ATTACHMENT 1.
Maps and Drawings

to

Application for
301(h)-Modified NPDES Permit Reissuance

for

TAFUNA WASTEWATER TREATMENT PLANT

NPDES Permit No. AS0020010

Submitted By

AMERICAN SAMOA POWER AUTHORITY

May 4, 2004

ATTACHMENT 2.

Supporting Technical Analysis for 301(h) Waiver Renewal for the Tafuna Wastewater Treatment Plant

to

Application for 301-Modified NPDES Permit Reissuance for Small Dischargers

for

TAFUNA
WASTEWATER TREATMENT PLANT

NPDES Permit No. AS0020010

Submitted By

AMERICAN SAMOA POWER AUTHORITY

May 4, 2004

SUPPORTING TECHNICAL ANALYSIS FOR 301(H) WAIVER RENEWAL FOR THE TAFUNA WASTEWATER TREATMENT PLANT

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April 2004

1. Introduction

The current NPDES permit (AS 0020010) and associated 301(h) Waiver for the American Samoa Power Authority's (ASPA) Tafuna wastewater treatment plant (WWTP) became effective on 2 November 1999 and will expire on 1 November 2004. The permit and waiver renewal application is being submitted by ASPA. This Supporting Technical Analysis provides information required to complete the application for the 301(h) Waiver renewal.

The material presented in this document provides detailed data compilations, calculations, and descriptions of methods that were used to develop responses to various technical questions in the 301(h) questionnaire required as a part of the permit/waiver renewal process. The information presented is not intended to be a comprehensive data review, serve as the basis for engineering design, or provide complete guidance for implementation of permit requirements. The material presented is specifically aimed at addressing the following areas:

- Summarize effluent monitoring results over the current permit period to characterize the effluent properties required to address specific technical questions.

- Summarize receiving water oceanographic characteristics, using available data, to characterize the ambient environmental properties required to address specific technical questions. In addition, summarize receiving water monitoring results over the current permit period to assess effects of the discharge and characterize the receiving water chemical characteristics required to address specific technical questions.

- Determine the hydraulic characteristics of the diffuser to the extent required to assess dilution performance of the existing diffuser configuration and potential modifications to the diffuser configuration.

- Using EPA approved methods, predict the initial dilution and trapping level of the effluent plume. This is done for the existing diffuser configuration as well as potential modifications to the diffuser configuration. The evaluation of dilution performance is based on the application of critical conditions as generally defined and accepted by EPA. Therefore, the dilution predictions provide results that will generally be considerably lower than expected.

- Based on data and evaluations described in the items above, and using EPA approved methods, an evaluation of sedimentation caused by the discharge is developed.

- Based on data and evaluations described in the items above, and using EPA approved methods, an evaluation of effects on ambient dissolved oxygen caused by the discharge is developed.

A set of conclusions and recommendations is also presented. The areas addressed by the conclusions and recommendations include the possible modifications to the diffuser configuration, the need and design of a continuing receiving water quality monitoring program, and the requirements for effluent monitoring.

2. Effluent Characteristics

Effluent characteristics required to support the various analyses in this report and in the responses to the 301(h) Waiver questionnaire include the following:

- Flow is required to determine the critical conditions for assessing dilution performance. The most critical condition is the maximum flow expected, which is essentially the hydraulic capacity of the WWTP of 6 million gallons per day (mgd). Average or typical flows are also of interest, and the currently permitted (2 mgd annual average) and proposed permit limit increase (3 mgd annual average) are also considered in the assessment of dilution performance below (Section 5).

- Total suspended solids (TSS) are required to provide an assessment of sedimentation effects of the discharge. The average, maximum, and permitted values are all of interest. Maximum daily permitted TSS is 150 mg/l.

- Biochemical oxygen demand (BOD) is needed to assess the effects of the discharge on ambient dissolve oxygen (DO) levels. Maximum daily permitted BOD_5 is 200 mg/l.

- Other parameters that are monitored are useful in assessing the impact of the effluent on the receiving water. Using the dilution performance and known or estimated receiving water concentrations, the effluent data can be used to predict the expected changes in the receiving water concentrations of the parameters of concern. In addition to flow, TSS, and BOD, the current NPDES permit requires periodic monitoring for pH, settleable solids, oil and grease, and whole effluent toxicity (WET).

The complete data set for the effluent constituents in terms of average, maximum, and minimum is provided in Appendix 1.

3. Receiving Water Conditions

The receiving water characteristics needed to support the required responses and analyses in the 301(h) waiver application questionnaire are presented in this section of the supporting technical analysis document. There are three categories of data required:

- Current speed and direction is important for predicting initial dilution and the fate and transport of the effluent plume.

- Ambient density and vertical density profiles are important in prediction the initial dilution and trapping level of the effluent plume.

- Receiving water chemical and physical data in the vicinity of the discharge is used to assess any observed impacts of the discharge on the receiving water.

Available data for each of the three items listed above is presented. The data provided are those available and considered most suitable to do the required analyses. A comprehensive data review is not presented. However, it is noted that the data provided likely represent most of the data available and useable for the required analyses.

Currents

Current speed is an important variable in the effectiveness of the initial mixing from the diffuser and is one of the primary parameters that controls the initial dilution achieved. Current direction relative to the discharge direction is a variable of secondary importance and has a much smaller effect on initial dilution than current speed. However, the current direction as related to the overall circulation patterns is important in assessing the ultimate fate of the effluent plume in the farfield, after initial dilution.

No recent current data has been measured at the discharge site. The previous NPDES permit and 301(h) waiver application used the data from the 1979 Baseline Water Quality Report[1]. These data were used by EPA[2] in 1994 to determine the initial dilution and mixing zone dimensions. The same data are used for the analyses in this report, and are considered to accurately reflect the expected conditions on the open coastline setting involved. The frequency distribution and current rose from the 1979 study are reproduced in Figures 1 and 2.

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[1] M&E Pacific, Inc. 1979. Baseline Water Quality Survey in American Samoa. prepared for the U.S. Army Engineer Division, Pacific Ocean. October 1979.

[2] USEPA, 1994. Tafuna Mixing Zone Calculation. Memorandum prepared by Walter Frick, Coastal Ecosystems Team, for David Stuart, Region 9 U.S. Environmental Protection Agency. 26 May 1994

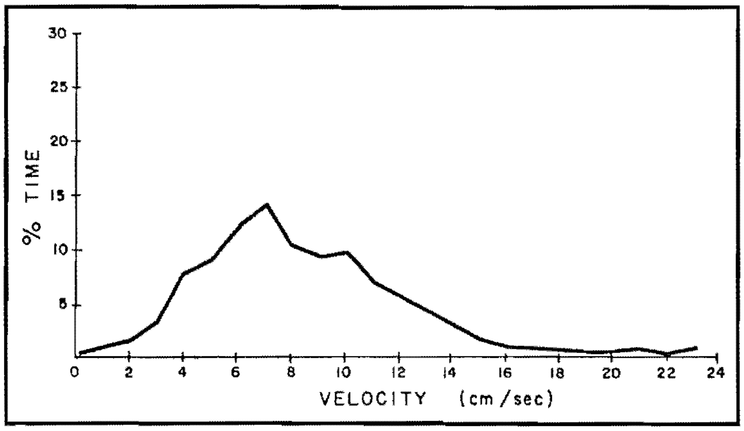

Figure 1.
Current Speed Frequency Diagram

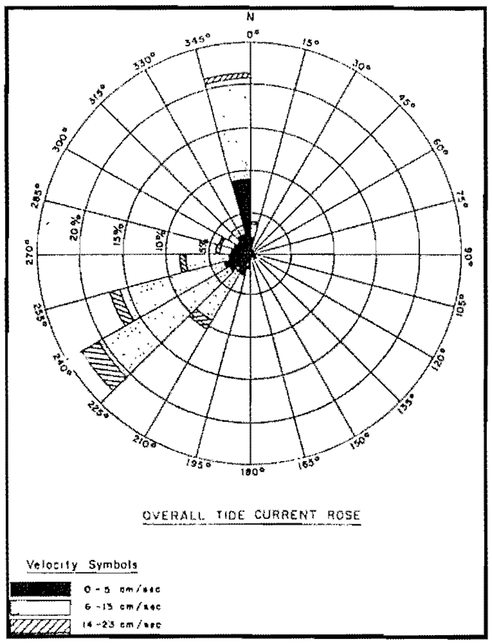

Figure 2
Current Rose

The frequency distribution in Figure 1 indicates a 10-percentile current speed of about 5 cm/sec. For open coastal systems the 10-percentile current is typically taken as the critical condition for calculation of minimum dilution for a diffuser. The current data in Figures 1 and 2 is from a meter located close to the existing outfall as shown in Figure 3. The current rose indicates two primary directions of flow: SW to SSW and NNW. These direction indicate curvilinear flow path flow parallel to the shoreline as illustrated in Figure 3. The same study from which the frequency distribution an current rose were developed presents drawings showing the overall circulation pattern in the vicinity of the outfall. This qualitative evaluation of circulation patterns was based largely on drogue studies and is reproduced in Figure 3. The currents tend to be parallel to the shoreline, as would be expected, and would transport and disperse the diluted plume away from the discharge area.

