University of Nevada, Reno

Wastewater Treatment Design: NAS Fallon WWTP

A thesis submitted in partial fulfillment
of the requirements for the degree of

BACHELOR OF SCIENCE, ENVIRONMENTAL ENGINEERING

by

KAITLIN KIMBROUGH and THOMAS STROUD

Eric Marchand, Ph.D., Thesis Advisor

August, 2014

UNIVERSITY OF NEVADA RENO

THE HONORS PROGRAM

We recommend that the thesis
prepared under our supervision by

KAITLIN C. KIMBROUGH

entitled

Wastewater Treatment Design: NAS Fallon WWTP

be accepted in partial fulfillment of the
requirements for the degree of

BACHELOR OF SCIENCE, ENVIRONMENTAL ENGINEERING

Eric Marchand, Ph.D., Thesis Advisor

Tamara Valentine, Ph.D., Director, Honors Program

August, 2014

Executive Summary

Wastewater Treatment Facility Proposal

Fallon, Nevada

A new wastewater treatment facility is needed at Naval Air Station (NAS) Fallon. NAS Fallon recently became the new home for the Naval Fighter Weapons School (TOPGUN), ensuring that the base will remain highly active for years to come. Between residential and commercial flows, the existing facilities accepts and treats approximately 0.25 MGD of wastewater. The existing facility holds an outdated design and compromised structures. In addition to weak infrastructure, ever increasing effluent regulations have created the need for a new facility a priority. While wastewater loading flows are not expected to drastically increase, a reliable, effective facility is needed for the future operation of NAS Fallon. Upon completion of this new facility, NAS Fallon will be capable of successfully accepting, treating, and disposing of wastewater flows up to 1 MGD for the foreseeable future.

The new facility will be composed of a full array of wastewater treatment processes. Influent wastewater flows will be collected in wet and dry well system at the mouth of the facility. From there, flows will be pumped through a coarse and fine screening process to remove large debris. Following screening, water will enter an aerated grit chamber for high rate particle removal to promote water clarity. Next, a step-feed biological nutrient removal process will be utilized to remove nitrogen and phosphorus content in order to meet effluent regulations. Then, circular sedimentation tanks will allow for any residual solids to be removed. Finally, the wastewater will pass through disinfection where viruses and bacteria will be managed before the water leaves the facility. Similar to the existing facility, effluent water will be discharged into the drain system feeding the Stillwater Nation Wildlife Refuge.

Full considerations regarding design alternatives and cost effectiveness were analyzed during the design process for this facility. The following proposal holds the summation of that analysis.

Table of Contents

Tables

Figures

1.0 Introduction

1.1 Existing Infrastructure

Naval Air Station (NAS) Fallon operates a wastewater treatment plant (WWTP) that receives primarily residential and commercial waste with a small amount of waste from industrial sources. The WWTP also receives stormwater. The WWTP was constructed in 1985 and subsequent upgrades including the addition of two sequencing batch reactors (SBRs) and a chlorine contact tank was built in 1995. The configuration of the WWTP includes a lift station with three chopper pumps, teacup grit separator system, two SBRs, chlorine contact tank, mechanical building (control system, chemical storage, pumps, and blowers), two aerated sludge lagoons, and standby ponds for drying and accumulation of sludge. Fallon's current WWTP configuration is shown in Figure 1.

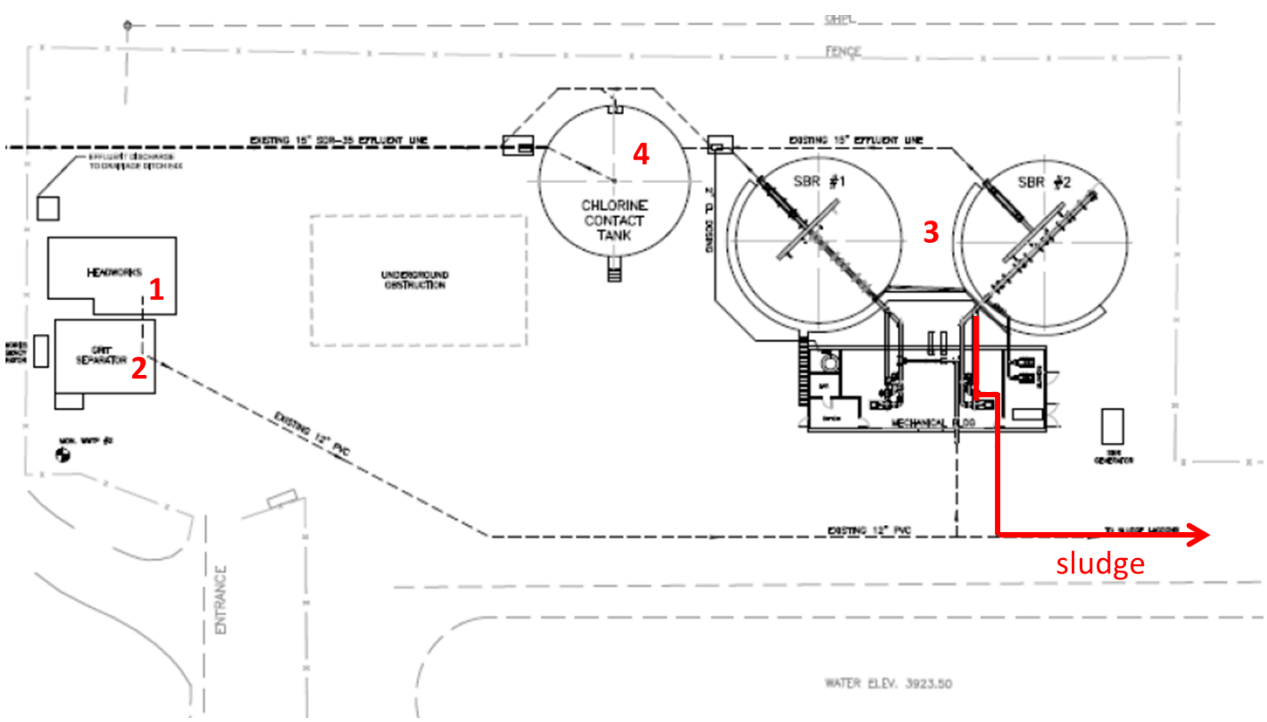


Figure 1: Configuration of NAS Fallon's current WWTP.

