhole buried	OBZ	2
broken pipe	prior repairs made	B, RP	4
broken pipe	ground soil visible; from 7-3; repairs required	BSV or HSV	5
camera under water	possible sag in line	MCU, MWLS	4, 4
Channel	Invert and Crown Structure Diameter Is too Small for 6" Camera to Pass Through	OBS	3
crack in pipe	crack in line unable to proceed	FM (or B)	4
DEBRI	More jetting required heavy grease	OBZ, DAGS	5
Debris	Large Rock in Main Line More jetting required	OBR	3
Debris	debris in line further jetting required debris in line more jetting required More Jetting Required	OBZ	2
debris in line	heavy; more jetting required.	OBZ	2
debris& sag in line	jetting to commence	OBZ, MWLS	3, 2
Grease	heavy grease build -up in line	DAGS	2
hole in pipe	small; before LC; ground soil visible; immediate attention not required; future repairs to be made small; due to location of hole (invert) immediate attention required; ground soil visible; exfiltration occurring.	HSV	5
hole in pipe @ 12	small; ground soil visible	HSV	5
infiltration @ 7	from joint connection	IGJ	5
joint gasket exposed	small missing piece of pipe	B	3
joint off-set	light exfiltration	JOM	1
Liner Failure	Lining is blistering.	LFB	3
liner failure	inner liner missing 12o'clock inner liner missing; 10-2o'clock inner liner missing; 9-2o'clock	LFD	3
Lining failure	Blistering	LFB	
Lining failure	blistering & peeling	LFB, LFDL	
Lining failure	Peeling	LFDL	3
off set joint	ground soil visible; repairs may be required	JOLD	4
offset joint connection	ground soil visible; possible exfiltration	JOL	2

Table D-2. Conversion of GWA non-PACP Codes to PACP Codes			
GWA Codes		Estimated PACP Codes	
Comment	Modifier(s)	Equivalent Code(s)	Grades Based on Structure Condition
Pending		MSA	
Pipe	Offset Joint Cannot Proceed Up Stream 4" Joint Difference Also Cannot Survey from Up Stream Due to Turn of Channel	JOM (or MSC?)	1
Pipe	pipe Blistering from Start	LFB	3
Pipe	Pipe Material Change from ACP to CLAY	MMC (AC to CLC)	
pipe continues	change of direction to the right; no manhole.	LR	4
pipe puncture	small; a rock visible	HSV	5
puncture in pipe	small; minor; ground soil not visible	H	3
roots @ LC	medium;	RMC	3
roots in pipe	medium;	RMB	4
Sag	sag from 4.6 feet to 12.7 feet at 50%	MCU, MWLS	4, 4
Sag	End of Sag Sag in Line About 40% sag in main line about 40%	MWLS	3
sag in line	cause of grease build up.	MWLS, DAGS	2, 5
sag in line	camera will be submerged severe; from the 6" LC a steep drop detected; camera will be submerged.	MWLS, MCU	4, 4
TD	MORE JETTING REQUIRED	OBZ	2

Consequence of Failure Factors

Table D-3 lists the scoring breakdown for the consequence of failure factors.

Table D-3. Consequence of Failure Factors	
Input Value	Score
C1, Damage or Disruption to Sensitive Locations	
Distance > 400 feet from sensitive location	1
Distance =301 to 400 feet from sensitive location	2
Distance =201 to 300 feet from sensitive location	3
Distance =101 to 200 feet from sensitive location	4
Distance =0 to 100 feet from sensitive location	5
C3, Damage or Disruption to Roadways	
Pipes not near a roadway	1
<= 50 feet from minor roadway	3
<= 50 feet from major roadway	5
C7, Service Outage – Number of Customers	

Table D-3. Consequence of Failure Factors	
Input Value	Score
Serves no customers	1
Serves 1 to 6 customers	2
Serves 7 to 12 customers	3
Serves 13 to 21 customers	4
Serves more than 21 customers	5
C12, Flooding Potential – Model Flows	
Flow = 0 mgd	1
Flow < = 1 mgd	2
Flow > 1 and <= 10 mgd	3
Flow > 10 and <= 50 mgd	4
Flow > 50 mgd	5
C16, General Disruption to Life – Population Density	
Population density <= 250 persons per square mile	1
Population density = 251 – 500 persons per square mile	2
Population density = 501 – 750 persons per square mile	3
Population density = 751 – 1,500 persons per square mile	4
Population density > 1,500 persons per square mile	5

Manhole Inspection Reports

Approximately 615 recent manhole inspection assessments were provided by GWA, all located in the Hagåtña basin. A summary of the assessments is shown in the table in the following pages. Of the 615 manholes assessments, 577 of the manholes could be found in the GIS based on the manhole ID.

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations	Can Not Locate (CNL)	Can Not Access (CNA)
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)			
Area 1 - Chalan Pago / Ordot																										
1485ACPO	16.9	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
1485CPO	18.1	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/2	N	1	N	N			
1486CPO	9.7	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
1490CPO	5.8	A	S	32	N	1	1	1	-	N	2	1	2	-	N	1	1	N	1/4	N	1	N	N	FC (circumferenntial fractures), CL (longitudinal crack) on the chimney interior		
1491CPO	7.6	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
1493CPO	4.7	A	S	32	N	2	1	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
1518CPO	18.1	G	S	32	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
1519CPO	12.4	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/2	N	1	N	N			
1520CPO	5.7	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
1521CPO	5.6	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
1521ACPO	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
1529CPO	4.6	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	L	N			
1530CPO	4.9	A	S	32	N	2	1	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
1531CPO	6.2	A	S	32	N	2	1	2	-	N	2	1	2	-	N	1	1	N	NF	N	1	N	N	CM (cracks multiple) on the chimney interior		
1532CPO	5.3	A	S	32	N	2	1	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
1533CPO	7.3	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
1534CPO	4.9	A	S	32	N	2	2	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
Area 2 - Sinajana																										
425H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
426H_H	9.7	A	S	26	N	2	1	2	-	N	1	3	3	-	Y	1	1	N	F	N	1	N	N	MM (missing Mortar) on the chimney interior, **Repair required**		
430H_H	3.6	G	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
431H_H	3.5	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
432H_H	6.2	A	S	26	N	1	1	1	-	N	2	1	2	-	N	1	1	N	3/4	N	1	N	N	SAV (surface aggregate visible) on the chimney interior. SRV (surface reinforcement visible) on the wall interior.		

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

	General			Cover and Frame							Cone and Riser					Bench and Channel			Other							
Manhole No.	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)	Comments and Other Observations	Can Not Locate (CNL)	Can Not Access (CNA)
1003AH_H	9.8	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	Not in the original list. New found manhole.		
1004H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
4344Sin	10.3	A	S	24	N	2	1	2	-	N	1	1	1	2 DI	N	1	1	N	1/4	N	1	N	N			
4345Sin	9.2	A	S	24	N	1	1	1	-	N	1	1	1	2 DI	N	1	1	N	NF	N	1	N	N			
4346Sin	11.6	A	S	24	N	1	1	1	-	N	1	1	1	2 DI	N	1	1	N	NF	N	1	N	N			
4348Sin	5.6	A	S	24	N	1	1	1	-	N	1	1	1	2 DI	N	1	1	N	1/2	N	1	N	N			
4349Sin	8.7	A	S	24	N	1	1	1	-	N	1	1	1	1 DI	N	1	1	N	F	N	1	N	N	MGO (ring is chipped)		
4350Sin	5.7	A	S	24	N	2	1	2	-	N	2	1	2	2 DI	N	1	1	N	F	N	1	N	N	FM (fracture multiple) on the chimney interior		
4370Sin	6.6	A	S	24	N	1	1	1	-	N	1	1	1	2 DI	N	1	1	N	1/2	N	1	N	N			
7589Sin	5.6	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
7590Sin	5.2	A	S	24	N	2	1	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
7927Sin	10.8	A	S	24	N	1	1	1	-	N	1	1	1	2 DI	N	1	1	N	1/4	N	1	N	N			
7928Sin	9.5	A	S	24	N	1	1	1	-	N	1	1	1	2 DI	N	1	1	N	1/4	N	1	N	N			
Area 3 - Mongmong / Toto / Maite																										
194MTM	5.7	A	S	32	N	1	2	2	-	N	2	1	2	-	N	1	1	N	1/4	N	1	N	N	CC (circumfrential crack) on the chimney interior		
195MTM	7.8	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
196MTM	6.6	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
197MTM	6.6	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
198MTM	4.8	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
199MTM	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
200MTM	5.1	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
201MTM	4.7	A	S	30	N	1	1	1	-	N	2	1	2	-	N	1	1	N	1/8	N	1	N	N	FC (fracture circumferential) on the chimney interior		
202MTM	5.5	A	S	30	N	2	2	2	-	N	2	1	2	-	N	1	1	N	NF	N	1	N	N	FC (fracture circumferential) on the chimney interior		
204MTM	9	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	3	N	NF	N	1	N	N	DSGV (deposits gravel) in channel, Cleaning required		