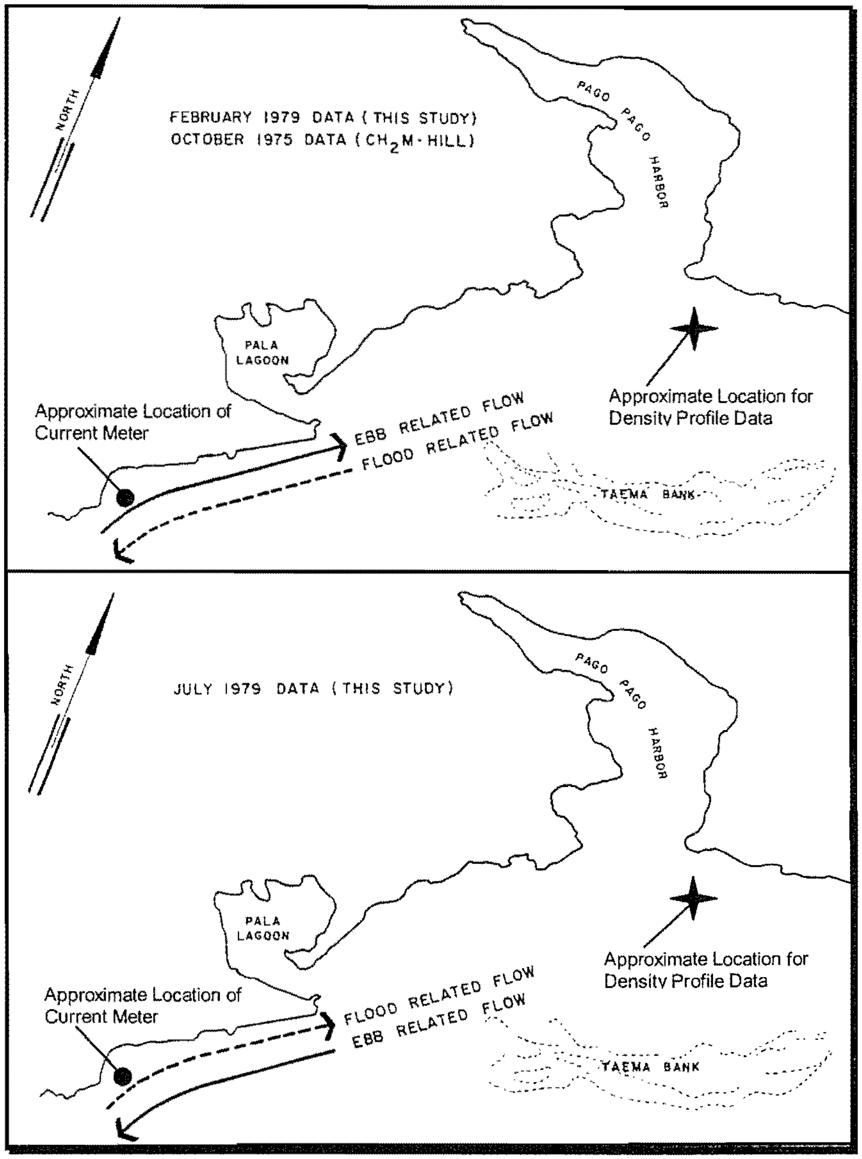

Figure 3.
General Current Patterns in the Vicinity of the Outfall

Density Profiles

Vertical density profiles as well as the overall density differences between the effluent and receiving water are a primary factor in the dilution and plume trapping depth achieved by a particular diffuser configuration. Density is calculated from salinity and temperature, and pressure effects are unimportant at the depths being considered. The dilution is quite sensitive to small changes in density and therefore precise and accurate information is required for application of the initial dilution model.

The receiving water monitoring data provides some measurements of temperature and salinity but only at three depths, and using an instrument that may not have the necessary resolution to accurately define the small changes in vertical density structure at the location of the outfall. Some previous studies provide a few density profiles, but the data have the same constraints as those from the current monitoring program. The best data is that from the monitoring done in the ocean offshore of the mouth of Pago Pago Harbor (see location shown in Figure 3). These data were collected with a very accurate and precise instrument (SeaBird) designed for the necessary resolution for oceanographic calculations, at the reference site for the Harbor Water Quality Monitoring program for Pago Pago Harbor. Although the data were collected some distance to the east of the discharge site, they are considered representative of the southern shoreline of Tutuila Island. The density profiles derived from the Pago Pago Harbor monitoring data are shown in Table 1 and Figure 4.

Table 1. Station 5 (Open Coast) Density Profiles

Date	Mar-01	Oct-01	Mar-02	Aug-02	Mar-03	Aug-03
Depth (m)	Density as sigma-t					
0	21.80	22.73	21.93	22.72	21.80	22.24
5	21.91	22.92	22.01	22.72	21.89	22.33
10	21.96	22.94	22.11	22.74	21.89	22.34
15	21.99	22.95	22.14	22.78	21.91	22.34
20	22.02	22.97	22.17	22.80	21.96	22.34
25	22.03	22.97	22.23	22.84	22.02	22.35
30	22.05	22.97	22.35	22.85	22.02	22.40
Delta	0.25	0.24	0.42	0.13	0.22	0.16
Average	21.97	22.92	22.13	22.78	21.93	22.33

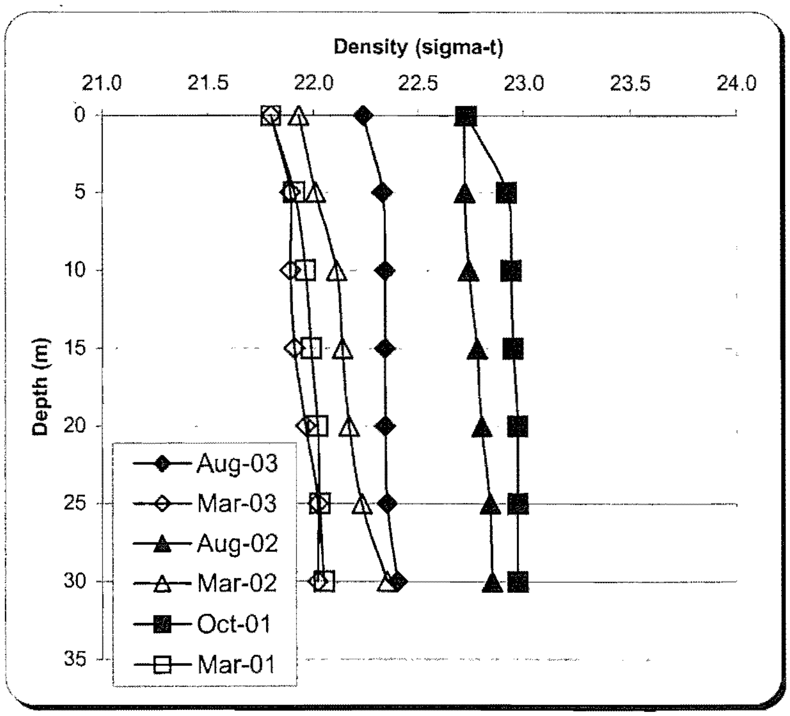

Figure 4.
Offshore Density Profiles

The most critical profile for dilution performance is the lowest density (smallest difference from the effluent density) and/or the largest vertical gradient in the area where the initial mixing will take place. The lowest density and the largest gradient may not be in the same profile, and it is not always obvious which will result in the lowest dilution. Therefore the initial dilution model is often tested with each profile to determine which s the most critical. This was done and the March 2002 profile was found to be the critical profile, and is used in the dilution modeling discussed below in Section 5.