Wastewater initially enters the WWTP at the v-shaped wet well where it is pumped. The three chopper pumps are currently not used at the facility. Grit from the wastewater is removed by the teacup grit separator. Next the wastewater enters the SBR system which alternatively receives influent at 3-hour intervals. After the SBR fills, the wastewater is aerated until complete biodegradation is theoretically achieved. Next, the sludge is allowed time to settle and the liquid effluent is decanted. The decanted effluent then flows to the chlorine contact chamber. The chlorinated effluent is discharged to the E4X canal and subsequently to the Lower Diagonal Drain (LDD). The resulting sludge in the SBRs is transferred to two aerobic sludge lagoons and then the sludge is transferred to two 5-million-gallon standby ponds.

1.2 Issues and Proposal Objectives

Existing wastewater treatment processes at NAS Fallon fail to meet the NPDES Permit for total-N discharge limits. Existing data from December 2007 to April 2008 shows that NAS Fallon experienced 7 total-N concentrations that exceeded the threshold set by the NPDES permit (10 mg/L total-N). Also, treatment efficiency at NAS Fallon is impacted by hot and cold winter temperatures, organic mass loading fluctuations, hydraulic retention times, variations in flows, and possible toxicity in SBRs due to arsenic, chloride, and ammonia/nitrate. Also, NAS Fallon experiences inflow and infiltration issues due to high groundwater and aged infrastructure. The current wet well at the facility is not accessible due to high carbon monoxide concentrations. Also, the chopper pumps are not currently used due to continual clogging and maintenance issues. The teacup grit settlers are currently not used due to the low efficiency at the small flow.

Currently, NAS Fallon allows sludge to accumulate in standby ponds for longer than the permitted two year period.

Reconstruction of the current NAS Fallon WWTP will target improved treatment efficiency of total-N (<10 mg/L), near future regulations, flexibility in flow fluctuations, solids handling, cold weather enhancements, reliability and redundancy, and efficient use of land space. The proposed design suggests demolition of the current WWTP and construction of an entirely new treatment facility. Significant improvements will be made to the headworks and primary treatment processes in order to improve pumping capacity, grit removal, and solids removal.

2.0 Site Specifications

2.1 Geographical Information

NAS Fallon is located approximately 6 miles southeast of Fallon, Nevada and 60 miles east of Reno, Nevada. The base itself occupies a total of 8,670 acres while the current wastewater facility occupies about three acres. Figure 2 displays the location of the wastewater treatment facility property relative to the rest of NAS Fallon.

Figure 2: The location of the current wastewater treatment facility property (outlined in red) relative to NAS Fallon.

2.2 Site Constraints

NAS Fallon faces several site constraints. In particular, the water table at NAS Fallon resides 7 feet below grade. All future construction on site should be shallow or located on grade. In turn, NAS Fallon has a generally flat elevation profile. This will limit the use of gravity flow onsite and will lead to higher energy costs associated with pumping.

Also, NAS Fallon has limited land space available for construction. Expansion to the north of the SBRs is limited by the location of a fence line and utility. If needed, provisions will have to be made in order to construct to the north of the SBRs. The plant is also located close (~100 feet) to a runway. In order to expand toward the runway, permission will have to be granted. The elevation of new structures must not exceed the elevation of the SBRs. Land space is further limited by a subsurface obstruction which has been partially removed. Location of the retired Imhoff Tank is shown in Figure 1. Abundant land space is available to the south where the drying beds are currently located. In order to use the space to the south, the site will have to be filled in and smoothed out.

The current design proposal suggests using the land to the south of the current WWTP; provisions will have to be made in order to prepare this land for construction.

Due to the location of NAS Fallon, the region experiences climatic seasonal variations with hot summers and cold winters. The minimum temperatures can lead to freezing and inadequate biomass concentration. Cold weather enhancements should be incorporated in renovations and new constructions.

3.0 Wastewater Characteristics

3.1 Water Flow

Current site specifications rate NAS Fallon's WWTP capacity at 0.75 MGD with an average daily flow (ADF) of 0.25 MGD. Site personnel state that flow data from March 2008 to March 2009 is the most representative of wastewater flow into the plant. A flow analysis was conducted comparing data from March 2008 to March 2009 to data collected in the last 5 years (2008-2013) in order to justify whether 0.25 MGD was a valid assumption. Generally, water flow analysis requires more than three years of flow data, so the most recent 5 year time period was chosen. Flow data for the given time periods was analyzed for expected wet and dry periods in order to gauge the effects inflow and infiltration (I&I) as well as the input of storm water has on wastewater influent flows. Figures 3 and 4 depict the ADFs for the given time periods as well as the ADF expected year around and for wet and dry periods.