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations		
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)		Can Not Locate (CNL)	Can Not Access (CNA)
205MTM	8.3	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	1/2	N	1	N	N			
206MTM	4.4	A	S	32	N	1	2	2	-	N	1	1	1	-	N	3	1	N	NF	N	1	N	N	MGO (pipe and rubber debris) in bench, Cleaning required		
207MTM	5.9	A	S	32	N	1	2	2	-	N	2	1	2	-	N	1	1	N	1/4	N	1	N	N	FC (fracture circumferencial) in chimney interior		
208MTM	6.4	A	S	32	N	3	2	3	-	N	1	3	3	-	N	1	1	N	1/4	N	1	N	N	SRV (Surface reinforcement visible) in wall interior		
209MTM	9.6	A	S	24	N	1	3	3	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
210MTM	-	-	-	-	N	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
211MTM	5.2	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
212MTM	5.6	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
6453MTM	10.5	S	S	32	N	2	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N	DSGV (deposits gravel @ 10.5' on the channel)		
6496MTM	4.5	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	-	N	S	N	1	N	N	DSGV (deposits gravel) in channel, **Cleaning required**		
6497MTM	10.5	A	S	24	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
6498MTM	7.6	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
6499MTM	10.5	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
6500MTM	10.8	A	S	24	N	3	3	3	-	N	1	1	1	-	N	-	-	-	S	N	1	N	N	Pump Station Wet Well		
6502MTM	14.6	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
6503MTM	6.3	A	S	24	N	2	1	2	-	N	1	1	1	-	N	1	4	N	S	N	1	N	N	DAGS (deposits grease) in channel, **Cleaning required**		
6504MTM	7.1	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
6505MTM	5.8	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
6506MTM	9	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
6510AMTM	9.3	A	S	30	N	3	2	3	-	N	1	1	1	-	N	1	1	N	3/4	N	1	N	N			
6510MTM	5.1	A	S	24	N	3	3	3	-	Y	1	3	3	-	Y	1	2	N	F	N	1	N	N	FL (fracture longitudinal) on the chimney interior, MGO (chimney separated from cone), DSGV (deposits grease) in channel. **Need repair.**		
6511MTM	-	-	-	-	N	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
6512MTM	7.2	A	S	24	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			

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Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations		
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)		Can Not Locate (CNL)	Can Not Access (CNA)
6513MTM	4.6	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	3	N	S	N	1	N	N	DSGV (deposits gravel) in channel, **Cleaning required**		
6983MTM	7.7	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
6993MTM	8	S	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
6994MTM	9.5	A	S	24	N	1	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
6995MTM	8.8	A	S	24	N	1	2	2	-	N	1	1	1	-	N	1	1	N	1/2	N	1	N	N			
6996MTM	7.3	A	S	24	N	1	2	2	-	N	2	1	2	-	N	3	1	Y	1/4	N	1	N	N	SRV (surface reinforcement visible) on the cone interior , H (hole in bench), **Repair on the bench required**		
6997MTM	7.7	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	3/4	N	1	N	N			
7001MTM	7.2	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
7944MTM	9	G	S	24	N	1	1	1	-	N	4	1	4	-	Y	1	1	N	F	N	1	N	N	MGO (frame not attached), **Need repair**		
7946DBarr	11	G	S	26	N	2	2	2	-	N	5	1	5	-	Y	3	1	Y	F	N	1	N	N	H (hole @ 6" on the chimney interior), OBR (chimney debris @ 10.6' on the bench), **Need repair, Bench cleaning required.**		
8556AMTM	5.1	A	S	24	N	2	1	2	-	N	1	1	1	3 DI	N	1	1	N	2/3	N	1	N	N			
8556BMTM	6.9	A	S	24	N	2	1	2	-	N	1	1	1	2 DI	N	1	1	N	F	N	1	N	N			
8556CMTM	6.2	A	S	24	N	2	1	2	-	N	1	1	1	2 DI	N	1	1	N	F	N	1	N	N			
8556DMTM	8.2	A	S	24	N	2	2	2	-	N	1	1	1	2 DI	N	1	1	N	1/4	N	1	N	N			
8556EMTM	8	A	S	24	N	1	1	1	-	N	1	1	1	2 DI	N	1	1	N	1/8	N	1	N	N			
10415MTM	8.3	A	S	25	N	1	1	1	-	N	1	3	3	-	Y	1	1	N	F	N	1	N	N	MM (missing mortar) on the chimney interior, **Repair required**		
10416MTM	9.4	A	S	25	N	2	2	2	-	N	1	3	3	-	Y	1	1	N	F	N	1	N	N	MM (missing mortar) on the chimney interior, **Repair required**		
10417MTM	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
10418MTM	5.1	A	S	25	N	1	2	2	-	N	1	1	1	-	N	1	1	N	3/4	N	1	N	N			
10426MTM	6.9	A	S	25	N	1	2	2	-	N	1	3	3	-	Y	1	3	Y	NF	N	1	N	N	MM (missing mortat) on the chimney interior, DSGV (deposits gravel) in channel. Chimney interior repair required, **Repair and channel cleaning required.**		
10427MTM	4.7	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
10428MTM	4.1	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
10429MTM	3.7	A	S	24	N	2	2	2	-	N	2	1	2	-	N	1	1	N	S	N	1	N	Y	H (hole) on the chimney interior		

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations	Can Not Locate (CNL)	Can Not Access (CNA)
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)			
10430MTM	13.3	G	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	S	N	1	N	Y 1.6'	CS (crack spiral on the wall interior)		
10431MTM	19.5	C	O	27	N	2	-	-	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N	Lift station, MGO (pipe is open with no cap @ 1.2' on the chimney interior)		
Area 4 - Tamuning																										
648Tam	3.4	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/2	N	1	N	N			
649Tam	5	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
650Tam	2.7	S	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N			
651Tam	3.8	A	S	26	N	1	1	1	-	N	2	1	2	-	N	1	1	N	1/2	N	1	N	N			
652Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
653Tam	3.9	A	S	25	N	2	2	2	-	N	1	3	3	-	N	1	1	N	F	N	1	N	N	FC (fracture circumferential @ 0.8' on the chimney interior)		
654Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
655Tam	4.6	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 0.8' on the chimney interior)		
656Tam	4.1	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
657Tam	3.3	A	S	26	N	1	3	3	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	Need more photos to identify defects on the top of the chimney		
658Tam	3.9	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
1146Tam	4.7	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior)		
1147Tam	5.8	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
1148Tam	5.8	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
1149Tam	5.5	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
1150Tam	5.6	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
1151Tam	3.8	A	S	26	N	1	1	1	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
1152Tam	3.9	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
1153Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
1154Tam	3.4	A	S	26	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
1155Tam	5.5	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			