Monitoring Data

The current NPDES permit requires periodic monitoring for a number of chemical and biological parameters. The available data over the period of the current permit (1999 to the present) include the following parameters:

- Hydrographic parameters including temperature, salinity, pH, and DO, and turbidity
- Nutrients and associated biological parameters including nitrogen, phosphorus, and chlorophyll-a
- Bacteria of potential anthropogenic origin (*Enterococci*)

The data were collected at two stations designated at the zone of initial dilution (ZID) of the effluent plume, two stations at the zone of mixing (ZOM) established for the discharge, and a farfield or reference station (REF). Sample were taken at three depths. The complete data set is provided in Appendix 2. An examination of the data indicates no significant discernable difference in the water quality at the ZID, ZOM, or REF locations. This is an indication of the high dilution and subsequent dispersion of the effluent.

4. Diffuser Hydraulics

The diffuser is a linear extension of the outfall pipeline with the diffuser barrel the same diameter as the outfall pipe (24-inch HDPE, 21-inch ID). The slope of the seabed along the diffuser is approximately 4.5°, deeper at the terminal end of the diffuser. Water depth is approximately 95 feet at the offshore end of the diffuser. There are six ports spaced 10 feet apart on 3-foot vertical risers. The risers are 8-inch HDPE (7.75-inch ID) and terminate in 90-degree elbows, which discharge effluent horizontally, parallel to the sea floor. The ports discharge in alternating directions, perpendicular to the diffuser barrel

The diffuser ports are sized with orifice plates on the end of each riser. The port diameters are 7.5, 6.5, 5.5, 5, 4.5, and 4 inches in diameter. The diffuser design intended that the 7.5-inch port be located at the offshore end of the diffuser, with sequentially smaller ports along the diffuser in the onshore direction. The as-built diffuser, however, was constructed with the 7.5-inch port on the inshore end of the diffuser with sequentially smaller ports along the barrel in the offshore direction. ASPA intends to correct this and modify the diffuser to match initial design conditions during the next inspection of the outfall. However, based on the hydraulic and diffuser performance analyses presented in this Technical Memorandum, it may be more reasonable to modify the diffuser by removing the orifice plates entirely.

The hydraulic characteristics of the diffuser were calculated to determine the flow distribution between the ports. The flow from each of the ports is required to accurately estimate the dilution performance of the diffuser discussed in Section 5 below. The diffuser port flows and headloss were calculated using HYDRO, a hydraulic software package developed by CH2M HILL. The diffuser hydraulics was modeled for three port configurations and three flow conditions. The diffuser configurations were, as discussed above:

- Design configuration: the largest port (7.5-inch) at the end of the diffuser (offshore) with progressively smaller ports moving upstream (towards the WWTP).
- As-built configuration: the smallest (4-inch) port at the end of the diffuser (offshore) with progressively larger ports moving upstream (towards the WWTP).
- Modified diffuser: all port orifice plates removed and all ports the same size (7.75-inch).

The flow conditions considered include the following:

- The currently permitted annual average flow of 2 mgd.
- A proposed increase in the annual average permitted flow of 3 mgd.
- The hydraulic capacity of the treatment system is 6 mgd, which is approached during high discharge flow conditions.

The results of the diffuser hydraulic analyses are presented in Table 2 where ports are, by convention numbered sequentially, with the downstream most port labeled as port No. 1. Detail results of the calculations are provided in Appendix 3. Table 2 shows the flow distribution between the ports and the exit velocity from each port. For the design and as-built cases the flow and speed are essentially the same for the same port sizes regardless s of the order of the ports along the diffuser. There is a fairly wide range of flows from individual ports for the design and as-built configurations, varying by a factor of about 2.6:1. The range of variation in current speeds is less, about 1.3:1. For the alternate configuration, with the orifice plates removed, the flow and exit speed distributions are nearly uniform between the ports. Exit speeds are less because the port size is larger than the ports for the other two cases.

Table 2 also lists the head loss for each configuration and discharge case considered. The design and as-built configurations are nearly the same. As expected, the head loss for the alternate case is substantially less (less than half) than for the other two configurations. The head loss calculations are only for the diffuser configuration, and do not include the outfall pipeline. The diffuser performance and head loss calculations were done based on typical friction and loss coefficients and are intended only for comparisons between cases. The analyses was not intended to provide hydraulic design information, but was intended for, and is considered sufficient for, the evaluation of diffuser dilution performance presented in Section 5.

Table 2. Diffuser Port Flows for Various Configurations

Port Number	Design Configuration			As-built Configuration			Alternate Configuration		
	Port Size (in)	Flow (mgd)	Exit Speed (ft/s)	Port Size (in)	Flow (mgd)	Exit Speed (ft/s)	Port Size (in)	Flow (mgd)	Exit Speed (ft/s)
Discharge = 2 mgd (Permitted Annual Average)									
1	7.5	0.52	2.60	4.0	19	3.40	7.7	0.34	1.64
2	6.5	0.43	2.90	4.5	24	3.34	7.7	0.34	1.64
3	5.5	0.33	3.12	5.0	29	3.25	7.7	0.34	1.61
4	5.0	0.28	3.23	5.5	34	3.15	7.7	0.33	1.57
5	4.5	0.24	3.35	6.5	43	2.90	7.7	0.32	1.53
6	4.0	0.20	3.49	7.5	52	2.60	7.7	0.33	1.57
Loss (ft)		0.23			0.21			0.10	
Discharge = 3 mgd (Proposed Increased Annual Average)									
1	7.5	0.77	3.91	4.0	0.29	5.10	7.7	0.52	2.47
2	6.5	0.65	4.35	4.5	0.36	5.01	7.7	0.52	2.47
3	5.5	0.50	4.68	5.0	0.43	4.88	7.7	0.51	2.42
4	5.0	0.43	4.84	5.5	0.50	4.72	7.7	0.49	2.36
5	4.5	0.36	5.02	6.5	0.65	4.35	7.7	0.48	2.28
6	4.0	0.29	5.22	7.5	0.77	3.90	7.7	0.49	2.35
Loss (ft)		0.52			0.48			0.23	
Discharge = 6 mgd (Hydraulic Capacity of WWTP)									
1	7.5	1.55	7.84	4.0	0.57	10.18	7.7	1.04	4.95
2	6.5	1.30	8.70	4.5	0.71	10.01	7.7	1.04	4.95
3	5.5	1.00	9.36	5.0	0.86	9,76	7.7	1.01	4.85
4	5.0	0.85	9.66	5.5	1.01	9.44	7.7	0.99	4.71
5	4.5	0.71	10.00	6.5	1.30	8.70	7.7	0.95	4.56
6	4.0	0.59	10.39	7.5	1.55	7.82	7.7	0.98	4.68
Loss (ft)		2.04			1.89			0.88	

5. Diffuser Performance

The dilution performance of the three diffuser configurations under the three flow conditions described in Section 4 above was predicted using the EPA initial dilution model UDKHDEN. This model is appropriate for application to the diffuser because it accounts for and simulates circular ports, discharging horizontally into a stratified water column. The model accounts for the effect of ambient current speed, and can include a vertical current profile (not available for this application). UDKHDEN also accounts for merging of the individual plumes from each port and terminates the simulation when the plume encounters the water surface or reaches maximum elevation in the water column prior to trapping below the surface.

The initial dilution model requires the specification of various parameters describing the diffuser configuration, effluent properties, and ambient conditions. The required diffuser configuration includes the following:

- Port diameter, port spacing, and port orientation (discharge angle relative to horizontal) are required. The port diameters for the various diffuser configurations being considered were described in Section 4. Ports are spaced 10 feet apart and discharge horizontally.

- The port depth below the surface is also required and based on available information was taken to be 91.5 feet for the seaward-most port (No. 1). This represents a low tide condition and accounts for the distance of the ports above the seabed. For other ports the depth was adjusted based on the slope of the seabed (4.5°) and the distance between the ports. It is noted that tidal ranges are small and water depths do not vary by more than 1 to 3 feet over tidal extremes. The port depths used in the model, with port No.1 being the furthest offshore, were:

Port No.	1	2	3	4	5	6
Depth (meters)	27.9	27.7	27.4	27.2	26.9	26.7

- The model requires that the ports are assumed to discharge in a direction perpendicular to the line of the diffuser barrel, which is actually the case for all three configurations considered in this analysis. The model also requires that the ports are all assumed to discharge in the same direction and in the same direction of the ambient current. In the case considered here, for the Tafuna WWTP outfall diffuser, the ports discharge in alternate directions along the diffuser, and thus every other port discharges into the ambient current rather than with the current direction. Experience indicates that if the ports are all assumed to discharge in the same direction the results are conservative (predicted dilution is lower than would be observed) for two reasons: mixing into opposing currents is more intense that for co-flowing currents, and plumes do not merge as quickly.