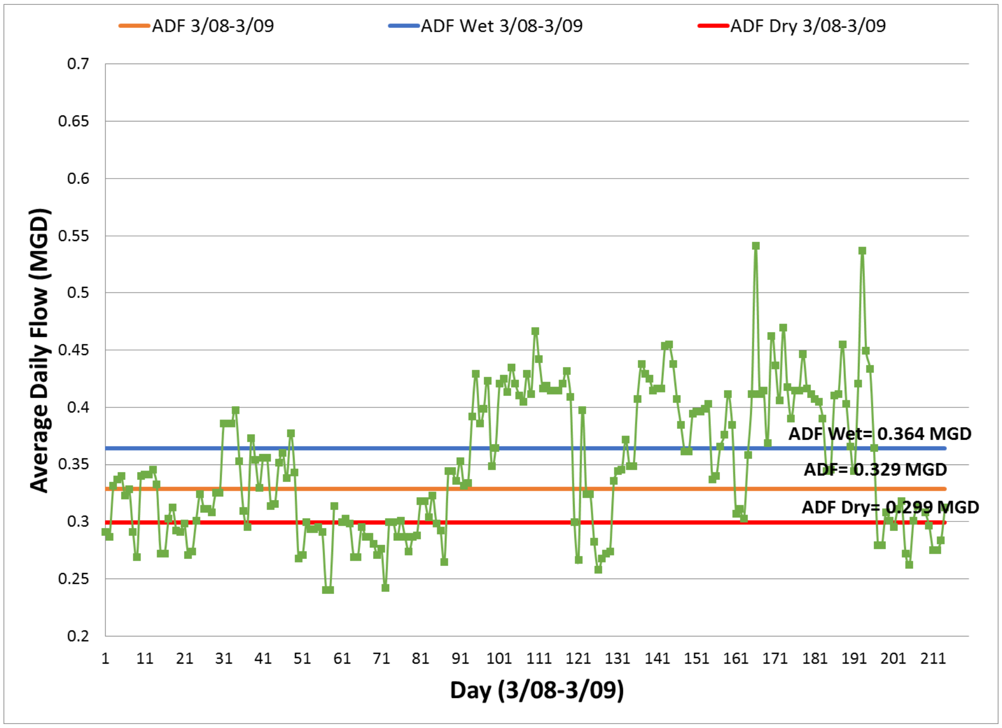


Figure 3: Flow parameters derived from flow analysis of March 2008-March 2009 data.

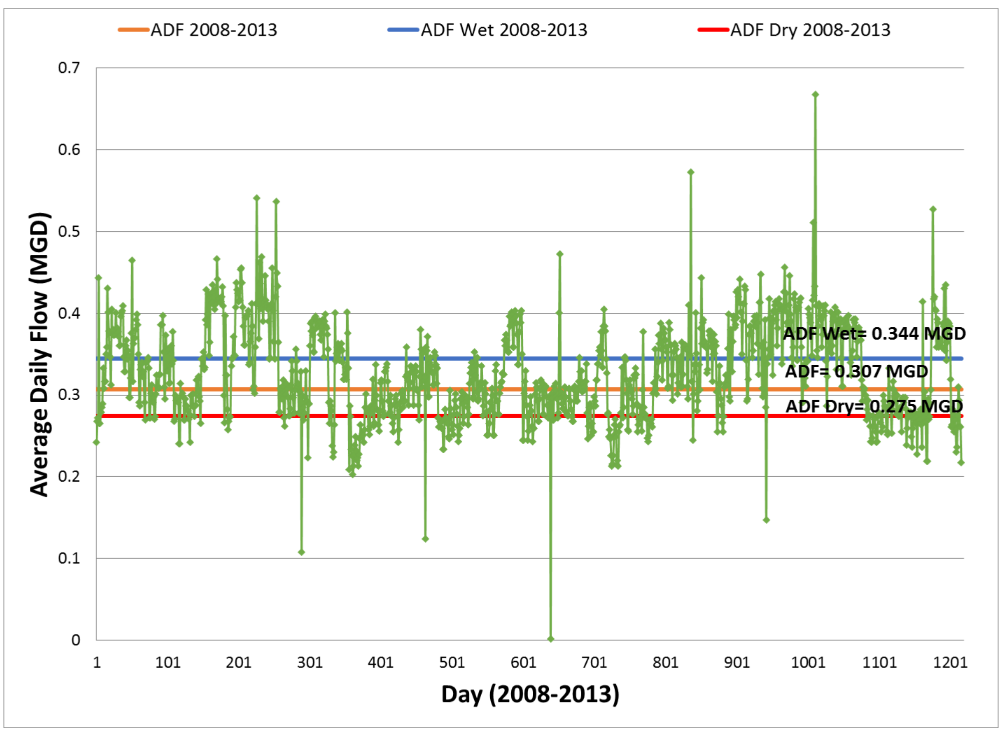


Figure 4: Flow parameters derived from the flow analysis of 2008-2013 data.

Flow trends have shown to generally decrease over time, and the flow data from 2008 to 2009 is approximately .02 MGD higher than flow data from 2008 to 2013. The data from the 5 year period also showed a greater variation in expected high and low flows. A large variation in flows is typically expected for smaller plants (Metcalf & Eddy, 2014). In order to plan for the expected lower flows and greater variations in peak and low flows, flow parameters derived from the 5 year flow analysis were used to design the future construction of NAS Fallon's WWTP. Additional data analysis for the two time periods is located in Appendix A. Flow parameters derived from the data analysis are shown in Table 1. A peaking factor of 4.00 was used to estimate peak hour flow (PHF) (Metcalf & Eddy, 2014).

Table 1: Flow measurements from analysis of 2008-2013 flow data.

	MGD	GPM	CFS	m^3/d
ADF	0.307	213	0.474	1,161
Wet ADF	0.344	239	0.533	1,304
Dry ADF	0.275	191	0.425	1,039
Min. ADF	0.00144	1.00	0.002	5.45
Max. ADF	0.668	464	1.03	2,529
Min. MDF	0.219	0.309	0.219	830
Max. MDF	0.402	0.402	0.307	1,522
PF(PHF/ADF)	4.00	4.00	4.00	4.00
PHF	1.23	852	1.90	4,642

The population of NAS Fallon is expected to remain fairly constant at 5,000. Additional capacity is not expected to exceed the current capacity of the WWTP at 0.75 MGD. In order to justify this claim, wastewater flows were estimated for the current NAS Fallon population (Table 2).

Table 2: Estimation of plant capacity for NAS Fallon population of 5,000 using typically accepted flow values for a variety of sources (Metcalf & Eddy, 2014).

