American Samoa Power Authority
P.O Box PPB
Pago Pago, American Samoa 96799
Phone: (684) 699-5282
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In reply refer to:

June 28, 2012

Via Electronic Mail & Postal Mail

Mr. Greg Arthur, Environmental Engineer,
United States Environmental Protection Agency
Region IX
75 Hawthorne Street
San Francisco, CA 94105

Re: CWA-309(a)-11-016 and 017 - Administrative Orders (AO)
Submittal of Scoping Summary of Projects to Increase Critical Initial Dilution Factors (CID).
Submittal of Preliminary Engineering Plans for WWTP Effluent Disinfection and De-chlorination.

Dear Mr. Arthur:

American Samoa Power Authority (ASPA) is pleased to submit to USEPA the above referenced documents that we believe provide us a roadmap to the final resolution of on-going issues regarding NPDES permits and 301(h) waivers for our Tafuna and Utulei Wastewater Treatment Plants (WWTPs). ASPA has done its best under strict AO scheduling requirements to prepare these documents as accurately and completely as possible with the support of our professional consultants, contractors, and internal staff.

You will notice in these reports that there are currently some data gaps from our wastewater operations and outside water testing laboratories that will be filled and incorporated into these reports as soon as possible. Furthermore, new technology allowing for the production of 15% chlorine solution on-site was discovered late in the evaluation of disinfection options. We believe this may be promising, particularly given our remote location, and would like to further explore this option in the prescribed AO timeframe with the manufacturer, our design-build team, and USEPA before finalizing our direction with this vital project.

Importantly, the Preliminary Engineering Plan for disinfection currently estimates capital costs that are about triple the bid range. While our team will do its best to discover the reason for this substantial escalation of estimated preliminary costs and will seek to reduce them to what we believe are reasonable levels, we - and surely USEPA - must proceed very cautiously during this review period. Not only are we concerned that these costs are not reasonable for capital expenditures, but we are also mindful of significant operation and maintenance costs, especially given utility rate schedules among the highest in the world being paid by many on very modest incomes. Finally, such significant capital costs could impact construction funding for the East Side Village Wastewater Collection system – which may arguably have a greater impact on water quality than effluent disinfection - and other critical USEPA funded projects intended to improve groundwater, marine waters, public drinking water systems and the overall health of the island and our people.

ASPA sincerely appreciates the assistance you and everyone at USEPA, along with our partners at ASEPA, provide to us in helping improve the Territory's public and environmental health. Thank you and we look forward to further advancing these projects in the coming year.

Best Regards,

Andra Samoa, Chief Executive Officer

Attachment

COE & VAN LOO
CONSULTANTS, INC.

SCOPING REPORT

IDENTIFICATION OF PROJECTS TO INCREASE THE CRITICAL INITIAL DILUTION FACTOR FOR BOTH THE TAFUNA AND UTULEI OCEAN OUTFALLS

AMERICAN SAMOA POWER AUTHORITY

June 30, 2012

Prepared for:
American Samoa Power Authority
PO Box PPB
Pago Pago, American Samoa 96799

Prepared by:
Coe & Van Loo Consultants, Inc.
4550 N. 12th Street
Phoenix, Arizona 85014

Registered Professional Engineer (Civil)
CERTIFICATE NO. 14478
ERIC LAURIN
Date Signed 30/June/12
ARIZONA, U.S.A.
Expires: 6/30/15

1.07.0216801

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Registered Professional Engineer (Civil)
CERTIFICATE NO. 14478
ERIC LAURIN
Date Signed 30 June '12
ARIZONA, U.S.A.
Expires: 6/30/15

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CERTIFICATION STATEMENT

I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, I certify that the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I certify that all wastewater samples analyzed and reported herein are representative of the ordinary process wastewater flow from this facility. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

Coe & Van Loo Consultants, Inc.

Signature:
Print Name: Eric Laurin, PE, PEng
Title: Associate, Director
Date: 30 June '12

American Samoa Power Authority

Signature:
Print Name: Andra Samoa
Title: Chief Executive Officer
Date: June 27, 2011

EXECUTIVE SUMMARY

The United States Environmental Protection Agency (USEPA) in Administrative Orders (AO) issued on July 27, 2011, ordered the American Samoa Power Authority (ASPA) to perform an investigation to identify projects that could be taken to optimally increase the critical initial dilution (CID) factor for the Utulei and Tafuna Sewage Treatment Plants (STP) ocean outfall discharge. The results of this analysis are to be summarized and presented in a scoping report to address the following projects, at a minimum:

- o A reduction in the expected daily-maximum mean and peak discharge flow rates through infiltration and inflow upgrades to the sewer system.
- o A reduction in the expected daily-maximum mean and peak discharge flow rates through increases in sewer system storage capacities and optimized delivery.
- o A reduction in the expected daily-maximum mean and peak discharge flow rates through the installation and operation of on-site wet-weather storage.
- o A doubling of the diffuser length of the Utulei ocean outfall.
- o Any other project to increase the size of the zone of initial dilution.

The scoping summary report shall include a description of the project considered, the resulting estimated mean and peak discharge flow rates, the capital cost of the project, and a construction schedule to not extend beyond June 30, 2013. Compliance with these orders is necessary for ASPA to retain the 301(h) waivers to discharge primary treated effluent from the Utulei and Tafuna STPs.

Coe & Van Loo Consultants, Inc. (CVL) has performed the tasks associated with a, b, c, and e. GDC, under separate contract with ASPA, has addressed item d above and performed the necessary modeling of the outfalls and diffusers to describe the relative effects of reduced outflows on the CID. See Appendix M for a copy of GDC's technical memorandum with conclusions and recommendations. The resulting outfall models have been used by CVL to determine the efficacy of projects selected to reduce STP outflows in achieving the desired increase to the CID.

Having obtained all available information on the collection, pumping, and treatment systems of the Utulei and Tafuna STPs, daily flows to each facility.

A comparison of these flows during precipitation events was made to establish a relationship between rainfall and sewage flows. Periods of relatively dry weather were used to establish base flow conditions. Periods of wet weather with significant rain events were used to establish wet weather flows. The difference in flows between wet and dry periods was defined as excess Infiltration and Inflow (I&I) that are to be reduced by implementing collection system repairs, storage or on-site wet weather storage. Table ES-1 summarizes the results of the I&I flow analysis.

Table ES-1 – Results of I&I Flow

	Dry Weather	Wet Weather	Excess I&I
TAFUNA			
Average Daily Flow	1.6 MGD	2.7 MGD	1.1 MGD
Peak Flow	3.7 MGD	6.2 MGD	2.5 MGD
UTULEI			
Average Daily Flow	1.0 MGD	2.8 MGD	1.8 MGD
Peak Flow	3.4 MGD	6.0 MGD	2.6 MGD

MGD = Million Gallons per Day

It is noted that the I&I rate for the Utulei system is larger than that of the Tafuna system based on a per acre and per capita basis as shown in Table 13 of the report. A one-time sampling of the incoming wastewater at the Utulei STP for salinity indicates a "salinity" equivalent of approximately 12 percent seawater. This strongly suggests that seawater is infiltrating the collection system and lift stations along the Harbor and contributing in some measure to the increased I&I values noted for the Utulei system. Review of BOD_5 concentrations of the STP inflows also indicates that the Utulei system experiences greater I&I because it average BOD_5 concentration for 2011 is approximately 25 percent less than that calculated for Tafuna indicating a sewer greatly diluted by I&I.

The report investigates various means to reduce expected daily-maximum mean[1] and peak[2] discharge flow rates through I&I upgrades in the collection system, lift stations, sewer system storage, and on-site storage at the STPs. These projects were ranked and a selection was made based on cost, potential increase in the critical initial dilution (CID) at each outfall and the ability to meet the 30 June 2013 construction deadline. Tables ES-2 and 3 below provides a summary of our results.

Table ES-2 – List of Projects, Tafuna Sewer System

	Items	Cost ($)
Infiltration and Inflow Upgrades		
Sewers below Sea Level	Video, Flushing, Grouting Pipe, Grouting Manholes	251,800
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	380,722
	Dollar per Gallon Reduction in Flow (Peak)	*5.08*
Areas with Suspected I&I	Video, Flushing, Grouting Pipe, Grouting Manholes	367.613
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	555,830
	Dollar per Gallon Reduction in Flow (Peak)	*9.11*
Sewer System Storage Capacity Upgrade		
Equalization, Vaitele LS	Equalization Basin, EQ Return Pumps, EQ Diversion Structure, Aeration, Mixing, Piping, Odor Control,	680,000

[1] Defined as Average Day flows in the report.
[2] Defined as Peak Day flows.

	Items	Cost ($)
	Electrical & Controls, Generator, Site Improvements	
Storage in the System, Vaitele LS	Video, Flushing, Grouting Pipe, Grouting Manholes	208,760
	Subtotal	888,760
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	1,343,805
	Dollar per Gallon Reduction in Flow (Peak)	2.05
Equalization, Airport LS	Equalization Basin, EQ Return Pumps, EQ Diversion Structure, Aeration, Mixing, Piping, Odor Control, Electrical & Controls, Generator, Site Improvements	930,000
Storage in the System, Airport LS	Video, Flushing, Grouting Pipe, Grouting Manholes	27,200
	Subtotal	957,200
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	1,447,286
	Dollar per Gallon Reduction in Flow (Peak)	1.00
On Site Wet Weather Storage		
Equalization at Tafuna	Equalization Basin, EQ Return Pumps, EQ Diversion Structure, Aeration, Mixing, Piping, Odor Control, Electrical & Controls, Generator, Site Improvements	2,245,000
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	3,394,440
	Dollar per Gallon Reduction in Flow (Peak)	0.97
Tafuna Ocean Outfall		
Diffuser Modifications	12.5 inch end gate port, restriction of existing port areas from 8 inches to 6 inches	150,000[1]
	Dollar per Gallon Reduction in Flow (Peak) not applicable	

Table ES-3 – List of Projects, Bay Area

	Items	Cost ($)
Infiltration and Inflow Upgrades		
Sewers below Sea Level	Video, Flushing, Grouting Pipe, Grouting Manholes	1,089,200
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	1,646,870
	Dollar per Gallon Reduction in Flow (Peak)	1.65
Areas with Suspected I&I	Video, Flushing, Grouting Pipe, Grouting Manholes	226,830
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	342,967
	Dollar per Gallon Reduction in Flow (Peak)	4.63

	Items	Cost ($)
Sewer System Storage Capacity Upgrade		
Equalization, Malaloa LS	Equalization Basin, EQ Return Pumps, EQ Diversion Structure, Aeration, Mixing, Piping, Odor Control, Electrical & Controls, Generator, Site Improvements	1,230,000
Storage in the System, Malaloa LS	Video, Flushing, Grouting Pipe, Grouting Manholes	314,198
	Subtotal	1,544,198
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	2,334,827
	Dollar per Gallon Reduction in Flow (Peak)	0.81
Equalization, Faga'alu LS	Equalization Basin, EQ Return Pumps, EQ Diversion Structure, Aeration, Mixing, Piping, Odor Control, Electrical & Controls, Generator, Site Improvements	580,000
Storage in the System, Faga'alu LS	Video, Flushing, Grouting Pipe, Grouting Manholes	74,318
	Subtotal	654,318
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	989,328
	Dollar per Gallon Reduction in Flow (Peak)	2.03
On Site Wet Weather Storage		
Equalization at Utulei	Equalization Basin, EQ Return Pumps, EQ Diversion Structure, Aeration, Mixing, Piping, Odor Control, Electrical & Controls, Generator, Site Improvements	2,320,000
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	3,507,840
	Dollar per Gallon Reduction in Flow (Peak)	1.10
Utulei Ocean Outfall		
Diffuser Modifications	10.5 inch end gate port, possible restriction of existing port areas from 6 inches to 5.5 inches performed in tandem with underwater outfall repair/maintenance work.	190,000[1]
	Dollar per Gallon Reduction in Flow (Peak) not applicable	

Of these project only the top two highest scores were selected based on the selection criteria defined in the report as seen in the tables below:

Table ES-4 – Selection Matrix for Tafuna

	Effect on CID		Capital Cost		Technical/ Operational Feasibility		Construct-ability		Time to Complete		Final Score
Project Description		**5**		**4**		**2**		**1**		**5**	
Sewer Collection System Improvements											
Improvements to known problem areas in the existing collection system facilities	1	5	1	4	5	10	5	5	2	10	34
Improvements to areas where the collection system is installed below high tide elevations	1	5	1	4	5	10	5	5	3	15	39
Lift Station Operation Improvements											
Storage at Vaitele	3	15	4	16	3	6	3	3	2	10	50
Storage at the Airport	4	20	5	20	3	6	3	3	2	10	**59**
Equalization Basin Installation at Treatment Plants											
Tafuna	5	25	4	16	2	4	2	2	1	5	**52**
Outfall Adjustments											
Diffuser Modifications	5	25	5	20	5	10	3	3	3	15	**73**

Table ES-5 – Selection Matrix for Utulei

	Effect on CID		Capital Cost		Technical/ Operational Feasibility		Construct-ability		Time to Complete		Final Score
Project Description		**5**		**4**		**2**		**1**		**5**	
Sewer Collection System Improvements											
Improvements to known problem areas in the existing collection system facilities	1	5	1	4	5	10	5	5	2	10	34
Improvements to areas where the collection system is installed below high tide elevations	4	20	2	8	5	10	5	5	1	5	48
Lift Station Operation Improvements											
Storage at Malaloa	3	15	5	20	3	6	3	3	3	15	**59**
Storage at Faga'alu	2	10	5	20	3	6	3	3	3	15	**54**
Equalization Basin Installation at Treatment Plants											
Utulei	5	25	3	12	1	2	1	1	1	5	45
Outfall Adjustments											
Diffuser Modifications	5	25	5	20	5	10	3	3	3	15	**73**

The installation of an equalization basin at the Tafuna STP site has a high score (52) but cannot be completed by 30 June 2013. ASPA should consider funding this project at a future time to further increase the CID at this outfall.

The final increase in CID and reduction in flows for these project is shown in Tables 30, 31, 32, and 33.

1.0 INTRODUCTION

1.1 Purpose

Coe & Van Loo Consultants, Inc. (CVL) has been retained by the American Samoa Power Authority (ASPA) to provide engineering service to determine corrective measures necessary to mitigate ongoing discharge limit violations identified by the United States Environmental Protection Agency (USEPA) in Administrative Orders issued for the Utulei and Tafuna Sewage Treatment Plants (STP). The purpose of this report is to identify a selection of possible improvements and make recommendations for system improvements that will satisfy the requirements of the Administrative Orders.

The scope of work for this project includes the preparation of a report in which projects to increase the critical initial dilution factor at the Zones of Initial Dilution (ZID) for both STP outfalls are identified. The remaining AO stipulations will be addressed in other proposal solicitations and are not part of this scope.

1.2 Background

American Samoa consists of several small islands located at the eastern end of the Samoan archipelago in the South Pacific Ocean (see Figure 1). The American Samoa Power Authority (ASPA) owns and operates two Sewage Treatment Plants (STP) on the island of Tutuila (American Samoa) known as the Tafuna STP and the Utulei STP. Figure 2 and Figure 3 show the location of the treatment plants with their outfalls to the Pacific Ocean. Both facilities are subject to the requirements and stipulations contained in a USEPA issued Administrative Order (AO) dated July 27, 2011, (CWA-309(a)-11-016 for Tafuna STP) and CWA-309(a)-11-017 for Utulei STP). ASPA is required to engage in corrective measures to mitigate ongoing discharge limit violations within the established Zone of Initial Dilution (ZID). A copy of the Administrative Orders is found in Appendix A, Administrative Order.

1.2.1 USEPA Administrative Order

The AO identifies the following exceedances to be remedied.

Tafuna STP	**Utulei STP**
Total Nitrogen	Total Nitrogen
Enterococci	Enterococci
Total Phosphorus	
Chlorophyll-a	

The tasks to be performed to achieve the goal of mitigating these exceedances are stated in the AO as follows:

The scoping summary shall cover the following projects to increase the critical initial dilution factor:

- o A reduction in the expected daily-maximum mean and peak discharge flow rates through infiltration upgrades to the sewer system;

- A reduction in the expected daily-maximum mean and peak discharge flow rates through increases in sewer system storage capacities and optimized delivery;
- A reduction in the expected daily-maximum mean and peak discharge flow rate through the installation and operation of on-site wet-weather storage;
- A doubling of the diffuser length of the Tafuna ocean outfall;
- Any other project to increase the size of the zone of initial dilution.

1.2.2 Study Area

The area under study may be characterized has having a tropical, wet, marine climate with little significant seasonal temperature change. Precipitation is abundant, with a decidedly wet period extending from December through February, although periods of wet weather may be experienced at any time of the year. Average annual precipitation at the Pago Pago Airport is 119 inches and the average annual temperature is 80° F.

The island's topography is the result of volcanic forces that have created a steep, rocky terrain and relatively narrow coastal plain. Pago Pago harbor was formed as the result of the collapse of a caldera and evidence of lava flows may be seen at the shoreline.

A coral reef rings most of the island. It is of variable width and extent.

1.2.3 Study Sites

The areas to be investigated include the sewer collection systems for both Treatment Plants, including the system lift stations, the treatment plant sites, and each outfall used to discharge treated effluent. See Figure 4 for an overview over the sewer systems and their locations on the Island.

The Tafuna STP's collection system serves an area that encompasses the Airport and the developed area north of the Airport. The coastal plain is wider in this area of Tutuila Island and sewer service has been extended to the middle of the island. The land use served is also a mix of residential, commercial and municipal types. See Figure 5.

The Utulei STP's collection system serves a narrow area adjacent to the shore, extending from the Village of Atuu to Faga'alu. It includes Pago Pago. The existing Starkist and Tri-Marine canneries are not served by the ASPA collection system. Each has its own wastewater collection, treatment and outfall discharge. A portion of the canneries' bathroom facility and cafeteria waste is conveyed into the ASPA Bay Area sewer collection system. The service area is a mix of residential, commercial and municipal land use. See Figure 6.

1.2.4 Rainfall Data

Rainfall data was used to determine the extent of inflow and infiltration (I&I) resulting from storm events on the island. All rainfall data was obtained from the National Oceanic and Atmospheric Administration (NOAA). Data for the years prior to 2012 was obtained online and was collected at the rain gauge located at Tafuna/Pago Pago International Airport (Gauge No. USAF 917650, WBAN 61705). The data is collected hourly. The data set indicates "missing data" by an asterisk (*). The assumption is made, confirmed by NOAA employees, that these data points represent periods of no precipitation.

The data was reported in Greenwich Mean Time (GMT) and converted to American Samoa local time.

Rain data for dates in the year 2012 were collected at a rain gauge from NOAA's Weather Office (WSO) in Tafuna, American Samoa, Pago Pago and made available to ASPA for this report by Simona Taufau at the local NOAA office. (Office Number: 684-699-9130, Cell: 684-770-6061)

Rain Data obtained from an automated website system at NOAA (www.ncdc.noaa.gov) for the years 2008 through 2011 is listed as daily totals in Appendix B, NOAA Precipitation Data.

1.3 Approach

Collection system and the treatment plants data was collected from ASPA as part of this study. This data included topographical information, physical system properties of pipes, manholes, and lift stations, operating parameters such as flow measurements, pump run times, and data obtained from previous studies. In addition rainfall data was obtained from NOAA as noted above.

The original approach considered by CVL was to review flows experienced at the Lift Stations by using pump run times, and obtain STPs flows from available circular flow charts. A comparison of these flows during precipitation events was made to establish a relationship between rainfall and sewage flows. Periods of relatively dry weather were used to establish base flow conditions at the Lift Stations and STPs. Likewise, flows during wet weather periods were identified to establish wet weather flows. The difference in flows measured during base flow periods and those measured during wet weather periods is defined in this report as excess Infiltration and Inflow (I&I). ASPA reported that only minor expansion of the service areas was performed during the period 2009 to 2012. It was assumed that the service area population for the Bay Area service area had not changed significantly during the 2009 to 2012 period and that base flows would be equal for the entire period studied. The Tafuna system has been expanded to serve additional portions of Tualauta during that timeframe. The expansion was assumed to not be substantial enough to invalidate the use of data available for the whole period ranging from 2009 to 2012.

There being no flow metering at the lift stations, pump run times were initially used to quantify the amount of excess I&I. The rationale for this approach is that pump run times will increase during period of increased inflows to the wet wells as the pumps operate for a longer time to convey the additional flows to the system downstream. This approach was found to not be feasible because of lack of reliable data as further described in Sections 6.0 and 7.0 and was substituted with the establishment of an area wide flow factor derived from the STP inflow data as described in these sections. Flows to the lift stations were calculated for both dry and wet weather periods. The difference between the two factors was defined as excess I&I. It is this I&I flow that was used to investigate I&I mitigation alternatives at each lift station.

STP flows during the dry and wet weather periods selected for this study were obtained from ASPA's daily monitoring reports (DMRs) rather than the circular flow chart recorders that currently monitor discharges instantaneously at each STP's outfall weir. The flow charts continuously record flows at the outfall chamber using an ultrasonic level control system that converts flow depth over the weir to a discharge rate. It was determined that this flow monitoring system is influenced by the pump discharges from the STP's influent lift station, resulting in immediate and erratic fluctuations in flow rates as constant speed pumps turn on and off during their operating cycle, making the determination of increased flows caused by I&I difficult. See Sections 8.0 and 9.0 for further discussion.

The resulting quantification of I&I flows at the STPs was used to size the required flow attenuation or equalization improvements necessary to reduce outfall discharges and increase the ZID as mandated by the AOs.

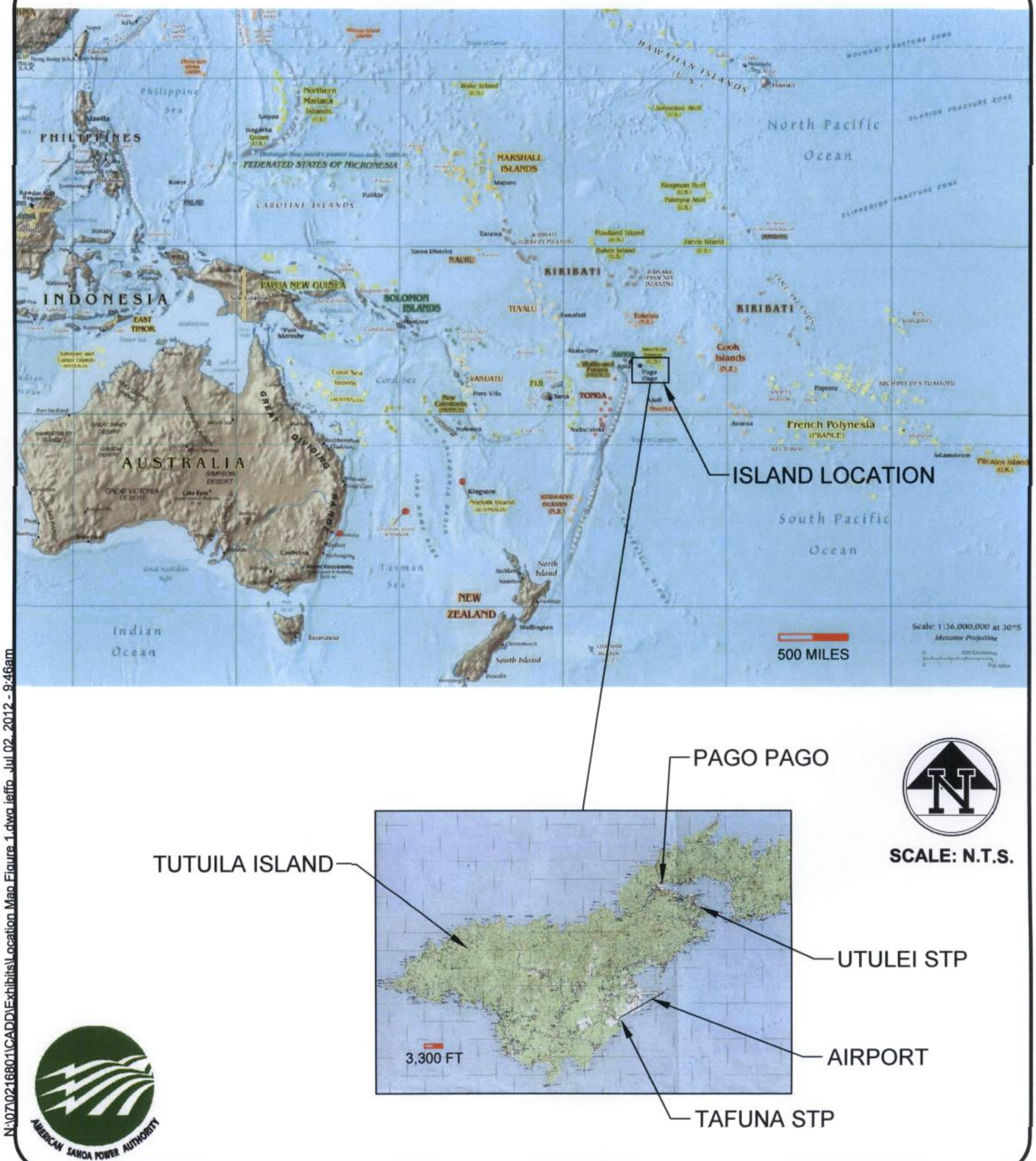
The expected STP effluent flows were taken from the I&I study results and provided to Glatzel Da Costa (GDC) for inclusion in the outfall dilution and mixing models. GDC provided CVL with its recommendations for modifying the outfall structures. These recommendations were included in the report. See Section 10.0.

Each alternative was evaluated and ranked and recommendations made to proceed with those projects that would cost effectively increase the ZID.

1.4 Limitations

The success of identifying problems and solutions is directly tied to the available data provided to CVL by ASPA. The Scope of this investigation did not include an extensive on-site system monitoring and evaluation. Due to the system operating setup and impaired equipment, data gaps were encountered creating challenges in determining the magnitude of I&I:

- Available flow measurement information was found to be useful to identify flows over weekly or longer periods. Daily flow data was only partially reliable because read times were not consistent. Diurnal flows cannot be obtained for either collection system from the available data.
- Some of the equipment in the Bay Area system (Utulei) has been impaired by the September 2009 Tsunami. Normal equipment function is being restored but operational issues continue to exist.



LOCATION MAP	AMERICAN SAMOA	JOB NO 1.07.0216801
4550 NORTH 12TH STREET PHOENIX, ARIZONA 85014 TELEPHONE (602) 264-6831	COE & VAN LOO PLANNING • ENGINEERING • LANDSCAPE ARCHITECTURE	FIGURE 1

SCALE: N.T.S.

LOCATION MAP	TAFUNA STP & OUTFALL	JOB NO 1.07.0216801
4550 NORTH 12TH STREET PHOENIX, ARIZONA 85014 TELEPHONE (602) 264-6831	COE & VAN LOO PLANNING • ENGINEERING • LANDSCAPE ARCHITECTURE	FIGURE 2

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SCALE: N.T.S.

LOCATION MAP	UTULEI STP & OUTFALL	JOB NO 1.07.0216801
4550 NORTH 12TH STREET PHOENIX, ARIZONA 85014 TELEPHONE (602) 264-6831	COE & VAN LOO PLANNING•ENGINEERING•LANDSCAPE ARCHITECTURE	FIGURE 3

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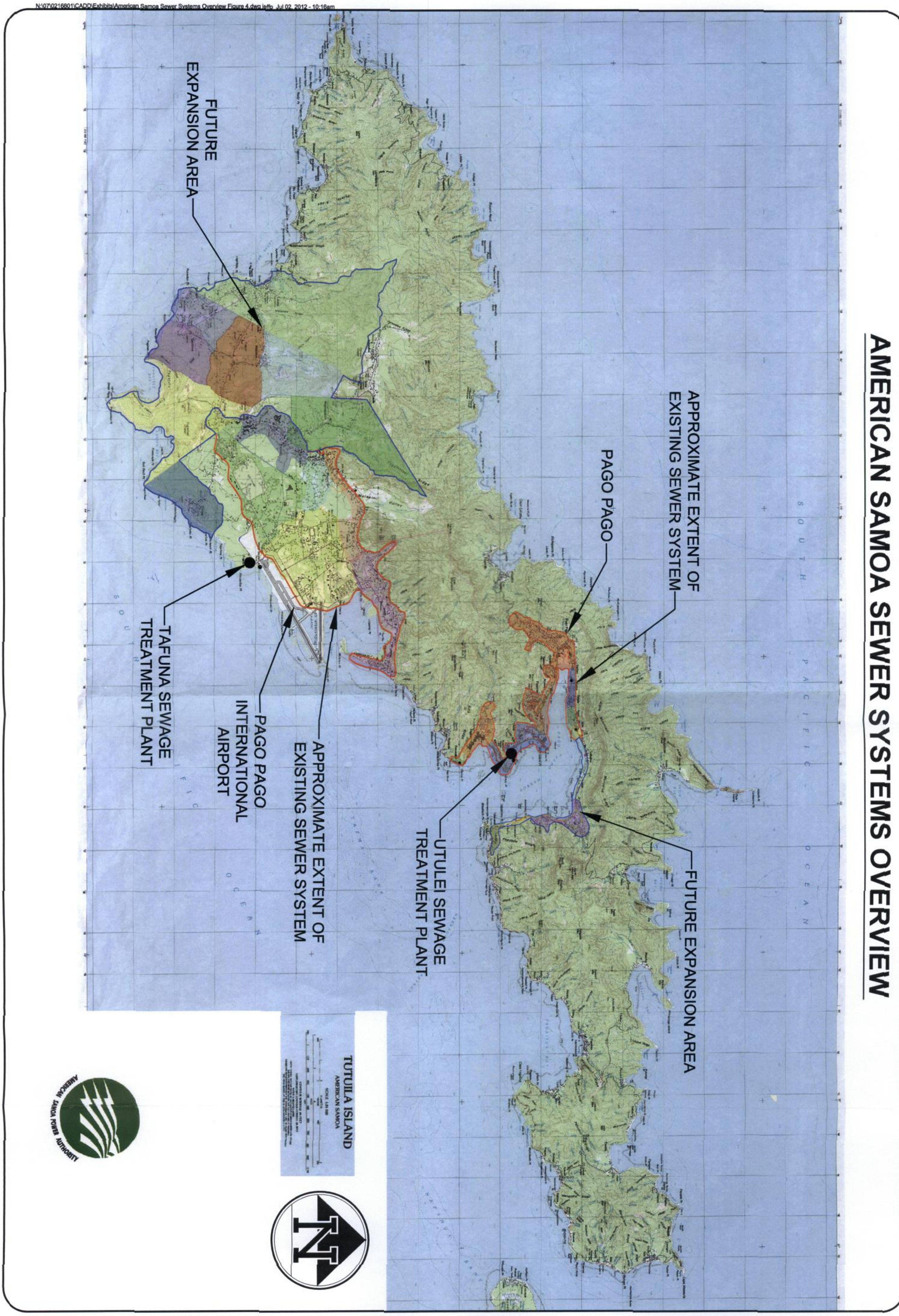


JOB NO. 07-0216801	AMERICAN SAMOA SEWER SYSTEM OVERVIEW	AMERICAN SAMOA POWER AUTHORITY
FIGURE 4	4550 NORTH 12TH STREET *PHOENIX, ARIZONA 85014* TELEPHONE (602) 264-6831	COE & VAN LOO PLANNING • ENGINEERING • LANDSCAPE ARCHITECTURE

2.0 EXISTING CONDITIONS

The treatment of wastewater in American Samoa is currently accomplished in two ways:

- By community wastewater collection, treatment and disposal systems owned and operated by the American Samoa Power Authority (ASPA)
- By individual cesspools and on-site septic systems that are owned and operated by private businesses and individuals.

The community wastewater systems primarily include the Tafuna and Bay Area systems shown in Figure 5 and Figure 6. From the approximately 10,580 housing units in the Western and Eastern Districts of Tutuila Island, 5,744[3] or 54% are currently served by the community sewer systems.

The individual on-site systems are, with few exceptions, soil-based treatment systems that rely on the percolation of wastewater flows through the soil to remove pathogens. Variable soil conditions in the Territory impact the effectiveness of on-site wastewater treatment and failed or failing systems are evident. ASPA is embarked in a long term program to connect more homes and businesses to the Tafuna and Utulei collection systems. On 22 May 2012, ASPA inaugurated a significant expansion to the Utulei STP collection system in which sewer service will be provided to the following villages east of the current terminus of the collection system: Onesospo, Leloaloa, and Aua. This project is known as the East Side Village Wastewater Collection System Improvements (ESV). See Figure 6 for the location of the proposed improvements. Plans are also being made for the future expansion of the Tafuna STP collection system to Leone Village (see Figure 5). The anticipated flows from both expansion areas have been included in this study.

2.1 Tafuna STP Tributary Area

2.1.1 Collection System

The Tafuna sewer system serves the villages of Faleniu, Iliili, Malaeimi, Mesepa, Nu'uuli, Pava'ia'i, and Tafuna (see Figure 5). The system serves approximately 21,000 individuals or 4,137 housing units. A future expansion could include Auma, Leone, Puapua, Futiga, Malaeloa/Huau/Aitulagi, Taputimu, Vailoatai, Mapusagaou, and Vaitogi. See Table 1 below for population and housing numbers and areas for each village.

It is generally assumed that as the population increases, peaking factors decrease. This reflects the fact that as the population increases and, especially, diversifies, residents are less likely to follow similar schedules. This tends to flatten the demand curve, resulting in less obvious peaks over the course of the day. Common formulas to approximate peaking factors such as the Harmon formula demonstrate this inverse relationship of decreasing peaking factor with increasing population size.

The diameter of the sewer mains in the collection system varies from 6 to 24 inches. Manholes are constructed of fiberglass or of precast concrete. The sewer system consists of approximately

[3] All population figures were obtained from the 2010 US Census and adjusted as necessary by ASPA to more accurately reflect the known service area populations. See Appendix I, Villages Census Population.

200,000 LF of pipe. Pipe materials are a mix of Asbestos Cement Pipe (ACP), Vitrified Clay Pipe (VCP), Polyvinyl Chloride (PVC) and Ductile Iron Pipe (DIP). Pump station discharge force mains vary in size from 1.5 to 8 inches. Service lines are nearly all 4 inches in diameter.

Table 1 Tafuna Tributary Area Statistics

Village	Population[1]	Housing Units[1]	Population Assuming 6 people per Housing Unit	Sewer System Gross Area (ac)
Existing Areas				
Faleniu	1,897	347	2,082	200
Iliili	3,195	642	3,852	827
Malaeimi	1,182	228	1,368	162
Mesepa	444	97	582	70
Nu'uuli Area 1	659	115	690	420
Nu'uuli Area 2	3,296	660	3,960	
Pava'ia'i	2,450	432	2,592	328
Tafuna	7,945	1,616	9,696	914
Total	**21,068**	**4,137**	**24,822**	**2921**
Future Areas				**Village Gross Area (ac)**
Auma	254	52	312	-
Leone	1,919	418	2,508	1,648
Puapua	965	190	1,140	-
Futiga	723	116	696	915
Malaeloa/Ituau/Aitulagi	1,248	235	1,410	1,082
Taputimu	841	159	954	374
Vailoatai	1,447	263	1,578	308
Mapusagao	1,126	221	1,326	892
Vaitogi	1,959	382	2,292	405
Total	**10,482**	**2036**	**12,216**	**5,624**
Grand Total	**31,550**	**6,173**	**37,038**	**8,545**

[1] From summary spreadsheet provided by ASPA – Appendix C, Villages Census Population

2.1.2 Lift Stations

The Tafuna system includes 11 wet well pump stations with submersible grinder or non-clog type pumps. The material of construction is typically precast concrete. The smallest lift stations are of the package type system. The pumps are all Flygts or Hydromatic. Manual transfer switches have been provided at each lift station for hook up of a portable emergency generator. ASPA has indicated that no emergency generators are currently installed at the lift stations. The lift stations and tributary areas are shown in Figure 7. Table 16 in Section 6.0 lists the currently active lift stations serving the Tafuna STP service area. Recommendations for improvements to these Lift Stations are made in Section 6.3.

2.1.3 Treatment Plant

Wastewater conveyed through the Tafuna collection system is treated at the Tafuna Sewage Treatment Plant (Tafuna STP). The existing plant, constructed in 1971, is located in Fogagogo near the western end of the main runway at Pago Pago International Airport. The as-built design dry-weather and daily peak design criteria are 2.16 MGD and 6.0 MGD respectively. The plant provides primary treatment only. Wastewater is processed by directing incoming flow to the influent pump station, which utilizes 4 constant speed submersible pumps (two 10 hp, 1,200 gpm pumps and two 14 hp, 1,750 gpm pumps). From the influent pump station, wastewater is piped to the headworks that includes two grit channels, a bar rack, and comminutor. A splitter weir is used to channel flows to the three 45-foot diameter clarigesters. The clarigesters separate settleable solids and floating debris from the influent wastewater. Settleable solids sink to the digester compartment where they undergo digestion and eventual removal as sludge. Sludge removed from the clarigester is dewatered by means of three covered drying beds. One (95 x 40 feet) is located adjacent to the clarigesters and the other two (39 x 78 feet each) are located near the headworks. The drying bed is divided into 4 bays for operational flexibility. Plant drainage flows from the clarigesters and drying beds to the plant pump station which pumps it back to the headworks. Clarigester effluent flow is taken by gravity to the outlet structure that contains an outfall weir and an ultrasonic level measuring device that allows for the measurement of flow by means of sending a signal to a control panel located in the Operations Room. The signal is converted to a flow rate recorded on a 7 day circular flow chart recorder. A manual transfer switch at the site allows for the connection of a portable diesel powered engine set during power outages.

Installation of a disinfection system in the near future is anticipated as part of the stipulated improvements to the STP contained in the AO. Figure 9 shows a site plan of the treatment plant and Figure 10 shows the process flow or hydraulic profile for the facility.

The treatment plant receives and treats the following wastewaters:

- Domestic sewage collected through the Tafuna sewer system.
- Digested sewage from the Utulei STP sludge drying beds.
- Restaurant grease.

2.1.4 Outfall

The disposal of treated effluent from the Tafuna STP is made through the use of an ocean outfall. The Tafuna outfall is a 24-inch polyethylene (PE) discharge line that extends from the outlet structure at the STP to the diffuser. The outfall line extends over 1,500 feet from shore and discharges plant effluent through a linear diffuser assembly at a depth of 95 feet. The maximum capacity of the outfall is approximately 15 MGD. The outfall location is shown on Figure 2. For a full discussion see Section 10.0.

2.2 Utulei STP Tributary Area

2.2.1 Collection System

The Bay Area sewer system serves the villages in the vicinity of Pago Pago Harbor. These villages are Anua, Atu'u, Faga'alu, Fagatogo, Fatumafuti, Pago Pago, Satala, Utulei, and in the future Onesosopo, Leloaloa, and Aua (see Figure 5). The system serves approximately 7,774[4] individuals or 1,505 housing units. Including the planned area of Onesosopo, Leloaloa, and Aua, the system would serve 10,299 individuals or 1,999 housing units. See Table 2 below for population and housing numbers and areas for each village.

The diameter of the sewer mains varies from 6 to 24 inches. Pipe materials used include ACP, VCP, PVC, and DIP. Force mains vary from 3 to 14 inches in diameter. Service lines are generally four inches in diameter. Manholes are constructed of pre-cast concrete and fiberglass. Similar to the Tafuna system, low-lying manholes have been fitted with removable inserts in selected areas to reduce inflow when water pools over the lid. The total length of the collection system is approximately 76,000 LF.

Table 2 Utulei Tributary Area Statistics

Village	Population[1]	Housing Units[1]	Area (ac)
Existing Areas			
Anua	18	5	32
Atu'u	359	62	21
Faga'alu	910	197	119
Fagatogo	1,737	347	144
Fatumafuti	113	22	10
Pago Pago	3,656	691	259
Satala	297	61	48
Utulei	684	120	200
Total	**7,774**	**1505**	**833**
Future Areas			
Aua	2,077	392	116
Leloaloa	448	102	34
Onesosopo	Included in Aua		11
Total	**2,525**	**494**	**161**
Grand Total	**9,851**	**2,000**	**1,000**

[1] From Census data provided by ASPA – Appendix C, Villages Census Population

2.2.2 Lift Station

The Bay Area system includes seven wet well pump stations with non-clog type Flygt or Hydromatic pumps. The material of construction is typically precast concrete. The smallest facilities are of a package type construction. Manual transfer switches have been provided at the

[4] All population figures were obtained from the 2010 US Census and for the Tafuna area adjusted as necessary by ASPA to more accurately reflect the known service area populations. See Appendix I, Villages Census Population.

larger lift station for the connection of portable emergency generators during power outages. ASPA reports that no permanent engine sets have been installed at the lift stations. The lift stations and tributary areas are shown in Figure 8.

2.2.3 Treatment Plant

Sewer flows collected from the Bay Area collection system are treated at the Utulei Sewage Treatment Plant (Utulei STP). The Utulei STP, which was originally constructed in 1963, is generally located near the south end of Pago Pago Harbor. More specifically, the plant is situated in the village of Utulei, east of the fuel tank farm. The as-built design dry-weather and daily peak design criteria are 2.21 MGD and 6.0 MGD respectively. The plant provides primary treatment only. Wastewater is processed by directing incoming flow to the influent pump station, which utilizes 4 constant speed submersible pumps (two 20 hp, 900 gpm pumps and two 35 hp, 1,600 gpm pumps). The ESV Project includes the modernization of the Influent Pumping Station that includes the replacement of the 4 constant speed pumps with new pumps to be equipped with variable speed drives. The new pumps are all Flygt model NP 3171-34 hp. This work is expected to be completed by the end of 2012. From the influent pump station, wastewater is piped to the headworks which includes an influent screen, grinder and a bar screen. A splitter weir is used to channel flow to four clarigesters. Three are presently used for primary sludge sedimentation and stabilization. The fourth, Clarigester 1, is currently being rehabilitated as part of the Eastside Villages Wastewater Collection System Sewer Upgrade and will be on line in late 2012. Clarigester 1 and 2 are 35 feet in diameter; Clarigesters 3 and 4 are 40 feet in diameter.

Sludge is transported by truck to covered drying beds that are located at the Tafuna STP. Plant drainage flows from the clarigesters to the influent pump station where it is pumped back to the headworks. Clarigester effluent flows are transported by gravity to an existing chlorine contact chamber that is currently unused for disinfection purposes. Primary treated effluent is split and flows through two separate chambers within the Chlorine Contact Chamber where is it recombined in an outlet structure containing an ultrasonic flow measurement device that measures the depth of flow over the discharge weir. A signal from the level measuring device is sent to the control panel located in the Operations Room where it is converted to a flow rate and recorded on a 7 day circular flow chart. Effluent is discharged into the 24-inch diameter ocean outfall line that is located adjacent to the plant. Installation of a disinfection system in the near future is anticipated. Figure 11 shows a site plan of the treatment plant and Figure 12 shows the process flow or hydraulic profile.

The treatment plant receives and treats the following wastewaters:

- Domestic sewage collected through the Utulei sewer system.
- Some sewage from the Ship Yard and Canneries located along the harbor.

2.2.4 Outfall

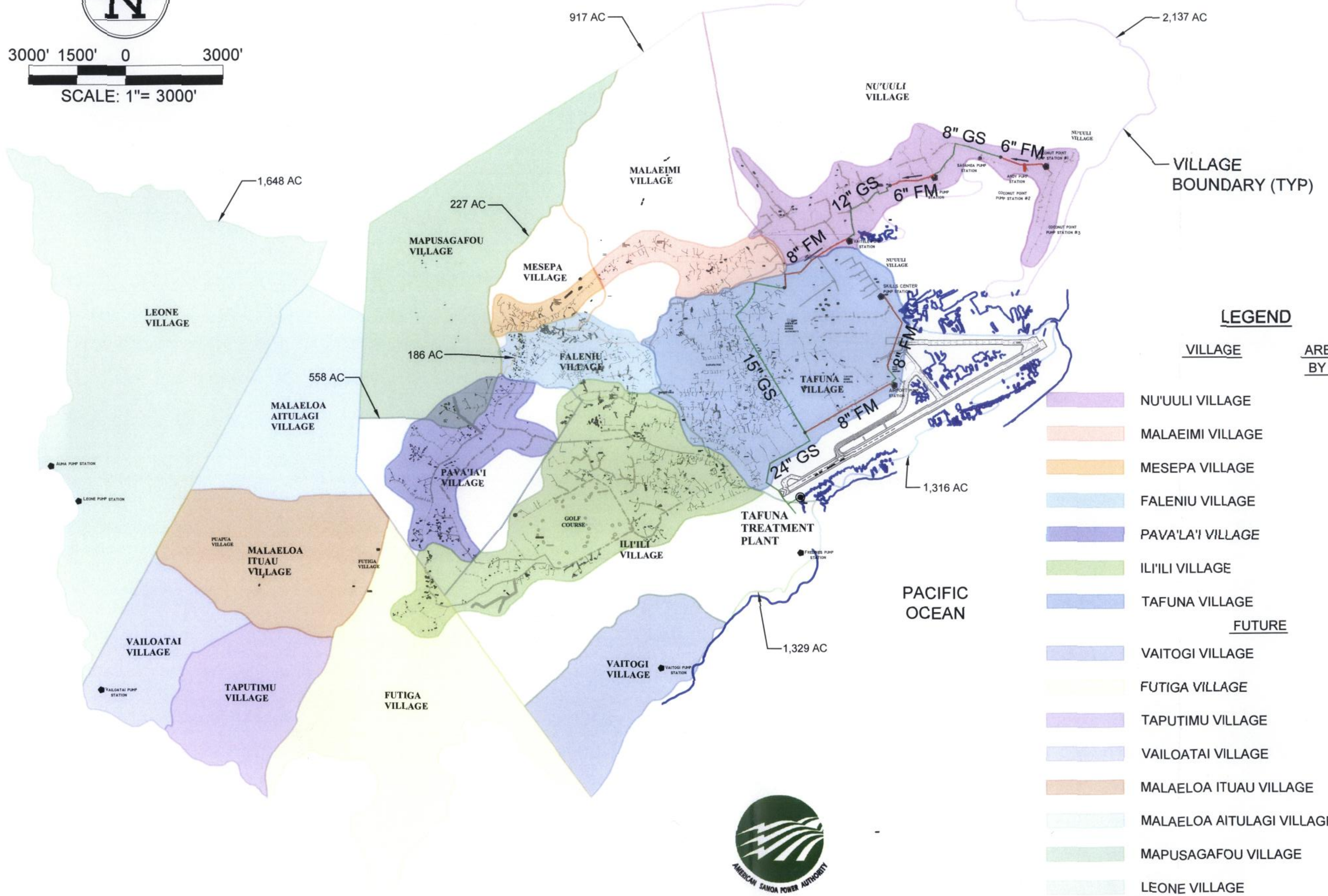
The disposal of treated effluent from the Utulei STP is made through the use of an ocean outfall. The original outfall at Utulei was constructed in the 1960's when the Utulei STP was built. The outfall was 12 inches in diameter and designed for a maximum flow of 2.35 MGD. It was replaced in two phases. The reef top portion of this outfall line was replaced with 24-inch polyethylene (PE) pipe in 1993. This new segment extended approximately 600 feet from the plant outlet structure to the edge of the reef. At this point, the original 12-inch cast iron pipe extended down the reef face and along the bottom for approximately 200 feet. However, a new 24-inch outfall was installed in January 1996 to replace the 12-inch cast iron line. The new outfall extends 270 feet out beyond the reef edge and discharges plant effluent through a linear diffuser assembly at a depth of 150 feet. The maximum capacity of the outfall is approximately 15 MGD. See Figure 3 and Section 10.0 for a full description of the outfall.

2.2.5 Bay Area Environmental Assets

Coral Reef ecosystems are threatened by land-based sources of pollution (LBSP). The United States Coral Reef Task Force (USCRTF) initiated a Watershed Partnership Initiative in 2009. Collaboration through this partnership is intended to increase federal capacity for coral conservation by concentrating upland conservation practices, habitat restoration, and protection efforts to reduce impacts of LBSP on near-shore coral reefs. In 2012, the village of Faga'alu will be recognized at the US Coral Reef Task Force's August meeting that will be held in American Samoa for its watershed program. Faga'alu was designated the Pacific Plus One Watershed under the US Coral Reef Task Force's Priority Watershed Partnership Initiative. Faga'alu is the first village in American Samoa to implement a village-based watershed management program where the village's main environmental concerns were identified and a conservation action plan to protect the natural resources will be realized.

We note that the mitigation of I&I in the Bay Area sewers will reduce impacts to the near shore ocean environment by eliminating the potential for exfiltration of sewage from the collection system.

TAFUNA AREA VILLAGES SERVED BY THE SEWER SYSTEM



LEGEND

VILLAGE	AREA SERVED BY SEWER
NU'UULI VILLAGE	420 AC
MALAEIMI VILLAGE	162 AC
MESEPA VILLAGE	70 AC
FALENIU VILLAGE	200 AC
PAVA'LA'I VILLAGE	328 AC
ILI'ILI VILLAGE	827 AC
TAFUNA VILLAGE	914 AC
FUTURE	
VAITOGI VILLAGE	405 AC
FUTIGA VILLAGE	915 AC
TAPUTIMU VILLAGE	374 AC
VAILOATAI VILLAGE	308 AC
MALAELOA ITUAU VILLAGE	556 AC
MALAELOA AITULAGI VILLAGE	526 AC
MAPUSAGAFOU VILLAGE	892 AC
LEONE VILLAGE	1,650 AC

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FIGURE 5

BAY AREA VILLAGES SERVED BY THE SEWER SYSTEM

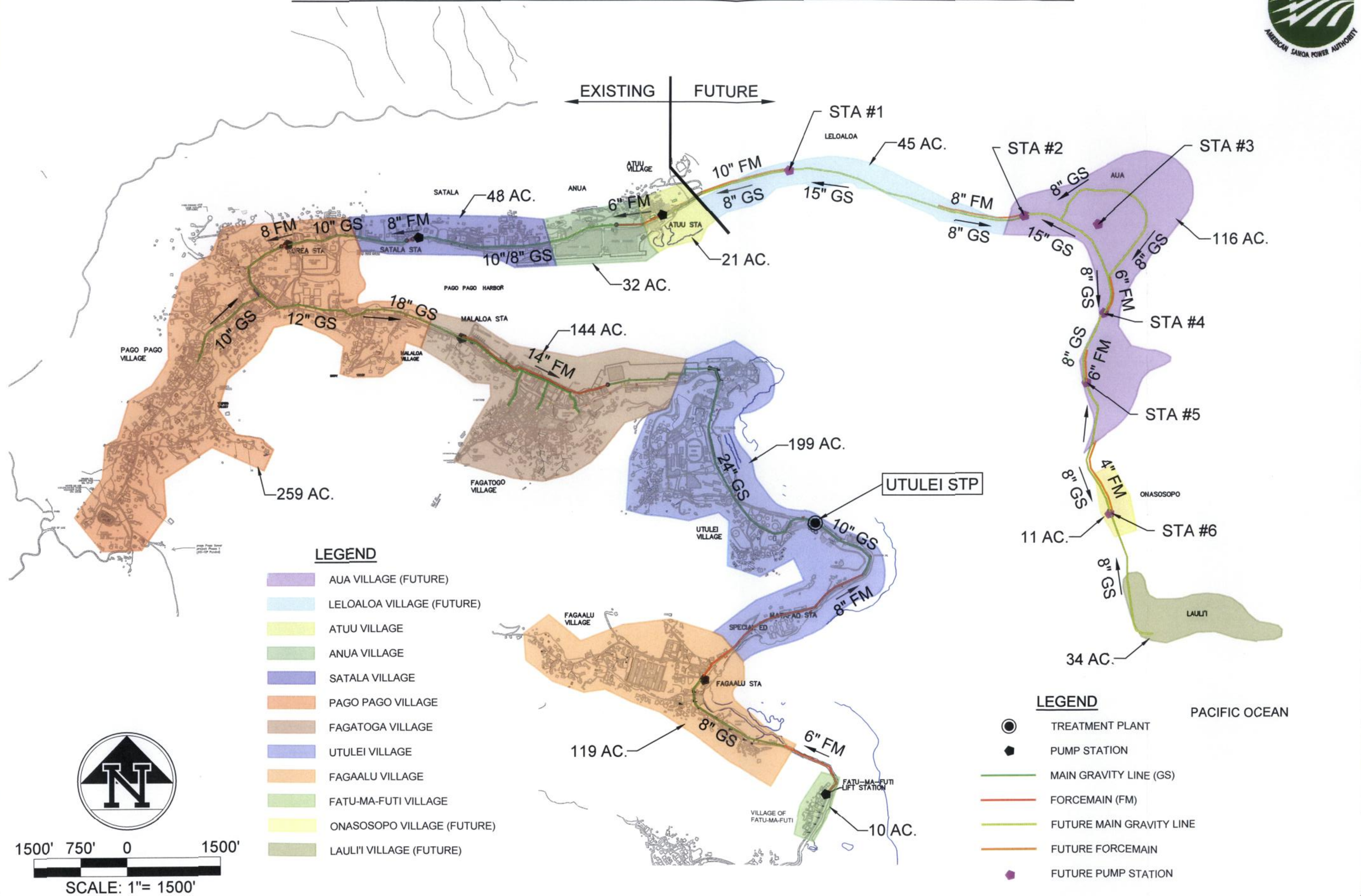

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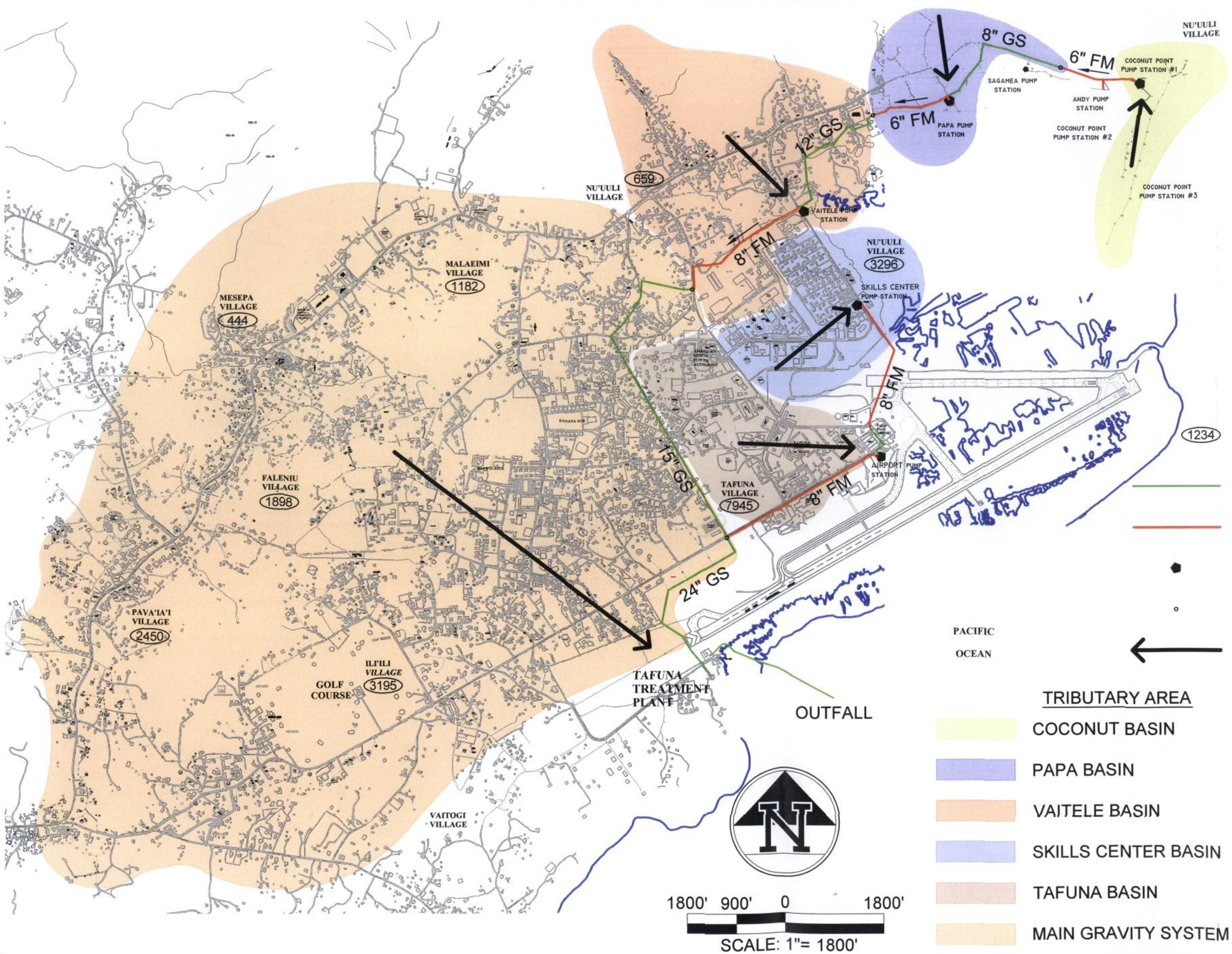
FIGURE
6

TAFUNA SEWER SYSTEM



LEGEND

1234 POPULATION SERVED BY SEWER SYSTEM

MAIN GRAVITY LINE (GS)

FORCEMAIN (FM)

PUMP STATION (P.S.)

JUNCTION MANHOLE

FLOW DIRECTION

TRIBUTARY AREA	DISCHARGES TO
COCONUT BASIN	COCONUT P. S.
PAPA BASIN	PAPA P.S.
VAITELE BASIN	VAITELE P.S.
SKILLS CENTER BASIN	SKILLS CENTER P.S.
TAFUNA BASIN	AIRPORT P.S.
MAIN GRAVITY SYSTEM	TAFUNA S.T.P.

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FIGURE 7

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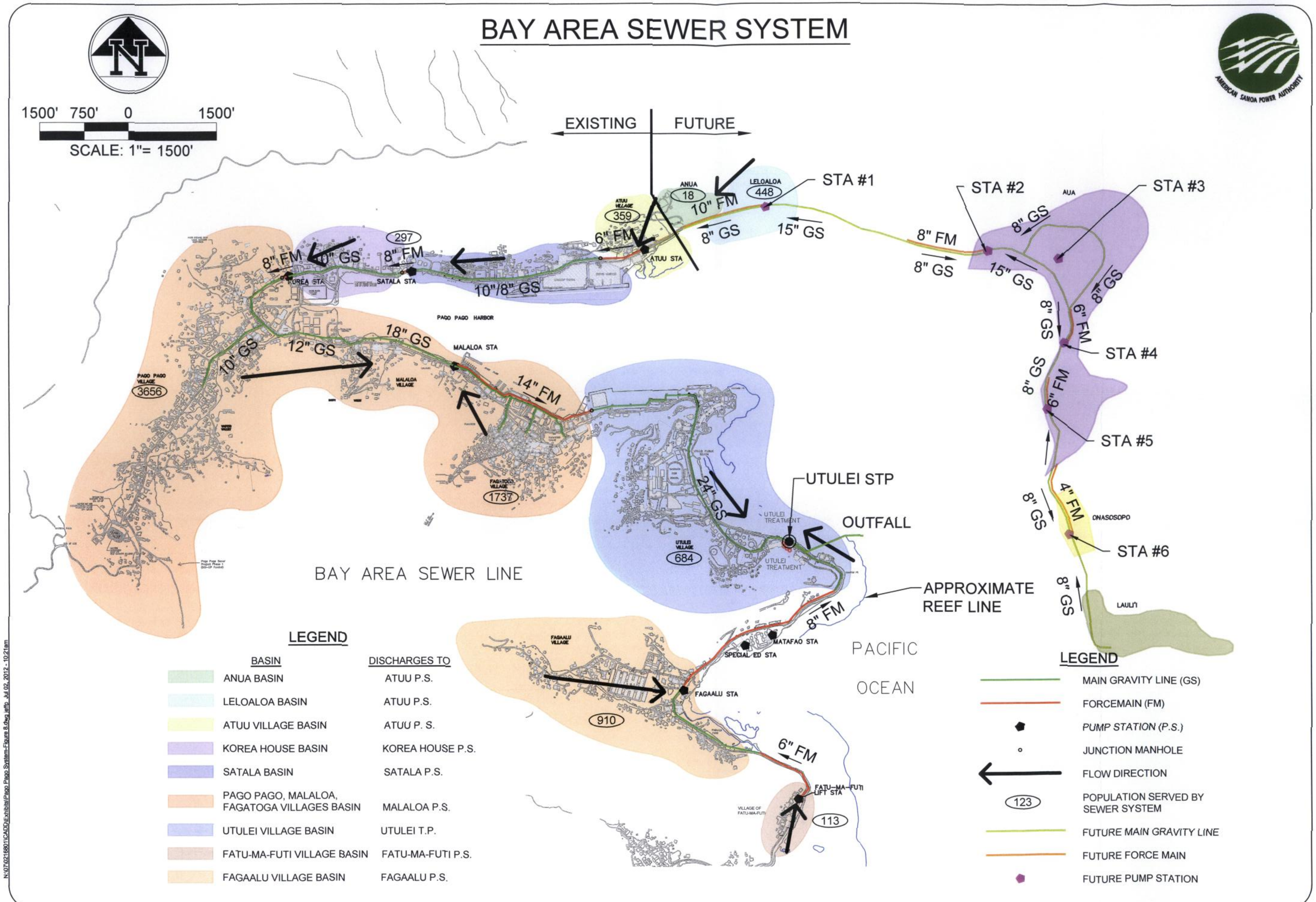

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BAY AREA SEWER SYSTEM

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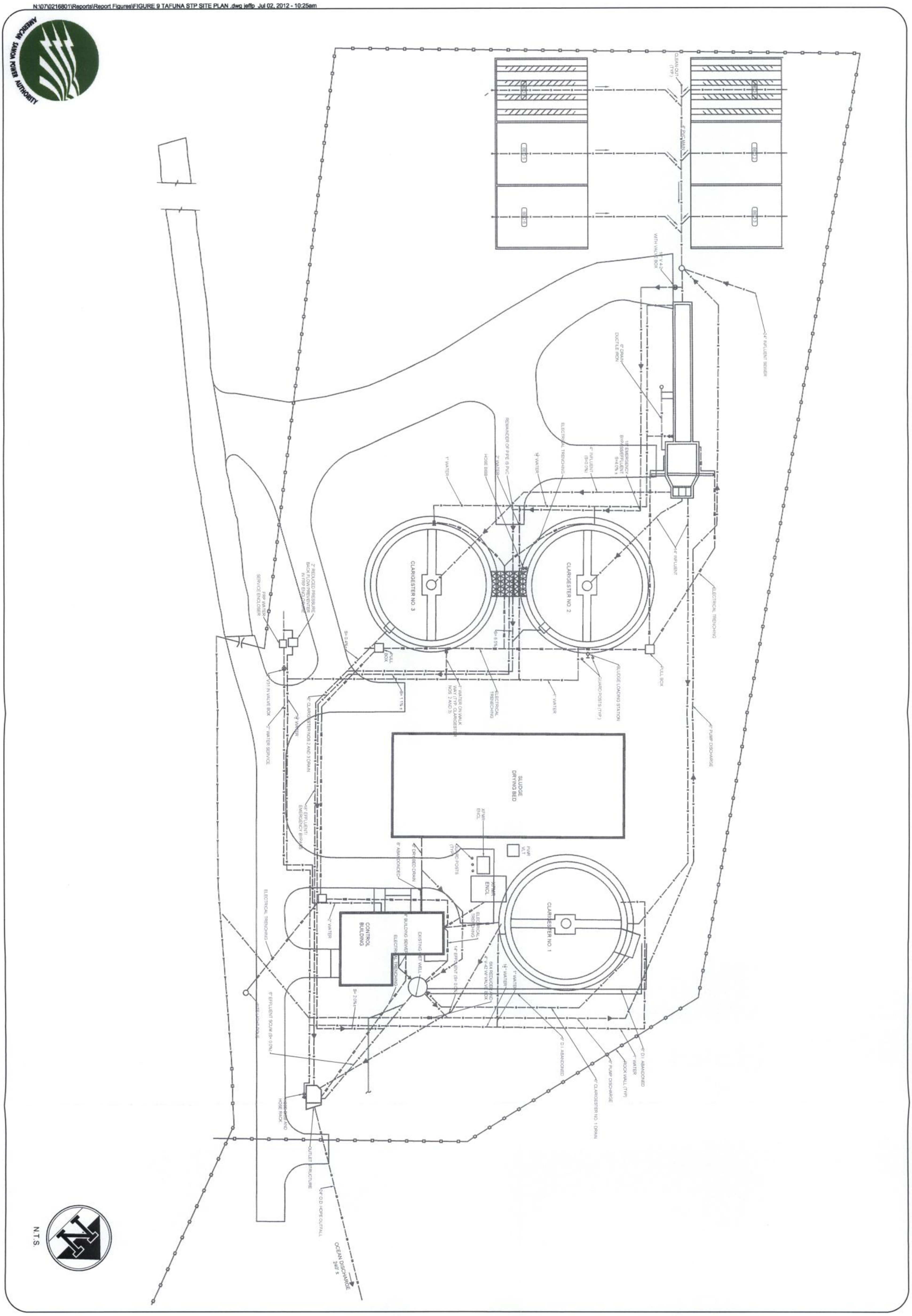
FIGURE
8



JOB NO 07-0216801 FIGURE 9	TAFUNA STP PIPING PLAN	AMERICAN SAMOA POWER AUTHORITY
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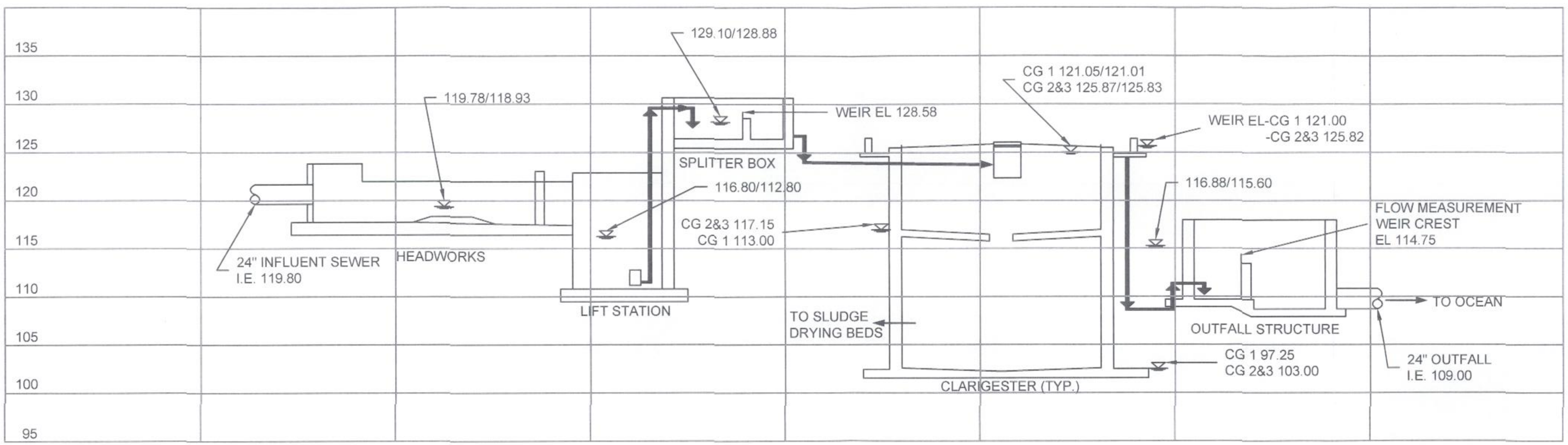

LEGEND

CG CLARIGESTER

000.00/000.00 PEAK WATER SURFACE/MINIMUM WATER SURFACE

→ PROCESS FLOW

TAKEN FROM: WESTERN ENGINEERING INC. FROM THE 1994 WWTP EXPANSION PLANS

AMERICAN SAMOA POWER AUTHORITY

TAFUNA STP HYDRAULIC GRADE LINE

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FIGURE
10

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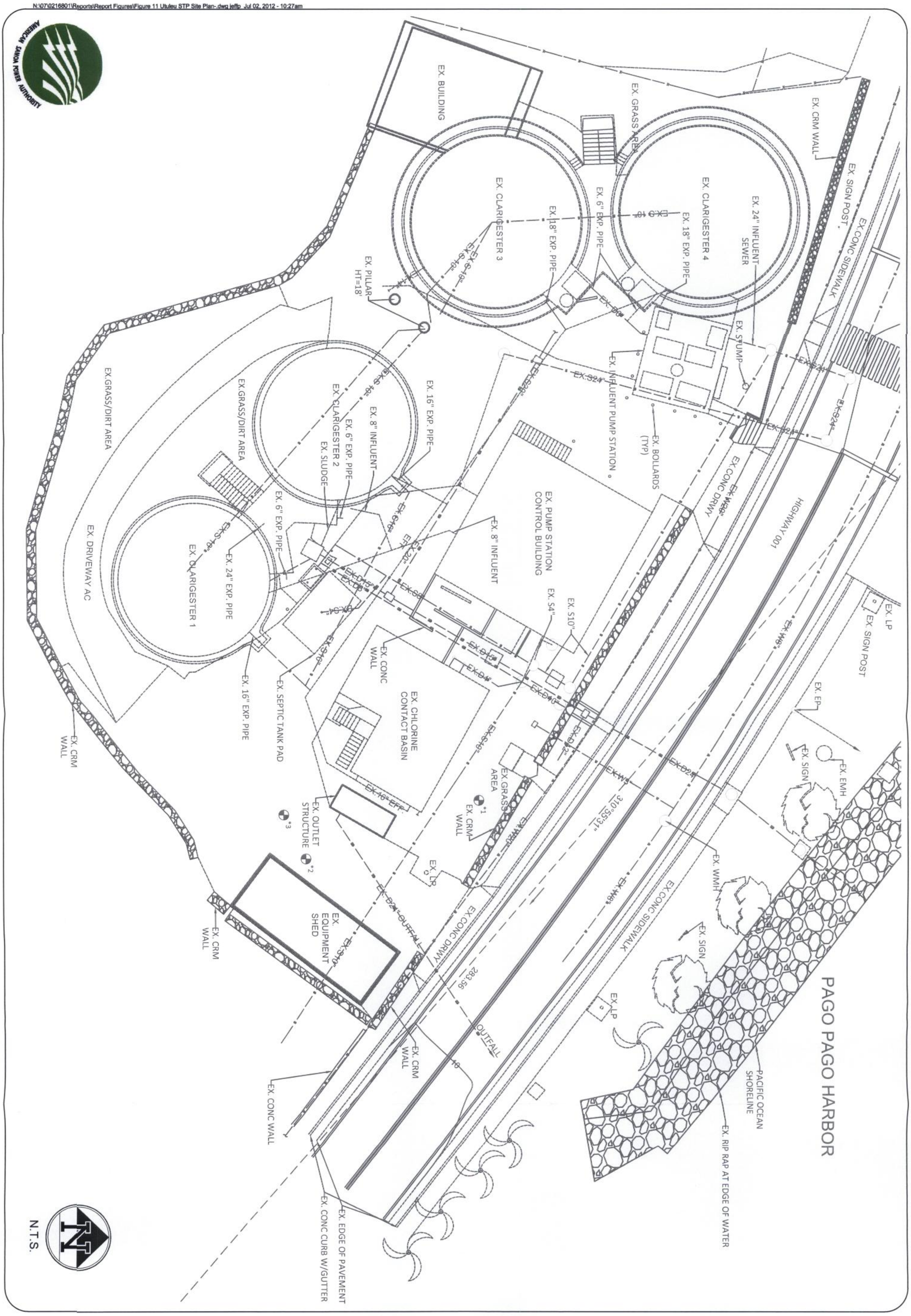
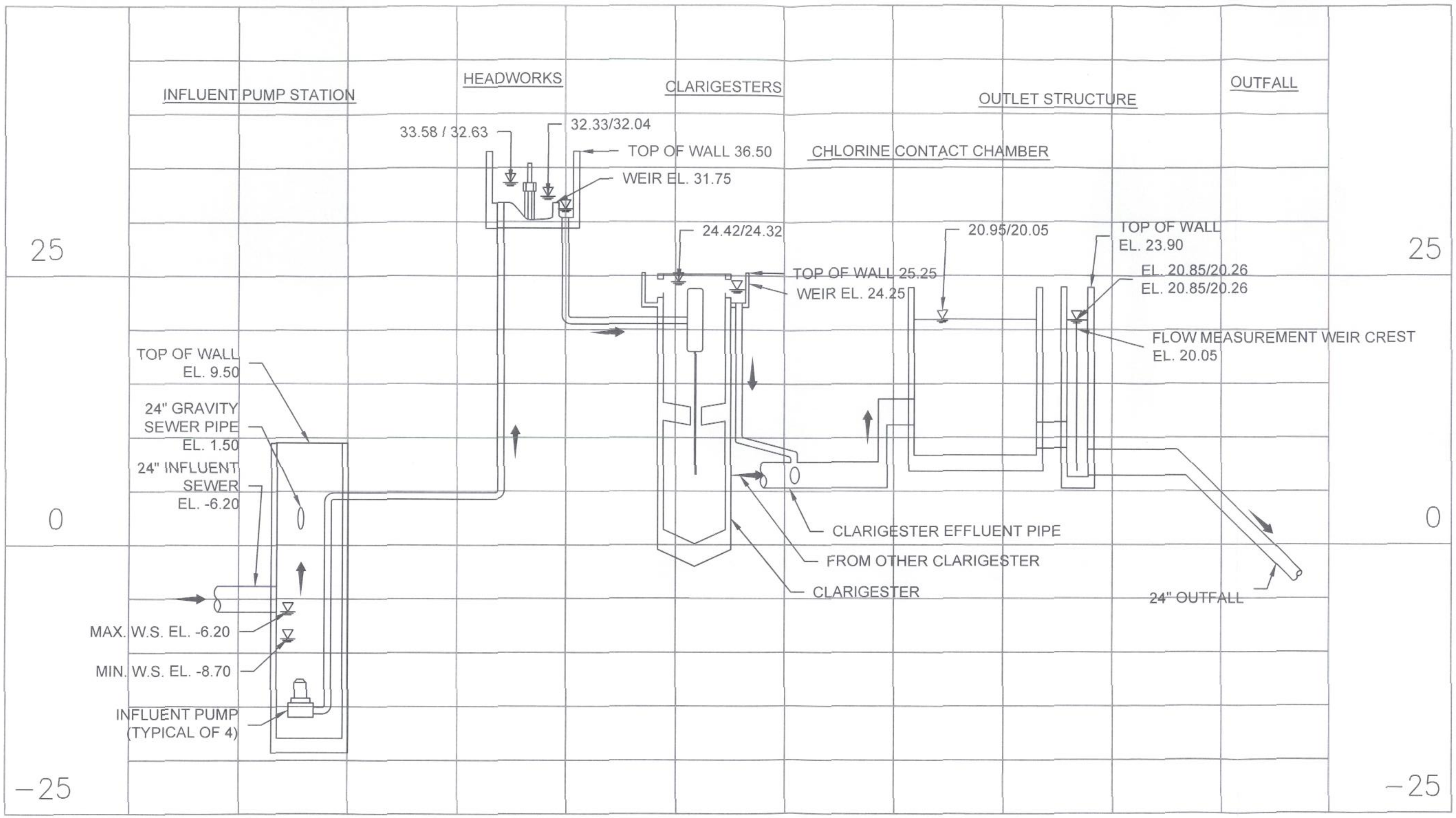


JOB NO 07-026801	UTULEI STP SITE PLAN	AMERICAN SAMOA POWER AUTHORITY
FIGURE 11	4550 NORTH 12TH STREET PHOENIX, ARIZONA 85014 TELEPHONE (602) 264-6831	COE & VAN LOO PLANNING • ENGINEERING • LANDSCAPE ARCHITECTURE



LEGEND

CG CLARIGESTER

000.00/000.00 PEAK WATER SURFACE/MINIMUM WATER SURFACE

PROCESS FLOW

TAKEN FROM: ASPA WASTEWATER ENGINEERING DIVISION 2011 UTULEI WWTP IMPROVEMENTS

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UTULEI STP HYDRAULIC GRADE LINE

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JOB NO
07-0216801

FIGURE
12

3.0 INFILTRATION AND INFLOW ANALYSIS – TAFUNA

3.1 Collection System Methodology

Sanitary sewer gravity lines and pump stations are designed to convey wastewater to the treatment facilities. The introduction of water in the form of infiltration and/or inflow (I&I) from precipitation, groundwater, or seawater into the collection system periodically increases the flow rate and causes conditions outside the threshold of the permitted conditions such as:

- Surcharging Manholes
- Decreased outfall dilution
- Exceedances of hydraulic capacities
- Impacts on treatment process

Flow monitoring combined with precipitation monitoring is a basic method for the quantification of I&I entering a sanitary sewer system. The purpose is to measure flow fluctuations with precipitation. Seawater intrusion can be measured by comparing the conductivities and/or chloride concentrations of the sewage flows at various points in the system and at the STP. The characteristics of the flow changes can often be related to the pattern of precipitation and/or seawater conditions to quantify inflow and infiltration rates.