Required effluent characteristics include the flow rate and density (determined by temperature and salinity) of the effluent:

- Effluent discharge rates of 2mgd, 3 mgd, and 6 mgd were considered as described in Section 4 above.

- The effluent density was based on freshwater at a representative temperature of 30°C. Dilution is not sensitive to small changes in effluent density.

The initial dilution is quite sensitive to ambient current speed and density conditions. These conditions can vary significantly in time at the discharge location, and therefore initial dilution for a given diffuser configuration and effluent discharge depends on the particular time of interest. Typically, those critical conditions resulting in the lowest expected dilution of the most interest. Therefore, the model predictions presented below are for critical ambient conditions. The required ambient conditions to calculate initial dilution are as follows:

- Current direction, based on available data and examination of the orientation of the diffuser relative to the shoreline, is typically perpendicular to the diffuser. The effects of variations in current direction are small, so this is not a key input to the model.

- Ambient current speed was taken as the 10-percentile current based on available data. and was selected to be 5 cm/sec. This is an important variable that has a significant effect on dilution performance. The value is supported by available data, minimum current speeds suggested in the 301(h) TSD[3], and professional judgment and experience. It is unlikely that current speeds lower than this will be observed in the vicinity of the diffuser.

- The critical ambient density profile was determined by running the model for the six available density profiles and the case producing the lowest initial dilution was selected as the critical case. This was a profile taken in March 2002 with a density gradient of 0.42 sigma-t units between the surface and the 100 foot depth. The profiles used were based on data collected offshore of Pago Pago Harbor, and are considered to represent the general area along the south central coastal area of Tutuila Island. The limited salinity and temperature data (and thus the calculated density) collected in the immediate vicinity of the diffuser are not considered adequate in terms of spatial detail or (based on the type of instrument used) accuracy for use with the model. It is noted that the data from the monitoring program in the vicinity of the discharge were intended only to characterize the conditions near the discharge, for which they are adequate, and were not collected with the modeling application in mind.

The model predictions using UDKHDEN were done accounting for plume merging. This is a conservative assumption (predicts dilutions lower than expected) since the alternating direction of port discharge is not accounted for. The dilution performance model UDKHDEN, like all other available similar models, can not be run with varying port sizes, varying depths, or with varying flows through each port. The typical approach for a multi port diffuser with different size ports is to run each port size (with the appropriate flow and depth) separately, determine the dilution from each port, and use a calculated flux average dilution as the overall performance metric for the diffuser. However, the diffuser being considered here only has a single port of each size and modeling a single port will not account for merging of adjacent ports. To account for this affect, which reduces dilution, the typical approach is to run the model for two ports of the same size, with twice the flow that is discharged through the single port. The simulations were done following the above procedures. Results are summarized in Table 3. Detailed model results are provided in Appendix 4.

The results in Table 3 indicate that there is virtually no difference, within the normally accepted confidences levels for the dilution model, between the three diffuser configurations considered. For each configuration the dilution is lower with increased effluent discharge

[3] EPA (1994). Amended Section 301(h) Technical Support Document, U.S. Environmental Protection Agency (EPA 842-B-94-007), September 1994.

flow rate, as is generally expected (except at very high port exit velocities). The dilution is quite high for all cases and indicates that the effluent will be mixed with ambient sea water very quickly and will be immeasurable within a few meters and within a few seconds of the discharge point.

Table 3. Diffuser Dilution and Plume Trapping Level (Below Surface) for Various Configurations

Port Number	Design Configuration			As-built Configuration			Alternate Configuration		
	Port Size (in)	Dilution	Trapping Level (m)	Port Size (in)	Dilution	Trapping Level (m)	Port Size (in)	Dilution	Trapping Level (m)
Discharge = 2 mgd (Permitted Annual Average)									
1	7.5	283	8.3	4.0	486	14.7	7.7		
2	6.5	325	8.8	4.5	456	12.3	7.7		
3	5.5	394	9.6	5.0	429	10.2	7.7		
4	5.0	441	10.1	5.5	382	9.4	7.7		
5	4.5	472	11.1	6.5	316	8.5	7.7		
6	4.0	510	12.3	7.5	276	7.6	7.7		
	Flux Average	378	9.6	Flux Average	366	9.7	Total	≈ 380	≈ 9.5
Discharge = 3 mgd (Proposed Increased Annual Average)									
1	7.5	220	6.6	4.0	414	11.7	7.7		
2	6.5	247	7.3	4.5	375	9.8	7.7		
3	5.5	294	8.5	5.0	328	9.0	7.7		
4	5.0	325	8.9	5.5	293	8.3	7.7		
5	4.5	368	9.4	6.5	243	6.9	7.7		
6	4.0	429	10.0	7.5	215	5.9	7.7		
	Flux Average	291	8.1	Flux Average	289	8.0	Total	289	8.0
Discharge = 6 mgd (Hydraulic Capacity of WWTP)									
1	7.5	149	2.6	4.0	280	8.8	7.7		
2	6.5	165	4.0	4.5	240	7.8	7.7		
3	5.5	191	5.8	5.0	211	6.6	7.7		
4	5.0	211	6.6	5.5	190	5.6	7.7		
5	4.5	237	7.3	6.5	168	3.5	7.7		
6	4.0	268	8.0	7.5	148	1.6	7.7		
	Flux Average	190	5.1	Flux Average	192	4.8	Total	187	4.9

It is noted that for the 2 mgd case with orifice plates removed (all ports 7.7 inches in diameter, the model would not run since the densimetric Froude number was below the arbitrary cut-off value in the model code. A different model (e.g. UOUTPLM) could be used, but instead the results were simply extrapolated since UDKHDEN would run at just over 2 mgd. The limitations inherent with the use of alternate other models were considered more detrimental than the small extrapolation required. Figure 5 illustrates the dilution achieved for the alternate (constant port size) configuration as a function of effluent discharge rate.

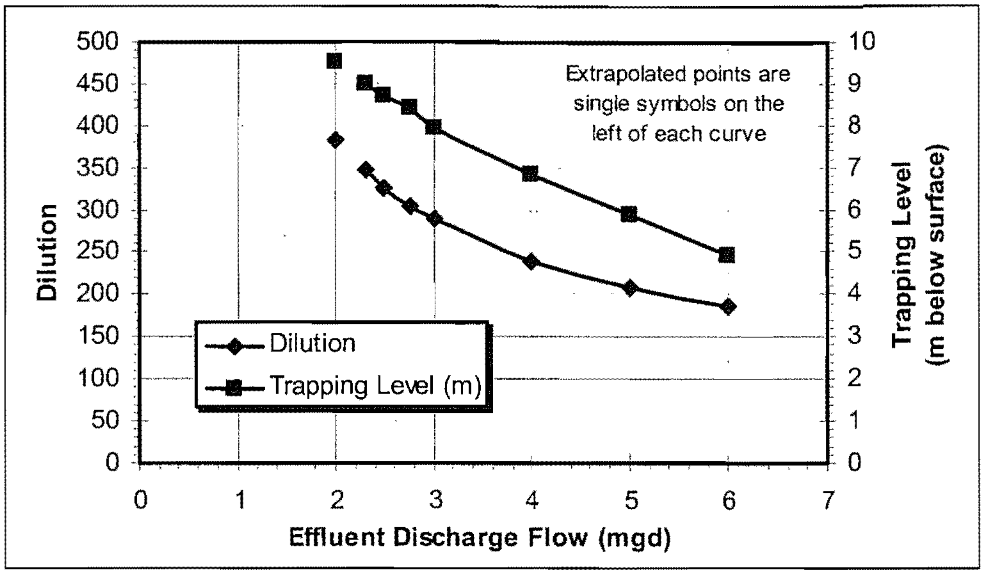

Figure 5.
Dilution and Trapping Level Variation with Discharge Flow Rate for Alternate Diffuser Configuration with Six 7.7-inch Ports

6. Suspended Sediment and Sedimentation

The total suspended sediment load in the effluent can affect the sediment concentration in the water column and can also result in the deposition and accumulation of sediment on the seabed in the vicinity of the discharge. Both of these potential impacts need to be addressed to respond to specific questions in the 301(h) waiver questionnaire. Each is addressed below.