	Low	High	Average
	(gal/cap-d)	(gal/cap-d)	(gal/cap-d)
Industry WW	10.0	100	10.0
Domestic WW	51.0	189	99.0
Loss and Waste	5.00	20.0	12.5
Influx and Infiltration	5.28	39.6	26.4
Total (MGD)	0.356	1.74	0.740

The estimation of the total water flows is an overestimate compared to Table 1 values. This is most likely attributed to the high flow contribution attributed to industrial wastewater. NAS Fallon industrial wastewater values are characterized as "light" and probably closer to the lower end (10.0 gal/cap-d). Also, the flow attributed to loss and waste and I&I are expected to decrease in the future as NAS Fallon continues to improve I&I issues and reduces industrial discharges. Also, water conservation measures are expected to take place in the near future and domestic wastewater production will decrease.

4.0 Headworks

4.1 Wet/ Dry Well

Currently, NAS Fallon's WWTP has a v-shaped wet well that intermittently pumps wastewater from a lower elevation to the teacup grit settlers at a higher elevation. The influent wet well is housed in a headworks building. The v-shaped wet well is not well ventilated and the plant operator is unable to perform maintenance on the wet well and pumps due to high carbon monoxide concentrations. Plant operators have expressed dissatisfaction with the three chopper pumps that are currently used to pump water from the wet well. The new design proposes an entirely new construction implementing a combined wet well and dry well lift station with installation of three Tourque-Flow pumps rated to handle abrasive grit and prevent clogging. The existing wet well will be not be utilized and the new wet well and dry will include the following: installation of equipment control and alarm system, 3 individual intake ports, odor control system, and a ventilation system. In addition, a dry well will be constructed adjacent to the new wet well structure with a roof access hatch and bridge crane in order to remove pumps for maintenance. The three chopper pumps will be replaced with more reliable and heavy duty vortex pumps. A ventilation system and sump will also be installed in the new dry well. Drawings of the wet well and dry well are shown in Figure 5.

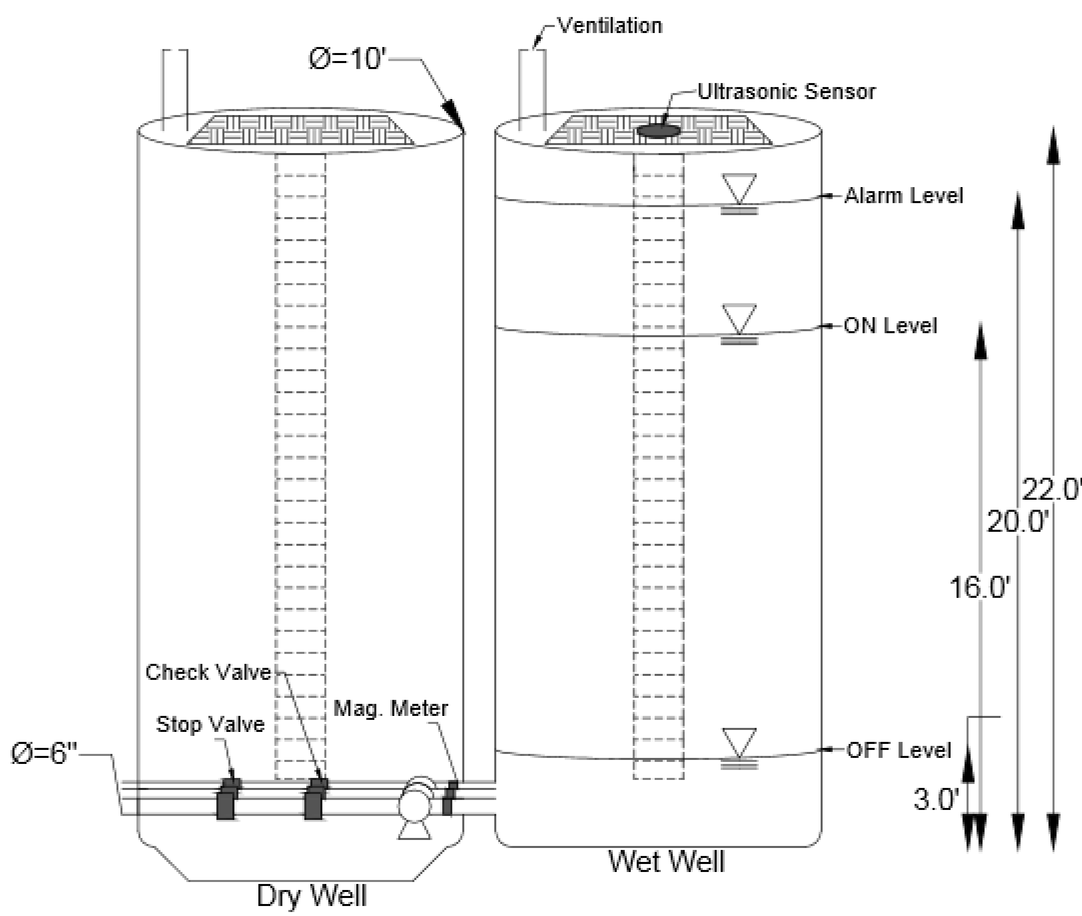


Figure 5: Profile view of the wet well and dry well lift station.

A combined wet well and dry well lift station was chosen over a submersible lift station for a variety of reasons. Though dry well pumping is usually more expensive, this kind of pumping allows for easy inspection and maintenance of pumps. Due to the abrasive nature of wastewater, it is important that operators can easily inspect and repair pumps. Also, dry well pumping allows for easier redundancy in pumps by installing three pumps in the dry well rather than relying on a single submersible pump. The redundancy implemented with the dry well design allows for more reliable control of wastewater flow to the NAS Fallon WWTP.