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Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations	Can Not Locate (CNL)	Can Not Access (CNA)
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)			
Area 5 - Sinajana / Hagatna																										
394Haga	11.9	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior)		
491Haga	8.7	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MGO (wood nalled to chimney @ 1.9' on the chimney interior), OBR (obstacle rocks @ 8' on the channel)		
492Haga	7.8	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
493Haga	8	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
494Haga	11.1	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 7" on the chimney interior)		
842Haga	6.3	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
935Haga	11.8	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior)		
936Haga	11.8	A	S	25	N	2	2	2	-	N	2	1	2	-	Y	1	1	N	F	Y	2	N	N	MM (missing mortar @ 7" on the chimney interior), ID (infiltration dripper @ 8.5' on the wall interior)		
937Haga	12.1	A	S	25	N	1	2	2	-	N	1	1	1	-	Y	1	1	N	F	Y	2	N	N	IS (infiltration stain @ 8.6' on the wall interior), IW (infiltration weeper @ 9.8' on the wall exterior)		
938Haga	11.9	A	S	25	N	1	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
939Haga	11.6	G	S	25	N	2	2	2	-	N	4	1	4	-	Y	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior), JOM (medium offset frame joint). **Need repair**		
940Haga	9.3	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
941Haga	6.1	A	S	24	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
942Haga	10	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
943Sin	5.4	A	S	25	N	3	1	3	-	N	1	3	3	-	N	1	1	N	F	N	1	N	N	FC (fracture circumferential @ 1.4' on the chimney interior)		
944Sin	4.7	A	S	25	N	3	2	3	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
945Haga	4.1	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
946Sin	5	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
947Sin	5.2	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	FC (fracture circumferential @ 1.8' on the chimney interior)		
948Sin	5.4	A	S	24	N	2	2	2	-	N	1	3	3	-	N	1	1	N	F	N	1	N	N	SCP (surface corrosion @ 11" on the chimney interior), MB (missing brick @ 1.8' on the chimney interior)		
949ASin	10.5	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 9.6' on the chimmney interior)		
949Sin	6	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N			

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Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations		
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)		Can Not Locate (CNL)	Can Not Access (CNA)
950Sin	13.7	A	S	25	N	2	3	3	-	N	2	2	2	-	N	1	1	N	3/4	N	1	N	N	MM (missing mortar @ 8" on the chimney interior), SRI (surface roughnss-increased @ 4.8' on the wall interior)		
951Sin	5.1	A	S	25	N	2	3	3	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
952Sin	4.6	A	S	25	N	2	3	3	-	N	2	2	2	-	N	2	1	N	F	N	1	N	N	FC (fracture circumferential @ 1.1' on the chimney interior), DAZ (unknown deposits @ 1.9' on the wall interior), DSGV (deposits gravel @ 3.8' on the bench)		
953Sin	8.8	A	S	25	N	2	3	3	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 7" on the chimney interior)		
954Sin	6.4	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	3/4	N	1	N	N			
955ASin	5.4	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
955Sin	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	Cement on cover		CNA
956Sin	10.3	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	1/2	N	1	N	N	MM (missing mortar @ 1.5' on the chimney interior)		
957Sin	12.9	A	S	25	N	2	2	2	-	N	4	1	4	-	Y	1	1	N	1/2	N	1	N	N	FM (fracture multiple @ 1.5' on the chimney interior), **Need repair**		
958Sin	13.4	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	2	N	F	N	1	N	N	MGO (wood object intruding @ 11" on the chimney interior), DSGV (deposits gravel @ 13.4' on the channel)		
6885Sin	5.2	A	S	33	N	2	2	2	-	N	2	1	2	-	N	1	1	N	1/8	N	1	N	N	MM (missing mortar @ 11" on the chimney interior)		
6886Sin	8.8	A	S	33	N	2	2	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
6887Sin	5.3	A	S	33	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 11" on the chimney interior)		
6888Sin	5.2	A	S	33	N	2	2	2	-	N	1	2	2	-	N	1	1	N	F	N	1	N	N	SSS (surface spalling @ 4.3' on the wall interior)		
6889Sin	5.8	A	S	32	N	2	3	3	-	N	2	1	2	-	N	1	1	N	1/8	N	1	N	N	MM (missing mortar @ 9" on the chimney interior), SRI (surface roughness increased @ 1.7' on the cone interior)		
6890Sin	5.4	A	S	33	N	2	2	2	-	N	2	1	2	-	N	1	1	N	1/8	N	1	N	N	MB (missing brick @ 11" on the chimney interior), FM (fracture multiple @ 1.7' on the chimney interior)		
6891Haga	6.2	A	S	33	N	2	2	2	-	N	2	1	2	-	N	1	1	N	1/8	N	1	N	N	MB (missing brick @ 9" on the chimney interior)		
6892Haga	6.3	A	S	32	N	2	3	3	-	N	2	1	2	-	N	1	1	N	1/8	N	1	N	N	FC (fracture circumferential @ 1.6' on the chimney interior)		
6893Haga	5.2	A	S	33	N	2	2	2	-	N	2	1	2	-	N	1	1	N	1/2	N	1	N	N			
6894Haga	5.2	A	S	32	N	2	2	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
6895AHaga	7.9	A	S	32	N	2	2	2	-	N	2	1	2	-	N	1	1	N	1/4	N	1	N	N	MM (missing mortar @ 7" on the chimney interior)		
6895Haga	5.1	A	S	33	N	2	2	2	-	N	1	1	1	-	N	1	1	N	1/2	N	1	N	N			
6896Haga	9.2	A	S	33	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior)		

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations	Can Not Locate (CNL)	Can Not Access (CNA)
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)			
6907Haga	14.5	A	S	32	N	2	1	2	-	N	3	1	3	-	N	1	1	N	F	N	1	N	N	H (hole @ 1.2' on the chimney interior), SRV (surface reinforcement- visible @ 1.3' on the chimney interior)		
6908Haga	12.8	A	S	32	N	2	1	2	-	N	3	1	3	-	N	1	1	N	F	N	1	N	N			
Area 6 - Adelup / Hagatna / Tamuning																										
13Asan	5.5	G	S	25	N	3	3	3	-	N	1	2	2	-	N	1	1	N	S	N	1	N	Y 3.3'	FC (fracture circumferential on the wall interior)		
14Asan	7.1	A	S	25.5	N	2	2	2	-	N	1	1	1	-	N	1	1	N	S	N	1	N	Y 4.8'			
15Asan	7.5	A	S	25.5	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	FM (multiple fractures on concrete collar), DAZ (mud on the chimney interior @ 7", Defect Wanders on the chimney interior @ 2.2')		
16Asan	7.4	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	FM (multiple fractures on concrete collar), DAZ (mud on the chimney interior @ 7")		
17Asan	7.6	A	S	25.5	N	3	2	3	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
180Haga	13.7	A	S	33	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 1.5' on the chimney interior)		
181AHaga	13	A	S	32	N	2	1	2	-	N	2	1	2	-	Y	1	1	N	F	Y	4	N	N	CM (crack multiple @ 8" on the chimmney interior), IG (infiltration gusher @ 10.8' on the wall exterior), **Need to be repaired**		
181Haga	13.1	A	S	32	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
182Haga	-	-	-	-	N	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
183Haga	12.3	A	S	32	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	CM (crack multiple @ 2.2' on the chimney interior)		
184Haga	12.5	A	S	32	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	CM (crack multiple @ 1.3' on the chimney interior)		
190Tam	15	G	O	31	N	1	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	Y 2'	MH cover is a circular steel place.		
191Tam	13.6	A	S	32	N	2	1	2	-	N	1	1	1	-	N	1	2	N	F	N	1	N	N	MGO (plywood in channel @ 13.5' on the channel)		
193Tam	12.2	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	2	N	F	N	1	N	N	DAR (deposits attached - ragging @ 10.9' on the channel)		
323Haga	13	A	S	25	N	3	3	3	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
324Haga	12.7	A	S	25	N	2	4	4	-	Y	1	1	1	-	N	1	1	N	F	N	1	N	N	5" frame offset joint, **Need repair**		
326AHaga	10.7	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
326Haga	10.5	A	S	25	N	3	2	3	-	N	1	2	2	-	Y	1	1	N	F	Y	2	N	N	MM (missing mortar @ 8" on the chimney interior), IW (infiltration weeper @ 9.1' on the wall interior)		
327Haga	11.4	A	S	25	N	2	3	3	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
328Haga	11.9	A	S	25	N	2	2	2	-	N	1	2	2	-	Y	1	1	N	F	Y	2	N	N	IW (Infiltration weeper @ 8.9' on the wall interior), H (hole on wall @ 8.5' on the wall interior)		
329Haga	12.3	A	S	25	N	2	2	2	-	N	2	2	2	-	Y	1	1	N	F	Y	2	N	N	MM (missing mortar @ 8" on the chimney interior), IW (infiltration weeper @ 11.5' on the wall interior)		
330Haga	5.3	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
331Haga	10	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
332Haga	9.9	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 7" on the chimney interior)		
333Haga	10	A	S	25	N	3	2	3	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
335Haga	9.9	A	S	25	N	4	2	4	-	Y	1	1	1	-	N	1	1	N	F	N	1	N	N	Corroded cover. **Need to be replaced.**		