Suspended Sediment in the Water Column

The effect of effluent TSS on receiving water quality depends on the ambient levels of receiving water TSS. The receiving water monitoring required under the current permit does not require TSS measurements. However, recent data from the ocean reference station offshore of Pago Pago Harbor indicate that TSS along the open ocean coastline can be less than 5 mg/l[4]. Since the largest relative effect will occur for the lowest receiving water value and the highest effluent concentration, these values are used in the calculations below. The effluent value is taken as the maximum permitted concentration of 150 mg/l. Following initial dilution, the suspended solids (TSS=SS) concentration is calculated from,

$$SS_f = SS_a + \frac{(SS_e - SS_a)}{S_a}$$

where the subscripts *f, a,* and *e* represent the final, ambient, and effluent levels of suspended solids, respectively, and S_a is the flux-averaged dilution.

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[4] Recent data collected for the Pago Pago Harbor Water Quality Monitoring requirements under the NPDES permits for StatKist Samoa, Samoa packing, and the ASPA Utulei WWTP. These data will be published in the 2004 non-tradewind season monitoring reports during 2004 and supplied to USEPA and ASEPA. Samples at three depths (3 feet, 60 feet, and 120 feet, at Station 5 (the reference station offshore of the Harbor mouth) were all below detection at 5 mg/l.

The concentration of TSS immediately following initial dilution was calculated for the three diffuser configurations described in Section 4 above and for the existing permitted flow (2 mgd annual average), the proposed flow (3 mgd annual average), and the maximum flow (6 mgd). All of the situations examined were based on the most critical period of water column stratification and the maximum permitted TSS limitation of 150 mg/l.

The calculation was made for a range of ambient values, although the lower end of the range of 5 mg/l represents a detection limit and actual values may be lower (therefore a value of 1 mg/l was also considered). The range of TSS increase for all cases considered was 0.26 to 0.80 mg/l. Larger increases are associated with the higher effluent flows and lower ambient concentrations. The results of this calculation are shown in Table 4. The changes in TSS caused by the effluent following initial dilution are indistinguishable from expected natural variability and below the typical resolution of analytical measurements.

Table 4. Suspended Solids Concentration (mg/l) Following Initial Dilution

Flow	2 mgd				3 mgd				6 mgd			
Design Diffuser Configuration												
SS_e	150				150				150			
Dilution (S_a)	378				291				190			
SS_a	1	5	10	50	1	5	10	50	1	5	10	50
SS_f	1.39	5.38	10.37	50.26	1.51	5.50	10.48	50.34	1.78	5.76	10.74	50.53
Δ TSS	0.39	0.38	0.37	0.26	0.51	0.50	0.48	0.34	0.78	0.76	0.74	0.53
As-built Diffuser Configuration												
SS_e	150				150				150			
Dilution (S_a)	366				289				192			
SS_a	1	5	10	50	1	5	10	50	1	5	10	50
SS_f	1.41	5.40	10.38	50.27	1.52	5.50	10.48	50.35	1.78	5.76	10.73	50.52
Δ TSS	0.41	0.40	0.38	0.27	0.52	0.50	0.48	0.35	0.78	0.76	0.73	0.52
Alternate Diffuser Configuration (Six 7.7-inch Ports)												
SS_e	150				150				150			
Dilution (S_a)	380				289				187			
SS_a	1	5	10	50	1	5	10	50	1	5	10	50
SS_f	1.39	5.38	10.37	50.26	1.52	5.50	10.48	50.35	1.80	5.78	10.75	50.53
Δ TSS	0.39	0.38	0.37	0.26	0.52	0.50	0.48	0.35	0.80	0.78	0.75	0.53

Seabed Sedimentation

The analysis and prediction of sediment deposition and steady state accumulation rates can be relatively complex and depends on the loading, plume dynamics, ambient circulation and current patterns, seabed slope, organic content of the TSS load, and size distribution of the discharged sediment. The concomitant effect on sediment oxygen demand can also relatively difficult to estimate. However, the 301(h) TSD provides a simplified method for small discharges that predicts deposing rates and DO demand graphically based on plume height of rise and TSS loading rates.

The simplified 301(h) TSD method is shown in Figure 6. To use this figure the plume height of rise and TSS load are entered as coordinates and if the point defined by these variables is below the dashed line, no additional analyses are required. The plume height of rise, under average conditions, is based on the trapping levels given in Table 3. The trapping level is 9.7 meters below the surface and the discharge is 27.3 meters below the surface and 1.1 meter above the bottom. Therefore, the plume height of rise is 18.7 meters above the seabed. The average permitted TSS loading (average monthly) is 1252 lbs/day (568 kg/day). Plotting these values on Figure 6, as shown, indicates that the steady state sediment accumulation rate is less than 50 g/m^2 and the steady state sediment oxygen demand is less than 0.2 mg/l. Therefore, based on the 301(h)TSD, no additional analyses are required for either parameter.

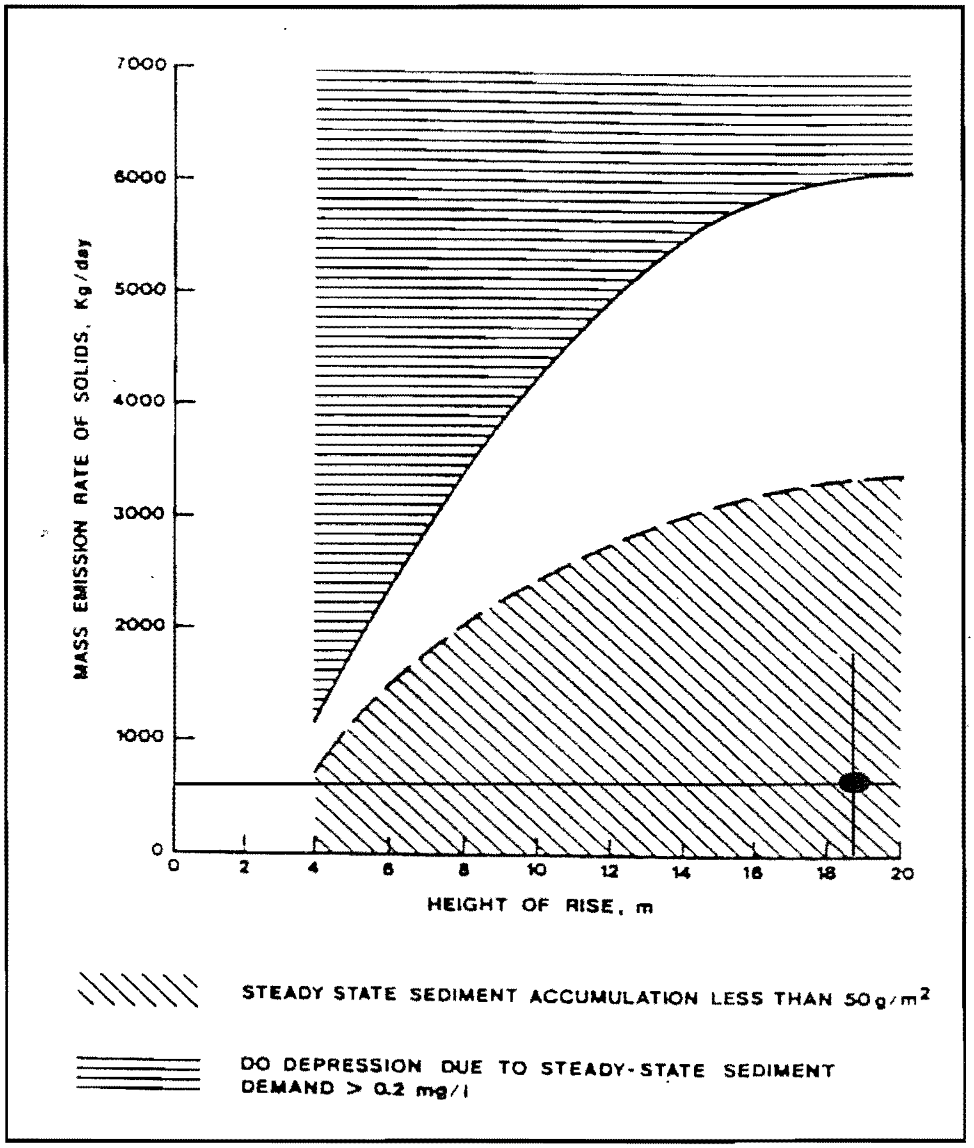

Figure 6.
Sedimentation and Sediment Oxygen Demand: 301(h) Simplified Method for Small Discharges

7. Dissolved Oxygen

The effects of the effluent discharge on dissolved oxygen can be in the nearfield, as the effluent plume mixes with the ambient water and in the farfield when the oxygen demand of the BOD effluent load is exerted. The nearfield DO demand is a mixing phenomenon with the addition of an immediate dissolved oxygen demand (IDOD) included. Predicting the potential oxygen sag in the farfield, as the plume moves away from the immediate discharge region, requires a farfield dispersion model. Both of these potential impacts are evaluated below.