To improve lift-station reliability, torque-flow, vortex pumps were chosen due to their abrasion resistance, recessed impeller design, and non-clogging nature. Additional manufacturing information for the pumps is in Appendix B. The pumps were sized in order to handle the PHF to minimize the chance of overflow (~850 gpm), but the operating point of the pumps will be 600 gpm and will provide 28.4 feet of head (Figure 6) in order to overcome subsequent headlosses. All three pump sizes were kept consistent in order to allow for easier maintenance. The pumps will each be connected to a 6 inch intake port placed 1 foot from the bottom of the wet well. The pumps will be equipped with check valves, isolation valves (stop valves), and magnetic flow meters. An ultrasonic sensor system will be installed in the wet well in order to control the pumps. The ultrasonic sensor will be mounted at the center of the wet well 0.5 feet below grade (Appendix B). The ultrasonic sensor will trigger a single pump to switch on when wastewater is 6 feet below grade. The pump will switch off when wastewater is 3 feet from the bottom of the wet well or 19 feet below grade. Also, at 2 feet below ground surface elevation the ultrasonic sensor will sound an alarm in the control room and will turn on multiple pumps in order to prevent overflow. The pumps will be run sequentially in order to reduce wear on a single pump, and two pumps will be run simultaneously in case of an overflow event at PHF. The drainage time for the wet well at full capacity is approximately 13 minutes. Figure 6 depicts the system curve overlaid on the pump curve. The operating point of the torque flow pump was determined from the intersection of the system curve and pump curve to be 650 gpm at 28.4 feet of head. Additional manufacturing data for WEMCO Weir Speciality Pumps is located in Appendix B.

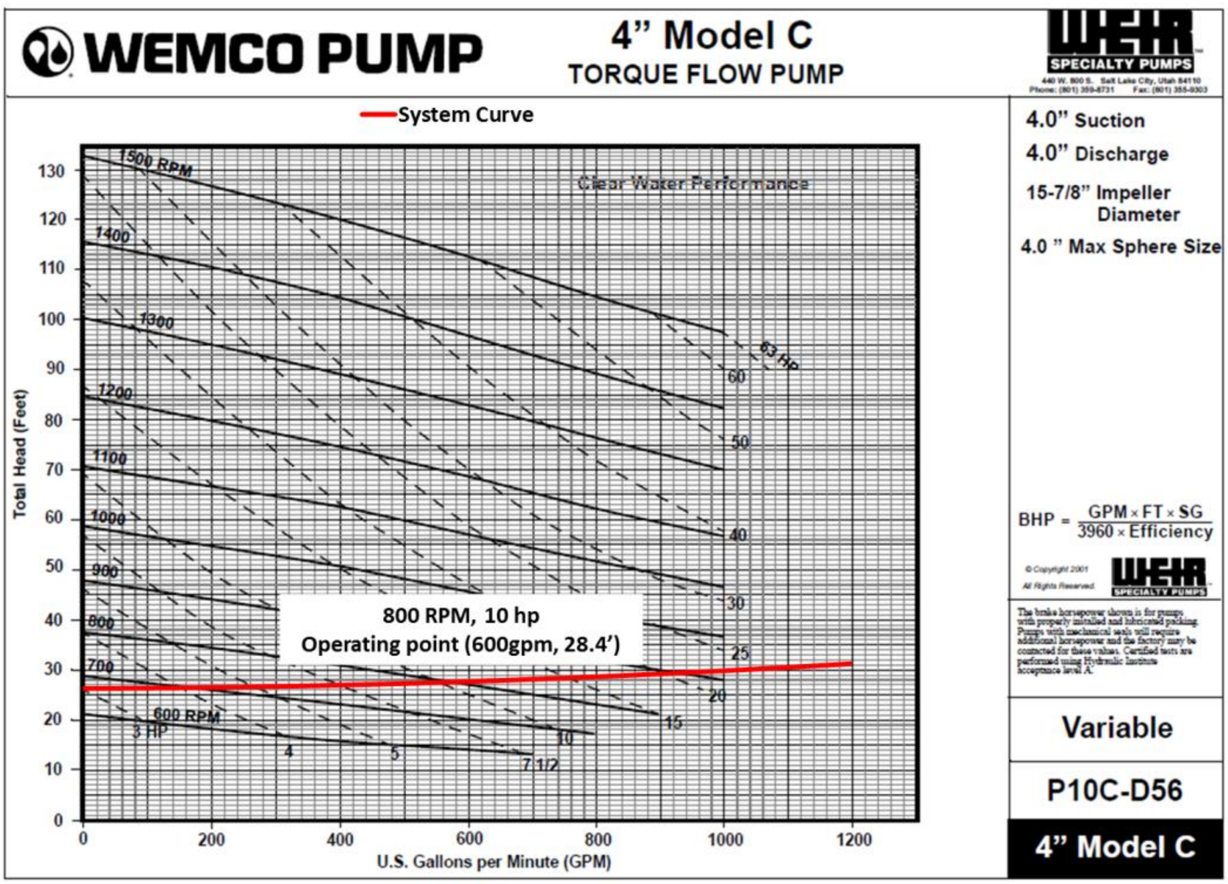


Figure 6: Pump curve and system curve were analyzed in order to determine the operating point of the pump.

After the wastewater is pumped above grade, the pipe will discharge into a channel that will lead to subsequent coarse screening, fine screening, and grit removal.

5.0 Primary Treatment

5.1 Coarse Solids Screening

Mechanically cleaned bar screens were designed based on the average daily and peak daily flows. In order to maintain the proper velocity, a rectangular open channel with the dimensions 5.0’x0.88’x3.0’ (LxWxD) was designed. The depth of flow during average influent conditions is designed to be approximately 0.2 feet and 0.4 feet during peak conditions. There will be a bar screen located four feet from the mouth of the channel. The bar screen will span the width of the channel. The screen design calls for five evenly spaced bars, each with a width of 0.6 inches. In addition, the channel will be fitted with sluice gates before and after the coarse screen in order to provide flow controls during periods of irregular flow or downtime. Cleaning of the bar screen can be accomplished by a reciprocating rake device. Two identical channels will be constructed for redundancy. Figure 7 depicts physical design parameters of the coarse screen installation. Table 3 summarizes important design criteria regarding the coarse screening process. Because of the relatively shallow water depth through the channels, the channels should be inspected daily for deposited material. If unwanted deposits are seen, they can be mobilized with a pressurized hose, or removed with a rake.