ASPA staff has identified several areas in either system that have increased occurrence of inflow and infiltration (I&I). These areas are shown in Figure 14. Known areas of I&I contributions should be characterized by ASPA and eliminated through a long term, dedicated capital improvement program. According to the Pedersen 2003 Report, the ASPA Wastewater Division owns a TV camera, smoke testing equipment, as well as a portable high-velocity cleaner that can be used to detect leaks in the collection system and be used to identify areas needing repair.

3.2 Analysis

Flow data was made available by the American Samoa Power Authority (ASPA). Flow data consists of measured effluent flow at the treatment plant. Flow data from lift stations in the form of pump run time was discovered unreliable and not used in this study. Flows arriving at the treatment plant include flows from the whole system including flows conveyed through one or a series of lift stations and flows conveyed by gravity to the facility. Precipitation data was collected from NOAA. The relative proportions of seawater and groundwater to the total I&I are not known and were not calculated for this analysis. For an estimate of seawater intrusion as determined by sampling for chlorine/conductivity in the inflow to both STPs, see Section 3.5, Seawater Intrusion.

3.2.1 Base Flow

Base flow data was collected for the most recent extended periods that experienced minimal rainfall. Dates used to estimate the Tafuna System base flows were obtained from NOAA rainfall data and are identified below:

- September 15, 2009, through October 12, 2009
- September 15, 2011, through October 12, 2011

The available data was given as daily effluent flow totals. The average flow from these periods was determined to represent the collection system base flow for this study. It is noted that this base flow also includes an undetermined amount of I&I from groundwater and seawater intrusion. It is assumed that the amount of I&I is at minimal quantities during these dry weather periods. This base flow only serves to establish a magnitude of increased I&I observed during wet weather periods. See Appendix D, Tafuna Flow Data, for complete data on periods chosen for base flow estimation.

3.2.2 Wet Weather Flow

Flow data was collected for periods with significant rain events. These periods included days with rain preceding and/or following major rain events with accumulated precipitation of 5 inches or more in one day during the period. Dates selected for the Tafuna System are:

- December 8, 2009, through December 11, 2009
- December 25, 2009, through December 31, 2009
- January 21, 2010, through January 27, 2010
- January 19, 2011, through January 26, 2011

Data was obtained from the STP DMRs reported as daily effluent flow totals with a maximum value flow for each day. The average of each observed period was calculated and the maximum flow of any day during the period was selected as the period maximum flow. Table 3 demonstrates the type of data that was available for the analysis. This is an example of data for the January 2010 Tafuna system rain event.

Table 3 Example data for the January 2010 Rain Event From NOAA for the Tafuna System

Date	Daily Flow Total (MGD)	Maximum Flow (MGD)	Total Precipitation (inch)	Max Hourly Precipitation (inches)
January 21, 2010	2.5	4.3	0.38	0.27
January 22, 2010	2.5	4.3	0.37	0.12
January 23, 2010	2.8	Not Reported[1]	0.32	0.24
January 24, 2010	2.7	4.6[2]	6.91	0.89
January 25, 2010	2.6	4.6	2.19	0.44
January 26, 2010	2.9	4.6	1.28	0.25
January 27, 2010	3.1	5.0	5.50	1.73
Period	Average 2.7	Maximum 5.0	Total 16.95	Maximum 1.73

[1] Not reported = not recorded in DMRs.
[2] Not reported in DMRs but provided by ASPA personnel from weekly circular flow charts (records instantaneous flows)

Table 3 illustrates how periods of wet weather flows were determined. A major rain went with daily accumulated precipitation of 5 inches or more was recorded. Days with rainfall directly prior to the event were included up to a day with no rainfall or rainfall of less than 0.15 inch. Days with rainfall directly following the event were also included up to a day with no rainfall.

This process was repeated for all four periods above.

3.3 Discussion

Table 4 lists the results of the evaluation of flow increases during rain events at the Tafuna STP. For major rain events (total rainfall for one day exceeding 5 inches) the increase in flow rates measured at the plant is significant. The increase also appears to correlate well with the severity of the rain event which is noted in this report as the ratio of the maximum daily rainfall to the rainfall average for the period. The base flow derived from the DMR flow data at the treatment plant is 1.6 MGD as noted in Table 4. I&I in the Tafuna collection system increases flows by an estimated 25 to 69% during a month with high precipitation.

Rainfall intensities will have an impact on I&I rates into the system. CVL believes that the available flow data does not permit a correlation of rainfall intensity vs. sewage flows.

The percent increase was calculated using the following equation:

Percent (%) increase in average flow:

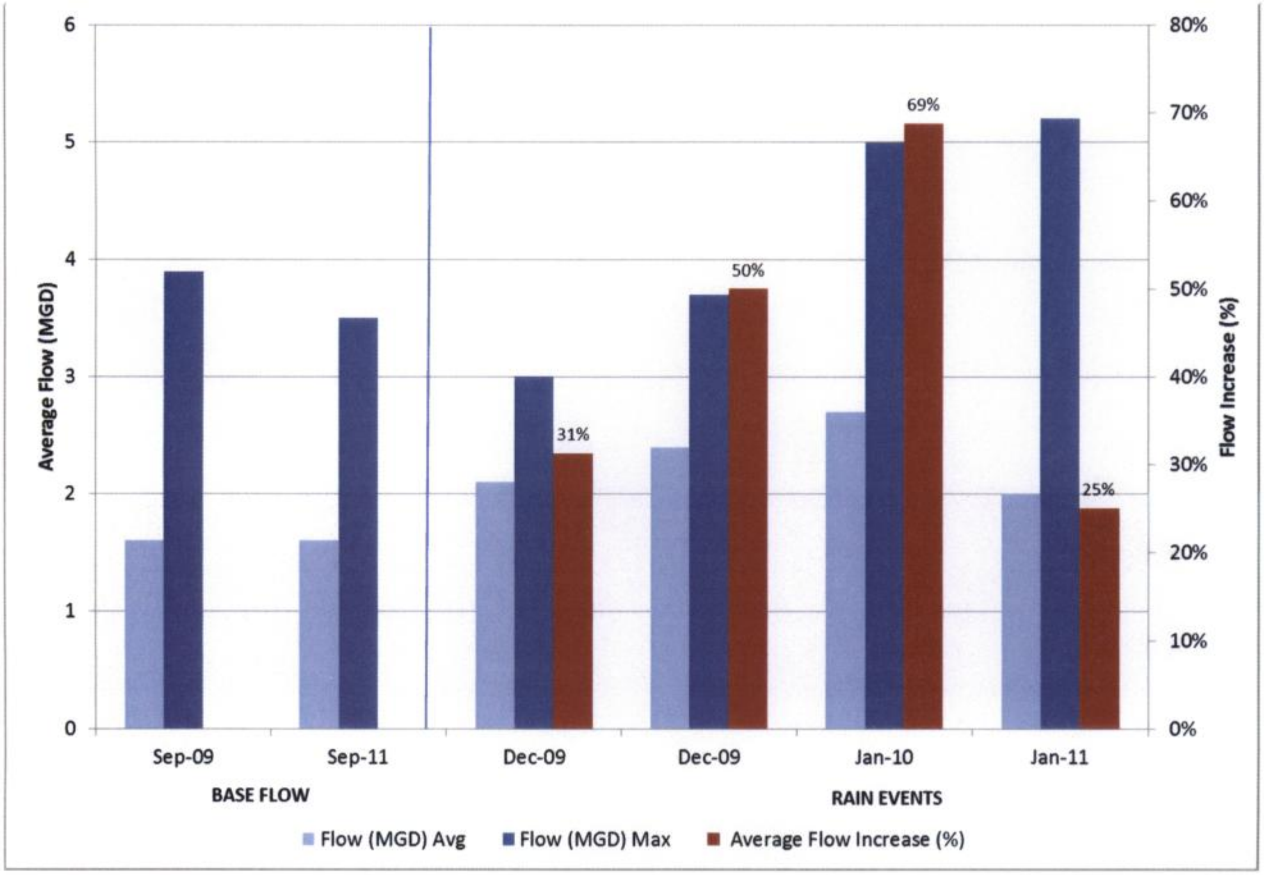
$$Increase = \frac{(Event\ Average - Base\ Average)}{Base\ Average}$$

Table 4 Tafuna Summary I&I Contribution

Event Type	**Period**	**Precipitation (inches)**			**Flow (MGD)**		**Avg Flow Increase**
		Avg	Max	Total	Avg	Max	(%)
Base Flow	9/2009			0.43	1.6	3.9	
Base Flow	9/2011			0.16	1.6	3.5	
Base Flow	**Average**				**1.6**		
Rain Event	12/2009	2.5	6.6	10.11	2.1	3.0	31
Rain Event	12/2009	2.2	5.4	15.55	2.4	3.7	50
Rain Event	1/2010	2.4	6.9	16.9	2.7	5.0	69
Rain Event	1/2011	1.7	5.9	13.9	2.0	5.2	25

Figure 13 and Table 4 show the estimated I&I calculated for those periods of high rainfall. The increase in average daily flows compared to the estimated base flow is 25% to 69% for the Tafuna sewer collection system.

Figure 13 Tafuna Summary I&I Contribution



Review of Table 4 and Figure 13 reveals that the average and maximum day flows measured during the dry weather or base flow events to be similar for the two periods investigated. Average day flows for both periods are approximately 1.6 MGD. Maximum day flows are 3.5 and 3.9 MGD. The peaking factor of Max Day to Average Day ranges from 2.2 to 2.4. A peaking factor of 2.3 will be used to generate the Max Day value of 3.7 MGD for the base flow periods.

The wet weather flows measured during the three time periods exhibit a greater degree of variability. Some factors contributing to this variability include the length of rainfall events, the duration of high intensity rainfall, antecedent soil moisture conditions and the frequency of given storms during the period of interest. It is noted that long periods of steady, moderate rains generally favor infiltration into the collection system because runoff is slowed, allowing for percolation into the soils and entry into the sewers through open joints or cracked pipe. Higher intensity rainfalls cause more runoff and accumulation of water in low spots and generalized flooding. The poor drainage noted in the service area and subsequent pooling and ponding of water in the roadways and adjacent lands would result in greater inflow into the collection system through manhole covers submerged in these ponds and leakage in cracked or damaged masonry supporting the manhole frames. The amount of precipitation within a period is not a sufficient indicator of I&I by itself and the maximum rainfall measured during each period was also considered in calculating a Maximum Day Flow value for a wet weather event.

It has already been noted that Average Daily flows as measured during the selected wet weather periods increase from 25% to 69% above the Average Daily flows measured during dry weather periods. In an effort to select a representative figure to be used in calculating typical I&I flows for this study, CVL selected the January 2010 event as a conservative approach to determining the expected increase of I&I during wet weather periods. In this period, both high intensity rainfalls and large amounts of rain were recorded, maximizing the potential for I&I. The wet weather Average Day flows are taken to be 69% greater than a comparable dry weather period for a total value of 2.7 MGD.

Review of the data in Table 4 and Figure 13 indicates that the peaking factors for the four wet weather events range from 1.4 to 2.6. Knowing that a peaking factor of 2.3 was calculated for the dry weather period as discussed above and assuming that the typical diurnal cycle would not be necessarily impacted by a wet weather period, the calculated peaking factor 2.3 was used to compute a Maximum Day flow of 6.2 MGD (2.7 MGD x 2.3). It is noted that this value is above the Maximum Day flows recorded during the three wet weather periods, but, in reviewing DMRs for the 2008 to 2012 period, several instances of maximum day flows were found above the 5.2 MGD recorded. Wet weather flows should be representative of a worst case scenario to not understate peak wet weather flows. Assuming the same peaking factor for wet weather as for dry weather calculates a reasonable value as indicated by DMRs. We believe that the use of a Maximum Day flow of 6.2 MGD to the Tafuna STP will be sufficiently conservative to adequately model expected wet weather I&I flows and serve as the basis for the analysis of measures to mitigate excessive I&Is in the collection, lift station and STP systems.

Lift station run time data was also collected to compare rain events with increased flow at lift stations throughout the system to determine those areas more prone to I&I. Due to the recording method of lift station run times this data did not correlate reliably with flows measured at the treatment plants and was subsequently not included in the evaluation and the report. See discussion in Section 6.2 for further information.

3.4 Tafuna Results Summary

Table 5 summarizes the flows to be used in our analyses as discussed above.

Table 5 Tafuna Flow Summary Table

	Dry Weather	Wet Weather
Average Day Flow	1.6 MGD	2.7 MGD
Peak Flow	3.7 MGD	6.2 MGD
Peaking Factor	2.3	2.3

3.5 Seawater Intrusion

The existence and magnitude of seawater intrusion in the Tafuna STP collection system was estimated by measuring conductivity and salinity of the STP effluent and tap water at the STP site. These were compared to determine if an increase in salinity was present in the wastewater. The salinity of bottled water was also measured as a control for this sampling exercise. Increased salinity in the wastewater would indicate seawater infiltration into the collection system between the customer and the treatment plant.

Measurements were conducted by GDC with an YSI 30 meter on June 5, 2012 and are provided in Table 6 below. Precipitation was substantial prior to measuring and the tide was near low water at the time measurements were done at Tafuna.

Table 6 Results of Salinity Measurements of the Tafuna STP Effluent:

Sample	Date	Time	Temperature °C	Specific Conductivity µS @ 25 °C	Conductivity µS	"Salinity" ppt
Tap Water	5 June 2012	15:30	21.5	1075	996	0.5
STP Effluent	5 June 2012	13:25	21.2	1600	1500	0.8
Bottled Water	5 June 2012	15:45	24.8	--	320	--

The measurements at Tafuna indicate only a slight elevation above expected conductivity but could indicate a potential for minor infiltration.

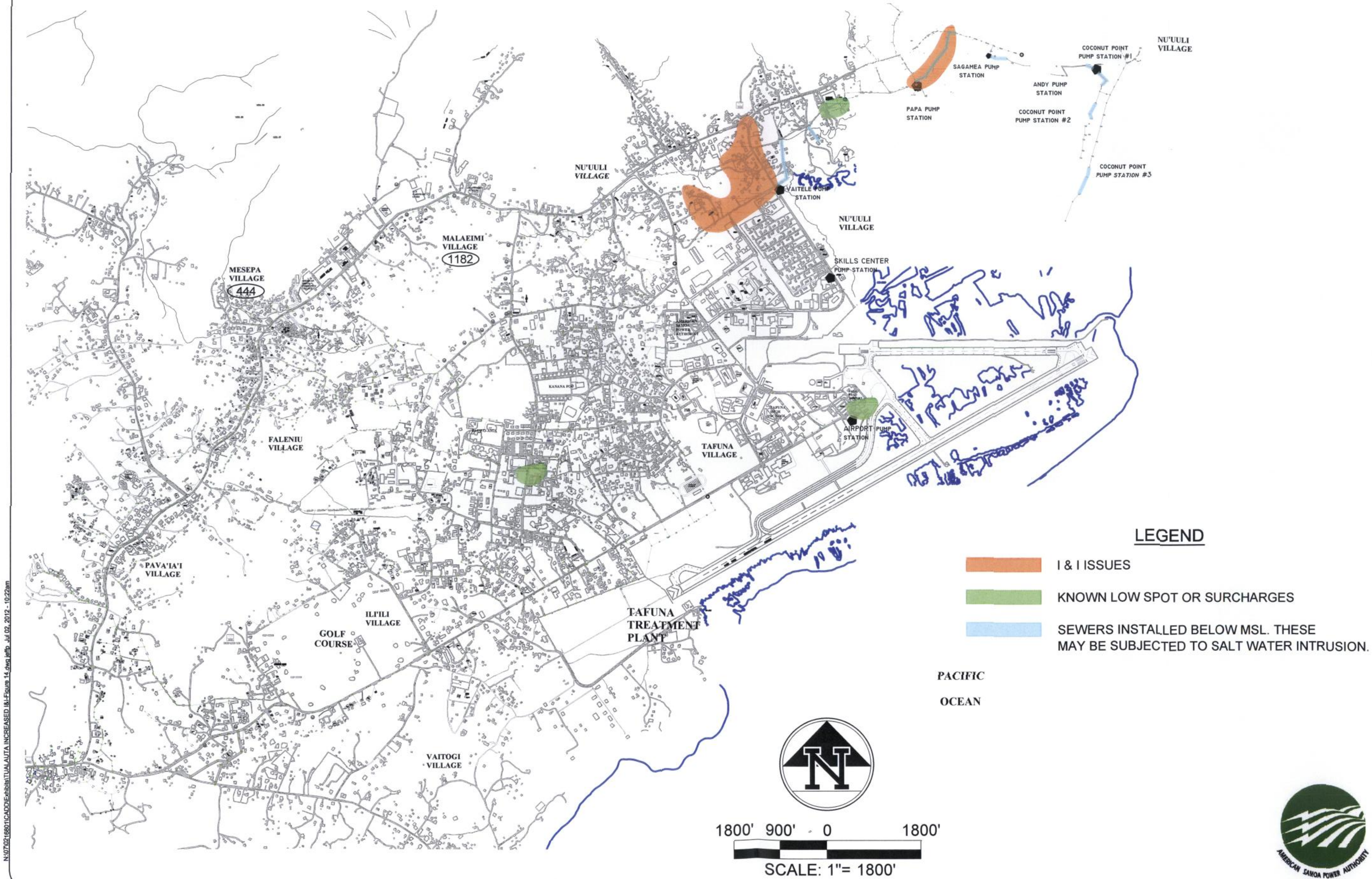
The tap water at a local hotel (Lefalepule – served by the Bay Area collection system) was tested and bottled drinking water was also tested. The results indicate the meter was working as expected. The zero offset of less than 20 µS, based on readings in air, indicate that the meter, although calibrated for seawater, is reasonably linear over the entire range.

CVL assumes there are no other significant sources adding salt to the sewer flows such as water softeners or industrial processes.

3.5.1 Sewers Located Below Sea Level

The Tafuna system has a few areas where sewers are installed along the coastal line below sea level. These sewers could be subjected to saltwater intrusion. The locations of sewers below sea level were determined from plans provided by ASPA and are shown on Figure 14. The total length of identified sewers likely subjected to this I&I is approximately 4,440 feet.

TAFUNA SYSTEM - AREAS OF SUSPECTED I&I



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TAFUNA SYSTEM
AREAS OF SUSPECTED I&I

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JOB NO.
07-0216801

FIGURE
14

4.0 INFILTRATION AND INFLOW ANALYSIS – UTULEI

The "Small Community Wastewater Facilities Plan" (GDC, 2003) and our discussions in Section 3.0 have clearly established a relationship between rainfall and increased wastewater flows resulting from I&I. In this section, the relationship between rainfall and I&I flows experienced in the Bay Area collection system is studied.

4.1 Collection System Methodology

The methodology to analyze I&I is discussed in detail in Section 3.1 and also applies to the Bay Area System. ASPA has identified known areas of I&I for this service area, see Figure 16.

4.2 Analysis

Flow data and precipitation data for the analysis were obtained as described in Section 3.2.

4.2.1 Base Flow

Base flow data was collected for the same recent periods experiencing minimal rainfall as for the Tafuna system. Dates used to estimate the Utulei System base flows are:

- September 15, 2009, through October 12, 2009
- September 15, 2011, through October 12, 2011

See Appendix E, Utulei Flow Data, for complete data on periods chosen for base flow estimation.

4.2.2 Wet Weather Flow

Flow data was collected for the same periods with significant rain events as for the Tafuna system (see Section 3.2.2.). The dates are:

- December 8, 2009, through December 11, 2009
- December 25, 2009, through December 31, 2009
- January 21, 2010, through January 27, 2010
- January 19, 2011, through January 26, 2011

Data was obtained from the STP DMRs reported as daily effluent flow totals with a maximum value flow for each day. The average of each observed period was calculated and the maximum flow of any day during the period was selected as the period maximum flow. Table 7 demonstrates the type of data that was available for the analysis. This is an example of data for the January 2010 Utulei system rain event. See Appendix E, Utulei Flow Data, for data on all selected rainfall periods.

Table 7 Example data for the January 2011 Rain Event for the Utulei System

Date	Daily Flow Total (MGD)	Maximum Flow (MGD)	Total Precipitation (inch)	Max Hourly Precipitation (inches)
January 19, 2011	1.5	2.0	0.28	0.07
January 20, 2011	1.7	3.6	0.89	0.55
January 21, 2011	2.3	3.6	1.92	0.35
January 22, 2011	3.3	Not Reported[1]	2.17	0.64
January 23, 2011	3.3	Not Reported[1]	1.54	0.28
January 24, 2011	3.3	4.8	5.85	1.04
January 25, 2011	4.0	4.2	0.77	0.38
January 26, 2011	2.8	3.1	0.48	0.29
Period	Average 2.8	Maximum 4.8	Total 13.9	Maximum 1.04

[1] Not reported = not recorded in DMRs.

4.3 Discussion

Table 8 lists the results of the evaluation of flow increases during rain events at the Utulei STP. For major rain events (total rainfall for one day exceeding 5 inches) the increase in flow rates measured at the plant is significant. The increase also appears to correlate well with the severity of the rain event which is noted in this report as the ratio of the maximum daily rainfall to the rainfall average for the period. The base flow derived from the DMR flow data at the treatment plant is 1.0 MGD as seen in Table 8. I&I in the Utulei collection system is estimated to increase flows from 40% to 180% during the wet weather periods studied.

The percent increase for average day flows was calculated as discussed in Section 3.3.

Table 8 Utulei Summary I&I Contribution

Event Type	Period	Precipitation (inches)[1]			Flow (MGD)		Avg Flow Increase
		Avg	Max	Total	Avg	Max	(%)
Base Flow	9/2009			0.43	0.69	3.2	
Base Flow	9/2011			0.16	1.31	3.5	
Base Flow	**Average**				**1.0**		
Rain Event	12/2009	2.5	6.6	10.11	1.4	2	40%
Rain Event	12/2009	2.2	5.4	15.55	0.9	3.2	-10%[2]
Rain Event	1/2010	2.4	6.9	16.9	2.0	3.7	100%
Rain Event	1/2011	1.7	5.9	13.9	2.8	5.0	180%

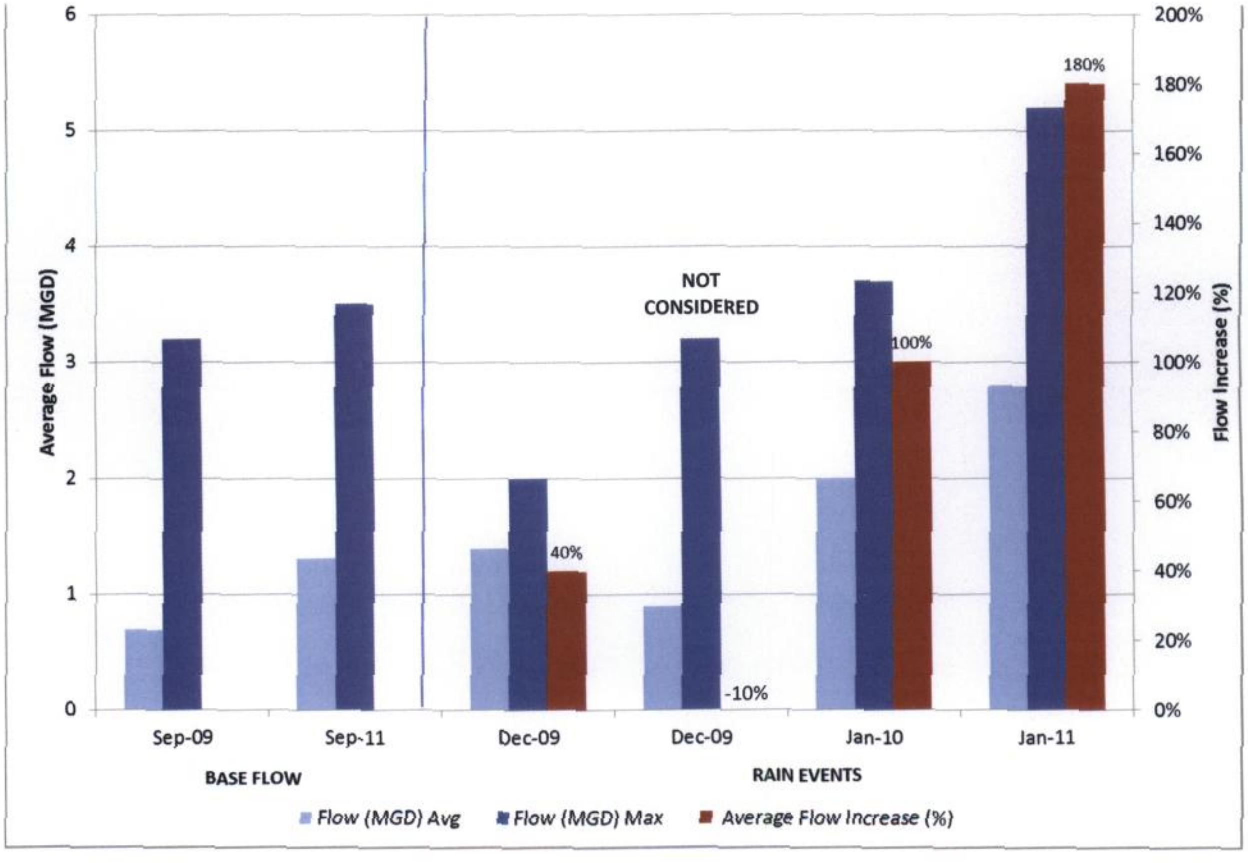
[1] For the period described in Section 4.2
[2] This rain event period will not be considered

During the base flow period in September 2009 the Island was subjected to a major Tsunami. The data indicates some irregularities. The peaking factor for the 2009 base flow period is calculated to be 4.6 which appears to be very high. As explained below a different approach is appropriate in calculating representative flows.

The second December 2009 rain event period also includes data that seems to be corrupted in some way. During this rain event period the average flows decreased by 10% as compared to the average base flow value. We also note that no flows were recorded for maximum day flows for 25-27 December indicating a potential issue in flow data collection during a period of heavy rain over a three day period. This period will be excluded from the calculations for the Utulei system.

Figure 15 and Table 8 show estimated I&I calculated for these periods of high rainfall. The increase in average daily flows compared to the estimated base flow is 40% to 180% for the Utulei sewer collection system.

Figure 15 Utulei Summary I&I Contribution



Review of Table 8 and Figure 15 reveals that the average day flows measured during the dry weather or base flows to vary significantly for the two periods investigated. Average day flows for the two periods range from 0.7 to 1.3 MGD with an average of 1.0 MGD. Maximum day flows are quite similar and range from 3.2 to 3.5 MGD. The peaking factor of Maximum to Average Day ranges from 2.7 to 4.6 indicating that the Utulei collection system exhibits a greater variability in its diurnal flows. A peaking factor of 3.7 calculated by averaging the values from the base flow events would result in an estimated peak day dry weather flows of 1.0 x 3.7 = 3.7. This value appears to be too high for dry weather flows. Recorded flows for Utulei since the Tsunami event indicate an Average Maximum value of 3.9 MGD for all wet and dry periods (see Appendix F, Average and Maximum Flows at STP 2006-2011). The use of the 3.7 peaking factor will overstate the dry weather Maximum Flow. For Utulei a Peaking Factor value of 3.4 was selected which is the average of the two recorded maximum flow values for the dry weather periods. The expected Maximum Day flow to the plant during base flow periods is then 3.4 MGD.

As discussed in Section 3.3, wet weather flows exhibit a great amount of variability owing to rainfall types and intensities. A similar approach was taken to determine an Average Daily Flow for the Utulei collection system during wet weather periods. It has already been established that Average Daily Flows increase from 40% to 180% above Average Daily Flows observed during dry weather periods (see Table 8). CVL selected the January 2011 wet weather event as being sufficiently conservative to determining the I&I flow increases during wet weather. This period exhibits both high intensity and long duration rainfalls. The wet weather Average Day Flows are taken to be 180% greater than a comparable dry weather period for a total value of 2.8 MGD as taken from Table 8.

Taking the previously derived peaking factor of 3.4, the Maximum Daily Flow during wet weather periods is calculated to be 9.5 MGD (2.8 MGD x 3.4). This value is significantly above the Maximum Day flows recorded during the three wet weather periods and also higher than any maximum flows recorded at Utulei during the last 5 years. The highest maximum flow recorded at Utulei from 2007 to 2011 was 6.0 MGD in July 2010 as derived from the DMRs. This period exhibited small but steady rainfall. CVL selects 6.0 MGD as the wet weather peak day flow value as being appropriate for the purposes of the study given the data available. The focus of this section was to establish representative flows. A peaking factor was not determined for dry and wet weather flows for the Bay Area system.

Lift station run time data was also collected to compare rain events with increased flow at lift stations throughout the system to determine those areas more prone to I&I. Due to the recording method of lift station run times this data did not correlate reliably with flows measured at the treatment plants and was subsequently not included in the evaluation and the report. See discussion in Section 7.2.

4.4 Utulei Results Summary

Table 9 Utulei Flow Summary Table

	Dry Weather	Wet Weather
Average Day Flow	1.0 MGD	2.8 MGD
Peak Flow	3.4 MGD	6.0 MGD

4.5 Seawater Intrusion

4.5.1 Seawater Intrusion Measurements

As for the Tafuna System, (see Section 3.5), measurements were conducted by GDC with an YSI 30 meter on June 5, 2012 and are provided in Table 10 below. Precipitation was substantial prior to measuring and the tide was mid water at the time of the measurements at Utulei.

Table 10 Results of Salinity Measurements of the Utulei STP Effluent:

Sample	Date	Time	Temperature °C	Specific Conductivity µS @ 25 °C	Conductivity µS	"Salinity" ppt
Tap Water	5 June 2012	15:30	21.5	1075	996	0.5
STP Effluent	5 June 2012	11:15	21.2	8000	7400	4.4
STP Effluent	5 June 2012	11:20	21.5	8100	7600	4.5
Bottled Water	5 June 2012	15:45	24.8	--	320	--

The measurements at Utulei indicate a "salinity" equivalent to about 12% seawater. Salinity at these levels is not necessarily or strictly equivalent to actual seawater - but seawater intrusion is the only reasonable mechanism to explain the results. The "salinity" measurements indicate seawater infiltration to the collection system and lift stations along the Harbor. The actual salinity of the ground water is not known, and may be mixed with freshwater, so the 12% value might be considered a minimum estimate of infiltration.

4.5.2 Sewers Located Below Sea Level

The Bay Area system has several stretches of sewers along the coastal line that are installed below sea level. These sewers are likely subjected to saltwater intrusion. As built plans were available for the area ranging from the Malaloa lift station to the Utulei STP. The location of sewers below sea level is shown on Figure 16. The total length of identified sewers likely subjected to I&I is 11,181 feet. This number is likely substantially larger and likely includes portions of coastal sewer on the north side of the Bay and also south of the Utulei STP.

5.0 INFILTRATION AND INFLOW RESULTS

5.1 I&I Calculations

The system wide I&I quantities for each service area were calculated by subtracting the dry weather day flow from the calculated wet weather flows and dividing the resulting number by the entire area served in acres where flows were obtained from Sections 3.0 and 4.0. As an example, the wet weather Average Day for Tafuna is 2.7 MGD and the dry weather flows are 1.6 MGD:

$$Tafuna\ System\ I\&I = 2.7 - 1.6 = 1.1\ MGD$$

$$I\&I\ per\ Area = \frac{1.1\ MGD}{2{,}921\ Acres} = 380\ gpad\ (see\ \text{Table 11})$$

Table 11 I&I Flows Summary

Tafuna	Wet Weather Flows	Dry Weather Flows	I&I Quantity	System Area	I&I Rate
Average Day	2.7 MGD	1.6 MGD	1.1 MGD	2,921 ac	380 gpad
Peak Day	6.2 MGD	3.7 MGD	2.5 MGD	2,921 ac	860 gpad

Utulei	Wet Weather Flows	Dry Weather Flows	I&I Quantity	System Area	I&I Rate
Average Day	2.8MGD	1.0 MGD	1.8 MGD	840 ac	2,143 gpad
Peak Day	6.0 MGD	3.4 MGD	2.6 MGD	840 ac	3,095 gpad

5.2 Comparison of I&I Values

CVL performed a search of the technical literature to review I&I quantities reported by others and perform a comparison with values calculated for the Utulei and Tafuna STP collection systems. Several previous reports estimated I&I flows based on standards or flow conditions and are listed below in Table 12.

Table 12 Comparison of I&I Flows

		Climate Condition	
Source	**Comment**	**Dry Weather**	**Wet Weather**
Honolulu, HI Standards	Sewers Laid Below Normal Ground Water Table	35 gallons per capita per day	2,750 gallons per acre per day
Honolulu, HI Standards	Sewers Laid Above Normal Ground Water Table	5 gallons per capita per day	1,250 gallons per acre per day
GDC Report for ASPA (GDC, 2007)	Assumes 50% of pipes below and 50% pipes above GWT	50,000 gpd, assumes 2,500 people, 20 gpcd	256,000 gpd, assumes 128 ac, 2,000 gpad.
Hart Report (Hart, 2005)	-	-	160,000 gpd, for 128 ac, 1,250 gpad.
This report, Tafuna[1]	Derived from measured flows compared with base flows, Average Day Basis	N/A	1.1 MGD, or approximately 380 gpad
This report, Tafuna[1]	Derived from measured flows compared with base flows, Maximum Day Basis	N/A	2.5 MGD, or approximately 860 gpad
This report, Utulei[1]	Derived from measured flows compared with base flows, Average Day Basis	N/A	1.8 MGD, or approximately 2,143 gpad
This report, Utulei[1]	Derived from measured flows compared with base flows, Maximum Day Basis	N/A	2.6 MGD, or approximately 3,095 gpad

[1] See Table 11 for flows and areas.

Two I&I values were computed: one for increases in Average Daily flows from base to wet weather periods and another for increases in Maximum Daily flows from base to wet weather periods.

The Tafuna STP collection system appears to exhibit area wide I&I flows that are less than those reported in the literature reviewed. The figure for Maximum Day is 860 gpad compared with values of 1,250 to 2,700 gpad in the literature. CVL believes that, for the purposes of this study, an I&I area wide flow value of 900 gpad adequately describes expected wet weather I&I in the system. We note that the large collection system area located well away from the shoreline may serve to reduce the expected I&I flows. The data implies that the sewer lines are generally above the normal groundwater table because of the relatively low rates observed when compared with values found in the literature. Area wide flow factors tend to underestimate the I&I in low areas and the benefit of targeting reductions in those areas. It is recommended that ASPA purchase two (2) portable flow monitors for each of the systems for installation at various points in the collection systems, to better monitor sewer flows and I&I contributions in a specific area.

The Utulei STP collection system exhibits a greater magnitude in I&I quantities. Comparison of Average Daily flows yields values of 2,143 gpad while Maximum Day flows jump to 3,095 gpad, indicating significant I&I into the system. We also note that much of the service area is located in a narrow strip adjacent to the Pago Pago Harbor with salt water intrusion likely as reported in Section 4.5. These facts support the findings that a good portion of the Utulei STP collection system is below the normal groundwater table and below sea level. CVL proposes to base its I&I calculations for Utulei using an area wide I&I flow factor of 3,000 gpad. As a check, CVL calculated the Average Day and Maximum Day per capita discharges for the base and wet weather periods using population data obtained from Table 1 and Table 2 and presented in Table 13.

Table 13 Average Day and Maximum Day per Capita Estimated Flows

		Average Day Base		Average Day Wet Weather	
Collection System	Population	Flow (MGD)	Per Capita (gpd)	Flow (MGD)	Per Capita (gpd)
Tafuna STP Area	21,068	1.6	75.9	2.7	128.2
Utulei STP Area	7,774	1.0	128.6	2.8	360.0

		Maximum Day Base		Maximum Day Wet Weather	
Collection System	Population	Flow (MGD)	Per Capita (gpd)	Flow (MGD)	Per Capita (gpd)
Tafuna STP Area	21,068	3.7	175.6	6.2	294.3
Utulei STP Area	7,774	3.4	437.4	6.0	771.8

The average per capita flows listed above were compared with flows used for designing sewer system facilities in other juristictions. See Appendix G, Typical Sewage Design Flows:

City of Phoenix	100 gpcd
Arizona Administrative Code	80 gpcd

The data in Table 13 is instructive. It indicates that the Average Day per capita flows from the Utulei STP collection system are approximately 1.7 times higher than Tafuna's and the Maximum Day per capita flows are 2.5 times higher than those calculated for the Tafuna collection system. The Utulei STP collection system is subject to greater I&I flows than the Tafuna system as shown by this increase in the per capita factors. The area wide I&I values presented above will be used in sizing proposed mitigation measures to reduce I&I. BOD values observed from the DMRs confirm that the Bay Area system has a larger percentage of I&I in the sewer flows. The yearly average BOD value for the Bay Area system in 2011 is at 109 mg/l relatively low and definitely lower than in the Tafuna system where the 2011 yearly average influent BOD value was 137 mg/l.

5.3 Recommendations

5.3.1 Tafuna System

In the Tafuna system sewer pipe lengths influenced by seawater intrusion only make up about 3% of the system. The total length in the Tafuna system is approximated to be 200,000 feet, the length of sewers below mean sea level is about 5,439 feet as identified in see Figure 14.

The areas shown with suspected I&I problems on Figure 14 make up about 2.5% of the whole area served (total area is 2,921 ac, problem areas are estimated at 71 ac). Repairing sewers in those areas would result in a reduction of flows by 27,000 gpd during average wet weather flows and 61,100 gpd during peak wet weather flows.

5.3.2 Utulei System

Sewers along the harbor potentially influenced by seawater intrusion make up about 30% of the total Bay Area system sewer length; the total length in the Utulei system is approximated to be 76,000 feet with 22,360 feet of sewers along the coastline. This has only been verified for 11,180 feet from available as-built drawings as shown in Figure 16.

In the Utulei system seawater intrusion is assumed to contribute approximately 12% of I&I as discussed in Section 4.5. Only one conductivity measurement was obtained. Additional testing could further validate the estimation of contribution from seawater. Table 14 lists the contributions in flows possible in the Bay Area system resulting from an estimated contribution of I&I from either seawater I&I and/or groundwater/stormwater I&I.

Table 14 I&I Source Contributions

	Total System I&I	I&I Contribution Assumed from Seawater	I&I Contribution from complete system from Groundwater/ Stormwater
Average	1.8 MGD	0.22 MGD	1.58 MGD
Peak	2.6 MGD	0.31 MGD	2.29 MGD

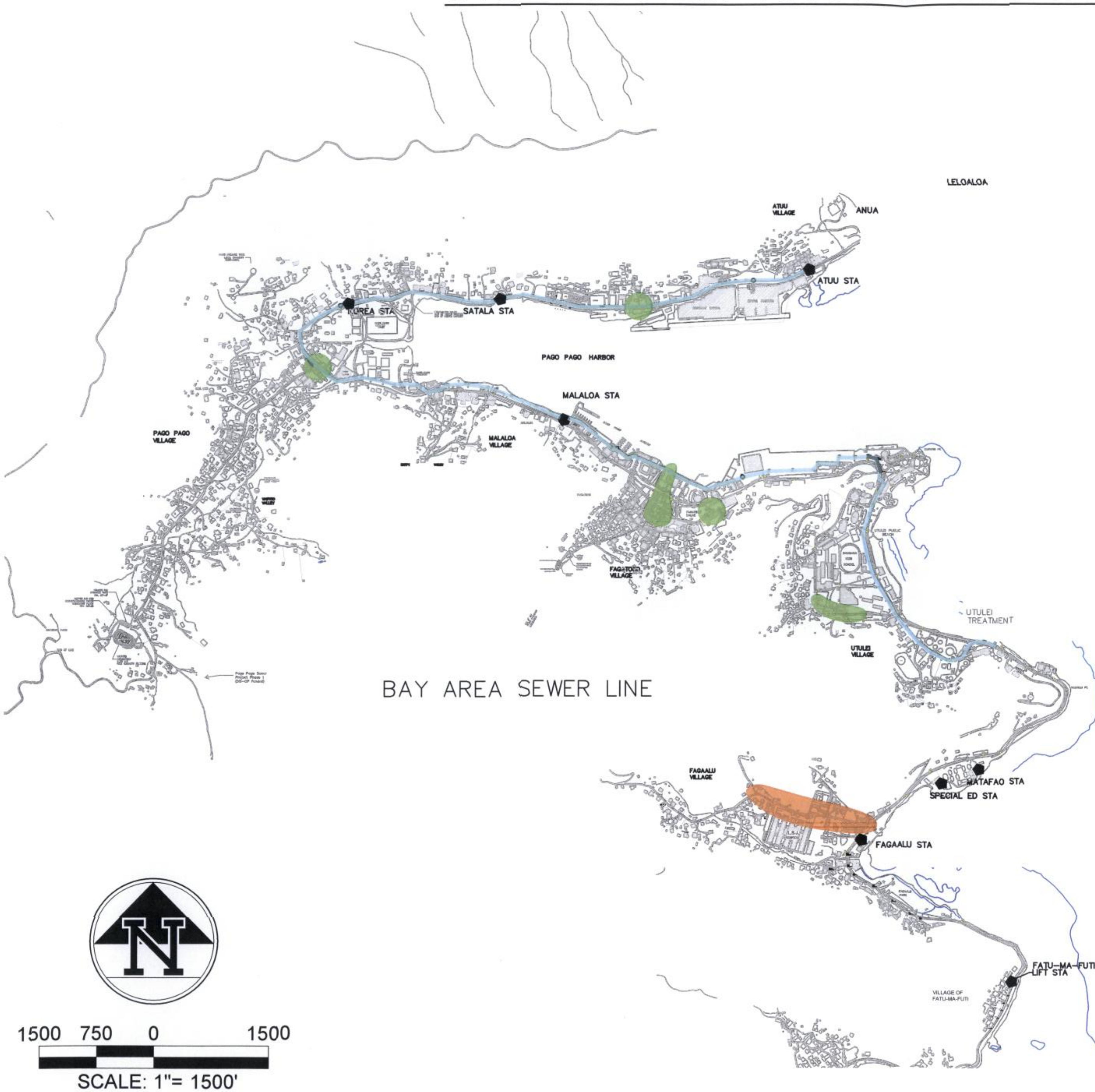
The anticipated reduction in I&I from seawater and groundwater/stormwater sources in the Bay Area collection system is presented in Table 15. It has been assumed the repair to the sewers in this area would eliminate the I&I contribution from seawater and a further reduction of approximately 30% of the I&I flows originating from groundwater/stormwater reported in Table 14.

Table 15 Anticipated I&I Reductions

	I&I Contribution Seawater	I&I Contribution (Groundwater/ Stormwater)	Total Contribution
Average	0.216 MGD	0.475 MGD	0.691MGD
Peak	0.312 MGD	0.686 MGD	0.998 MGD

The areas shown with suspected I&I problems not subject to seawater intrusion shown on Figure 16 make up about 3% of the whole area served. The total service area is 840 ac, area with problems is estimated at 24 ac. Repairing those areas would result in a reduction of flows by 54,430 gpd during average wet weather flows and 74,280 gpd during peak wet weather flows.

UTULEI SYSTEM - AREAS OF SUSPECTED I&I



LEGEND

- I & I ISSUES
- KNOWN LOW SPOT OR SURCHARGES
- SEWERS INSTALLED BELOW TIDE. THESE MAY BE SUBJECTED TO SALT WATER INTRUSION.
- PUMP STATION

NOTE:
LOW LYING SEWERS ALONG COASTAL ROAD IN OTHER AREAS OF PAGO PAGO BAY MAY ALSO BE SUBJECT TO SALT WATER INTRUSION. AS-BUILT INFORMATION IS NOT CURRENTLY AVAILABLE.

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JOB NO.
07-0216801

FIGURE
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6.0 LIFT STATIONS – TAFUNA

The Tafuna sewer system serves the villages of Faleniu, Iliili, Malaeimi, Mesepa, Nu'uuli, Pava'ia'i, and Tafuna (see Figure 5). It includes 11 wet well pump stations equipped with submersible grinder or non-clog type pumps as shown in Table 16. All pumps are manufactured by Flygt Corporation and Hydromatic and are of the constant speed type.

Table 16 Tafuna Sewer System Pump Stations

Station Name	Rated Capacity	Total Dynamic Head (feet)	Pump Type	Remarks
Existing				
Coconut Point #1	140 gpm	21	Non-clog	Duplex
Coconut Point #2	22 gpm	21	Grinder	Duplex
Coconut Point #3	19 gpm	23	Grinder	Duplex
Andy's	18 gpm	40	Grinder	Duplex
Sagamea	36 gpm	42	Grinder	Duplex
Papa Stream	390 gpm	36	Non-clog	Duplex
Vaitele	390 gpm	103	Non-clog	Triplex
Skill Center	225 gpm	24	Grinder	Duplex
Lavatai	155 gpm	42	Non-clog	Duplex
Airport	570 gpm	80	Non-clog	Duplex
Freddie's Beach	18 gpm	Unknown	Non-clog	Simplex
Future				
5 Lift Stations[1]	Varies	Varies	Non-clog	varies

[1] 5 Additional lift stations, future expansion west and north-west of the treatment plant.

6.1 Methodology

The following information for each lift station was obtained from ASPA: pump station run times, wet well configurations, system plans, and maintenance logs.

Our initial evaluation focused on lift station run times compared with rain events to determine the magnitude of I&I for each tributary area served by the lift stations and thereby quantifying the necessary mitigation alterations to reduce I&I. Table 17 shows pump run time given for the selected lift stations in the Tafuna system for the January 2011 rain event. Pump run times are given in hours. These lift stations were selected because of their importance to the system.

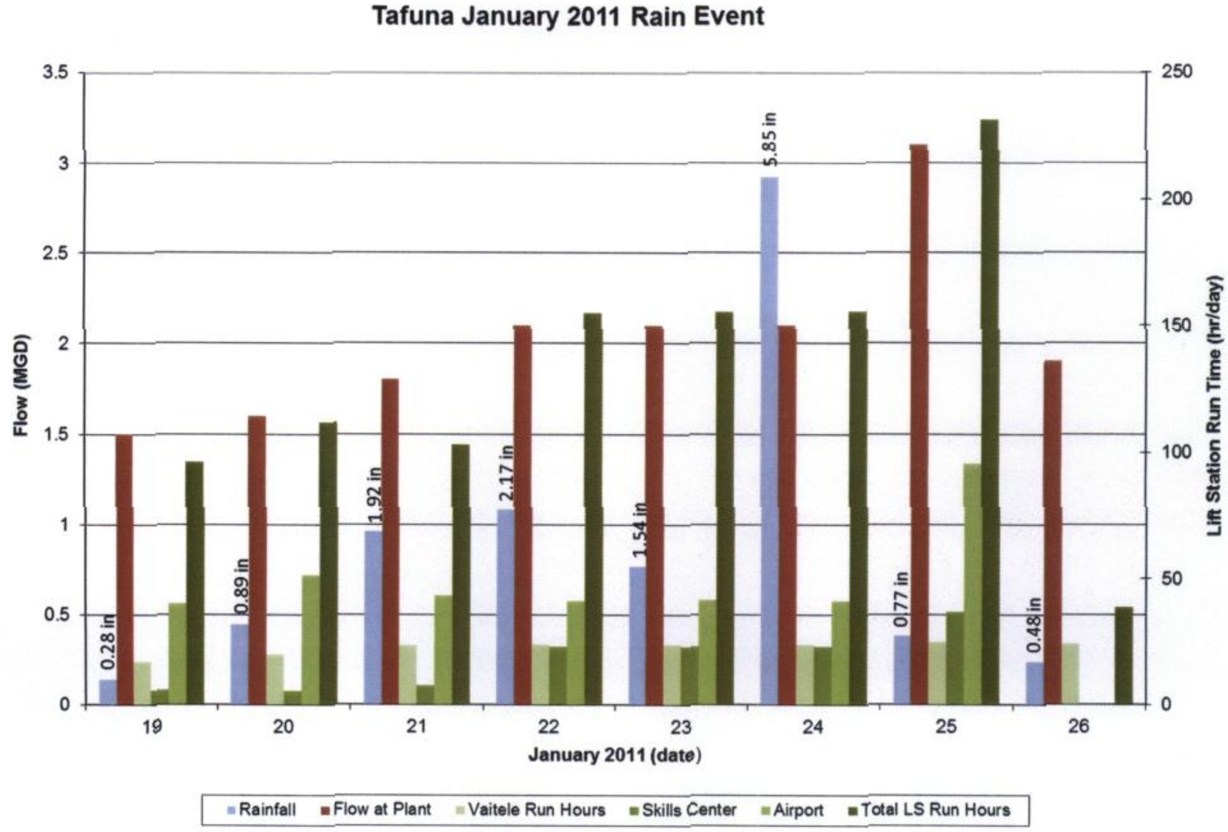
Table 17 Lift Station Run Times

	Vaitele Pump (hr)				Skill Center Pump (hr)			Airport Pump (hr)		
Day	#1 Pump	#2 Pump	#3 Pump	Total	#1 Pump	#2 Pump	Total	#1 Pump	#2 Pump	Total
19	17	0	0	17	3.9	1.8	5.7	20	20.5	40.5
20	19.9	0	0	19.9	3.8	1.8	5.6	25	26.2	51.2
21	23.3	0	0	23.3	4.8	2.8	7.6	21	22.2	43.2
22	23.9	0	0	23.9	13.9	9.1	23	20	21.4	41.4
23	23.9	0	0	23.9	13.9	9.2	23.1	20	21.4	41.4
24	23.9	0	0	23.9	13.9	9.2	23.1	20	21.4	41.4
25	24.8	0	0	24.8	18.9	18	37	45	50.4	95.4
26	24.1	0	0	24.1			0			0

Run times for each pump in the lift station were added to obtain the total run time per day per lift station. These total lift station run times were then compared against Treatment Plant flow data and precipitation data in an attempt to quantify the relationship between precipitation, I&I and pump run times at these lift stations.

In Figure 17 it is apparent that the inflow at the treatment plant correlates well with increased rain events. This confirms the existence of I&I in the system as discussed in Section 3.0 of this report. It is also apparent that some of the lift station run hours do not parallel the increase in rainfall and expected increase in flow to the lift station caused by I&I. Vaitele lift station data does not appear to be complete. See also Table 17, where pumps #2 and #3 were either not running or data was not reported for this lift station during periods of presumed high flows caused by I&I. Total run hours in Figure 17 include all lift stations in the Tafuna system listed in Table 16. Total run hours correlate somewhat with plant inflows but do not allow for pinpointing the locations of problem areas.

Figure 17 Tafuna System Lift Station Run Times, January 2011



6.2 Limitations of Available Data

It was hoped to use the pump run times at selected lift stations in the system to determine the magnitude of I&I in the system in general and for those areas tributary to each lift station. With this information, areas of high I&I could be identified and I&I mitigation measures tailored for each lift station tributary area. Review of the pump run time data shown in Table 17 and Figure 17 indicates that the use of pump run time data does not provide a reliable record of I&I. Several factors influence lift station run time records and cause the data to be skewed.

- Pumping equipment condition causes some pumps to run for extended periods. Pump maintenance logs by ASPA note for example that pump bases on two pumps in the Vaitele were broken which caused pumps not to discharge directly into the risers. The complete maintenance logs are attached in Appendix H, Lift Station Maintenance Logs.
- Pump run times are not recorded at equal intervals each day causing some days to record significantly more than 24 hours and shortening the recorded run time for the following day.
- Pump run times were frequently averaged over weekends starting on Friday at 8 am and ending on Monday at 8 am. This results in averaged data over almost half the weekly period.

Following discussions with ASPA and USEPA, it was agreed to modify our approach in calculating I&I at each lift station. The use of pump run times to determine I&I was abandoned. The magnitude of I&I to the lift stations will be calculated using the area wide I&I factors determined for the Tafuna or Utulei STP collection system as presented in Section 5.2.

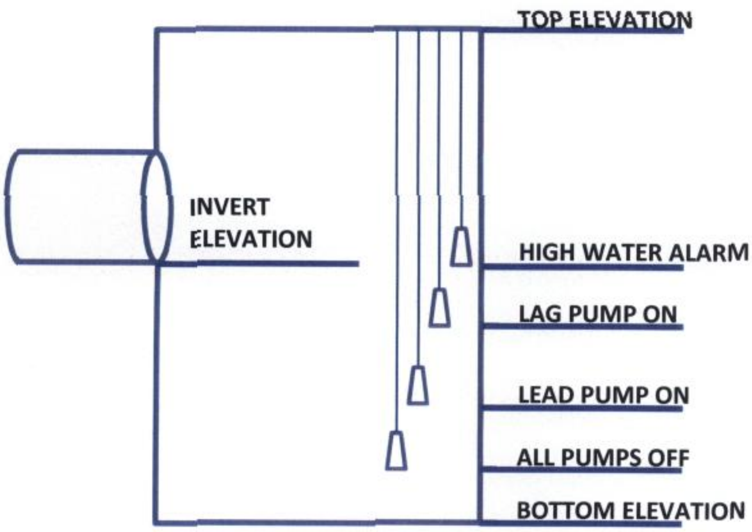
6.3 Consideration of Future Improvements

This Section discusses possible improvements to lift station facilities to attenuate peak flows.

6.3.1 Pump Operating Levels

Pumps in wet wells are typically operated using float levels set at certain elevations. For a wet well operating with two pumps (one pump, one backup) the following example figure (Figure 18) shows the arrangement of typical level settings.

Figure 18 Example Pump Station Float Elevation Setup



For lift stations where the average flows can be handled with one pump, flow equalization can be achieved by maximizing the working volume. The discussion below illustrates the effect of changes in float level elevations for a system facility. Parameters for this example were taken from the plans for the Vaitele lift station in the Tafuna system. In this example the lead pump and the lag pump are set to start at the same time. See Table 18, existing elevation settings for pump 1 and pump 2.

Table 18 Float Level Settings at Vaitele

	Existing		Proposed	
	Elevations (ft)	**Flows (gpm)**	**Elevations (ft)**	**Flows (gpm)**
Inflow Invert	93.07		93.07	
Finished Floor	85.17		85.17	
Average Influent Flow		209		209
Peak Influent Flow		480		480
Pump Design Flow		325[1]		325[1]
All Pumps off	86.67		86.67	
Pump 3 off	87.67			
Pump 1 on	89.17		89.17	
Pump 2 on	89.17		91.67	
Pump 3 on	92.07		92.67	

[1] Per ASPA

The Vaitele lift station wetwell has a diameter of 15 feet. The pump flow for the three pumps at Vaitele as measured by ASPA is 390 gpm. The average inflow was assumed to be 209 gpm and the peak inflow 480 gpm. As calculated by using population numbers for the village of Nu'uuli tributary area with approximately 3,960 people, see Table 1 in Section 2.1.1. As determined in Section 5.2 the expected average discharge is 76 gpcd. A peaking factor of 2.3 was used to calculate the peak flow. Figure 19, 'Average Flow Existing' shows how the accumulated sewage volume is pumped out of the wet well very rapidly during average flows owing to both pumps being activated. One of the constant speed pumps has a discharge capacity greater than the average inflow rate (325 gpm v. 209 gpm). If the level floats are set to only start one pump initially, discharge during average flows is more equalized as shown in Figure 19.

Figure 19 Discharge at Average Flows at Vaitele Lift Station

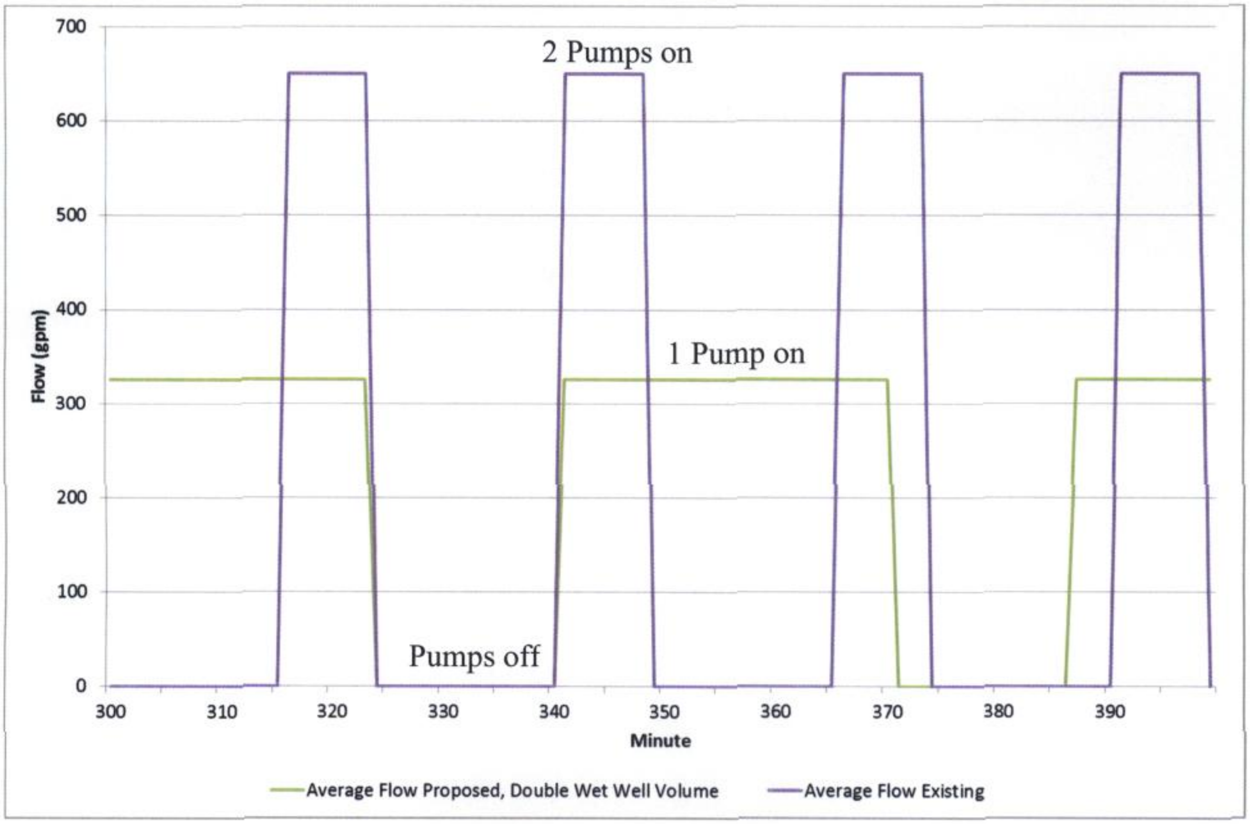
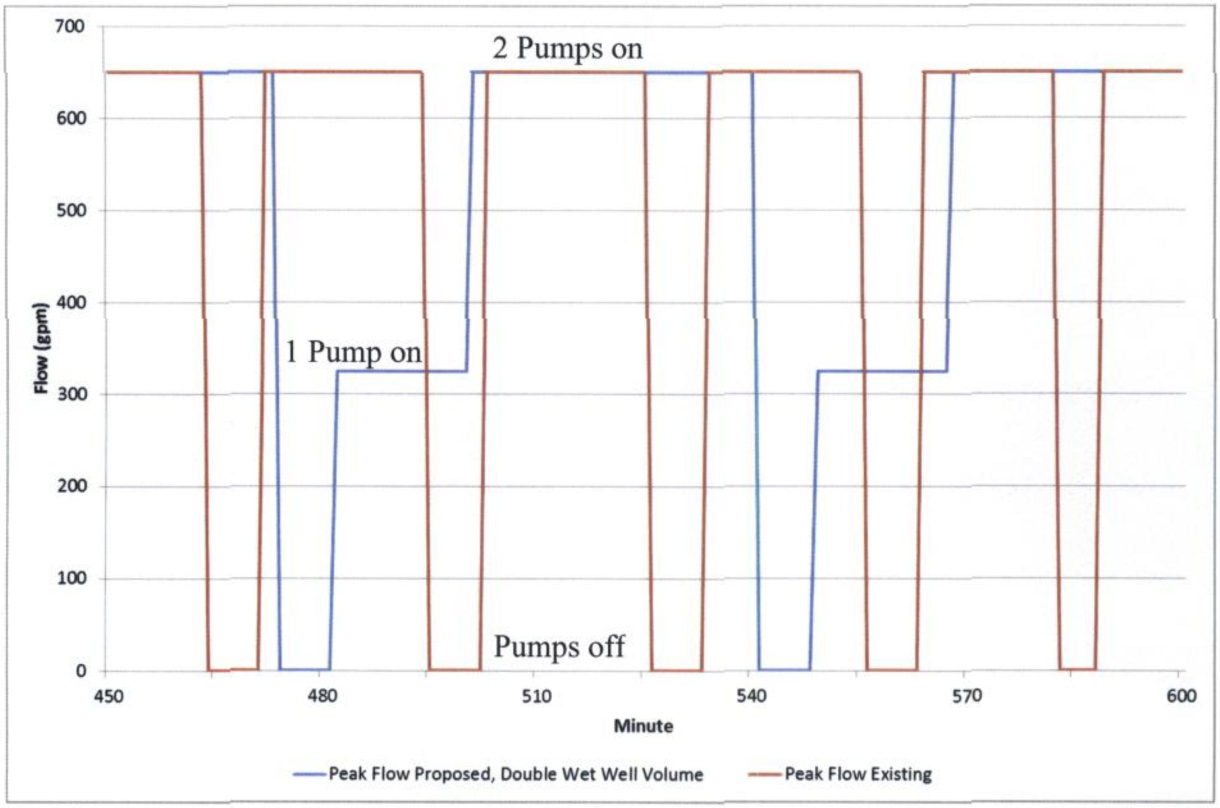


Figure 20 shows the same setting but for peak flows. During peak flows the existing level settings allow for more equalized flows as two pumps running are able to pump the peak inflow. The existing settings start the second pump at 89.17 feet and run two pumps for a volume of 3,830 gal (2.8 feet elevation difference, 15 feet wetwell diameter). However, the proposed settings start the second pump at 91.67 feet and run two pumps for a volume of 1,324 gal (1.0 foot elevation difference, 15 feet wetwell diameter).

The existing settings would be preferred for peak flows because inflows are in magnitude much closer to two pumps running and for short periods even three pumps are needed to pump the incoming flows out of the wet well. Represented in the figures is a time period of 100 minutes assuming constant average flows in Figure 19 and a period of 150 minutes assuming constant peak flows in Figure 20. Flows and level settings are summarized in Table 18.

Figure 20 Discharge at Peak Flows at Vaitele Lift Station



Float levels in the lift stations can be adjusted to maximize pump cycle time and reduce short term peaks.

6.3.2 Variable Frequency Drives

All pumps currently used in the ASPA sewer collection systems are constant speed pumps. In a lift station where the inflow and outflow are not matched, a constant speed pump operates with on and off cycling at certain variable intervals dictated by inflows and wet well volumes. This creates unnecessary wear and reduces equipment life. It also means that discharge flows in the system vary greatly. Installing variable frequency drives (VFD) allows for the outflow to match the inflow up to the equipment rating. For the ASPA sewer systems where flow equalization is a priority, variable frequency drives would help attenuate flows within the limits of the lift station capacities.

Other advantages of installing VFDs are energy savings due to maximizing pump efficiencies to match flows. During low flow conditions pumps will not be operating at full capacity for example. Pumping at lower flows results in pumping against lower heads due to decreased friction. Cycle times are also greatly reduced which reduces the number of starts and increases operating times. This increases the overall life of the pumping equipment and saves cost on the long run. Flows above the design maximum pump discharge will accumulate in the wet well.

The VFDs would need to be sized to a maximum discharge rate so as to attenuate peak flows during rain events.

6.3.3 Equalization Volume at Lift Station Wet Wells

Installing additional volume at the lift stations may further equalize the flows to the plant and ultimately the effluent flows. Many of the lift stations do not have space for large equalization basins. A smaller basin, for example a parallel structure of the same dimensions as the wet well could be installed where space allows. Figure 21 and Figure 22 show a graph of the flows at the Vaitele lift station for the proposed condition in which a second manhole was installed, doubling the available wet well volume. Calculations are shown in Appendix I, Lift Station Volume Calculations.

For the average flows the proposed scenario allows for more equalization. Doubling of the wet well volume allows for one pump to run for a period twice as long as shown in Figure 19 above thereby limiting the run time of a second pump that would have otherwise been activated for a short time to keep up with the inflow.

For periods of extended peak flows discharged to the lift station with the proposed pump level settings and additional wet well volume (see Figure 22), flows would be more equalized as compared to those shown above in Figure 20.

The additional volume will not be enough to attenuate flows and reduce extended peak flow periods to average flow conditions.