Dissolved Oxygen after Initial Dilution

Following initial dilution, the DO is calculated from

$$DO_f = DO_a + \frac{(DO_e - IDOD - DO_a)}{S_a}$$

where the subscripts *f*, *a*, and *e* represent the final, ambient, and effluent levels of DO, respectively. S_a is the flux-averaged dilution and IDOD is the immediate dissolved oxygen demand. The parameters required to calculate the DO after initial dilution are determined as follows:

- The IDOD for the effluent was not measured, but the recommended value in the 301(h)TSD is used. The estimated IDOD is a function of effluent BOD load and travel time of the effluent through the outfall pipe. Longer travel times result in higher IDOD and lower (more critical) predicted values of DO_f. Therefore travel time based on the average flow (2 mgd) was assumed as a conservative estimate. At 2 mgd, for a 21-inch ID pipeline 1550 feet long the travel time is 33 minutes. The maximum permitted BOD_5 concentration is 200 mg/l. Using Table B-3 in the 301(h) TSD for primary treated wastewater, the suggested IDOD is 5 mg/l.

- The effluent DO is unknown, but can conservatively be assumed to be zero.

- Based on the 301(h) TSD, the ambient DO, should be taken immediately up current of the diffuser averaged over the tidal period (12.5 hours) and from the diffuser port depth to the trapping level. The monitoring data provides DO from singe point measurements at a specific point in time. To investigate the range of possible effects two values of ambient DO, from the Tafuna WWTP receiving water monitoring reference station (Station C), were selected for use in the calculation:

    - Based on the monitoring data (see Appendix 2) the minimum DO averaged over the lower half of the water column is 5.55 mg/l, observed in the first quarter of 2001. This is the <u>minimum</u> water column averaged value at the reference station used in the receiving water monitoring data. The value is consistent with other data from the open coastline off the mouth pf Pago Pago Harbor. This value represents a case that is more restrictive than suggested by the 301(h) TSD guidance, but represents a critical condition.

- o Using data from the same station, the average value over the bottom half of the water column for all monitoring events is 6.5 mg/l. This value appears more consistent with the guidance of the 301(h)TSD.

- The CID (from Section 5) for the existing and proposed average flows and the maximum flow and considering all three of the diffuser configurations described in Section 4 above.

Using the above values, the calculated DO_f after initial dilution is presented in Table 5. The affect of the discharge on DO following initial dilution is insignificant.

Table 5. DO Following Initial Dilution

Flow	2 mgd		3 mgd		6 mgd	
Design Diffuser Configuration						
DO_e (mg/l)	0	0	0	0	0	0
DO_a (mg/l)	5.55	6.5	5.55	6.5	5.55	6.5
IDOD (mg/l)	5	5	5	5	5	5
S_a	378	378	291	291	190	190
DO_f (mg/l)	5.522	6.470	5.514	6.460	5.494	6.439
ΔDO (mg/l)	-0.028	-0.030	-0.036	-0.040	-0.056	-0.061
As-built Diffuser Configuration						
DO_e (mg/l)	0	0	0	0	0	0
DO_a (mg/l)	5.55	6.5	5.55	6.5	5.55	6.5
IDOD (mg/l)	5	5	5	5	5	5
S_a	366	366	289	289	192	192
DO_f (mg/l)	5.521	6.469	5.513	6.460	5.495	6.440
ΔDO (mg/l)	-0.029	-0.031	-0.037	-0.040	-0.055	-0.060
Alternate Diffuser Configuration (Six 7.7-inch Ports)						
DO_e (mg/l)	0	0	0	0	0	0
DO_a (mg/l)	5.55	6.5	5.55	6.5	5.55	6.5
IDOD (mg/l)	5	5	5	5	5	5
S_a	380	380	289	289	187	187
DO_f (mg/l)	5.522	6.470	5.513	6.460	5.494	6.439
ΔDO (mg/l)	-0.028	-0.030	-0.037	-0.040	-0.056	-0.061

Farfield Dissolved Oxygen

The method presented in the 301(h) TSD, to estimate the impact of the discharge on DO concentrations in the farfield, is a stepwise procedure as described below. The first step is a test to determine whether farfield analysis is required. This test requires the determination of final 5-day biochemical oxygen demand (BOD_5) concentration following initial dilution as follows:

$$BOD_f = BOD_a + \frac{(BOD_e - BOD_a)}{S_a}$$

where,

BOD_f = final BOD_5 concentration of receiving water at plume trapping level in mg/l,

BOD_a = ambient BOD_5 concentration in mg/l,

BOD_e = effluent BOD_5 concentration in mg/l, and

S_a = initial dilution (flux-averaged).

The BOD_a value is not know and has not be measured in the receiving water. Typically there is very low BOD in coastal open ocean waters, and a value of 1 mg/l would be considered high. Using this value can be considered to yield a conservative analysis (i.e., these values will result in an overestimation of DO demand and an underestimation of DO concentration in the farfield).

The permit limit for BOD_5 is 200 mg/l (maximum daily limitation), which is used in the calculations below. The CID for the maximum flow and existing diffuser configuration is 190:1. For the average flow the CID is 378:1. The BOD_f concentration evaluated using these values is:

- For the CID for maximum discharge BOD_f = 2.04 mg/l
- For the average discharge under critical conditions, the BOD_f = 1.53 mg/l

The 301(h) TSD guidance requires the DO at the end of initial dilution estimated above to immediately support the full demand of the BOD load. If it does not, a more detailed analysis of farfield DO effects is required. The test is stated as follows:

$$DO_s < DO_f - BOD_{fu}$$

where,

DO_s = applicable water quality standard DO limitation

DO_f = dissolved oxygen concentration at the completion of initial dilution, mg/l, as calculated above, and

BOD_{fu} = ultimate BOD at the completion of initial dilution = (1.46 x BOD_f) mg/l, where BOD_f is calculated above.

The DO water quality standard is stated in terms of both saturation and concentration. For open coastal waters the DO shall not be less than 80% saturation, or less than 5.5 mg/l. Under conditions that provide for the lowest saturation values, the ambient DO saturation value would be approximately 6.7 mg/l (temperature of 30 °C and salinity of 36 ppt) and 80% of that value is 5.4 mg/l. Therefore, DO_s is taken to be 5.5 mg/l for the required calculation. Since water quality standard is stated in terms of average values for DO, and because 301(h) TSD suggests an average ambient DO, as described in the calculation for nearfield effects above, the average ambient do in the lower half of the water column (6.5 mg/l)is used.

If the above inequality is true, it can be assumed that the discharge cannot possibly violate the DO standard and no further analysis of farfield BOD exertion is required. If the inequality is not true, then the further analysis is required. In this case,

$$5.5 > [[6.5 - 2.04 \times 1.46] = 3.52]$$

and the test equality is not true. Therefore, the farfield analysis must be conducted and is described below.