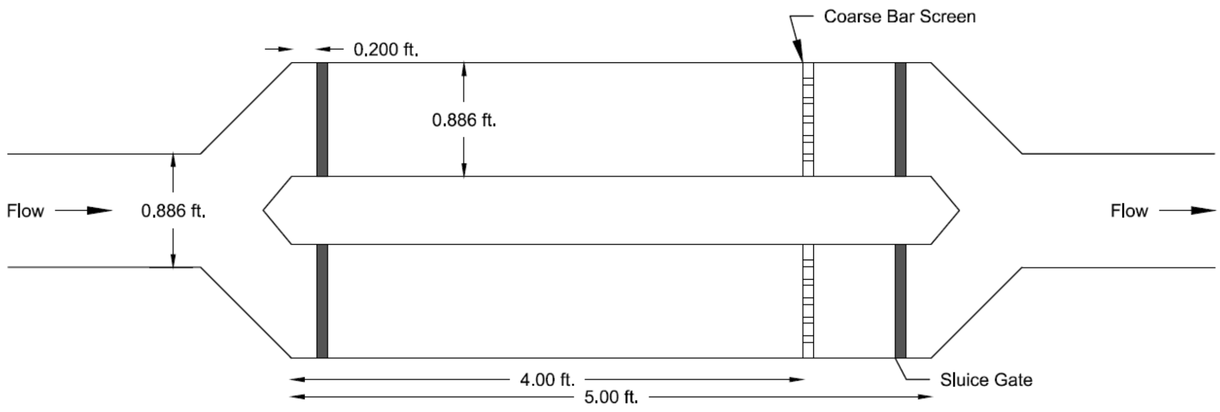


Figure 7: Plan view of proposed coarse screening process.

Table 3: Design parameters at different flow regimes for coarse bar screening.

	Q_{ADF}	Q_{PHF}
Approach Velocity (ft/s)	1.00	3.28
Screen Opening (in.)	3.50	3.50
Clear Screen Headloss (in.)	0.79	0.79
Clogged Screen Headloss (in.)	1.50	11.6
Allowable Headloss (in.)	1.50	13.5

5.2 Fine Solids Screening

A fully automated fine screening process was designed using a step screen unit from Hydro-Dyne Engineering, Inc. A step screen unit was chosen for two distinct reasons. With relatively low influent flows below 1 MGD, an effective step screen can eliminate the need for a coarse screen and primary clarifier. The design opted to include a fine solids screening unit rather than a primary clarifier. For comparative purposes, the design of a primary clarifier will be discussed as well. In addition, the step screen unit design works well with the small channel dimensions necessary for acceptable flow velocities. The step screen will be located four feet from the mouth of the channel in an open channel with dimensions 7.0'x2.5'x3.0' (LxWxD) and a 0.5% slope. The step screen has 0.35 inch openings, a full channel width of 2.5 feet and an effective height of 95 inches. The screen itself is angled at sixty degrees from the horizontal. The screenings collected may sum a maximum of 31.5 ft^3/week. Two 2-yard (54 ft^3) dumpsters should be purchased for handling and disposal of the screenings. The dumpsters will then be cycled each week or as necessary (Appendix B). The depth of flow approaching the screening unit will be approximately 0.2 feet and 0.4 feet during average and peak conditions respectively. Approach velocities will vary between 2.4 and 3.6 fps. Headloss across the fine screen units will vary between 1 and 8.25 inches depending on the flow and screen efficiency conditions. The channel will be fitted with sluice gates before and after the step screen unit to provide flow control during periods of irregular flow or downtime. Two identical channels will be constructed for redundancy. Figure 8 depicts physical design parameters of the proposed fine screening process and Table 4 summarizes important design criteria.

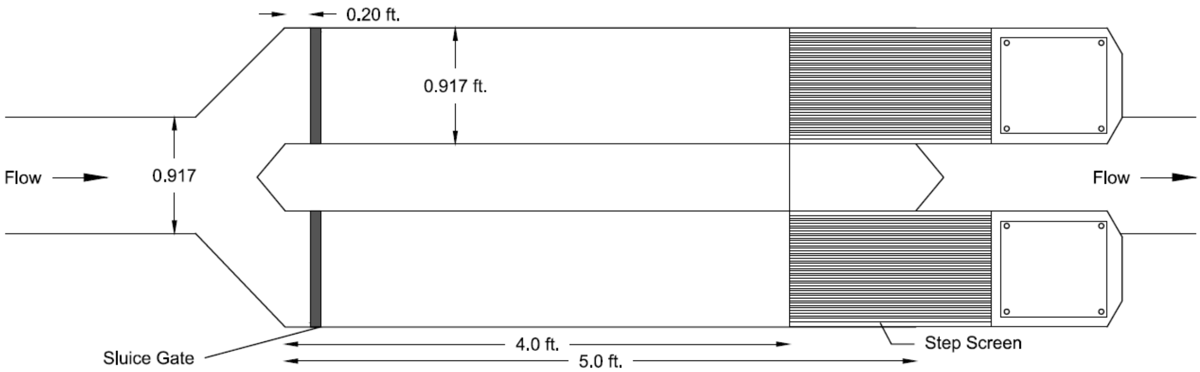


Figure 8: Plan view of proposed fine screening process (End process sluice gates are located underneath the step screen units).

Table 4: Summary of important design criteria regarding the fine screen process.

	$Q_{Average}$	Q_{Peak}
Approach Velocity (ft/s)	2.40	3.60
Screen Opening (in.)	0.35	0.35
Clean Screen Headloss (in.)	1.00	3.20
Clogged Screen Headloss (in.)	3.48	8.25
Allowable Headloss (in.)	6.00	24.00
Screening Mass (lb/d)	94.0	280
Volume of Screening (ft^3/d)	1.50	4.50
Typical Screening Volume (ft^3/d)	1.00 – 2.00	3.00 - 6.00

5.3 Aerated Grit Chamber

Currently, NAS Fallon has a teacup grit separator in order to remove grit, but plant operators have not been satisfied with the grit removal efficiency. Plant operators have requested a new grit removal system. The existing teacup grit separator will be used as a last resort backup.