Figure 21 Discharge at Average Flows, Double Wet Well Volume

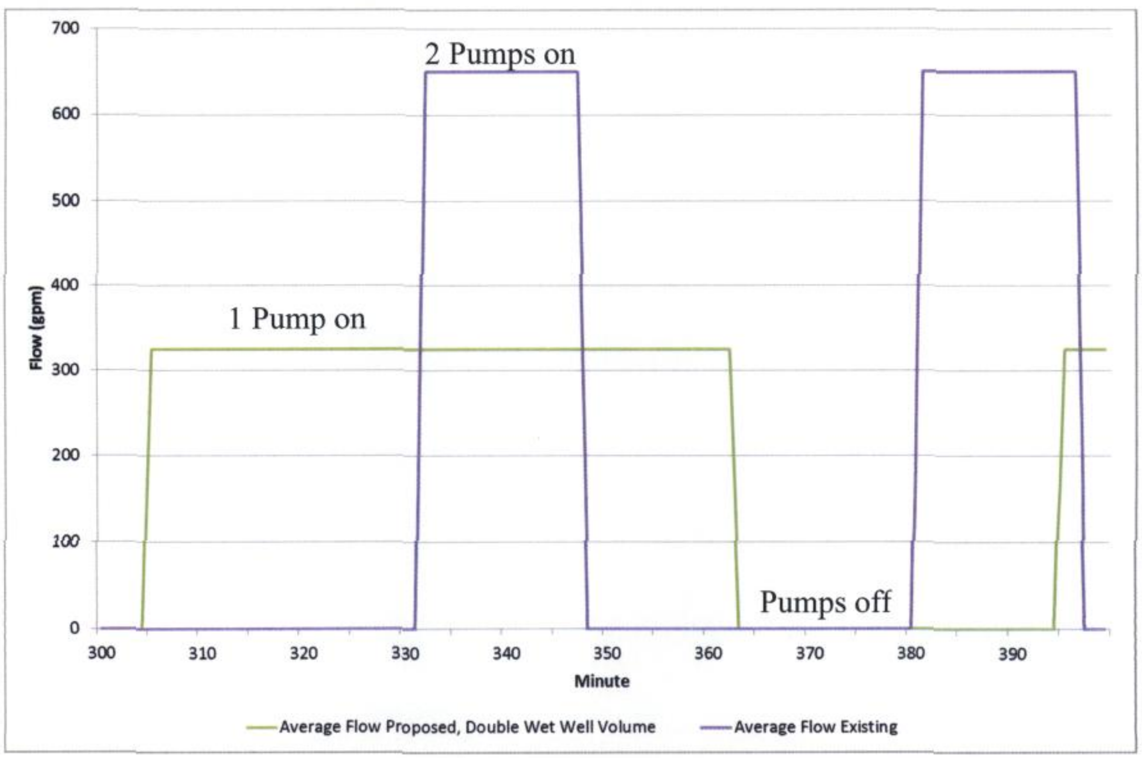
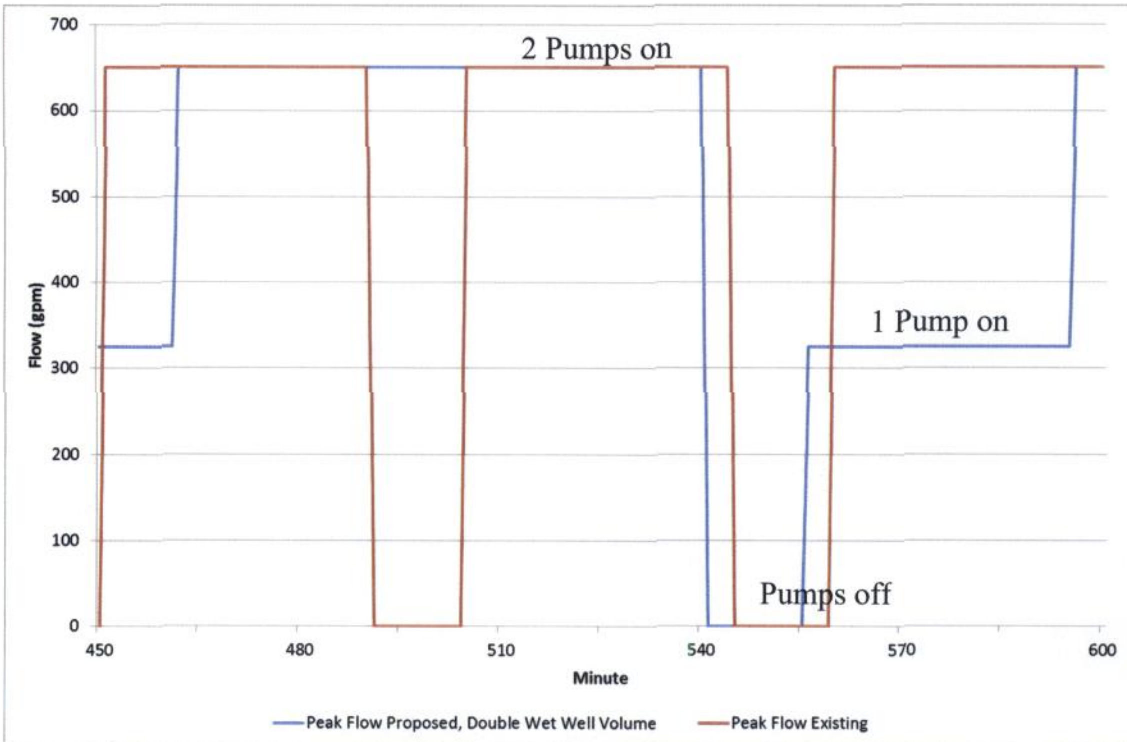


Figure 22 Discharge at During Peak Flows, Double Wet Well Volume



6.4 Equalization in the Collection System

Equalization Volume may also be found in the sewer system. Especially large diameter pipes can offer sufficient equalization volume provided the site topography does not cause manholes to surcharge. In general level areas with minimal elevation changes offer more equalization volume. Table 19 lists possible storage volume by pipe diameter for reference.

Table 19 Potential Storage Volume in Pipe

Pipe Diameter (in)	Per Length (ft)	Volume (gal)
6	1,000	1,469
8	1,000	2,611
10	1,000	4,080
12	1,000	5,875
15	1,000	9,179
16	1,000	10,444
18	1,000	13,218
24	1,000	23,499

To assess the potential storage volume in the collection system, the larger lift stations and incoming gravity sewers were investigated. The lowest rim elevation of the reach (manhole or lift station) was assumed to be the maximum allowable rise in sewage levels in the system. Manhole sections, pipe lengths, and wet well sections below that elevation were included in the equalization volume. Table below lists the potentially available storage volume in the Tafuna sewer system. Storage in the system can be achieved by changing the pump float elevations.

Table 20 System Storage Potential

Pump Station	Pump Station Volume (gal)	Gravity Pipe Volume (gal)	Manhole Volume (gal)	Total Storage (gal)
Papa	643	7,916	5,103	13,662
Vaitele	3,454	19,820	9,702	32,977
Skill Center	253	2,037	2,299	4,588
Airport	108	2,285	3,703	6,095
Coconut Point 1	404	6,878	4,559	11,841
Tafuna[1] STP influent Pump Station	1,769	17,589	0	19,358

[1] Manhole elevations in the system need to be verified.

Expected flows for each village are estimated from populations found in Table 1. The area around the Papa, Vaitele, and Skills Center lift stations is all considered Nu'uuli Village. The estimated population numbers may need to be adjusted by ASPA in order to more precisely estimate expected flows.

Table 21 Required Collection System Volume by Pump Station

Lift Station	Village	Population	Expected Flows (gal)		Required Storage Volume[1]	Available Volume
			AVG[2]	Max[2]	(gal)	(gal)
Coconut Point 1	Nu'uuli Part	659	84,352	193,746	34,821	11,841
Papa	Nu'uuli Part	1,500	192,000	441,000	79,259	13,662
Vaitele	Nu'uuli Part	3,955	506,240	1,162,770	208,980	32,977
Skills Center	Nu'uuli Part	796	101,888	234,024	42,060	4,588
Airport	Tafuna & Nu'uuli Part	8,741	1,118,848	2,569,854	461,870	6,095

[1] For calculations of storage volume see Section 8.2.
[2] See Table 13.

As seen in Table 21, storage volumes in the system are small and not sufficient to eliminate peaks into the treatment plant. Some reduction would be expected, the magnitude of the reductions could be determined with a further study. Adding additional equalization at the Airport pump station and at the treatment plant would have the greatest effect on attenuating flows to the treatment plant.

In order to get the maximum storage volume, the float levels would have to be set as follows (a safety elevation of 3 feet below the lowest rim is assumed for the last pump to turn on):

Table 22 Float Levels[1] to Maximize System Storage

Lift Station	Lowest Rim Elevation (ft)	LS Inflow Invert Elevation (ft)	Pump 1 On Elevation (ft)		Pump 2 On Elevation (ft)		Pump 3 On Elevation (ft)	
			Existing	Proposed[2]	Existing	Proposed[2]	Existing	Proposed[2]
Airport	108	93.3	101.35	104.5	101.85	105.00	-	
Coconut Point 1	104.3	98.6	95.75	100.55	96.5	101.30	-	
Skill Center	105.9	100.6	99.60	102.40	100.10	102.90	-	
Papa	103.5	93.9	92.63	99.90	93.22	100.50	-	
Vaitele	105.8	93.1	89.17	101.8	89.17	101.8	90.17	102.8

[1] Elevations are mean sea level + 100 feet
[2] During high flow periods only, level controls to match existing during normal operations.

Some of the lift stations are arranged in series. Providing system storage at lift stations upstream of Vaitele and the Airport lift station will not attenuate flows to the treatment plant sufficiently. Attenuating flows at Vaitele and the Airport lift stations would reduce system flows to average flows at the STP except for flows from the gravity portion of the system. Flows from the gravity system make up a much larger portion of total system flows than flows conveyed through Vaitele and the Airport lift stations. The gravity system includes I&I contributions from stormwater and possible groundwater. An equalization basin at the Tafuna STP would have to be added in addition to basins at Vaitele and at the Airport lift station to attenuate peak flows into the plant.

7.0 LIFT STATIONS – UTULEI

The Bay Area sewer system serves the villages in the vicinity of Pago Pago Harbor. These villages are Anua, Atu'u, Faga'alu, Fagatogo, Fatumafuti, Pago Pago, Satala, Utulei, and in the future Aua, Onesosopo, and Leloaloa (see Figure 6). It includes seven wet well pump stations equipped with non-clog type pumps as shown in Table 23. All pumps are manufactured by Flygt Corporation or Hydromatic and are of the constant speed type.

Table 23 Utulei Sewer System Pump Stations

Station Name	Rated Capacity	Total Dynamic Head (feet)	Pump Type	Remarks
Existing				
Atuu	150 gpm	21	Non-clog	Duplex
Satala	285 gpm	17	Non-clog	Duplex
Korea House	380 gpm	18	Non-clog	Duplex
Malaloa	1,420 gpm	29	Non-clog	Triplex
Matafao School	75 gpm	31	Non-clog	Simplex
Matafao Special Ed	75 gpm	31	Non-clog	Simplex
Faga'alu	310 gpm	48	Non-clog	Duplex
Future				
6 Lift Stations[1]	Varies	Varies	Non-clog	varies

[1] 6 additional lift stations are planned in the ESV area.

7.1 Methodology

The following information for each lift station was obtained from ASPA: pump station run times, wet well configurations, system plans, and maintenance logs.

Our initial evaluation focused on lift station run times compared with rain events to determine the magnitude of I&I for each tributary area served by the lift stations and thereby quantifying the necessary mitigation alterations to reduce I&I. Table 24 shows pump run times given for the selected lift stations in the Utulei system for the April 2008 rain event. Pump run times are given in hours. These lift stations were selected because of their importance to the system.

Table 24 Lift Station Run Times – April 2008

	Malaloa Pump (hr)				Korea House Pump (hr)			Faga'alu Pump (hr)		
Day	#1 Pump	#2 Pump	#3 Pump	Total	#1 Pump	#2 Pump	Total	#1 Pump	#2 Pump	Total
13	13	13	0	26	1	1	2	9	9	18
14	12.9	13.1	0	26	1.6	3.6	5.2	8.7	9	17.7
15	13.8	13.7	0	27.5	1.1	0	1.1	0	18.7	18.7
16	14.5	14.5	0	29	1.2	3.8	5.0	3.4	13	16.4
17	17	17	0	34	1	2	3	18	21	39
18	17	17	0	34	1	2	3	18	21	39
19	17	17	0	34	1	2	3	18	21	39

Run times for each pump in the lift station were added to obtain the total run time per day per lift station. These total lift station run times were then compared against Treatment Plant flow data and precipitation data in an attempt to quantify the relationship between precipitation, I&I and pump run times at these lift stations.

7.2 Limitations of Available Data

As discussed in Section 6.2 available pump runtime data cannot be used reliably to approximate the extent of I&I. In addition to limitations listed in Section 6.2, the Utulei collection system experienced the following event further limiting the use of available data:

- The Tsunami from September 29, 2009, damaged several of the lift stations. Although the sewer system remained functional, pump run times have not been regularly recorded at the damaged Lift Stations (Satala, Korea, and Malaloa).

7.3 Consideration of Future Improvements

A discussion of future improvements including adjustment of pump operating levels, installation of variable frequency drives, and addition of equalization volume at lift station wet wells is included for the Tafuna system in Section 6.3 and is also valid for the Bay Area system.

7.4 Equalization in the Collection System

Equalization Volume is also available in the Utulei collection system and was determined as discussed in Section 6.4 above. Information on sewers was not available for the whole Bay Area system.

Table 25 System Storage Potential

Pump Station	Pump Station Volume (gal)	Gravity Pipe Volume (gal)	Manhole Volume (gal)	Total Storage (gal)
Utulei	8,393	192,109	45,581	246,083
Malaloa	3,172	98,314	17,955	119,441
Faga'alu	705	15,502	1,679	246,083

Expected flows for each village are estimated from populations listed in Table 13.

Table 26 Required System Volume by Pump Station

Lift Station	Village	Population	Expected Flows (gal)		Required Storage Volume[1]	Available Volume
			AVG	Max	(gal)	(gal)
Utulei Influent Lift Station	Fagatogo	1,737	625,320	1,340,964		
	Utulei	684	246,240	528,048		
	Atuu	359	129,240	277,148		
	Anua	18	6,480	13,896		
	Pago Pago	3,656	1,316,160	2,822,432		
	Satala	297	327,600	702,520		
	Faga'alu	910	40,680	87,236		
	Fatu-Ma-Futi	113	106,920	229,284		
Total			2,798,640	6,001,528	1,019,511	246,083
Malaloa LS	Fagatogo	1,737				
	Atuu	359				
	Anua	18				
	Pago Pago	3,656				
	Satala	297				
Total		6,067	2,184,120	4,683,724	795,649	119,441
Faga'alu	Faga'alu	910				
	Fatu-Ma-Futi	113				
Total		1,023	368,280	789,756	134,160	17,886

[1] For calculations of storage volume see Section 8.2.
[2] As listed in Table 13

The storage volume at Utulei is not sufficient to reduce peak flows into the treatment plant.

Table 27 Float Levels[1] to Maximize System Storage

Lift Station	Lowest Rim Elevation (ft)	LS Inflow Invert Elevation (ft)	Pump 1 On Elevation (ft)		Pump 2 On Elevation (ft)		Pump 3 On Elevation (ft)	
			Existing	Proposed[2]	Existing	Proposed[2]	Existing	Proposed[2]
Faga'alu	105.70		94.20	102.20	94.70	102.70		
Malaloa	103.56	89.00	86.50	99.56	87.00	100.06	87.50	100.56
Utulei STP	102.2	94.00	94.00	98.20	94.60	98.80	95.00	99.20

[1] Elevations are mean sea level + 100 feet
[2] During high flow periods only, level controls to match existing during normal operations.

Since some of the lift stations are arranged in series, attenuating flows at the Malaloa and the Faga'alu lift stations would reduce total system flows to average flows at the Utulei STP.

8.0 TAFUNA STP FLOW EQUALIZATION

Flow equalization is used to minimize the variability of wastewater flow rates and sometimes its composition. Variations in flows in the American Samoa wastewater systems are due to I&I resulting from rainfall, groundwater and seawater intrusion, and variations in municipal sewer discharges. To reduce these variations in flow, equalization basins may be provided in the system or at the treatment facilities. The equalization basins will equalize flows to allow the discharge of sewer at a uniform rate to the treatment facilities.

Equalization basins in a treatment system can be located in-line or off line. In in-line equalization, 100% incoming raw wastewater directly enters into the equalization basin, sufficiently sized to attenuate peak flows. Wastewater would then be pumped directly to other treatment units. However, for offline equalization, the basin does not directly receive the incoming wastewater. Rather, an overflow structure diverts excess flow from the incoming raw wastewater into a basin which stores the excess flows that are released on a measured basis to the treatment process after the peak flow period has passed.

8.1 Tafuna Treatment Plant

Wastewater conveyed through the Tafuna collection system is treated at the Tafuna Sewage Treatment Plant (Tafuna STP). See Section 2.1 for a full description of the plant processes.

Expected I&I flows to the STP were calculated from an analysis of DMRs for discrete dry weather and wet weather periods as described in Section 3.3 and summarized in Section 3.4. From this data, two I&I flow scenarios have been investigated.

1. Reduction of I&I peak discharge (wet weather Max Day) flows to wet weather Average Day flows:

   I&I Max Day = 6.2 MGD (see Table 13)
   Wet Weather Average Day = 2.7 MGD (see Table 13)

   This scenario may be described as a worst case for the system

2. Reduction of Max wet weather flows indicated in USEPA's AO to Average Day flows as reported in the AO:

   Max Wet weather flows = 5.3 MGD (see Appendix A, Administrative Order)
   Average Day flows = 1.94 MGD (see Appendix A, Administrative Order)

The scenarios provide a range of flow volumes to be equalized at the STP. An optimal volume for equalization has been determined for each.

8.2 Equalization Volume Calculation Methodology

Computation of the volume of an equalization basin is the key design requirement and is based on inflow variation over time. In the case of the treatment facilities in this study inflow variation over time is not available. Flows are recorded at the effluent weir of the treatment plant and are a reflection of the number of constant speed pumps operating at the influent pump station. Plant flow data includes daily average, minimum, and maximum flows, as well as flow data recorded on circular flow charts. A large percentage of flow in both systems is conveyed to the facility through a series of pump stations which skews a diurnal curve. This is reflected in the circular flow charts in which measured flows oscillate very quickly as pumps turn on and off. (See Appendix J, Sample Circular Flow Charts, for example charts).

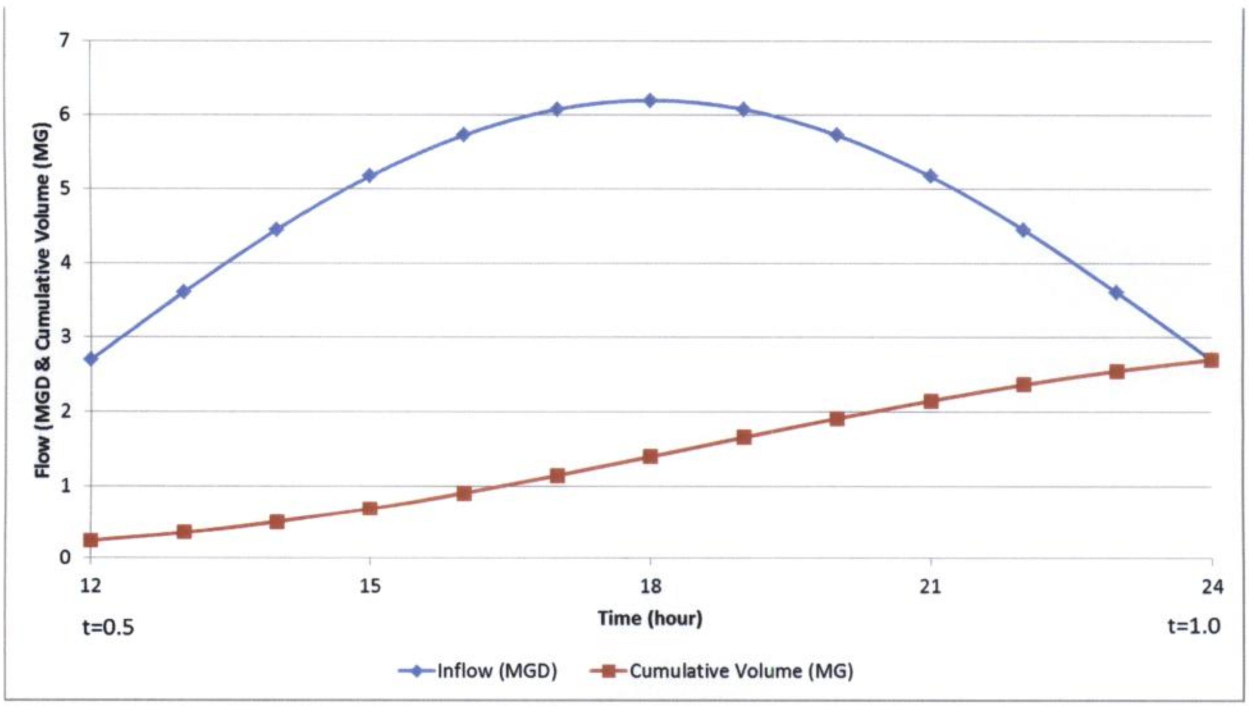
A diurnal variation of flows to the STP may be approximated by a sine wave function, where reliable inflow over time data is not available (EPA 1979, see Appendix K, Equalization Volume Calculations). This relationship is shown by the following equation.

Equation 1 $$Q_{in}(t) = Q_A - (Q_P - Q_A)\sin 2\pi t$$

$Q_{in}(t)$ = Influent flow as a function of time
Q_P = Peak Influent Flow
Q_A = Average Flow, or attenuated flow, a constant outflow
t = time in days

It is assumed that the time (t) segment is one day beginning at midnight. Minimal flows are simulated at 6 am and maximum flows at 6 pm:

Figure 23 Sine Wave Flow Approximation for Scenario 1, Tafuna



Then, To approximate the equalization volume needed the sine wave equation can be simplified to include the portion from noon, t=0.5 to midnight, t=1.0 and integrating Equation 1 from t=0.5 to 1.0.

Equation 2 $$V = \int_{t=0.5}^{t=1.0} (Q_P - Q_A)dt$$

V = Equalization Volume, Working Range
Q_P = Peak Influent Flow
Q_A = Average Influent Flow
t = time in days

This yields:

Equation 3 $$V = \frac{Q_P - Q_A}{\pi}$$ V = Equalization Volume, Working Range
Q_P = Peak Influent Flow
Q_A = Average Influent Flow

Simplified sine wave method equation

Using Equation 3 listed above, the required equalization volume for Scenario 1 is 1.11 MG.

Using Equation 3 listed above, the required equalization volume for Scenario 2 is 1.07 MG.

8.3 Mixing and Aeration Requirements

Both in-line and side-line equalization basins require mixing and aeration. Mixing should be designed to blend the contents of the tank and to prevent deposition of solids. Aeration is required to prevent the wastewater from becoming septic. Ideally, grit should be removed prior to flows entering the equalization basin to reduce mixing requirements but this was not considered.

Mixing requirements: 0.02 to 0.04 HP per 1,000 gal of storage
Aeration requirements: 1.25 to 2 cfm air per 1,000 gal of storage
(EPA, 1976)

8.4 Discussion of Results

Table 28 summarizes the flows used to calculate equalization volumes, and the approximate capacities of the basins. These volumes are calculated for current flows and do not assume a reduction of I&I through collection system repairs. Future expansion to include Leone Village will likely increase the required equalization volume. Alternatively, repairs to the collection system and equalization in the system and at lift stations will likely decrease the equalization volume required.

Table 28 Tafuna Equalization Volumes and Mixing/Aeration Requirements

Scenario	Peak Flow (MGD	Average Flow (MGD)	Volume (MG)	Volume (gal)	Mixing (hp)	Aeration (cfm)
1	6.2	2.7	1.11	1,114,100	44.0	2,220
2	5.3	1.94	1.07	1,069,640	42.8	2,140

Using a side water depth of 10 feet, a rectangular concrete basin of 100 ft by 150 ft would be necessary to provide 1.11 MG of equalization storage. A 100 ft by 143 ft basin would provide sufficient equalization storage for 1.07 MG in Scenario 2.

It is noted that Scenario 1 provides for a slightly larger equalization volume and is selected as the basis for an equalization system for the Tafuna STP.

The equalization basin would fill from a diversion structure placed upstream of the headworks on the 24-inch inflow sewer whenever plant inflows are greater than 2.7 MGD. An adjustable side weir in the structure would regulate the diversion of flows to the basin. Emptying the basin would be accomplished by pumping the collected sewage to the headworks at low flow periods.

The pumping rates must be set during design but would be approximately equal to the 2.7 MGD average flows (1,875 gpm).

In conclusion a 1.11 MG equalization basin at Tafuna STP is required to maintain plant inflows to 2.7 MGD, a rate equal to the anticipated Average Day of a wet weather period. It is this flow that will be evaluated in the discussion of outfalls and Critical Dilution Zones for the Tafuna STP, Section 10.0.

9.0 UTULEI STP FLOW EQUALIZATION

9.1 Utulei Treatment Plant

See Section 2.2 for a full description of the plant processes and Section 4.3 for a description of dry and wet weather flows.

9.1.1 I&I Influenced Flow

Equalization volumes have been calculated for two I&I scenarios:

1. Reduction of I&I wet weather Max Day flows to wet weather Average Day flows (worst case):

I&I Max Day	= 6.0 MGD (see Table 13)
Wet Weather Average Day	= 2.8 MGD (see Table 13)

2. Reduction of Max Wet weather flows indicated in USEPA's AO to Average Day flows as reported in the AO:

Max wet weather flows	= 4.4 MGD (see Appendix A, Administrative Order)
Average Day flows	= 1.21 MGD (see Appendix A, Administrative Order)

The scenarios provide a range of flow volumes to be equalized at the STP. An optimal volume for equalization has been determined for each.

9.2 Equalization Volume Calculation Methodology

The method used to calculate the flow equalization volumes is discussed in Section 8.2 using Equations 1, and 2, and 3.

Using Equation 3, the required equalization volume is 1.02 MG for Scenario 1.

Equation 3 calculates the required equalization volume to be 1.02 MG for Scenario 2.

9.3 Mixing and Aeration Requirements.

Mixing and aeration requirements are discussed in Section 8.3.

9.4 Discussion of Results

Table 29 summarizes the flows used to calculate equalization volumes, and the approximate capacities of the basins. These volumes are calculated for current flows and do not assume a reduction of I&I through collection system repairs. Future expansion to include Leloaloa and Aua Villages will likely increase the required equalization volume. Alternatively, repairs to the collection system will likely decrease the equalization volume required by reducing I&I.

Table 29 Utulei Equalization Volumes and Mixing/Aeration Requirements

Scenario	Peak Flow (MGD	Average Flow (MGD)	Volume (MGD)	Volume (gal)	Mixing (hp)	Aeration (cfm)
1	6.0	2.8	1.02	1,018,590	41	2,040
2	4.4	1.21	1.02	1,015,410	41	2,040

The equalization volumes for both scenarios are approximately equal and a concrete basin having the dimensions of approximately 100 ft by 138 ft by 10 ft side water depth would be required. Scenario 1 with average flows of 2.8 MGD is selected for further discussion in Section 10.0, Outfalls and Critical Dilution Zones. There being no space at the treatment plant site, a nearby location for this facility must be selected during the design phase of this work. The adjacent tank farm site may offer an opportunity for locating the facility. Diversion from the main interceptor would be accomplished with a splitter box and flows above the 2.8 MGD Average Flow quantity established for this scenario would be pumped to the equalization basin. Stored sewage in the basin would be returned to the STP inlet upon resumption of normal flows.

10.0 OUTFALLS AND CRITICAL DILUTION ZONES

10.1 General

The Administrative Orders issued by USEPA require that scoping studies be performed to investigate the feasibility of increasing the critical initial dilution (CID) and/or zone of initial dilution (ZID) for estimated mean and peak -outfall discharges from the Tafuna and Utulei STPs. This scoping study investigates two means of achieving this goal. Firstly, the configuration of the existing outfalls and diffusers are studied and recommendations made to modify or upgrade the outfall structure so as to increase the CID and ZID. Secondly, knowing that an increase in the CID and ZID may also be achieved by reducing the peak discharges from both STPs, possible projects are identified that would yield a reduction in effluent flows from the STPs.

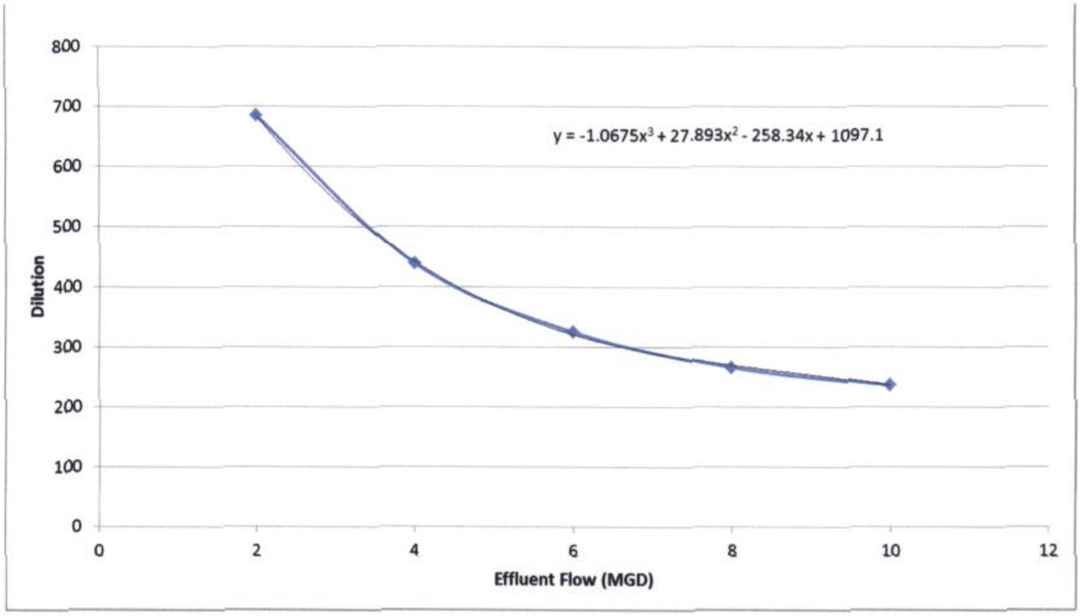
ASPA has retained GDC to perform a study that describes the relative effects on the CID and ZID by modifying the diffusers found in the outfalls. These effects are described as a function of effluent flow so that the effects of reducing effluent flows on the CID and ZID may be easily determined. In its study, CVL has identified several projects that would reduce I&I and equalize peak flows to the STPs thereby resulting in reductions in effluent discharges to the outfalls. The GDC and CVL studies are complementary and the resulting impacts to the CID and ZID are presented in this report. Appendix L contains a full copy of GDC's technical memorandum in which diffuser configuration and performance for each STP outfall is described.

The results of diffuser performance analyses for the STP outfalls are presented in this section as a summary of GDC's report. Increasing the CID and the ZID concurrently is not always the result of diffuser modifications as can be seen in the analyses presented in Appendix M. Increases in CID may not result in increases in the ZID. The overall goal of regulatory agencies is generally to maximize the CID with a minimum mixing zone size (in this case a minimum ZID). The GDC analyses was focused on achieving the maximum practicable CID, which has the most desirable effect on minimizing impacts of a discharge of pollutants to the receiving water. Therefore, the evaluation of the projects presented is based on the achievable CID.

10.2 Tafuna Outfall

GDC has stated that substantial increases in dilution could be achieved by modifying the existing diffuser configuration without extending the diffuser barrel or outfall. It is recommended to reduce existing ports from 8-inches to 6-inches in diameter and provide an end port of 12.5 inches in diameter. The dilution model was run using the appropriate flows for the modified port configuration at the point of maximum rise of the discharge plume based on realistic merging behavior. See Appendix L, Section 6-1 for a full discussion. The resulting evaluation yielded the following dilution-effluent flow relationship graphed below, taken from Table 6-3 of the GDC Technical Memorandum.

Figure 24 Dilution for Modified Tafuna Diffuser Configuration



The equation calculating the change in diffusion shown in the graph in Figure 24 is

$$y = -1.0675\,x^3 + 27.893\,x^2 - 258.34\,x + 1097.1$$

The effective dilution increases with decreasing flows. For example a flow of 10 MGD has a resulting dilution value of approximately 235; a flow of 6 MGD has a dilution value of approximately 325. Reducing flows from 10 to 6 MGD in the diffuser barrel results in a 38% increase in dilution and corresponding increase in the CID:

$$Percent\ Dilution\ Increase = \frac{325 - 235}{235} = 38\%$$

Table 30 and Table 31 below list the reductions in STP effluent flows resulting from improvements projects identified in Sections 5.0, 6.0, and 8.0 and the resulting increases in dilution. The dilution values are taken directly from Figure 24.

Table 30 Tafuna: Increase in Dilution at Maximum Mean Flows

Project to Increase Dilution	Max Mean Flows (MGD)	Reduced[1] Max Mean (MGD)	Dilution of Max Mean Flow		Increase in Dilution	% Increase	Reference
			Existing	With Improve-ment			
Infiltration and Inflow Upgrades							
Sewers below Sea Level	2.7	2.667	582	586	4	0.7%	Section 5.3.1
Areas with Suspected I&I	2.7	2.673	582	585	4	0.6%	Section 5.3.1
Entire Sewer System	2.7	1.600	582	684[2]	118	20.3%	Table 11
Sewer System Storage Capacity Upgrade							
Storage in the System							
Vaitele LS	0.506	0.473	582	586	4	0.7%	Table 21
Airport LS	1.119	1.113	582	583	1	0.1%	Table 21
Equalization at Lift Stations							
Vaitele LS	0.506	0.301	582	610	28	4.8%	Table 21
Airport LS	1.119	0.664	582	646	64	10.9%	Table 21
On-site Wet Weather Storage							
Equalization at Tafuna	2.7	1.6	582	684[2]	118	20.3%	Table 28

[1] Reductions based on implementation of listed projects to increase dilution.
[2] These calculations may not be reliable below 2.0 MGD as noted in the GDC Technical Memorandum. The minimum reliable dilution at 2 MGD is shown.

Table 31 Tafuna: Increase in Dilution at Peak Flows

Project to Increase Dilution	Peak Discharge Flows (MGD)	Reduced[1] Peak Discharge (MGD)	Dilution of Peak Flow		Increase in Dilution	% Increase	Reference
			Existing	With Improve -ment			
Infiltration and Inflow Upgrades							
Sewers below Sea Level	6.2	6.167	313	316	3	0.9%	Section 5.3.1
Areas with Suspected I&I	6.2	6.173	313	315	2	0.7%	Section 5.3.1
Entire Sewer System	6.2	5.1	313	469	156	49.8%	Table 11
Sewer System Storage Capacity Upgrade							
Storage in the System							
Vaitele LS	1.16	1.130	313	314	1	0.4%	Table 21
Airport LS	2.57	2.564	313	313	0	0.1%	Table 21
Equalization at Lift Stations							
Vaitele LS	1.16	0.506	313	340	27	8.7%	Table 21
Airport LS	2.57	1.119	313	385	72	22.9%	Table 21
On-site Wet Weather Storage							
Equalization at Tafuna	6.2	2.7	313	582	269	85.8%	Table 28

[1] Reductions based on possible projects to increase dilution.

10.3 Utulei Outfall

GDC notes that although the existing diffuser configuration is expected to provide sufficient dilution, an improvement in the dilution effectiveness of the Utulei Outfall may be achieved by reducing the size of the existing ports from 6 inches to 5.5 inches in diameter and adding a 10.5 inch diameter end port. The length of the outfall structure is to remain the same. The values for dilution at the maximum rise of the discharge plume assuming realistic merging behavior are shown in Table 6-5 of the report in Appendix L and are plotted in Figure 25 below. See Section 6-2 for a full discussion.

Figure 25 Dilution for Modified Utulei Diffuser Configuration

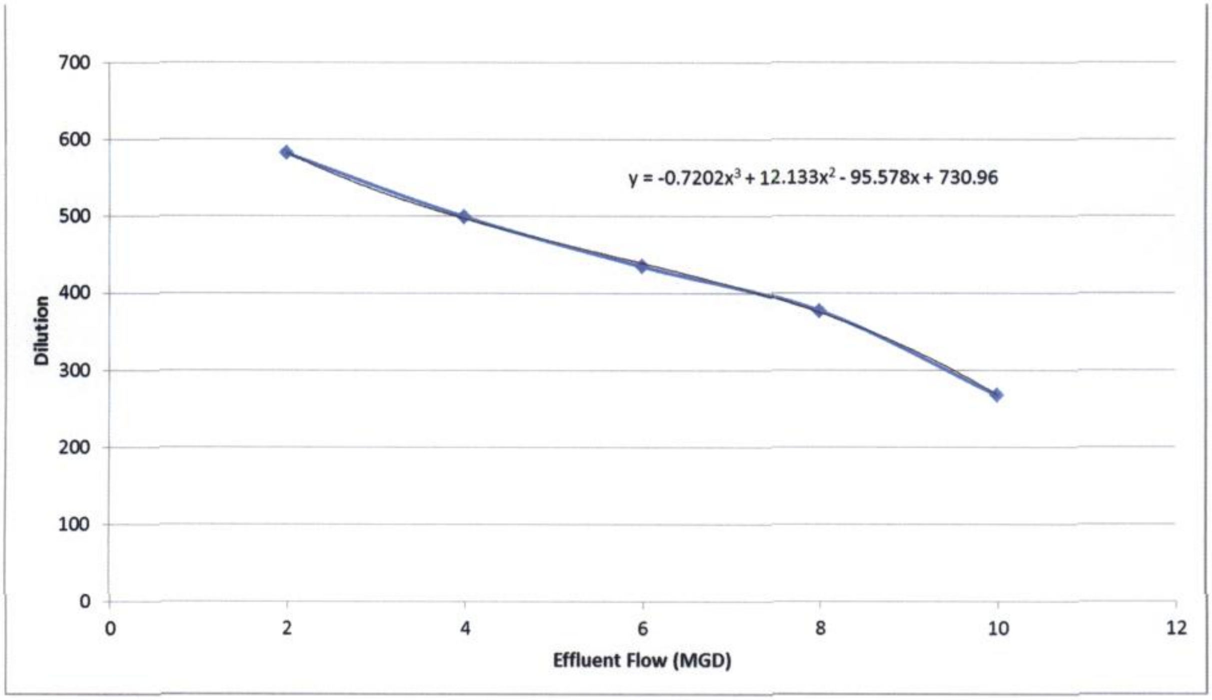


Table 32 and Table 33 below list the reductions in STP effluent flows resulting from improvements projects identified in Sections 5.0, 7.0, and 9.0 and the resulting increases in dilution.

The equation calculating the change in diffusion shown in the graph in Figure 25 is

$$y = -0.7202\, x^3 + 12.133\, x^2 - 95.578\, x + 730.96$$

Table 32 Utulei: Increase in Dilution at Maximum Mean Flows

Project to Increase Dilution	Max Mean Flows (MGD)	Reduced[1] Max Mean (MGD)	Dilution of Max Mean Flow		Increase in Dilution	% Increase	Reference
			Existing	With Improve-ment			
Infiltration and Inflow Upgrades							
Sewers below Sea Level	2.8	2.109	544	578	34	6.2%	Section 5.3.2
Areas with Suspected I&I	2.8	2.749	544	545	2	0.4%	Section 5.3.2
Entire Sewer System	2.8	1.000	544	583[2]	39	7.3%	Table 11
Sewer System Storage Capacity Upgrade							
Storage in the System							
Utulei Inflow LS	2.8	2.554	544	554	11	2.1%	Table 26
Equalization at Lift Stations							
Malaloa LS	2.2	0.783	544	583[2]	39	7.1%	Table 26
Faga'alu LS	0.4	0.132	544	557	13	2.4%	Table 26
On-site Wet Weather Storage							
Equalization at Utulei	2.8	1.000	544	583[2]	39	7.3%	Table 29

[1] Reductions based on implementation of listed projects to increase dilution.
[2] These calculations may not be reliable below 2.0 MGD as noted in the GDC Technical Memorandum. The minimum reliable dilution at 2 MGD is shown.

Table 33 Utulei: Increase in Dilution at Peak Flows

Project to Increase Dilution	Peak Discharge Flows (MGD)	Reduced[1] Peak Discharge (MGD)	Dilution of Peak Flow		Increase in Dilution	% Increase	Reference
			Existing	With Improve-ment			
Infiltration and Inflow Upgrades							
Sewers below Sea Level	6.0	5.002	440	468	28	49.4%	Section 5.3.2
Areas with Suspected I&I	6.0	5.926	440	441	2	40.7%	Section 5.3.2
Whole Sewer System	6.0	3.400	440	518	79	65.4%	Table 11
Sewer System Storage Capacity Upgrade							
Storage in the System							
Utulei Inflow LS	6.0	5.754	440	446	7	42.3%	Table 26
Equalization at Lift Stations							
Malaloa LS	4.7	2.184	440	531	90	69.5%	Table 26
Faga'alu LS	0.8	0.368	440	454	13	44.9%	Table 26
On-site Wet Weather Storage							
Equalization at Utulei	6.0	2.800	440	543	104	73.3%	Table 29

11.0 CONCLUSIONS AND RECOMMENDATIONS

In this section, techniques and procedures available to mitigate excess infiltration/inflow, control lift station pumping operation, and provide flow equalization in the collection system and at the treatment plants so as to increase the Critical Initial Dilution Factor (CIDF) at the outfalls is presented. A list of improvement projects is shown, evaluated, and ranked. The most cost effective solutions are presented and recommended for future study.

11.1 Description of Proposed Improvements to Increase Initial Dilution Factor

11.1.1 Sewer Collection System Improvements

- Improvements to sewer collection facilities
  - Chemical or Cement Grouting

    Cement or chemical grouting sometimes can be used to rehabilitate sewer lines externally by excavating adjacent to the pipe. Internal chemical grouting is a more common technique to rehabilitate leaking joints, manhole walls, and minor cracks in non-pressurized pipelines. Chemical grouting is not a good option if the cracks in the pipe are large. Grouting does not improve or reinforce the structural integrity of the pipes, and large joints and cracks and misaligned pipe may be impossible to seal or may require excessive amounts of grout. Types of chemical grouts used for sewer pipe repairs include acrylic-based gels, urethane gel, and polyurethane foam. The sewer lines must be cleaned thoroughly prior to application of grout.

  - Liners

    Sections of pipeline can be rehabilitated internally by sliding a somewhat smaller diameter flexible liner pipe inside an existing pipe and then reconnecting the service lines to the new liner. The space between the existing pipe and liner pipe may be grouted to provide added strength to the line. Slip-linings for sewers can be constructed of polyethylene, fiberglass reinforced polyester, polyvinyl chloride (PVC), or other materials resistant to the corrosive atmosphere in the collection system. They may be inserted as a single flexible continuous pipe or in short sections that can be jointed to form a continuous lining. Cured-in-place pipe (CIPP) is a practical method for correcting I&I and rehabilitating pipes that need minor structural reinforcement. CIPP is formed by inserting a polyester or epoxy resin-filled felt tube into a pipe, which is inverted against the inner wall of the existing pipe and then allowed to cure. A remote cutting device is used with a closed-circuit camera to reopen service connections. This method is practical because the resin can bridge gaps, fill cracks, and maneuver around pipe defects.

    CIPP lining may be used in vertical or horizontal pipes, with inside diameter ranging from 0.5" to 48". For pipes larger than 48", a spray-on epoxy pipe lining process is used. Pipe lining is sometimes referred to as "Trenchless Pipe Lining", because there are little or no excavation required and less invasive to landscape and surrounding environment.

    The advantages of CIPP and pipe lining are listed below:

    - Less excavation, less digging and pipe cutting required. Less invasive to landscape and the surrounding environment. Less of heavy equipment or machineries used.

- May not require construction permit from municipalities. Less delaying time dealing with other utilities companies and less utilities down time. Sectional "repair" lining is not considered "replacement", therefore no permitting require and the work can commence immediately.
- CIPP lining can be installed in area with high underground water tables.
- CIPP lining can be installed in area of cracked or damaged pipe section,
- CIPP lining material has up to 100-year engineered life and typically comes with a 50-year warranty, depending on the liner manufacturers (Perma-Lateral Lining System).
- Provided smoother lining surface which has lower coefficient of friction than concrete or clay pipe material, therefore flow capacity may be improved.
- CIPP liner offers the structural strength of a new pipe condition.
- Total cost is much less compared to "traditional pipe replacement" methods, such as excavation and repair or replacement of damaged pipe sections.

The disadvantages of CIPP and pipe lining are listed below:

- CIPP lining reduce the inner pipe diameter by 5%. This may not affect the flow capacity because CIPP has lower coefficient of friction of the smooth liner surface.
- Mobilization costs are high due to the location of the work in American Samoa.

- Manhole Repair

  Old, degraded, or leaky manhole structures that are low lying are subject to inflow from accumulating stormwater runoff and are likely a major source of I&I. Many of the options for reducing I&I through manholes are similar to the options listed above for rehabilitating sewer lines. For example, chemical grouting or sealing is a cost effective method. Cement coatings and chemical patching compounds have both been used to coat manhole walls to reduce flow. Manholes can be structurally rehabilitated with the use of linings. Poured-in-place concrete linings, for example, have been used effectively. Other choices include PVC rib-lock liners, prefabricated reinforced plastic mortar or fiberglass reinforced plastic, prefabricated high density polyethylene, spiral-wound liner, epoxy liners, and cured-in-place structural manhole liners. In addition, unwanted inflow can enter manholes through ill-fitting covers and leaky frames. Leaky manhole covers can be replaced with new watertight covers, or they can be fixed with rubber gaskets, rain dishes, or by installing hole-plugs or watertight inserts under the existing covers.

- Sewer Joint Repair

  Sewer joints may be a major source of I&I if they allow ground water and/or seawater into the collection system through open or misaligned joints. Leaking joints may be rehabilitated by sealing the leaking joints with a pressure grout. Special machines are pulled through the sewer to test a joint with air pressure and then add grout under pressure to those joints that fail the pressure test.

o Replace or install new piping with alternative low pressure systems

Alternative collection systems may be preferred in areas with high groundwater or areas where sewer pipes are located near the shore below the high tide elevation where water may seep into the system. Small diameter gravity sewers (SDGS) convey effluent by gravity from an interceptor tank (or septic tank) to a centralized treatment location or pump station. These systems generally use smaller diameter pipes with a slight slope or follow the surface contour of the land, reducing the amount of excavation and construction costs.

Most suspended solids are removed from the waste stream prior to pumping by septic tanks, or reduced in size by grinder pumps decreasing the potential for clogging to occur and allowing for smaller diameter piping downstream. Cleanouts are used to provide access for flushing; manholes are rarely used. Air release risers are required at or slightly downstream of summits in the sewer profile. Odor control is important at all access points since the SDGS carries odorous septic tank effluent. Because of the small diameters and flexible slope and alignment of the SDGS, excavation depths and volumes are typically much smaller than with conventional sewers. Minimum pipe diameters can be three inches. Plastic pipe is typically used because it is economical in small sizes and resists corrosion. (USEPA, 2000)

We note that due to the additional maintenance required by the utility to pump septic tanks and dispose of the septage, the requirement for additional easements for access make this option impractical.

- Identify interceptors/sewers below high tide elevation

  Sewer installations located below the high tide elevation are susceptible to infiltration from sea water. These sewers are identified on Figure 14 and Figure 16 and should be investigated by video inspection for the presence of cracks or leaks. Flow through manholes and lift stations can also be tested for salinity as discussed in Section 4.5.1 to help pinpoint target areas for inspection.

- Determine chloride concentrations on inflow or effluent to assess seawater infiltration

  The existing collection system has many miles of sewer within the Island's coastal roadways and is subject to potential saltwater intrusion. The magnitude of saltwater infiltration along the shoreline where sewers are located below the high tide level can be determined by measuring the chloride concentrations in the inflow or effluent at the plant and compare to potable water or upstream sewage. Some work, limited to one measurement at each STP, has been performed recently that indicates that up to 12% of the flow in the Utulei interceptor to the treatment plant is sea water. The results also indicate that the Tafuna STP collection system is not subject to significant seawater intrusion. See Section 4.5 for a more detailed analysis.

- Trenches in Lava Rock

  Many areas on the Island have volcanic bedrock extending close to the surface. Sewer facilities in these areas were installed in trenches and backfilled with granular material, as specified by ASPA requirements. These may behave like troughs to concentrate infiltrated water and create the potential for additional infiltration in the system. Sewer facilities in these areas also need to be monitored closely for cracks or leakage through a program of video inspection. Sewers and manholes found to have cracks and leaks should be repaired.

The approximate footage of pipes in these areas may be determined by reviewing the drilling and blasting contracts issued during installation of the system. Dig conditions may also be noted on as built drawings.

11.1.2 Lift Station Operation Improvements

- Install pump run meters/data loggers

  ASPA does not presently monitor flow in the collection portion of the Tafuna and Utulei sewer systems. An attempt was made to calculate the amount of sewage conveyed in the collection system and at the lift stations from the pump run times. This is typically a dependable and simple flow measurement alternative for systems using constant speed pumps and pump run time meters. Pump run times are determined and logged by facility personnel. Time intervals at which the run times are logged are not consistent. Occasionally run times are averaged over several days. This allows only for run time approximations on a weekly basis or for longer time periods. Daily pump run time information is not accurate. See Section 6.2 for a full discussion. The installation of data loggers would provide a reliable and effective means of recording pump run times and provide ASPA with an important tool to more cost effectively operate lift station pumps.

- Upgrade/overhaul pumping equipment and controls

  All pump stations but especially those located in the Bay Area system that were affected by the 2009 Tsunami must be fully inspected and a list of necessary repairs identified. The original design of the pump stations appears to be adequate but some of the equipment is not performing at expected levels and ASPA is engaged in a long term program to upgrade its lift stations (see Appendix H, Lift Station Maintenance Logs).

- Install Variable Frequency Drives

  Section 6.3.1 discusses the advantages of VFD installation. This installation of VFD does not reduce the quantity of I&I but does result in an attenuation of discharge from the lift station. Other benefits include increased equipment life, increased pumping efficiency, and a reduction in poser costs.

  The installation of VFDs for the pumps at the Utulei STP influent pump station is under construction (as of June 2012). Installation of VFDs at six new lift stations is also recommended as part of the East Side Villages Wastewater Collection System Sewer Upgrade. The installation of VFDs is also recommended at large lift stations. In the Tafuna System VFDs are recommended for the Vaitele and the Airport lift stations as well as for the STP influent pump station; in the Bay Area system VFDs are recommended for the Malaloa LS and the Faga'alu LS.

- Adjust pump operating levels

  Pumps in wet wells are typically operated via float levels set a certain elevations. For duplex lift stations, where the average flows can be handled with one pump, flow alternation may be achieved by maximizing the volume at which only one pump operates to reduce spiking caused by the activation of a second pump pumping for a short time. For lift stations adjustment of the operating levels should be carefully investigated. Greatest equalization can be achieved by maximizing the storage volume for the operating condition at which the pumping capacity matches expected inflows. For low flows that is likely one pump running and for higher flows that is when two pumps are running. For a complete discussion with an example using the Vaitele lift station, see Section 6.3.1.

11.1.3 Equalization in the System

The purpose of flow equalization is to minimize or control the fluctuations in effluent flows to avoid peaks that could cause exceedances in discharge limits for the STP.

- Equalization at lift stations

  Providing additional storage volume at the lift stations may equalize discharges to the STP by providing capacity to regulate flows to a predetermined rate and temporarily storing all flows above the selected rate. See discussion in Section 6.3.3.

- Equalization in the collection system

  The existing piping system may be used for temporary storage volume. In areas where the system geometry permits and manholes would not overflow, pump levels may be adjusted to start pumps at or above the inflow pipe invert. Equalization in the existing piping requires upgrading the system prior to changing float levels. If the system is susceptible to infiltration, exfiltration will also be a problem. The integrity of the lines targeted for system storage should be confirmed first and where issues are noted the lines should be repaired.

  Larger diameter lines are able to store a greater volume of water. Collection system storage for smaller diameter lines is generally not cost effective. We note that the system's topography is important in evaluating the desirability of this method as hilly terrain will reduce the available potential storage in pipes installed at steep slopes. See Section 6.4 and 7.4 for a discussion of potential system storage in the Tafuna and Utulei STP service areas.

11.1.4 Equalization Basin Installation at Treatment Plants

Adding equalization volume at the treatment plants is the most beneficial option from an operational perspective but it has the greatest capital cost. Because total flows from the entire system are attenuated, equalization at the STP requires a larger volume and more space. See section 8.0 for a discussion of equalization at the Tafuna STP site and Section 9.0 for a discussion of the Utulei STP.

11.1.5 Road Drainage Issues

At many locations, the poor condition of island roads under designed road geometry and lack of storm drains do not allow for the rapid drainage of stormwater. Runoff accumulates in low spots creating ponding resulting in additional potential for inflow into manholes and the sewer system. As roadway design issues are resolved and a storm drain system is installed the potential for I&I to the sewer system will decrease. A quantification of this magnitude of I&I reduction is beyond the scope of this project.

11.2 Selection Criteria

This Section discusses the selection criteria that were used to compare the various projects aimed at increasing the initial dilution of effluent discharged from the treatment plants.

11.2.1 Costs of Alternatives

An estimate of cost for each of the projects discussed in Section 11.1 has been completed. To compare the different projects with each other, the cost was normalized to a comparative cost/gallon of I&I removed. Table 34 and Table 35 list the unit costs of improvement projects and associated estimated costs. Costs assume contractor provided and installed and include adjustments for American Samoa. Land costs are not included. Units and detailed costs for each item of work may be found in Appendix M.

Table 34 List of Projects, Tafuna Sewer System

	Items	Cost ($)
Infiltration and Inflow Upgrades		
Sewers below Sea Level	Video, Flushing, Grouting Pipe, Grouting Manholes	251,800
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	380,722
	Dollar per Gallon Reduction in Flow (Peak)	5.08
Areas with Suspected I&I	Video, Flushing, Grouting Pipe, Grouting Manholes	367.613
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	555,830
	Dollar per Gallon Reduction in Flow (Peak)	9.11
Sewer System Storage Capacity Upgrade		
Equalization, Vaitele LS	Equalization Basin, EQ Return Pumps, EQ Diversion Structure, Aeration, Mixing, Piping, Odor Control, Electrical & Controls, Generator, Site Improvements	680,000
Storage in the System, Vaitele LS	Video, Flushing, Grouting Pipe, Grouting Manholes	208,760
	Subtotal	888,760
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	1,343,805
	Dollar per Gallon Reduction in Flow (Peak)	2.05
Equalization, Airport LS	Equalization Basin, EQ Return Pumps, EQ Diversion Structure, Aeration, Mixing, Piping, Odor Control, Electrical & Controls, Generator, Site Improvements	930,000
Storage in the System, Airport LS	Video, Flushing, Grouting Pipe, Grouting Manholes	27,200
	Subtotal	957,200
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	1,447,286
	Dollar per Gallon Reduction in Flow (Peak)	1.00
On Site Wet Weather Storage		
Equalization at Tafuna	Equalization Basin, EQ Return Pumps, EQ Diversion Structure, Aeration, Mixing, Piping, Odor Control, Electrical & Controls, Generator, Site Improvements	2,245,000

	Items	Cost ($)
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	3,394,440
	Dollar per Gallon Reduction in Flow (Peak)	0.97

Tafuna Ocean Outfall		
Diffuser Modifications	12.5 inch end gate port, restriction of existing port areas from 8 inches to 6 inches	150,000[1]
	Dollar per Gallon Reduction in Flow (Peak) not applicable	

[1] From GDC Technical Memorandum, Appendix L

Table 35 List of Projects, Bay Area Sewer System

	Items	Cost ($)
Infiltration and Inflow Upgrades		
Sewers below Sea Level	Video, Flushing, Grouting Pipe, Grouting Manholes	1,089,200
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	1,646,870
	Dollar per Gallon Reduction in Flow (Peak)	1.65
Areas with Suspected I&I	Video, Flushing, Grouting Pipe, Grouting Manholes	226,830
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	342,967
	Dollar per Gallon Reduction in Flow (Peak)	4.63
Sewer System Storage Capacity Upgrade		
Equalization, Malaloa LS	Equalization Basin, EQ Return Pumps, EQ Diversion Structure, Aeration, Mixing, Piping, Odor Control, Electrical & Controls, Generator, Site Improvements	1,230,000
Storage in the System, Malaloa LS	Video, Flushing, Grouting Pipe, Grouting Manholes	314,198
	Subtotal	1,544,198
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	2,334,827
	Dollar per Gallon Reduction in Flow (Peak)	0.81

	Items	Cost ($)
Equalization, Faga'alu LS	Equalization Basin, EQ Return Pumps, EQ Diversion Structure, Aeration, Mixing, Piping, Odor Control, Electrical & Controls, Generator, Site Improvements	580,000
Storage in the System, Faga'alu LS	Video, Flushing, Grouting Pipe, Grouting Manholes	74,318
	Subtotal	654,318
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	989,328
	Dollar per Gallon Reduction in Flow (Peak)	2.03

On Site Wet Weather Storage		
Equalization at Utulei	Equalization Basin, EQ Return Pumps, EQ Diversion Structure, Aeration, Mixing, Piping, Odor Control, Electrical & Controls, Generator, Site Improvements	2,320,000
	Total including 8% Engineering, 10% Overhead & Profit, 8% Bonding & Insurance, 20% Contingency	3,507,840
	Dollar per Gallon Reduction in Flow (Peak)	1.10

Utulei Ocean Outfall		
Diffuser Modifications	10.5 inch end gate port, possible restriction of existing port areas from 6 inches to 5.5 inches performed in tandem with underwater outfall repair/maintenance work.	190,000[1]
	Dollar per Gallon Reduction in Flow (Peak) not applicable	

[1] From GDC Technical Memorandum, Appendix L

Table 36 identifies other improvements to the sewer systems that may be considered to improve operational efficiency and provide better flow data.

Table 36 List of Miscellaneous Projects, Tafuna and Bay Area Sewer System

Other Improvements						
	Tafuna System			**Bay Area System**		
	Unit Cost ($)	Quantity	Total Cost ($)	Unit Cost ($)	Quantity	Total Cost ($)
Treatment Plants						
Flowmeters	40,000	1	40,000	40,000	1	40,000
Data Logger	30,000	1	30,000	30,000	1	30,000
VFD	75,000	4	300,000			
Total			370,000			70,000

Lift Stations						
	Vaitele (0.5 MGD Max Mean)			Malaloa (2.2 MGD Max Mean)		
Flowmeters	15,000	1	15,000	25,000	1	25,000
Data Logger	15,000	1	15,000	15,000	1	15,000
VFD	30,000	2	60,000	60,000	2	120,000
Total			90,000			160,000
	Airport (1.1 MGD Max Mean)			Faga'alu (0.4 MGD Max Mean)		
Flowmeters	20,000	1	20,000	15,000	1	15,000
Data Logger	15,000	1	15,000	15,000	1	15,000
VFD	50,000	2	100,000	25,000	2	50,000
Total			135,000			80,000

11.2.2 Technical/Operational Feasibility

The improvement projects are also examined for technical and/or operational challenges.

Rehabilitation of the collection systems to prevent inflow and infiltration would eliminate the need for equalization facilities and associated equipment at the lift stations or treatment plants. This option would likely require a much higher capital investment, but would benefit ASPA by reducing operations and maintenance expenses.
The advantages include:

- no maintenance of equalization basins, pumps and associated equipment
- reduced pump sizes
- reduced power costs
- reduced treatment costs
- increased available treatment capacity
- improved treatment plant performance

Construction of equalization facilities at the pump stations or treatment plants would likely cost significantly less than rehabilitation of the collection systems. However, unless some investment is made in preventing inflow and infiltration in the collection system, inflow and infiltration flows will continue to increase as the systems deteriorate further over time.

11.2.3 Constructability

The improvement projects are also assessed for constructability and ease of implementation.

Acquisition of additional land may be required to build the equalization facilities near the lift station and treatment plant sites. Negotiations for land could become a lengthy and cumbersome process, and there would be no guarantee that sufficient agreements could be made. However, if land is acquired the necessary improvements could be constructed relatively quickly after they are designed.

Rehabilitation of the collection systems would likely be a long and drawn out process. The collection systems would first have to be investigated by videotaping the lines to determine the locations and degree of repairs that would likely be required. It would need to be determined which areas require immediate repair and which areas could be repaired at a later point in time. The budget and schedule for making the repairs would then need to be established. Repair work would likely impede traffic to some degree and traffic control measures would need to be taken.

11.2.4 Completion Time

This section contains a brief estimate of time requirements to build each of the improvement projects. Projects that cannot be completed by the AO imposed deadline of June 30, 2013, are dropped from consideration.

It is unlikely that a complete investigation and repair of both collection systems could be completed by the June 30, 2013 deadline. It is possible that progress could be made in known problematic areas, especially in Faga'alu to coincide with greater watershed protection efforts now underway in this region, and some new areas identified through video investigation could be repaired before the deadline. However, investigation of the entire collection system is likely to be a lengthy process, and repairs should be implemented on the most problematic areas first beginning with the sewer interceptors in the Bay Area sewer collection system identified in Figure 16 and Section 5.3.2.

It is unlikely that any of the equalization facilities could be completed by the June 30, 2013 deadline, unless they could be built within or with minor expansion of existing lift station sites. Land would have to be acquired and an engineer and a contractor would need to be procured. Design and construction would likely take a minimum of a year to complete.

11.2.5 Critical Initial Dilution and Zone of Initial Dilution Improvements

Increasing the critical initial dilution (CID) and ZID may be achieved in the following ways, singly or in combination:

- Decrease effluent flows overall by eliminating system I&I
- Equalize effluent flows to eliminate peaks
- Changes in outfall structure design to improve dilution. See Section 10.0 and Appendix L for discussion.

11.3 Selection Matrix

In this section the improvement projects proposed above are evaluated and ranked in order of most desirable to least recommended. The selection criteria listed above are assigned different weights to account for the importance of a criterion to achieve the required CID and to prevent future exceedances. A selection Matrix was completed that compares the improvement projects with each other by assigning scores for each of the criteria.

11.3.1 Selection Criteria Weight

Selection criteria were assigned numerical weights as follows, with "5" being the most important and "1" of lesser importance:

CID/ZID	5
Capital Cost	4
Technical/Operational Feasibility	2
Constructability	1
Construction Duration	5

The effect on CID and construction duration were determined to be the most important of the criteria since projects must be completed by June 30, 2013, as stipulated by AO. They are therefore assigned the highest value. The capital cost of design was also ranked high meaning less desirable. When an improvement project score is multiplied by the criteria weight listed above, the project with the larger score will be more desirable for implementation. Table 37 below contains a matrix of the proposed projects investigated and their ranking.

11.3.2 Scores

Each project is given a numerical value from 5 to 1 to describe the relative weight of the effort or complexity to achieve the desired result. Scores are numerical and signify "best" for 5 and "least desirable" for 1 as listed below:

Best	5
Excellent	4
Good	3
Acceptable	2
Least Desirable	1

Table 37 Selection Matrix for Tafuna

	Effect on CID		Capital Cost		Technical/ Operational Feasibility		Construct-ability		Time to Complete		Final Score
Project Description		**5**		**4**		**2**		**1**		**5**	
Sewer Collection System Improvements											
Improvements to known problem areas in the existing collection system facilities	1	5	1	4	5	10	5	5	2	10	34
Improvements to areas where the collection system is installed below high tide elevations	1	5	1	4	5	10	5	5	3	15	39
Lift Station Operation Improvements											
Storage at Vaitele	3	15	4	16	3	6	3	3	2	10	50
Storage at the Airport	4	20	5	20	3	6	3	3	2	10	**59**
Equalization Basin Installation at Treatment Plants											
Tafuna	5	25	4	16	2	4	2	2	1	5	**52**
Outfall Adjustments											
Diffuser Modifications	5	25	5	20	5	10	3	3	3	15	**73**

Table 38 Selection Matrix for Utulei

	Effect on CID		Capital Cost		Technical/ Operational Feasibility		Construct-ability		Time to Complete		Final Score
Project Description		**5**		**4**		**2**		**1**		**5**	
Sewer Collection System Improvements											
Improvements to known problem areas in the existing collection system facilities	1	5	1	4	5	10	5	5	2	10	34
Improvements to areas where the collection system is installed below high tide elevations	4	20	2	8	5	10	5	5	1	5	48
Lift Station Operation Improvements											
Storage at Malaloa	3	15	5	20	3	6	3	3	3	15	**59**
Storage at Faga'alu	2	10	5	20	3	6	3	3	3	15	**54**
Equalization Basin Installation at Treatment Plants											
Utulei	5	25	3	12	1	2	1	1	1	5	45
Outfall Adjustments											
Diffuser Modifications	5	25	5	20	5	10	3	3	3	15	**73**

11.4 Recommendations

It is recommended that ASPA proceed immediately with the diffuser modifications for both outfalls as described in Section 10.0. These improvements will provide the greatest increase in effluent dilution for the least amount of capital expense.

The design and construction of the equalization storage facilities in the Tafuna system at the Airport lift station and at the Malaloa lift station for the Utulei system (see Section 8.0 for more detail) are also recommended as viable projects that significantly increase outfall dilution.

Other projects are not recommended because of construction periods extending beyond the 30 June 2013 deadline established by the Administrative Orders or do not significantly increase dilution values at the outfall. These projects may be considered by ASPA for future improvements to the system as funding becomes available.

It is also recommended that ASPA consider improvements to the Utulei collection system where the system is known to be installed below the high tide elevation (see Section 5.3.2) and the improvements to the system at Faga'alu. This work will result in significant reduction in I&I to the Utulei STP and improve the disinfection efficiency of contemplated chlorine based disinfection systems by reducing chlorine demands from nitrogen compounds found in seawater. Improvements to the system at Faga'alu is also recommended due to the coral reef conservation efforts to reduce the impacts from land based sources of pollution described in Section 2.2.5.

This project cannot be recommended for implementation because it will not likely be completed within the 30 June 2012 deadline established by the AO. ASPA may consider phasing this work as part of a long term program to improve its collection system.

The anticipated schedules shown below are conceptual and will be refined during design.

Diffuser Modifications – Tafuna Outfall

NTP	1 August 2012
Design Phase	1 August – 1 September 2012
Bidding Phase	1 September – 15 October 2012
Award / NTP	1 November 2012
Mobilization	1 November – 1 December 2012
Outfall Repairs	1 December 2012– 1 February 2013

Diffuser Modification – Utulei Outfall
As above for Tafuna

Equalization Basin at Airport Lift Station

NTP	1 August 2012
Design Phase	1 August – 1 November 2012
Permits / Approvals	1 November – 1 December 2012
Bidding Phase	1 December – 15 January 2013
Mobilization	15 January – 15 February 2013
Construction	15 February – 15 May 2013
Commissioning / Approvals	15 May – 15 June 2013

Equalization Basin at Malaloa

As above for Airport

12.0 REFERENCES

GDC, 2007. Small Community Wastewater Facilities Plan for the Villages of Leloaloa, Aua, and Onesosopo, American Samoa.

Hart Pacific Engineering, 2004. Harbor Sewer System Evaluation, American Samoa.

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USEPA, 2000. Decentralized Systems Technology Fact Sheet, Small Diameter Gravity Sewers.

USEPA, 1976. Evaluation of Flow Equalization at a Small Wastewater Treatment Plant, US Environmental Protection Agency, Cincinnati, OH. EPA-600/2-76-181

USEPA, 1979. Evaluation of Flow Equalization in Municipal Wastewater Treatment, US Environmental Protection Agency, Cincinnati, OH. EPA-600/2-79-096

Appendix A Administrative Order

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
REGION IX
75 Hawthorne Street
San Francisco, CA 94105

February 17, 2012

In Reply Refer To:
CWA-309(a)-11-016 and 017

Andra Samoa, CEO
American Samoa Power Authority
P.O. Box PPB
Pago Pago, AS 96799

**Re: Modification of CWA-309(a)-11-016 and 017
To Extend Deadlines for the Submittal of Preliminary Engineering Plans**

Dear Ms. Samoa:

This letter modifies the Administrative Orders issued on July 27, 2011 to the American Samoa Power Authority for the Tafuna and Utulei Sewage Treatment Plants. The modifications extend the order deadlines to submit preliminary engineering plans and scoping summaries of projects by an additional two months to **June 30, 2012**. All other requirements of the orders remain unchanged. The key dates are now as follows:

KEY DATES	ADMINISTRATIVE ORDERS CWA-309(a)-11-016 and CWA-309(a)-11-017
09/30/11	1. Submit a short response to the June 10, 2011 EPA inspection report.
10/01/11	6. Begin reporting sewage spills on the DMRs
12/01/11	7. Begin effluent sampling for nitrogen, phosphorus, turbidity, and ammonia.
06/30/12	2. Submit preliminary engineering plans for disinfection and de-chlorination.
06/30/12	5. Submit scoping summary of projects to increase the critical initial dilution factor.
06/30/13	3. Install and begin the use of disinfection and de-chlorination. 8. Upon start-up begin effluent sampling for bacteria, and residual chlorine.
1st report due 02/28/12	12-14. Quarterly status reports and quarterly self-monitoring results. (due 02/28 for Oct-Dec, 05/30 for Jan-Mar, 08/30 for Apr-Jun, 11/30 for Jul-Sep)

If you have any questions regarding this matter, please contact Greg V. Arthur of my staff at (415) 972-3504 or at arthur.greg@epa.gov.

Sincerely,

Original signed by:
Nancy Woo for

Alexis Strauss
Director, Water Division

cc: To'afa Vaigaga'e, Director, ASEPA
Robert Kerns, ASPA Engineering

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
REGION IX
75 Hawthorne Street
San Francisco, CA 94105

CERTIFIED MAIL 7008 3230 0000 3863 1666
RETURN RECEIPT REQUESTED

July 27, 2011

In Reply Refer To: CWA-309(a)-11-016 and 017

Andra Samoa, CEO
American Samoa Power Authority
P.O. Box PPB
Pago Pago, AS 96799

Re: Administrative Orders the ASPA Sewage Treatment Plants

Dear Ms. Samoa:

Enclosed are two Administrative Orders issued today to the American Samoa Power Authority in order to resolve violations of the NPDES permits for the Tafuna and Utulei Sewage Treatment Plants by **June 30, 2013**. The Orders are identical, with identical schedules for each sewage treatment plant requiring (1) the installation of disinfection and de-chlorination, (2) expanded treatment plant self-monitoring, (3) spill reporting, and (4) an investigation of ways to increase the effectiveness of the deep-water outfalls. The key dates are as follows:

KEY DATES	ADMINISTRATIVE ORDER CWA-309(a)-11-016 for the TAFUNA STP
09/30/11	1. Submit a short response to the June 10, 2011 EPA inspection report.
10/01/11	6. Begin reporting sewage spills on the DMRs
12/01/11	7. Begin effluent sampling for nitrogen, phosphorus, turbidity, and ammonia.
04/30/12	2. Submit preliminary engineering plans for disinfection and de-chlorination.
04/30/12	5. Submit scoping summary of projects to increase the critical initial dilution factor.
06/30/13	3. Install and begin the use of disinfection and de-chlorination.
	8. Upon start-up begin effluent sampling for bacteria, and residual chlorine.
1st report due 02/28/12	12-14. Quarterly status reports and quarterly self-monitoring results. (due 02/28 for Oct-Dec, 05/30 for Jan-Mar, 08/30 for Apr-Jun, 11/30 for Jul-Sep)

KEY DATES	ADMINISTRATIVE ORDER CWA-309(a)-11-017 for the UTULEI STP
09/30/11	1. Submit a short response to the June 10, 2011 EPA inspection report.
10/01/11	6. Begin reporting sewage spills on the DMRs
12/01/11	7. Begin effluent sampling for nitrogen, phosphorus, turbidity, and ammonia.
04/30/12	2. Submit preliminary engineering plans for disinfection and de-chlorination.
04/30/12	5. Submit scoping summary of projects to increase the critical initial dilution factor.
06/30/13	3. Install and begin the use of disinfection and de-chlorination.
	8. Upon start-up begin effluent sampling for bacteria, and residual chlorine.
1st report due 02/28/12	12-14. Quarterly status reports and quarterly self-monitoring results. (due 02/28 for Oct-Dec, 05/30 for Jan-Mar, 08/30 for Apr-Jun, 11/30 for Jul-Sep)

The enclosed Orders and the findings that constitute the basis behind the Orders are issued pursuant to Sections 308(a) and 309(a)(3), (a)(4) and (a)(5)(A) of the Clean Water Act ("the Act") as amended [33 U.S.C. Sections 1318(a) and 1319(a)(3), (a)(4) and (a)(5)(A)]. Sections 309(a), (b), (d), and (g) of the Act [33 U.S.C. Sections 1319(a), (b), (d) and (g)], provide administrative and/or judicial relief for failure to comply with the Clean Water Act. In addition, Section 309(c) of the Act [33 U.S.C. Section 1319(c)], provides criminal sanctions for negligent or knowing violations of the CWA and for knowingly making false statements.