The procedure specified in the 301(h) TSD was applied using a Microsoft® Excel spreadsheet application. This method typically gives conservative results in coastal waters (over-predicts DO depression) because the method used is very conservative and <u>does not</u> account for the replenishment of oxygen in the receiving water as a result of contact with the atmosphere (re-aeration) or photosynthesis. The method for farfield analysis is described in the following equation:

$$DO(t) = DO_a + \frac{DO_f - DO_a}{D_s(t)} - \left[\frac{L_{fc}}{D_s(t)}(1 - \exp(K_c t))\right] - \left[\frac{L_{fn}}{D_s(t)}(1 - \exp(-K_n t))\right]$$

where,

> $DO(t)$ =dissolved oxygen as a function of time, mg/l,
>
> DO_a = ambient dissolved oxygen, mg/l,
>
> DO_f = final dissolved oxygen at end of initial dilution, mg/l,
>
> D_s = subsequent (farfield) dilution (calculated using the Brooks method as described in the 301(h) TSD),
>
> L_{fc} = ultimate carbonaceous BOD (CBOD) above ambient after initial dilution, mg/l,
>
> L_{fn} = Nitrogenous BOD (NBOD) above ambient after initial dilution, mg/l,
>
> K_c = CBOD decay rate constant (day^{-1}),
>
> K_n = NBOD decay rate constant (day^{-1}), and
>
> t = travel time (days).

The calculation of DO(t) using the above relation has four main terms (on the right hand side of the equation):

1. The first term is simply the DO in the ambient receiving water.

2. The second term accounts for the mixing of the effluent and the receiving water over the farfield dilution phase. This term is numerically equal to the difference between the DO at the end of initial dilution, DO_f, and the DO of the receiving water, DO_a, divided by the farfield dilution, D_s. DO_f must include the IDOD effect as described in Equation 8-9.

3. The third term accounts for both the dilution and decay of the carbonaceous BOD as a function of time, based on measured BOD_5 in the effluent and receiving water.

4. The fourth term accounts for both the dilution and decay of the nitrogenous BOD as a function of time, based on the TKN levels measured in the effluent.

All of the terms and coefficients used above are identical to those in the 301(h) TSD. The calculations were done for 10 days, at 6-hr intervals to account for the majority of the oxygen demand load as it depletes ambient DO, using the average observed value of ambient DO levels at the plume trapping depth. To examine the DO impacts more closely, the calculation was also done in more detail for 1 day at 0.6-hour intervals. The results of the calculations are shown in Figure 7 and were based on the procedure and inputs described below.

Input values for the farfield calculations shown on the figure were as follows:

- The ambient average water column DO was taken as 6.5 mg/l.

- The final DO concentration to begin the farfield calculation (after CID) was 6.44 mg/l, as determined above.

- A CBOD decay rate of 0.347/day (base e) was used based on a value of 0.23/day, as specified in the 301(h) TSD, adjusted for the ambient water temperature. Ambient water temperature varies seasonally, and the maximum values are typically between 28°C and 30°C. A value of 29°C was used to calculate the CBOD decay rate using the expression:

$$K_c = 0.23x(1.047)^{(T-20)} = 0.23 \times 1.047^9 = 0.347$$

- Similarly, an NBOD decay rate of 0.151/day (base e) was used based on a value of 0.1/day adjusted for the ambient water temperature, based on a similar expression:

$$K_n = 0.10 \times (1.047)^{(T-20)} = 0.10 \times 1.047^9 = 0.151$$

- The ultimate CBOD concentration was based on an effluent BOD_5 of 200 mg/l, as described above, increased by a factor of 1.46 for conversion to ultimate BOD as recommended in the 301(h)TSD.

- An effluent TKN concentration has not been measured recently a typical extreme value for municipal WWTP can be assumed to below 25 mg/l.

- Ultimate NBOD was calculated by multiplying the effluent TKN concentration by 4.57, as recommended in the 301(h) TSD

- Following the 301(h) TSD guidelines, the ultimate NBOD and CBOD are to be decreased to the ambient ultimate NBOD and CBOD and divided by the CID of 190. The ambient BOD_5 and TKN available measured values not know but are very small. The ambient BOD_5 was assumed to be 1 mg/l and the ambient TKN was assumed to be less than 1 mg/l as well. The total nitrogen water quality standard is 0.13 mg/l and the avaible monitoring data indicates that the receiving water values are usually near the water quality standard. However, since smaller ambient values will result in lower predicted DO, based on the farfield equation above, a value of 0.0 was used for each of these parameters in order to remain conservative (predict the lowest ambient farfield DO).

The values of farfield dilution, D_S, as a function of travel time were calculated using methods given in the *301(h) TSD* for the initial condition of a maximum field width with a variable diffusion coefficient applicable to open ocean conditions. The relationship used to calculate the farfield dilution, D_S, was:

$$D_s = \left[erf \left[\frac{1.5}{\left(1 + \frac{8 \cdot e_0 \cdot t}{b^2}\right)^3 - 1} \right]^{1/2} \right]^{-1}$$

where,

erf = the error function,

e_0 = the initial diffusion coefficient (ft^2/sec) = $0.001 \times b^{4/3}$,

b = the effective diffuser length (ft), and

t = travel time (sec).

The effective diffuser length, b, is the initial width of the plume after initial dilution. Consideration of the equation for farfield or subsequent dilution, D_s, shows that the inverse of D_s is related to b by an error function. The larger the value of b, the smaller the value of D_s

and, therefore, the lower the value of DO(t). To be as conservative as possible, the calculations use the largest possible value of b. This occurs when the ambient current flows perpendicular to the axis of the diffuser. Therefore, b was set equal to the length of the diffuser plus the half widths of the plume on either end of the diffuser for the critical conditions and maximum effluent flow. This procedure results in an initial width of 38 meters.

The results of the farfield DO demand calculations, shown in Figure 7 indicate an insignificant DO demand in the farfield after initial dilution. The actual maximum DO sag can be calculated as 0.0016 mg/l occurring 6 minutes after the completion of initial dilution. This number is meaningless given the accuracy of the input data and the inability to measure DO at these levels. The DO water quality standard for American Samoa will be achieved.

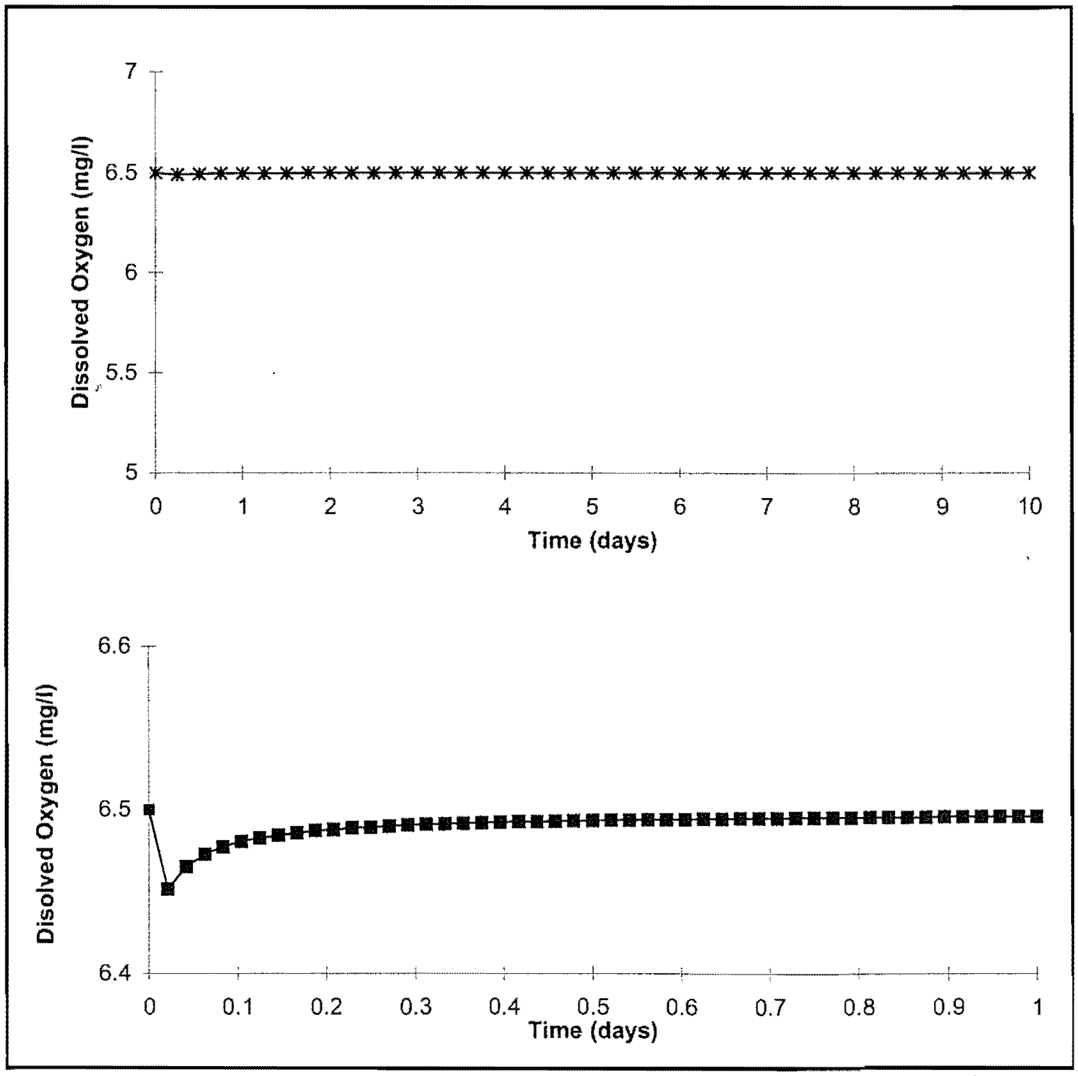

Figure 7

DO Sag in the Receiving Water for 10-day and 1-day Periods.