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations		
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)		Can Not Locate (CNL)	Can Not Access (CNA)
336Haga	9.8	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	FC (fracture circumferential @ 1.4' in the chinmey interior)		
337Haga	9.4	A	S	25	N	2	2	2	-	N	1	2	2	-	N	1	1	N	F	N	1	N	N	H (hole in pipe @ 8.9' on the wall interior)		
338Haga	9.5	A	S	25	N	3	2	3	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
339Haga	9.5	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 4" in the chimney interior)		
340Haga	9.3	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
341Haga	9.1	A	S	25	N	2	2	2	-	N	1	2	2	-	Y	1	1	N	F	Y	2	N	N	DAZ (appears to be mud @ 7" in the chimney interior), IW (Infiltration weeper @ 6.9' on the wall interior)		
342Haga	9.5	A	S	25	N	1	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 7" in the chimney interior)		
343Haga	9.4	A	S	25	N	2	2	2	-	N	1	1	1	-	Y	1	1	N	F	Y	2	N	N	IW (Infiltration weeper @ 8.2' in the wall interior)		
344Haga	8.7	A	S	25.5	N	2	2	2	-	N	1	1	1	-	Y	1	1	N	F	Y	2	N	N	IW (Infiltration weeper @ 8.6' in the wall interior)		
345Haga	8.6	A	S	25	N	2	2	2	-	N	1	1	1	-	Y	1	1	N	F	Y	3	N	N	IR (Infiltration runner @ 7.3' in the wall interior)		
346Haga	8.9	A	S	25	N	2	2	2	-	N	1	1	1	-	Y	1	1	N	F	Y	2	N	N	DAZ (appears to be mud @ 7" in the chimney interior), IW (Infiltration weeper @ 7.5' on the wall interior)		
347Haga	9.1	A	S	25	N	2	2	2	-	N	1	1	1	-	Y	1	1	N	F	Y	2	N	N	IW (Infiltration weeper @ 6.9' in the wall interior)		
348Asan	9.5	A	S	25.5	N	2	2	2	-	N	2	2	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 5" in the chimney interior), FM (fracture multiple @ @7.3' on the wall interior)		
349Asan	9.6	A	S	25	N	2	2	2	-	N	2	2	2	-	N	1	1	N	F	N	1	N	N	MM (Missing mortar @ 10" in the chimney interior), SAV (surface aggregate visible @ 10" & 10.9' on the chimney and wall interior)		
350AHaga	10.5	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	2	N	F	N	1	N	N	DAR (deposits attached - ragging @ 10.5' on the channel)		
350BHaga	10.3	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAR (deposits attached - ragging @ 10.3' on the channel)		
350CHaga	10.1	A	S	25	N	1	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
350Haga	9.9	A	S	25.5	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 0.5" in the chimney interior)		
351Haga	10	A	S	25.5	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 7" in the chimney interior)		
352Haga	10.2	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 7" on the chimney interior)		
353Haga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	Manhole paved over		CNA
355Asan	7.6	A	S	25.5	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
356Asan	9.2	A	S	25.5	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 0.5" on the chimney interior)		
357Haga	5.1	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
372Haga	6.6	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 5" on the chimney interior)		
373Haga	5.3	A	S	25	N	1	2	2	-	N	2	1	2	-	N	1	2	N	NF	N	1	N	N	MM (missing mortar @ 7" on the chimney interior), DSGV (Deposits gravel @ 5.1' on the channel)		
374Haga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
375Haga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
376Haga	4.6	A	S	25	N	2	2	2	-	N	3	1	3	-	Y	1	1	N	1/4	N	1	N	N	MM (missing mortar @ 0.7" on the chimney interior) **Need Repair**		
377Haga	4.8	A	S	25	N	1	2	2	-	N	2	1	2	-	N	1	2	N	NF	N	1	N	N	MM (missing mortar @ 0.8"on the chimney interior), DSGV (deposits gravel @ 4.8' on the channel)		
378Haga	5.5	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	1/2	N	1	N	N	MM (missing mortar @ 0.6" on the chimney interior)		

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations		
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)		Can Not Locate (CNL)	Can Not Access (CNA)
379Haga	5.1	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	1/8	N	1	N	N	MM (missing mortar @ 0.7" on the chimney interior)		
380Haga	6.3	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	2/3	N	1	N	N	MM (missing mortar @ 0.7" on the chimney interior)		
381Haga	3.2	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	1/4	N	1	N	N	MM (missing mortar @ 0.7" on the chimney interior)		
382Haga	4.3	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	1/8	N	1	N	N	MM (missing mortar @ 0.6" on the chimney interior), SRV (surface reinforcement visible @ 1.2' on the chimney interior)		
383Haga	5.1	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
384Haga	4.7	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
385Haga	6.9	A	S	25	N	1	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N	DSZ (appears to be mud @ 6.2' in the channel)		
386Haga	7.8	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	FM (fracture multiple @ 10" on the chimney interior), DAR (deposits attached @ 7.4' in the channel)		
388Haga	5.9	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	3/4	N	1	N	N			
388AHaga	6.5	A	S	30	N	1	2	2	-	N	2	1	2	-	N	1	2	N	F	N	1	N	N	MB (missing bricks @ 11" on the chimney interior), DSGV (deposits gravel @ 6.3' in the channel)		
389Haga	6.9	A	S	25	N	1	1	1	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	FM (fractures multiple @ chimney interior), SRV (surface reinforcement - visible @ 1.9' on the chimney interior)		
390Haga	7.5	A	S	25	N	1	1	1	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	FL (fracture longitudinal @ 1' on the chimney interior), CC (circumferential crak @ 1.3' on the chimney interior), SRV (surface reinforcement - visible @ 1.8' on the chimney interior)		
391Haga	7.2	A	S	25	N	1	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
561Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
562Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
563Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
564Tam	3.9	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
565Tam	6.3	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
566Tam	6.2	A	S	22	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
887Tam	11	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 4" on the chimney interior)		
888Tam	11.9	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 3" on the chimney interior)		
889Tam	12	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
890Tam	12.2	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 3" on the chimney interior)		
891Tam	9.8	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 4" on the chimney interior)		
892Tam	8.3	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	CM (crack multiple @ 1.7' on the chimney interior)		
893Tam	8.4	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 5" on the chimney interior)		
894Tam	8.9	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 4" on the chimney interior)		
895Tam	8.5	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 7" on the chimney interior)		
896Tam	7.1	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 5" on the chimney interior)		
897Tam	7.7	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 8" on the chimney interior)		
898Tam	8	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 3" on the chimney interior)		
899Tam	8.2	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 5" on the chimney interior)		
900Tam	6.3	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N			

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Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations	Can Not Locate (CNL)	Can Not Access (CNA)
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)			
901Tam	7.7	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
902Tam	7.6	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 8" on the chimney interior)		
903Tam	8.5	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 4" on the chimney interior)		
904Tam	8.5	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 3" on the chimney interior)		
905Tam	7.6	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MGO (frame is damaged), DAZ (appears to be mud @ 5" on the chimney interior)		
906Tam	5.4	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
907Tam	5.7	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	FC (fracture circumferential @ 1.3' on the chimney interior)		
908Tam	9.5	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
909Tam	4.3	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAZ (deposits attached - appears to be mud @ 3" on the chimney interior)		
910Tam	5.9	A	S	25	N	2	1	2	-	N	1	1	1	-	N	3	1	N	F	N	1	N	N	OBR (ovstacle rocks @ 4.2' on the bench)		
911Tam	6	A	S	25	N	3	2	3	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 7" on the chimney interior)		
912Tam	7.25	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior)		
913Tam	6.9	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
914ATam	5	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 3" on the chimney interior), DSGV (deposits gravel @ 4.9' on the channel)		
914Tam	5.8	A	S	25	N	3	1	3	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 3" on the chimney interior)		
915Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-			CNA
916Tam	5.7	A	S	25	N	2	1	2	-	N	3	1	3	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 8" on the chimney interior), MM (missing mortar @ 8" on the chimney interior)		
917Tam	5.8	A	S	25	N	2	1	2	-	N	3	1	3	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 3" on the chimney interior), MM (missing mortar @ 7" on the chimney interior)		
918ATam	6.3	A	S	25	N	2	1	2	-	N	4	1	4	-	Y	1	1	N	F	N	1	N	N	MM (missing mortar @ 7" on the chimney interior), DB (displaced brick @ 1.4' on the chimney interior), **Need to repair**		
918Tam	6.3	A	S	25	N	2	1	2	-	N	4	1	4	-	Y	1	1	N	F	N	1	N	N	MM (missing mortar @ 7" on the chimney interior), MB (missing brick @ 9" on the chimney interior), DB (displace brick @ 9" on the chimney interior), **Need to repair**		
919Tam	6.8	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 3" on the chimney interior)		
920Tam	6.9	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior), DAZ (appears to be mud @ 8" on the chimney interior), FM (fracture multiple @ 1' on the chimney interior)		
921Tam	5.3	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior), DAZ (appears to be mud @ 8" on the chimney interior)		
922Tam	4.7	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 9" on the chimney interior), MM (missing mortar @ 1.1' on the chimney interior), SRV (surface reinforcement - visible @ 1.1' on the chimney interior)		
923Tam	-	-	-	-	N	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-			CNA
924Tam	5.7	A	S	25	N	3	1	3	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 7" on the chimney interior), DAZ (appears to be mud @ 7" on the chimney interior)		
925Tam	4.4	A	S	26	N	3	1	3	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior)		
926Tam	4.9	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 8" on the chimney interior), MM (missing mortar @ 8" on the chimney interior)		
927Tam	6.2	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior)		
928Tam	6.3	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior)		
929Tam	5.3	A	S	25	N	3	2	3	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 7" on the chimney interior)		
1066Tam	4.7	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
1076Tam	4.6	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	3/4	N	1	N	N			