Two aerated grit chambers were designed in place of the teacup grit separator. The aerated grit chamber was chosen due to its high grit removal efficiency and built in flexibility. Also, alternative grit remover systems were proprietary and manufacturers did not make systems compatible with the small design volume required. The aerated grit chamber can achieve high grit removal for various grit contents and flow regimes by implementing a variable air flow. Two aerated grit chambers are proposed for redundancy and reliability. The aerated grit chambers will share a common wall and air supply unit (Figures 9 and 10). A common channel will feed the two chambers and will be equipped with two influent and effluent weir gates in order to control flow. Effluent wastewater will flow from the grit chambers through a weir into a common channel. Each grit chamber can be isolated for maintenance. Figures 9 through 11 depict physical design parameters for the aerated grit chambers.

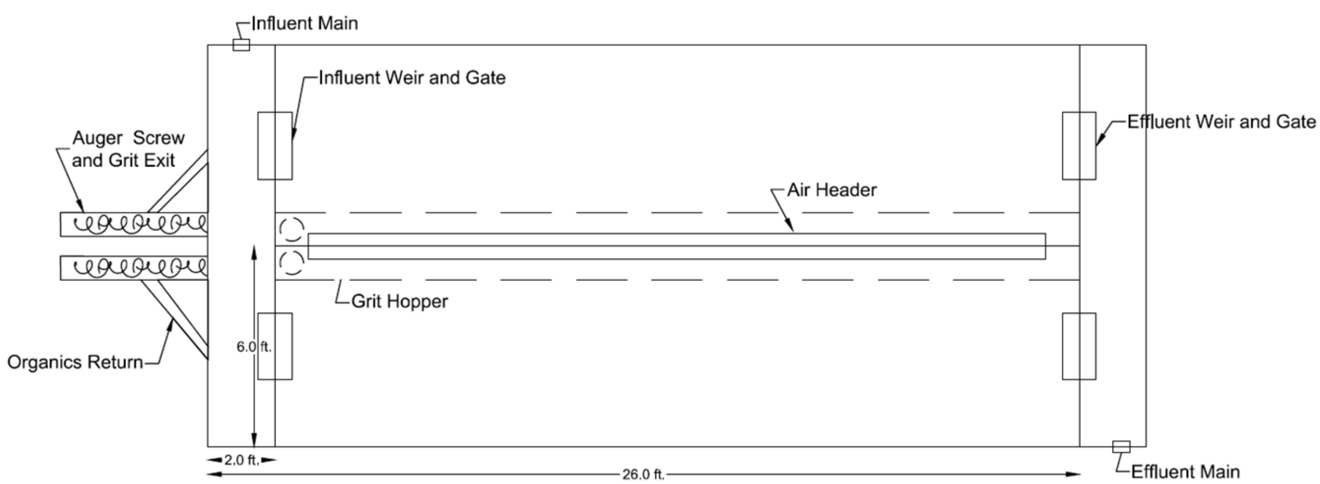


Figure 9: Plan view of aerated grit chamber.

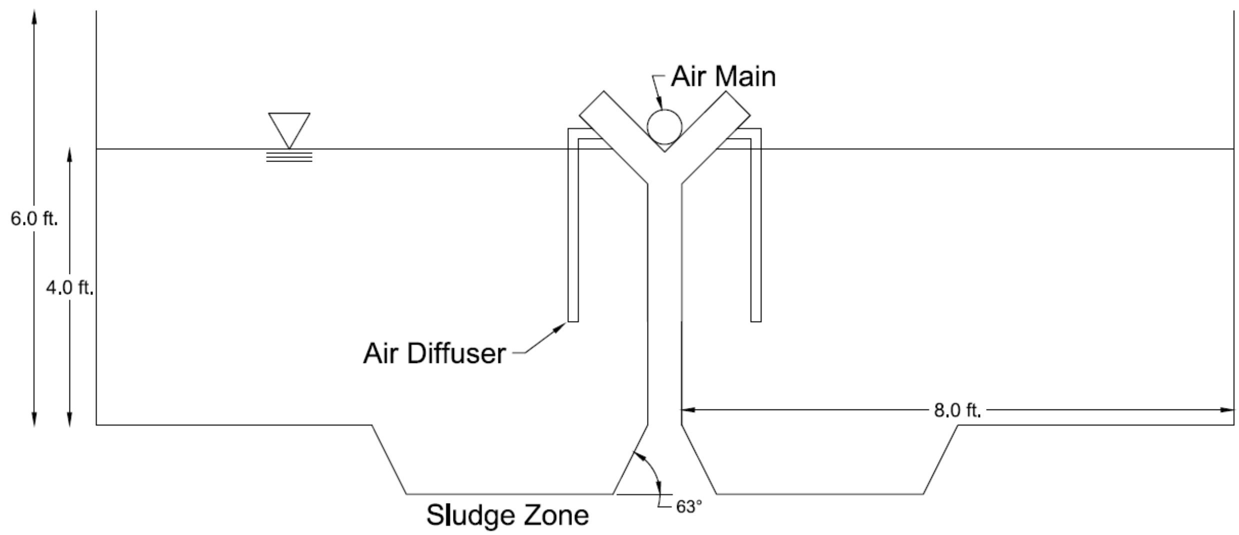


Figure 10: Profile view of aerated grit chamber.