The requests for information included in these Orders are not subject to review by the Office of Management and Budget (OMB) under the Paperwork Reduction Act because it is not an "information collection request" within the meaning of 44 U.S.C. Sections 3502(4), 3502(11), 3507, 3512, and 3518. Furthermore, it is exempt from OMB review under the Paperwork Reduction Act because it is directed to fewer than ten persons [44 U.S.C. Section 3502(4), 3502(11) and 5 CFR Section 1320.5(a)].

EPA has promulgated regulations to protect the confidentiality of the business information it receives. These regulations are set forth in 40 CFR Part 2, Subpart B and in the Federal Register at 41 F.R. 36902 (September 1, 1976) and 43 F.R. 40000 (September 8, 1978). A claim of business confidentiality may be asserted in the manner specified by 40 CFR Section 2.203(b) for part or all of the information requested. EPA will disclose business information covered by such a claim only as authorized under 40 CFR Part 2, Subpart B. If no claim accompanies the business information at the time EPA receives it, EPA may make it available to the public without further notice. The American Samoa Power Authority may not withhold from EPA any information on the grounds that it is confidential.

If you have any questions regarding this matter, please contact Greg V. Arthur of my staff at (415) 972-3504 or at arthur.greg@epa.gov.

Sincerely,

Original signed by:
Alexis Strauss

Alexis Strauss
Director, Water Division

Enclosure

cc: LCDR Matt Vojik, ASEPA
Brad Rea, ASPA Engineering

UNITED STATES
ENVIRONMENTAL PROTECTION AGENCY
REGION 9

In the Matter of	)	
	)	
American Samoa Power Authority Tafuna Sewage Treatment Plant, Ocean Outfall, and Sewage Collection System NPDES Permit No. AS0020010	))))	FINDING OF VIOLATION AND ORDER
	)	Docket No. CWA-309(a)-11-016
Proceedings under Section 308(a) and 309(a)(3), (a)(4) and (a)(5)(A) of the Clean Water Act, as amended, 33 U.S.C. Section 1318(a) and 1319(a)(3), (a)(4) and (a)(5)(A)	))))	

STATUTORY AUTHORITY

The following Finding of Violation and Administrative Order (Docket No. CWA-309(a)-11-016) is issued under the authority vested in the Administrator of the U.S. Environmental Protection Agency (EPA) pursuant to Sections 308(a) and 309(a)(3), (a)(4) and (a)(5)(A) of the Clean Water Act [33 U.S.C. Sections 1318(a) and 1319(a)(3), (a)(4) and (a)(5)(A)] (hereinafter the Act). This authority has been delegated by the Administrator and the Regional Administrator of EPA Region 9 to the Director of the Water Division of EPA Region 9.

FINDING OF VIOLATION

The Director of the Water Division of EPA Region 9 finds that the American Samoa Power Authority, as the owner and operator of the Tafuna Sewage Treatment Plant, violated Section 301(a) of the Act [33 U.S.C. Section 1317(d)]. This Finding is made on the basis of the following facts:

1. The American Samoa Power Authority ("ASPA") owns the Fogagogo-Tafuna Sewage Treatment Plant ("Tafuna Sewage Treatment Plant"), the Tafuna ocean outfall, and the southwestern Tutuila Island sewer collection system ("Tafuna sewer system").

2. Section 301(a) of the Act [33 U.S.C. Section 1311(a)] prohibits the discharge of any pollutant by any person from a point source into waters of the United States except in compliance with a National Pollutant Discharge Elimination System (NPDES) permit issued in accordance with Section 402(a) of the Act [33 U.S.C. Section 1342]:
   a. Section 502(5) of the Act [33 U.S.C. Section 1362(5)] defines "person" to mean an individual, corporation, partnership, association, State, municipality, commission, or political subdivision of a State, or any interstate body;
   b. Section 502(6) of the Act [33 U.S.C. Section 1362(6)] defines "pollutant" to mean sewage, garbage, sewage sludge, rock, sand, chemical wastes, biological

materials, dredged spoil, solid waste, incinerator residue, munitions, radioactive materials, heat, wrecked or discarded equipment, cellar dirt, and industrial, municipal, and agricultural waste discharged into water;

c. Section 502(12) defines the term "discharge of pollutants" to mean any addition of any pollutant to navigable waters from any point source;

d. Section 502(7) defines the term "navigable waters" to mean the waters of the United States, including the territorial seas;

e. Section 502(14) defines "point source" to mean any discernible, confined and discrete conveyance, including but not limited to any pipe, ditch, channel, tunnel, conduit, well, discrete fissure, container, rolling stock, concentrated animal feeding operation, or vessel, or other floating craft, from which pollutants are or may be discharged.

3. The American Samoa Government is a State, and is therefore a person within the meaning of Section 502(5) of the Act [33 U.S.C. Section 1362(5)], and thus subject to the provisions of the Act, [33 U.S.C. Section 1251 et seq].

4. The American Samoa Power Authority is a political subdivision of the State, and is therefore a person within the meaning of Section 502(5) of the Act [33 U.S.C. Section 1362(5)], and thus subject to the provisions of the Act, [33 U.S.C. Section 1251 et seq].

5. The Pacific Ocean is a water of the United States.

6. On September 28 and 29, 2010, EPA conducted a diagnostic evaluation inspection of the Tafuna Sewage Treatment Plant, and Tafuna sewer system, and determined the following:

a. Facility Description: The Tafuna Sewage Treatment Plant provides primary treatment and undisinfected discharge through an ocean outfall to the Pacific Ocean:

(1) The Tafuna Sewage Treatment Plant consist of grit removal channels, manual bar screens, a deep-well influent pump station, three clarigesters, sludge drying beds, on-site dried sludge stockpiles, and a deep-water ocean outfall into the Pacific Ocean at Vai Cove;

(2) The Tafuna sewer system has one main lift station and ten satellite lift stations that together feed collected sewage into the Tafuna Sewage Treatment Plant influent pump station;

(3) The average daily and maximum peak flows in 2010 of 1.94 and 5.3 mgd are approaching the as-built design dry-weather and daily peak design critieria of 2.16 mgd and 6.0 mgd, respectively, for the Tafuna Sewage Treatment Plant;

(4) The 1999 EPA final decision extending the 301(h) waivers and the 2009 EPA tentative decision denying the waivers cited the critical initial dilution factors to be 190:1 in 1999 and 187:1 in 2009 for the zone-of-initial-dilution established for the Tafuna ocean outfall.

b. Facility Operations: The Tafuna Sewage Treatment Plant accepts and handles the following domestic wastewaters:

(1) Domestic sewage collected into the Tafuna sewer system;
(2) Digested sewage from the Utulei Sewage Treatment Plant, trucked from Utulei to the Tafuna Sewage Treatment Plant sludge drying beds;
(3) Restaurant grease, collected island-wide from the school lunch program, and delivered by pumper truck to the Tafuna Sewage Treatment Plant for disposal into a dedicated clarigester.

7. EPA issued the NPDES permit No.AS0020010 for the Tafuna Sewage Treatment Plant to become effective on November 2, 1999 and to expire on November 1, 2004. The 1999 NPDES permit authorized the discharge of treated domestic sewage from the Tafuna Sewage Treatment Plant through the Tafuna ocean outfall into the Pacific Ocean at Vai Cove.

8. The Federal regulations in 40 CFR 122.21(d) allow the administrative extension of an NPDES permit if a permit application is submitted for renewal at least 180 days before it expires. ASPA submitted an application for permit renewal on May 4, 2004, before the 180 day deadline. Therefore the NPDES permit is administratively extended to be in effect beyond the November 1, 2004 permit expiration date.

9. The 1999 NPDES permit advances less-than-secondary technology-based limits (based on a 1999 EPA 301(h) waiver variance final decision), receiving water limitations, sludge limits, and self-monitoring requirements. The 2004 renewal application included a request to extend the section 301(h) variance waiver.

a. Effluent Limits: The less-than-secondary limits for BOD and TSS removal rates are based on a Federal minimum of 30% for primary treatment. The BOD and TSS concentration and loading limits reflect past performance data.

NPDES Permit No. AS0020010 - § A(1)	Tafuna STP Effluent Limits					Self-Monitoring	
	mo-av	7d-av	d-max	instant	geo-μ	frequency	type
flow (mgd)	-	-	-	-	-	continuous	flume
BOD (mg/l)	100	150	200	-	-	once/week inf and eff	8hr comp
BOD (lbs/day)	1669	2504	3338	-	-		
BOD (%removal)	>30%	-	-	-	-		
TSS (mg/l)	75	113	150	-	-	once/week inf and eff	8hr comp
TSS (lbs/day)	1252	1878	2504	-	-		
TSS (%removal)	>30%	-	-	-	-		
settleable solids (ml/l)	1.0	-	2.0	-	-	once/day	grab
pH (s.u.)	-	-	-	6.5-8.6	-	once/week	grab
oil and grease (mg/l)	-	-	-	-	-	quarterly	grab
toxicity (TUc)	-	-	-	-	-	quarterly	24hr comp

b. Receiving Water Limits: The permit establishes limits to apply at and beyond the Zone of Initial Dilution ("ZID") based on the American Samoa water quality standards for the Pacific Ocean at Vai Cove. The receiving water permit limits are for discharges that cause water column samples taken at three depths (top, mid, bottom) from defined ZID water column sampling stations to exceed the standards. For turbidity, nutrients, chlorophyll-a, and enterococci, the receiving water permit limits are for discharges that cause averages (over the water column, ZID sampling locations, and a 12-month period) to exceed the standards.

NPDES Permit No.	Tafuna Outfall Water Column Limits					Self-Monitoring	
AS0020010 - § A(3), E(1)	mo-av	12m-av	d-max	instant	geo-μ	frequency	type
turbidity (NTU)	-	0.25	-	-	-	semi-annual	metering
total phosphorus (μg/l)	-	15	-	-	-	semi-annual	grab
total nitrogen (μg/l)	-	130	-	-	-	semi-annual	grab
chlorophyll-a (μg/l)	-	0.25	-	-	-	semi-annual	grab
light penetrate (ft-50%)	-	-	-	<130	-	semi-annual	secchi disk
dissolved oxygen (mg/l)	-	-	-	5.5	-	semi-annual	grab
pH (s.u.)	-	-	-	6.5-8.6	-	semi-annual	grab
ΔpH (s.u.)	-	-	-	≤ 0.2	-		
enterococci (cfu/100ml)	-	-	-	124	35	semi-annual	grab

Federal regulations require discharge outfalls for 301(h) permittees to ensure water quality standards are not exceeded at or beyond the zone of initial dilution (40 CFR 125.62(a)(1).

c. Federal Sludge Standards: The permit also establishes additional limits and monitoring requirements of the sediment, benthic communities, and sludge.

NPDES Permit No.	Federal Sewage Sludge Limits					Self-Monitoring	
AS0020010 - § D(1), D(10)	Table 1	Table 3				frequency	type
arsenic (mg/kg-dry)	75	41	-	-	-	annually	grab
cadmium (mg/kg-dry)	85	39	-	-	-	annually	grab
copper (mg/kg-dry)	4300	1500	-	-	-	annually	grab
lead (mg/kg-dry)	840	300	-	-	-	annually	grab
mercury (mg/kg-dry)	57	17	-	-	-	annually	grab
molybdenum (mg/kg-dry)	75	-	-	-	-	annually	grab
nickel (mg/kg-dry)	420	420	-	-	-	annually	grab
selenium (mg/kg-dry)	100	100	-	-	-	annually	grab
zinc (mg/kg-dry)	7500	2800	-	-	-	annually	grab

d. Sewage Spills: Section A(1)(a) of the NPDES permit requires all domestic sewage contributions into the Tafuna sewer system to be discharged only through the Tafuna ocean outfall.

10. On January 14, 2009, EPA issued a Tentative Decision Document denying the 301(h) variance from the secondary treatment requirements in the next NPDES permit. A Final Decision Document has not been issued as of yet.

11. The American Samoa Power Authority, as owner and operator of the Tafuna Sewage Treatment Plant, violated Section 301(a) of the Act [33 U.S.C. Section 1311(a)], in that:

a. ASPA submits receiving water quality monitoring reports for the Tafuna Sewage Treatment Plant ocean outfall at least twice per year, in the spring/summer non-trade wind season, and in the fall/winter trade wind season;

b. As documented in the June 10, 2011 EPA inspection report for the ASPA wastewater systems:
   (1) EPA compared the 2005-2010 water column sampling results for the two Tafuna ocean outfall Zone-of-Initial-Dilution ("Tafuna ZID") sampling stations designated in the NPDES permit with those from a reference sampling station also designated in the permit;
   (2) EPA determined that the water quality standards are not consistently met at the Tafuna ZID boundaries for total nitrogen, total phosphorus, chlorophyll-a, and enterococci;
   (3) EPA determined that there are statistically significant increases in the enterococci levels between the Tafuna ZID sampling stations and the reference station;
   (4) Therefore EPA also determined that the discharge from the Tafuna sewage treatment plant is a likely cause or contributing source of the elevated enterococci levels in the receiving waters above the water quality standards.

c. In 2006-2010, the discharge from the Tafuna Sewage Treatment Plant ocean outfall violated the prohibitions in Section A(3) of the NPDES permit on 28 occasions accounting for 61 days of violation, as listed in Table 1 on the next page of this Finding of Violation.

d. As documented in the June 10, 2011 EPA inspection report for the ASPA waste-water systems, the ASPA daily work history for 2010 recorded the occurrence of spills from sewer line back-ups and line breaks. These sewer spills violated the prohibitions in Sections A(1)(a) of the NPDES permits for the Tafuna and Utulei Sewage Treatment Plants on 15 occasions accounting for 15 days of violations.

12. The June 10, 2011 EPA report of the September 28-29, 2010 inspection of the ASPA Sewage Treatment Works on Tutuila Island is by reference made part of this Order.

Table 1 - Tafuna STP
NPDES Permit No. AS0020010 Receiving Water Column Violations

2006 - 2010 Tafuna Sewage Treatment Plant Self-Monitoring Results					Permit Violations	
Sampling Date	Location	Sample Type	The discharge shall not cause ...		Violation	Days
Feb/Oct 2010	ZID A1/A2-all	geo-μ comps	§A(3)(j) enterococci geo-μ	35 CFU/100 ml	187	12
Feb/May 2008	ZID A1/A2-all	geo-μ comps	§A(3)(j) enterococci geo-μ	35 CFU/100 ml	64	12
Feb/Sep 2007	ZID A1/A2-all	geo-μ comps	§A(3)(j) enterococci geo-μ	35 CFU/100 ml	76	12
Oct 2010	ZID A1-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	959	1
Oct 2010	ZID A1-mid	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	691	1
Oct 2010	ZID A1-bot	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	959	1
Oct 2010	ZID A2-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	907	1
Oct 2010	ZID A2-mid	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	393	1
Feb 2010	ZID A1-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	1782	1
Feb 2010	ZID A2-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	987	1
Sep 2009	ZID A1-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	504	1
Sep 2009	ZID A2-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	3654	1
Sep 2009	ZID A2-bot	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	185	1
Feb 2009	ZID A1-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	2481	1
Feb 2010	ZID A2-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	1565	1
May 2008	ZID A1-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	1989	1
May 2008	ZID A1-mid	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	1100	1
May 2008	ZID A2-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	2755	1
May 2008	ZID A2-mid	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	269	1
Feb 2008	ZID A1-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	2481	1
Feb 2008	ZID A2-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	1565	1
Sep 2007	ZID A1-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	2723	1
Sep 2007	ZID A1-mid	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	2359	1
Feb 2007	ZID A1-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	7701	1
Feb 2007	ZID A1-bot	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	161	1
Feb 2007	ZID A2-mid	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	223	1
Nov 2006	ZID A1-top	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	12031	1
Nov 2006	ZID A1-bot	single grab	§A(3)(j) enterococci grab	124 CFU/100 ml	187	1

ADMINISTRATIVE ORDER

Taking these Findings into consideration and considering the potential environmental and human health effects of the violations and all good faith efforts to comply, EPA has determined that compliance in accordance with the following requirements is reasonable. Pursuant to Section 308(a) and 309(a)(3), (a)(4) and (a)(5)(A) of the Act [33 U.S.C. Section 1318(a) and 1319(a)(3), (a)(4) and (a)(5)(A)], IT IS HEREBY ORDERED that the American Samoa Power Authority comply with the following requirements:

Submission of Information

1. By **SEPTEMBER 30, 2011**, the American Samoa Power Authority shall submit short responses to the findings in Sections 2.0, 3.0, 4.0, 4.1, 4.2, 4.3, 4.4, and 4.5 of the June 10, 2011 EPA inspection report.

Disinfection and De-chlorination

2. By **APRIL 30, 2012**, ASPA shall submit a preliminary engineering plan for the steps to be taken in order to provide disinfection and de-chlorination (if necessary) of the primary treated effluent discharges from the Tafuna Sewage Treatment Plant into the Pacific Ocean through the Tafuna ocean outfall. This preliminary engineering plan shall include:
   a. A detailed description of all plant, equipment, hardware, management plans, and operating procedures to be used to provide disinfection of the primary treated effluent discharges from the Tafuna Sewage Treatment Plant;
   b. A detailed description of all plant, equipment, hardware, management plans and operating procedures to be used to provide de-chlorination of chlorinated effluent discharges from the Tafuna Sewage Treatment Plant;
   c. A description of the training needs for prospective operators of the disinfection and de-chlorination steps;
   d. An estimate of the capital and training costs;
   e. A schedule of all steps, outlined in Items 2(a), 2(b), and 2(c) above, not to extend beyond **June 30, 2013**.

3. By **JUNE 30, 2013**, ASPA shall (1) complete the steps (required in Item 2 above) necessary to provide disinfection and de-chlorination of the primary treated effluent discharges from the Tafuna Sewage Treatment Plant into the Pacific Ocean through the Tafuna ocean outfall, and (2) submit a Notice of Completion.

4. By **JUNE 30, 2013**, ASPA shall achieve consistent compliance with the receiving water limits for enterococci in Section A(3) of the NPDES permit for the Tafuna Sewage Treatment Plant, and with an interim limit of 0.1 mg/l residual chlorine.

Critical Initial Dilution Factor

5. By **APRIL 30, 2012**, ASPA shall submit a scoping summary of projects that could be taken in order to optimally increase the critical initial dilution factor for the Tafuna ocean outfall discharge. For each project, this scoping summary shall include a description of the project, the resulting estimated mean and peak discharge flow rates (if any), the estimated capital cost of the project, and a construction schedule not to extend beyond **June 30, 2013**. At a minimum, this scoping summary shall cover the following projects:
   a. A reduction in the expected daily-maximum mean and peak discharge flow rates through infiltration and inflow upgrades to the sewer system;
   b. A reduction in the expected daily-maximum mean and peak discharge flow rates through increases in sewer system storage capacities and optimized delivery;
   c. A reduction in the expected daily-maximum mean and peak discharge flow rate through the installation and operation of on-site wet-weather storage;
   d. A doubling of the diffuser length of the Tafuna ocean outfall;
   e. Any other project to increase the size of the zone of initial dilution.

Additional Self-Monitoring

6. **FROM OCTOBER 1, 2011** through the reissuance of the NPDES permit, ASPA shall report all sewage spills from the Tafuna sewer system. The sewage spill reports shall be submitted as an attachment to the Discharge Monitoring Reports required by the NPDES permit, and include the date, volume, duration, cause, and destination of the spills.

7. **ONCE EACH MONTH** from **December 1, 2011** through the reissuance of the NPDES permit, ASPA shall self-monitor the Tafuna Sewage Treatment Plant, for total nitrogen (influent and effluent), total phosphorus (influent and effluent), ammonia (influent and effluent), and turbidity (effluent only).

8. **ONCE EACH MONTH** from the start-up date of the disinfection required in Items 2 and 3 of this Order, through the reissuance of the NPDES permit, ASPA shall self-monitor the Tafuna Sewage Treatment Plant, for enterococci (effluent only), and residual chlorine (effluent only).

9. ASPA shall self-monitor and analyze using the sampling protocols listed below, and the EPA approved analytical methods in 40 CFR 136 (or equivalent) necessary to achieve the detection limits indicated below:

Pollutants	Sampling Location	Method Protocols	Detection Limits
total nitrogen	Tafuna STP influent and effluent	8-hour composite	0.1 mg/l
total phosphorus	Tafuna STP influent and effluent	8-hour composite	0.1 mg/l
ammonia	Tafuna STP influent and effluent	8-hour composite	0.1 mg/l as N
turbidity	Tafuna STP effluent	grab	1 NTU
enterococci	Tafuna STP effluent	grab	10 CFU/100 ml
residual chlorine	Tafuna STP effluent	grab	0.01 mg/l

10. For each sample and measurement, ASPA shall record the following:
    a. the sample or measurement results,
    b. the EPA analytical methods used,
    c. the date, and time of sampling, and sampling point,
    d. the type of sample (ie. 24-hour composite, grab, or manual composite), and
    e. the name of the laboratory used.

11. The monthly self-monitoring required in Items 7 and 8 of this Order is in addition to the self-monitoring requirements of Sections A(1) and E(1) of NPDES Permit No. AS0020010.

Quarterly Status Reports

12. **WITHIN TWO MONTHS** after the end of a quarter through June 30, 2013, ASPA shall submit a written quarterly status report that documents the status of the work required under the following items:

    a. Items 2 and 3 - The installation of disinfection and de-chlorination;
    b. Item 5 - The scoping of potential critical initial dilution projects.

    The quarterly status reports shall also contain any Notices to Proceed and Notices of Completion issued during the quarter.

13. **WITHIN TWO MONTHS** after the end of a quarter through the reissuance of the NPDES permit, ASPA shall submit all self-monitoring results for the previous quarter as required in Items 7 and 8 of this Order.

14. The first quarter status reports for October to December will be due on **February 28th.** Second quarter reports for January to March will be due on **May 30th.** Third quarter reports for April to June will be due on **August 30th.** Fourth quarter reports for July to September will be due on **November 30th.**

15. All reports submitted pursuant to this Order shall be signed by a principal executive officer of the American Samoa Power Authority or by a representative of the American Samoa Office of the Governor, and shall include the following self-certifying statement:

    I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, I certify that the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I certify that all wastewater samples analyzed and reported herein are representative of the ordinary process wastewater flow from this facility. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

16. This Order is not and shall not be interpreted to be an NPDES permit under Section 402 of the Act [33 U.S.C. Section 1342], nor shall it in any way relieve the American Samoa Power Authority of obligations imposed by the Act, or any other Federal, State or local law.

17. All submittals shall be mailed to the following addresses:

U.S. ENVIRONMENTAL PROTECTION AGENCY
75 Hawthorne Street
San Francisco, California 94105
Attn: Greg V. Arthur (WTR-7)

AMERICAN SAMOA ENVIRONMENTAL PROTECTION AGENCY
P.O. Box PPA
Pago Pago, American Samoa 96799
Attn: LCDR Matt Vojik

18. This Order takes effect upon the date of receipt.

Original signed by:
Alexis Strauss

Alexis Strauss
Director, Water Division

July 27, 2011
Dated

UNITED STATES
ENVIRONMENTAL PROTECTION AGENCY
REGION 9

In the Matter of American Samoa Power Authority Utulei Sewage Treatment Plant, Ocean Outfall, and Sewage Collection System NPDES Permit No. AS0020001 Proceedings under Section 308(a) and 309(a)(3), (a)(4) and (a)(5)(A) of the Clean Water Act, as amended, 33 U.S.C. Section 1318(a) and 1319(a)(3), (a)(4) and (a)(5)(A)	)))))))))))	FINDING OF VIOLATION AND ORDER Docket No. CWA-309(a)-11-017

STATUTORY AUTHORITY

The following Finding of Violation and Administrative Order (Docket No. CWA-309(a)-11-017) is issued under the authority vested in the Administrator of the U.S. Environmental Protection Agency (EPA) pursuant to Sections 308(a) and 309(a)(3), (a)(4) and (a)(5)(A) of the Clean Water Act [33 U.S.C. Sections 1318(a) and 1319(a)(3), (a)(4) and (a)(5)(A)] (hereinafter the Act). This authority has been delegated by the Administrator and the Regional Administrator of EPA Region 9 to the Director of the Water Division of EPA Region 9.

FINDING OF VIOLATION

The Director of the Water Division of EPA Region 9 finds that the American Samoa Power Authority, as the owner and operator of the Utulei Sewage Treatment Plant, violated Section 301(a) of the Act [33 U.S.C. Section 1317(d)]. This Finding is made on the basis of the following facts:

1. The American Samoa Power Authority ("ASPA") owns the Utulei Sewage Treatment Plant, the Utulei ocean outfall, and the Pago Pago harbor area sewer collection system ("Utulei sewer system").

2. Section 301(a) of the Act [33 U.S.C. Section 1311(a)] prohibits the discharge of any pollutant by any person from a point source into waters of the United States except in compliance with a National Pollutant Discharge Elimination System (NPDES) permit issued in accordance with Section 402(a) of the Act [33 U.S.C. Section 1342]:

    a. Section 502(5) of the Act [33 U.S.C. Section 1362(5)] defines "person" to mean an individual, corporation, partnership, association, State, municipality, commission, or political subdivision of a State, or any interstate body;

b. Section 502(6) of the Act [33 U.S.C. Section 1362(6)] defines "pollutant" to mean sewage, garbage, sewage sludge, rock, sand, chemical wastes, biological materials, dredged spoil, solid waste, incinerator residue, munitions, radioactive materials, heat, wrecked or discarded equipment, cellar dirt, and industrial, municipal, and agricultural waste discharged into water;

c. Section 502(12) defines the term "discharge of pollutants" to mean any addition of any pollutant to navigable waters from any point source;

d. Section 502(7) defines the term "navigable waters" to mean the waters of the United States, including the territorial seas;

e. Section 502(14) defines "point source" to mean any discernible, confined and discrete conveyance, including but not limited to any pipe, ditch, channel, tunnel, conduit, well, discrete fissure, container, rolling stock, concentrated animal feeding operation, or vessel, or other floating craft, from which pollutants are or may be discharged.

3. The American Samoa Government is a State, and is therefore a person within the meaning of Section 502(5) of the Act [33 U.S.C. Section 1362(5)], and thus subject to the provisions of the Act, [33 U.S.C. Section 1251 et seq].

4. The American Samoa Power Authority is a political subdivision of the State, and is therefore a person within the meaning of Section 502(5) of the Act [33 U.S.C. Section 1362(5)], and thus subject to the provisions of the Act, [33 U.S.C. Section 1251 et seq].

5. The Pacific Ocean is a water of the United States.

6. On September 28 and 29, 2010, EPA conducted a diagnostic evaluation inspection of the Utulei Sewage Treatment Plant, and Utulei sewer system, and determined the following:

a. Facility Description: The Utulei Sewage Treatment Plant provides primary treatment and undisinfected discharge through an ocean outfall to Pago Pago Harbor:

(1) The Utulei Sewage Treatment Plant consist of a manual bar screen dropped into a deep-well influent pump station, a flow splitter box, four clarigesters, a decommissioned chlorine contact outlet structure, and a deep-water ocean outfall into outer Pago Pago Harbor;

(2) The Utulei sewer system has one main lift station and seven satellite lift stations that together feed collected sewage into the Utulei Sewage Treatment Plant influent pump station;

(3) The average daily and maximum peak flows in 2010 of 1.21 and 4.4 mgd are not approaching the as-built design dry-weather and daily peak design critieria of 2.21 mgd and 6.0 mgd, respectively, for the Utulei Sewage Treatment Plant;

(4) The 1999 EPA final decision extending the 301(h) waivers and the 2009 EPA tentative decision denying the waivers cited the critical initial dilution factors to be 202:1 in 1999 and 90:1 in 2009 for the zone-of-initial-dilution established for the Utulei ocean outfall.

b. Facility Operations: The Utulei Sewage Treatment Plant accepts and handles only domestic sewage collected into the Utulei sewer system. Digested sludge is trucked to the Tafuna Sewage Treatment Plant sludge drying beds.

7. EPA issued the NPDES permit No.AS0020001 for the Utulei Sewage Treatment Plant to become effective on October 9, 2001 and to expire on October 9, 2006. The 2001 NPDES permit authorized the discharge of treated domestic sewage from the Utulei Sewage Treatment Plant through the Utulei ocean outfall into outer Pago Pago Harbor.

8. The Federal regulations in 40 CFR 122.21(d) allow the administrative extension of an NPDES permit if a permit application is submitted for renewal at least 180 days before it expires. ASPA submitted an application for permit renewal on April 11, 2006, before the 180 day deadline. Therefore the NPDES permit is administratively extended to be in effect beyond the October 9, 2006 permit expiration date.

9. The 2001 NPDES permit advances less-than-secondary technology-based limits (based on a 1999 EPA 301(h) waiver variance final decision), receiving water limitations, sludge limits, and self-monitoring requirements. The 2006 renewal application included a request to extend the section 301(h) variance waiver.

a. Effluent Limits: The less-than-secondary limits for BOD and TSS removal rates are based on a Federal minimum of 30% for primary treatment. The BOD and TSS concentration and loading limits reflect past performance data.

NPDES Permit No.	Utulei STP Effluent Limits					Self-Monitoring	
AS0020001 - § A(1)	mo-av	7d-av	d-max	instant	geo-μ	frequency	type
flow (mgd)	-	-	-	-	-	continuous	flume
BOD (mg/l)	78.3	117	157	-	-	once/week inf and eff	8hr comp
BOD (lbs/day)	1085	1628	2170	-	-		
BOD (%removal)	>30%	-	-	-	-		
TSS (mg/l)	75	113	150	-	-	once/week inf and eff	8hr comp
TSS (lbs/day)	1377	2065	2754	-	-		
TSS (%removal)	>30%	-	-	-	-		
settleable solids (ml/l)	1.0	-	2.0	-	-	once/day	grab
pH (s.u.)	-	-	-	6.5-8.6	-	once/week	grab
oil and grease (mg/l)	-	-	-	-	-	quarterly	grab
toxicity (TUc)	-	-	-	-	-	quarterly	24hr comp

b. Receiving Water Limits: The permit establishes limits to apply at and beyond the Zone of Initial Dilution ("ZID") based on the American Samoa water quality standards for Pago Pago Harbor. The receiving water permit limits are for discharges that cause water column samples taken at three depths (top, mid, bottom) from defined ZID water column sampling stations to exceed the standards. For turbidity, nutrients, chlorophyll-a, and enterococci, the receiving water permit limits are for discharges that cause averages (over the water column, ZID sampling locations, and a 12-month period) to exceed the standards.

NPDES Permit No.	Utulei Outfall Water Column Limits					Self-Monitoring	
AS0020001 - § A(3), E(1)	mo-av	12m-av	d-max	Instant	geo-μ	frequency	type
turbidity (NTU)	-	0.75	-	-	-	semi-annual	metering
total phosphorus (μg/l)	-	30	-	-	-	semi-annual	grab
total nitrogen (μg/l)	-	200	-	-	-	semi-annual	grab
chlorophyll-a (μg/l)	-	1.0	-	-	-	semi-annual	grab
light penetrate (ft-50%)	-	-	-	<65	-	semi-annual	secchi disk
dissolved oxygen (mg/l)	-	-	-	5.0	-	semi-annual	grab
pH (s.u.)	-	-	-	6.5-8.6	-	semi-annual	grab
ΔpH (s.u.)	-	-	-	≤ 0.2	-		
enterococci (cfu/100ml)	-	-	-	104	35	semi-annual	grab

Federal regulations require discharge outfalls for 301(h) permittees to ensure water quality standards are not exceeded at or beyond the zone of initial dilution (40 CFR 125.62(a)(1).

c. <u>Sewage Spills</u>: Section A(1)(a) of the NPDES permit requires all domestic sewage contributions into the Utulei sewage collection systems to be discharged only through the Utulei ocean outfall.

10. On January 14, 2009, EPA issued a Tentative Decision Document denying the 301(h) variance from the secondary treatment requirements in the next NPDES permit. A Final Decision Document has not been issued as of yet.

11. The American Samoa Power Authority, as owner and operator of the Utulei Sewage Treatment Plant, violated Section 301(a) of the Act [33 U.S.C. Section 1311(a)], in that:

a. ASPA submits receiving water quality monitoring reports for the Utulei Sewage Treatment Plant ocean outfall at least twice per year, in the spring/summer non-trade wind season, and in the fall/winter trade wind season;

b. As documented in the June 10, 2011 EPA inspection report for the ASPA wastewater systems:

(1) EPA compared the 2005-2010 water column sampling results for the two Utulei ocean outfall Zone-of-Initial-Dilution ("Utulei ZID") sampling stations designated in the NPDES permit with those from a reference sampling station also designated in the permit;

(2) EPA determined that the water quality standards are not consistently met at the Utulei ZID boundaries for total nitrogen, and enterococci;

(3) EPA determined that there are statistically significant increases in the enterococci levels between the Utulei ZID sampling stations and the reference station;

(4) Therefore EPA also determined that the discharge from the Utulei Sewage Treatment Plant is a likely cause or contributing source of the elevated enterococci levels in the receiving waters above the water quality standards.

c. In 2006-2010, the discharge from the Utulei Sewage Treatment Plant ocean outfall violated the prohibitions in Section A(3) of the NPDES permit on 15 occasions accounting for 26 days of violation, as listed in Table 1 below.

d. As documented in the June 10, 2011 EPA inspection report for the ASPA wastewater systems, the ASPA daily work history for 2010 recorded the occurrence of spills from sewer line back-ups and line breaks. These sewer spills violated the prohibitions in Sections A(1)(a) of the NPDES permits for the Tafuna and Utulei Sewage Treatment Plants on 15 occasions accounting for 15 days of violations.

12. The June 10, 2011 EPA report of the September 28-29, 2010 inspection of the ASPA Sewage Treatment Works on Tutuila Island is by reference made part of this Order.

Table 1 - Utulei STP
NPDES Permit No. AS0020001 Receiving Water Column Violations

2006 - 2010 Utulei Sewage Treatment Plant Self-Monitoring Results					Permit Violations	
Sampling Date	Location	Sample Type	The discharge shall not cause ...		Violation	Days
Feb/Oct 2010	ZID A1/A2-all	geo-μ comps	§A(3)(j) enterococci geo-μ	35 CFU/100 ml	47	12
Oct 2010	ZID B1-mid	single grab	§A(3)(j) enterococci grab	104 CFU/100 ml	573	1
Feb 2010	ZID A1-top	single grab	§A(3)(j) enterococci grab	104 CFU/100 ml	2098	1
Feb 2010	ZID B1-top	single grab	§A(3)(j) enterococci grab	104 CFU/100 ml	1607	1
Feb 2010	ZID B1-mid	single grab	§A(3)(j) enterococci grab	104 CFU/100 ml	1106	1
Feb 2009	ZID A1-mid	single grab	§A(3)(j) enterococci grab	104 CFU/100 ml	213	1
Sep 2008	ZID B1-top	single grab	§A(3)(j) enterococci grab	104 CFU/100 ml	529	1
May 2008	ZID A1-mid	single grab	§A(3)(j) enterococci grab	104 CFU/100 ml	1043	1
May 2008	ZID B1-mid	single grab	§A(3)(j) enterococci grab	104 CFU/100 ml	246	1
Sep 2007	ZID A1-top	single grab	§A(3)(j) enterococci grab	104 CFU/100 ml	121	1
Sep 2007	ZID B1-mid	single grab	§A(3)(j) enterococci grab	104 CFU/100 ml	341	1
Feb 2007	ZID A1-mid	single grab	§A(3)(j) enterococci grab	104 CFU/100 ml	591	1
Nov 2006	ZID A1-top	single grab	§A(3)(j) enterococci grab	104 CFU/100 ml	171	1
Nov 2006	ZID B1-mid	single grab	§A(3)(j) enterococci grab	104 CFU/100 ml	223	1
Feb 2006	ZID B1-mid	single grab	§A(3)(j) enterococci grab	104 CFU/100 ml	197	1

ADMINISTRATIVE ORDER

Taking these Findings into consideration and considering the potential environmental and human health effects of the violations and all good faith efforts to comply, EPA has determined that compliance in accordance with the following requirements is reasonable. Pursuant to Section 308(a) and 309(a)(3), (a)(4) and (a)(5)(A) of the Act [33 U.S.C. Section 1318(a) and 1319(a)(3), (a)(4) and (a)(5)(A)], IT IS HEREBY ORDERED that the American Samoa Power Authority comply with the following requirements:

Submission of Information

2. By **SEPTEMBER 30, 2011**, the American Samoa Power Authority shall submit short responses to the findings in Sections 2.0, 3.0, 4.0, 4.1, 4.2, 4.3, 4.4, and 4.5 of the June 10, 2011 EPA inspection report.

Disinfection and De-chlorination

2. By **APRIL 30, 2012**, ASPA shall submit a preliminary engineering plan for the steps to be taken in order to provide disinfection and de-chlorination (if necessary) of the primary treated effluent discharges from the Utulei Sewage Treatment Plant into Pago Pago Harbor through the Utulei ocean outfall. This preliminary engineering plan shall include:
   a. A detailed description of all plant, equipment, hardware, management plans, and operating procedures to be used to provide disinfection of the primary treated effluent discharges from the Utulei Sewage Treatment Plant;
   b. A detailed description of all plant, equipment, hardware, management plans and operating procedures to be used to provide de-chlorination of chlorinated effluent discharges from the Utulei Sewage Treatment Plant;
   c. A description of the training needs for prospective operators of the disinfection and de-chlorination steps;
   d. An estimate of the capital and training costs;
   e. A schedule of all steps, outlined in Items 2(a), 2(b), and 2(c) above, not to extend beyond **June 30, 2013**.

3. By **JUNE 30, 2013**, ASPA shall (1) complete the steps (required in Item 2 above) necessary to provide disinfection and de-chlorination of the primary treated effluent discharges from the Utulei Sewage Treatment Plant into Pago Pago Harbor through the Utulei ocean outfall, and (2) submit a Notice of Completion.

4. By **JUNE 30, 2013**, ASPA shall achieve consistent compliance with the receiving water limits for enterococci in Section A(3) of the NPDES permit for the Utulei Sewage Treatment Plant, and with an interim limit of 0.1 mg/l residual chlorine.

Critical Initial Dilution Factor

5. By **APRIL 30, 2012**, ASPA shall submit a scoping summary of projects that could be taken in order to optimally increase the critical initial dilution factor for the Utulei ocean outfall discharge. For each project, this scoping summary shall include a description of the project, the resulting estimated mean and peak discharge flow rates (if any), the estimated capital cost of the project, and a construction schedule not to extend beyond **June 30, 2013.** At a minimum, this scoping summary shall cover the following projects:
   a. A reduction in the expected daily-maximum mean and peak discharge flow rates through infiltration and inflow upgrades to the sewer system;
   b. A reduction in the expected daily-maximum mean and peak discharge flow rates through increases in sewer system storage capacities and optimized delivery;
   c. A reduction in the expected daily-maximum mean and peak discharge flow rates through the installation and operation of on-site wet-weather storage;
   d. A doubling of the diffuser length of the Utulei ocean outfall;
   e. Any other project to increase the size of the zone of initial dilution.

Additional Self-Monitoring

6. **FROM OCTOBER 1, 2011** through the reissuance of the NPDES permit, ASPA shall report all sewage spills from the Utulei sewer system. The sewage spill reports shall be submitted as an attachment to the Discharge Monitoring Reports required by the NPDES permit, and include the date, volume, duration, cause, and destination of the spills.

7. **ONCE EACH MONTH** from **December 1, 2011** through the reissuance of the NPDES permit, ASPA shall self-monitor the Utulei Sewage Treatment Plant, for total nitrogen (influent and effluent), total phosphorus (influent and effluent), ammonia (influent and effluent), and turbidity (effluent only).

8. **ONCE EACH MONTH** from the start-up date of the disinfection required in Items 2 and 3 of this Order, through the reissuance of the NPDES permit, ASPA shall self-monitor the Utulei Sewage Treatment Plant, for enterococci (effluent only), and residual chlorine (effluent only).

9. ASPA shall self-monitor and analyze using the sampling protocols listed below, and the EPA approved analytical methods in 40 CFR 136 (or equivalent) necessary to achieve the detection limits indicated below:

Pollutants	Sampling Location	Method Protocols	Detection Limits
total nitrogen	Utulei STP influent and effluent	8-hour composite	0.1 mg/l
total phosphorus	Utulei STP influent and effluent	8-hour composite	0.1 mg/l
ammonia	Utulei STP influent and effluent	8-hour composite	0.1 mg/l as N
turbidity	Utulei STP effluent	Grab	1 NTU
enterococci	Utulei STP effluent	Grab	10 CFU/100 ml
residual chlorine	Utulei STP effluent	Grab	0.01 mg/l

10. For each sample and measurement, ASPA shall record the following:
    a. the sample or measurement results,
    b. the EPA analytical methods used,
    c. the date, and time of sampling, and sampling point,
    d. the type of sample (ie. 24-hour composite, grab, or manual composite), and
    e. the name of the laboratory used.

11. The monthly self-monitoring required in Items 7 and 8 of this Order is in addition to the self-monitoring requirements of Sections A(1) and E(1) of NPDES Permit No. AS0020001.

Quarterly Status Reports

12. **WITHIN TWO MONTHS** after the end of a quarter through June 30, 2013, the ASPA shall submit a written quarterly status report that documents the status of the work required under the following items:

    a. Items 2 and 3 - The installation of disinfection and dechlorination;
    b. Item 4 - The scoping of potential critical initial dilution projects.

    The quarterly status reports shall also contain any Notices to Proceed and Notices of Completion issued during the quarter.

13. **WITHIN TWO MONTHS** after the end of a quarter through the reissuance of the NPDES permit, ASPA shall submit all self-monitoring results for the previous quarter as required in Items 7 and 8 of this Order.

14. The first quarter status reports for October to December will be due on **February 28th**. Second quarter reports for January to March will be due on **May 30th**. Third quarter reports for April to June will be due on **August 30th**. Fourth quarter reports for July to September will be due on **November 30th**.

15. All reports submitted pursuant to this Order shall be signed by a principal executive officer of the American Samoa Power Authority or by a representative of the American Samoa Office of the Governor, and shall include the following self-certifying statement:

    I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, I certify that the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I certify that all wastewater samples analyzed and reported herein are representative of the ordinary process wastewater flow from this facility. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

16. This Order is not and shall not be interpreted to be an NPDES permit under Section 402 of the Act [33 U.S.C. Section 1342], nor shall it in any way relieve the American Samoa Power Authority of obligations imposed by the Act, or any other Federal, State or local law.

17. All submittals shall be mailed to the following addresses:

U.S. ENVIRONMENTAL PROTECTION AGENCY
75 Hawthorne Street
San Francisco, California 94105
Attn: Greg V. Arthur (WTR-7)

AMERICAN SAMOA ENVIRONMENTAL PROTECTION AGENCY
P.O. Box PPA
Pago Pago, American Samoa 96799
Attn: LCDR Matt Vojik

18. This Order takes effect upon the date of receipt.

Original signed by:
Alexis Strauss

Alexis Strauss
Director, Water Division

July 27, 2011

Dated

Appendix B NOAA Precipitation Daily Totals

Daily Total Rainfall

Year	Day	January	February	March	April	May	June	July	August	September	October	November	December
2008	1	0	0.01	0.02	0.45	0.02	0.11	0	0	0.22	0.01	0.05	
	2	0.27	0.1	0.3	0	0.19	0.18	0.61	0	1.14	0.14	0.15	
	3	0.57	0.3	2.89	0	0.04	0	0.03	0.05	0.73	0.36	0.12	
	4	0	0.08	1.8	0.85	0.11	0.06	0.09	0	0.04	0.21	0	
	5	0.1	0.11	0.21	0.21	0.14	1.22	0.23	0	0	0.88	0.28	
	6	3.6	0.02	0.2	0.01	0.06	0.11	0.07	0	0	0	0.49	
	7	0.27	0.11	0.05	0.02	0.01	1.47	0.01	0	0.01	0.06	0.22	
	8	2.44	0.15	0.02	0.24	0.96	0	0	0.96	0.02	0	2.43	
	9	0.23	0.06	0.06	0	0.2	0.26	0.51	0.4	0.01	0	0.02	
	10	0.02	0.16	0.12	0.12	0.02	0.01	0.25	0	0	0.23	0	
	11	0.77	0	0	0	0.02	0.47	0.01	0.04	0	0	1.01	
	12	1.19	0.52	0.02	0	3.78	0.18	0.01	0.24	0	0	0.41	
	13	0.09	0.05	0	0.62	0.05	0.22	0.17	0.03	0.05	0	0	
	14	*0.59*	*0*	*0.02*	*0.1*	*0*	*3.76*	*1.78*	*0*	*0*	*0.11*	*0.61*	
	15	0.04	0.15	0.16	1.18	0	0.11	0.5	0.19	0.14	0	0.2	
	16	1.01	0	0.09	1.49	0	0.01	0.07	0	1.45	0.01	0.03	
	17	0.25	0	0.37	5.19	0.09	0.17	0	0	0	0.03	0	
	18	0.02	0.12	0.02	0.05	2.39	0.96	0.02	0.12	0	0	0	
	19	2.59	0.54	0.32	0	3.39	0.05	0.01	0.72	0.02	0	0.92	
	20	2.31	0.04	0	0	0.21	0.03	0.09	0	0	0	0.49	
	21	1.34	0	0.12	0.38	1	1.31	0	0	0.01	0	0.25	
	22	0.23	0.03	0.17	0.05	0	0	0	0.04	0	0	0.62	
	23	0	0.56	0.11	0.36	0	0.2	0	0.13	0	0.01	0.04	
	24	0.85	0.13	1.9	0	0	0.83	0	0	0.15	0.38	0.88	
	25	1.3	0.2	0	0	0.56	0	0	0.05	0	0.09	0.13	
	26	0.14	0.24	0.01	0.08	0.65	0	0	0	0	0.01	0.21	
	27	0	0.49	0	1.16	0.15	0.28	0	0	0.46	0	0.83	
	28	0.12	0.13	0.17	0.29	0.77	0	0	0	0.07	0.01	1.27	
	29	0.3	0.6	0.37	0.03	0.3	0	0	0	1.61	0.49	0	
	30	1.33		0.52	0.01	0.17	0	0	0	0	0.2	0.21	
	31	0.18		0.04		4.55		0	0.62		0		
	Total	22.15	4.9	10.08	12.89	19.83	12	4.46	3.59	6.13	3.23	11.87	
	Year Total												111.13

Daily Total Rainfall

Year	Day	January	February	March	April	May	June	July	August	September	October	November	December
2009	1	0.05	0.27	0.1	0.56	0	0.18	0.46	0	0.42	0.01	0	0.62
	2	0.03	0.59	0.01	1.99	0	0.14	0.29	0.01	0.18	0	0.06	0.23
	3	1.09	0	0.23	0.5	0	0.21	2.43	0.01	1.12	0	0	0.1
	4	0.84	0.03	0	1.85	0.59	0.67	0.02	0	0.01	0.08	0.08	0.58
	5	*0.17*	*0.04*	*0.05*	*0.05*	*0*	*0.03*	*0*	*0.03*	*0.22*	*0*	*0*	*0.01*
	6	0.02	0.04	0.02	0.07	0.14	0.17	0	1.25	2.2	0	0.54	0.05
	7	0	0.22	0.02	0.1	0.68	0	0	0.34	0.08	0	3.43	0.01
	8	0.26	1.64	0.22	1.82	0.68	0	0.07	0.01	0.02	0.02	2.53	0.57
	9	0.04	0.05	0	0.06	1.1	0.01	0.17	0.03	0.1	0.04	1.6	0.68
	10	0.02	0	0.18	0.82	4.05	0.05	0.16	0.79	0.08	0	0.64	6.6
	11	0.5	0.11	0.79	0.16	0.48	0	0.18	0.04	0	0	2.04	2.26
	12	0.43	0.16	0.78	0.02	0.51	0	0.04	0.08	0	0	0	0
	13	0.58	0.5	0.44	0	0.48	0	0	0.01	0.08	2.35	0	0
	14	0.76	0.04	0.12	0.01	2.39	0	0	0.03	1.49	1.94	1.04	0
	15	0	0.03	0.07	0.38	0.64	0	0.18	1.75	0.01	0.36	0	0
	16	0.67	0	0.77	0.03	0.28	0	0.95	1.19	0.02	0.01	0	0
	17	0.28	0.06	0	0	4.21	0	0.07	0.03	0.11	0.05	0	0.13
	18	0.74	0	0	0.02	0.03	0.09	0	0.1	0	0.21	0.01	0
	19	1.08	0.91	0	0.15	0.42	0.03	0.01	0.58	0.01	0.02	0	0
	20	0.05	0.02	1.36	0.04	1.18	1.03	0.12	0.01	0.03	0.01	0	0
	21	0.09	1.49	0.52	0	0.49	0.12	0.14	0	0.07	0.24	0.06	0
	22	0.55	0.24	0.01	0.01	0.14	0.15	0.24	0	0	0	0.04	0
	23	0.32	0.41	0.01	0	0.39	0.08	0.02	0.77	0	0.05	0	0
	24	0.27	0.18	0.01	0	0	0.25	0.14	3.1	0	0.09	0	0.01
	25	0.16	0.46	0.02	0	0.04	0.11	0.09	0.09	0	0	0.35	0.15
	26	0.51	0	0.04	0.07	0.01	0.32	0.3	0	0	0	1.65	4.06
	27	0.08	0.02	0.06	0.15	0.08	0.01	2.92	0	0	0.04	1.32	5.36
	28	0.09	0.11	0.26	0.42	0	0	0.1	0	0.03	0	0.78	2.76
	29	0.91		0.17	0	0	0	0.12	0	0	0	0.29	1.07
	30	0.36		0.64	0	0.15	0	0.54	0.06	0	0.83	0.19	0.81
	31	0.39		3.09		0.01		0.2	0.1		1.18		1.34
	Total	11.34	7.62	9.99	9.28	19.17	3.65	9.96	10.41	6.28	7.53	16.65	27.4
	Year Total												139.28

Daily Total Rainfall

Year	Day	January	February	March	April	May	June	July	August	September	October	November	December
2010	1	0.43	0	0.22	0	0	0.29	0	0.08	0	0	0.07	1.25
	2	1.51	1.84	0	0.08	0.02	0.21	0.56	0	0	0.01	0.55	0.1
	3	0.53	0.08	0.03	0.25	0.22	0.21	0	0	0	3.38	0	0.53
	4	0.51	0.05	0.07	0.01	1.29	0.12	0.03	0	0	0	0.11	0.39
	5	*0.16*	*0*	*0.02*	*0.03*	*0*	*0.56*	*0.11*	*0.02*	*0.14*	*0*	*0*	*0.26*
	6	0.41	0.02	0.04	0.08	0.85	0.09	0	0	0.07	0	0.01	0.56
	7	0.2	0.09	0.12	0.44	1.65	0.14	0.19	0.12	0.04	3.1	0	0.22
	8	0.07	0	0.05	0.66	0.09	0.03	0.06	0.11	0.33	0.12	0	0.86
	9	0	0.98	0.17	0.01	0.37	0.42	0.01	0.01	0.63	0	0.9	0.14
	10	0.4	0.2	0.23	0.03	0.03	0.66	1.21	1.24	0.95	0.37	0.76	0.01
	11	0.06	0.01	0.26	0.01	0.64	0	0.56	0	0.04	0	0.15	0.46
	12	0.16	3.12	0.01	0.04	0	0.28	0.69	0.07	0.05	0	0.54	0.39
	13	0.2	2.8	0.01	3.3	0	0.08	0.21	0.12	1.45	0.01	0.14	0.12
	14	1.29	0.04	0	0.27	0	0.12	0.98	0.09	3.14	0.36	0	0.08
	15	0.39	2.86	0	0.01	0	0.19	0	0.41	0.32	0.01	0.01	0.5
	16	0.18	0.02	0	0.28	0	0.62	0.15	0.03	0.51	0.08	0	0.05
	17	0	0.18	0	1.15	0.15	0.18	0.02	0.17	0.14	0.56	1.28	0.35
	18	0.1	0.06	0.04	0.31	0.01	0.03	0.13	0	0.4	0.26	0.19	3
	19	0.06	0.07	0	0	0.02	0.08	0.43	0	0.46	0.1	1.97	0.54
	20	0.04	0.09	0	0	0	0.19	0	0.8	0.03	0.33	2.04	0.01
	21	0.38	0.76	0.04	0.41	0.1	0.02	0.1	0	0.01	0.2	0.06	0
	22	0.37	2.13	0	0.1	0.21	0.05	0.16	0	0.11	0.04	0.24	0
	23	0.32	0.22	0.02	0.48	0.18	0.26	0	0	0.01	0.07	0.04	0.07
	24	6.91	0.15	0.02	0	0.09	0.03	0.07	0.45	0.21	0.03	0	0
	25	2.19	0.02	0	0.13	0.05	0.01	0.01	0.01	0.1	0.11	0	0.02
	26	1.28	0.04	0.03	0.51	0	0.05	0.1	0	0.06	0.29	0	0.25
	27	5.5	0	0.69	0	0.13	0	0.24	0.14	0	1.67	0.02	1.1
	28	0.07	0	2.63	0.39	0.15	0.13	0.12	0.45	0.39	0.75	0.01	0.58
	29	0		0	1.35	0.13	0	0.01	0	0.03	0.1	0.06	0.01
	30	0.13		0.29	0	0	0	0.33	0	0.06	0.04	0	0.48
	31	4.46		0		0.23		1.82	0		3.34		0.01
	Total	28.31	15.83	4.99	10.33	6.61	5.05	8.3	4.32	9.68	15.33	9.15	12.34
	Year Total												130.24

Daily Total Rainfall

Year	Day	January	February	March	April	May	June	July	August	September	October	November	December
2011	1	0.72	2.15	0.03	1.32	0.2	0.01	0	0	0		0.11	
	2	1.12	0.17	0.32	0.4	0.09	0.41	0	0.97	0		0.07	
	3	0.56	0.01	0.58	0.01	0	2.44	0	0	0		0.17	
	4	0.62	0.43	0	0.02	0	1.52	0	0	0		0.03	
	5	1.21	0.43	0	0	0	0.14	0.05	0	0		0.37	
	6	0.28	0.68	0.45	0	0	0.02	0.12	0.14	0.03		0	
	7	0.02	0.45	0	0	0.02	0.01	0	0.04	0	0.05	0.03	
	8	0.08	0.19	0	0.02	0.83	0	0	0	0	0.06	0.32	
	9	0.58	0.22	0.49	0.02	0.14	0.23	0	0.26	0		2.44	
	10	2.25	0.12	0.61	0.06	0.01	0.28	0	0	0.2	0.01	0.07	
	11	0	0.78	0.01	0	0.07	0.09	0.01	0	0	0.02	0.87	
	12	0.68	0.59	0	0	0	0.03	0	0	0	0.01	0.28	
	13	0.51	3.89	0.06	0.02	0.54	0.02	0.18	0	0		0	
	14	0.14	0.37	0.1	0.01	0.3	0	0.17	0.06	0.79	0.16	0	
	15	0.44	1.14	0.29	0	0	0	0	0.03	0.16	0.99	0	
	16	0.15	0.05	0.05	0	0.02	0	0	0.01		0.01	0	
	17	0.63	0.6	0.11	0	0.59	0.29	0.16	0		0.05	0	
	18	0.81	0.85	0.01	0.07	0.04	0	0.05	0	0	0.01	0	
	19	0.28	0	0	0.08	0	0	0	0	0.08	0.12	0.62	
	20	0.89	0.01	0.08	0.3	0	0.9	0.07	0		0.09	0.91	
	21	1.92	0.04	0.18	0.05	0	0.32	0	0.07		0.05	2.47	
	22	2.17	0.06	0.26	0.28	0	0.09	0	0.52		0.54	0.03	
	23	1.54	0.11	0.13	0.39	0	0.01	0	0.22	0	0.16	0	
	24	5.85	0	0	0	0.09	0.02	0	0.08		0.08	0.07	
	25	0.77	0	0.02	0.02	0.01	0.09	0	0.09		0	0.75	
	26	0.48	0	0.08	0.1	0.04	0	0.11	0.23		0	0.71	
	27	0.32	0.05	0.04	0.05	0.03	0	0.41	0.23	0.08	0	0.5	
	28	0.09	0	0	0.76	0.17	0	0.77	0.09	0	0	0	
	29	0		0.13	0.03	0.46	0.06	0.19	0.18	0	0	0	
	30	0.63		0.31	0	0.04	0.03	0.02	0.42	0.12	0	0	
	31	0.02		1.43		0		0	0.21		0		
	Total	25.76	13.39	5.77	4.01	3.69	7.01	2.31	3.85	1.46	2.41	10.82	
	Year Total												80.48

Appendix C Villages Census Population

Census-Based Estimated Flow Increase to Tafuna Waste Water Treatment Plant from Expansion Area

	Village	2000 Census house count	2000 Census Population	2010 Census house count	2010 Census Population
	Auma	-	-	52	254
	Leone	600	3,568	418	1,919
	Puapua	-	-	190	965
Leone	Futiga	105	731	116	723
	Malaeloa/Ituau/ Aitulagi	180	1,224	235	1,248
	Taputimu	100	640	159	841
	Vailoatai	159	989	263	1,447
	Mapusagaou	285	1,642	221	1,126
Tualauta	Vaitogi	243	1,347	382	1,959
	Total:	1,672	10,141	2,036	10,482
	Persons (6 per house):	10,032	10,141	12,216	10,482
	100 gal/person/day:	1,003,200	1,014,100	**1,221,600**	1,048,200
	Peak flow (factor of 4.0):	4,012,800	4,056,400	**4,886,400**	4,192,800

Census-Based Estimated Flow to Tafuna Waste Water Treatment Plant from Existing Area

	Village	2000 Census house count	2000 Census Population	2010 Census house count	2010 Census Population
	Faleniu	315	2,056	347	1,898
	Iliili	470	2,513	642	3,195
	Malaeimi	189	1,067	228	1,182
Tualauta	Mesepa	80	481	97	444
	Nu'uuli (Part)	404	2,310	115	659
	Pava'ia'i	401	2,200	432	2,450
	Tafuna	1,488	8,409	1,616	7,945
	Nu'uuli (Part)	501	2,844	660	3,296
Ituau					
	Total:	3,848	21,880	4,137	21,069
	Persons (6 per house):	23,088	21,880	24,822	21,069
	100 gal/person/day:	2,308,800	2,188,000	**2,482,200**	2,106,900
	Peak flow (factor of 4.0)	9,235,200	8,752,000	**9,928,800**	8,427,600

Census-Based Estimated Flow Increase to Tafuna Waste Water Treatment Plant

	Village	2000 Census house count	2000 Census Population	2010 Census house count	2010 Census Population
	Auma	-	-	52	254
	Leone	600	3,568	418	1,919
	Puapua	-	-	190	965
Leone	Futiga	105	731	116	723
	Malaeloa/Ituau/ Aitulagi	180	1,224	235	1,248
	Taputimu	100	640	159	841
	Vailoatai	159	989	263	1,447
	Mapusagaou	285	1,642	221	1,126
Tualauta	Vaitogi	243	1,347	382	1,959
	Total:	1,672	10,141	2,036	10,482
	Persons (6 per house):	10,032	10,141	12,216	10,482
	70 gal/person/day:	702,240	709,870	**855,120**	733,740
	Peak flow (factor of 3.5):	2,457,840	2,484,545	**2,992,920**	2,568,090

Census-Based Estimated Flow to Tafuna Waste Water Treatment Plant

	Village	2000 Census house count	2000 Census Population	2010 Census house count	2010 Census Population
	Faleniu	315	2,056	347	1,898
	Iliili	470	2,513	642	3,195
	Malaeimi	189	1,067	228	1,182
Tualauta	Mesepa	80	481	97	444
	Nu'uuli (Part)	404	2,310	115	659
	Pava'ia'i	401	2,200	432	2,450
	Tafuna	1,488	8,409	1,616	7,945
	Nu'uuli (Part)	501	2,844	660	3,296
Ituau					
	Total:	3,848	21,880	4,137	21,069
	Persons (6 per house):	23,088	21,880	24,822	21,069
	70 gal/person/day:	1,616,160	1,531,600	**1,737,540**	1,474,830
	Peak flow (factor of 3.5):	5,656,560	5,360,600	**6,081,390**	5,161,905

SUMMARY:

FLOW ESTIMATING SCENARIOS	2012 TAFUNA WW SYSTEM CURRENT SERVICE AREA FLOW (GPD)	2012 TAFUNA WW SYSTEM EXPANSION AREA FLOW (GPD)	2012 TAFUNA WW SYSTEM TOTAL ESTIMATED FLOW (CURRENT AND EXPANSION AREAS)	**10%** ESTIMATED FUTURE GROWTH TO TAFUNA WW SYSTEM (25 YEARS)
AVG FLOW (100GPCD)	2,482,200	1,221,600	3,703,800	4,074,180
PEAK FLOW (4.0 FACTOR)	9,928,800	4,886,400	14,815,200	16,296,720
AVG FLOW (70GPCD)	**1,737,540**	**855,120**	**2,592,660**	**2,851,926**
PEAK FLOW (3.5 FACTOR)	**6,081,390**	**2,992,920**	**9,074,310**	***9,981,741***

BECAUSE USING THE 70GPCD/P.F.=3.5 CLOSELY MATCHES EXISTING TAFUNA WWTP FLOW CHARTS, WE WILL USE THEM TO DETERMINE EXPANSION AREA FLOWS TO TAFUNA WWTP AND ASSUME 10% FUTURE ADDITIONAL FLOWS.

SAY 10MGD **PEAK DESIGN FLOW FOR TAFUNA WWTP!**

Population by District, County and Village: 2000 and 2010

	2000	2010	% Change
American Samoa......................................	57,291	55,519	-3.1
Eastern District...	23,441	23,030	-1.8
Ituau County	4,312	4,676	8.4
Faganeanea	183	150	-18.0
Fagasa	900	831	-7.7
Matu'u	385	399	3.6
Nu'uuli (Part)	2,844	3,296	15.9
Maoputasi County	11,695	10,299	-11.9
Anua	265	18	-93.2
Atu'u	413	359	-13.1
Aua	2,193	2,077	-5.3
Fagaalu	1,006	910	-9.5
Fagatogo	2,096	1,737	-17.1
Fatumafuti	103	113	9.7
Leloaloa	534	448	-16.1
Pago Pago	4,278	3,656	-14.5
Satala	..	297	
Utulei	807	684	-15.2
Sa'ole County	1,768	2,187	23.7
Alofau	495	646	30.5
Amouli	520	920	76.9
Auasi	125	113	-9.6
Aunu'u	476	436	-8.4
Pagai (Part)	88	24	-72.7
Utumea (East)	64	48	-25.0
Sua County	3,417	3,323	-2.8
Afono	530	524	-1.1
Alega	54	54	0.0
Amaua	102	96	-5.9
Aumi	249	186	-25.3
Auto	258	262	1.6
Avaio	57	44	-22.8
Faga'itua	483	433	-10.4
Lauli'i	937	892	-4.8
Masausi	178	164	-7.9
Masefau	435	425	-2.3
Pagai (Part)	34	94	176.5
Sa'ilele	100	75	-25.0
Utusia	..	74	

Continued on next page

Population by District, County and Village: 2000 and 2010 (cont'd)

	2000	2010	% Change
Vaifanua County	2,249	2,545	13.2
Alao	528	495	-6.3
Aoa	507	855	68.6
Onenoa	153	150	-2.0
Tula	413	405	-1.9
Vatia	648	640	-1.2
Western District..	32,435	31,329	-3.4
Lealataua County	5,684	5,103	-10.2
Afao	188	182	-3.2
Agugulu	45	51	13.3
Amaluia	179	162	-9.5
Amanave	287	250	-12.9
Asili	250	224	-10.4
Auma	..	254	
Fagali'i	259	247	-4.6
Fagamalo	39	47	20.5
Failolo	128	108	-15.6
Leone	3,568	1,919	-46.2
Maloata	17	8	-52.9
Nua	207	141	-31.9
Poloa	203	193	-4.9
Puapua	..	965	
Se'etaga	270	299	10.7
Utumea West	44	53	20.5
Leasina County	1,739	1,807	3.9
Aasu	364	494	35.7
Aoloau	778	615	-21.0
Malaeloa/Aitulagi	597	698	16.9
Tualatai County	2,987	3,561	19.2
Futiga	731	723	-1.1
Malaeloa/Ituau	627	550	-12.3
Taputimu	640	841	31.4
Vailoatai	989	1,447	46.3
Tualauta County	22,025	20,858	-5.3
Faleniu	2,056	1,898	-7.7
Iliili	2,513	3,195	27.1
Malaeimi	1,067	1,182	10.8
Mapusagafou	1,642	1,126	-31.4
Mesepa	481	444	-7.7
Nu'uuli (Part)	2,310	659	-71.5
Pava'ia'i	2,200	2,450	11.4
Tafuna	8,409	7,945	-5.5
Vaitogi	1,347	1,959	45.4

Continued on next page

Population by District, County and Village: 2000 and 2010 (cont'd)

	2000	2010	% Change
Manu'a District...	1,378	1,143	-17.1
Faleasao County	135	162	20.0
Faleasao	135	162	20.0
Fitiuta County	358	270	-24.6
Leusoali'i	181	117	-35.4
Maia	177	153	-13.6
Ofu County	289	176	-39.1
Ofu	289	176	-39.1
Olosega County	216	177	-18.1
Olosega	206	172	-16.5
Sili	10	5	-50.0
Ta'u County	380	358	-5.8
Luma	288	183	-36.5
Si'ufaga	92	175	90.2
Swains Island...	37	17	-54.1

Source: U.S. Census Bureau, 2000 Census of American Samoa and the 2010 Census of American Samoa.

Housing Units by District, County and Village: 2010

	2000	2010
American Samoa....................................	10,052	10,963
Eastern District...	4,111	4,490
Ituau County	740	936
Faganeanea	31	31
Fagasa	137	154
Matu'u	71	91
Nu'uuli (Part)	501	660
Maoputasi County	2,031	1,999
Anua	11	5
Atu'u	59	62
Aua	386	392
Fagaalu	204	197
Fagatogo	359	347
Fatumafuti	19	22
Leloaloa	93	102
Pago Pago	742	691
Satala	..	61
Utulei	158	120
Sa'ole County	298	466
Alofau	86	140
Amouli	76	189
Auasi	19	20
Aunu'u	88	97
Pagai (Part)	16	5
Utumea (East)	13	15
Sua County	611	595
Afono	109	82
Alega	13	12
Amaua	23	23
Aumi	37	29
Auto	45	53
Avaio	9	12
Faga'itua	90	76
Lauli'i	155	150
Masausi	35	29
Masefau	73	88
Pagai (Part)	4	13
Sa'ilele	18	14
Utusia	..	14

Continued on next page

Housing Units by District, County and Village: 2010

	2000	2010
Vaifanua County	431	494
Alao	104	102
Aoa	96	168
Onenoa	29	26
Tula	81	82
Vatia	121	116
Western District...	5,610	6,090
Lealataua County	972	1,038
Afao	23	33
Agugulu	8	9
Amaluia	41	31
Amanave	58	52
Asili	36	40
Auma	..	52
Fagali'i	48	43
Fagamalo	14	11
Failolo	21	29
Leone	600	418
Maloata	5	2
Nua	28	25
Poloa	37	42
Puapua	..	190
Se'etaga	44	51
Utumea West	9	10
Leasina County	312	336
Aasu	57	79
Aoloau	162	120
Malaeloa/Aitulagi	93	137
Tualatai County	451	636
Futiga	105	116
Malaeloa/Ituau	87	98
Taputimu	100	159
Vailoatai	159	263
Tualauta County	3,875	4,080
Faleniu	315	347
Iliili	470	642
Malaeimi	189	228
Mapusagafou	285	221
Mesepa	80	97
Nu'uuli (Part)	404	115
Pava'ia'i	401	432
Tafuna	1,488	1,616
Vaitogi	243	382

Continued on next page

Housing Units by District, County and Village: 2010

	2000	2010
Manu'a District..	323	376
Faleasao County	37	40
Faleasao	37	40
Fitiuta County	64	70
Leusoali'i	34	34
Maia	30	36
Ofu County	75	103
Ofu	75	103
Olosega County	67	79
Olosega	62	76
Sili	5	3
Ta'u County	80	84
Luma	58	37
Si'ufaga	22	47
Swains Island..	8	7

Source: U.S. Census Bureau, 2000 Census of American Samoa and the 2010 Census of American Samoa.