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations	Can Not Locate (CNL)	Can Not Access (CNA)
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)			
1442Tam	-	-	-	-	N	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-			CNA
1443Tam	7.5	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	2	N	F	N	1	N	N	DSGV (deposits gravel @ 7.5' on the channel)		
1444Tam	5	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
1445Tam	6.5	G	S	25	N	2	2	2	-	N	4	1	4	-	Y	2	1	N	F	N	1	N	N	DSGV (deposits gravel @ 6' on the bench); JO (frame offset 7"). **Need Repair**		
1446Tam	4.5	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
1447Tam	6.6	A	S	25	N	2	1	2	-	N	1	1	1	-	N	4	1	N	F	N	1	Y	N	DAR (6" in of deposits - attached ragging @ 6.2' on the bench), need to clean the bench		
1448Tam	4.7	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
1449Tam	5.2	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	2	N	NF	N	1	N	N	DSGV (deposits gravel @ 5.2' on the channel)		
1450Tam	8.5	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	2	N	NF	N	1	N	N	DSGV (deposits gravel @ 8.5' on the channel)		
1451Tam	9.7	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	1/2	N	1	N	N			
1452Tam	3.1	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
1453Tam	4.5	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	4	N	3/4	N	1	N	N	DSGV (deposits gravel @ 4.5' on the channel), Needs cleaning		
1454Tam	6.7	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	4	N	3/4	N	1	N	N			
1455Haga	6.1	A	S	26	N	2	1	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
1456Haga	7.7	A	S	26	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
1458Tam	-	-	-	-	N	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
1459Tam	9.3	A	S	25	N	2	2	2	-	N	3	1	3	-	Y	1	1	N	F	Y	4	N	N	MB (missing brick @ 11" on the chimney interior), IG (infiltration gusher @ 8.5' on the wall interior), **Needs repair**		
1460Tam	11.8	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 1.3' on the chimney interior)		
1461Tam	2.9	A	S	25	N	2	2	2	-	N	3	1	3	-	N	1	1	N	NF	N	1	N	N	SSS (surface spalling @ 1.6' on the chimney interior)		
6533Tam	6.2	A	S	32	N	2	2	2	-	N	1	3	3	-	N	1	1	N	F	N	1	N	N	H (hole @ 4.2' on the wall interior)		
6921MTM	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
6922Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
6923Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
6924Tam	6	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	NF	N	1	N	N	CC (crack circumferential @ 1.6' on the chimney interior)		
7842Tam	12.1	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 4" on the chimney interior)		
7843Tam	12.1	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 4" on the chimney interior), H (hole @ 11" on the chimney interior), H (hole @ 11" on the chimney interior)		
8900Haga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
8901Haga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
8902Haga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
8903Haga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
8904Haga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
8905Haga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
8906Haga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
8907Haga	8.4	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	2	N	F	N	1	N	N	MM (missing mortar @ 1' on the chimney interior), DSF (deposits settled - fine @ 8.2' on the channel)		
8908Haga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
8909Haga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations	Can Not Locate (CNL)	Can Not Access (CNA)
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)			
8910Haga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
8911Haga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
10273Tam	4.2	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
10274Tam	5.7	A	S	25	N	1	2	2	-	N	1	1	1	-	N	2	1	N	F	N	1	N	N	MGO (cinder block @ 4.3' on the bench)		
10275Tam	6.8	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
10294Haga	13.2	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAZ (appears to be mud @ 1' on the chimney interior)		
10295Tam	10.4	S	S	25	N	3	4	4	-	Y	2	2	2	-	N	1	1	N	F	N	1	N	N	S (surface spalling @ 11" on the chimney interior), F (fracture @ 8.8' on the wall interior), Frame corroded, **Frame needs to be replaced.**		
10296Tam	12.1	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior), FM (fractures multiple @ 1.3' on the chimney interior)		
10297Tam	12.6	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
10298Tam	12.6	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
10299Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	Plywood blocking access to see in the manhole for inspection.		
10300Haga	11.3	A	S	25	N	2	1	2	-	N	2	2	2	-	N	1	1	N	F	N	1	N	N	FC (fracture multiple @ 1.6' chimney interior), H (hole @ 4.9' on the cone interior)		
10301Haga	12.2	A	S	25	N	2	4	4	-	Y	2	1	2	-	N	1	1	N	F	N	1	N	N	FM (fracture multiple @ 7" on the chimney interior), corroded frame interior, **Needs to be replaced**		
10303Haga	12.3	A	S	25	N	3	1	3	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 7" on the chimney interior), FL (fracture longitudinal @ 2.6' on the chimney interior)		
10304Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
10305Tam	6.2	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N		N	1	N	N	FC (fracture circumferential @ 1.4' on the chimney interior)		
10307Tam	5.3	G	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
10308Tam	5.9	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
10309Tam	7.2	A	S	25	N	2	2	2	-	N	1	1	1	-	N	2	1	N	F	N	1	N	N	DAR (deposits-attached ragging @ 7' on the bench)		
10310Haga	9.1	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
10311Haga	8.5	A	S	25	N	2	1	2	-	N	1	1	1	-	N	2	1	N	F	N	1	N	N	MWL (8 in water level @ 8.9' on the bench)		
10312AHaga	7	A	S	25	N	3	3	3	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 10" on the chimney interior)		
10312Haga	8.5	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
10313Haga	10.8	G	S	25	N	3	2	3	-	N	5	1	5	-	Y	1	1	N	F	N	1	N	N	FC (fracture circumferential @ 1.7' on the chimney interior), SAV (surface aggregate visible @ 1.7' and 9.6' on the chimney interior and the wall interior), JPM (Medium offset frame joint), **Need repair**		
10677ATam	8	A	S	25	N	1	1	1	-	N	1	3	3	-	N	1	1	N	F	N	1	N	N	MGO (rebar in wall @ 5.7' on the wall interior)		
10677BTam	7.7	A	S	24	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
10677Tam	5.9	A	S	25	N	1	1	1	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 1.2' on the chimney interior), SRV (surface reinforcement - visible @ 1.2' on the chinmey interior)		
10678Tam	5.3	A	S	26	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 11" on the chimney interior)		
10679Tam	2.9	A	S	25	N	2	2	2	-	N	3	1	3	-	N	1	1	N	F	N	1	N	N	DAR (deposits attached - ragging @ 5" on the chimney interior)		