(Note: change in scale for DO between the two plots)

The farfield oxygen demand predictions shown in Figure 7 were calculated on the anticipated maximum BOD_5 concentration of 200 mg/l and a maximum effluent discharge of 6 mgd. The calculation was done for a CID of 190:1 (it is noted that variations in the CID between 192 and 187 representing the three diffuser configurations has no effect on the farfield DO analysis, and the analysis was not done for all three cases). The subsequent diffusion was calculated based on open ocean conditions (4/3 power) for the diffusion coefficient. The methods and parameters used for the calculations are the same as described in the 301(h) TSD. Calculations were made for 10 days at 0.25-day increments and for 1 day at 0.5-hour increments.

8. Conclusions and Recommendations

This Supporting Technical Analysis was done to facilitate responses to the 301(h) waiver renewal questionnaire. There are a number of conclusions and recommendations that can be drawn form the results of the analysis. These conclusions and recommendations are directly or indirectly associated with the objectives of the analyses, although not all may be directly reflected in the responses given in the questionnaire. The conclusions and recommendations concern the diffuser and potential modifications, the need for and structure of receiving water quality monitoring, and the future requirements for effluent monitoring.

Diffuser Modifications

The diffuser configuration was designed with progressively larger ports along the barrel in the offshore direction, which is the typical design approach for multiport diffusers. It has been reported that the diffuser was constructed in reverse, with the larger ports inshore and progressively smaller ports offshore. The port sizes are achieved by orifice plates installed on the ends of the risers, and all riser diameters are identical. It has been suggested that the orifice plates can be removed resulting in six identical port sizes, larger than the existing ports. The following conclusions concerning the diffuser configuration can be developed form the analyses in this document:

- The dilution, under both average and maximum (critical) conditions is essentially identical, within the accuracy of the initial dilution model, and there is no advantage displayed by any of the three configurations.

- The head loss is essentially identical for the design and as-built configurations. However, the head loss through the diffuser would be substantially less for the alternate configuration with all orifice plates removed and constant port diameters equal to the riser diameters. This is a potential advantage for the alternate configuration, although the actual overall importance of lower head loss through the diffuser will depend on the total head loss through the outfall pipeline.

- The velocity in the diffuser barrel is important to prevent sedimentation and, under low discharge conditions, recirculation of ambient water. The design configuration provides higher velocities throughout the diffuser barrel than the as-built configuration. Therefore it would be an advantage to change the orifice plates to reflect the actual design. The alternate configuration, with all plates removed, provides barrel velocities that are intermediate between the design and as-built configurations. In all cases none of the barrel velocities are sufficient to prevent some sedimentation, particularly at lower flows, so the relative advantages may be minimal. It is noted, that higher barrel velocities can reduce future maintenance related to sedimentation and recirculation.

- The removal of the orifice plates, as suggested for the alternate configuration, lowers the risk of port blockage, particularly for the small ports, if unintended objects were introduced to the discharge stream. This could be a maintenance advantage.

It is recommended that the as-built diffuser configuration be modified to reflect the design configuration or the alternate configuration with orifice plates removed. The choice between these two configurations depends on the need for maintenance caused by sedimentation within the barrel, the potential for port blockage of the small ports, and the potential advantage of lower head losses. Since the dilution performance of all three configurations is nearly identical, the decision should be based on past experience with the existing diffuser and judgments about future maintenance requirements.

Receiving Water Monitoring

It is unlikely, because of the high dilution achieved by the diffuser and the good flushing characteristics of the receiving water, that any effect of the effluent discharge can be measured in a receiving water monitoring effort targeted at the ZID boundary and beyond. Examination of the available data (see Appendix 2) indicate that the variability in concentrations of targeted parameters are not attributable to the discharge. Therefore, the existing monitoring requirements do not provide any useful data concerning the effects of the discharge on the receiving water.

It is recognized that the 301(h) program requires a continuing assessment to determine if the discharge is having any detrimental effects on the environment. Water column monitoring will not provide this information for the reasons mentioned above. The best approach to examine this question is quite likely to look at the sediments, which tend to integrate effects over long periods of time. Since sediment quality and the response of the benthic community structure change slowly, this monitoring need not be done frequently to determine if the discharge is having any effect.

Since water column concentrations are dependent on the initial dilution and transport of the effluent plume, additional monitoring activities could be developed to examine and verify the dilution performance of the diffuser. This could be accomplished with a one time dye study to verify model predictions of plume behavior. However, a great deal of experience with the diffuser model applied to multiport diffusers in open coastal waters indicates that the model predictions are generally conservative (under predict dilution). Furthermore, there is little chance of substantial tidal reflux of the plume, shoreline trapping, or background increases in effluent concentration. These affects are associated with discharges in more confined locations where boundaries constrain the transport of the effluent plume.

Transport of bacteria to the shoreline after discharge could also be a perceived, although unlikely, issue. There are no nearby recreational areas, and other sources of bacteria (runoff from permanent and intermittent streams and exchange of water between Pala Lagoon and the open coastal waters, could easily dominate the bacterial concentrations, if any, along the shoreline. A survey of shoreline bacteria, adequately designed to account for other sources, might be useful to characterize the shoreline distribution and develop a baseline survey for future reference. However, an ongoing monitoring plan is not likely to be useful for characterizing the effects of the Tafuna WWTP discharge.

Based on the above discussions and a careful examination of the existing data, the following recommendations are made for future monitoring:

- Receiving water quality monitoring as currently conducted should be discontinued.

- A sediment monitoring study, including selected chemical parameters and benthic community enumeration should be conducted once per permit cycle. The study should include stations near the edge of the ZID, in the farfield along the expected trajectory of the plume, and at reference sites. A study plan could be developed and approved as a special condition of the renewal permit.

- A one time shoreline bacteria study is recommended. The study should be designed to enable identification, to the extent possible, of sources other than the effluent discharge. A study plan could be developed and approved as a special condition of the renewal permit.

- If the sediment study or the shoreline bacteria shows potential impacts then a dye study to define the plume dilution and transport (nearfield and farfield) could be done. But such a study is not recommended unless and until other monitoring indicates it is necessary.

Effluent Monitoring

Effluent monitoring as currently conducted should be continued generally as is, since it adequately characterizes the effluent quality in terms of permit limitations. It is further recommended that the permit should state any additional monitoring that will be required for subsequent permit renewals. Specific monitoring requirements could include:

- The number and timing of priority pollutant scans. it is recommended that only single scan in the year before permit renewal is necessary. Parameters tested should be consistent with Form 2A, Section D.

- The number and timing of other parameters that are required for permit renewal (for example parameters required in Form 2A, Section B.6 of the NPDES application).

- Testing for whole effluent toxicity should be maintained at the existing frequency of once per year using a single test organism for chronic and acute toxicity determination.

APPENDICES

TO THE
SUPPORTING TECHNICAL ANALYSIS FOR 301(H) WAIVER RENEWAL
FOR THE TAFUNA SEWAGE TREATMENT PLANT

Appendix 1: Effluent Monitoring Data

Appendix 2: Receiving Water Monitoring Data

Appendix 3: Results of Hydraulic Analyses of Diffuser Configurations

Appendix 4: Diffuser Initial Dilution Model Predictions