Appendix D Tafuna Flow Data

Tafuna Base Flows

Tafuna Base Flow September/October 2011

Day	Rainfall (in)	Average Flow at Plant (MGD)	Maximum Flow at Plant (MGD)
15	0.16	1.6	3.4
16	0	1.7	3.4
17	0	1.6	
18	0	1.6	
19	0.08	1.6	2
20	0	1.7	3
21	0	1.4	3.3
22	0	1.5	3.3
23	0	1.6	3.3
24	0	1.7	
25	0	1.7	
26	0	1.7	3.5
27	0.08	1.5	3.2
28	0	1.6	3.5
29	0	1.5	3.5
30	0.12	1.6	3.5
1	0	1.6	
2	0	1.6	
3	0	1.6	3.4
4	0	1.6	1.4
5	0	1.7	1.4
6	0	1.4	3.5
7	0.05	1.6	1.9
8	0.06	1.6	
9	0	1.6	
10	0.01	1.6	
11	0.02	1.6	3.3
12	0.01	1.5	2.5
Average		1.6	
Maximum			3.5

Tafuna Base Flow September/October 2009

Day	Rainfall (in)	Average Flow at Plant (MGD)	Maximum Flow at Plant (MGD)
15	0.01	1.9	
16	0.02	1.6	
17	0.11	1.6	
18	0	1.6	
19	0.01	1.5	
20	0.03	1	
21	0.07	2	
22	0	1.8	
23	0	1.4	
24	0	1.6	
25	0	1.6	
26	0	1.6	
27	0	1.6	
28	0.03	1.6	
29	0	1.6	
30	0	1.6	
1	0.01	0.9	1.2
2	0	1.5	3.5
3	0	1.7	
4	0.08	1.7	
5	0	1.7	3.7
6	0	1.7	3.9
7	0	1.3	2.6
8	0.02	1.5	2.9
9	0.04	1.7	3.2
10	0	1.7	
11	0	1.7	
12	0	1.7	
Average		1.6	
Maximum			3.9

Tafuna Rain Events

Tafuna Rain Event December 2009

Day	Rainfall (in)	Average Flow at Plant (MGD)	Maximum Flow at Plant (MGD)
8	0.57	1.8	2.2
9	0.68	1.7	2.9
10	6.6	2	3
11	2.26	2.7	3
Average		2.1	2.8
Maximum			3.0

Tafuna Rain Event December 2009

Day	Rainfall (in)	Average Flow at Plant (MGD)	Maximum Flow at Plant (MGD)
25	0.15	2.1	
26	4.06	1.8	
27	5.36	2.1	
28	2.76	3.2	3.7
29	1.07	2.6	2.9
30	0.81	3.1	
31	1.34	2.2	2.3
Average		2.4	3.3
Maximum			3.7

Tafuna Rain Event January 2010

Day	Rainfall (in)	Average Flow at Plant (MGD)	Maximum Flow at Plant (MGD)
21	0.38	2.5	4.3
22	0.37	2.5	4.3
23	0.32	2.8	
24	6.91	2.7	
25	2.19	2.6	4.6
26	1.28	2.9	4.6
27	5.5	3.1	5.0
Average		2.7	4.56
Maximum			5.0

Tafuna Rain Event January 2011

Day	Rainfall (in)	Average Flow at Plant (MGD)	Maximum Flow at Plant (MGD)
19	0.28	1.5	2.8
20	0.89	1.6	2.8
21	1.92	1.8	2.7
22	2.17	2.1	
23	1.54	2.1	
24	5.85	2.1	5.2
25	0.77	3.1	3.1
26	0.48	1.9	3.0
Average		2.0	3.3
Maximum		3.1	5.2

Appendix E Utulei Flow Data

Utulei Base Flows

Utulei Base Flow September/October 2009

Day	Rainfall (in)	Average Flow at Plant (MGD)	Maximum Flow at Plant (MGD)
15	0.16	0.8	2.2
16	0	0.8	2.3
17	0	0.8	2
18	0	0.8	2.3
19	0.08	0.8	
20	0	0.8	
21	0	0.8	2.5
22	0	0.9	2.3
23	0	0.8	2.1
24	0	0.8	3.2
25	0	0.7	1.5
26	0	0.7	
27	0.08	0.6	
28	0	0.8	2.5
29	0	0.7	3.2
30	0.12	0.7	2
1	0	0.6	2
2	0	0.5	2.1
3	0	0.6	
4	0	0.6	
5	0	0.6	2.2
6	0	0.5	
7	0.05	0.5	2.2
8	0.06	0.6	2
9	0	0.4	2.1
10	0.01	0.7	
11	0.02	0.7	
12	0.01	0.7	
Average		1.6	
Maximum			3.2

Utulei Base Flow September/October 2011

Day	Rainfall (in)	Average Flow at Plant (MGD)	Maximum Flow at Plant (MGD)
15	0.01	1.3	2
16	0.02	1.3	2
17	0.11	1.3	
18	0	1.3	
19	0.01	1.3	1.5
20	0.03	1.3	2.1
21	0.07	1.3	3.2
22	0	1.3	3
23	0	1.3	2.1
24	0	1.3	
25	0	1.3	
26	0	1.3	2.1
27	0	1.4	2.1
28	0.03	1.4	2.2
29	0	1.4	2.2
30	0	1.4	3.2
1	0.01	1.3	
2	0	1.3	
3	0	1.3	2
4	0.08	1.3	2
5	0	1.2	3.5
6	0	1.3	2
7	0	1.3	3.5
8	0.02	1.3	
9	0.04	1.3	
10	0	1.3	
11	0	1.3	3
12	0	1.3	2
Average		1.6	
Maximum			3.5

Utulei Rain Events

Utulei Rain Event December 2009

Day	Rainfall (in)	Average Flow at Plant (MGD)	Maximum Flow at Plant (MGD)
8	0.57	0.7	2
9	0.68	1.8	2
10	6.6	1	2
11	2.26	2.1	
Average		2.1	2.8
Maximum			3.0

Utulei Rain Event December 2009

Day	Rainfall (in)	Average Flow at Plant (MGD)	Maximum Flow at Plant (MGD)
25	0.15	0.7	
26	4.06	0.9	
27	5.36	0.9	
28	2.76	0.8	3
29	1.07	1.3	3
30	0.81	1.9	3.2
31	1.34	1.5	3.2
Average		2.4	3.3
Maximum			3.7

Utulei Rain Event January 2010

Day	Rainfall (in)	Average Flow at Plant (MGD)	Maximum Flow at Plant (MGD)
21	0.38	2	1.5
22	0.37	1	1.7
23	0.32	1.7	
24	6.91	1.7	
25	2.19	1.7	3.7
26	1.28	0	3.7
27	5.5	6.1	3.2
Average		2.7	4.56
Maximum			5.0

Utulei Rain Event January 2011

Day	Rainfall (in)	Average Flow at Plant (MGD)	Maximum Flow at Plant (MGD)
19	0.28	1.5	2
20	0.89	1.7	3.6
21	1.92	2.3	3.6
22	2.17	3.3	
23	1.54	3.3	
24	5.85	3.3	4.8
25	0.77	4	4.2
26	0.48	2.8	3.1
Average		2.0	3.3
Maximum		3.1	5.2

Appendix F Average and Maximum Flows at STP 2006-2011

Wastewater Treatment Planr Effluent Record

Utulei WWTP

YEAR	FLOW (MG)				RAIN	BOD (av)	TSS (av)	pH
2010	Mo.	min.	Max	Daily Av	Inches	% Removal	(mg/l)	Effluent
Jan	46.9	1.3	3.7	1.5	20.5	43.6	55.0	7.0
Feb	25.3	0.8	2.6	0.9	8	47.9	56.1	7.0
Mar	15.4	0.8	2.9	0.5	3.1	40.5	46.9	7.0
Apr	29.1	2.5	4.0	1.0	17.3	46.3	59.2	7.0
May	29.3	1.5	4.2	0.9	9.2	53.3	47.4	7.0
June	33.7	1.6	4.0	1.1	12.8	47.4	60.7	7.0
Jul	43.4	1.8	6.0	1.4	8.4	46.8	67.2	7.0
Aug	36.7	2.0	5.9	1.2	8.5	46.7	60.8	7.0
Sept	33.2	2.2	5.2	1.1	9.9	39.3	64.8	7.0
Oct	42.6	1.8	3.2	1.4	7.6	33.0	53.2	7.0
Nov	62.7	2.0	5.0	2.0	7.5	37.8	70.0	7.0
Dec	43.0	1.4	3.5	1.4	14	43.1	54.1	7.0
Avg.	36.8	1.6	4.2	1.2	10.6	43.8	58.0	7.0

Tafuna WWTP

YEAR	FLOW (MG)				Rainfall	BOD (av)	TSS (av)	pH
2010	Mo.	Min.	Max	Daily Av	Inches	% Removal	(mg/l)	Effluent
Jan	85.6	2.7	5.1	2.8	21.0	44.8	61.8	7.0
Feb	77.0	2.3	4.7	2.7	9.5	47.5	60.9	7.0
Mar	77.7	0.0	4.5	2.5	3.6	42.2	51.2	7.0
Apr	91.5	0.0	5.2	3.1	4.7	50.3	66.5	7.0
May	47.9	1.3	5.6	1.5	8.3	48.5	55.0	7.0
June	45.2	1.5	3.5	1.5	6.5	42.9	63.7	7.0
Jul	48.6	1.2	3.2	1.6	4.0	47.7	75.2	7.0
Aug	45.2	1.2	3.3	1.5	5.1	44.1	66.3	7.0
Sept	45.9	1.6	3.2	1.5	3.5	43.7	69.3	7.0
Oct	47.8	1.2	3.2	1.5	6.7	35.4	58.1	7.0
Nov	47.5	1.9	3.5	1.6	6.6	41.7	65.4	7.0
Dec	49.4	1.3	3.0	1.6	8.7	43.0	63.3	7.4
Avg.	59.1	1.4	4.0	2.0	7.4	44.3	63.1	7.0

Utulei WWTP

YEAR	FLOW (MG)				Rainfall	BOD (av)	TSS (av)	pH
2011	Mo.	Min	Max	Daily Av	Inches	% Removal	(mg/l)	Effluent
Jan	63.7	2.0	4.8	2.1	23.4	54.9	45.6	7.0
Feb	50.2	1.8	3.6	1.8	11.7	47.8	57.8	7.0
Mar	48.3	2.0	3.6	1.6	7.0	39.4	62.9	7.0
Apr	44.4	1.4	3.5	1.5	6.5	39.6	69.1	7.0
May	39.2	2.0	3.5	1.3	9.8	40.5	58.5	7.0
June	40.1	2.0	3.6	1.3	4.5	39.5	59.6	7.0
Jul	39.0	2.1	3.2	1.3	1.6	38.4	62.8	7.0
Aug	41.0	1.8	3.6	1.3	5.5	40.8	59.3	7.0
Sept	40.2	1.5	3.5	1.3	1.2	38.2	69.0	7.0
Oct	44.9	2.0	3.5	2.2	1.7	50.8	55.5	7.0
Nov	52.4	2.0	4.5	1.7	8.9	39.1	55.2	7.0
Dec	54.4	1.5	4.0	1.8	6.2	42.0	62.8	7.0
Avg.	46.5	1.8	3.7	1.6	7.3	42.6	59.8	7.0

Tafuna WWTP

YEAR	FLOW (MG)				Rainfall	BOD (av.)	TSS (av)	pH
2011	Mo.	Min.	Max.	Daily Av	Inches	% Removal	mg/l	Effluent
Jan	55.8	1.4	5.2	1.8	11.7	48.5	48.4	7.0
Feb	48.4	1.8	4.9	1.7	9.8	50.4	55.7	7.0
Mar	47.5	1.4	3.1	1.5	3.5	40.7	69.0	7.0
Apr	47.0	1.7	5.0	1.6	2.1	41.5	68.2	7.0
May	54.3	1.4	3.5	1.8	2.6	42.5	58.6	7.0
June	51.8	1.2	3.7	1.7	2.7	42.1	63.5	7.0
Jul	50.0	1.6	3.4	1.6	1.1	40.7	59.8	7.0
Aug	50.0	1.8	3.8	1.6	1.2	45.6	54.0	7.0
Sept	48.1	1.9	3.5	1.6	0.5	42.1	67.7	7.0
Oct	53.1	1.0	3.5	1.7	3.4	52.6	62.2	7.0
Nov	57.8	2.0	3.8	1.9	9.3	41.3	58.1	7.0
Dec	65.9	1.5	4.3	1.3	9.3	45.6	66.7	7.0
Avg.	52.5	1.6	4.0	1.7	4.8	44.5	61.0	7.0

Wastewater Treatment Planr Effluent Record

Utulei WWTP

YEAR	FLOW (MG)				Rainfall	BOD (av)	TSS (av)	pH
2008	Mon.	Min	Max	Daily Av	Inches	%Removal (mg/l)		Effluent
Jan	51.2	1.2	3.2	1.7	18.3	39.6	39.1	7.0
Feb	34.5	0.3	2.8	1.2	6.7	55.2	44.8	7.0
Mar	43.6	1.1	4.5	1.4	19.0	40.4	42.1	7.0
Apr	38.3	1.0	4.5	1.3	10.2	46.4	52.4	7.0
May	43.0	1.0	3.8	1.4	13.9	47.8	48.1	7.0
June	49.0	2.4	4.5	1.6	15.6	46.6	40.6	7.0
Jul	42.8	1.3	3.0	1.4	5.0	51.2	56.0	7.0
Aug	37.3	1.2	2.7	1.2	4.5	47.5	48.9	7.0
Sept	32.8	0.9	2.7	1.1	7.2	56.6	64.9	7.0
Oct	28.5	0.4	3.0	0.9	6.3	42.1	43.7	7.0
Nov	39.3	0.8	3.0	1.3	12.9	42.8	42.7	7.0
Dec	46.2	1.3	3.0	1.5	11.2	33.6	57.4	7.0
Avg.	40.5	1.1	3.4	1.3	10.9	45.8	48.4	7.0

Tafuna WWTP

YEAR	FLOW (MG)				Rainfall	BOD (av)	TSS (av)	pH
2008	Mon.	Min	Max	Daily Av	Inches	%Removal (mg/l)		Effluent
Jan	52.2	1.0	4.4	1.7	10.8	33.8	54.5	7.1
Feb	45.0	1.3	3.7	1.6	8	56.8	47.7	7.0
Mar	47.9	2.4	4.4	1.5	4.6	37.5	44.2	7.0
Apr	47.6	1.7	3.7	1.6	1.3	45.5	38.4	7.0
May	51.9	2.4	4.5	1.7	7.1	41.3	43.7	7.0
June	47.5	1.3	3.7	1.6	3.8	31.7	30.2	7.0
Jul	46.5	1.4	3.8	1.5	1.3	37.2	45.7	7.0
Aug	44.9	1.3	3.6	1.4	2.2	48.2	37.8	7.0
Sept	44.4	2.1	4.5	1.5	2.3	46.2	53.6	7.0
Oct	43.9	1.7	4.8	1.4	2.3	41.8	52.1	7.0
Nov	45.2	1.8	3.6	1.5	10.3	41.6	31.0	6.9
Dec	51.0	1.2	5.0	1.6	11.5	34.8	37.9	6.9
Avg.	47.3	1.6	4.1	1.6	5.5	41.4	43.1	7.0

Utulei WWTP

YEAR	FLOW (MG)				Rainfall	BOD	TSS	Ph
2009	Mon.	Min.	Max	Daily Av	Inches	%Removval (mg/l)		Effluent
Jan	43.3	1.5	3.3	1.4	10.8	34.7	56.8	7.0
Feb	32.0	1.1	3.0	1.1	5.5	33.9	64.1	7.0
Mar	25.8	0.2	2.7	0.8	5.2	31.9	52.8	7.0
Apr	39.7	2.1	4.5	1.3	9.9	43.0	49.2	7.0
May	49.3	1.2	4.7	1.6	17.4	53.3	43.1	7.0
June	44.7	1.3	5.0	1.5	7.7	35.6	54.8	7.0
Jul	43.7	0.6	4.4	1.4	15.3	41.4	56.4	7.0
Aug	28.3	1.3	2.6	0.9	10.9	53.7	45.9	7.0
Sept	27.8	1.5	3.2	0.9	8.1	41.2	37.2	7.0
Oct	19.3	0.7	3.3	0.6	6.2	38.3	53.7	7.0
Nov	32.4	1.0	3.2	1.1	9.7	47.6	45.2	7.0
Dec	32.2	1.4	3.2	1.0	20.8	40.1	52.2	7.0
Avg.	34.9	1.2	3.6	1.1	10.6	41.2	51.0	7.0

Tafuna WWTP

Year	FLOW (MG)				Raifall	BOD (av)	TSS (av)	Ph
2009	Mon.	Min	Max.	Daily Av	Inches	%Removal (mg/l)		Effluent
Jan	48.2	1.5	3.7	1.6	8.5	39.0	47.9	7.0
Feb	42.5	1.6	3.7	1.5	6.3	30.5	60.6	7.0
Mar	47.7	1.7	3.8	1.5	6.1	31.5	47.1	7.0
Apr	50.4	2.0	4.0	1.7	9.2	31.2	54.3	7.0
May	56.1	2.2	4.1	1.8	16.2	39.0	37.2	7.0
June	48.7	2.6	3.7	1.6	4.5	42.3	42.9	7.0
Jul	51.1	2.5	5.1	1.6	7.6	47.5	51.4	7.0
Aug	52.8	1.6	4.1	1.7	7.5	42.9	30.2	7.0
Sept	51.3	0.6	1.7	1.7	9.2	42.3	32.1	7.0
Oct	52.6	1.2	4.1	1.7	10.5	38.4	45.6	7.0
Nov	56.2	2.4	4.9	1.9	20.5	49.8	60.2	7.0
Dec	65.5	1.9	5.1	2.1	16.5	42.3	58.1	7.0
Avg.	51.9	1.8	4.0	1.7	10.2	39.7	47.3	7.0

Wastewater Treatment Planr Effluent Record

Utulei WWTP

YEAR	FLOW (MG)				Rainfall	BOD (av)	TSS (av)	Ph
2006	Mon.	Min	Max	Daily Av	Inches	%Removal (mg/l)		Effluent
Jan	51.2			1.7		39.6	39.1	7.0
Feb	34.5			1.2		55.2	44.8	7.0
Mar	43.6			1.4		40.4	42.1	7.0
Apr	38.3			1.3		46.4	52.4	7.0
May	43.0			1.4		47.8	48.1	7.0
June	49.0			1.6		46.6	40.6	7.0
Jul	42.8			1.4		51.2	56.0	7.0
Aug	37.3			1.2		47.5	48.9	7.0
Sept	32.8			1.1		56.6	64.9	7.0
Oct	28.5			0.9		42.1	43.7	7.0
Nov	39.3			1.3		42.8	42.7	7.0
Dec	47.5			1.5		33.6	57.4	7.0
Avg.	40.7			1.3		45.8	48.4	7.0

Tafuna WWTP

YEAR	FLOW (MG)				Rainfall	BOD (av)	TSS (av)	pH
2006	Mon.	Min	Max	Daily Av	Inches	%Removal (mg/l)		Effluent
Jan	52.2			1.7		33.8	54.5	7.1
Feb	45.0			1.6		56.8	47.7	7.0
Mar	47.9			1.5		37.5	44.2	7.0
Apr	47.6			1.6		45.5	38.4	7.0
May	51.9			1.7		41.3	43.7	7.0
June	47.5			1.6		31.7	30.2	7.0
Jul	46.5			1.5		37.2	45.7	7.0
Aug	44.9			1.4		48.2	37.8	7.0
Sept	44.4			1.5		46.2	53.6	7.0
Oct	43.9			1.4		41.8	52.1	7.0
Nov	45.2			1.5		41.6	31.0	6.9
Dec	51.0			1.6		34.8	37.9	6.9
Avg.				1.6		41.4	43.1	7.0

Utulei WWTP

YEAR	FLOW (MG)				Rainfall	BOD (av)	TSS (av)	pH
2007	Mon.	Min.	Max	Daily Av	Inches	%Removal (mg/l)		Effluent
Jan	61.1	1.4	3.1	2.0	14.1	46.1	47.3	7.0
Feb	44.2	1.6	2.8	2.8	10.9	56.5	49.2	7.0
Mar	45.9	1.5	3.1	1.5	26.9	43.2	43.4	7.1
Apr	37.3	1.0	3.8	1.2	5.7	43.2	42.7	7.0
May	44.5	1.0	3.0	1.4	14.0	42.6	47.0	7.2
June	40.0	1.2	2.6	1.4	5.8	35.5	51.2	7.1
Jul	37.5	1.1	2.3	1.2	4.1	34.2	50.9	7.1
Aug	38.9	0.8	2.9	1.3	6.7	40.6	51.7	7.2
Sept	43.1	1.0	3.0	1.4	9.8	38.3	41.7	7.4
Oct	43.5	1.2	3.6	1.4	16.4	39.1	54..2	7.2
Nov	43.3	1.2	4.0	1.4	13.6	50.3	46.5	7.0
Dec	48.1	1.0	3.7	1.6	20.3	41.8	43.3	7.0
Avg.	44.0	1.2	3.2	1.6	12.4	42.6	42.9	7.1

Tafuna WWTP

Year	FLOW (MG)				Rainfall	BOD (av)	TSS (av)	pH
2007	Mon.	Min	Max	Daily Av	Inches	%Removal (mg/l)		Effluent
Jan	68.4	3.3	7.7	2.2	18.1	54.7	48.6	7.0
Feb	53.7	4.5	6.3	1.9	16.5	54.9	45.8	7.0
Mar	51.3	4.1	7.7	1.7	27.3	50.3	46.6	7.0
Apr	37.3	2.3	7.0	1.2	20.9	51	48.2	7.0
May	52.6	3.0	3.7	1.7	11.1	34.8	39.8	7.0
June	47.2	1.6	3.5	1.6	8.9	52.5	53.3	7.0
Jul	44.6	1.0	4.3	1.4	4.5	55.3	45.1	7.0
Aug	44.8	1.0	4.1	1.4	5.6	47.2	44.5	7.1
Sept	49.5	1.5	3.8	1.7	7.6	40.3	44.0	7.0
Oct	50.6	1.6	3.7	1.6	3.2	46.8	46.8	7.0
Nov	50.9	2.3	5.2	1.7	11.7	51.9	44.3	7.0
Dec	52.9	1.5	4.7	1.7	20.9	53.2	39.2	7.0
Avg.	50.3	2.3	5.1	1.7	13.0	49.4	45.5	7.0

Appendix G Typical Sewage Design Flows

CHAPTER 5

Wastewater Collection System

5.1 GENERAL REQUIREMENTS

NOTE: *Any and all more stringent requirements by Federal, State, County or local codes or ordinances shall take precedence.*

5.1.1 Arizona Aquifer Protection Permit Requirements

The design of sewage collection systems shall conform to the requirements of the Aquifer Protection Permit General Permit rules in Arizona Administrative Code Title 18, Chapter 9. An *Application for Approval to Construct and/or Notice of Intent to Discharge* shall be submitted in accordance with AAC R18-9-A301(B) and E301(C). An *Approval to Construct and/or Provisional Verification of General Permit Conformance* shall be issued prior to commencing construction. *Approval to Construct and/or Provisional Verification of General Permit Conformance* include, but are not limited to, the following requirements:

1. Engineer's Design Report.
2. Complete Construction-Ready Design Plans.
3. Specifications (CIP projects).
4. All other relevant information to verify that the facility conforms to the terms of the 4.01 General Permit.

The design report, plans and specifications shall be signed and sealed by an Arizona Registered Professional Civil Engineer.

The sewage collection system shall not be placed in service until an *Approval of Construction and/or Verification of General Permit Conformance* has been issued. *Approval of Construction and/or Verification of General Permit Conformance* includes, but is not limited to, the following requirements:

1. An *Engineer's Certificate of Completion* sealed and signed by an Arizona Registered Professional Civil Engineer, attesting that the sewers have been constructed to the requirements of AAC R18-9-A301.
2. As-built drawings, with each changed sheet sealed and signed by an Arizona Registered Professional Civil Engineer, are submitted to the City of Phoenix, Water Services Department.
3. Satisfactory test results from deflection, leakage, and uniform slope testing are confirmed by the City of Phoenix.
4. All other relevant information to verify that the facility conforms to the terms of the 4.01 General Permit.

5.1.2 Maricopa County Health Code Requirements

Sewage collection systems shall conform to the requirements of the Maricopa County Health Code, Chapter 2. Sewage collection system projects to be installed as part of the City's CIP and projects to be constructed by private developers with City financial contribution shall be required to have plans approved and receive the certificates of approval and verifications of general permit conformance described above by the Maricopa County Department of Environmental Services.

5.1.3 City Code Requirements

Sewage collection systems shall comply with the requirements of Chapter 28 of the Phoenix City Code.

The sewer line extension policy of the City of Phoenix is contained in Article III of Chapter 28 of the Phoenix City Code.

Developers shall pay all costs for constructing all elements of the public wastewater system authorized by the City. Under certain circumstances as described in Section 28-23 of the Code repayment of the cost of "off-site" sewer lines may be available.

Sewer extensions shall be designed for projected flows even when the diameter of the receiving sewer is less than the diameter of the proposed extension at a manhole with special consideration of an appropriate flow channel to minimize turbulence when there is a change in sewer size. A relief sewer may be planned in the future.

5.1.4 Environmental and Cultural Regulatory Requirements

This section is not intended to be all encompassing, but is provided as an overview of environmental and cultural requirements and typical agency involvement. A thorough consideration of the environmental and cultural impact of the project at the project location or along the project route shall be evaluated to identify environmental and cultural requirements. Private developers shall be responsible for regulatory compliance and for obtaining the required permits for their projects.

Whenever a project impacts Waters of the United States, a Clean Water Act, Section 404 Permit will be required by the U.S. Army Corps of Engineers (Corps). A jurisdictional delineation shall be performed to gain the Corps concurrence on the jurisdictional limits of the Waters of the United States. The amount of acreage disturbed in jurisdictional areas in addition to other criteria will determine if an individual or nationwide permit will be required. A Preconstruction Notification (PCN) may be required to be submitted to the Corps to support a determination that the proposed construction activities comply with the terms and conditions of the applicable Nationwide Permit(s). The proposed activities will also need to comply with all terms and applicable general, regional, and 401 water quality conditions.

Compliance is required with the Arizona Pollution Discharge Elimination System (AZDES) general permit for storm water discharges from construction sites. The permit, issued on February 28, 2003 by ADEQ, replaces Stormwater National Pollutant Discharge Elimination System (NPDES) permits previously issued by the U. S. Environmental Protection Agency (EPA). Coverage under the general permit is required for all operators of construction sites that disturb one (1) or more acres of soil through grading, trenching, or excavation.

Projects shall not adversely impact threatened or endangered species or their habitat and shall comply with the Federal Endangered Species Act. To address any biological requirements, an assessment report of the project may be required by the U.S. Fish and Wildlife Service and the Arizona Game and Fish Department.

No project shall adversely impact historic or prehistoric properties. Projects shall comply with the National Historic Preservation Act, the City's Archaeological policy, the Arizona Antiquities Act and the State Historic Preservation Act. As part of the cultural resources consideration the City of Phoenix Archaeologist and the City of Phoenix Historical Preservation Office may be contacted for additional information and direction.

5.1.5 Community Notification and Involvement

The City has made a commitment to early citizen notification and involvement. The goal of identifying neighborhood concerns has a high priority. Communication through printed notice, and public presentations could be a necessary element in construction plan approval.

5.2 SUBMITTALS

5.2.1 Wastewater Master Plan and Report

Planned Community Districts (PCD's) require the submittal of a wastewater master plan and design report. Master plans are required to establish specific improvements and the sequence of improvements that must be completed prior to vesting of the PCD overlay zoning. Guidelines for preparing PCD wastewater and other master plans are provided in the *Planned Community Districts Master Plan Manual* available from the Development Services Department.

The Water Services Department may require the submittal of a wastewater master plan and design report for large non-PCD developments where significant offsite infrastructure is required.

5.2.2 Design Report

A design report shall be submitted for all proposed sewage collection systems. The design report shall include a description of the project, the basis of the design, calculations for project design flow, system capacity, capacity phasing and other information needed to gain a clear understanding of the project. The design report shall be signed and sealed by an Arizona Registered Professional Civil Engineer.

1. Project Description- Describe the type of development including number of units. Provide a site map of the project showing major streets and physical features such as canals, floodplains, railroads, washes, etc. Describe the proposed collection system including lift stations and force mains. Describe the existing system adjacent to proposed development and specific locations where the proposed collection system will connect to the existing system. Show the service area that will be served by the proposed sewage collection system, including offsite areas. Show alignment of proposed offsite mains and onsite mains with diameter of 12-inches and greater.

2. Design Flows- Provide the design average and design peak flows for the sewage collection system. The basis of the projection of initial and future flows shall be included and must be based upon the initial service area and the ultimate upstream service area that can be served by gravity even if it is outside a development's project area. Flow projections shall be based on the land use plan data calculated using the per capita flows, persons/home densities, and peaking factors in this Chapter.

3. Basis of Design- Provide the basis of design for the sewage collection system, including pipe sizes and slopes. Include the sizing calculations and calculations showing that there is sufficient hydraulic capacity to transport the design flows at the proposed sizes and slopes.

4. Conformance with Master Plans- The engineering report shall show that the proposed collection system conforms to the City's master plan for the area and the development's specific master plan.

5. Environmental Issues- The report shall address potential compliance issues with Clean Water Act Section 404, cultural resources, or any other environmental requirements.

5.2.3 Construction Plans – CIP Projects and Projects with City Financial Participation

For Capital Improvement Program (CIP) projects and private development projects where the City participates financially, signed and sealed design plans shall be submitted to WSD for approval. The plans shall also be submitted to Maricopa County Environmental Services Department (MCESD) for Approval to Construct and Provisional Verification of General Permit Conformance.

All plans for the wastewater collection system addition or modification shall bear a suitable title showing the City's name. They shall show a graphical scale, the north arrow, date and the name and signature of the engineer, with the certificate number and imprint of the professional engineering seal. A space shall be provided for signature and/or approval stamp of the WSD and MCESD.

The plans shall be clear and legible (suitable for microfilming). They shall be drawn to a scale, which will permit all necessary information to be plainly shown. The COP datum used shall be indicated. Gravity sewer slopes shall be indicated in percentages.

Plans shall conform to the City of Phoenix Administrative Procedure No. 13 (AP13). Refer to Appendix D.

There are four main views for plans; plan, elevation, section, and supplementary. The plans are used in conjunction with the specifications and general layouts to form the working information for contract and construction of the proposed facilities.

For preparation of CIP project wastewater system plans, refer to the appropriate checklist in Appendix C "Capital Improvement Construction Plan Required Checklists".

5.2.4 Construction Plans – Private Development Projects

All technical and engineering plans relating to private developer projects subject to the Development Review Process shall be submitted to the Development Services Department (DSD) for review and approval.

For preparation of private development wastewater system plans that will become a part of the Phoenix collection system, refer to the appropriate checklist in Appendix B.

5.2.5 Record Drawings

Three sets of construction plans shall be submitted to the inspector as record drawings. The record drawings shall be sealed and signed by an Arizona Registered Professional Engineer. The record drawings shall meet the requirements of WSD policy P-69 for private development projects and policy P-85 for CIP projects. For CIP projects, a CD of the sealed record drawings is also required to be submitted to the City.

5.2.6 Technical Specifications - CIP Projects and Projects with City Financial Participation

For City CIP projects and private development projects where the City participates financially, complete signed and sealed technical specifications shall be submitted for the construction of sewers and all other appurtenances, and shall accompany the plans.

The specifications accompanying construction drawings shall include, but not be limited to, specifications for the approved procedures for operation during construction, all construction information not shown on the drawings that is necessary to inform the builder in detail for the design requirements for the quality of materials, workmanship and fabrication of the project.

Technical specifications shall conform to the MAG Specifications and City Supplements.

5.3 WASTEWATER DESIGN FLOWS

Design flows utilized in the preparation of engineering design reports, plans and specifications shall as a minimum, conform to the criteria set forth in this section.

5.3.1 Average Daily Flows Based Land Use

Average daily flow estimates based on land use, shall conform to Table 5.1 "Average Daily Flows by Land Use". Where the project land use does not fit within the tabulated categories, an average daily unit flow of 100 gallons per person per day shall be used.

Table 5.1 Average Daily Flows by Land Use

LAND USE		AVERAGE DAILY FLOW
Low Density Residential (Less than 5 dwelling units per acre)	Use 3.2 persons per dwelling unit	100 Gallons per person per day
High Density Residential (More than 5 dwelling units per acre)	Use 2.5 persons per dwelling unit, unless otherwise directed by WSD	100 Gallons per person per day
Commercial		3,000 Gallons per acre per day
Industrial/Warehousing	Domestic flows only	1,000 Gallons per acre per day
Commercial Office High Rise		100 Gallons per 1,000 square feet per day
Industrial		50 Gallons per person per day or 5,200 Gallons per acre per day
Hotel/Motel		130 Gallons per room per day
Schools	Without lunch or shower facilities	50 Gallons per student per day
Schools	With lunch & shower facilities	75 Gallons per student per day
Malls/Retail Areas		0.5 Gallons per square foot per day

5.3.2 Peak Flow

All gravity sewers, lift stations, and force mains shall be designed for peak flow conditions. Peak flow is calculated as the product of the peaking factor and the average daily flow. The peaking factor should be calculated from Harmon's formula.

Design Flow Equation

Design Flow = Peak Flow = Q Peak = Q avg $[1+14/(4+\sqrt{P})]$, Where P = Population/1,000

5.3.3 Design Flows Based on Fixture Units

Some facilities, such as large hospitals, may require that design flows be estimated by using the "fixture-unit" method. A fixture-unit flow rate is defined as the total discharge flow in gallons per minute of a single fixture divided by 7.5, which provides the flow rate of that particular plumbing fixture as a unit of flow. Fixtures are rated as multiples of this unit of flow. Fixture unit values are included in the Uniform Plumbing Code. Fixture unit calculations shall be included in the design report.

TITLE 18. ENVIRONMENTAL QUALITY
CHAPTER 9. DEPARTMENT OF ENVIRONMENTAL QUALITY
ARTICLE 3. AQUIFER PROTECTION PERMITS
PART E. TYPE 4 GENERAL PERMITS

Table 1. Unit Design Flows

Wastewater Source	Applicable Unit	Sewage Design Flow per Applicable Unit, Gallons Per Day
Airport	Passenger (average daily number) Employee	4 15
Auto Wash	Facility	Per manufacturer, if consistent with this Chapter
Bar/Lounge	Seat	30
Barber Shop	Chair	35
Beauty Parlor	Chair	100
Bowling Alley (snack bar only)	Lane	75
Camp Day camp, no cooking facilities Campground, overnight, flush toilets Campground, overnight, flush toilets and shower Campground, luxury Camp, youth, summer, or seasonal	 Camping unit Camping unit Camping unit Person Person	 30 75 150 100-150 50
Church Without kitchen With kitchen	 Person (maximum attendance) Person (maximum attendance)	 5 7
Country Club	Resident Member Nonresident Member	100 10
Dance Hall	Patron	5
Dental Office	Chair	500
Dog Kennel	Animal, maximum occupancy	15
Dwelling For determining design flow for sewage treatment facilities under R18-9-B202(A)(9)(a) and sewage collection systems under R18-9-E301(D) and R18-9-B301(K), excluding peaking factor.	Person	80

Dwelling		
For on-site wastewater treatment facilities per R18-9-E302 through R18-9-E323:		
Apartment Building		
1 bedroom	Apartment	200
2 bedroom	Apartment	300
3 bedroom	Apartment	400
4 bedroom	Apartment	500
Seasonal or Summer Dwelling (with recorded seasonal occupancy restriction)	Resident	100
Single Family Dwellings	see R18-9-A314(D)(1)	see R18-9-A314(D)(1)
Other than Single Family Dwelling, the greater flow value based on:		
Bedroom count		
1-2 bedrooms	Bedroom	300
Each bedroom over 2	Bedroom	150
Fixture count	Fixture unit	25
Fire Station	Employee	45
Hospital		
All flows	Bed	250
Kitchen waste only	Bed	25
Laundry waste only	Bed	40
Hotel/motel		
Without kitchen	Bed (2 person)	50
With kitchen	Bed (2 person)	60
Industrial facility		
Without showers	Employee	25
With showers	Employee	35
Cafeteria, add	Employee	5
Institutions		
Resident	Person	75
Nursing home	Person	125
Rest home	Person	125
Laundry		
Self service	Wash cycle	50
Commercial	Washing machine	Per manufacturer, if consistent with this Chapter
Office Building	Employee	20

Park (temporary use)		
Picnic, with showers, flush toilets	Parking space	40
Picnic, with flush toilets only	Parking space	20
Recreational vehicle, no water or sewer connections	Vehicle space	75
Recreational vehicle, with water and sewer connections	Vehicle space	100
Mobile home/Trailer	Space	250
Restaurant/Cafeteria	Employee	20
With toilet, add	Customer	7
Kitchen waste, add	Meal	6
Garbage disposal, add	Meal	1
Cocktail lounge, add	Customer	2
Kitchen waste disposal service, add	Meal	2
Restroom, public	Toilet	200
School		
Staff and office	Person	20
Elementary, add	Student	15
Middle and High, add	Student	20
with gym & showers, add	Student	5
with cafeteria, add	Student	3
Boarding, total flow	Person	100
Service Station with toilets	First bay	1000
	Each additional bay	500
Shopping Center, no food or laundry	Square foot of retail space	0.1
Store	Employee	20
Public restroom, add	Square foot of retail space	0.1
Swimming Pool, Public	Person	10
Theater		
Indoor	Seat	5
Drive-in	Car space	10

Note: Unit flow rates published in standard texts, literature sources, or relevant area or regional studies are considered by the Department, if appropriate to the project.

Appendix H Lift Station Maintenance Logs

Drawdown Site: **Airport**

Date: 4/12/2012

Time: 7:40am

staff: Adriell Moa

dischared pipe: 6-in

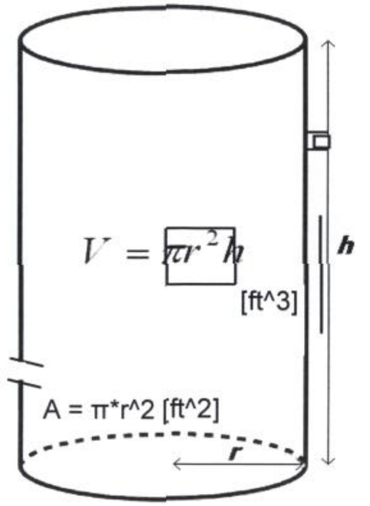
Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 12-in in el. ≈ 7.8-ft from cover	10	10.2	11.5	1.30		78.54	102.10	763.72

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time	7:46 AM	7	32	7.53		101.38		Pump # 1 Time
Pump # _1_ Time	7:47 AM	1	14	1.23	avg fill	98.32	+ 619.23	= 717.56
Fill Time	7:55 AM	8	1	8.02		95.27		Pump # 2 Time
Pump # _2_ Time	7:56 AM	1	33	1.55	avg fill	100.18	+ 492.72	= 592.91
Fill Time	8:03 AM	7	16	7.27		105.10		Pump # Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				

In. El.

i.d.



Pump 1 Average

717.56 gpm

Pump 2 Average

592.91 gpm

AM Inflow Average: **98.32 gpm**

Notes:

This was the second time testing these pumps this week. Pump 2 as efficient as pump 1 before, but is a lot better this time.

This test was intended to catch peak inflow

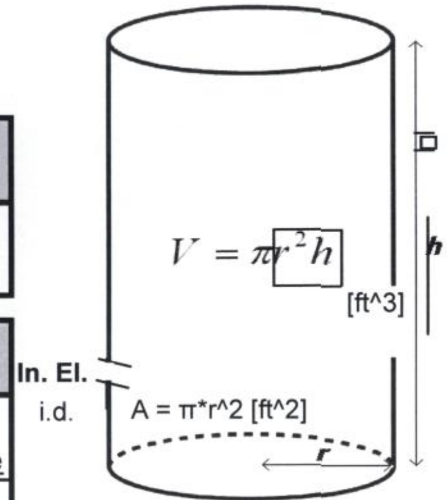
Drawdown Site: **Airport PS**

Date: 4/9/2012   Time: 9:25 AM   staff: Adriell Moa

dischared pipe: 6-in   Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 12-in in el. ≈ 7.8-ft from cover	10	9	10	1.00		78.54	78.54	587.48

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time	9:33 AM	3	26	3.43		171.11		Pump # 1 Time
Pump # _1_ Time	9:34 AM	1	14	1.23	avg fill	129.29	+ 476.33	= 605.62
Fill Time	9:41 AM	6	43	6.72		87.47		Pump # 2 Time
Pump # _2_ Time	9:44 AM	2	58	2.97	avg fill	111.52	+ 198.03	= 309.54
Fill Time	9:48 AM	4	20	4.33		135.57		Pump # 1 Time
Pump # _1_ Time	9:50 AM	1	22	1.37	avg fill	128.56	+ 429.86	= 558.42
Fill Time	9:55 AM	4	50	4.83		121.55		Pump # 2 Time
Pump # _2_ Time	9:59 AM	4	24	4.40	avg fill	121.55	+ 133.52	= 255.06
Fill Time	10:04 AM	4	50	4.83		121.55		Pump # 1 Time
Pump # _1_ Time	10:05 AM	1	31	1.52	avg fill	123.72	+ 387.35	= 511.07
Fill Time	10:10 AM	4	40	4.67		125.89		Pump # 2 Time
Pump # _2_ Time	10:14 AM	3	39	3.65	avg fill	136.38	+ 160.95	= 297.33
Fill Time	10:18 AM	4	0	4.00		146.87		



Pump 1 Average

558.37 gpm

Pump 2 Average

287.31 gpm

Inflow Average: 127.90 gpm

Notes:

Rainy Day. Pump2 was cavitating and taking twice as long.

The lowest manhole in Airport's Basin is the second manhole directly infront of the PS on the road.

Drawdown Site: **Andy PS**

Date: 4/11/2012

Time: 9:00am

staff: Adriell Moa

dischared pipe:

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: (two) 4-in's in el. ≈ -ft from cover	40-inches	4.6	5.7	1.10				

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump (gpm)
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
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Fill Time				0.00				Pump # Time
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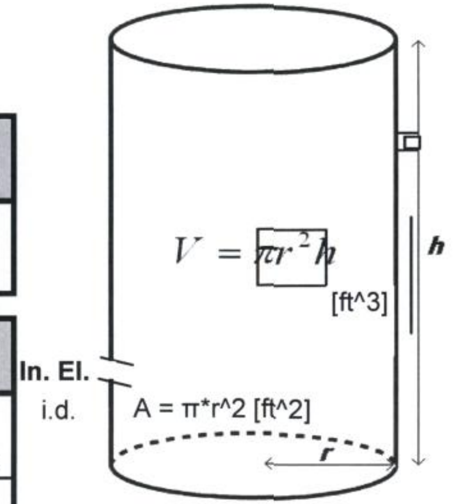
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				



Pump 1 Average

gpm

Pump 2 Average

gpm

AM Inflow Average:

gpm

Notes:

2-in discharge, flow is little to none.

Drawdown Site: **Atuu**

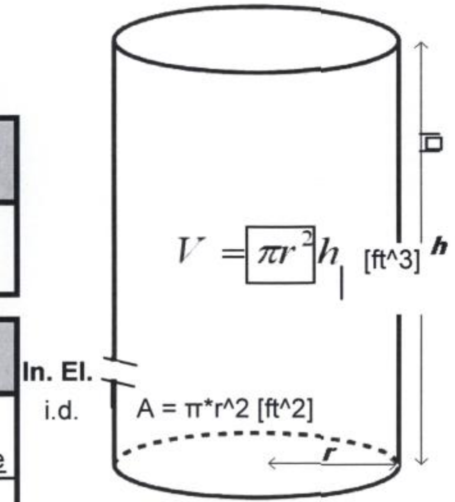
Date: 4/10/2012 | Time: 9:50 AM | staff: Adriell Moa

dischared pipe: 4-in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 8-in in el. ≈ 8-ft from cover	5	10.4	11.7	1.30		19.63	25.53	190.93

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time	10:18 AM	24	28	24.47		12.61		Pumps 1&2 Time
Pumps 1&2 Time	10:32 AM	13	7	13.12	avg fill	13.58	14.56	28.14
Fill Time	10:32 AM	13	7	13.12		14.56		Pump # 2 Time
Pump # _2_ Time	10:34 AM	1	55	1.92	avg fill	13.40	99.62	113.01
Fill Time	10:50 AM	15	36	15.60		12.24		Pump # 2 Time
Pump # _2_ Time	10:54 AM	2	5	2.08	avg fill	13.38	91.65	105.03
Fill Time	11:07 AM	13	9	13.15		14.52		Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				



Pump 2 Average

109.02 gpm

Inflow Average: **13.48 gpm**

Notes:

There is a hole around 8-12inches in diameter. There are a few old lines from the cannery, but this one was not plugged. The hole almost to the bottom, about 12.5-ft from the hatch covers.

Manually stopped pump 1 because it would simultaneously come on when pump 2 did. Also tried pump1 by itself and it was not pumping much at all.

Drawdown Site: **Coconut Pt #1**

Date: 4/11/2012

Time: 6:53am

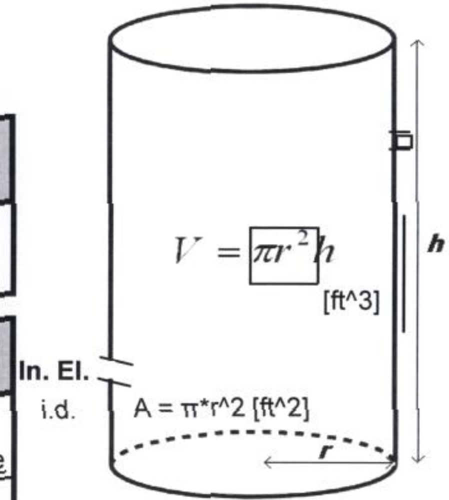
staff: Adriell Moa

dischared pipe: 6"

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 8" in el. 10-ft from cover	12:00 AM	12.4	16.6	4.20		28.27	118.75	888.27

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump (gpm)
Fill Time	7:11 AM	10	38	10.63		83.54		Pump # 1 Time
Pump # _1_ Time	7:20 AM	8	52	8.87	avg fill	81.19	+ 100.18	= 181.37
Fill Time	7:31 AM	11	16	11.27		78.84		Pump # 2 Time
Pump # _2_ Time	7:43 AM	11	57	11.95	avg fill	80.73	+ 74.33	= 155.07
Fill Time	7:53 AM	10	45	10.75		82.63		Pump # 1 Time
Pump # _1_ Time	8:02 AM	8	46	8.77	avg fill	73.11	+ 101.32	= 174.44
Fill Time	8:16 AM	13	58	13.97		63.60		Pump # 2 Time
Pump # _2_ Time	8:26 AM	9	46	9.77	avg fill	64.94	+ 90.95	= 155.89
Fill Time	8:39 AM	13	24	13.40		66.29		Pump # Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				



Pump 1 Average

177.90 gpm

Pump 2 Average

155.48 gpm

AM Inflow Average: **73.08 gpm**

Notes:

There is a hole about 6-in wide and about 5-ft under the inlet pipe.

Drawdown Site: **Coconut Pt #2**

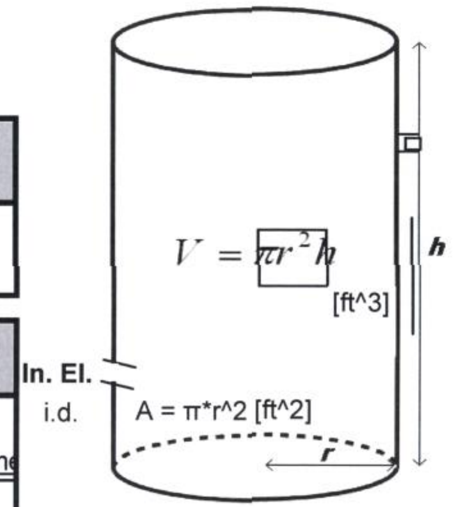
Date: 4/11/2012 | Time: 11:30am | staff: Adriell Moa

dischared pipe: 2"

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 6-in in el. ≈ 6.6-ft from cover	3.75	6.6	7.1	0.50		11.04	5.52	41.31

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump (gpm)
Fill Time	12:02 PM	21	12	21.20		1.95		Pump # 1&2 Time
Pump # _1&2_ Time	12:26 PM	24		24.00	avg fill	1.63	+ 1.72	= 3.35
Fill Time @ 6.9-ft	12:45 PM	19		19.00		1.30		Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				



Pump 1 Average

3.35 gpm

Pump 2 Average

gpm

AM Inflow Average: **1.63 gpm**

Notes:

2-in discharge. Laundry mat is connected to wetwell. Takes hours to pump down at set points (pumps not keeping up with the flow). Pump 2 is always on, Pump 1 with Pump 2 on takes 15 mins just to pump 6-in from wet well.

Drawdown Site: **Coconut Pt #3**

Date: 4/11/2012

Time: 10:30am

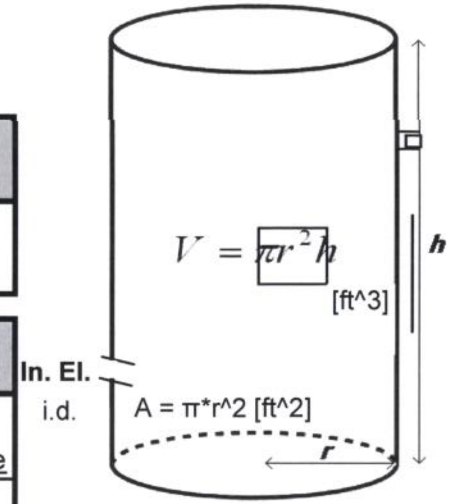
staff: Adriell Moa

dischared pipe:

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 6-in in el. ≈ 10.3-ft from cover	3.33	12.2	13.2	1.00		8.71	8.71	65.14

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump (gpm)
Fill Time	10:36 AM	26	4	26.07		2.50		Pump # 1 Time
Pump # _1_ Time	10:41 AM	5	0	5.00	avg fill	3.45	13.03	16.48
Fill Time	10:55 AM	14	47	14.78		4.41		Pump # 2 Time
Pump # _2_ Time	10:58 AM	2	7	2.12	avg fill	3.59	30.78	34.37
Fill Time	11:22 AM	23	29	23.48		2.77		Pump # __ Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				



Pump 1 Average

16.48 gpm

Pump 2 Average

34.37 gpm

AM Inflow Average:

3.45 gpm

Notes:

Drawdown Site: **Fagaalu (Hospital) PS**

Date: 4/10/2012

Time: 7:40 AM

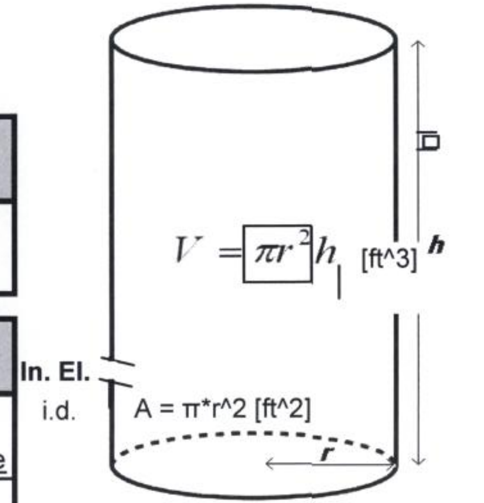
staff: Adriell Moa

dischared pipe: 6-in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 8-in in el. ≈ 12-ft from cover	5	13	15	2.00		19.63	39.27	293.74

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time	7:49 AM	2	53	2.88		101.87		Pump # 1 Time
Pump # _1_ Time	8:00 AM	11	2	11.03	avg fill	99.62	+ 26.62	= 126.25
Fill Time	8:03 AM	3	1	3.02		97.37		Pump # 2 Time
Pump # _2_ Time	8:07 AM	3	43	3.72	avg fill	93.65	+ 79.03	= 172.68
Fill Time	8:10 AM	3	16	3.27		89.92		Pump # 1 Time
Pump # _1_ Time	8:19 AM	9	0	9.00	avg fill	88.58	+ 32.64	= 121.22
Fill Time	8:23 AM	3	22	3.37		87.25		Pump # 2 Time
Pump # _2_ Time	8:26 AM	3	16	3.27	avg fill	90.25	+ 89.92	= 180.17
Fill Time	8:29 AM	3	9	3.15		93.25		Pump # Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				



Pump 1 Average

123.73 gpm

Pump 2 Average

176.42 gpm

Inflow Average: **92.82 gpm**

Notes:

Pump 1 was noticiably slower.

Special Ed and Elementry (Matafao) manifolds into the Fagaalu forcemain.

Drawdown Site: **Fagaalu (hospital)**

Date: 4/10/2012

Time: 1:18pm

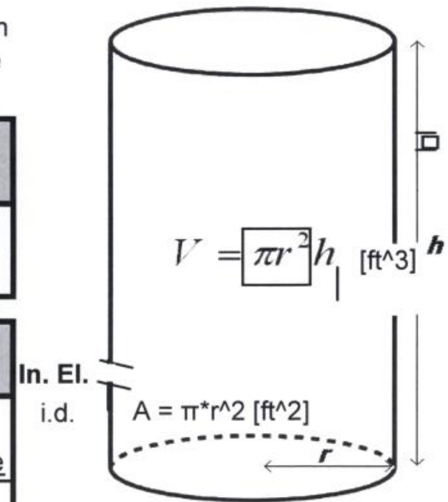
staff:
Adriell Shrikissoon
Moamoaniu Loke

dischared pipe: 6-in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 8-in in el. ≈ 12-ft from cover	5	12	15	3.00		19.63	58.90	440.61

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)		Actual Pump Rate (gpm)
Fill Time	1:26pm	3	41	3.68		119.62			Pump # 1 Time
Pump # _1_ Time	1:35pm	8	34	8.57	avg fill	133.25	+ 51.43	=	184.68
Fill Time	1:38pm	3	0	3.00		146.87			Pump # 2 Time
Pump # _2_ Time	1:41pm	2	42	2.70	avg fill	127.17	+ 163.19	=	290.36
Fill Time	1:45pm	4	6	4.10		107.47			Pump # 2 Time
Pump # _2_ Time	1:48pm	2	58	2.97	avg fill	115.50	+ 148.52	=	264.02
Fill Time	1:51pm	3	34	3.57		123.54			Pump # 2 Time
Pump # _2_ Time	1:54pm	2	56	2.93	avg fill	114.01	+ 150.21	=	264.22
Fill Time	1:58pm	4	13	4.22		104.49			Pump # Time
Pump # __ Time				0.00	avg fill		+	=	
Fill Time				0.00					Pump # Time
Pump # __ Time				0.00	avg fill		+	=	
Fill Time				0.00					



Pump 1 Average

184.68 gpm

Pump 2 Average

272.87 gpm

Inflow Average: **122.48 gpm**

Notes:

Tested stating earlier in the day, both times Pump 1 was pumping pretty slow.

Drawdown Site: **Fatu-ma-Futi**

Date: 4/10/2012

Time: 2:05pm

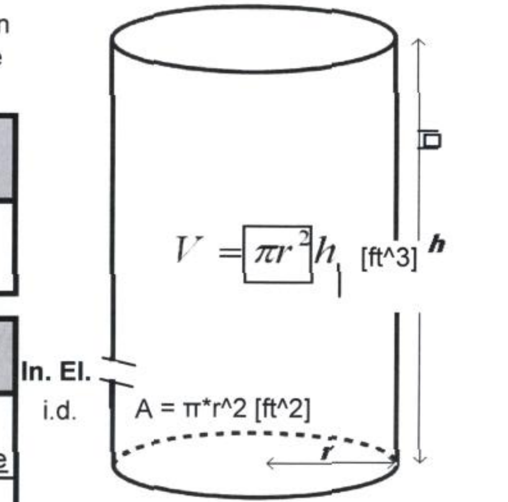
staff:
Adriell Shrikissoon
Moamoaniu Loke

dischared pipe: 4-in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 8-in and 4-in in el. ≈ 6.2-ft and 6.8-ft from cover	4	8.6	9.9	1.30		12.57	16.34	122.20

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)		Actual Pump Rate (gpm)
Fill Time				0.00					Pump # 1 Time
Pump # _1_ Time	2:08pm	1	50	1.83	avg fill	4.36	+ 66.65	=	71.01
Fill Time	2:36pm	28	1	28.02		4.36			Pump # 2 @ 8.6
Pump # 2 @ 8.6	2:38pm	1	43	1.72	avg fill	5.01	+ 71.18	=	76.19
Fill Time	3:06pm	28	15	28.25		5.66			Pump 1 @ 8.2
Pump 1 @ 8.2	3:08pm	2	24	2.40	avg fill	5.04	+ 66.58	=	71.62
Fill Time	3:36pm	27	37	27.62		4.42			
Pump # _2_ Time				0.00	avg fill	4.25	+	=	
Fill Time	4:06pm	29	57	29.95		4.08			Pump # 1 Time
Pump # _1_ Time	4:08pm	2	3	2.05	avg fill	4.23	+ 59.61	=	63.83
Fill Time	4:35pm	27	57	27.95		4.37			Pump # 2 @ 7.3
Pump # 2 @ 7.3	4:38pm	2	55	2.92	avg fill	4.37	+ 41.90	=	46.27
Fill Time				0.00					



Pump 1 Average

68.82 gpm

Pump 2 Average

61.23 gpm

Inflow Average: **4.54 gpm**

Notes:
Diameter at rim is 6-ft, but the actual diameter where the set points are is 4-ft.

Drawdown Site: **Korea PS**

Date: 4/10/2012

Time: 11:00 AM

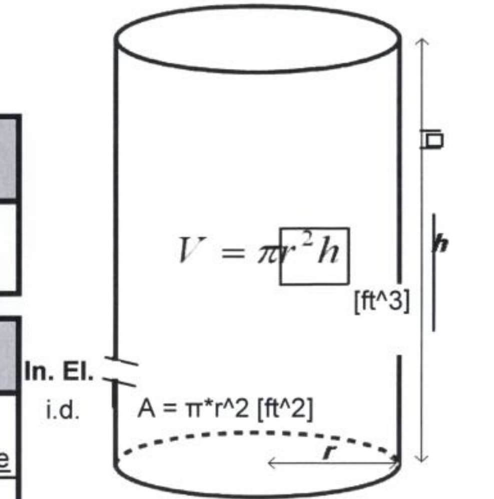
staff: Adriell Moa

dischared pipe: -in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 8-in in el. ≈ 15.9-ft from cover	5	16.5	18.5	2.00		19.63	39.27	293.74

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time	11:19 AM		52	0.87		338.93		Pump # 1 Time
Pump # _1_ Time	11:22 AM	2	4	2.07	avg fill	239.96	+ 142.13	= 382.09
Fill Time	11:24 AM	2	5	2.08		140.99		Pump # 2 Time
Pump # _2_ Time	11:25 AM	1	2	1.03	avg fill	112.66	+ 284.26	= 396.92
Fill Time	11:29 AM	3	29	3.48		84.33		Pump # 1 Time
Pump # _1_ Time	11:30 AM	1		1.00	avg fill	120.15	+ 293.74	= 413.89
Fill Time	11:32 AM	1	53	1.88		155.97		Pump # 2 Time
Pump # _2_ Time	11:33 AM	1	41	1.68	avg fill	209.51	+ 174.50	= 384.01
Fill Time	11:35 AM	1	7	1.12		263.05		Pump # 1 Time
Pump # _1_ Time	11:39 AM	4	35	4.58	avg fill	284.78	+ 102.54	= 387.32
Fill Time	11:41 AM	1	32	1.53		306.51		Pump # 2 Time
Pump # _2_ Time	11:45 AM	3	46	3.77	avg fill	287.53	+ 124.77	= 412.31
Fill Time	11:45 AM	1	45	1.75		268.56		



Pump 1 Average

394.43 gpm

Pump 2 Average

397.75 gpm

Inflow Average: **189.87 gpm**

Notes:

Drawdown Test Site: Korea

Date: 8/10/11

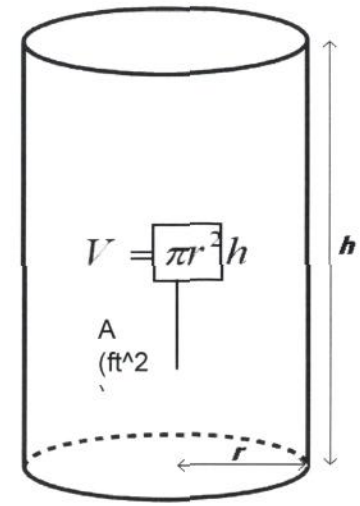
Time: 5:45 AM

5PM PM peak

staff: Adriell Lino

Diameter of WW (ft)	pump on (in)	pump off (in)	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
5			2		19.63	39.27	293.74

Time	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump (gpm)
Pump # _____ Time							
Fill Time	2	9	2.15		136.62		
Pump # ___1__ Time	1	18	1.30	avg fill	119.25	225.95	345.20
Fill Time	2	53	2.88		101.87		
Pump # __1___ Time		59	0.98	avg fill	92.90	298.72	391.62
Fill Time	3	30	3.50		83.93		
Pump # __2___ Time		41	0.68	avg fill	78.38	429.86	508.24
Fill Time	4	2	4.03		72.83		
Pump # __2___ Time		38	0.63	avg fill	64.57	463.80	528.37
Fill Time	5	13	5.22		56.31		



Pump 1 Average

368.41 **gpm**

Pump 2 Average

518.30 **gpm**

Notes:

Drawdown Site: **Lavatai**

Date: 4/12/2012

Time: 11:00am

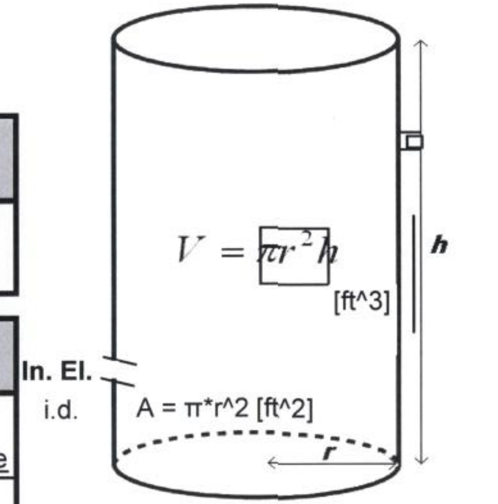
staff: Adriell Moa

dischared pipe: 4-in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 8-in in el. ≈ 8.7-ft from cover	6	10.5	13.1	2.60		28.27	73.51	549.88

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time	11:08 AM	65	44	65.73		8.37		Pump # 1 Time
Pump # _1_ Time	11:09 AM	2	29	2.48	avg fill	8.37	+ 221.43	= 229.79
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				



Pump 1 Average

229.79 gpm

Pump 2 Average

gpm

Inflow Average: **8.37 gpm**

Notes:

Returned on second day to test pump 1.

Pump "on" time is different for P1, 10.5-ft from hatch covers (P2 was 11.6).

Drawdown Site: **Lavatai PS**

Date: 4/11/2012

Time: 2:00 PM

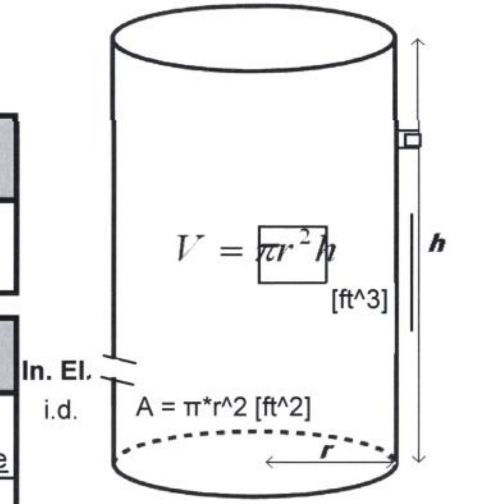
staff: Adriell Moa

dischared pipe: 4-in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 8-in in el. ≈ 8.7-ft from cover	6	11.6	13.1	1.50		28.27	42.41	317.24

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time	2:25pm	30	31	30.52		10.40		Pump # 2 Time
Pump # _2_ Time	2:27pm	2	6	2.10	avg fill	10.40	+ 151.07	= 161.46
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				



Pump 2 Average

161.46 gpm

Inflow Average: **10.40 gpm**

Notes:

Flow was taking un expectedly long.

FM manifolds into 8" FM from Vaitele

Drawdown Site: **Malaloa PS**

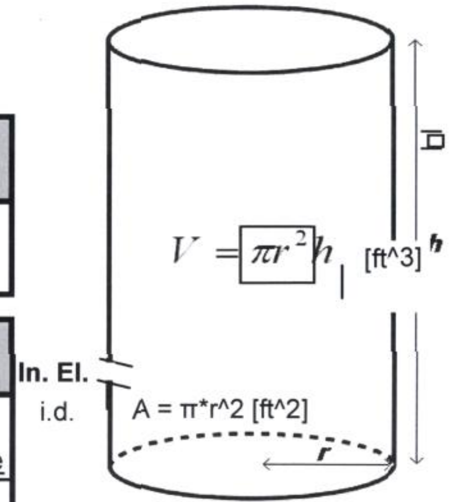
Date: 4/4/2012 | Time: 11:22 AM | staff: Adriell Moa

dischared pipe: -in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 15-in in el. ≈ 12-ft from cover	15	16.3	17.8	1.50		176.71	265.07	1982.74

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time	11:25 AM	4	55	4.92		403.27		Pump # 1 Time
Pump # _1_ Time	11:33 AM	8	36	8.60	avg fill	267.00	230.55	497.55
Fill Time	11:40 AM	15	10	15.17		130.73		Pump # 2 Time
Pump # _2_ Time	11:45 AM	19	43	19.72	avg fill	104.32	100.56	204.88
Fill Time	11:50 AM	25	27	25.45		77.91		Pump # 3 Time
Pump # _3_ Time	11:53 AM	28	21	28.35	avg fill	68.55	69.94	138.48
Fill Time	11:58 AM	33	30	33.50		59.19		Pump # 1 Time
Pump # _1_ Time	12:02 PM	36	51	36.85	avg fill	53.02	53.81	106.83
Fill Time	12:07 PM	42	19	42.32		46.85		Pump # 2 Time
Pump # _2_ Time	12:11 PM	46	15	46.25	avg fill	42.57	42.87	85.44
Fill Time	12:16 PM	51	47	51.78		38.29		Pump # 3 Time
Pump # _3_ Time	12:19 PM	54	47	54.78	avg f	35.93	36.19	72.13
Fill Time	12:24 PM	59	3	59.05		33.58		



Pump 1 Average 302.19 gpm

Even though impellers were changed, it was still hard for the pumps to keep up with the inflow at a low flow time, hence making the pump rates seem lower than they should be.

Pump 2 Average 145.16 gpm

Pump 3 Average 105.31 gpm

Inflow Average: 95.23 gpm

Notes:

Impellers were changed earlier that week, pumps could not previously keep up with the inflow as well. (Still not true p flow)

Drawdown Site: **Matafao (Elementry)**

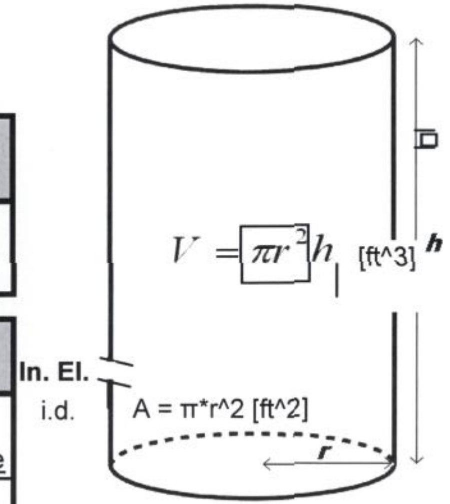
Date: 4/10/2012 | Time: 8:50 AM | staff: Adriell Moa

dischared pipe: -in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 4-in in el. ≈ 4.4-ft from cover	4	4.9	7	2.10		12.57	26.39	197.39

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time				0.00				Pump # 1 Time
Pump # _1_ Time	8:54 AM	5	50	5.83	avg fill	32.72	33.84	66.56
Fill Time	9:01 AM	6	2	6.03		32.72		Pump # 1 Time
Pump # _1_ Time	9:07 AM	5	47	5.78	avg fill	29.97	34.13	64.10
Fill Time	9:14 AM	7	15	7.25		27.23		Pump # 1 Time
Pump # _1_ Time	9:20 AM	5	42	5.70	avg fill	26.95	34.63	61.58
Fill Time	9:27 AM	7	24	7.40		26.67		Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				



Pump 1 Average

64.08 gpm

Inflow Average: **29.83 gpm**

Notes:

Shells in WW. Pump Station is ~60-ft from the ocean.

Only 1 pump at station.

Drawdown Site: **PAPA PS**

Date: 4/12/2012

Time: 9:10 AM

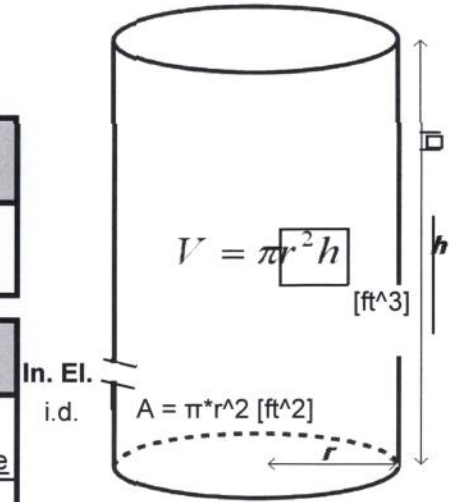
staff: Adriell Moa

dischared pipe: 6-in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 8-in in el. ≈ 17.2-ft from cover	7	18.2	19.2	1.00		36.75	36.75	274.89

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time	9:10 AM	3	53	3.88		70.79		Pump # 2 Time
Pump # _2_ Time	9:17 AM	7	2	7.03	avg fill	98.35	+ 39.08	= 137.43
Fill Time	9:20 AM	2	11	2.18		125.90		Pump # 2 Time
Pump # _2_ Time	9:22 AM	2	41	2.68	avg fill	124.04	+ 102.44	= 226.48
Fill Time	9:25 AM	2	15	2.25		122.17		Pump # 1 Time
Pump # _1_ Time	9:27 AM	2	26	2.43	avg fill	139.63	+ 112.97	= 252.60
Fill Time	9:29 AM	1	45	1.75		157.08		Pump # 1 Time
Pump # _1_ Time	9:38 AM	8	53	8.88	avg fill	149.63	+ 30.94	= 180.58
Fill Time	9:40 AM	1	56	1.93		142.18		Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				



Pump 1 Average

181.96 gpm

Pump 2 Average

203.53 gpm

Inflow Average: **129.20 gpm**

Notes:

Drawdown Site: **Papa AM Flow Test**

Date: 4/12/2012

Time: 8:20AM

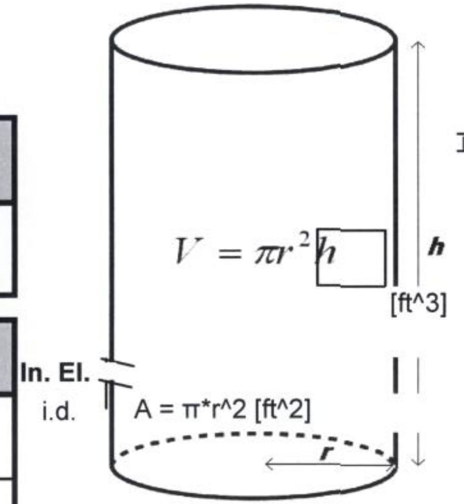
staff: Adriell Moa

dischared pipe: 6-in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 8-in in el. ≈ 17.2-ft from cover	7	18.2	19.2	1.00		36.75	36.75	274.89

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)		Actual Pump Rate (gpm)
Fill Time	8:23 AM	2	14	2.23		123.09			
Pump # _1&2_ Time	8:26 AM	2	13	2.22	avg fill	110.34	+ 124.01	=	234.35
Fill Time	8:28 AM	2	49	2.82		97.59			
Pump # _1&2_ Time	8:31 AM	1	57	1.95	avg fill	181.81	+ 140.97	=	322.78
Fill Time	8:33 AM	2	4	2.07		266.02			
Pump # 1&2 Time; on @ 20.2	8:40 AM	7	52	7.87	avg fill	196.20	+ 69.89	=	266.09
Fill Time	8:45 AM	4	21	4.35		126.39			
Pump # 1&2 Time; on @ 20.2	8:49 AM	3	46	3.77	avg fill	273.57	+ 72.98	=	346.55
Fill Time	8:52 AM	3	16	3.27		420.75			
Pump # 1&2 Time; on @ 20.2	8:59 AM	6	50	6.83	avg fill	282.09	+ 80.46	=	362.54
Fill Time	9:02 AM	3	50	3.83		143.42			
Pump # 1&2 Time; on @ 20.2	9:07 AM	4	3	4.05	avg fill	142.50	+ 135.75	=	278.25
Fill Time	9:10 AM	3	53	3.88		141.57			



Pump 1&2 Average

301.76 gpm

Inflow Average: 200.94 gpm

Notes:

Pumps were not keeping up independently in the AM (they were a little after 9am though).

Decided to change the test levels after quick cycles (doubled the volume).

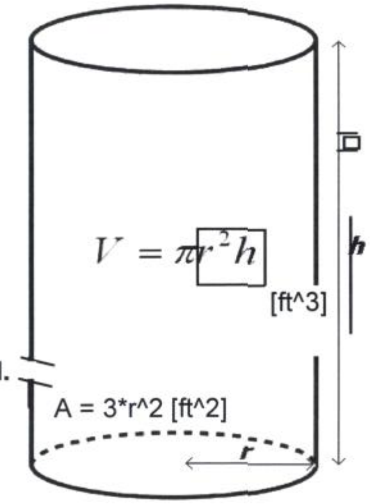
Drawdown Site: **Papa**
Inflow Test Only

Date: 4/9/2012 Time: 2:30pm staff: Adriell Moa

dischared pipe: 6-in Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 8-in in el. ≈ 17.2-ft from cover	7	18.7	19.2	2.00		36.75	73.50	549.78

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time	2:29pm	3	34	3.57		154.14		Pumps 1&2 Time
Pumps 1&2 Time	2:37pm	4	8	4.13	avg fill	146.37	+ 133.01	= 279.38
Fill Time	2:40pm	3	58	3.97		138.60		Pumps 1&2 Time
Pumps 1&2 Time	2:51pm	10	22	10.37	avg fill	143.26	+ 53.03	= 196.29
Fill Time	2:55pm	3	43	3.72		147.92		Pumps 1&2 Time
Pumps 1&2 Time	3:05pm	10		10.00	avg fill	151.40	+ 54.98	= 206.37
Fill Time	3:08pm	3	33	3.55		154.87		Pumps 1&2 Time
Pumps 1&2 Time	3:12pm	4	2	4.03	avg fill	141.86	+ 136.31	= 278.17
Fill Time	3:16pm	4	16	4.27		128.85		Pumps 1&2 Time
Pumps 1&2 Time	3:25pm	8	27	8.45	avg fill	132.86	+ 65.06	= 197.93
Fill Time	3:29pm	4	1	4.02		136.87		Pumps 1&2 Time
Pumps 1&2 Time				0.00	avg fill		+	=
Fill Time				0.00				



Pump 1 Average

227.89 gpm

Pump 2 Average

237.23 gpm

Inflow Average: **146.54 gpm**

Notes:

Pumps were not keeping up with inflow (afternoon) independently.