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations		
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)		Can Not Locate (CNL)	Can Not Access (CNA)
Area 7 - Adelup / Agana Heights																										
128H_H	13.8	A	S	30	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/2	N	1	N	N			
129H_H	5.8	A	S	30	N	2	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
130H_H	8	G	S	32	N	3	2	3	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
131H_H	7.3	G	S	29	N	3	2	3	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
132H_H	7.4	G	S	25	N	3	3	3	-	N	4	1	4	-	Y	1	2	N	F	N	1	N	N	DSGV (deposits grave @ 7' on the channel); JO (frame offset 6"). **Need Repair**		
140H_H	5.3	A	S	30	N	2	1	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
141H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
142H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
154H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
155H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
156H_H	4.6	A	S	30	N	2	1	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
157AH_H	16	A	S	30	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
157H_H	4.3	A	S	30	N	2	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
158H_H	4.4	A	S	30	N	1	1	1	-	N	2	1	2	-	N	1	1	N	NF	N	1	N	N	FC (fracture circumferential @ 1.6' on the chinmey interior)		
159H_H	4.9	A	S	30	N	2	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N	FC (fracture circumferential @ 1.4' on the chinmey interior)		
160H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
161H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
162H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
163H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
168H_H	15.2	A	S	30	N	2	2	2	-	N	2	1	2	-	N	1	1	N	1/4	N	1	N	N	CC (crack circumferential @ 1.1' on the chimney interior)		
173H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
175H_H	5.6	A	S	32	N	2	1	2	-	N	1	1	1	-	N	3	1	N	NF	N	1	N	N	DSGV (deposits gravel @ 4.8' on the bench)		
176H_H	4.7	A	S	30	N	3	3	3	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
462H_H	2.6	A	S	25	N	2	1	2	-	N	2	1	2	-	N	3	1	N	NF	N	1	N	N	MM (missing mortar @ 8" on the chimney interior), DSGV (deposits gravel @ 2.3' on the bench)		
463H_H	4.3	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
464H_H	4.5	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	2	N	1/4	N	1	N	N	DSGV (deposits - gravel @ 4.5' on the channel)		
465H_H	3.8	A	S	25	N	1	1	1	-	N	2	1	2	-	N	1	1	N	1/4	N	1	N	N	MM (missing mortar @ 8" on the chimney interior)		
466H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
467H_H	10.9	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
468H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
469H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
470H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
471H_H	6.8	S	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N	DSGV (deposits gravel @ 6.8' on the channel)		
472H_H	11.3	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			

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Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations	Can Not Locate (CNL)	Can Not Access (CNA)
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)			
473H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
474H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
475H_H	2.7	S	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	NF	N	1	N	N	CC (circumferential crack @ 0.7' on the chimney interior), SRV (surface reinforcement - visible @ 1.1' on the chimney interior)		
476H_H	6.3	A	S	25	N	1	1	1	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior)		
477H_H	6.8	G	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	1/8	N	1	N	N			
478H_H	17.7	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
479H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
480H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
481AH_H	8.5	A	S	25	N	2	1	2	-	N	2	1	2	-	N	2	1	N	F	N	1	N	N	DAZ (appears to be mud @ 5" on the chimney interior, mud @ 8' on the bench)		
481H_H	7.2	A	S	25	N	2	2	2	-	N	2	1	2	-	N	2	1	N	F	N	1	N	N	MM (missint mortar @ 7" on the chimney interior), DAR (deposits-attached ragging @ 10" on the chimney interior)		
482H_H	6.6	A	S	25	N	1	2	2	-	N	2	1	2	-	N	1	1	N	1/8	N	1	N	N	MM (missing mortar @ 8" on the chimney interior)		
483H_H	5.6	A	S	25	N	1	1	1	-	N	2	1	2	-	N	1	1	N	1/4	N	1	N	N	FM (fracture multiple) @ 8" on the chimney interior		
484H_H	6.6	A	S	25	N	1	2	2	-	N	2	1	2	-	N	1	2	N	1/4	N	1	N	N	MM (missing mortar @ 8" on the chimney interior), DSGV (deposits gravel @ 6.6' on the channel)		
485H_H	4.2	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
486H_H	8.5	A	S	25	N	1	1	1	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	FC (fracture circumferential @ 8" on the chimney interior)		
487H_H	12.9	A	S	25	N	1	1	1	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior)		
488H_H	11	A	S	25	N	1	1	1	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	MM (missing mortar @ 8" on the chimney interior)		
489H_H	10.9	G	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
843H_H	3.3	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
844Haga	2.5	G	S	25	N	3	2	3	-	N	5	1	5	-	Y	1	1	N	F	N	1	N	N	DSGV (deposits gravel @ 2.3' on the bench), MGO (broken chimney @ 1.4' on the chimney interior), JPM (Medium offset frame joint) **Broken chimney needs to be repaired, Frame needs to be replaced.**		
845Asan	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
846Asan	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
847Asan	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
848Asan	8.7	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
849Asan	10.3	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	FC (fracture circumferential @ 2.2' on the chimney interior)		
850H_H	5.3	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	2	N	F	N	1	N	N	DSGV (deposits gravel @ 4.8' on the channel)		
851H_H	3.6	G	S	25	N	2	3	3	-	N	1	1	1	-	N	1	2	N	F	N	1	N	N	DSGV (deposits gravel @ 3.2' on the channel)		
852H_H	10.3	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
853H_H	9	A	S	25	N	2	1	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	CM (crack multiple @ 7" on the chimney interior)		
854H_H	6.7	G	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	FM (fracture multiple @ 0.8' on the chimney interior)		
855H_H	9	G	S	25	N	3	2	3	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
856H_H	6.1	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
857H_H	8.1	G	S	25	N	3	N	N	-	Y	1	1	1	-	N	1	1	N	F	N	1	N	N	JOM (frame offset @ 0.3' on the chimnety interior), **Need repair**		
858H_H	7.4	G	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	FC (fracture circumferential @ 9" on the chimney interior)		
866H_H	5.4	A	S	25	N	3	3	3	-	N	1	1	1	-	N	1	2	N	1/4	N	1	N	N	DSGV (deposits gravel @ 5.5' on the channel)		