Used a 2-ft Δh instead of the set points which were 0.5-ft apart.

The wet well is actually a dodecagon (12 sides). A=3r^2

The lowest manhole for Papa's collection system is on the other side of the river.

Drawdown Site: **Sagamea**

Date: 4/11/2012

Time: 9:15am

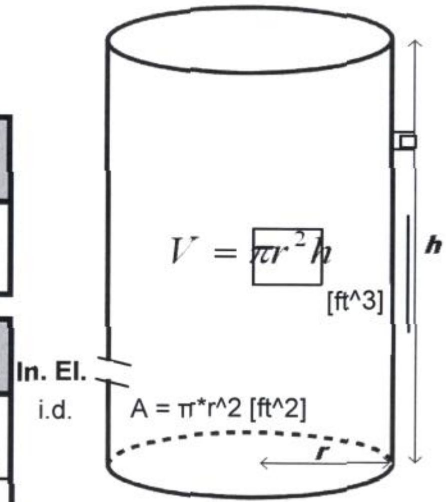
staff: Adriell Moa

dischared pipe:

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 6-in in el.≈ 8.4-ft from cover								

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump (gpm)
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				



Pump 1 Average

gpm

Pump 2 Average

gpm

AM Inflow Average:

gpm

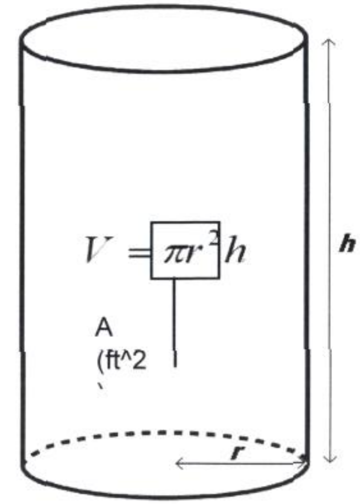
Notes:

Flow is little to none.

Drawdown Test Site: Satala Date: 8/9/11 Time: 7AM staff: Adriell Lino

Diameter of WW (ft)	pump on (in)	pump off (in)	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
5			2		19.63	39.27	293.74

Time	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump (gpm)
Pump # _____ Time			0.00				
Fill Time	3	54	3.90		75.32		
Pump # ___1__ Time	3	3	3.05	avg fill	69.24 +	96.31 =	165.55
Fill Time	4	39	4.65		63.17		
Pump # __1___ Time	4	2	4.03	avg fill	63.40 +	72.83 =	136.23
Fill Time	4	37	4.62		63.63		
Pump # __2___ Time	2	39	2.65	avg fill	62.20 +	110.84 =	173.04
Fill Time	4	50	4.83		60.77		
Pump # __2___ Time	2	45	2.75	avg fill	56.46 +	106.81 =	163.27
Fill Time	5	38	5.63		52.14		



Pump 1 Average

150.89 gpm

Pump 2 Average

168.16 gpm

Notes:

Drawdown Site: **Skills Center**

Date: 4/12/2012

Time: 6:47am

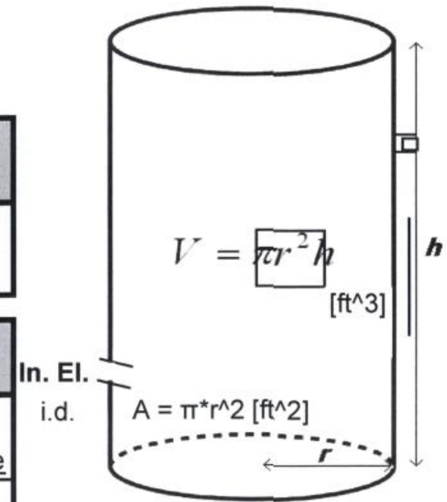
staff: Adriell Moa

dischared pipe:

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 8-in in el. ≈ 11-ft from cover	5	14.1	15.1	1.00		19.63	19.63	146.87

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump (gpm)
Fill Time	6:58 AM	6	52	6.87		21.39		Pump # 2 Time
Pump # _2_ Time	6:59 AM	1		1.00	avg fill	23.69	+ 146.87	= 170.56
Fill Time	7:05 AM	5	39	5.65		25.99		Pump # 1 Time
Pump # _1_ Time	7:06 AM		50	0.83	avg fill	25.37	+ 176.24	= 201.62
Fill Time	7:12 AM	5	56	5.93		24.75		Pump # 2 Time
Pump # _2_ Time	7:13 AM	1	3	1.05	avg fill	24.22	+ 139.88	= 164.10
Fill Time	7:19 AM	6	12	6.20		23.69		Pump # 1 Time
Pump # _1_ Time	7:20 AM		53	0.88	avg fill	21.70	+ 166.27	= 187.97
Fill Time	7:27 AM	7	27	7.45		19.71		Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				



Pump 2 Average

167.33 gpm

Pump 1 Average

194.79 gpm

AM Inflow Average:

23.20 gpm

Notes:

Drawdown Site: **Skills center**

Date: 4/9/2012

Time: 11:13 AM

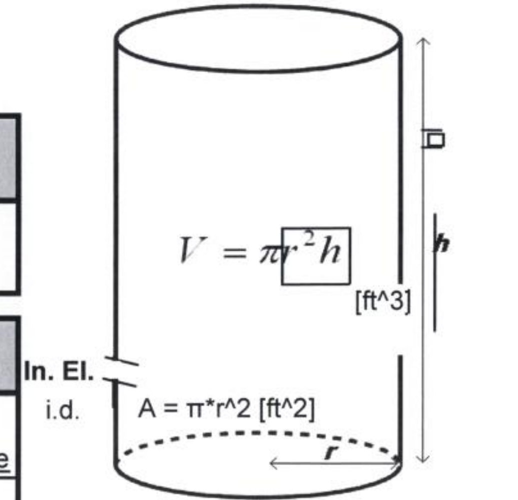
staff: Adriell Moa

dischared pipe: 6-in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 8-in in el. ≈ 11-ft from cover	5	12	14	2.00		19.63	39.27	293.74

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time	11:35 AM	3	19	3.32		88.56		Pump # 1 Time
Pump # _1_ Time	11:41 AM	5	48	5.80	avg fill	83.27	50.64	133.92
Fill Time	11:45 AM	3	46	3.77		77.98		Pump # 2 Time
Pump # _2_ Time	11:53 AM	8	31	8.52	avg fill	78.69	34.49	113.18
Fill Time	11:57 AM	3	42	3.70		79.39		Pump # 1 Time
Pump # _1_ Time	12:02pm	4	35	4.58	avg fill	75.66	64.09	139.75
Fill Time	12:06pm	4	5	4.08		71.94		Pump # 2 Time
Pump # _2_ Time	12:12pm	6	9	6.15	avg fill	66.25	47.76	114.01
Fill Time	12:17pm	4	51	4.85		60.56		Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				



Pump 1 Average

136.83 gpm

Pump 2 Average

113.59 gpm

Inflow Average: **75.06 gpm**

Notes:

Pumps were turning on and off within seconds (sensors may have been blocked).

Manually operated pumps with a Δ of 2-ft

Drawdown Site: **Special Ed**

Date: 4/10/2012 | Time: 8:34 AM | staff: Adriell Moa

dischared pipe: -in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 4-in (two) in el. ≈ 3.9-ft and 5.5-ft from cover	4	6.6	8.7	2.10		12.57	26.39	197.39

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill			
Fill Time				0.00				Pump # Time
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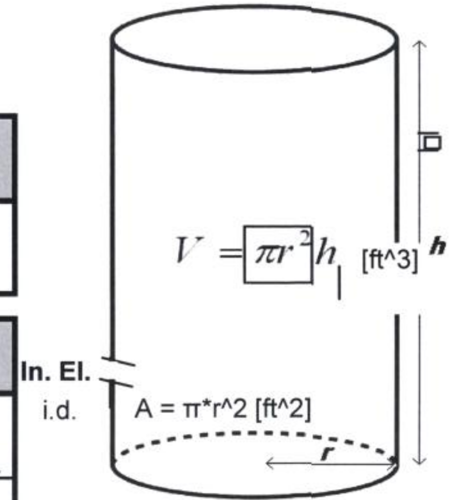
Fill Time				0.00				



Pump 1 Average

gpm

Pump 2 Average

gpm

Inflow Average: gpm

Notes:

Drawdown Site: **Vaitele AM inflow**

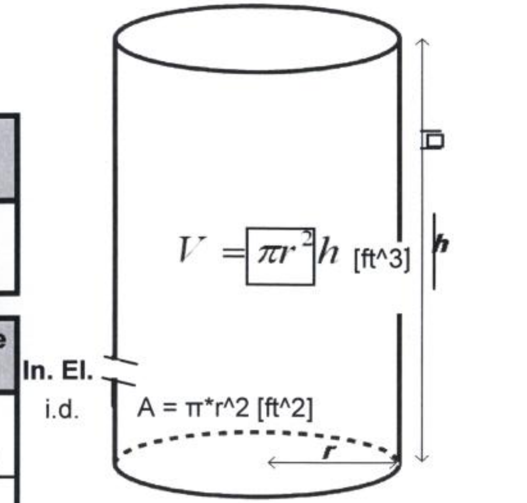
Date: 3/21/2012 | Time: 6:15am | staff: Adriell James

dischared pipe: -in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: -in in el. ≈ -ft from cover	15	28.2	32.5	4.30		176.71	759.87	5683.85

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time				20.87		364.33		Pumps 1&2 Time
Pumps 1&2 Time				51.65	avg fill	392.99	+ 110.05	= 503.03
Fill Time				13.48		421.65		Pumps 1,2&3 Time
Pumps 1,2&3 Time				0.00	avg fill		+	=
Fill Time				12.50		454.71		Pump # Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				



Pumps Average

503.03 gpm

Inflow Average: **413.56 gpm**

Notes:

Had a pressure gauge on pump1, initially 35psi, stable at 30psi once pumps were running.

Drawdown Site: **Vaitele PS**

Date: 3/21/2012

Time: 10:55 AM

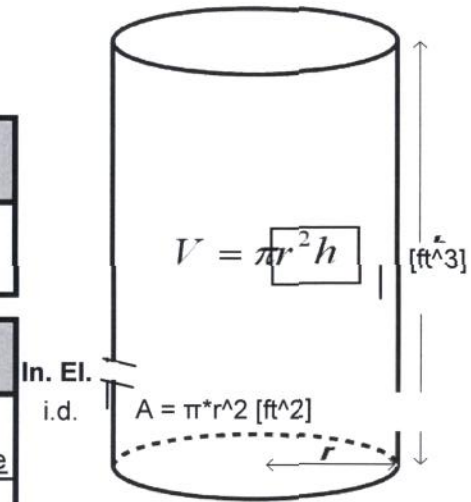
staff: Adriell Lino

dischared pipe: 8-in

Distances measured from hatch cover

Inlet Pipe	Diameter of WW (ft)	pump on el	pump off el	Δ h [off-on] (ft)		Area (ft^2)	Vol pumping (ft^3)	Vol pumping (gal)
i.d.: 15-in in el. ≈ -ft from cover	15	18.2	22.5	4.30		176.71	759.87	5683.85

Status	Time Stamp	Min	Sec	Minutes		Inflow (gpm)	Pump (gpm)	Actual Pump Rate (gpm)
Fill Time		21	38	21.63		262.74		Pump # 1 Time
Pump # _1_ Time		59		59.00	avg fill	228.97	+ 96.34	= 325.31
Fill Time		29	7	29.12		195.21		Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				Pump # __ Time
Pump # __ Time				0.00	avg fill		+	=
Fill Time				0.00				



Pump 1 Average

325.31 gpm

Pump 2 Average

gpm

Inflow Average: **228.97 gpm**

Notes:

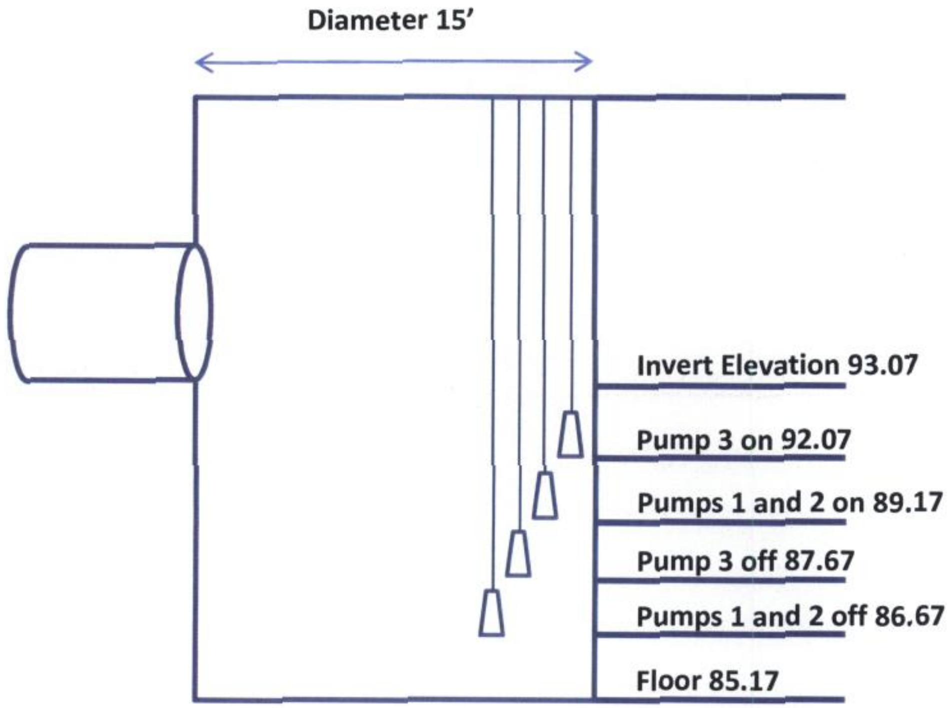
Pressure gauge on P1 was stable at 32psi, peaked at 38 initally.
P2 was 32psi.

Tested pump 1 once (not sufficent), pumps 2 and 3 pump bases were broken, so pumps were not discharging directly into the risers. Also, new check valves were installed on all 3 discharge valves (pumps still were not keeping up with inflow independently).

Appendix I Lift Station Volume Calculations

Wet Well Volume Calculations

Existing Conditions:



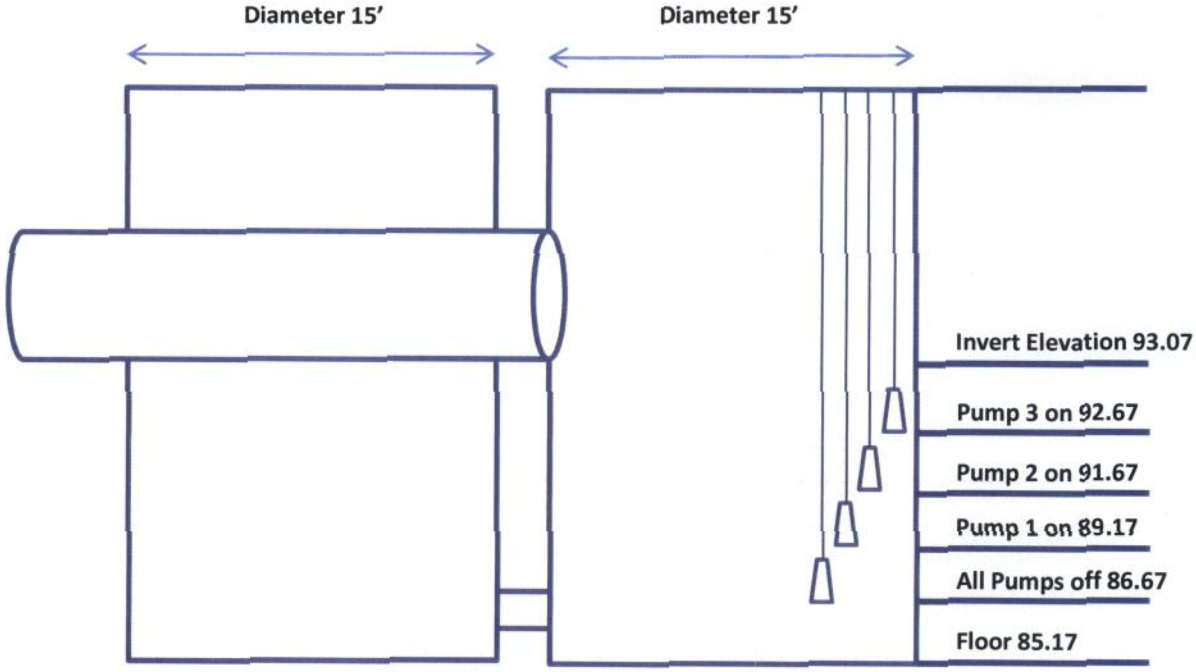
Wet Well Area:

$A = \pi(\frac{15'}{2})^2,$ A=177 sf

Volume Pumps 1 and 2 on to Pump 3 on:

$V = (92.07 - 89.17) * 177$ V=512 cf V=3,833 gal

Proposed Conditions



Wet Well Area, One Manhole:

$A = \pi(\frac{15'}{2})^2$, A=177 sf

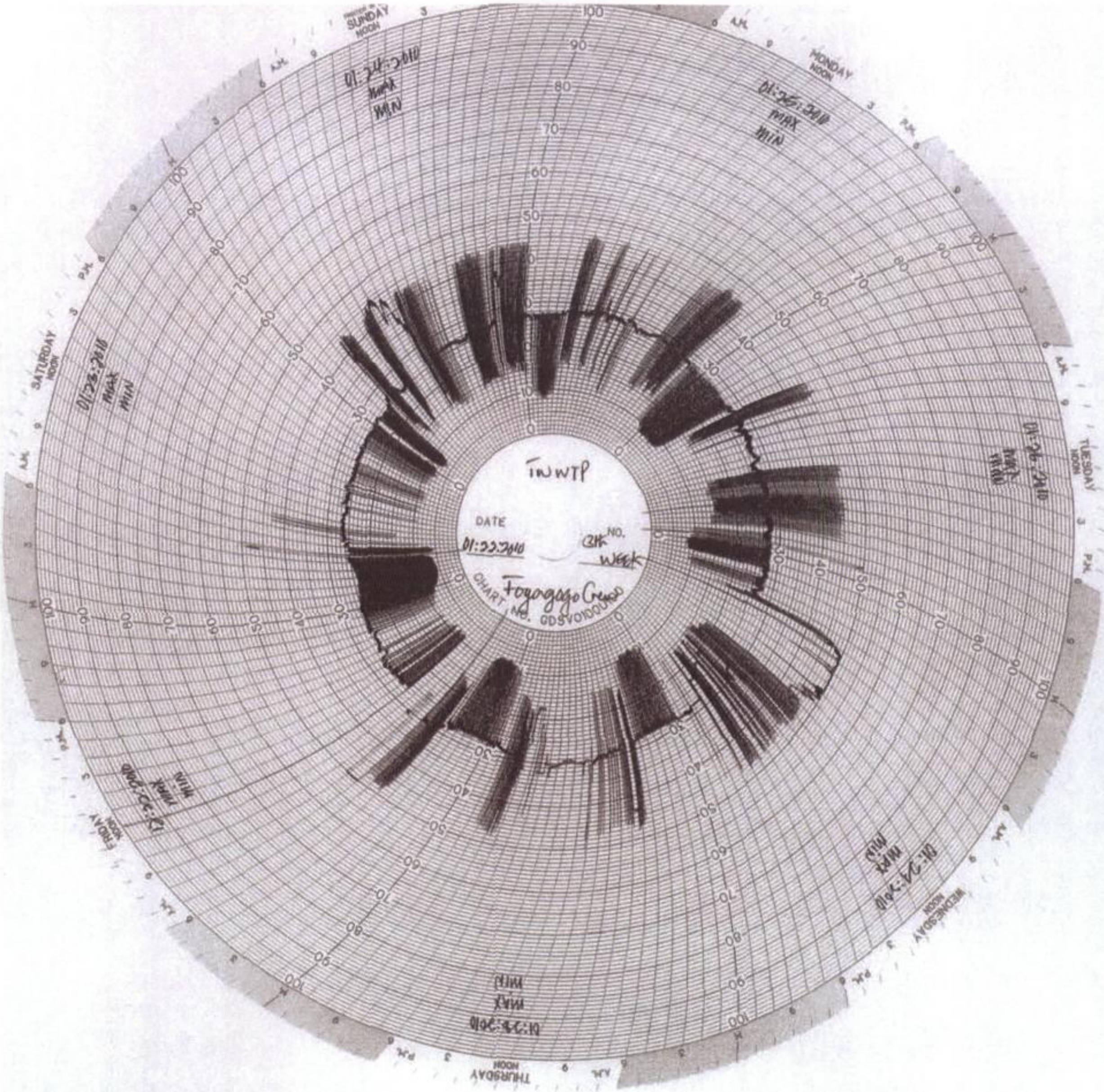
Wet Well Area, Two Manholes:

With two 15' manholes installed next to each other, the Area doubles to A=353sf

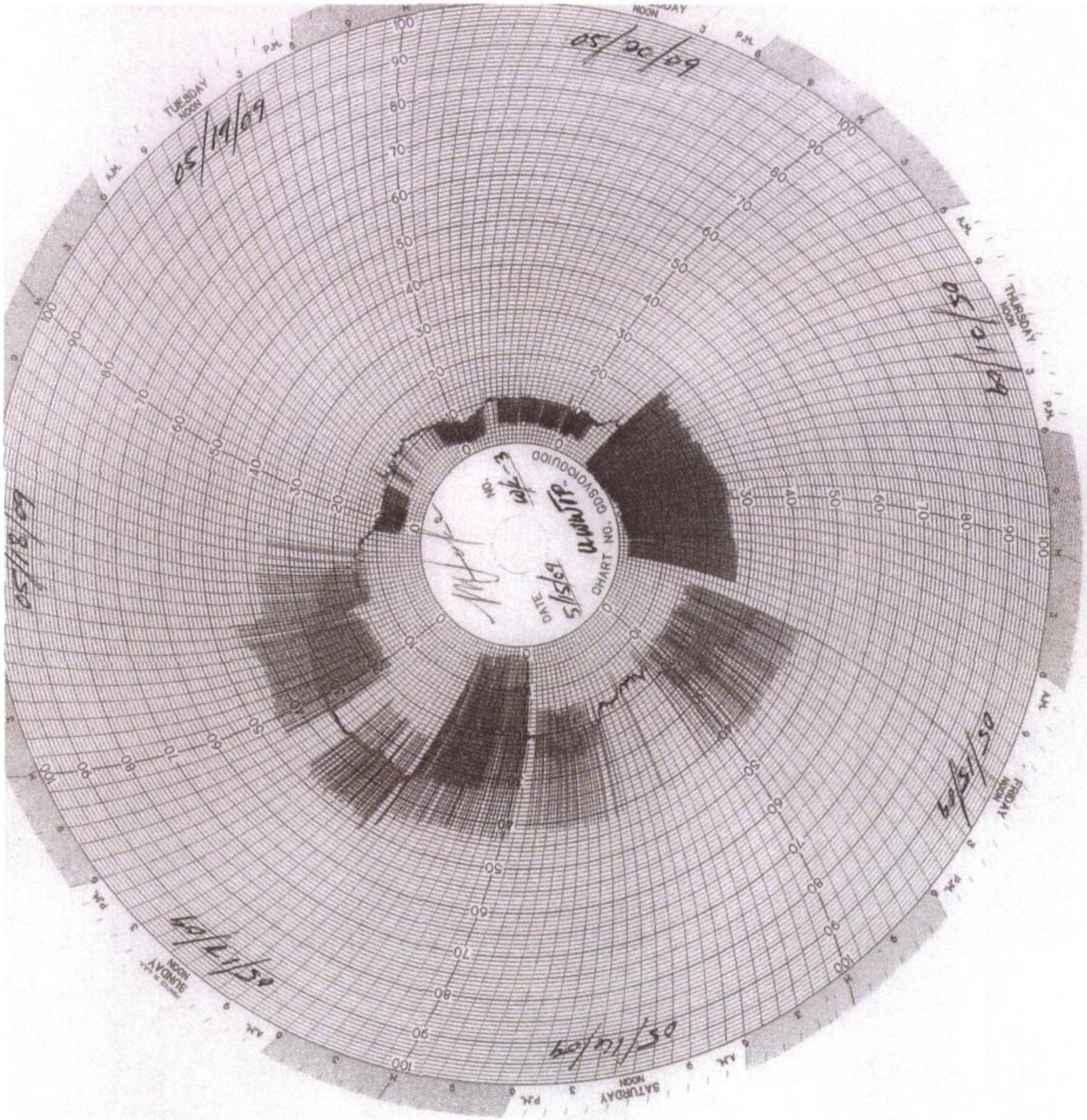
The Volumes are as follows:

	Lower Elevation (ft)	Upper Elevation (ft)	Elevation Difference (ft)	Volume for one Wet Well (cf)	Volume for two Wet Wells (cf)
Pump 1 on to Pump 2 on	89.17	91.67	2.5	443	885
Pump 2 on to Pump 3 on	91.67	92.67	1	177	353

Appendix J Sample Circular Flow Charts



Tafuna
week
of
1/22/10



Utulei
Week w/
5/17/09

Appendix K Equalization Volume Calculation

EPA-600/2-79-096
May 1979

EVALUATION OF FLOW EQUALIZATION IN MUNICIPAL WASTEWATER TREATMENT

by

J. E. Ongerth
Brown and Caldwell, Incorporated
Seattle, Washington 98119

Contract No. 68-03-2512

Project Officer

Francis Evans, III
Wastewater Research Division
Municipal Environmental Research Laboratory
Cincinnati, Ohio 45268

MUNICIPAL ENVIRONMENTAL RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

volume. The peak-to-average BOD concentration ratio was reduced from 1.69 to 1.20, compared to the reduction from 1.69 to 1.51 realized by just meeting requirements for flow equalization. The average load reduction due to BOD decay increased to approximately 6 percent, compared to 3 percent for Example 4.

The average residence time in the larger basin (assuming well mixed conditions) is approximately one half day compared to approximately 4-1/2 hours for Example 4. The large dead volume and longer residence time would warrant serious consideration of aeration and/or mixing equipment to prevent odor problems and solids accumulation. Increased costs for the larger volume, and any additional equipment, should be justified in terms of balancing cost savings in downstream processes; such as reduced peaking capacity in biological processes designed for nitrification and denitrification.

Method 4--Sine Wave Method

In Method 3, stepwise equations were presented, based on conservation of volume of wastewater and on mass balance of constituents. If influent concentration, flow, and outflow are certain very simple functions, then it is not necessary to solve the equations stepwise; a direct solution is possible. Such a direct solution requires little computation and little inlet data, since simple functions are represented by very few numbers.

A direct solution has been obtained assuming that both flow and concentration may be represented by sine waves, in phase with each other and with a period of one day.(15) The flow equation used was of the form:

$$Q_{in}(t) = Q_A - (Q_P - Q_A)\sin 2\pi t \qquad (2\text{-}28)$$

where $Q_{in}(t)$ = influent flow as a function of time

Q_A = average flow

Q_P = peak influent flow

t = time, in days

From such equations, simple and rapid estimates can be made of certain equalization operations. For instance, if a constant outflow is desired, Equation 2-28, minus the constant outflow, may be integrated from t = 0.5 to t = 1.0 day, yielding the working volume of storage:

$$V = \int_{0.5}^{1.0} (Q_{in} - Q_A)\,dt \qquad (2\text{-}29)$$

$$V = \frac{Q_P - Q_A}{\pi} \qquad (2\text{-}30)$$

Equations such as 2-30 are very easy to compute. Concentration and mass loading fluctuations as above were considered,(15) assuming completely mixed basins and first-order decay.

For comparison, Equation 2-30 was applied to the flow variation of Example 2, Method 1. Equation 2-30 yielded a working volume of 0.65 million gallons, whereas 0.79 million gallons were indicated in Example 2. Therefore, Equation 2-30 and similar equations must be used with great caution, because they may give a lower answer.

The limitation of this approach is that municipal wastewater variations do not follow simple sine waves. This was recognized in the reported work because a rough approximation was sufficient for the purpose of the example illustration. Considering that iterative methods are quite workable for diurnal variations, that the sine wave method may give lower answers, and that the sine wave method is applicable only to diurnal variations, it should be considered as approximate and used only to develop rough estimates.

Method 5--Rectangular Wave Method

The rectangular wave method is similar to the sine wave method, except that flows are approximated by rectangular waves. This method has been described (16) and applied assuming the peak-to-average flow ratio to equal the average-to-minimum ratio; which is a rough but reasonable approximation for many sewer systems. With this assumption, and constant outflow, the necessary volume is:

$$V = Q_A \frac{(x-1)^2}{(x^2-1)} \qquad (2\text{-}31)$$

where V = equalization volume, working range

Q_A = average flow

x = peak-to-average flow ratio = average-to-minimum flow ratio

This equation is easily solved. A similar equation, not much more difficult, could be developed without assuming that the peak-to-average ratio equals the average-to-minimum ratio.

Also, the rectangular wave approach could be used for concentration and mass loading.

Generally, the rectangular wave method has similar limitations to the sine wave method. For peak-to-average input ratios less than (π-1) the rectangular wave model gave a somewhat more conservative estimate; for ratios greater than (π-1) the sine wave model becomes rapidly more conservative (see Figure 13). When a rough estimate is required and iteration is impractical, the rectangular wave or sine wave methods may be used to estimate volumes required to equalize daily diurnal variations. Nevertheless, it should be recognized that iteration is more flexible and provides reasonable accuracy.

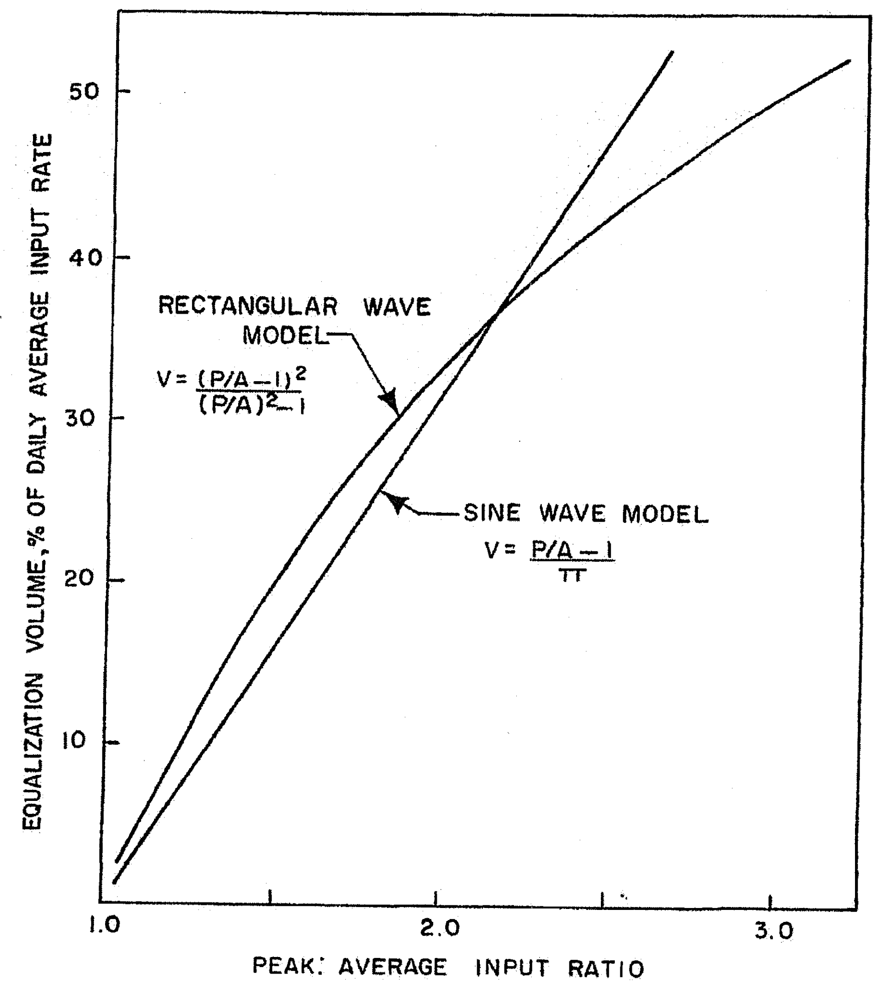


Figure 13. Equalization volume estimation by sine wave and rectangular wave models

Appendix L GDC Technical Memorandum and AUS Diving Reports

TECHNICAL MEMORANDUM **gdc**

Diffuser Configuration and Performance: Scoping Studies for the Tafuna and Utulei WWTPs

Prepared For:	American Samoa Power Authority [1]
Prepared By:	**gdc**
Date:	June 30, 2012

1 Introduction

The American Samoa Power Authority (ASPA) owns and operates the Tafuna and Utulei wastewater treatment plants (WWTPs)[2]. The WWTPs discharge primary treated domestic wastewater into coastal waters on the southwest shore of Tutuila Island (Tafuna WWTP) and into Pago Pago Outer Harbor (Utulei WWTP). The U.S. Environmental Protection Agency (USEPA) issued Administrative Orders (AOs) [3] that require scoping studies to investigate the feasibility of increasing the critical initial dilution (CID)[4] and/or the size of the zone of initial dilution (ZID)[5] for the discharges from the Tafuna and Utulei WWTPs. The required scoping studies include possible projects that would reduce peak flows through the WWTPs, thus increasing the CID and /or ZID, and diffuser configuration changes that would increase CID and/or ZID. This Technical Memorandum (TM) describes the relative changes in CID as a function of effluent flow for several diffuser configurations.

1.1 Purpose

The purpose of the analyses presented in this TM is to describe the relative effects on the CID and the ZID of modifications to the diffusers for both WWTPs. The effects on CID and ZID are described as a function of effluent flow so that the effects of decreasing flows on the CID and ZID can easily be determined. The primary objective is to select the most appropriate changes in diffuser configuration, if any, for increasing the CID and ZID. The

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[1] The results of this Technical Memorandum are intended to be incorporated into Scoping Studies Required by USEPA.

[2] NPDES Permits AS0020010 and AS0020001 for the Tafuna and Utulei WWTPs, respectively.

[3] CWA-309(a)-11-016 and CWA-309(a)-11-017. Issued by USEPA-Region 9 on July 27,2011

[4] The CID is the initial dilution under discharge and ambient conditions resulting in the lowest expected dilution of the effluent plume.

[5] The ZID is defined in various ways. For this TM the ZID is defined by the distance from the discharge point to the edge of the plume at the point where the CID is established.

results presented into this TM are intended to be incorporated into the Scoping Study document being prepared by Coe & Van Loo (CVL) as described in Section 1.2 below.

1.2 Background

The Tafuna and Utulei WWTPs operate under administratively extended NPDES permits. USEPA issued National Pollutant Discharge Elimination System (NPDES) permit No. AS0020010 for the Tafuna Sewage Treatment Plant with an effective date of permit (EDP) of November 2, 1999, which had an expiration date of November 1, 2004. The NPDES permit was administratively extended to be in effect beyond the November 1, 2004 permit expiration date. USEPA issued the NPDES permit No.AS0020001 for the Utulei Sewage Treatment Plant with an EDP of October 9, 2001, which had an expiration date of October 9, 2006. The NPDES permit was administratively extended to be in effect beyond the October 9, 2006 permit expiration date.

The existing NPDES permits provide for less-than-secondary technology-based effluent limitations based on USEPA issued 301(h) waivers from secondary treatment levels, receiving water limitations, sludge limits, and self-monitoring requirements. The permit renewal applications for both facilities included a request to extend the section 301(h) waivers. On January 14, 2009, USEPA issued a Tentative Decision Document denying the 301(h) variance from the secondary treatment requirements in the next NPDES permits for both facilities. A Final Decision Document has not been issued as of the time this TM was prepared.

USEPA determined that the American Samoa Water Quality Standards (ASWQS) are not consistently met at the Tafuna ZID boundaries for total nitrogen, total phosphorus, chlorophyll-a, and *Enterococcus* and that the discharge from the Tafuna WWTP is a likely cause or contributing source of the elevated *Enterococcus* levels in the receiving waters. Exceedances of ASWQS for total nitrogen, total phosphorus, and chlorophyll-a are not clearly a result of the discharge and to some extent reflect natural variability.[6]

USEPA determined that the ASWQS are not consistently met at the Utulei ZID boundaries for total nitrogen, and *Enterococcus* and that the discharge is a likely cause or contributing source of the elevated *Enterococcus* levels in the receiving waters. Exceedances of ASWQS for total nitrogen are not clearly a result of the discharge and to some extent reflect natural variability.

The USEPA issued AOs require certain actions including scoping studies targeted at improving diffuser performance at both facilities.[7] The AOs require that ASPA submit a "*scoping summary of projects that could be taken in order to optimally increase the critical initial dilution factor*" for the Tafuna and Utulei WWTPs ocean outfall discharges:

> "For each project, this scoping summary shall include a description of the project, the resulting estimated mean and peak discharge flow rates (if any), the

[6] The ASWQS are currently under revision, and there may be future revision of the criteria for these parameters that reduce or remove the potential for future exceedances.

[7] The AOs also require the design and implementation of disinfection of the effluent streams and supplementary monitoring of the effluent for specific parameters.

estimated capital cost of the project, and a construction schedule not to extend beyond June 30, 2013. At a minimum, this scoping summary shall cover the following projects:

a. A reduction in the expected daily-maximum mean and peak discharge flow rates through infiltration and inflow upgrades to the sewer system;

b. A reduction in the expected daily-maximum mean and peak discharge flow rates through increases in sewer system storage capacities and optimized delivery;

c. A reduction in the expected daily-maximum mean and peak discharge flow rates through the installation and operation of on-site wet-weather storage;

d. A doubling of the diffuser length of the Utulei ocean outfall;

e. Any other project to increase the size of the zone of initial dilution."

The analyses in this TM are intended to provide predictions of CID and ZID to be used in assessing the efficacy of items a through c above being addressed by CVL. The analyses include the evaluation of performance of the diffusers for a range of flow conditions for a number of diffuser configurations (items d and e). The results of diffuser performance analyses will be included for potential projects required by the AO in the Scoping Studies being prepared by CVL

1.3 Approach

The approach used in this TM addresses the changes in diffuser configuration that can be considered to improve diffuser performance. As required in the AOs, extension of the diffuser length is addressed. Changes in the diffuser configuration of the existing diffusers are addressed based on modifying the number and size of diffuser ports. The standard modeling approach is addressed to better define the expected dilution performance of the diffusers; results of both the standard and modified modeling approaches are presented. For each diffuser configuration considered, including the existing diffusers, the dilution and size of the ZID are determined as a function of effluent flow. Using these results the effect of managing peak flows on CID and ZID can be determined and applied in the evaluation of projects required in the AO being addressed by CVL, as well as other projects that may be identified as a result of the scoping studies.

1.3.1 Doubling of the Diffuser Length

Increasing the diffuser length has the potential to increase both the CID and the ZID. This option, although it appears attractive, must be considered in terms of overall diffuser operation and hydraulics. Care must be taken not to reduce velocities in the diffuser barrel to a point where sedimentation in the barrel becomes likely. In addition, care must be taken not to produce flows that could lead to persistent seawater intrusion and recirculation at transient low flows that cannot be overcome by normal flows. Analyses for this approach for both outfalls is based on the application of accepted hydraulic and dilution models.[8]

1.3.2 Other Projects to Increase the CID

There are other approaches that can be used to increase either the CID or the establishment of a mixing zone (including the ZID) as discussed in Section 3.4 below. These include reconfiguration of the port size and port configuration of the existing diffusers. This approach is considered in this TM and appropriate reconfigurations of both diffusers are analyzed.

1.3.3 Modeling Considerations

Re-evaluation of the existing CID was conducted based on more extensive and recent ambient data. Recent ambient density profiles data were reviewed and modified. The effect of ambient current was also considered. Recent effluent data for nitrogen and phosphorus were reviewed to determine the effectiveness of the various approaches to reduce the expected concentrations to acceptable levels.

The modeling was done using two approaches for each diffuser configuration considered:

- The "standard" approach for ports with alternating direction, which assumes that all ports are discharging in the same direction. The model, as is the case for all initial dilution models, does not simulate port discharge direction opposed to the current. This assumption predicts merging of plumes from individual ports much sooner than is actually the case and substantially under predicts actual dilution because lateral dilution (mixing) is suppressed beyond the predicted point of merging.
- A "realistic merging" modeling approach that models the ports along the diffuser barrel in two groups, essentially doubling the port spacing. This results in higher dilution by delaying the point of merging. The model still provides conservative (under predicted dilution) because vertical separation of the plumes is not modeled and merging is still predicted somewhat sooner than actually occurs.

In addition, the dilution predictions for both modeling approaches are presented for dilution predicted at the trapping level of the plume (point where the plume and ambient density are equal) and the point of maximum rise of the plume. As the plume rises in the water column it continues past the trapping level because of the buoyancy induced momentum of the plume. The model used predicts the dilution up to the point of maximum rise, which is considered a better estimate of the overall dilution performance of the various diffuser configurations.

1.4 Scope and Limitations

This TM provides a scoping level (feasibility, planning level) consideration of a number of factors that could be addressed to increase the CID and the ZID, including revised diffuser configurations and better model input definition. The range of potential approaches is not

[8] The use of check values on diffuser ports is sometimes used as a method to increase dilution under higher flows and avoid seawater intrusion at low flows. This approach is not explicitly considered in the TM because of the potential problems with maintence of these valves in American Samoa. Relocation of the diffusers to deeper water was not explicitly considered because the reconstruction or major extension of the outfalls is outside the scope of this TM and the requirements of the AOs.

exhaustive and is limited to examples of those that appear most promising and feasible for the existing discharges and under a strict schedule imposed by the AOs. The evaluations are based on existing data; no new data were collected for the scoping studies. Data considered are purposely limited to recent data collected over the last two to five years. This TM does not provide a final design or a definitive design basis for whatever alternative is ultimately selected, but is intended to provide sufficient guidance to select the most appropriate alternative and provides guidance for developing the design of the alternative selected, if appropriate.

2 Existing Diffuser Descriptions

The outfalls and diffusers for the two WWTPs have undergone modifications and repairs in the past. A number of diffuser configurations have been reported and discussed in various documents. This TM addresses only the most recently documented diffuser configurations, based on detailed inspections conducted by Associated Underwater Services (AUS) in December 2011. Based on the current configurations of the diffusers, a *base case* diffuser configuration for each WWTP was selected and used for baseline dilution modeling. The following diffuser descriptions are taken directly from the AUS inspections with minor modifications. The figures showing the diffuser locations used in this section are from the routine receiving water monitoring reports submitted semi-annually to USEPA.

2.1 Tafuna WWTP Outfall and Diffuser

The Fogagogo (Tafuna) WWTP outfall is a 24-inch diameter HDPE pipeline that extends approximately 1,600 feet from the treatment plant and is anchored to the ocean floor with a steel anchorage system attached to concrete weights placed on coral reef, sand and rock. The outfall is located on the south side of Tutuila Island adjacent the Pago Pago International Airport (Figure 2-1). The outfall terminates at a depth of approximately 95 feet with effluent released through six, eight-inch diameter diffusers spaced over a length of 50 feet (based on drawings submitted with the inspection report. The outfall replaced a 12-inch diameter iron pipeline that remains within close proximity to the replacement outfall.

The diffuser consists of the blind flange on the end of the diffuser barrel, 6 gooseneck diffuser ports and is connected to the outfall pipeline by a flange. The total length of the diffuser section is 61 feet, the diffuser ports span a distance of 50 feet. At the time of the AUS inspection, the blind flange was partially buried in the sandy bottom. Each gooseneck riser extends directly up from the top of the diffuser barrel 28" high with a spacing of 10 feet between risers. The risers have 8" diameter ports discharging horizontally and orientated in alternating east-west direction, perpendicular to the diffuser barrel. The depth of each diffuser was measured during the AUS inspection. The six ports were observed to have strong and equal discharge. The offshore end of the outfall is in 96 feet of water. The port depths range from 87 feet to 94 feet. AUS inspected the seafloor 100 feet out from the end of the outfall to a depth of 106 feet and the seafloor consisted of a flat sandy bottom with no obstruction beyond 100 ft (visibility at the time of the inspection was 80 feet).

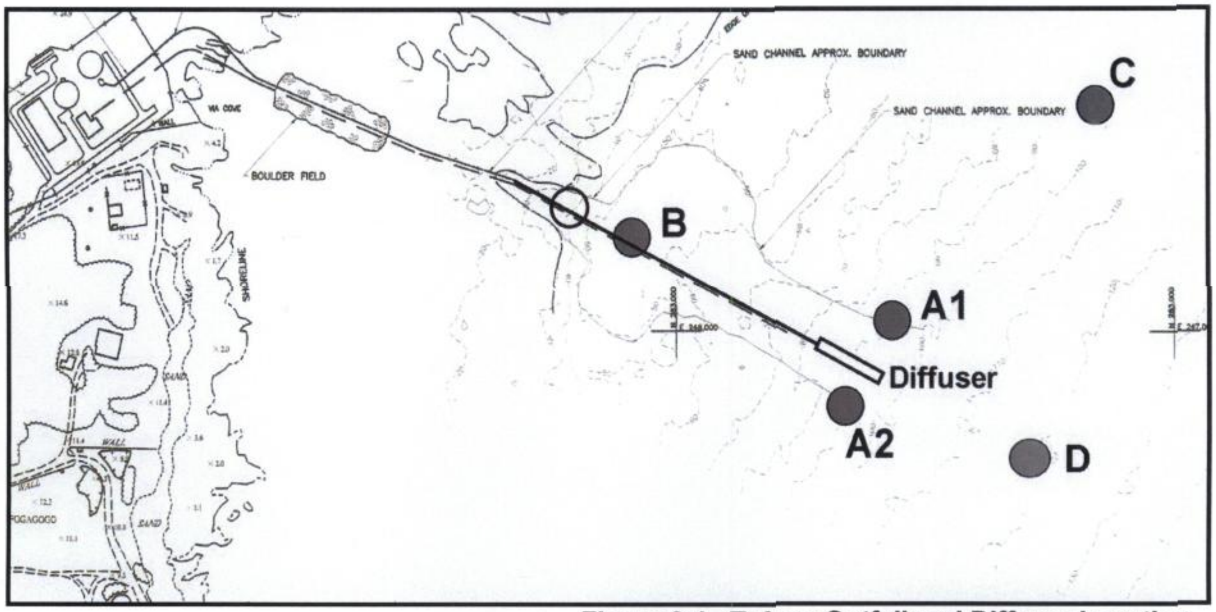

Figure 2-1. Tafuna Outfall and Diffuser Location
(The shaded circles are the current sampling points used for receiving water monitoring.)

2.2 Utulei WWTP Diffuser

The Utulei WWTP outfall discharges to Outer Pago Pago harbor (Figure 2-2). The outfall is a 24-inch diameter ductile iron and high density polyethylene (HDPE) pipeline extending approximately 1,050 feet from the treatment plant with approximately 700 feet buried in the coral reef flat and the exposed offshore portion descending the reef slope. The inshore section of the outfall is ductile iron and the offshore section is HDPE. The outfall is secured to the reef by a unique gimbaled frame that holds the outfall in place on the top of the reef slope. The outfall drops off the reef flat at a steep angle and touches the Harbor bottom 65 feet below the gimbaled frame. Effluent is released through five, six-inch diameter ports and one four-inch port spaced over a length of 35 feet (based on drawings submitted with the AUS inspection report).

There are six gooseneck risers on the diffuser section of the outfall that rises directly from the top of the diffuser barrel. The ports on the gooseneck risers discharge horizontally in alternate directions perpendicular to the diffuser barrel. Three of the gooseneck risers were replaced in 2007 with ROMAC repair gooseneck risers. These repaired risers have 6-inch diameter openings and are 16" tall. The remaining 3 gooseneck risers are original and are 27" tall. Two of the risers have 6-inch diameter ports and port number 2 has a restrictor plate over the opening which reduces the 6-inch port opening to 4 inches in diameter. The depth for each diffuser port was measured from the top of the diffuser. The depths for each port are shown on the drawing submitted with the AUS inspection report and vary from 146 feet to 154 feet. The ROMAC repair gooseneck risers are showing signs of moderate corrosion with one having a small leak at its base. In general, the flows were strong and equal from all ports, although debris was observed hindering flow from at least one of the ports. The need to clean the diffuser is noted in Section 7. The diffuser terminates with a blind flange that is partially buried at a depth of approximately 160 feet in a sandy/muddy bottom. The AUS inspection included the seafloor 100 feet beyond the end of the outfall to a depth of 177 feet. Soft sand was encountered with no large coral heads or boulders.

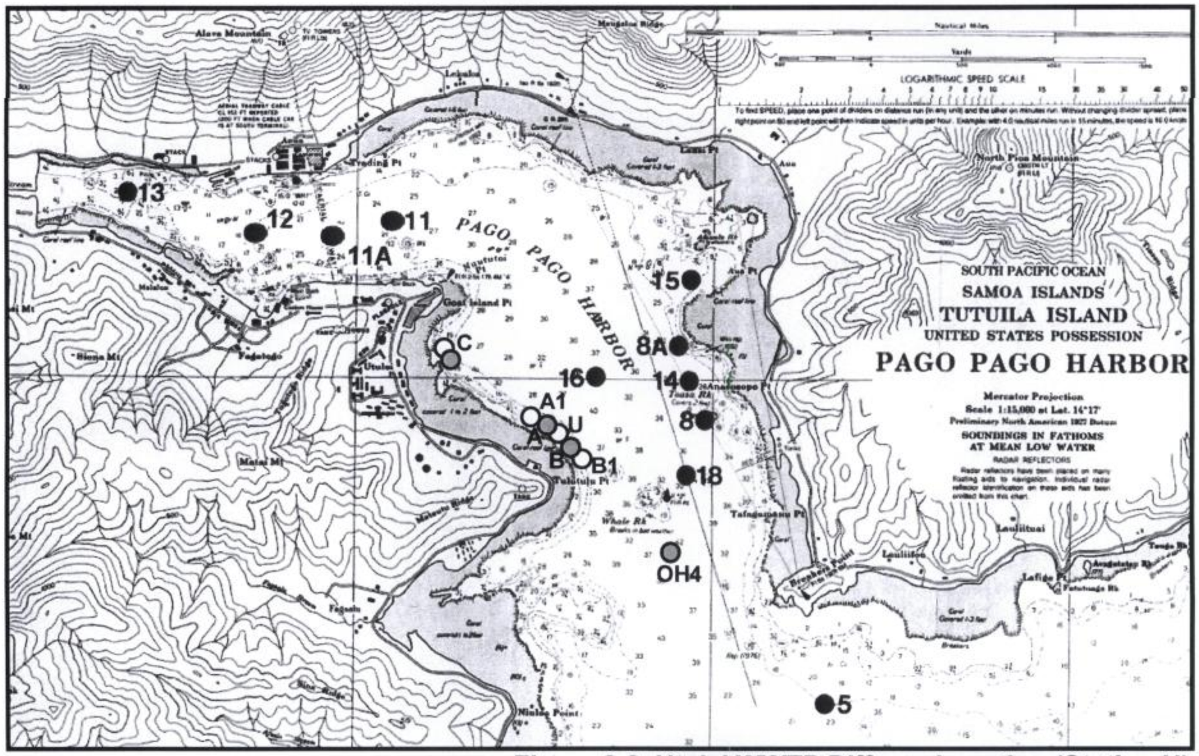

Figure 2-2. Utulei WWTP Diffuser Location (Station U)
(The circles are sampling points used for receiving water monitoring either currently or in the past, throughout Pago Pago Harbor; Stations C, A1, B1, U, 16, and 18, are used for the current routine Utulei receiving water monitoring.)

2.3 Selected Base Case Diffuser Configurations

Based on the AUS inspections the base case diffuser configurations are described in Table 2-1. The dilution and plume geometry for these base case configurations were used to evaluate the effectiveness of changes in diffuser configurations.

Table 2-1. Base-case Diffuser Configuration

Parameter	Tafuna WWTP Diffuser	Utulei WWTP Diffuser
Diffuser Barrel Diameter - inches (meters)	24 (0.609)	24 (0.609)
Riser Diameter - inches (meters)	8 (0.2032)	6 (0.1524)
Riser Length - inches (meters)	28 (0.711)	16 (0.406) [1]
Number of Ports	6	6
Port Discharge Angle (Vertical Angle)	0° (Horizontal)	0° (Horizontal)
Port Discharge Angle (Horizontal Relative to Diffuser Barrel)	±90° (Perpendicular in alternating directions)	±90° (Perpendicular in alternating directions)
Port Spacing - feet (meters)	10 (3.05)	7.1 (2.16)
Port Diameter - inches (meters)	8 (0.203)	6 (0.152) [2]
Average Port Depth - feet (meters)	89.2 (27.19)	150.4 (45.84)

[1] Three risers are 16 inches long and three are 27 inches long. It is assumed, for this TM, that the longer risers will eventually be replaced with shorter ones. The effect on the hydraulics and dilution caused by the longer risers is insignificant.
[2] One of the ports is 4 inches in diameter because an orifice plate is mounted on the riser exit. It is assumed that this orifice plate will be removed.

3 Principles of Diffuser Operation and Dilution Performance

This section briefly describes the parameters that are used for input to the dilution model and their relative importance. The base case values of the parameters, in addition to those tabulated in Section 2 (Table 2-1), are described. The modeling for the base case will provide a baseline from which to evaluate the effects of changes in the diffuser configurations considered. The model used for this TM is the USEPA developed model UDKHDEN[9]. Although other models are available, UDKHDEN is well suited for marine discharges and accounts for ambient density variations. Recent dye studies of six marine discharges in Puerto Rico[10] have shown that this model is quite conservative; typically under predicting the actual dilution by a factor of two or more (the average result for the six discharges considered was a factor of between five and six). The results of dye studies conducted for the joint cannery outfall in Pago Pago Harbor are consistent with these results and indicated actual dilutions substantially higher than predicted by the model.

The design of diffuser configurations is discussed in terms of practical limitations on diffuser design. One of the diffuser modifications investigated in this TM is to lengthen the diffuser, with additional ports. Although the model may predict higher dilutions for such a configuration, it will not directly determine if seawater intrusion and recirculation is a potential problem. The result is that such model predictions may not be accurate under realistic conditions with variable effluent flow. Therefore, a brief discussion of this phenomenon is included in this section of the TM and used to evaluate the potential for seawater intrusion.

3.1 Design Constraints

The design of high-rate (high port velocity and/or high dilution performance) multiport diffusers must account for a number of practical constraints that limit the physical configuration of the diffuser. As a general approach dilution can be maximized by using many ports of small diameter with large spacing. However, there are limitations on the design of such multiport diffusers. These include:

- Minimum velocities in the diffuser barrel should be sufficient, under expected flow regimes, to prevent sedimentation in the diffuser barrel. This typically requires flows in the diffuser to be sufficient to maintain effluent derived sediments in suspension or to be sufficient to resuspend sediments in the diffuser under higher flows. For primary treated effluent a diffuser velocity of 1 ft/sec to 2 ft/s during the expected higher flows is often used as a design basis. The "rule-of-thumb" for initial design considerations is that the total port area along the diffuser barrel should be no more than about 50% of the cross sectional area of the diffuser barrel. For larger systems the diameter of the diffuser barrel can be decreased along the length of the diffuser. A common approach to maximize port area and dilution performance is to install a port on the end-gate of the diffuser; this maintains sufficient velocities in the diffuser barrel while increasing the total port area available.

[9] W.P. Muellenhoff et al., 1985. *Initial Mixing Characteristics of Municipal Ocean Discharges. Volume I. Procedures and Applications.* U.S. Environmental Protection Agency, EPA/600/3-85/073a.

[10] S.L. Costa, R.W. Darby, and M.D. Hernandez, 2012. *Dilution Model Validation for Puerto Rico Ocean Mixing Zones.* Puerto Rico Water Environment Association, Wastewater and Environmental Seminar, May 11, 2012.

- Optimum dilution performance is generally obtained when the discharge through the diffuser ports is balanced (nearly equal flows through each port). For longer diffusers with high-volume discharges, this is typically addressed by varying port diameters along the diffuser (larger ports towards the seaward end of the diffuser). For short, low volume diffusers this is generally not required. Diffuser hydraulics must be considered to select appropriate variations in port sizes.
- The size of the individual ports should be sufficient to eliminate the potential for blockage of the port by debris that might inadvertently be discharged though the outfall. The depth and location of the diffuser and the associated difficulty of maintenance operations is a primary consideration in limiting the minimum size of the diffuser ports. For a remote location, such as American Samoa, with limited facilities and technical support a minimum port opening of about 5-inch to 6-inch diameter ports is prudent.
- The physical characteristics of the diffuser location often limit the length of the diffuser and the spacing of the individual ports. The length of the diffuser is also a major consideration in the cost of construction. This is often addressed by installing multiple ports on a single riser.
- Flows must be adequate to maintain discharge sufficient to avoid seawater intrusion into the seaward end of the diffuser. This is discussed in more detail in Section 3.5 below.
- The primary head loss in an outfall system with a multiport high-rate diffuser is associated with the port discharge. For relatively short outfalls the head loss associated with the port discharge comprises most of the total system head loss. The diffuser design must account for, and not exceed, the available hydraulic head under conditions of maximum peak discharge.

3.2 Parameters of Importance

In addition to the diffuser configuration parameters shown in Table 2-1, parameters based on the effluent and ambient receiving water characteristics are required to predict the dilution performance of the diffuser. These are discussed below and the selected base line condition is described. Although the selected conditions may vary based on additional data, the relative effects on diffuser performance of the configurations considered in this TM are expected to remain the same.

3.2.1 Effluent Flow

Effluent flow, with all other parameters held constant, is a primary variable that has a substantial effect on diffuser performance. An evaluation to reduce and better manage peak flows for both WWTPs is being conducted separately under the AOs (see Section 1.2). The modeling presented in this TM considers a range of flows for each of the diffuser configurations considered from 1 mgd to 10 mgd. Results are presented for this range of flows allowing results of the flow management studies to be evaluated with respect to diffuser performance and incorporated in evaluating the efficacy of projects to reduce peak WWTP flows as required in the AO.

3.2.2 Effluent Density

Effluent density is an input variable for the dilution models because the difference in density between the lighter effluent and the denser seawater affects the momentum of the plume as it rises through the receiving water column thus controlling the momentum and relative speed (and shear) that drives the mixing process between the effluent plume and the receiving water. Effluent density does not vary much and a representative effluent density is usually sufficient for dilution performance evaluations. Effluent density is a function of effluent temperature and salinity. The effluent temperature varies only slightly and is typically close to the ambient air temperature. Past modeling for the diffusers being considered has used the following:

- Effluent temperature is taken as 29 °C for both discharges
- Effluent salinity (more correctly TDS) is taken as 0.50 ppm for both discharges

This results in an effluent density of about 0.99638 g/cm^3, which is used as the base case input for the dilution model[11].

3.2.3 Ambient Current Speed and Direction

Ambient current speed has a substantial effect on dilution. Previous modeling for the Utulei WWTP and the Tafuna WWTP diffusers has assumed a zero ambient current. This is considered overly conservative (results in lower predicted dilutions).A more realistic current speed is used in this TM. Selected current speeds and the rationale for using them are as follows:

- There is no available recent current data for the location of the Tafuna WWTP diffuser[12]. The diffuser is located on an open coastal setting and currents, primarily driven by tides, wind, and regional oceanic current patterns, are seldom if ever zero. USEPA's 301(h) Technical Support Document [301(h) TSD][13] suggests that open coastal average currents should be taken as 5 cm/sec in the absence of site-specific data. The 301(h) TSD also recommends the use of the 10th percentile current speed for assessing the CID. Based on this guidance and experience with open coastal discharges, a critical velocity of 2 cm/sec is selected as the base case current speed for the Tafuna WWTP diffuser location. It is noted that this is probably lower than the actual current speed, but will result in conservative (lower than expected) dilution predictions.
- There is very little current information for the Utulei WWTP diffuser site. The most recent data are from dye studies conducted in Outer Pago Pago Harbor in 1993[14] and 1994[15]. The results of the limited data from these studies support the use of a 2 cm/sec 10 percentile current speed as a conservative assumption at the depth and location of

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[11] Recent, limited, data has indicated that effluent densities may be somewhat higher than those used in this analysis. This may somewhat depress the predicted dilutions. However, the *relative* effects on dilution for all cases considered will be similar and are appropriate for application to the ongoing Scoping Studies.

[12] There are some reported drogue releases that are not directly useful for assessing current speeds in the area, but are generally consistent with the assumptions used in this TM.

[13] U.S. Environmental Protection Agency, 1994. *Amended Section 301(h) Technical Support Document.* EPA 842-B-94-007.

[14] CH2M HILL, 1993. Joint Cannery Outfall Dye Study Report: Non-tradewind Season. July, 1993.

[15] CH2M HILL, 1994. Joint Cannery Outfall Dye Study Report: Tradewind Season. July, 1993.

the Utulei WWTP diffuser. This is used as the base case current speed for the Utulei diffuser in this TM.

Current direction has a smaller effect on dilution than current speed, except for diffuser ports that are very close together. In both cases considered here the prevailing currents are bathymetrically controlled and therefore generally perpendicular to the diffuser barrels. Currents perpendicular to the diffuser barrel are used as the base case condition for the modeling presented in this TM. It is noted that none of the readily available dilution models, including UDKHDEN, can simulate currents opposed to the direction of port discharge. The accepted practice is to assume the port discharge is in the direction of the current in such cases, which explains some the observed conservatism in the model predictions compared to measured predictions mentioned in the introduction to this section.

3.2.4 Ambient Density Structure

The ambient vertical density profile has a substantial effect on dilution performance and plume behavior. A variety of assumptions have been used in past dilution modeling for the Tafuna and Utulei diffusers, which accounts largely for variations in reported model results. In particular the density profile for the Utulei diffuser used in the most recent application for a 301(h) waiver from secondary standards was problematic. The profile used was from the location of the diffuser (Station U in Figure 2-2) and exhibited anomalies that are likely the result of interaction of the local water column with the plume. The ambient density profiles should be from a background location that represents a water mass that is involved in, but unaffected by, the mixing process. That is, the profiles should be close to the discharge but far enough away to not be directly influenced by the discharge plume.

The ongoing receiving water monitoring for both Tafuna and Utulei WWTPs does not require reporting of vertical profiles. However, there are data available for these locations from the Joint Cannery and ASPA receiving water quality monitoring studies in Pago Pago Harbor and in some cases data taken during the Tafuna and Utulei monitoring that was not required and may not have been reported.

For the Tafuna WWTP diffuser there are some data from Station C (see Figure 2-1), which would be a reasonable background station for density profiles because it is not likely affected by the plume and there is a substantial amount of data from Station 5 (see Figure 2-2) that can be, and has been, used as representative for the Tafuna open coastal location. It is noted that data from Station 5 should be used with some caution for the Tafuna discharge since it is influenced by the water masses within Pago Pago Harbor.

For the Utulei WWTP diffuser location there is periodic data (twice a year) from Station 16 from the Joint Cannery monitoring studies. This station is considered representative of background conditions for the Utulei location. There are also occasional data from Stations A1 and B1 and Station 18 (see Figure 2-2). Data from Station 5 could also be used, but is not as representative as the other stations.

Seawater density is a function of temperature, salinity, and depth. At the depths of the diffusers considered here the effect of pressure (depth) on density is minor and can be ignored. The initial dilution model can accept inputs of temperature and salinity at specified depths and calculate the appropriate density profile data, or density (calculated independently) can be entered into the model directly.

The most recent density data available was collected in March 2012. Temperature and salinity were recorded continuously through the water column at Station C (applicable for the Tafuna diffuser modeling and at Station 16 (applicable for the Utulei diffuser modeling). These profiles are used as the base case profiles in this TM. The profiles for Station C (Tafuna) are shown in Figure 3-1 and the profiles for Station 16 (Utulei) are shown in Figure 3-2. Monitoring in the vicinity of both outfalls is typically conducted in the tradewind and non-tradewind seasons. Non-tradewind density profiles will typically show more stratification, and result in lower predicted dilutions. The March 2012 non-tradewind monitoring occurred during the most intense non-tradewind conditions in recent years based on observed winds recorded during the data collection event and is quite likely representative of the most critical density profile on record.

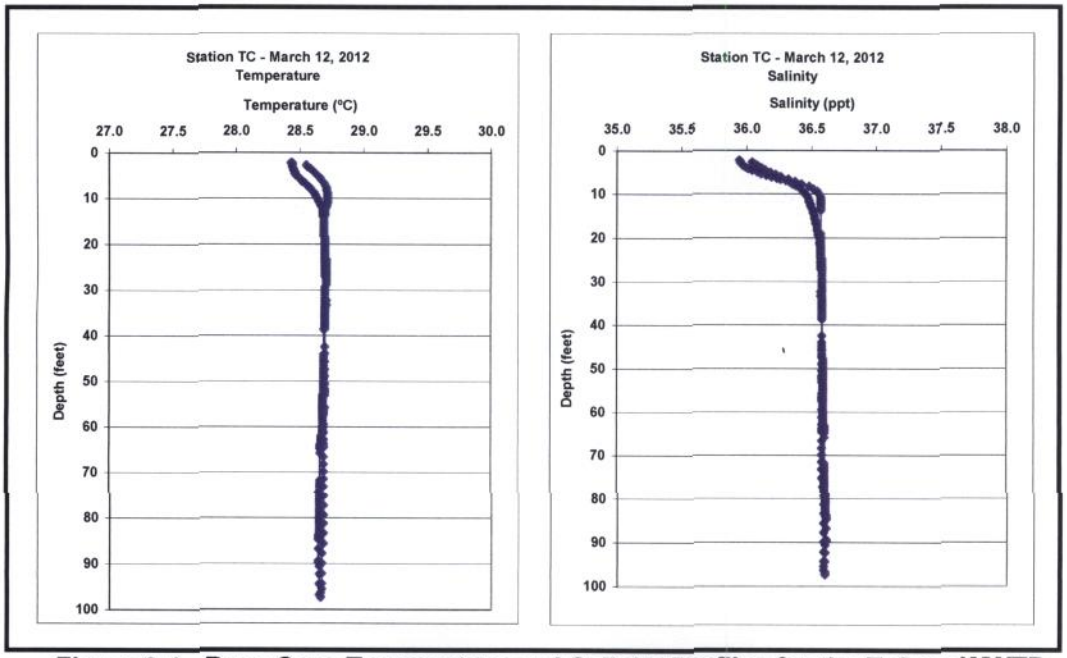

Figure 3-1. Base Case Temperature and Salinity Profiles for the Tafuna WWTP
(The two traces represent downcast and upcast data, which are averaged to develop the density profile used in the modeling. Note that Station TC is Station C in Figure 2-1; "T" is used to avoid confusion with Utulei Station C in Pago Pago Harbor.)

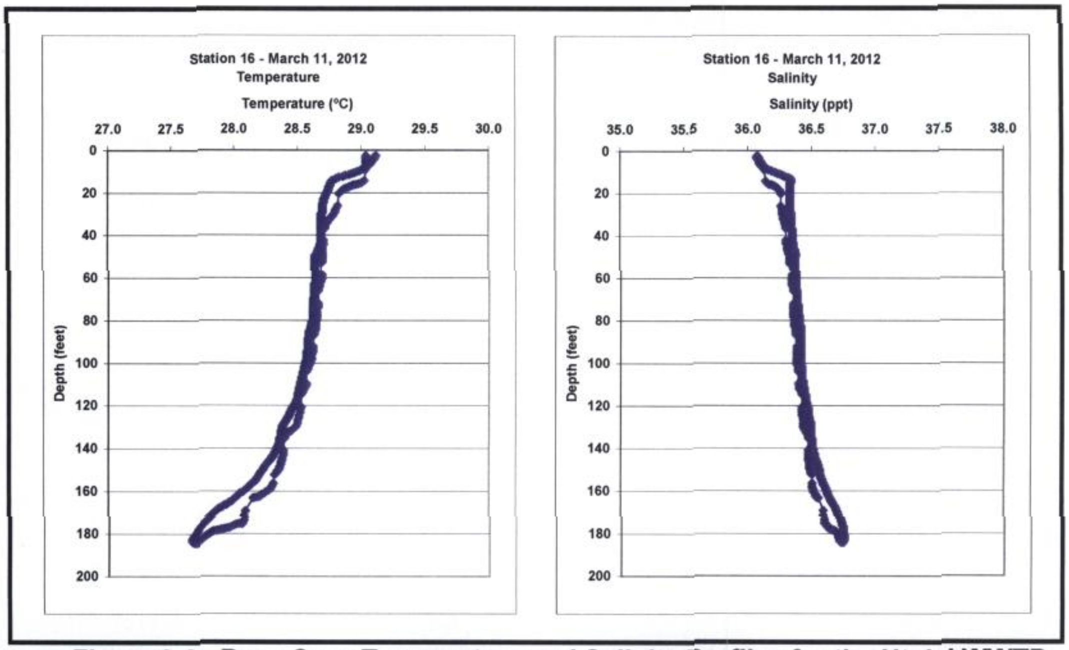

Figure 3-2. Base Case Temperature and Salinity Profiles for the Utulei WWTP
(The two traces represent downcast and upcast data, which are averaged to develop the density profile used in the modeling.)

3.2.5 Base Case Effluent and Ambient Parameters

Based on the descriptions in Sections 3.2.1 through 3.2.4, the base case effluent and ambient parameters are defined as shown in Table 3-1.

Table 3-1. Base-case Effluent and Ambient Parameters

Parameter	Tafuna WWTP Diffuser	Utulei WWTP Diffuser
Effluent Flow	1 mgd through 10 mgd	1mgd through 10 mgd
Effluent Temperature	29 °C	29 °C
Effluent Salinity	0.5 ppt	0.5 ppt
Effluent Density	0.99638 g/cm^3	0.99638 g/cm^3
Ambient Current Speed	2 cm/sec	2 cm/sec
Ambient Current Direction	Perpendicular to Diffuser Barrel	Perpendicular to Diffuser Barrel
Ambient Density Profile	Station C - March 2012	Station 16 - March 2012

3.3 Determination of Critical Initial Dilution and Zone of Initial Dilution

To evaluate the CID and ZID dimensions, the USEPA-approved dilution model UDKHDEN was used to predict initial dilution for the various simulations. The UDKHDEN model input parameters for the base case are listed in Tables 2-1 and 3.1. The model provides the dilutions for the plume continuously along the plume trajectory. Unless blocked by the water surface, the plume initially rises through the equilibrium or trapping level (where ambient and plume water are of the same density), overshooting the equilibrium level, and then collapsing back to the equilibrium level. The UDKHDEN model tracks the plume to the point of maximum rise and then terminates execution (see Figure 3-

3 below). Dilution continues during the rise of the plume past the trapping level. However, to be conservative, the dilution as the plume first passes the trapping level is generally taken as the initial dilution. The dilution at the point of maximum rise can also be considered. Both dilutions, at the trapping level and at maximum rise, are presented for the analyses in this TM

In cases where different size ports are considered the ports of the same size are modeled independently as separate groups. The CID was calculated for critical conditions as a flux-averaged value (CID_A), accounting for the flow (Q) through the various port sizes. This calculation is described as follows:

$$CID_A = \frac{\sum (CID_i \, x \, Q_i)}{\sum Q_i}$$

where *i* represents the individual port sizes. The manner in which the model handles multiple ports provides for the full effect of merging of adjacent plumes.

The ZID dimensions are taken at the point of initial dilution (either at the trapping level or at the point of maximum rise) and are calculated by the following equation:

$$L_{ZID} = Y_i + \left(\frac{W_i}{2}\right) \times \cos(90 - \theta_i)$$

where

L_{ZID}=the distance from the discharge port to the edge of the plume at the point of interest (in this case both the trapping level and the point of maximum rise are considered)

Y_i= horizontal distance from port "i" to the centerline of the plume; ($Y_I = [X^2 + Y^2]^{½}$ in terms of the UDKHDEN output variables)

W_i = width of the plume from port "i"

θ_i = angle of the plume centerline to the horizontal.

Calculation of the L_{ZID} at the trapping level requires the interpolation of the various plume parameters at the point of interest. A linear interpolation scheme is used. This calculation is described as follows:

$$P_{CID} = P_2 - (P_2 - P_1)/(S_2 - S_1)\, x\, (S_2 - CID_A)$$

where

S_1 and S_2 = the dilutions lower and higher than the CID_A, respectively

P_1 and P_2 = the values of the parameter Y_i, W_i, or θ_i at the same point in the plume trajectory as the respective dilutions, S_1 and S_2, and

P_{CID} = the value of the parameter at the CID_A.

A schematic representation of the parameters and variables described above is shown in Figure 3-3.

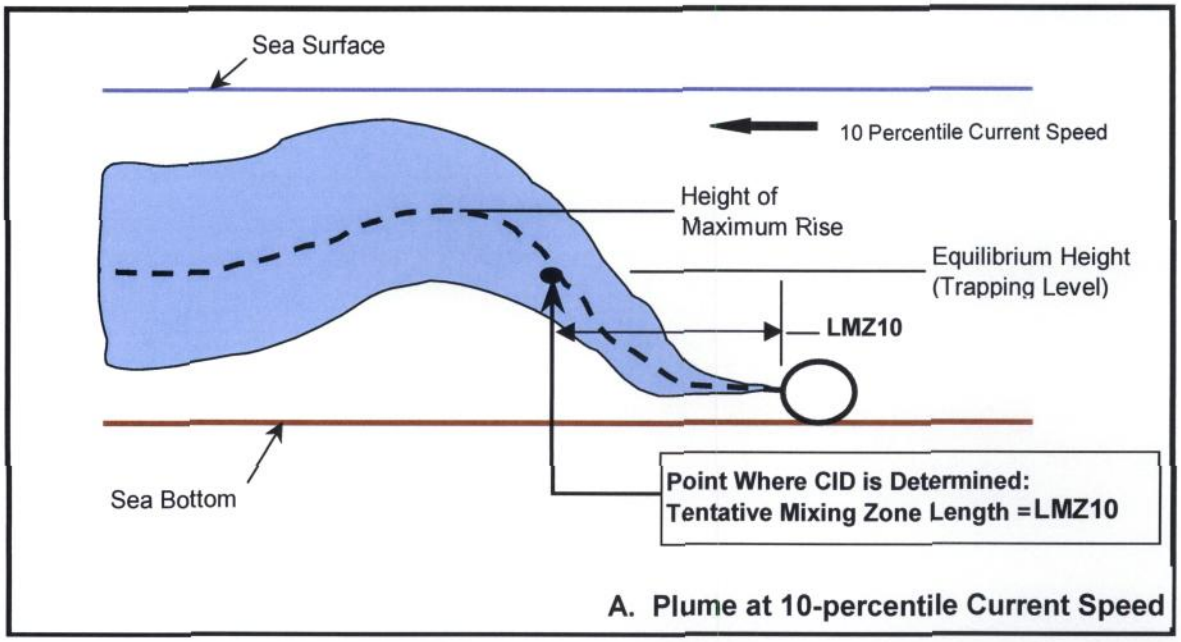

Figure 3-3. Schematic of Plume Characteristics for 10 percentile Current Speed
(L_{MZ} is the same as L_{ZID} at the trapping level in the analyses in this TM; L_{ZID} at the point of maximum rise is defined at the point of maximum rise shown)

The ZID is often taken as the compliance point for effluent constituents. That is the calculated concentration of parameters of concern (POCs) must meet water quality standards (WQSs) at that point after being diluted in the plume. However, the distance to the ZID, as described above, is based on the CID and under higher current conditions the CID is often further from the discharge than under critical conditions (10 percentile current speed). This occurs because under higher current speeds the plume is transported further prior to a dilution equal to the CID (and the overall dilution is much higher at the end of initial dilution). This situation is illustrated schematically in Figure 3-4. This effect should be considered to accurately determine the ZID, but there are no current data for the diffuser locations to accurately make such an assessment. For the results presented in this TM a representative L_{ZID} (L_{MZ}) is reported for the 10th percentile current speed, although the actual L_{ZID} may be larger if higher current speeds were considered. To compare the relative performance of the various diffuser configurations presented in this TM the ZID defined under the 10th percentile current are used, but it is recognized that this is potentially conservative (smaller than expected) and could be larger if higher currents were considered.

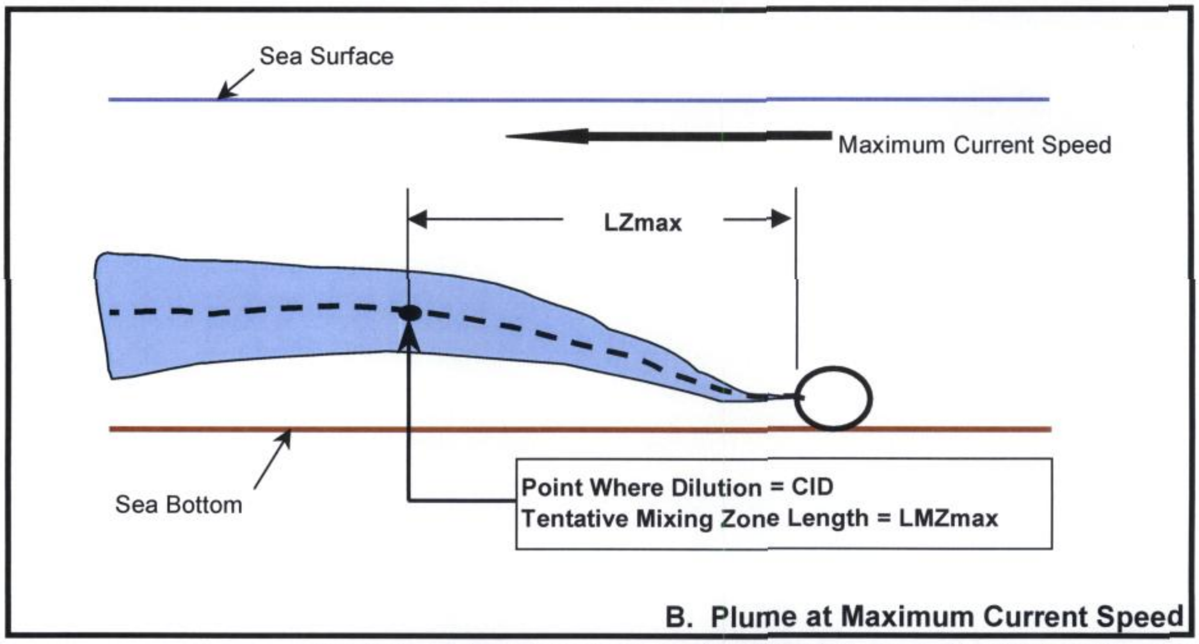

Figure 5. Schematic of Plume Characteristics for High Current Speed

3.4 Definition of the ZID and Mixing Zones

A mixing zone is a volume of water within which water quality standards (WQSs) can be exceeded if approved by the appropriate regulatory agencies. The ZID is one form of a mixing zone and can be defined in various ways as described above. Other definitions are provided in EPA's TSD for Water Quality-based Toxics Control (WQ TSD)[16] . The treatment in this TM is to define the ZID based on dilution modeling results. There are, depending on the various state jurisdictions, other definable mixing zones that are larger than the ZID. The ASWQS allows larger mixing zones for non-toxic substances (§24.0207).

The ASWQS defines the ZID "*as that area of a plume where dilution is achieved due to the combined effects of momentum and buoyancy of the effluent discharged from an orifice.*" It is noted that the ASWQS does not define the ZID in terms of the trapping level, but implies that the dilution achieved (which could be interpreted as the dilution at the maximum rise height or even during subsequent collapse of the plume) could be taken to define the ZID.

The ASWQS requires the ZID to be determined using the PLUMES model using a zero current speed. The PLUMES model is no longer supported by EPA and has been superseded by Visual Plumes. Visual Plumes has three initial dilution models, one of which is essentially the same model as UDKHDEN. Based on USEPA and the American Samoa Environmental Protection Agency (ASEPA) previous acceptance of UDKHDEN as appropriate to determine dilution and mixing zones, the continued use of this model appears acceptable. The use of a different model and the use of zero current can be waived by ASEPA and USEPA. Because the primary POCs are non toxic (nitrogen and phosphorous) it is presumed that the agencies will follow previous procedures and allow the use of a reasonable current using the UDKHDEN model. It is recognized that USEPA

[16] U.S. Environmental Protection Agency, 1991. *Technical Support Document For Water Quality-based Toxics Control.* EPA/505/2-90-001. March 1991 (The second printing of this document corrects some errors in the original and should be used.)

and ASEPA may consider mixing zones extended past the point of the trapping level or the past point of maximum rise in the future, but the definition of such mixing zones is not discussed further in this TM[17].

3.5 Seawater Intrusion and Re-circulation

Seawater intrusion and recirculation is a phenomenon that occurs in diffusers when effluent flows drop low enough that seawater can intrude into the diffuser and block the effluent discharge from the seaward ends of the diffuser as described below. One of the scenarios to be investigated following the AOs is to extend the length of the diffuser and, presumably, add additional ports. This could result in flows low enough for the initiation of seawater intrusion. Therefore, the potential of seawater intrusion needs to be considered when evaluating alternative diffuser configurations.

3.5.1 Dynamics of Seawater Intrusion

Seawater intrusion and recirculation (see Figure 3.4) occurs when flows drop below a critical level (usually defined as a densimetric Froude number [Frρ] below 2). A "flushing flow" substantially higher than the critical flow that allows initiation of seawater intrusion is generally required to re-establish normal diffuser operation (that is, flows out of all open ports). The intrusion of seawater into the diffuser can have a number of adverse effects including marine growth within the diffuser and sediment ingestion from the ambient water into the diffuser barrel. Over time these effects may result in the inability of the diffuser to operate as designed and may physically prevent flow from the seaward-most ports. Initial dilution achieved by the diffuser may be affected and compliance with water quality standards based on estimates of dilution and plume behavior may be inaccurate.

The densimetric Froude number can be used to characterize conditions that induce seawater intrusion and that conversely result in flushing flows that will subsequently clear the diffuser. The Froude number is a representation of the relative strengths of inertial and gravitational (or buoyant) forces. It is noted that each diffuser configuration is different and the use of the Froude number represents a generalized case that should be applied only for overall guidance. The densimetric Froude number is defined as:

$$F_{r\rho} = \frac{U_j}{\left[gD(\rho_o - \rho_d)/\rho_o\right]^{1/2}}$$

where

U_j = the port jet velocity
D = the port diameter
ρ_o = the ambient (seawater) density at the discharge depth
ρ_d = the density of the effluent
g = acceleration due to gravity

The value of Frρ can be calculated from the port velocities and the appropriate port diameter, ambient density, and effluent density. The value is provided as a part of the normal output of the initial dilution model UDKHDEN.

17 For example the mixing zone for nitrogen and phosphorous for the tuna canneries discharge into Pago Pago Harbor is substantially larger than the ZID and is based on an average current speed in the farfield dilution model.

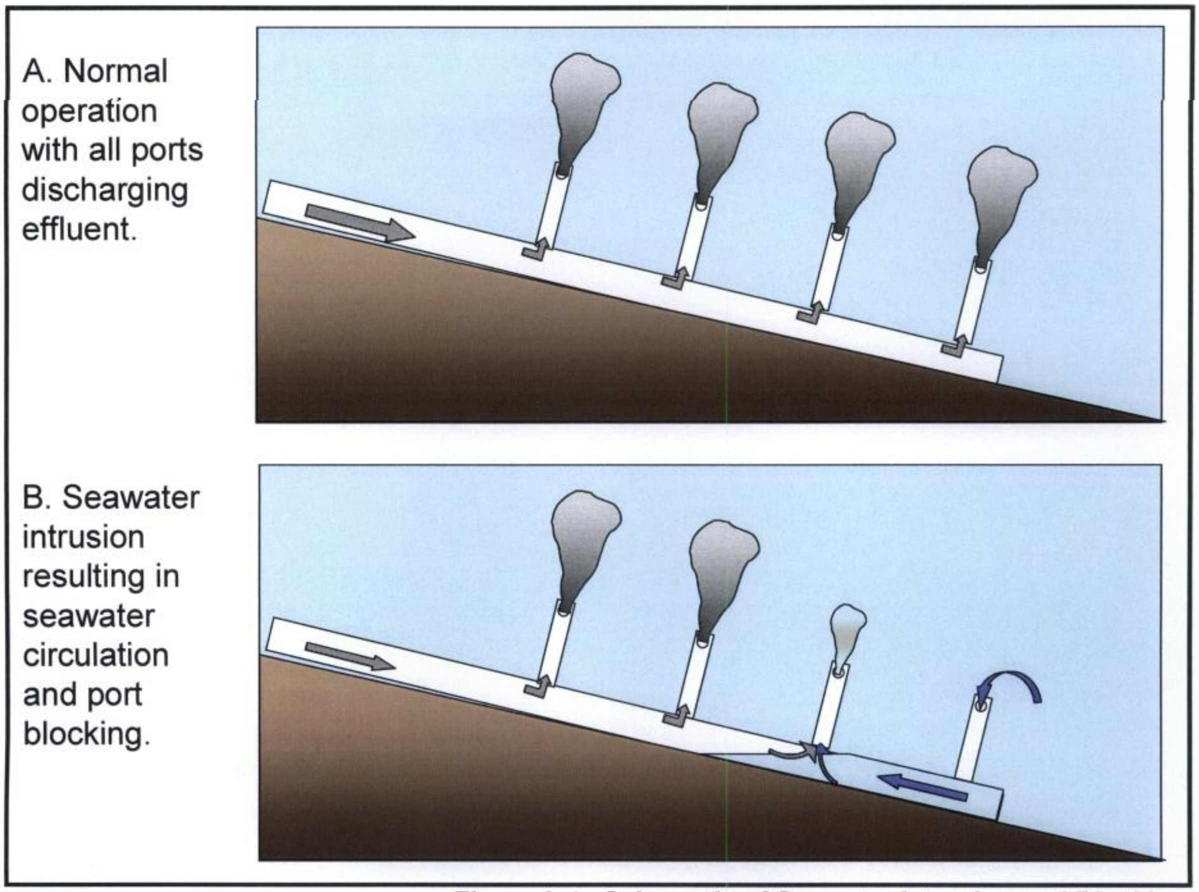

Figure 3-4. Schematic of Seawater Intrusion and Blocking

3.5.2 Avoiding Seawater Intrusion

There is no comprehensive or well-accepted model or predictor for flow conditions that will result in seawater intrusion or the subsequent flushing flows required to correct the condition. However, there are some documented investigations that provide general guidelines. Typically, flows resulting in an Frρ of approximately 1 to 2 will result in seawater intrusion if the diffuser barrel is on a slope. Once seawater intrusion has been established, an Frρ between 5 and 10 is typically required to flush the diffuser and initiate a resumption of normal operation. The definitions of Frρ values for initiation of seawater intrusion and for flushing are based on generally accepted levels and judgment; site-specific determinations of these values have not been done for the Utulei and Tafuna diffusers. The best determination of the occurrence of persistent seawater intrusion is based on actual diffuser inspections. As mentioned in Section 2, recent inspections of both diffusers have indicated that persistent seawater intrusion is not an issue for the current total port discharge area.

The occurrence of seawater intrusion is sensitive to the slope of the diffuser. Because the Tafuna diffuser is nearly level, the initiation of seawater diffusion for this facility is minimal (but still exists) and flushing flows are very likely within the normal range of the effluent flow regime. The Utulei diffuser is on a somewhat steeper slope and the potential for seawater intrusion is enhanced. Based on previous modeling and the observation of flow through all ports during the recent diver inspection, it appears that a densimetric Froude number of less than 7 is sufficient to flush the existing diffuser.

4 Base Case Diffuser Performance

The defined "base case" performance of the Tafuna and Utulei diffusers is described in this section. The diffuser configurations and ambient conditions are described in Sections 2 and 3. The modeling for the cases considered was done in two ways: the standard modeling approach and a modified modeling approach that more realistically accounts for plume merging behavior. Extended diffuser configurations are considered in Section 5 and modifications of the existing diffuser are considered in Section 6.

4.1 Tafuna WWTP Diffuser Base Case Performance

The base case configuration for Tafuna (see Section 2) is six 8-inch ports spaced 10 feet apart at a discharge depth of 89.2 feet (the average depth of the individual port discharges)[18]. Using standard hydraulic calculations, assuming sharp-edged ports on a 90 degree elbow, the flow distribution through the ports is as shown in Table 4-1 for the range of flows considered. The results indicate that over the range of expected flows the flow distribution is essentially uniformly distributed among the six ports on the diffuser. There is no need to consider varying port opening sizes along the diffuser. Head loss is well within the available head for the diffuser. Note that in this and all other descriptions ports are numbered sequentially from the seaward end going inshore; port number 1 is the last port on the diffuser barrel.

Table 4-1. Tafuna Base Case Diffuser Hydraulics - Port Velocities

Effluent Flow (mgd)	Port Velocity (ft/sec)						Head Loss (feet)
	Port 1	Port 2	Port 3	Port 4	Port 5	Port 6	
1	0.75	0.75	0.74	0.73	0.72	0.73	0.02
5	3.79	3.79	3.73	3.66	3.57	3.63	0.48
10	7.59	7.58	7.47	7.31	7.13	7.25	1.88

Table 4-2 shows the velocities in diffuser barrel upstream of each port. The velocities are somewhat lower than desirable to avoid sedimentation in the diffuser barrel, which indicates that larger ports or additional ports would not be prudent and lead to increased potential for sediment accumulation in the diffuser barrel. The exception to this might be a terminal port on the end gate of the diffuser barrel as described in Section 6, which results in higher pipe velocities up to the terminal port.

Table 4-2. Tafuna Base Case Diffuser Hydraulics - Pipe Velocities

Effluent Flow (mgd)	Pipe Velocity Upstream of Port (ft/sec)						Head Loss (feet)
	Port 1	Port 2	Port 3	Port 4	Port 5	Port 6	
1	0.08	0.17	0.25	0.33	0.41	0.49	0.002
5	0.42	0.84	1.26	1.66	2.06	2.46	0.031
10	0.84	1.69	2.51	3.33	4.12	4.92	0.104

The dilution performance of the base case diffuser is shown in Table 4-3 for the range of flows considered. Figures 4-1 and 4-2 provide graphical descriptions of the results for dilution and L_{ZID} (L_{MZ}), respectively. The model results (dilution projections) account for

18 The depth variation between the ports is small, allowing the ports to be modeled as a single group.

individual plumes merging, which limits dilution beyond the point where they merge. This leads to an extremely conservative prediction because the model calculations are done with all ports discharging in the same direction. This is an internal model constraint of virtually all dilution models commonly in use; the model cannot calculate the dilution for discharges opposed to the current. Because of this the plumes are predicted to merge considerably sooner than they actually do, which reduces predicted dilution. A modified modeling approach is used to account for this as described in Section 5.

The densimetric Froude number appears to be sufficient to avoid persistent seawater intrusion and recirculation. Seawater intrusion would occur at the lowest flows expected, but higher effluent flows, probably around 5 mgd would likely re-establish full flow through all ports. It would not be prudent to lower the Froude number by increasing port area (number of or size of the ports) along the diffuser barrel.

Table 4-3. Tafuna Base Case Diffuser Performance

Flow (mgd)	Froude Number	Trapping Level[1] (m)	Maximum Rise Level[2] (m)	Dilution		L_{ZID}[3] (meters)	
				Trapping Level	Maximum Rise	Trapping Level	Maximum Rise
1	0.97	RD[4]					
2	1.94	RD[4]					
3	2.91	2.65	231.62	211.96	231.62	13.81	16.75
4	3.88	2.18	186.70	173.51	186.70	14.26	16.22
6	5.81	1.87	139.80	131.09	139.80	15.42	16.85
8	7.75	1.58	114.25	108.89	114.25	16.74	17.69
10	9.69	1.09	98.31	95.71	98.31	18.24	18.79

[1] Trapping level is the Distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] L_{ZID} is the distance of the edge of the plume from the discharge point
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.

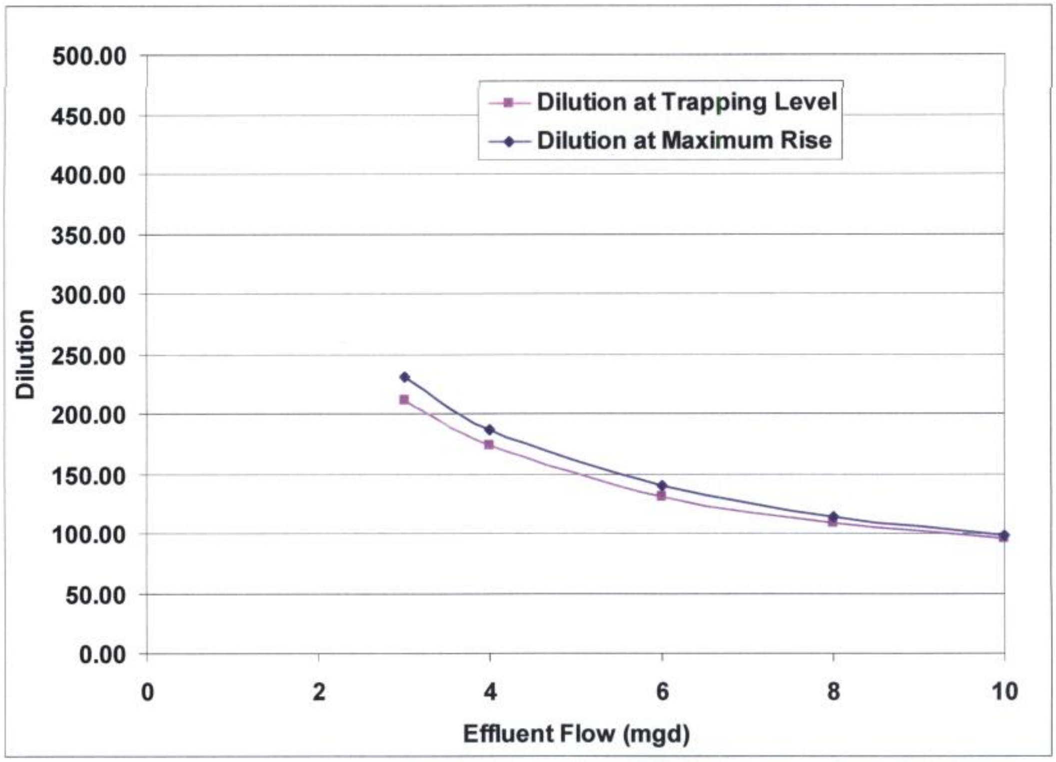

Figure 4-1. Dilution for Tafuna Diffuser - Base Case Conditions

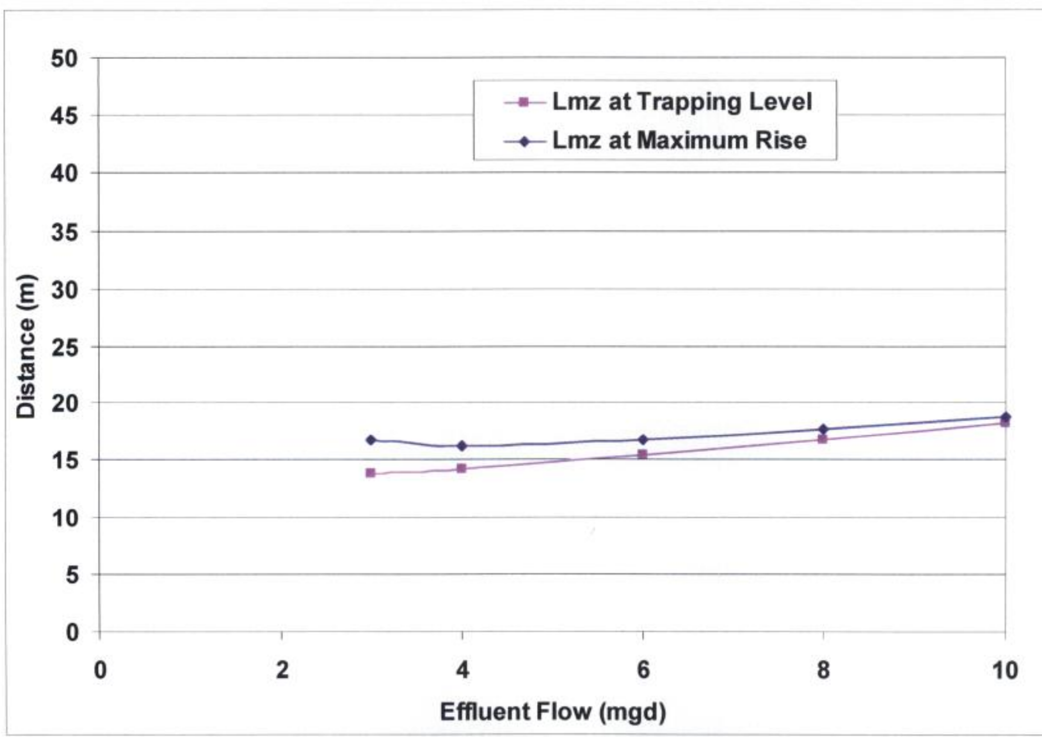

Figure 4-2. L_{ZID} for Tafuna Diffuser - Base Case Conditions

4.2 Utulei WWTP Diffuser Base Case Performance

The base case configuration for Utulei (see Section 2) is six 6-inch ports spaced 7.1 feet apart at a discharge depth of 150.4 feet (the average depth of the individual port discharges)[19]. Using standard hydraulic calculations, assuming sharp-edged ports on a 90 degree elbow, the flow distribution is as shown in Table 4-4. The results indicate that over the range of expected flows the flow distribution is essentially uniformly distributed among the six ports on the diffuser. There is no need to consider varying port opening sizes along the diffuser to avoid sedimentation in the diffuser barrel. Head loss is well within the available head for the diffuser. Note that ports are numbered sequentially from the seaward end going inshore; port number 1 is the last port on the diffuser barrel.

Table 4-4. Utulei Base Case Diffuser Hydraulics - Port Velocities

Effluent Flow (mgd)	Port Velocity (ft/sec)						Head Loss (feet)
	Port 1	Port 2	Port 3	Port 4	Port 5	Port 6	
1	1.32	1.32	1.32	1.31	1.30	1.31	0.05
5	6.62	6.62	6.59	6.55	6.49	6.52	1.27
10	13.25	13.25	13.19	13.09	12.98	13.03	5.06

Table 4-5 shows the velocities in diffuser barrel upstream of each port. The velocities are somewhat lower than desirable, which indicates that larger ports or additional ports would not be prudent and lead to increased potential for sediment accumulation in the diffuser barrel. The exception to this would be a terminal port on the end gate of the diffuser barrel as discussed in Section 6, which results in higher pipe velocities up to the terminal port.

Table 4-5. Utulei Base Case Diffuser Hydraulics - Pipe Velocities

Effluent Flow (mgd)	Pipe Velocity Upstream of Port (ft/sec)						Head Loss (feet)
	Port 1	Port 2	Port 3	Port 4	Port 5	Port 6	
1	0.08	0.17	0.25	0.33	0.41	0.49	0.001
5	0.41	0.83	1.24	1.65	2.05	2.46	0.022
10	0.83	1.66	2.48	3.30	4.11	4.92	0.073

The dilution performance of the base case diffuser is shown in Table 4-6 for the range of flows considered. Figures 4-3 and 4-4 provide graphical descriptions of the results for dilution and L_{ZID} (L_{MZ}), respectively. As in the case of the Tafuna diffuser, the model results (dilution projections) account for individual plumes merging, which limits dilution beyond the point where they merge and again leads to an extremely conservative prediction because the model calculations are done with all ports discharging in the same direction.

The densimetric Froude number appears to sufficient to avoid persistent seawater intrusion and recirculation. Seawater intrusion would occur at the lowest flows expected, but higher effluent flows, probably around 5 to 6 mgd would likely re-establish full flow through all ports. It would not be prudent to lower the Froude number by increasing port area (number of or size of the ports) along the diffuser barrel.

[19] The depth variation between the ports is small, allowing the ports to be modeled as a single group.

Table 4-6. Utulei Base Case Diffuser Performance

Flow (mgd)	Froude Number	Trapping Level[1] (m)	Maximum Rise Level[2] (m)	Dilution		L_{ZID}[3] (meters)	
				Trapping Level	Maximum Rise	Trapping Level	Maximum Rise
1	1.99	RD[4]					
2	3.97	15.06	41.94	264.47	344.2	20.88	33.62
4	7.94	6.27	45.7	197.76	222.52	25.19	37.45
6	11.91	4.95	44.99	153.26	165.84	26.52	30.55
8	15.88	3.97	45.38	129.31	137.97	28.93	32.01
10	19.86	3.57	45.65	113.46	120.54	31.58	34.27

[1] Trapping level is the Distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] L_{ZID} is the distance of the edge of the plume from the discharge point
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.

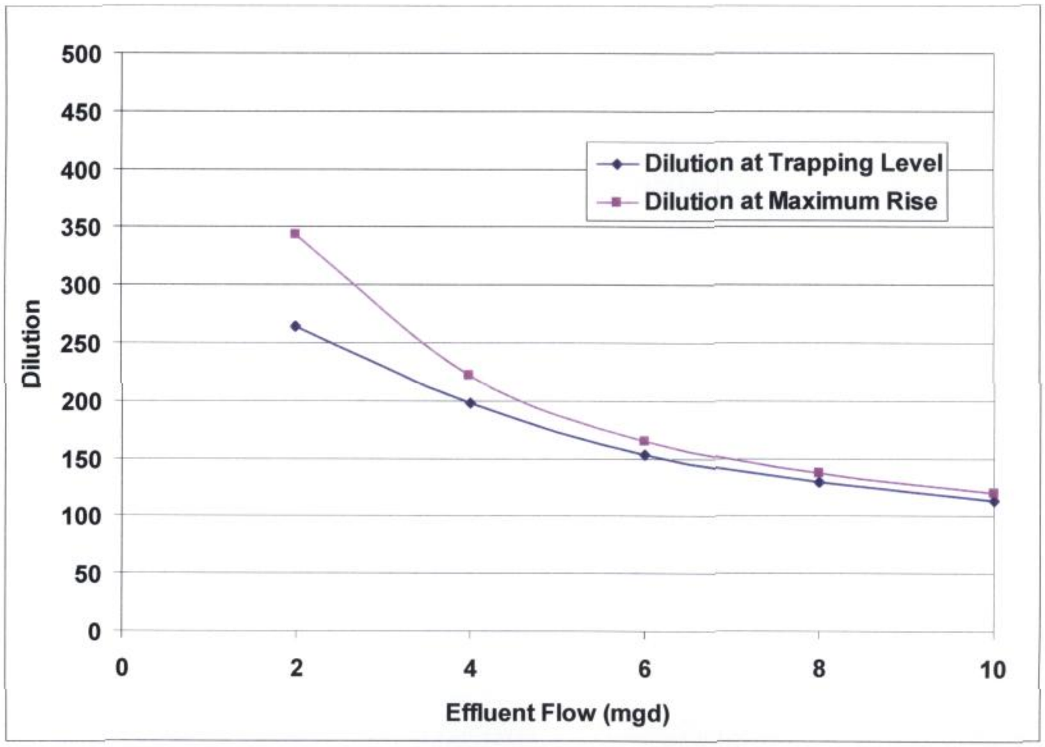

Figure 4-3. Dilution for Utulei Diffuser - Base Case Conditions

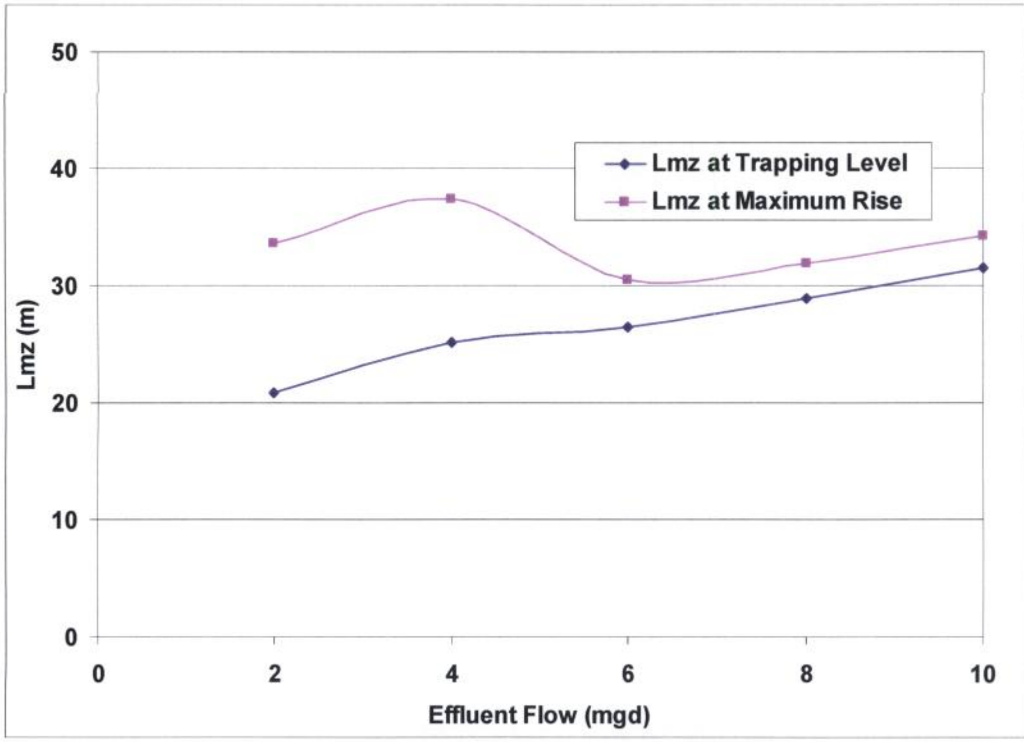

Figure 4-4. L_{ZID} for Utulei Diffuser - Base Case Conditions

4.3 Evaluation of Existing Diffuser Configurations

Overall the base case diffuser configurations appear well suited for the existing effluent flows for both the Tafuna and Utulei diffusers. The head losses are well below the available heads for the discharges. At low flows there may be some potential for seawater intrusion, but this is not expected to be persistent because higher flows appear sufficient to flush the diffusers and re-establish flow through all ports. The best evidence for this is the observed full flow condition during the AUS inspections.

4.4 Realistic Merging Approach

The predicted dilutions presented above are almost certainly quite conservative. The primary reason for this is that the discharges from adjacent ports are in opposite directions, while the model predictions are based on discharges that are all in the same direction. Therefore, the model predicts merging, and cessation of lateral mixing, well before merging actually occurs. Based on experience with similar diffusers this results in a considerable under prediction of dilution by a factor of 2 to 6 times. Figure 4-5 shows a schematic of this situation. The example on the left of Figure 4-5 illustrates how the model configuration results in merging of adjacent plumes and the example on the right is an example of the actual behavior of the plumes. The discharge from the middle port is opposed to the current direction and initially travels to the right as it rises, as the plume velocities decrease the current bends the plume back over the diffuser and it eventually merges with the other two plumes well beyond the model predicted location. This results in substantially higher dilutions than predicted by the model.

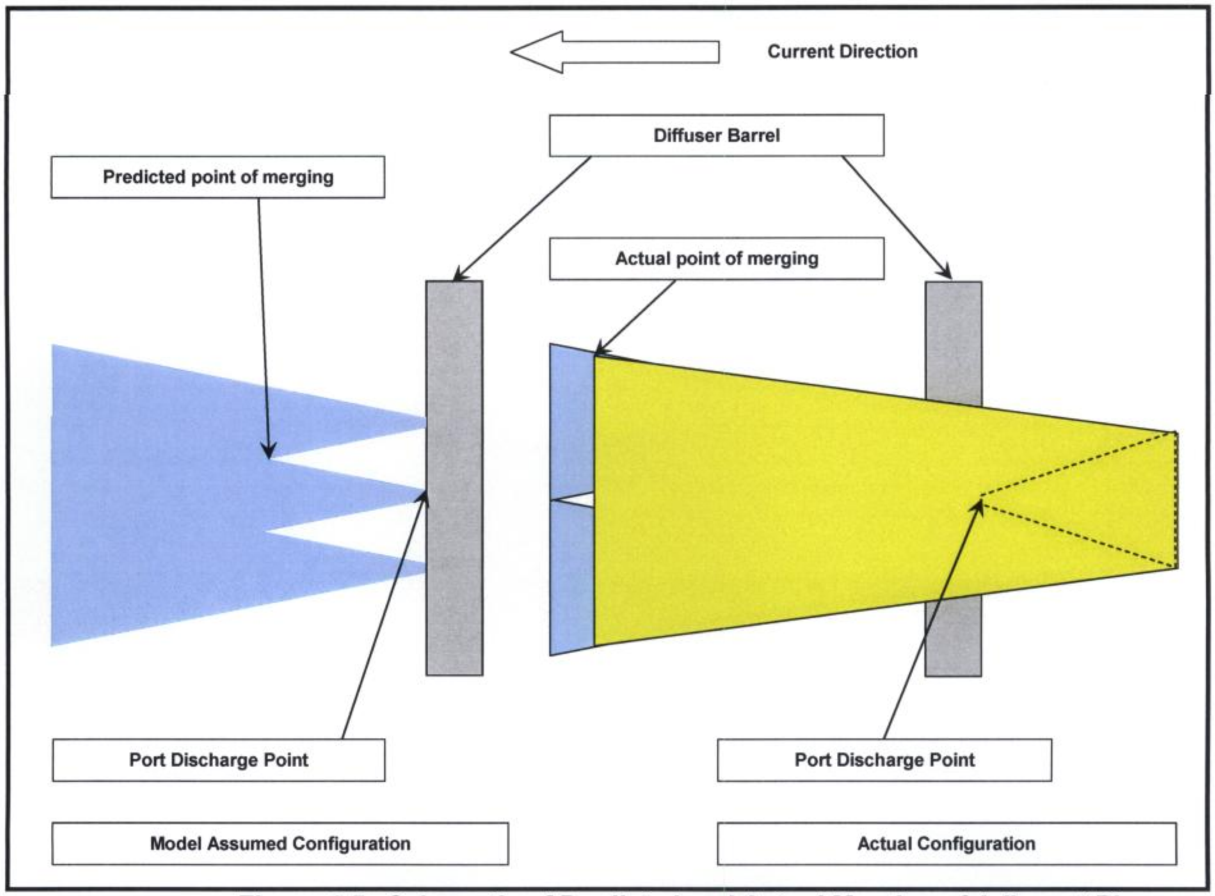

Figure 4-5. Schematic of Predicted and Actual Merging of Adjacent Plumes

To evaluate the issue of artificially premature merging of plumes from adjacent ports, the diffusers were run for a single port (with the flow appropriate for a single port) and zero current velocity. The model predicted plume geometry was examined for both diffusers and the results show that the plumes from each side of the diffuser *do not merge prior to reaching maximum rise* for total effluent flows up to and including 10 mgd. Therefore, a better evaluation of actual dilution will be obtained by using three ports at twice the spacing (and one-half the total flow) for each side of the diffuser. This modified modeling approach was done for both diffusers. It is noted that the results are still conservative (under predict dilution) based on the dye study investigations referenced in Section 3.

4.5 Tafuna WWTP Diffuser Base Case Performance - Realistic Merging

The results for the realistic merging model approach are shown for the Tafuna diffuser in Table 4-7. The dilutions predicted are substantially higher than those using the standard modeling approach. The approach described here is considered more realistic as described in Sections 1.3 and 4.4 and it is recommended that these results be used for assessing dilution and ZID dimensions. The results are shown graphically in Figures 4-6 and 4-7.

Table 4-7. Tafuna Base Case Diffuser Performance Evaluated for Realistic Merging Behavior

Flow (mgd)	Froude Number	Trapping Level[1] (m)	Maximum Rise Level[2] (m)	Dilution		L_{ZID}[3] (meters)	
				Trapping Level	Maximum Rise	Trapping Level	Maximum Rise
1	0.97	RD[4]					
2	1.94	RD[4]					
3	2.91	3.18	27.16	361.21	404.71	15.73	22.65
4	3.88	2.95	26.98	292.57	322.56	15.83	20.38
6	5.81	2.59	26.90	221.48	239.60	16.69	19.4
8	7.75	2.37	27.39	183.61	199.42	17.89	20.58
10	9.69	2.13	27.15	160.07	170.88	19.09	21.03

[1] Trapping level is the Distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] L_{ZID} is the distance of the edge of the plume from the discharge point
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.

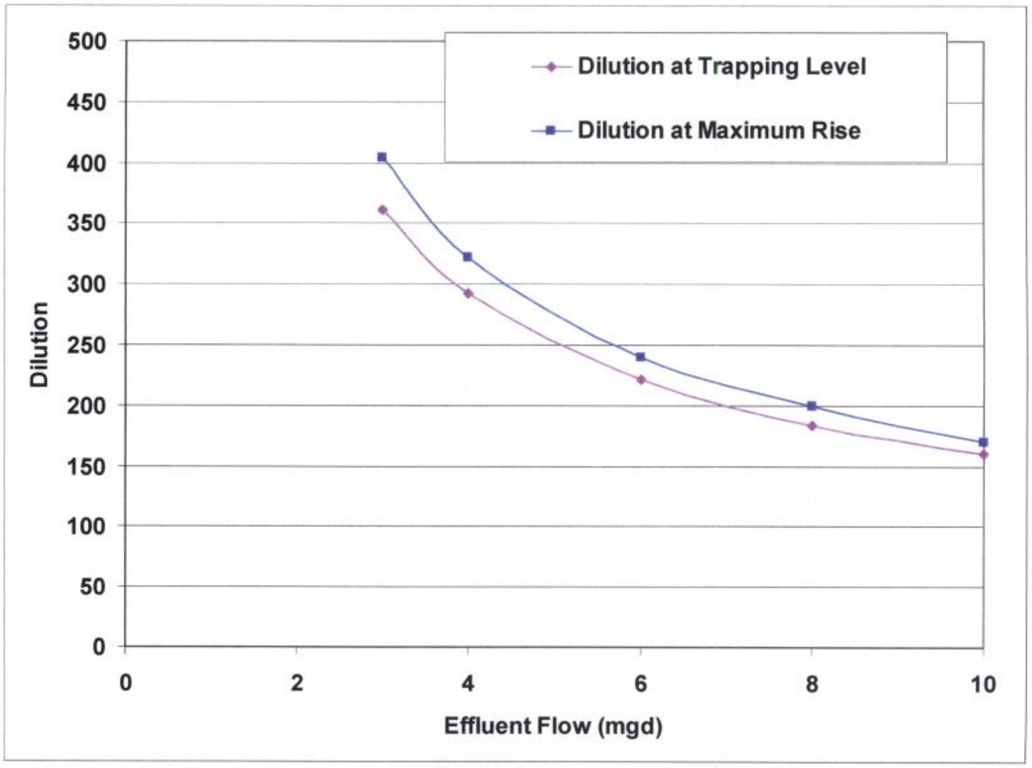

Figure 4-6 - Tafuna Dilution Evaluated for Realistic Merging Behavior

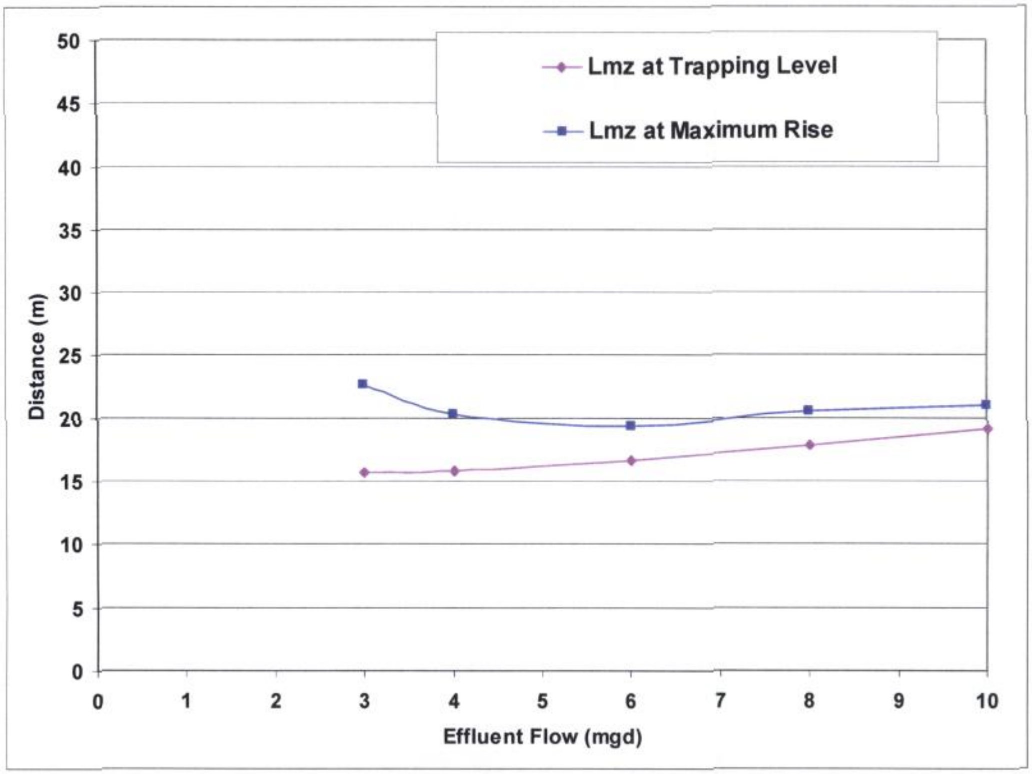

Figure 4-7 - Tafuna L_{ZID} Evaluated for Realistic Merging Behavior

4.6 Utulei WWTP Diffuser Base Case Performance - Realistic Merging

The results for the realistic merging model approach are shown for the Utulei diffuser in Table 4-8. The dilutions predicted are substantially higher than those using the standard modeling approach. The realistic merging approach described here (see Section 4.3) is considered more realistic and it is recommended that these results be used for assessing dilution and ZID dimensions. The results are shown graphically in Figures 4-8 and 4-9.

Table 4-8. Utulei Base Case Diffuser Performance Evaluated for Realistic Merging Behavior

Flow (mgd)	Froude Number	Trapping Level[1] (m)	Maximum Rise Level[2] (m)	Dilution		L_{ZID}[3] (meters)	
				Trapping Level	Maximum Rise	Trapping Level	Maximum Rise
1	1.99	RD[4]					
2	3.97	25.69	29.92	311.46	443.63	16.29	21.79
4	7.94	15.97	40.94	265.87	344.14	23.49	41.45
6	11.91	12.77	44.16	218.45	275.89	25.98	46.24
8	15.88	10.33	45.70	191.25	232.53	28.77	51.62
10	19.86	6.09	45.31	181.49	200.35	33.12	41.01

1 Trapping level is the Distance below the surface of the plume centerline
2 Maximum rise is the distance above the discharge point of the plume centerline
3 L_{ZID} is the distance of the edge of the plume from the discharge point
4 RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.

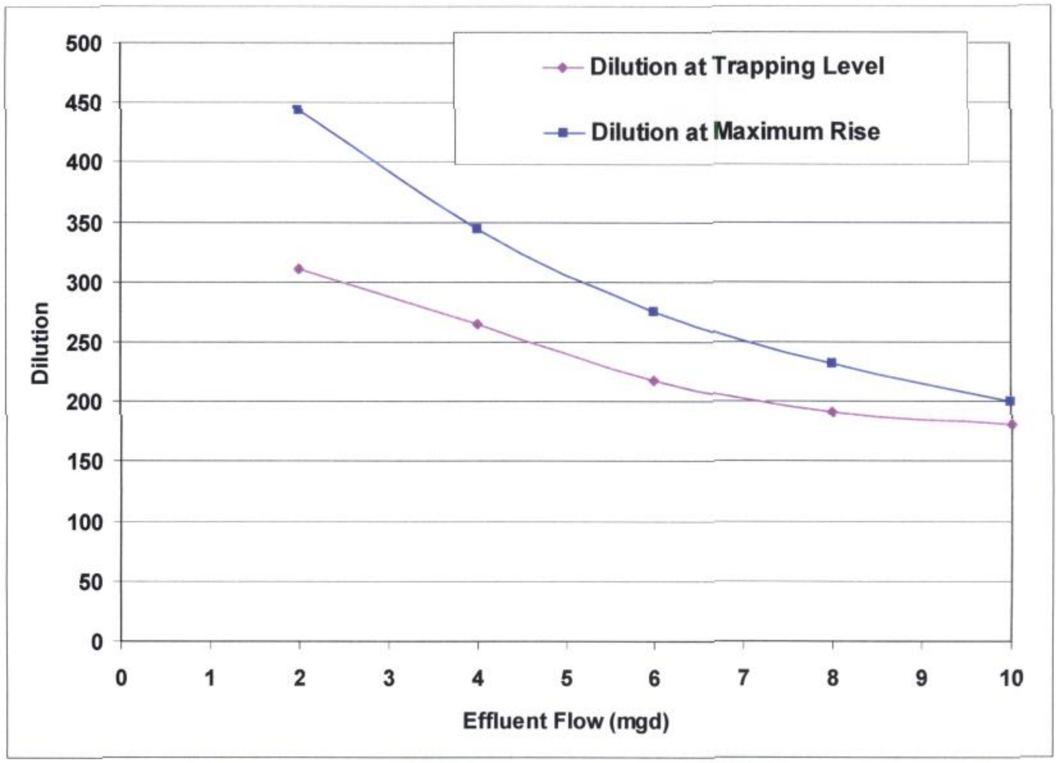

Figure 4-8 - Utulei Dilution Evaluated for Realistic Merging Behavior

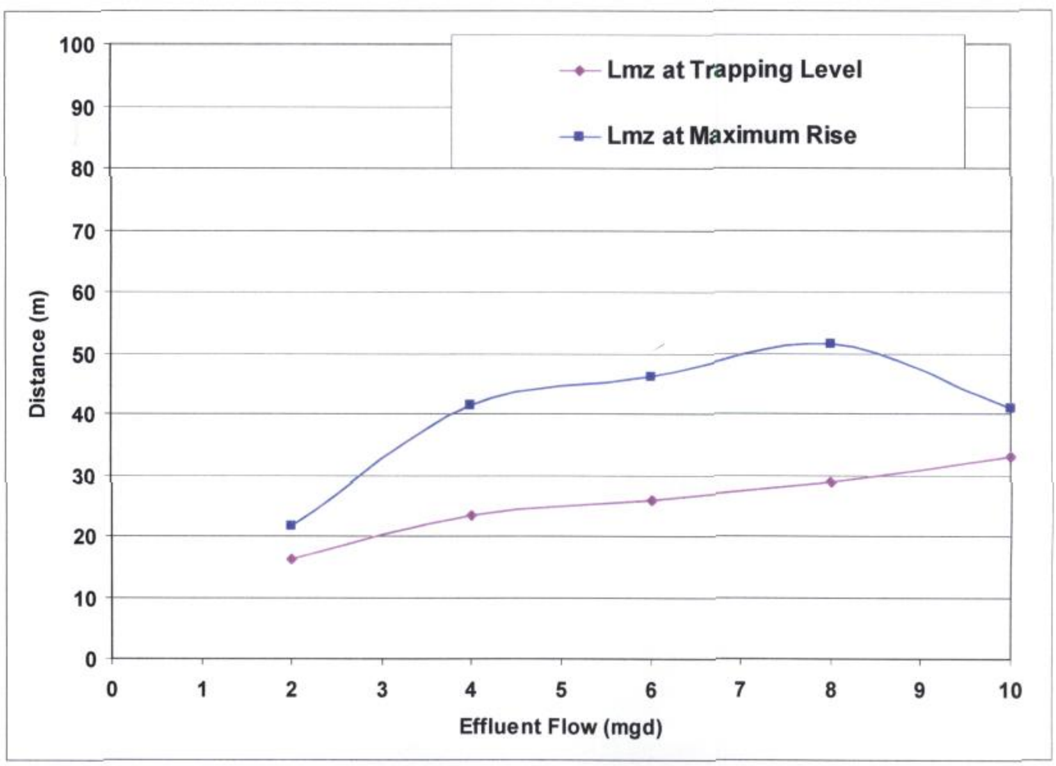

Figure 4-9 - Utulei L_{ZID} Evaluated for Realistic Merging Behavior

5 Diffuser Performance for Increased Diffuser Length

The AOs specifically require the consideration of increased diffuser lengths (doubling the existing diffuser length) as a means to potentially increase the CID and/or ZID dimensions. Increasing the diffuser length can be considered in three ways:

- Doubling the diffuser length in a linear fashion and adding ports (for example doubling the number of ports at the existing port spacing. This could potentially increase the CID and may, or may not, increase the lateral dimension of the ZID. Port size may have to be adjusted to maintain sufficient flow in the diffuser barrel and avoid seawater intrusion.
- Doubling the diffuser length in a linear fashion and maintaining the existing number of ports. This increases the port spacing and therefore will result in plume merging further from the discharge point, which will potentially increase dilution (CID) and may increase the lateral dimension of the ZID.
- Doubling the diffuser length by replacing the existing diffusers with a "Y"-shaped diffuser with each leg the same length as the existing diffuser. This approach is not considered feasible for the Tafuna outfall because the outfall is purposely located in an area of sandy bottom surrounded by hard bottom and coral that would require substantial disturbance to the existing habitat to create the space for the "Y"-shaped diffuser. This approach is not considered reasonable for the Utulei outfall because of the steep slope and difficulty of construction at the existing depth of the diffuser.

5.2.1 Doubling the Tafuna Diffuser Length

The existing port area of the Tafuna diffuser is currently about 66 percent of the diffuser barrel diameter. Because the diffuser is fairly level, persistent seawater intrusion is not considered a problem. However, increasing the port area is not considered prudent to avoid increased potential of persistent seawater intrusion and is not considered here. The most reasonable approaches would be to:

- Increase the diffuser length using the same number of ports at twice the spacing and maintaining the existing port diameters.
- Increasing the diffuser length using the existing port spacing. This would require maintaining the current port area to diffuser barrel area as is currently the case. This would require port sizes of approximately 5.5 inches.

Both of these approaches were considered. For the first approach (doubling the port spacing) the results are shown in Table 5-1 using base case ambient conditions (Section 3.2.4). These results are, as expected, identical to the results obtained when the realistic merging behavior of the base case configuration is used (see Table 4-7). If the realistic merging modeling approach described above is applied to the extended diffuser is applied to this case the results are as shown in Table 5-2. At the approximate design flow of the Tafuna WWTP (6 mgd) the increase in dilution is about 65 %.

Table 5-1. Tafuna Double Diffuser Length with Six Ports Diffuser Performance for Standard Modeling Approach

Flow (mgd)	Froude Number	Trapping Level[1] (m)	Maximum Rise Level[2] (m)	Dilution		L_{ZID}[3] (meters)	
				Trapping Level	Maximum Rise	Trapping Level	Maximum Rise
1	0.97	RD[4]					
2	1.94	RD[4]					
3	2.91	3.18	27.16	361.21	404.71	15.75	22.85
4	3.88	2.95	29.98	292.57	322.56	15.83	20.38
6	5.81	2.59	26.90	221.48	239.60	16.70	19.40
8	7.75	2.37	27.39	183.61	199.42	17.89	20.58
10	9.69	2.13	27.15	160.07	170.88	19.09	21.03

[1] Trapping level is the Distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] L_{ZID} is the distance of the edge of the plume from the discharge point
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.

Table 5-2. Tafuna Double Diffuser Length with Six Ports Diffuser Performance Evaluated for Realistic Merging Behavior

Flow (mgd)	Froude Number	Trapping Level[1] (m)	Maximum Rise Level[2] (m)	Dilution		L_{ZID}[3] (meters)	
				Trapping Level	Maximum Rise	Trapping Level	Maximum Rise
1	0.97	RD[4]					
2	1.94	RD[4]					
3	2.91	3.71	26.12	575.84	647.03	16.40	20.93
4	3.88	3.38	26.60	466.49	524.76	17.31	21.57
6	5.81	3.16	27.18	350.01	396.00	18.11	24.72
8	7.75	2.98	27.09	289.72	323.60	19.24	24.29
10	9.69	2.81	27.13	252.52	279.78	20.52	24.03

[1] Trapping level is the Distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] L_{ZID} is the distance of the edge of the plume from the discharge point
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.

The alternative approach, doubling the number of ports at the existing port spacing with the appropriate reduction in port diameter results in dilutions shown in Tables 5-3 and 5-4 for the standard and realistic merging modeling cases using base case ambient conditions, respectively. This configuration shows only modest increases in dilution compared to the case with fewer larger ports (Tables 5-1 and 5-2).

Table 5-3. Tafuna Double Diffuser Length with 12 Ports Diffuser Performance for Standard Modeling Approach							
Flow (mgd)	Froude Number	Trapping Level[1] (m)	Maximum Rise Level[2] (m)	Dilution		L_{ZID}[3] (meters)	
				Trapping Level	Maximum Rise	Trapping Level	Maximum Rise
1	1.24	RD[4]					
2	2.47	3.63	26.48	500.82	558.94	16.57	27.12
4	4.94	3.02	26.91	293.50	323.62	15.85	20.23
6	7.42	2.70	26.93	219.17	238.47	15.77	19.12
8	9.89	2.22	27.00	181.54	194.18	17.26	19.20
10	12.36	2.21	27.32	155.73	167.87	18.19	20.33

[1] Trapping level is the Distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] L_{ZID} is the distance of the edge of the plume from the discharge point
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.

Table 5-4. Tafuna Double Diffuser Length with 12 Ports Diffuser Performance Evaluated for Realistic Merging Behavior							
Flow (mgd)	Froude Number	Trapping Level[1] (m)	Maximum Rise Level[2] (m)	Dilution		L_{ZID}[3] (meters)	
				Trapping Level	Maximum Rise	Trapping Level	Maximum Rise
1	1.24	RD[4]					
2	2.47	5.12	25.31	826.27	947.41	18.10	17.12
4	4.94	3.51	26.49	502.38	558.67	17.99	22.41
6	7.42	3.32	27.14	372.61	417.60	18.04	24.72
8	9.89	3.05	27.10	305.87	338.14	18.69	24.01
10	12.36	2.86	27.08	264.10	289.08	19.64	23.60

[1] Trapping level is the Distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] L_{ZID} is the distance of the edge of the plume from the discharge point
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.

5.2.2 Doubling the Utulei Diffuser Length

The existing port area of the Utulei diffuser is currently about 37 percent of the diffuser barrel diameter. Because the diffuser is fairly level in its current position, persistent seawater intrusion is not considered a problem. However, substantially increasing the port area is not considered prudent and is not considered here. Some increase in port area appears justified and is discussed below and in Section 5.3. The most reasonable approaches to extend the diffuser would be to:

- Increase the diffuser length using the same number of ports at twice the spacing and maintaining the existing port diameters.
- Increasing the diffuser length using the existing port spacing. This would require increasing the total port area somewhat to maintain a reasonable individual port

opening size. Port sizes of approximately 5 inches (the minimum recommended) would increase the port area to about 52 percent of the diffuser barrel cross sectional area (the maximum deemed prudent.

Both of these approaches were considered. For the first approach (doubling the port spacing) the results are shown in Table 5-5. These results are, as expected, identical to the results obtained when the realistic merging behavior of the base case configuration is used (see Table 4-8). If the realistic merging modeling approach described above is applied to this case the results are as shown in Table 5-6. At the approximate design flow of the Utulei WWTP (6 mgd) the increase in dilution is about 33%.

Table 5-5. Utulei Double Diffuser Length with Six Ports Diffuser Performance for Standard Modeling Approach

Flow (mgd)	Froude Number	Trapping Level[1] (m)	Maximum Rise Level[2] (m)	Dilution		L_{ZID}[3] (meters)	
				Trapping Level	Maximum Rise	Trapping Level	Maximum Rise
1	1.99	RD[4]					
2	3.97	25.71	29.91	311.71	443.53	16.27	21.61
4	7.94	15.99	41.02	266.16	345.21	23.49	29.34
6	11.91	12.79	44.14	218.70	276.32	25.97	40.87
8	15.88	10.37	45.68	191.45	232.87	28.78	43.59
10	19.86	6.10	45.31	181.77	200.68	33.14	41.51

[1] Trapping level is the Distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] L_{ZID} is the distance of the edge of the plume from the discharge point
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.

Table 5-6. Utulei Double Diffuser Length with Six Ports Diffuser Performance Evaluated for Realistic Merging Behavior

Flow (mgd)	Froude Number	Trapping Level[1] (m)	Maximum Rise Level[2] (m)	Dilution		L_{ZID}[3] (meters)	
				Trapping Level	Maximum Rise	Trapping Level	Maximum Rise
1	1.99	RD[4]					
3	5.96	26.77	27.05	363.04	505.10	16.69	20.48
4	7.94	24.26	31.17	334.35	464.25	19.54	23.76
6	11.91	20.73	36.10	291.84	393.95	23.49	25.72
8	15.88	18.05	40.27	264.55	353.72	27.00	28.82
10	19.86	15.16	42.11	249.76	316.90	30.79	34.42

[1] Trapping level is the Distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] L_{ZID} is the distance of the edge of the plume from the discharge point
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.

The alternative approach, doubling the number of ports at the existing port spacing with the appropriate reduction in port diameter results in dilutions shown in Tables 5-7 and 5-8 for the standard and realistic merging modeling cases, respectively. This configuration shows only modest increases in dilution compared to the case with fewer larger ports (Tables 5-5 and 5-6).

Table 5-7. Utulei Double Diffuser Length with 12 Ports Diffuser Performance for Standard Modeling Approach

Flow (mgd)	Froude Number	Trapping Level[1] (m)	Maximum Rise Level[2] (m)	Dilution		L_{ZID}[3] (meters)	
				Trapping Level	Maximum Rise	Trapping Level	Maximum Rise
1	1.57	RD[4]					
2	3.13	26.42	28.87	300.91	428.69	15.81	19.20
4	6.26	15.31	41.44	265.69	343.10	22.19	33.45
6	9.40	11.66	43.63	217.90	272.57	23.76	38.94
8	12.53	6.48	45.78	200.13	225.72	27.59	41.09
10	15.66	5.86	45.34	173.69	192.52	28.48	35.84

[1] Trapping level is the Distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] L_{ZID} is the distance of the edge of the plume from the discharge point
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.

Table 5-8. Utulei Double Diffuser Length with 12 Ports Diffuser Performance Evaluated for Realistic Merging Behavior

Flow (mgd)	Froude Number	Trapping Level[1] (m)	Maximum Rise Level[2] (m)	Dilution		L_{ZID}[3] (meters)	
				Trapping Level	Maximum Rise	Trapping Level	Maximum Rise
1	1.57	RD[4]					
2	3.13	32.80	19.32	357.80	521.58	11.35	13.22
4	6.26	26.13	29.12	311.90	438.74	17.05	19.17
6	9.40	21.30	35.54	282.09	384.15	21.06	22.65
8	12.53	19.19	39.98	247.31	342.84	22.98	23.85
10	15.66	15.14	42.30	238.82	307.41	26.78	31.49

[1] Trapping level is the Distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] L_{ZID} is the distance of the edge of the plume from the discharge point
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.

6 Diffuser Performance for Modified Port Configurations

As described in Section 5, modifying the diffuser length provides substantial increases in dilution compared to the base case (essentially existing conditions). There are a variety of other approaches to increasing dilution, which involve modifying the existing diffuser configuration without extending the diffuser barrel. These include changing port sizes and orientation and adding ports to the existing diffuser barrel. There are a large number of such approaches. Preliminary model runs indicate the simplest and most cost effective approach is to use existing ports with installation of orifice plates to reduce the existing port diameters and with an added port on the end gate (currently a blind flange). This approach is considered for both diffusers in this section.[20]

6.1 Modified Tafuna Diffuser configuration

As noted above the existing total port area of the Tafuna diffuser is quite likely as large as is prudent to be assured that persistent seawater intrusion is not an issue. Therefore, the approach here is to reduce the sizes of the existing ports by installing orifice plates and adding a larger port on the end gate. The total port area is kept approximately the same as the existing diffuser. As a representative example reducing the existing ports from 8 inches to 6 inches in diameter with an end port of 12.5 inches in diameter is considered. The hydraulic characteristics of this diffuser configuration are shown in Table 6-1, where Port 1 is the 12.5-inch diameter end port and Ports 2 through 7 are the existing ports reduced to 6 inches in diameter. The flow through the six-inch ports is nearly uniform for all ports, so these ports can be modeled as a port group. The head loss is well below the available head loss for the outfall.

The dilution model was run using the appropriate flows for the modified port configuration. The 6-inch ports and 12.5-inch port are run separately and the dilution is calculated as the flux averaged dilution. The results are shown in Table 6-2 for the standard modeling approach. For the evaluation where the oppositely directed side ports do not merge (realistic modeling approach) the results are provided in Table 6-3. The results for both modeling approaches are shown in Figure 6-1.

[20] It is noted that the Scoping Studies are investigating means of reducing flows under peak flow conditions, which would result in increased dilutions overall.

Table 6-1. Tafuna Diffuser Hydraulics - Modified Configuration with End Gate Port								
Effluent Flow (mgd)	Port Velocity (ft/sec)							Port Head Loss (feet)
	Port 1	Port 2	Port 3	Port 4	Port 5	Port 6	Port 7	
	12.5"	6"						
1	0.62	0.88	0.87	0.86	0.85	0.85	0.88	0.02
2	1.24	1.76	1.74	1.72	1.70	1.68	1.75	0.08
3	1.86	2.65	2.61	2.58	2.55	2.52	2.62	0.17
4	2.49	3.54	3.49	3.45	3.40	3.36	3.49	0.31
5	3.11	4.42	4.36	4.31	4.26	4.20	4.36	0.48
6	3.73	5.31	5.24	5.17	5.11	5.03	5.22	0.68
7	4.35	6.20	6.11	6.04	5.96	5.87	6.09	0.93
8	4.98	7.08	6.99	6.90	6.81	6.71	6.95	1.21
9	5.60	7.97	7.86	7.77	7.66	7.54	7.82	1.53
10	6.22	8.86	8.74	8.63	8.51	8.38	8.69	1.88
	Port Flow (mgd)							
	Port 1	Port 2	Port 3	Port 4	Port 5	Port 6	Port 7	
1	0.34	0.11	0.11	0.11	0.11	0.11	0.11	
2	0.68	0.22	0.22	0.22	0.22	0.21	0.22	
3	1.03	0.34	0.33	0.33	0.32	0.32	0.33	
4	1.37	0.45	0.44	0.44	0.43	0.43	0.44	
5	1.71	0.56	0.55	0.55	0.54	0.53	0.55	
6	2.06	0.67	0.66	0.66	0.65	0.64	0.66	
7	2.40	0.79	0.78	0.77	0.76	0.75	0.77	
8	2.74	0.90	0.89	0.88	0.86	0.85	0.88	
9	3.08	1.01	1.00	0.99	0.97	0.96	0.99	
10	3.43	1.12	1.11	1.10	1.08	1.06	1.10	
	Pipe Velocity Upstream of Port (ft/sec)							Pipe Head Loss (feet)
	Port 1	Port 2	Port 3	Port 4	Port 5	Port 6	Port 7	
1	0.17	0.22	0.28	0.33	0.38	0.44	0.49	0.002
2	0.34	0.45	0.56	0.66	0.77	0.88	0.98	0.008
3	0.51	0.67	0.83	1.00	1.16	1.31	1.48	0.017
4	0.67	0.90	1.11	1.33	1.54	1.75	1.97	0.028
5	0.84	1.12	1.39	1.66	1.93	2.19	2.46	0.041
6	1.01	1.34	1.67	1.99	2.31	2.63	2.95	0.056
7	1.18	1.57	1.95	2.33	2.70	3.07	3.45	0.074
8	1.35	1.79	2.23	2.66	3.09	3.51	3.94	0.094
9	1.52	2.02	2.51	2.99	3.47	3.94	4.43	0.115
10	1.69	2.24	2.79	3.33	3.86	4.38	4.92	0.139

Table 6-2. Diffuser Performance for Tafuna Modified Diffuser with End Gate Port for Standard Modeling Approach

Flow (mgd)	Froude Number		Trapping Level[1] (m)		Maximum Rise Level[2] (m)		Flux Average Dilution		Lmz[3] (m)	
							Trapping Level (m)	Maximum Rise (m)	Trapping Level (m)	Maximum Rise (m)
	12.5"	6"	12.5"	6 "	12.5"	6 "				
1	0.65	1.31	RD[4]							
2	1.30	2.63	RD[4]	3.41	RD[4]	27.02	420.74[5]	480.82[5]	RD/ 15.56	RD/ 19.84
4	2.61	5.23	2.96	2.69	26.82	27.18	272.46	310.52	15.08/ 15.53	19.01/ 19.15
6	3.92	7.84	2.75	2.38	26.96	27.11	204.76	229.86	16.32/ 16.31	19.51/ 18.59
8	5.22	10.46	2.44	1.93	26.96	27.04	171.86	188.65	17.90/ 17.64	20.27/ 19.07
10	6.53	13.07	2.36	1.81	23.22	26.89	132.65	139.78	19.46/ 18.86	21.10/ 20.01

[1] Trapping level is the distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] Lmz is the distance of the edge of the plume from the discharge point. The upper number is for the end gate port and the lower number is for the side ports.
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.
[5] The dilution through the end port (12.5-inch port) was estimated by trend line analysis.

Table 6-3. Diffuser Performance for Tafuna Modified Diffuser with End Gate Port: Evaluated for Realistic Merging Behavior

Flow (mgd)	Froude Number		Trapping Level[1] (m)		Maximum Rise Level[2] (m)		Flux Average Dilution		Lmz[3] (m)	
							Trapping Level (m)	Maximum Rise (m)	Trapping Level (m)	Maximum Rise (m)
	12.5"	6"	12.5"	6 "	12.5"	6 "				
1	0.65	1.31	RD[4]							
2	1.30	2.63	RD[4]	3.82	RD[4]	25.72	612.41[5]	684.22[5]	RD/ 18.06	RD/ 20.11
4	2.61	5.23	2.96	3.32	26.82	27.14	382.66	438.69	15.08/ 17.36	19.01/ 23.52
6	3.92	7.84	2.75	3.06	26.96	27.28	286.86	325.14	16.32/ 17.96	19.51/ 23.91
8	5.22	10.46	2.44	2.83	26.96	27.16	238.91	265.92	17.90/ 18.88	20.27/ 22.78
10	6.53	13.07	2.36	2.58	23.22	27.12	217.32	236.20	19.46/ 20.08	21.10/ 23.25

[1] Trapping level is the distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] Lmz is the distance of the edge of the plume from the discharge point. The upper number is for the end gate port and the lower number is for the side ports.
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.
[5] The dilution through the end port (12.5-inch port) was estimated by trend line analysis.