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

	General			Cover and Frame							Cone and Riser					Bench and Channel			Other							
Manhole No.	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)	Comments and Other Observations	Can Not Locate (CNL)	Can Not Access (CNA)
867H_H	6	G	S	25	N	3	2	3	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	SRV (surface reinforcement - visible @ 1.8' on the chimney interior)		
868H_H	8.2	G	S	25	N	2	2	2	-	N	2	1	2	-	Y	1	1	N	F	N	1	N	N	MGO (Frame broke off chimney @ 8" on the chimney interior), **Need repair**		
869H_H	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
Area 8 - Mangilao																										
213Mang	5.3	A	S	32	N	2	2	2	-	N	1	1	1	-	N	2	1	N	NF	N	1	N	N	DSGV (deposits gravel @ 5'3" on the bench)		
214Mang	7.2	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
215Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
216Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
217Mang	7.8	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
218Mang	8.1	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
219Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
220Mang	6.6	A	S	32	N	2	1	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
221Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
222Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2179Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2180Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2181Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2182Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2183Mang	4.6	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2184Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2185Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2186Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2187Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2188Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2189Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2190Mang	6.3	A	S	24	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2191Mang	5.5	A	S	25	N	1	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2192Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2193AMang	9.1	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2193Mang	9.7	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2194Mang	14.2	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2195Mang	13.9	A	S	25	N	1	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2196Mang	10.3	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	2	N	F	N	1	N	N	DAR (Deposits attached ragging @ 10.3' on the channel)		
2197Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2198Mang	9.8	G	S	25	N	1	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2199Mang	16.6	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations		
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)		Can Not Locate (CNL)	Can Not Access (CNA)
2200Mang	11.4	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2201Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2202Mang	10.8	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2203Mang	12.7	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2204Mang	5.3	G	S	32	N	2	2	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
2205Mang	5.6	G	S	32	N	2	1	2	-	N	1	1	1	-	N	1	2	N	NF	N	1	N	N	DSZ (appears to be mud @ 5.6' on the channel), OBZ (appears to be rebar @ 5.6' on the channel)		
2206Mang	6.2	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
2207Mang	5.1	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2208Mang	5.8	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
2209Mang	10.3	A	S	32	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2210Mang	10.2	A	S	32	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	MGO (rebars in manhole @ 9.7' on the bench)		
2211Mang	5.4	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N	DSZ (appears to be mud @ 5.4' on the channel)		
2212Mang	4.6	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
2213Mang	5.1	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
2214Mang	3.5	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
2215Mang	5.9	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2216AMang	7.6	A	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2216Mang	7.7	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2217Mang	6.3	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2218Mang	8.8	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2219Mang	11.3	G	S	25	N	1	2	2	-	N	3	1	3	-	N	1	1	N	F	N	1	N	N	FM (fractures multiple @ 9" on the chimney interior)		
2220Mang	5.6	A	S	32	N	2	2	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
2221Mang	9.3	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/2	N	1	N	N			
2222Mang	3.6	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
2223Mang	10.7	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2224Mang	5.5	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
2225AMang	6.2	G	S	23	N	2	4	4	-	Y	4	2	4	-	Y	1	1	N	NF	N	1	N	N	MGO (broken / unattached frame @ 1.1'), FL (fracture longitudinal and multiple @ 5.9' on the wall interior), **Need repair**		
2225BMang	5.3	G	S	23	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2225CMang	5.2	G	S	23	N	2	2	2	-	N	4	1	4	-	Y	2	1	N	F	N	1	N	N	MGO (frame not attached @ 0.6'), OBR (large rock @ 5' on the bench), **Need repair**		
2225DMang	5.6	G	S	23	N	1	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2225Mang	4.9	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2226Mang	3.6	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2227Mang	9.7	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2228Mang	5.1	G	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations	Can Not Locate (CNL)	Can Not Access (CNA)
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)			
2229Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2230Mang	7.9	A	S	32	N	2	2	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2231Mang	11	A	S	32	N	2	2	2	-	N	2	1	2	-	N	1	1	N	1/2	N	1	N	N	FC (fracture circumferential @ 1' on the chimney interior)		
2232Mang	12.5	A	S	25	N	1	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N	DAR (deposits attached ragging @ 12.2' on the channel)		
2233Mang	4.9	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2234Mang	5	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2235Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2236Mang	7.2	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
2237Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2238Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2239Mang	6.4	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
2241Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2242Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2243Mang	7.1	A	S	32	N	1	1	1	-	N	2	1	2	-	N	1	1	N	1/2	N	1	N	N			
2244Mang	6	A	S	32	N	1	1	1	-	N	2	1	2	-	N	1	1	N	1/4	N	1	N	N			
2245Mang	3.6	A	S	32	N	1	1	1	-	N	2	1	2	-	N	1	1	N	NF	N	1	N	N			
2246Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2247Mang	4.2	A	S	32	N	1	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N			
2248Mang	4.5	A	S	32	N	1	2	2	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N			
2249Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2250Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2251Mang	6.5	A	S	32	N	1	1	1	-	N	2	1	2	-	N	1	1	N	F	N	1	N	N			
2252Mang	5.2	A	S	32	N	1	2	2	-	N	2	1	2	-	N	1	1	N	1/2	N	1	N	N			
2254Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2255Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2256Mang	9.2	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	2/3	N	1	N	N			
2258Mang	5.1	A	S	32	N	1	1	1	-	N	2	1	2	-	N	1	1	N	1/4	N	1	N	N			
2259Mang	6.6	A	S	32	N	1	1	1	-	N	2	1	2	-	N	1	1	N	1/2	N	1	N	N			
2260Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2261Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2262Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2263Mang	6.2	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
2264Mang	5.2	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2265Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2266Mang	3.3	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	2/3	N	1	N	N			
2267Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations		
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)		Can Not Locate (CNL)	Can Not Access (CNA)
2268Mang	3.6	A	S	32	N	2	2	2	-	N	1	1	1	-	N	1	1	N	2/3	N	1	N	N			
2269Mang	9.5	A	S	32	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2270Mang	6.2	A	S	32	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N	DSZ (debris @ 6.2' on the channel)		
2272Mang	17	A	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
2273Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2274Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2275Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2276Mang	14.1	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2277Mang	13.8	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	3/4	N	1	N	N			
2278Mang	14.9	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2279Mang	4.1	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
2280Mang	12.2	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
2281Mang	14.3	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2282Mang	2.7	G	S	32	N	1	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
2283Mang	10	S	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
2284Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2285Mang	10.3	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2286Mang	3.2	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	2	N	NF	N	1	N	N	DSGV (deposits gravel @ 3.2' in the channel)		
2287Mang	10.6	A	S	32	N	2	2	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2288Mang	3.5	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	2	N	NF	N	1	N	N			
2289Mang	10.6	A	S	32	N	1	1	1	-	N	1	1	1	-	N	2	1	N	1/2	N	1	N	N	MGO (Large rock @ 10.1' on the bench)		
2290Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2291Mang	3.9	A	S	32	N	2	1	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
2292Mang	3.3	A	S	32	N	2	1	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2293Mang	4.2	A	S	32	N	2	1	2	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
2294Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
2296Mang	4.2	A	O	34	N	1	2	2	-	N	1	1	1	-	N	2	1	N	F	N	1	N	N	Rectangular steel plate manhole cover, OBR (obstacle rocks @ 2.6' on the bench)		
2297Mang	8.3	A	S	25	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2298AMang	5.8	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2298Mang	14.9	A	S	24	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
2299Mang	7.8	A	S	26	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
3048Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
3064Mang	8.7	A	S	26	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
3065Mang	7.5	G	S	26	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
3066Mang	6.9	A	S	26	N	3	2	3	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
3067Mang	10.9	G	S	26	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations	Can Not Locate (CNL)	Can Not Access (CNA)
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)			
3068Mang	7.2	A	S	26	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
3069Mang	8.2	A	S	32	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
3315Mang	10.9	G	S	26	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
4089Mang	7.7	A	S	26	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
4089AMang	7.2	A	S	30	N	2	2	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
4090Mang	12.2	A	S	26	N	1	1	1	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
4091Mang	6.4	A	S	26	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
4092Mang	5.1	A	S	26	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
4093Mang	4.6	G	S	26	N	1	2	2	-	N	1	1	1	-	N	1	1	N	1/8	N	1	N	N			
4094Mang	2.4	A	S	26	N	1	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
5007Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
5009Mang	10.4	A	S	26	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/4	N	1	N	N			
6125Mang	3.9	A	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
6126Mang	6.2	G	S	24	N	2	2	2	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
6127Mang	2.7	A	S	24	N	1	1	1	-	N	1	1	1	-	N	1	1	N	NF	N	1	N	N			
6413Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
6916CPO	3	G	S	26	N	3	3	3	-	N	4	1	4	-	Y	1	1	N	F	N	1	N	N	MGO (frame is broken off chimney @ 5" on the chimney interior). **Need Repair**		
6918CPO	4.2	G	S	26	N	2	2	2	-	N	3	1	3	-	Y	1	1	N	2/3	N	1	N	N	MGO (frame is broken off chimney). **Need to Repair**		
6919CPO	5.8	G	S	26	N	2	3	3	-	N	1	1	1	-	N	1	1	N	2/3	N	1	N	N			
6920CPO	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
6940Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
6941Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
6942AMang	10.5	A	S	30	N	2	2	2	-	N	1	1	1	-	N	-	1	N	F	N	1	N	N			
6942BMang	10.3	A	S	30	N	2	3	3	-	N	1	1	1	-	N	-	1	N	F	N	1	N	N	JOS (small offset frame joint)		
6942Mang	6.5	G	S	30	N	2	2	2	-	N	1	1	1	-	N	-	1	N	S	N	1	N	Y 6.5'	DSZ (appears to be mud @ 6.5' on the bench)		
10696AMang	3	G	S	25	N	1	1	1	-	N	4	1	4	-	Y	1	1	N	1/8	N	1	N	N	MGO (missing material) on the chimney interior, MGO missing concrete collar around frame. **Need to Repair**		
10696BMang	3.7	G	S	24	N	1	1	1	-	N	1	1	1	-	N	1	1	N	1/2	N	1	N	N	New found manhole not on the original list		
10696Mang	3	A	S	25	N	3	2	3	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
10697Mang	2.5	G	S	27	N	2	2	2	-	N	1	1	1	-	N	1	1	N	3/4	N	1	N	N	DSGV (deposits gravel @ 2.5' on the channel)		
10698Mang	3.2	G	S	23	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
10700Mang	3.5	G	S	23	N	2	1	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
10701Mang	4	G	S	23	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
10702Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
10703Mang	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	

Table 3-11
Central Sanitary Sewer Evaluation Survey (SSES)
Summary of Manhole Condition Findings