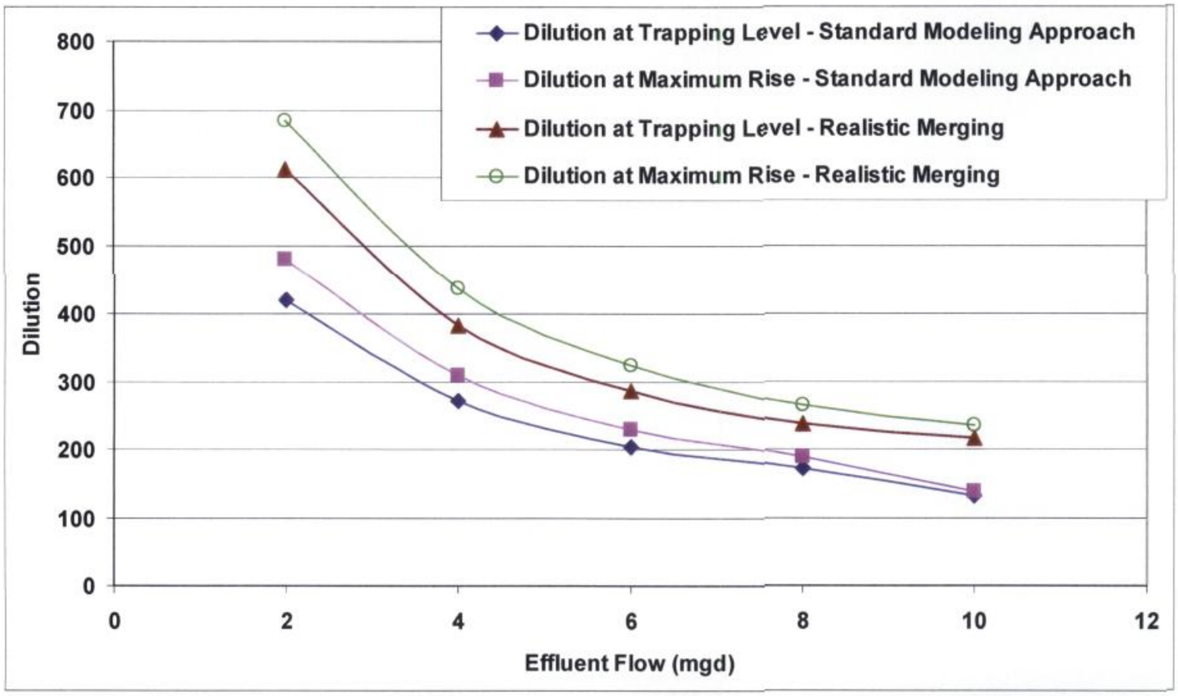

Figure 6-1. Dilution for Modified Tafuna Diffuser Configuration with End Gate Port

6.2 Modified Utulei Diffuser Configuration

As noted above the existing total port area of the Utulei diffuser appears to be sufficient to be assured that persistent seawater intrusion is not an issue. However, it appears that the total port area can be increased somewhat. Therefore, the approach here is to reduce the sizes of the existing ports by installing orifice plates and adding a larger port on the end gate. The total port area is increased but kept to about 50 percent of the barrel cross sectional area. As a representative example reducing the existing ports from 6 inches to 5.5 inches in diameter with an end port of 10.5 inches in diameter is considered. The hydraulic characteristics of this diffuser configuration are shown in Table 6-4, where Port 1 is the 10.5-inch diameter end port and Ports 2 through 7 are the existing ports reduced to 5.5 inches in diameter. It is noted that slight variations in port diameter, for example keeping the 6-inch ports as is and using a slightly smaller end gate port may be as effective as the configuration considered (this can be considered during final design of the port diameter chages). The flow through the 5.5-inch ports is nearly uniform for all ports, so these ports can be modeled as a port group. The head loss is well below the availbe head loss for the outfall.

The dilution model was run using the appropriate flows for the modified port configuration. The 5.5-inch ports and 10.5-inch port are run separately and the dilution is calculated as the flux averaged dilution. The results are shown in Table 6-4 for the standard modeling appraoch. For the realistic merging approach, where the oppositely directed side ports do not mergethe results are provided in Table 6-5. The results for both modeling approaches are shown in Figure 6-2.

Table 6-4. Utulei Diffuser Hydraulics - Modified Configuration with End Gate Port

Effluent Flow (mgd)	Port Velocity (ft/sec)							Port Head Loss (feet)
	Port 1	Port 2	Port 3	Port 4	Port 5	Port 6	Port 7	
	10.5"	5.5"						
1	0.84	1.07	1.06	1.05	1.04	1.04	1.06	0.03
2	1.68	2.14	2.12	2.10	2.09	2.07	2.11	0.13
3	2.52	3.22	3.18	3.16	3.13	3.11	3.16	0.28
4	3.35	4.29	4.25	4.21	4.18	4.14	4.22	0.50
5	4.19	5.36	5.31	5.27	5.22	5.18	5.27	0.77
6	5.03	6.44	6.37	6.33	6.27	6.21	6.32	1.11
7	5.86	7.51	7.44	7.38	7.32	7.25	7.37	1.51
8	6.70	8.59	8.50	8.44	8.36	8.28	8.43	1.97
9	7.54	9.66	9.57	9.49	9.41	9.32	9.48	2.48
10	8.37	10.73	10.63	10.55	10.46	10.35	10.53	3.06
	Port Flow (mgd)							
	Port 1	Port 2	Port 3	Port 4	Port 5	Port 6	Port 7	
1	0.33	0.11	0.11	0.11	0.11	0.11	0.11	
2	0.65	0.23	0.23	0.22	0.22	0.22	0.23	
3	0.98	0.34	0.34	0.34	0.33	0.33	0.34	
4	1.30	0.46	0.45	0.45	0.45	0.44	0.45	
5	1.63	0.57	0.57	0.56	0.56	0.55	0.56	
6	1.95	0.69	0.68	0.67	0.67	0.66	0.67	
7	2.28	0.80	0.79	0.79	0.78	0.77	0.79	
8	2.60	0.92	0.91	0.90	0.89	0.88	0.90	
9	2.93	1.03	1.02	1.01	1.00	0.99	1.01	
10	3.25	1.14	1.13	1.12	1.11	1.10	1.12	
	Pipe Velocity Upstream of Port (ft/sec)							Pipe Head Loss (feet)
	Port 1	Port 2	Port 3	Port 4	Port 5	Port 6	Port 7	
1	0.16	0.22	0.27	0.33	0.38	0.44	0.49	0.002
2	0.32	0.43	0.55	0.66	0.77	0.87	0.99	0.006
3	0.48	0.65	0.82	0.98	1.15	1.31	1.48	0.012
4	0.64	0.87	1.09	1.31	1.53	1.75	1.97	0.020
5	0.80	1.08	1.36	1.64	1.91	2.19	2.46	0.029
6	0.96	1.30	1.64	1.97	2.30	2.62	2.96	0.040
7	1.12	1.52	1.91	2.30	2.68	3.06	3.45	0.053
8	1.28	1.73	2.18	2.62	3.06	3.50	3.94	0.067
9	1.44	1.95	2.45	2.95	3.45	3.93	4.43	0.082
10	1.60	2.17	2.73	3.28	3.83	4.37	4.92	0.099

Table 6-4. Diffuser Performance for Utulei Modified Diffuser with End Gate Port for the Standard Modeling Approach

Flow (mgd)	Froude Number		Trapping Level[1] (m)		Maximum Rise Level[2] (m)		Flux Average Dilution		L_{ZID}[3] (m)	
	1 Port 10.5"	6 Ports 5.5"	1 Port 10.5"	6 Ports 5.5"	1 Port 10.5"	6 Ports 5.5"	Trapping Level (m)	Maximum Rise (m)	Trapping Level (m)	Maximum Rise (m)
1	RD[4]									
2	RD[4]	3.33	RD[4]	21.57	RD[4]	34.88	341.02[5]	519.83[5]	RD/ 17.67	RD/ 28.69
4	3.82	6.66	19.61	12.55	34.85	43.79	287.85	417.26	18.99/ 22.34	29.24/ 37.20
6	5.73	10.00	14.90	6.41	39.20	45.64	265.68	350.22	23.61/ 26.23	30.11/ 38.41
8	7.65	13.33	13.67	5.42	41.05	45.32	228.20	298.22	25.59/ 27.32	32.47/ 33.29
10	9.56	16.66	12.86	4.65	42.03	45.90	203.56	265.56	27.23/ 27.71	40.96/ 34.13

[1] Trapping level is the distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] Lmz is the distance of the edge of the plume from the discharge point. The upper number is for the end gate port and the lower number is for the side ports.
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.
[5] The dilution through the end port (10.5-inch port) was estimated by trend line analysis.

Table 6-5. Diffuser Performance for Utulei Modified Diffuser with End Gate Port Evaluated for Realistic Merging Behavior

Flow (mgd)	Froude Number		Trapping Level[1] (m)		Maximum Rise Level[2] (m)		Flux Average Dilution		Lmz[3] (m)	
	1 Port 10.5"	3 Ports 5.5"	1 Port 10.5"	3 Ports 5.5"	1 Port 10.5"	3 Ports 5.5"	Trapping Level (m)	Maximum Rise (m)	Trapping Level (m)	Maximum Rise (m)
1	RD[4]									
2	RD[4]	3.33	RD[4]	29.83	RD[4]	23.35	376.35[5]	581.91[5]	RD/ 13.59	RD/ 20.04
4	3.82	6.66	19.61	21.75	34.85	34.60	333.96	499.31	18.99/ 19.85	29.24/ 28.29
6	5.73	10.00	14.90	17.15	39.20	40.69	306.64	434.75	23.61/ 23.60	30.11/ 39.69
8	7.65	13.33	13.67	14.07	41.05	43.11	272.20	376.72	25.59/ 26.25	32.47/ 38.50
10	9.56	16.66	12.86	12.05	42.03	43.98	246.94	267.59	27.23/ 28.35	40.96/ 43.08

[1] Trapping level is the distance below the surface of the plume centerline
[2] Maximum rise is the distance above the discharge point of the plume centerline
[3] Lmz is the distance of the edge of the plume from the discharge point. The upper number is for the end gate port and the lower number is for the side ports.
[4] RD indicates the model run was discontinued because the Froude number was below a critical value and calculations may not be reliable.
[5] The dilution through the end port (10.5-inch port) was estimated by trend line analysis.

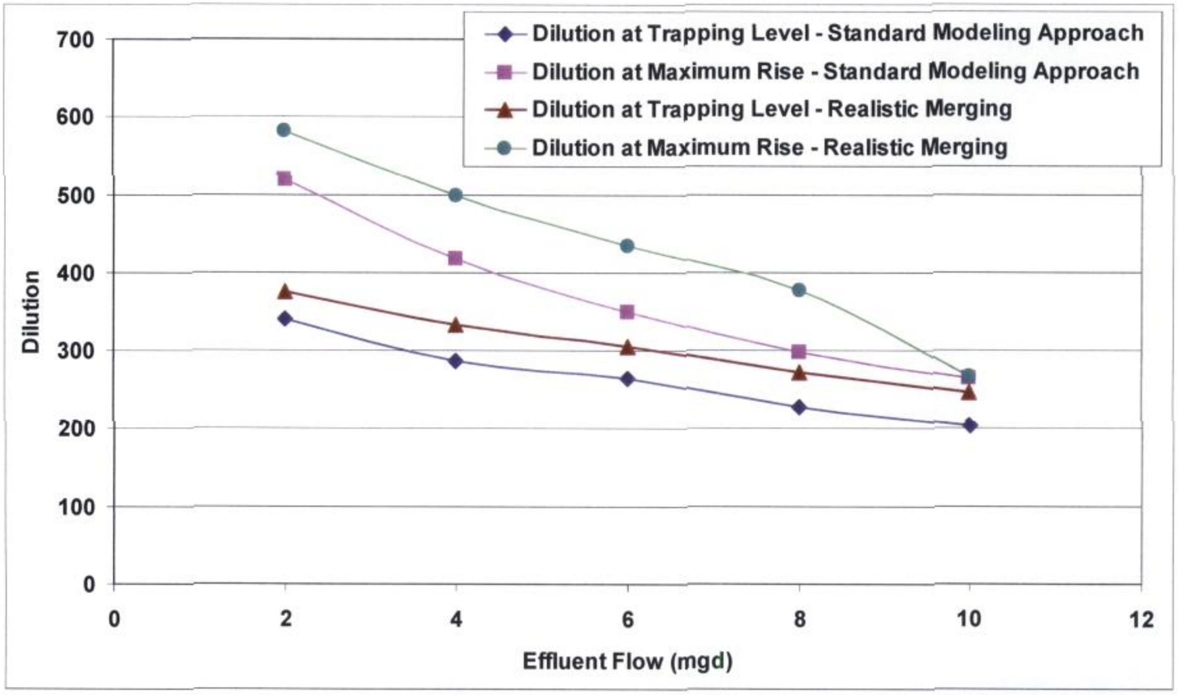

Figure 6-2. Dilution for Modified Utulei Diffuser Configuration with End Gate Port

7 Analysis of Alternatives

This section ranks the alternatives considered in terms of feasibility, effectiveness, constructability, and cost. Constructability includes potential permitting issues that would increase both costs and time needed for implementation.

7.1 Alternatives Considered

Four primary alternatives were considered in the evaluation of diffuser performance addressed in this TM. In each case the modeling was done using a "standard approach" that considered all ports on one side of the diffuser and a "modified" approach that artificially increased the spacing between the ports to better reflect the actual merging behavior of plumes from individual ports. *Based on experience and judgment, including a number of field studies referenced in this TM, it is believed that the modified modeling approach provides a more realistic and still conservative (under prediction) of actual dilution and should be used to evaluate the relative performance of the various diffuser configurations and the dilution as a function of flow described in this TM.* In addition the current was always considered collinear with the discharge direction, except for the end ports in the modified existing diffuser configuration analysis. This is required because of the dilution model constraints and leads to conservative (underestimation) of the actual dilution. Whether or not EPA accepts the modified modeling approach, and regardless of whether the dilution at the trapping level or at maximum plume rise is used for compliance calculations, the *relative* ranking of diffuser performance remains the same. The alternatives considered are:

- Base case diffusers using the existing diffuser configuration (assuming some minor repairs to the Utulei diffuser such as removing the orifice plate on one of the diffuser ports and repairing the leaks around some of the risers.)
- Doubling the length of the diffusers with the same number of ports, which effectively increases port spacing
- Doubling the length of the diffusers with twice the number of existing ports, which would result in ports being smaller to maintain an acceptable total port area
- Modifying the existing diffuser by reducing the size of the existing ports and adding a terminal port on the end gate directed seaward.

7.2 Effectiveness of Alternatives Considered

The primary measure of effectiveness in the context of the requirements of the AOs is the ability to reduce nitrogen and phosphorus to acceptable levels accounting for the initial dilution of the effluent. Conversations with USEPA indicate that the desired level of effectiveness is the reduction of *effluent derived* concentrations to the ASWQS criteria. The water quality standards for total nitrogen and total phosphorous are different in the receiving waters affected by the Tafuna and Utulei WWTPs discharge, which dictates the dilution required to achieve compliance with ASWQS.[21]

7.2.1 Tafuna WWTP

The Tafuna WWTP discharges into open coastal waters. The criterion for nitrogen is 0.130 mg/l. The criterion for phosphorous is 0.015 mg/l. These criteria are based on averages, but have been applied as single point criteria by ASEPA and USEPA in the past. The single point approach does not account for natural variability of the POCs in the receiving water. The effluent concentration that will be reduced to these levels as a function of dilution is shown in Figure 7.1. The vertical axis on Figure 7-1 shows a range of effluent concentration of nitrogen and phosphorous (upper and lower panels, respectively). The horizontal axis is the dilution required to reduce the effluent-derived concentrations to the current ASWQS criterion. For example an effluent concentration of nitrogen of 40 mg/l would require a dilution of 308 to reduce the effluent derived concentration to 0.13 mg/l.

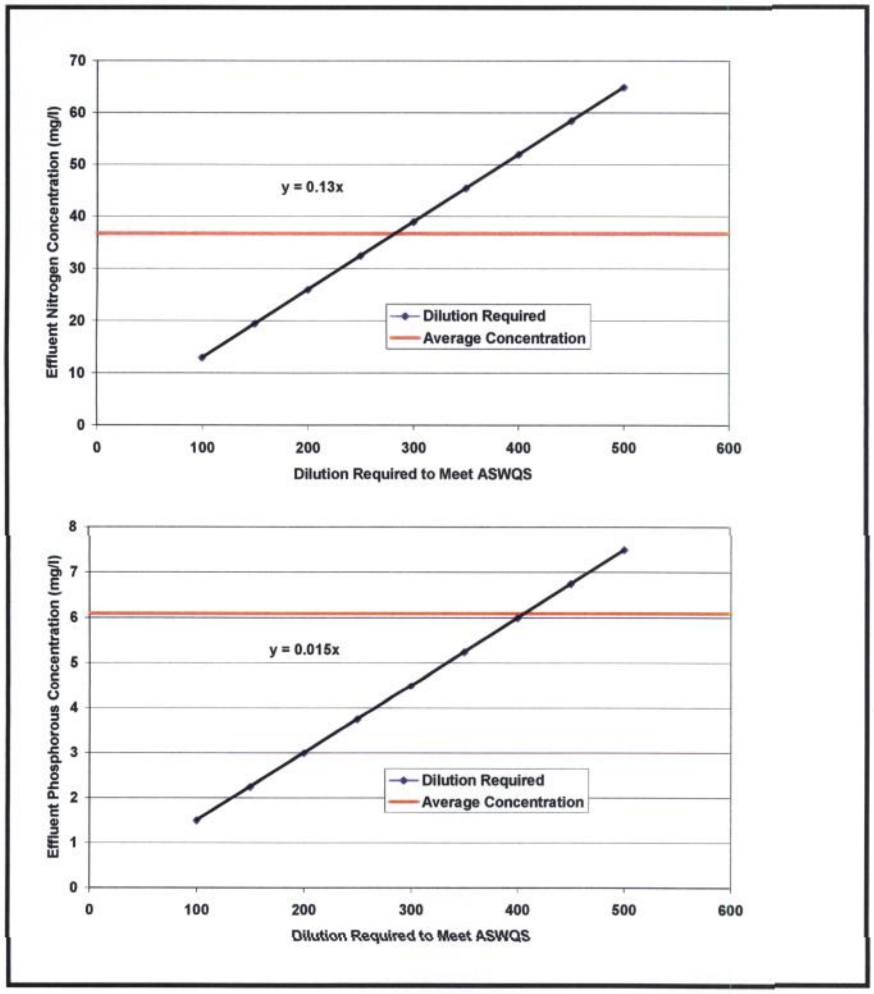

Figure 7-1. Dilution Required to Reduce Effluent-derived Concentrations to the WQS for Tafuna

[21] The ASWQS are currently under review. If the criteria for nitrogen and phosphorous are modified as anticipated the overall evaluation of diffuser configurations should not change, although it is expected that many ,if not all, of the past exceedances would not be exhibited.

There is little data on the effluent values for these POCs. There are five months of recently initiated monitoring data. These data show total nitrogen values of 25 mg/l to 44 mg/l with an average of about 37 and total phosphorus data values of 3.9 to 7.6 with an average of 6.1. The five-month average values are indicated as the horizontal line on Figure 7-1.

The average daily flow at the Tafuna WWTP from 2006 through 2011 was 1.72 mgd; the highest monthly average flow for the same time period was 2.02 mgd. As a preliminary evaluation of the effectiveness of the diffuser configurations considered in this TM, the dilutions required (Figure 7-1) can be compared to dilutions predicted for an effluent flow of 4 mgd (at the maximum rise height of the plume), which are presented in Table 7-1. It is noted that the dilutions are estimated for critical conditions, not average conditions. However, there is little variation in marine water conditions (density profiles or expected 10 percentile currents) in American Samoa and the conditions used in the modeling are likely representative of typical conditions.[22]

Table 7-1 Predicted Dilution for the Tafuna Diffuser at an Effluent Flow of 4 mgd at Maximum Plume Rise Height

Diffuser Configuration	Predicted Dilution
Base Case - Standard Modeling Approach	186.70
Base Case - Realistic Merging	322.56
Increased Diffuser length - Six Ports - Standard Modeling Approach	322.56
Increased Diffuser length - Six Ports - Realistic Merging	524.76
Increased Diffuser length - Twelve Ports - Standard Modeling Approach	323.62
Increased Diffuser length - Twelve Ports - Realistic Merging	558.67
Modified Diffuser (Seven Ports) - Standard Modeling Approach	310.52
Modified Diffuser (Seven Ports) - Realistic Merging[1]	438.69

[1] This is the recommend approach: modify existing diffusers, use maximum rise and realistic merging behavior for evaluations and compliance.

The increased diffuser length configurations and the modified diffuser with the end gate port all appear to be effective in reducing effluent-derived concentrations to the ASWQS based on the limited effluent data available (when considering the modified, realistic merging modeling approach). The increased length configuration appears to be somewhat more effective. However, it is likely that the modified configuration with end gate port presented in this TM can be adjusted (port sizes changed) to increase dilution somewhat and may ultimately be as effective as the increased length configuration.

7.2.2 Utulei WWTP

The Utulei WWTP discharges into Pago Pago Harbor. The criterion for nitrogen is 0.250 mg/l. The criterion for phosphorous is 0.030 mg/l. These criteria are based on averages, but have been applied as single point criteria by ASEPA and USEPA in the past. The effluent concentration that will be reduced to these levels as a function of dilution is shown in Figure 7.2. The vertical axis on Figure 7-2 shows a range of effluent concentration of nitrogen and phosphorous (upper and lower panels, respectively). The horizontal axis is the dilution required to reduce the effluent-derived concentrations to the current ASWQS

22 The exception to this may be the variation in current speeds at the Tafuna WWTP discharge site.

criterion. For example an effluent concentration of nitrogen of 20 mg/l would require a dilution of 80 to reduce the effluent derived concentration to 0.25 mg/l.

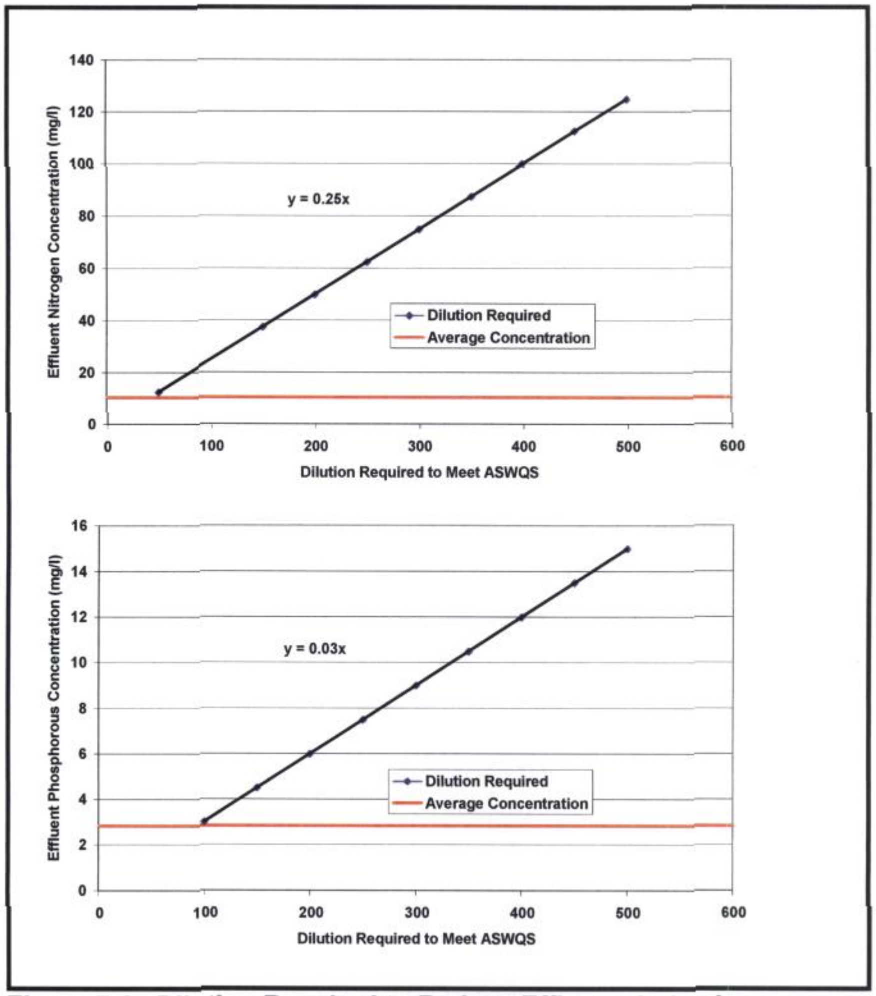

Figure 7-2. Dilution Required to Reduce Effluent-derived Concentrations to the WQS for Utulei

There is little data on the effluent values for these POCs. There are five months of recently initiated monitoring data. These data show total nitrogen values of 6.2 mg/l to 15 mg/l with an average of about 10.5 and total phosphorus data values of 1.3 to 4.9 with an average of 2.9. The five-month average values are indicated as the horizontal line on Figure 7-2. The average daily flow at the Utulei WWTP from 2006 through 2011 was 1.35 mgd with highest monthly average of 1.92 mgd. Both nitrogen and phosphorous water quality standards are based on average values. As a preliminary evaluation of the effectiveness of the diffuser configurations considered in this TM, the dilutions required (Figure 7-2) can be compared to dilutions predicted for an effluent flow of 4 mgd (at the maximum rise height of the plume), which are presented in Table 7-2. It is noted that the dilutions are estimated for critical conditions, not average conditions. However, there is little variation in marine water conditions (density profiles or expected 10 percentile currents) in American Samoa and the conditions used in the modeling are likely representative of typical conditions.

Table 7-2 Predicted Dilution for the Utulei Diffuser at an Effluent Flow of 4 mgd at Maximum Plume Rise Height	
Diffuser Configuration	**Utulei**
Base Case - Standard Modeling Approach	**222.52**
Base Case - Realistic Merging	344.14
Increased Diffuser length - Six Ports - Standard Modeling Approach	345.21
Increased Diffuser length - Six Ports - Realistic Merging	464.25
Increased Diffuser length - Twelve Ports - Standard Modeling Approach	343.10
Increased Diffuser length - Twelve Ports - Realistic Merging	438.74
Modified Diffuser (Seven Ports) - Standard Modeling Approach	417.26
Modified Diffuser (Seven Ports) - Realistic Merging[1]	499.31
[1] This is the recommend approach: modify existing diffusers, use maximum rise and realistic merging behavior *for evaluations and compliance.*	

All configurations considered, including the current diffuser configuration, appear to be effective in reducing effluent-derived concentrations to the ASWQS based on the limited effluent data available. The modified diffuser configuration presented in this TM appears to be the most effective overall.

7.3 Constructability and Maintenance

All alternative diffuser configurations are constructible using standard techniques but with wide variation in cost. Extending the diffusers is obviously considerably more difficult than modifying the existing diffusers because of extended dive time needed to place and secure additional pipe lengths. Modifications of the existing diffusers are remove-and-replace operations that can be done with far less overall effort and substantially less required dive time than adding a new diffuser section. Maintenance of an extended diffuser is more difficult to some extent, particularly at the Utulei discharge site because of water depths. Modifications involving installation of port orifices and replacing the existing end gate (blind flange) can also be done in tandem with outfall pigging operations necessary to clean outfalls and ensure proper hydraulic conditions exist along with any other proposed work. Thus, some cost savings should be realized by combining this work.

7.4 Habitat Disturbance

There exists a potential for habitat disturbance by adding diffuser extensions to the existing diffusers. The extent of this disturbance is not likely severe because of the depth of the Utulei outfall and the mostly sandy bottom in the locations where the both extensions would occur. However, there would be disturbances of the benthic communities during construction activities. Modification of the existing diffusers as recommended would have far less potential for habitat disturbance because the operations involved would take less time and involve only the existing structures.

It is also recognized that extending the diffusers results in larger volumes of water subjected to effluent-derived constituents. The objective should be to minimize the size of the mixing zone while maximizing the initial dilution performance of the diffusers.

7.5 Permitting

Outfall extensions, because of the potential for habitat disturbance would require some level of permitting effort. At a minimum an Environmental Assessment would likely be required. Modifying the existing diffusers would normally be considered a maintenance activity and not require any permitting action.

7.6 Planning Level Costs of Alternatives

The cost of using the existing diffusers is limited to some minor repairs and future maintenance. The cost of extending the diffusers (doubling the lengths of the diffusers) is substantial regardless of the approach used in terms of number and size of ports. The cost, including design, permitting, and construction is likely in the range of $600K to $1,000K each, with higher costs probably associated with the Utulei site, although weather contingencies for the Tafuna site could raise the cost at this site substantially. The cost of modifying the existing diffusers is likely about $100K to $150K for each site. However, if the diffuser modifications were conducted concurrently with diffuser inspection and maintenance operations the marginal costs of diffuser modification would be substantially lower. Estimated costs by major category are provided in Table 7-3

Table 7-3. Cost Estimate in $1000 for Diffuser Alternatives - Cost for Each Site

Cost Element	Existing Diffuser Repairs and Maintenance	Extended Length Diffusers	Modify Existing Diffusers
Design	0	30	5
Fabrication[1]	5	50	15
Materials[2]	5	0	0
Shipping[3]	1	2	0.5
Site Mobilization	6	10	1
Diver Mobilization[4]	60	0	0
Site Preparation (diver)	0	36	12
Installation (diver)	24	240	24
Contingency (weather)	24	168	48
Permitting and Environmental[5]	0	80	1
Total[6]	125	616	106.5
Minus 10%	138	554	96
Plus 50%	189	924	160

[1] Utulei only for Existing case
[2] Materials for maintenance is included in existing diffuser repairs and cleaning
[3] From Hawaii
[4] Assume work done concurrently with pigging and repairs to existing diffusers
[5] Does not include NPDES Related Permitting
[6] In each case total cost will be Existing Repairs PLUS Modifications

The costs presented above are planning level costs based on previous experience in American Samoa and should be considered only in relative (order of magnitude) terms.

7.7 Time Required for Construction

It is estimated that design, permitting, and construction of outfall extensions would require a minimum of 18 months to two years, or possibly longer (Note that the AOs require project completion within one year). Modifications of the existing diffusers could probably be accomplished within a matter of months and likely no longer than a year once the decision to proceed is made.

7.8 Evaluation of Impacts of Alternatives

Table 7-2 presents an initial screening of the relative attributes of the various alternatives (effectiveness is discussed separately above in Section 7.2). The two approaches to outfall extension are considered together because they are quite similar and would be ranked the same. The best overall alternative, based on the screening criteria in Table 7-2 is the no action alternative based on the project ranking categories. However, this would not improve dilution. Of the remaining alternatives, based on the effectiveness of the proposed modifications discussed in Section 7.2, modification of the existing alternative is clearly superior to extending the diffusers.

Table 7-2. Evaluation of Diffuser Modification Alternatives

Category	Tafuna WWTP			Utulei WWTP		
	Existing Diffuser Configuration	Double Diffuser Length	Modify Existing Diffuser	Existing Diffuser Configuration	Double Diffuser Length	Modify Existing Diffuser
Construction Difficulty	0	2	1	0	2	1
Habitat Disturbance	0	2	1	0	2	1
Permitting Difficulty	0	3	1	0	3	1
Relative Cost	1	3	1	1	3	1
Time Required	1	3	2	1	3	2
Total	2	13	6	2	13	6

0 = No impact

1 = Minor Impact

2 = Significant Impact

3 = Substantial Impact

7.9 Conclusions and Recommendations

Based on the analyses presented in this TM it appears clear that *the modification of the existing diffusers (addition of an end gate port and reduction of the existing port diameters with orifice plates) is the best overall approach for both outfalls.* The following specific recommendations are indicated:

- Because dilutions are greater and believed to be a more consistent with observed diffuser behavior, it is recommended that the scoping projects targeted at reducing effluent flows evaluate dilution improvements based on the modified diffuser configurations, with dilution predicted at maximum rise and the realistic merging approach. It is further recommended that future effluent limitations and compliance assessments be based on the dilutions predicted at maximum rise using the realistic merging modeling approach.

- Modification of the Tafuna diffuser with a 12.5-inch end gate port and restriction of the existing port areas from 8-inches to 6-inches presented in this TM should be considered and implemented as presented in Section 4.1. Additional investigation of the Tafuna diffuser configuration could be conducted to further refine the optimum port configurations and increase the dilution somewhat by adjusting port sizes. This would require additional time and may not be substantially different than what is now recommended.

- Modification of the Utulei diffuser with a 10.5-inch end gate port and restriction of the existing port areas from 6-inches to 5.5-inches has been presented. Although the existing diffuser configuration is expected to provide sufficient dilution to reduce effluent-derived POC concentrations to below the ASWQS criteria, the addition of an end gate port to further increase dilution is recommended. Additional analyses of port sizes is unlikely to result in significantly increased dilution but could be conducted to determine if restriction of the side ports is necessary. It is noted that, dilution predicted using reasonable ambient conditions and the realistic modeling approach is substantially higher that the dilution currently used to evaluate compliance with ASWQS.

  Significant inflow and infiltration is suspected in the Bay area collection system that feeds the Utulei outfall and identifying and repairing this should be considered a priority in tandem with diffuser modifications (see the appropriate sections of the CVL Scoping Report). Recent data (limited to a single measurement) indicate that seawater intrusion into the Utulei collection system may be significant. Increased effluent density will reduce dilution and the recommended modification of the diffuser will offset this effect.

- Modifications of both diffusers as recommended above should be conducted during routine maintence operations to minimize costs. In particular it is recommended that both diffusers be cleaned to remove any sediment and debris that has accumulated during the lifetime of these structures. Outfall structures should also be protected from the potential introduction of large debris as part of any initial work. Diffuser modifications can be easily implemented during this operation. Additional cost savings can be realized if ASPA coordinates operations with planned inspections and repairs to the Joint Cannery outfall.

- Additional temperature and conductivity (salinity) measurements should be made at the landside effluent discharge structures to better define the effluent density. This will facilitate more accurate dilution predictions for future permitting purposes. It is noted that changes in effluent density *will not significantly affect the relative ranking* of the various alternatives considered in this TM.

- Ambient density profiles should be collected routinely at the outfall discharge sites (particularly the Tafuna discharge site) during receiving water monitoring (this is not currently required in the current NPDES permits). These profiles will be useful to more accurately predict dilution. In addition, existing profile data should be compiled and examined to determine the range of dilutions expected for future permitting purposes. It is noted that variations in ambient density *will not significantly affect the relative ranking* of the various alternatives considered in this TM. Within the context of this recommendation ASPA should also consult with USEPA and ASEPA to reach agreement on the appropriate background sites for such density profiles.

ASSOCIATED UNDERWATER SERVICES

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March 15, 2012

American Samoa Power Authority
Pago Pago, AS 96799
Via email: Robert@aspower.com

Attention: Mr. Robert Kerns, P.E., Senior Environmental Engineer
Subject: Final Inspection Report of Fogagogo WWTP Outfall – REVISION 2

Dear Mr. Kerns:

This report documents the underwater inspection of the Fogagogo (Tafuna) WWTP outfall performed by Associated Underwater Services (AUS) on December 10 & 11, 2011.

Underwater Inspection Summary

The Fogagogo WWTP outfall is a 24-inch diameter HDPE pipeline that extends approximately 1,600 feet from the treatment plant and is anchored to the ocean floor with a steel anchorage system attached to concrete weights placed on coral reef, sand and rock. The outfall is located on the south side of Tutuila Island adjacent the Pago Pago International Airport runway. The outfall terminates at a depth of approximately 95 feet with effluent released through six, eight-inch diameter diffusers spaced over a length of 60 feet. The outfall replaced a 12-inch diameter iron pipeline that remains within close proximity to the replacement outfall.

Inspection Method:

This underwater inspection was performed using surface supplied diving equipment with a helmet mounted video camera. The diver was tended from a diving support boat which was secured to the outfall in two locations. For purposes of operational efficiency and safety, the inspection began at the deep (offshore) end of the outfall and proceeded toward the shore. The inspection diver reached as far inshore as safety and swell conditions would allow. The diver and the diving supervisor were communicating in real time and the video was recorded in DVD format.

Report of Findings:

A) Outfall diffuser section (blind flange to block 4):

This section of the outfall consists of the blind flange, 6 gooseneck diffuser ports, 4 anchor blocks and another flange that connects the diffuser sections to the rest of the outfall. Both of the flanges are 20 bolt flanges with 1 ¼" diameter galvanized bolts. The total length of the diffuser section is 61 feet. The hardware on the blind flange and the flange that connects the diffuser section with the rest of the pipe is in good condition. These flanges are protected by two 24 lbs. anodes on each side that are in good condition with approximately 60-70% remaining. The blind flange is partially buried in the sandy bottom from the 2-10 o'clock position. Each gooseneck riser extends directly up from the top of the outfall 28" high, and they are approximately 10 feet apart. The depth of each diffuser is measured from the base of orifice (top of pipe) and indicated on the attached drawing. The six gooseneck diffusers risers on the diffuser section of the pipe are intact, and have strong and equal discharge of effluent flow. The gooseneck risers have 8" diameter openings and they are orientated in alternating east-west direction. We did not observe any debris or obstructions in the diffuser openings. The anodes that sit on top of the anchor block and protect the all thread rod that hold the blocks together are in good condition with approximately 90% remaining on them.

The offshore end of the outfall sits in 96 feet of water. AUS inspected the seafloor 100 feet out from the end of the outfall to a depth of 106 feet and the seafloor consisted of a flat sandy bottom with no obstruction beyond 100 ft. Visibility at the time of the inspection was 80 feet.

B) Block 4 to block 60:

In this area the outfall lies in wide sandy plain. The outfall in this area is not perfectly straight. As it travels offshore it curves moderately to the southwest. The outfall is also buried up the top of the pipe in this area and the anchor blocks are approximately 11 feet apart. The anchor blocks are secured to the seafloor by triangular plates that are attached to turnbuckles, chain and soil anchors into the sand. The tops of the soil anchors in this area were buried in 6-12" of sand and not visible. All the visible hardware in this area was in good condition. The anodes that were installed on top of the anchors blocks in 2007 & 2008 are intact and in excellent condition (90%) remaining. There are boulders and some steel debris touching the outfall at blocks 36 & 51. The boulders can be moved away by chipping them into smaller pieces and rigging to a lift bag to move away from the outfall. The steel debris can be moved by cutting it into small pieces with a cutting torch and moved the same way as the boulders.

C) Block 60 to block 112:

In this area the outfall rises from the sandy plain and climbs up the slope of the reef as it approaches the shore. The depth at block #60 is approximately 50 feet. The anchor blocks in this zone of the outfall have additional anchor hardware because of the increase in wave energy. The anchor blocks are secured with chain, turnbuckles, shackles and anchor bolts into the reef on each side of the outfall. Some anchor blocks from the sand to the top of the reef are missing their anchoring hardware on one or both sides however there is no evidence that hardware was ever installed at these locations. All of the anodes that were installed on top of the anchor blocks in 2007 & 2008 are intact and in excellent condition (90%) remaining. The anodes that were installed on all of the turnbuckles in 2007 & 2008 range from 50% to 90% consumed. The teardrop anodes installed on the soil anchor baseplates are in excellent condition (90%) remaining. There is a connection flange between blocks 69-70 which is also a 20 bolt flange with 1 ¼" galvanized bolts. The flange is heavily encrusted with coral but in good condition and no leaks. The fiber optic cable that was installed just west of the outfall is touching the anchoring hardware in a few locations but is otherwise free of the outfall. Due to sea and swell conditions block #112 (of 123 totals) is the last block inspected during this inspection.

Conclusions:

Overall, the outfall is in good condition and functioning properly. There is no visible damage to the outfall or any damage that may have been caused by the earthquake and Tsunami of 2009. The six gooseneck diffuser risers are in good condition. AUS observed strong and equal flow from the diffusers and did not observe any obstructions in the diffuser openings. The blind flange at the end of the outfall is buried over 50%. The outfall is almost completely buried in the sandy plain area (blind flange to block 60). The seafloor 200 feet beyond the end of the pipe is free of any obstructions that would prevent the extension of the outfall. The anodes that were installed in 2007 & 2008 are intact and in good condition with the exception of the turnbuckle anodes in the reef area (block 60-block 112) The AUS inspection and repair report from 2007 & 2008 details the locations and amount of repairs completed on the outfall.

Recommendations:

1. Replace turnbuckle anodes that have deteriorated past 50%.
2. Consider running a pipeline cleaning device (pipe pig) though the outfall to force debris and encrusted effluent out the pipe. This will increase the flow and operational effectiveness of the outfall.
3. Remove boulder and steel debris touching the pipe at blocks 36 & 51 using a chipping gun and cutting torch to move smaller pieces.
4. Consider replacing chain and turnbuckles that are worn but still connected.
5. Consider replacing chain & turnbuckles that are very loose but still connected.

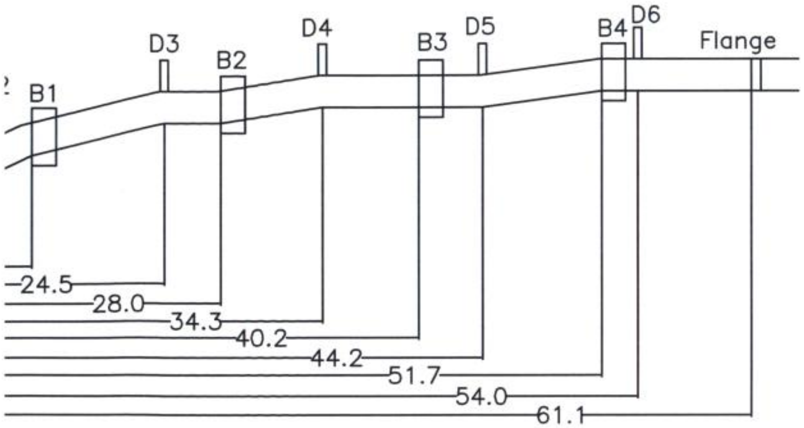
Respectfully Submitted,

David E. Cleary

David E. Cleary
Project Manager

FOGAGOGO WWTP OUTF

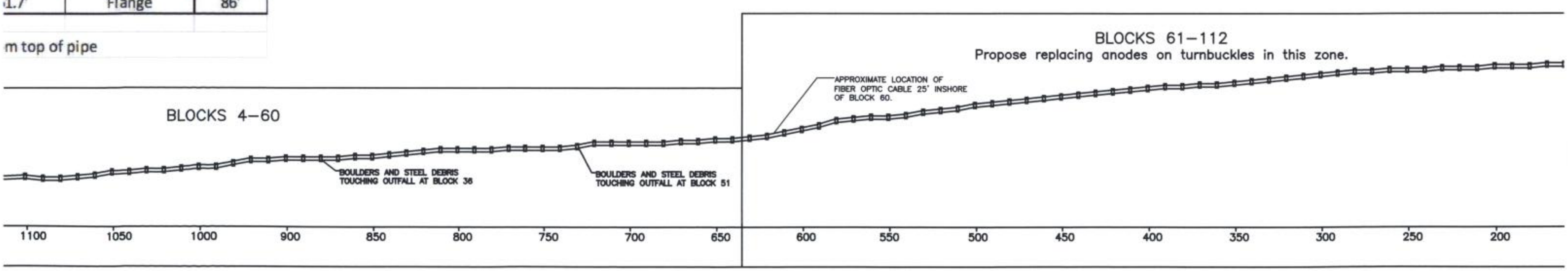
PAGO PAGO, AMERICAN SAMOA
DECEMBER 2011



DIFFUSER SECTION

tance	Anchor/Diffuser #	Depth
-	BF	95'
.2'	D1	94'
4.2'	D2	92'
6.3'	B1	91'
4.5'	D3	90'
28'	B2	89'
4.3'	D4	89'
0.2'	B3	88'
4.2'	D5	88'
1.7'	B4	87'
54'	D6	87'
1.7'	Flange	86'

m top of pipe



FOGAGOGO WWTP OUTFALL

ELEVATION VIEW

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March 6, 2012

American Samoa Power Authority
Pago Pago, AS 96799
Via email: Robert@aspower.com

Attention: Mr. Robert Kerns, P.E., Senior Environmental Engineer
Subject: Final Inspection Report of Utulei WWTP Outfall – Revision 1

Dear Mr. Kerns:

This report documents the underwater inspection of the Utulei WWTP outfall performed by Associated Underwater Services (AUS) on December 8 & 9, 2011

Underwater Inspection Summary

The Utulei outfall is a 24 inch diameter ductile iron and high density polyethylene (HDPE) pipeline extending approximately 1,050 feet from the treatment plant with approximately 700 feet buried in the coral reef. The inshore section of the outfall (WWTP to Flange #2) is ductile iron with the offshore section being HDPE. The outfall is secured to the reef by a unique gimbaled frame that holds the outfall in place on the edge of the reef wall. The outfall drops off the reef wall at a steep angle and touches the harbor bottom 65 feet below the gimbaled frame. The outfall terminates at a depth of approximately 160 feet in a sand/muddy bottom. Effluent is released through five, six-inch diameter diffusers and one four-inch diffuser spaced over a length of 51 feet.

Inspection Method: This underwater inspection was performed using surface supplied diving equipment and a helmet mounted video camera. The diver was tended from a diving support boat which was secured to the outfall in two locations. For purposes of operational efficiency, the inspection was started at the shallow portion of the outfall to the gimbaled frame. Then the diffuser section was inspected, then the sections in between. The diver and the diving supervisor were communicating in real time and the video was recorded in DVD format.

Report of Findings:
A.) Rock over reef area to gimbaled frame:

The outfall in this area consists of backfilled rock over reef. From the first visible anchor block the diver was able to see the outfall 35ft inshore. This point was designated as the outfall zero point for the purposes of this inspection. In this area the outfall enters an excavated trench as it turns and descends gradually toward the gimbaled frame in 21 feet of water. The outfall has a Y fitting at flange #1. The other side of the Y is smaller and is closed by a blind flange (20" diameter). Flange #2 marks the transition of the pipe from ductile iron to HDPE. The hardware on all flanges is in good condition. The offshore side of this flange was replaced in 2007 with a new stainless steel flange ring, rubber gasket and wo new 24lb anodes were installed. These anodes were found to be 100% deteriorated during the 2011 ıspection. The rapid deterioration is possibly caused by the wind wave, swell and tidal action because of their hallow depth. *These deteriorated anodes were replaced during the 2011 inspection. The hardware on flanges* 1 & #2 is in good condition. There is a pipe retraining ring just offshore of flange #2. This ring is secured to the eef by rigging to an anchor plate bolted to the reef. The rigging is in good condition but the anchor plate on the ast side of the pipe is missing (2) bolts and the remaining (2) bolts are in poor condition. There are 2 dditional flanges (#3 & #4) prior to reaching the gimbaled frame. The bolts and anodes on these flanges are lso in good condition. Flanges 1-4 are (20) bolts flanges with 1 1/4" bolts. The hardware on flanges #1&2 is tainless steel. The bolts on flanges #3&4 are galvanized. Just inshore of flanges #3&4 are U shaped pipe rackets that hold the pipe to the side of the coral trench. The brackets, bolts and most of the anodes on these ipe clamps are in good condition. Twenty inches offshore of flange #4 is a double female PVC sleeve fitting. Ve could not determine if this is covering a repair or part of the original construction. Immediately past the leeve fitting, the outfall takes a 45 degree turn north and down.

3.) <u>Gimbaled frame</u>:

he gimbaled frame holds the outfall in place at the edge of the reef cliff and allows the outfall to flex during eismic events. The overall condition of the gimbaled frame is excellent. The bolts, turnbuckles and structural teel show only minor corrosion. There is a hard rubber bellows between the frame and the clamp which is in ood condition. Two bolts are missing on the offshore section of the flange that captures the rubber bellows iece. This is not effecting the function of the bellows and is not recommended for repair at this time. The stud-nk anchor chains that brace the frame to the reef and the anchor points are also in good condition. There are wo beams which the frame of the gimbaled frame sits on. The beams are in good condition, but are leaning ffshore (towards the reef cliff) at approximately15 degree angle. This appears to be a recent movement based n our previous inspections in 2007 and 2008 that may have been caused by the 2009 tsunami. This could resent a problem if the beams continue to move towards the reef cliff.

C.) <u>Gimbaled frame to diffuser section</u>:

From the gimbaled frame (21ft deep) to anchor block #1 (86ft deep), the outfall descends rapidly. Anchor block 1, 2 & 3 are suspended above the harbor bottom because of the angle of the outfall decline. From anchor lock 1- 13 (last block before diffuser section) all of the anodes and connections nuts and bolt are in good ondition with 10-15 % deterioration. There is no damage to the outfall in this area or any debris or rocks ouching it.

D.) <u>Diffuser Section</u>:

There are six diffuser openings on the diffuser section of the outfall. Each diffuser consists of a gooseneck iser that rises directly from the top of the outfall. The discharge opening of the gooseneck risers alternates lirection between east and west. Three of the gooseneck risers were replaced in 2007 with ROMAC repair ooseneck risers. These repaired risers have 6 inch diameter openings and are 16" tall. The remaining 3 ooseneck risers are original and are 27" tall. Diffuser # 2 has a restrictor plate over the opening which reduces he opening to 4" diameter. The depth for each diffuser port was measured from the top of the diffuser. Depths or each diffuser are shown in the chart on the attached drawing. The ROMAC repair gooseneck risers are howing signs of moderate corrosion with one having a small leak at its base. It is likely that the corrosion in his section will continue due to abrasion that is caused from the high velocity of water/debris flowing through he risers. The discharge of effluent from each gooseneck riser was not consistent. Solid debris (bottles & lastics lids) was found to be blocking the openings on gooseneck riser #2 and # 1 respectively. Some solid vaste appears to be getting into the effluent stream and reaching the diffuser. The blind flange that marks the erminus of the outfall is partially buried (30%) but is in good condition. The anodes and the bolts show only ninor corrosion.

:.) Harbor bottom offshore of outfall terminus:

)ur inspection included an inspection of the seafloor 100ft beyond the end of the outfall to 177 feet. Soft sand vas encountered with no large coral head or boulders that could obstruct the outfall if it were to be extended.

:onclusions:

Overall, the outfall is in good condition and functioning properly. There is no visible damage to the outfall. The gimbaled frame is in good condition and appears to have functioned as designed during the earthquake and Tsunami of 2009. With the exception of some moderate corrosion on the gooseneck risers all the repairs made in 2007 are in place and functioning. The anodes on flange #2 and the adjacent blind flange were 100% consumed, possibly due to wind wave, swell and tidal action, and were replaced with new anodes. There is evidence that solid debris is inside the outfall and the diffuser section. Some of the solid debris has clogged the diffuser risers.

Recommendations:

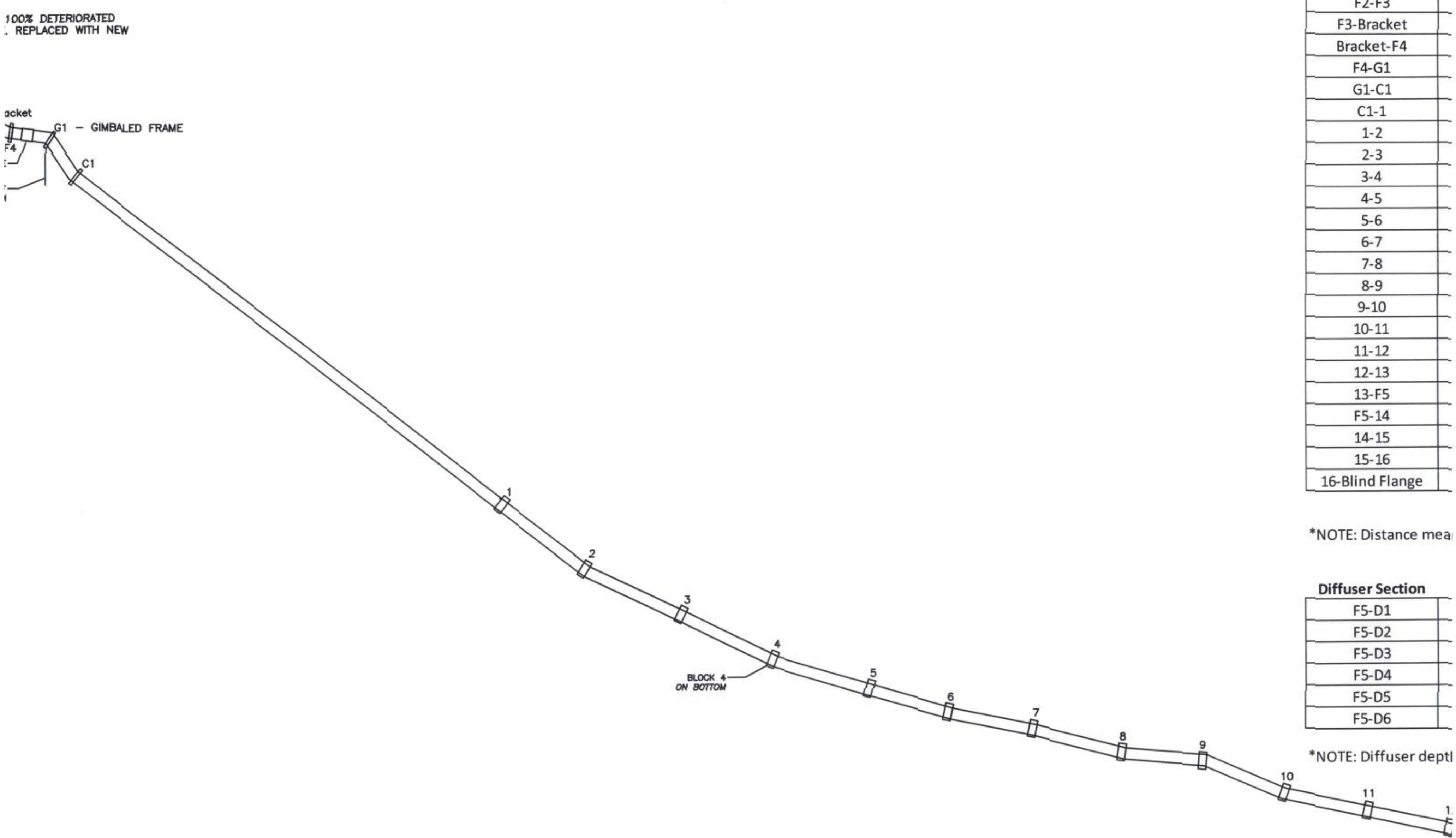
1. Disconnect the outfall blind flange and remove the solid waste debris that has collected in the diffuser section.
2. Consider running a pipeline cleaning device (pipe pig) though the outfall to force sold debris and encrusted effluent out the pipe. This will increase the flow and operational effectiveness of the outfall.
3. Replace the 4 bolts on the east anchor plate of the pipe retraining ring near flange #2.
4. Replace the ROMAC repair gooseneck risers within the next 4 years with something more heavy duty.
5. Drill additional anchor bolts into the reef to prevent box beams from rolling over the edge of the reef.

Respectfully Submitted,

David E. Cleary

David E. Cleary
Project Manager

PAGO PAGO, AMERICAN SAMOA
DECEMBER 2011



Pipe Section
0-Group 1
Group 1-Group2
Group 2-F1
F1-F2
F2-F3
F3-Bracket
Bracket-F4
F4-G1
G1-C1
C1-1
1-2
2-3
3-4
4-5
5-6
6-7
7-8
8-9
9-10
10-11
11-12
12-13
13-F5
F5-14
14-15
15-16
16-Blind Flange

*NOTE: Distance mea

Diffuser Section
F5-D1
F5-D2
F5-D3
F5-D4
F5-D5
F5-D6

*NOTE: Diffuser dept

UTULEI WWTP OUTFALL

Appendix M Unit Costs and Detailed Costs for Improvement Projects

Description	Units	Unit Price
Sewers		
Video Inspection	LF	$5.00
Flushing	LF	$2.50
Manhole Grouting	EA	$1,000.00
Manhole Replacement	EA	$5,000.00
Sewer Grouting	LF	$25.00
Sewer Pipe Slip Lining	LF	$100.00
Sewer Pipe Replacement	LF	$100.00
Lift Stations		
Vaitele Lift Station		
Land Acquisition	AC	$300,000.00
EQ Storage Tank	Gal	$1.00
EQ Return Pumps	EA	$25,000.00
EQ Diversion Structure	EA	$50,000.00
Aeration/Blowers/Mixing	LS	$30,000.00
Wet Well	EA	$30,000.00
Pumps	EA	$30,000.00
Piping and Valves	LS	$30,000.00
Odor Control	EA	$30,000.00
Flow Meter	EA	$15,000.00
Data Logger	EA	$15,000.00
VFD	EA	$30,000.00
Emergency Generator	EA	$50,000.00
Electrical and Controls	LS	$100,000.00
Site Improvements	LS	$30,000.00
Airport Lift Station		
Land Acquisition	AC	$300,000.00
EQ Storage Tank	Gal	$1.00
EQ Return Pumps	EA	$25,000.00
EQ Diversion Structure	EA	$50,000.00
Aeration/Blowers/Mixing	LS	$75,000.00
Wet Well	EA	$30,000.00
Pumps	EA	$50,000.00
Piping and Valves	LS	$40,000.00
Odor Control	EA	$40,000.00
Flow Meter	EA	$20,000.00
Data Logger	EA	$15,000.00
VFD	EA	$50,000.00
Emergency Generator	EA	$75,000.00
Electrical and Controls	LS	$150,000.00
Site Improvements	LS	$30,000.00
Malaloa Lift Station		
Land Acquisition	AC	$300,000.00
EQ Storage Tank	Gal	$1.00
EQ Return Pumps	EA	$25,000.00
EQ Diversion Structure	EA	$75,000.00
Aeration/Blowers/Mixing	LS	$75,000.00
Wet Well	EA	$50,000.00
Pumps	EA	$75,000.00
Discharge Piping	LS	$50,000.00
Odor Control	EA	$50,000.00

Flow Meter	EA	$25,000.00
Data Logger	EA	$15,000.00
VFD	EA	$60,000.00
Emergency Generator	EA	$100,000.00
Electrical and Controls	LS	$200,000.00
Site Improvements	LS	$30,000.00
Treatment Plants		
Land Acquisition	AC	$300,000.00
EQ Storage Tank	Gal	$1.00
EQ Return Pumps	EA	$35,000.00
EQ Diversion Structure	EA	$100,000.00
Aeration/Blowers/Mixing	LS	$75,000.00
Pumps	EA	$75,000.00
Piping and Valves	LS	$75,000.00
Odor Control	EA	$75,000.00
Flow Meter	EA	$40,000.00
Data Logger	EA	$15,000.00
VFD (Tafuna only)	EA	*$75,000.00*
Emergency Generator	EA	$150,000.00
Electrical and Controls	LS	$250,000.00
Site Improvements	LS	$50,000.00
Other Improvements		
Drainage Improvements	LF	$10.00

Tafuna STP Service Area					
Description	Units	Unit Price	Quantity	Total	
Sewers Below Sea Level					
Video Inspection	LF	$5.00	5,440	$	27,200.00
Flushing	LF	$2.50	5,440	$	13,600.00
Manhole Grouting	EA	$1,000.00	75	$	75,000.00
Sewer Grouting	LF	$25.00	5,440	$	136,000.00
Subtotal				$	251,800.00
Egineering	8%			$	20,144.00
Overhead and Profit	10%			$	25,180.00
Bonding & Insurance	8%			$	20,144.00
				$	-
Subtotal				$	65,468.00
Subtotal				$	317,268.00
Contingencies	20%			$	63,453.60
Subtotal (Sub-Sea-level)				**$**	**380,721.60**
Reduction in Flows	gal	75,000			
Dollar per Gallon Reduction in Flow (Peak)		$5.08			
Areas with Suspected I&I					
Video Inspection	LF	$5.00	9,865	$	49,325.00
Flushing	LF	$2.50	9,865	$	24,662.50
Manhole Grouting	EA	$1,000.00	47	$	47,000.00
Sewer Grouting	LF	$25.00	9,865	$	246,625.00
Subtotal				$	367,612.50
Egineering	8%			$	29,409.00
Overhead and Profit	10%			$	36,761.25
Bonding & Insurance	8%			$	29,409.00
				$	-
Subtotal				$	95,579.25
Subtotal				$	463,191.75
Contingencies	20%			$	92,638.35
Subtotal (I&I)				**$**	**555,830.10**
Reduction in Flows	gal	61,000			
Dollar per Gallon Reduction in Flow (Peak)		$9.11			

Estimate of Probable Construction Costs
Scoping Study
American Samoa Power Authority

Tafuna STP Service Area					
Description	**Units**	**Unit Price**	**Quantity**	**Total**	
Tafuna STP Service Area					
Description	**Units**	**Unit Price**	**Quantity**	**Total**	
Lift Stations					
Vaitele Lift Station					
EQ Storage Tank	Gal	$1.00	250,000	$	250,000.00
EQ Return Pumps	EA	$25,000.00	2	$	50,000.00
EQ Diversion Structure	EA	$50,000.00	1	$	50,000.00
Aeration/Blowers/Mixing	LS	$30,000.00	1	$	30,000.00
Pumps	EA	$30,000.00	2	$	60,000.00
Piping and Valves	LS	$30,000.00	1	$	30,000.00
Odor Control	EA	$30,000.00	1	$	30,000.00
Emergency Generator	EA	$50,000.00	1	$	50,000.00
Electrical and Controls	LS	$100,000.00	1	$	100,000.00
Site Improvements	LS	$30,000.00	1	$	30,000.00
Subtotal				$	680,000.00
Video Inspection	LF	$5.00	4,208	$	21,040.00
Flushing	LF	$2.50	4,208	$	10,520.00
Manhole Grouting	EA	$1,000.00	72	$	72,000.00
Sewer Grouting	LF	$25.00	4,208	$	105,200.00
				$	208,760.00
Subtotal				**$**	**888,760.00**
Egineering	8%			$	71,100.80
Overhead and Profit	10%			$	88,876.00
Bonding & Insurance	8%			$	71,100.80
				$	-
Subtotal				$	231,077.60
Subtotal				$	1,119,837.60
Contingencies	20%			$	223,967.52
Subtotal (Vaitele LS)				**$**	**1,343,805.12**
Reduction in Flows	gal	657,000			
Dollar per Gallon Reduction in Flow (Peak)		$2.05			

Tafuna STP Service Area				
Description	**Units**	**Unit Price**	**Quantity**	**Total**
Tafuna STP Service Area				
Description	**Units**	**Unit Price**	**Quantity**	**Total**
Airport Lift Station				
EQ Storage Tank	Gal	$1.00	500,000	$ 500,000.00
EQ Return Pumps	EA	$25,000.00	2	$ 50,000.00
EQ Diversion Structure	EA	$50,000.00	1	$ 50,000.00
Aeration/Blowers/Mixing	LS	$30,000.00	1	$ 30,000.00
Pumps	EA	$30,000.00	2	$ 60,000.00
Piping and Valves	LS	$30,000.00	1	$ 30,000.00
Odor Control	EA	$30,000.00	1	$ 30,000.00
Emergency Generator	EA	$50,000.00	1	$ 50,000.00
Electrical and Controls	LS	$100,000.00	1	$ 100,000.00
Site Improvements	LS	$30,000.00	1	$ 30,000.00
Subtotal				$ 930,000.00
Video Inspection	LF	$5.00	560	$ 2,800.00
Flushing	LF	$2.50	560	$ 1,400.00
Manhole Grouting	EA	$1,000.00	9	$ 9,000.00
Sewer Grouting	LF	$25.00	560	$ 14,000.00
				$ 27,200.00
Subtotal				**$ 957,200.00**
Egineering	8%			$ 76,576.00
Overhead and Profit	10%			$ 95,720.00
Bonding & Insurance	8%			$ 76,576.00
				$ -
Subtotal				$ 248,872.00
Subtotal				$ 1,206,072.00
Contingencies	20%			$ 241,214.40
Subtotal (Airport LS)				**$ 1,447,286.40**
Reduction in Flows	gal	1,451,000		
Dollar per Gallon Reduction in Flow (Peak)		$1.00		

Tafuna STP Service Area				
Description	**Units**	**Unit Price**	**Quantity**	**Total**
Tafuna STP Service Area				
Description	**Units**	**Unit Price**	**Quantity**	**Total**
Treatment Plant				
EQ Storage Tank	Gal	$1.00	1,100,000	$ 1,100,000.00
EQ Return Pumps	EA	$35,000.00	2	$ 70,000.00
EQ Diversion Structure	EA	$100,000.00	1	$ 100,000.00
Aeration/Blowers/Mixing	LS	$75,000.00	1	$ 75,000.00
Pumps	EA	$75,000.00	2	$ 150,000.00
Piping and Valves	LS	$75,000.00	1	$ 75,000.00
Odor Control	EA	$75,000.00	1	$ 75,000.00
VFD	EA	$75,000.00	2	$ 150,000.00
Emergency Generator	EA	$150,000.00	1	$ 150,000.00
Electrical and Controls	LS	$250,000.00	1	$ 250,000.00
Site Improvements	LS	$50,000.00	1	$ 50,000.00
Subtotal				$ 2,245,000.00
Egineering	8%			$ 179,600.00
Overhead and Profit	10%			$ 224,500.00
Bonding & Insurance	8%			$ 179,600.00
				$ -
Subtotal				$ 583,700.00
Subtotal				$ 2,828,700.00
Contingencies	20%			$ 565,740.00
Subtotal (Tafuna STP)				**$ 3,394,440.00**
Reduction in Flows	gal	3,500,000		
Dollar per Gallon Reduction in Flow (Peak)		$0.97		

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Utulei STP Service Area				
Description	**Units**	**Unit Price**	**Quantity**	**Total**
Sewers Below Sea-level				
Video Inspection	LF	$5.00	23,360	$ 116,800.00
Flushing	*LF*	*$2.50*	*23,360*	*$ 58,400.00*
Manhole Grouting	EA	$1,000.00	330	$ 330,000.00
Sewer Grouting	LF	$25.00	23,360	$ 584,000.00
Subtotal				$ 1,089,200.00
Egineering	8%			$ 87,136.00
Overhead and Profit	10%			$ 108,920.00
Bonding & Insurance	8%			$ 87,136.00
				$ -
Subtotal				$ 283,192.00
Subtotal				$ 1,372,392.00
Contingencies	20%			$ 274,478.40
Subtotal (Sub-Sea-level)				$ 1,646,870.40
Reduction in Flows	gal	998,000		
Dollar per Gallon Reduction in Flow (Peak)		$1.65		
Areas with known I&I				
Video Inspection	LF	$5.00	5,964	$ 29,820.00
Flushing	LF	$2.50	5,964	$ 14,910.00
Manhole Grouting	EA	$1,000.00	33	$ 33,000.00
Sewer Grouting	LF	$25.00	5,964	$ 149,100.00
Subtotal				$ 226,830.00
Egineering	8%			$ 18,146.40
Overhead and Profit	10%			$ 22,683.00
Bonding & Insurance	8%			$ 18,146.40
				$ -
Subtotal				$ 58,975.80
Subtotal				$ 285,805.80
Contingencies	20%			$ 57,161.16
Subtotal (I&I)				$ 342,966.96
Reduction in Flows	gal	74,000		
Dollar per Gallon Reduction in Flow (Peak)		$4.63		

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Utulei STP Service Area				
Description	**Units**	**Unit Price**	**Quantity**	**Total**
Utulei STP Service Area				
Description	**Units**	**Unit Price**	**Quantity**	**Total**
Lift Stations				
Malaloa Lift Station				
EQ Storage Tank	Gal	$1.00	800,000	$ 800,000.00
EQ Return Pumps	EA	$25,000.00	2	$ 50,000.00
EQ Diversion Structure	EA	$50,000.00	1	$ 50,000.00
Aeration/Blowers/Mixing	LS	$30,000.00	1	$ 30,000.00
Pumps	EA	$30,000.00	2	$ 60,000.00
Piping and Valves	LS	$30,000.00	1	$ 30,000.00
Odor Control	EA	$30,000.00	1	$ 30,000.00
Emergency Generator	EA	$50,000.00	1	$ 50,000.00
Electrical and Controls	LS	$100,000.00	1	$ 100,000.00
Site Improvements	LS	$30,000.00	1	$ 30,000.00
Subtotal				$ 1,230,000.00
Subtotal				$ 1,230,000.00
Egineering	8%			$ 98,400.00
Overhead and Profit	10%			$ 123,000.00
Bonding & Insurance	8%			$ 98,400.00
				$ -
Subtotal				$ 319,800.00
Subtotal				$ 1,549,800.00
Contingencies	20%			$ 309,960.00
Subtotal (Malaloa LS)				$ 1,859,760.00
Reduction in Flows	gal	2,888,000		
Dollar per Gallon Reduction in Flow (Peak)		$0.64		

Utulei STP Service Area					
Description	Units	Unit Price		Quantity	Total
Utulei STP Service Area					
Description	Units	Unit Price		Quantity	Total
Fagaalu Lift Station					
EQ Storage Tank	Gal	$1.00		150,000	$ 150,000.00
EQ Return Pumps	EA	$25,000.00		2	$ 50,000.00
EQ Diversion Structure	EA	$50,000.00		1	$ 50,000.00
Aeration/Blowers/Mixing	LS	$30,000.00		1	$ 30,000.00
Pumps	EA	$30,000.00		2	$ 60,000.00
Piping and Valves	LS	$30,000.00		1	$ 30,000.00
Odor Control	EA	$30,000.00		1	$ 30,000.00
Emergency Generator	EA	$50,000.00		1	$ 50,000.00
Electrical and Controls	LS	$100,000.00		1	$ 100,000.00
Site Improvements	LS	$30,000.00		1	$ 30,000.00
Subtotal					$ 580,000.00
Subtotal					**$ 580,000.00**
Egineering	8%				$ 46,400.00
Overhead and Profit	10%				$ 58,000.00
Bonding & Insurance	8%				$ 46,400.00
					$ -
Subtotal					$ 150,800.00
Subtotal					$ 730,800.00
Contingencies	20%				$ 146,160.00
Subtotal (Fagaalu LS)					$ 876,960.00
Reduction in Flows	gal	487,000			
Dollar per Gallon Reduction in Flow (Peak)		$1.80			

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Utulei STP Service Area				
Description	**Units**	**Unit Price**	**Quantity**	**Total**
Utulei STP Service Area				
Description	**Units**	**Unit Price**	**Quantity**	**Total**
Treatment Plant				
EQ Storage Tank	Gal	$1.00	1,100,000	$ 1,100,000.00
EQ Return Pumps	EA	$35,000.00	2	$ 70,000.00
EQ Diversion Structure	EA	$100,000.00	1	$ 100,000.00
Aeration/Blowers/Mixing	LS	$75,000.00	1	$ 75,000.00
Pumps	EA	$75,000.00	2	$ 150,000.00
Piping and Valves	LS	$150,000.00	1	$ 150,000.00
Odor Control	EA	$75,000.00	1	$ 75,000.00
VFD	EA	$75,000.00	2	$ 150,000.00
Emergency Generator	EA	$150,000.00	1	$ 150,000.00
Electrical and Controls	LS	$250,000.00	1	$ 250,000.00
Site Improvements	LS	$50,000.00	1	$ 50,000.00
Subtotal				$ 2,320,000.00
Egineering	8%			$ 185,600.00
Overhead and Profit	10%			$ 232,000.00
Bonding & Insurance	8%			$ 185,600.00
				$ -
Subtotal				$ 603,200.00
Subtotal				*$ 2,923,200.00*
Contingencies	20%			$ 584,640.00
Subtotal (Utulei STP)				**$ 3,507,840.00**
Reduction in Flows	gal	3,200,000		
Dollar per Gallon Reduction in Flow (Peak)		$1.10		