Manhole No.	General			Cover and Frame							Cone and Riser					Bench and Channel			Other					Comments and Other Observations	Can Not Locate (CNL)	Can Not Access (CNA)
	Depth of Invert to Grade (ft)	Ground Cover	Manhole Type	Cover/Frame Dia (in)	Inflow Guard? (Y/N)	Cover Condition	Frame Condition	Greater of cover and frame values	Frame Grout Collar Condition	Replace Frame and Cover? (Y/N)	Cone Condition & Material*	Riser Condition & Material*	Greater of Cone and Riser	Rungs Condition & Type**	Rehabilitate Cone and Riser (Y/N)	Bench Condition	Channel Condition	Rehabilitate Bench/Channel? (Y/N)	Flow Condition	Infiltration (Y/N)	Infiltration Rating	Grease	Surcharge Signs? (Y/N; depth, ft)			
Area 12 - Tamuning																										
540Tam	4.1	A	S	25	N	2	2	2	-	N	1	1	1	-	N	4	1	N	NF	N	1	N	N	DSF (deposits settled fine @ 3.9' on the bench), **Need to clean**		
541Tam	6	A	S	25	N	2	2	2	-	N	2	1	2	-	N	1	1	N	3/4	N	1	N	N	CC (circumferential crack @ 0.7' on the chimney interior)		
542Tam	7.4	A	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	3/4	N	1	N	N			
543Tam	3.5	G	S	25	N	2	2	2	-	N	1	1	1	-	N	1	2	N	NF	N	1	N	N	DSF (deposits fine @ 3.2' on the channel)		
544Tam	3	A	S	25	N	2	2	2	-	N	4	1	4	-	Y	1	2	N	NF	N	1	N	N	JOM (medium offset frame joint), **Need repair**		
545Tam	11.5	S	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	S	N	1	N	Y 5.9'			
1047Tam	8	S	S	25	N	2	2	2	-	N	1	1	1	-	N	1	1	N	F	N	1	N	Y 5.2'			
1048Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
1049Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
1050Tam	15.5	S	S	25	N	1	1	1	-	N	1	1	1	-	N	1	1	N	S	N	1	N	Y 5.4'			
1052Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
1053Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
1054Tam	7.2	S	S	25	N	2	2	2	-	N	4	1	4	-	N	1	1	N	S	N	1	N	Y 4.2'	RBL (rootball interior @ 3' on the cone interior), **Remove roots going into the pipe**		
1055Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
1056Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
1057Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
1058Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
1059Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
1064Tam	9.7	S	S	25	N	3	3	3	-	N	1	1	1	-	N	1	1	N	F	N	1	N	N			
6929Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
6930Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	
6931Tam	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-		CNL	

Ground Cover:
A - Asphalt
S - Sand/Gravel
D - Dirt
C - Concrete
G - Grass

Manhole Type:
(See C&C Standard Details)
S - Standard Plain MH
SD - Shallow Drop MH
D - Drop MH
O - Other (see comments)

Cover/Frame/Rungs/Cone/Riser/Bench
Channel Condition Code and Defect Types:
1 - Good
2 - Minor
3 - Moderate
4 - Severe
5 - Very Severe
MA - Misaligned
CR - Cracks
CF - Coating Failure
LF - Liner Failure
SP - Spalls/Cracks
Note: Unless otherwise noted, condition generally refers to degree of corrosion/deterioration.

Cone/Riser Material:*
Concrete unless indicated as;
B - Brick
L - Lined

Rung Type:**
Stainless Steel unless indicated as;
P - Plastic/Non-metallic
DI - Ductile/Cast iron
N - None

Flow Condition:
NF - No Flow
1/8 Pipe Depth
1/4 Pipe Depth
1/2 Pipe Depth
3/4 Pipe Depth
F - Full Depth
S - Surcharged

Infiltration:
1 - None
2 - Dripper (< 1/2 gpm)
3 - Runner (1/2 - 4 gpm)
4 - Gusher (> 4 gpm)
S - Single
M - Multiple

Grease:
N - None
L - Light
M - Medium
H - Heavy
SB - Scum Blanket

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Appendix E

Lift Station, Force Main Condition Assessment Data

Likelihood of Failure Factors

Table E-1 lists the scoring for each likelihood of failure factor for force mains from Section 5.2. Likelihood of failure factors are explained for lift stations in Section 6.2.

Table E-1. Likelihood of Failure Scoring for Force Mains

Input Value	Score
L1, Age	
< 20 years	1
20 to < 30 years	2
30 to < 40 years	3
40 to < 50 years	4
>= 50 years	5
L2, Material	
Asbestos Cement	5
Cast Iron	4
Ductile Iron	3
Polyethylene	1
PVC	1
Reinforced Concrete	2
Unknown	2
L3, Condition	
No reported condition problems	1
Minor condition problem reported	3
Major condition problem reported	5

Consequence of Failure Factors

Table E-2 lists the scoring for each consequence of failure factor. Most of the factors were used for force mains and lift stations. The factors that only apply to force mains are noted in the table.

Table E-2. Consequence of Failure Scoring for Force Mains and Lift Stations		
Input Value	Score	Facility
C1, Diameter		
< 8”	1	Force mains only
8” to < 14”	2	
14” to < 30”	3	
30” to < 36”	4	
>= 36”	5	
C2, Major roadways		
Does not cross or run along major Route	1	Force mains only
Crosses or runs along major Route	5	
C3, Proximity to surface water		
> 1000 feet	1	Force mains and lift stations
500 to 1,000 feet	2	
200 to 500 feet	3	
100 to 200 feet	4	
< 100 feet	5	
C4, Proximity to water well		
> 1000 feet	1	Force mains and lift stations
500 to 1,000 feet	2	
200 to 500 feet	3	
100 to 200 feet	4	
< 100 feet	5	
C5, Serves important area		
No	1	Force mains and lift stations
Yes	5	
C6, Serves important facilities		
None	1	Force mains and lift stations
Airport	2	
School	3	
Hospital	5	
C7, Average flow		
< 0.5 mgd	1	Force mains and lift stations
0.5 to < 1.0 mgd	2	
1.0 to < 5.0 mgd		
5.0 to < 10.0 mgd		
>= 10.0 mgd		

Hydraulic Capacity Evaluation – Pump Drawdown Tests

An evaluation of the hydraulic capacity of each lift station is recommended to identify potential problems with pumping capacity and operating efficiency of the pumps. A manual pump drawdown test can be implemented at each lift station to evaluate the operation of the pumps. A review of the as-built drawings should be completed prior to the test to determine the design size of the station wet well and pump curves to determine the rated capacity of the pumps. The collected data will be used to analyze and compare to the designed operation of the lift station, which will support the hydraulic assessment of the overall wastewater collection system. It is recommended that GWA conduct a basic drawdown test to estimate the maximum and average pumping rates for each pump and configuration of pumps present at each station. Figure E-1 shows a sample pump test form that GWA can use when testing their wastewater pumps. The following items that GWA should consider when conducting a basic drawdown test are:

1. For some of the larger lift stations that are currently equipped with flow meters, the pump flowrates will be determined by using the existing station meters.
2. Pumps with variable frequency drives must be operating at 100% speed so that the pumps are operating at full RPM during the testing.
3. The plan cross sectional area of the wet well is determined (by measuring the diameter for circular wells or length and width for rectangular shaped wet wells).
4. A tape measure is used at submersible type lift stations to measure the changes in water level during drawdown. For wet well/dry well type lift stations (where the wet well may be inaccessible), a portable level measurement device or existing level transducers may be used to record changes in water level during drawdown.
5. To ensure flow data is collected over the normal operating level range in the wet well, the test is started when the water level reaches the lift station high operating level set point.
6. The first pump is turned on and the pump "on time" is noted. The falling water level in the wet well is monitored by noting the change in depth versus time. Time elapsed is noted (using a stop watch) at set changes in water level (1 foot intervals). The pump is turned off when the low operating level is reached and the "off time" is recorded.
7. The wet well area and water depth readings in the wet well are used to calculate the incremental and cumulative volumes of pumped flow. Gallons per minute (gpm) estimates for the pump are obtained by dividing the changes in volume by the time elapsed between readings.
8. Multiple trials of the pump drawdown test should be performed to obtain reliable flow rate data.
9. The pump test should be repeated for the second (or remaining pumps) and also performed for different configurations of pumps that the station is expected to experience (1 and 2, 1 and 3, etc.).

	FLOW METER MEASUREMENT							DRAWDOWN TESTING MEASUREMENT							
PUMP ON	METER INDICATOR (GPM)	TIME	MINUTES	METER TOTALIZER (1000 GAL)	GALLONS	FLOW (GPM)	FLOW (MGD)	WETWELL PLAN AREA (SF)	DRAWDOWN DEPTH (FT)	TIME	MINUTES	DRAWDOWN VOLUME (CUFT)	GALLONS	FLOW (GPM)	FLOW (MGD)

Figure E-1. Sample Pump Test Form

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