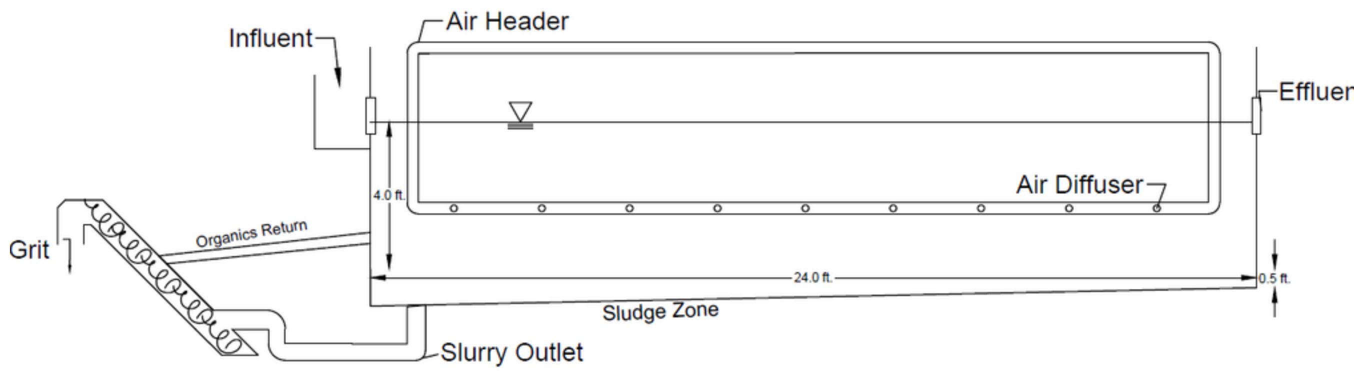


Figure 11: Profile view of aerated grit chamber.

Each grit chamber was sized based on the expected PHF, so that in case one chamber needs to be offline, the other grit chamber can handle the PHF. Table 5 compares typical design criteria against the proposed design. The aerated grit chamber required a smaller volume than the design criteria allowed. In this case, the dimension ratios were maintained while the depth, length, and width values were below the accepted low threshold. The detention time is on the high range (~5 min.). Table 6 shows the expected detention times for a variety of flow regimes. All flows below

PHF have a detention time greater than five minutes. Grit quantity and air supply were also estimated (Table 5). Grit quantity for the proposed design assumes an average value of 2 ft^3/Mgal. Values for the proposed design for air and grit quantities have units of ft^3/min and ft^3/day.

Table 5: Proposed aerated grit chamber design compared to typical design criteria.

	Design Criteria			
	Low	High	Average	Proposed Design
t_d @ PHF (min.)	2.00	5.00	3.00	5.06
Depth (ft)	7.00	16.0	11.5	4.00
Length (ft)	25.0	65.0	45.0	24.0
Width (ft)	8.00	23.0	15.5	6.00
W:D	1:1	5:1	1.5:1	1.5:1
L:W	3:1	5:1	4:1	4.5:1
Air supply/L (ft^3/ft-min)	3	8	5.5	132
Grit Quantity (ft^3/Mgal)	0.5	27	2	2.45

Table 6: Expected detention times in aerated grit chamber for various flow regimes.

	Q	t_d
	MGD	min
ADF	0.307	20.2
Wet ADF	0.344	18.0
Dry ADF	0.275	22.6
Min. ADF	0.001	4,308
Max. ADF	0.668	9.28
Min. MDF	0.219	28.3
Max. MDF	0.402	15.4
PHF	1.23	5.06

After grit settles to the bottom of the steeply sloped hopper, a conveyer belt will cause grit to move towards the influent end of the grit chamber at the slurry outlet. A recessed impeller pump will pump the grit slurry from the outlet to an auger screw. The auger screw will be inclined 60 degrees and will be equipped with water sprayers to wash the grit. The auger screw will have an outlet that allows organic matter to return back to the aerated grit chamber. After grit is washed, grit will be disposed of in a 2-yard dumpster and transported to the local landfill (Appendix B).

5.4 Primary Clarifier

A primary clarifier was considered as an alternative to the fine solids screening unit. The clarifier design calls for a circular tank with a depth of 13 feet and a diameter of 18 feet. The clarifier will be fitted with a radial wiper that will rotate along the water surface to collect floating scum and direct it to a scum trough located within the unit. In addition, the clarifier will have a rotating scraper along the bottom of the unit, which will effectively remove settled solids from the process. Both the wiper and the scraper will be attached to a shaft and motor located at the center of the unit. Because of the relatively small dimensions of the clarifier, a relatively slow rotation speed of approximately 0.02 revolutions per minute should be considered. A single catwalk will provide access to the central motor. Process water will be pumped up a well located at the center of the clarifier. The upper portion center well will be fitted with a circular energy dissipation wall to reduce water velocity throughout the process. At average flows, the overflow rate will be 980 gal/ft^2-d with a retention time of approximately 2.5 hours. At peak flows, the overflow rate increases to 3900 gal/ft^2-d with a detention time of just 0.6 hours. At peak flow, the design overflow rate is outside of the ideal range for primary clarifiers; however, with effective operation of the wet well, long periods of high loading can be avoided. The peak horizontal flow velocity was calculated to be less than the typical scour velocity of wastewater grit; implying that the overall hydraulic design of the clarifier will be effective. A second clarifying unit should be constructed for redundancy. Table 7 contains a summary of the clarifiers' physical design parameters. Table 8 contains a summary of important hydraulic design parameters. While primary clarification is a successful and useful process, it is ultimately unnecessary for the expected volume of wastewater entering this facility. For this reason, it has been left out of the final design proposal. Instead, the coarse and fine screening process was chosen.

Table 7: Summary of important design considerations for the primary clarifying process.

	Q_{ADF}	Q_{PHF}
Detention Time (hr)	2.40	0.60
Overflow Rate (gal/ft^2-d)	980	3,900
Min. Overflow Rate (gal/ft^2-d)	800	2,000
Max. Overflow Rate (gal/ft^2-d)	1,200	3,000
Scour Velocity (ft/s)	0.63	
Peak Horizontal Flow Velocity (ft/s)	0.005	
BOD/TSS Removal (%)	36.4/58.4	20.0/37.7

Table 8: Summary of design parameters for the primary clarifier.

	Proposed Design	Design Criteria
Depth (ft)	13	10 - 16
Diameter (ft)	18	10 - 200
Diameter : Depth (ft)	1.4 : 1.0	1 : 1 – 12.5 : 1.0
Surface Area (ft^2)	255	-
Volume (ft^3)	3,309	-
Wiper/Scraper Speed (rpm)	0.02	0.02 – 0.05

6.0 Biological Treatment

Conventional treatment of wastewater has long required that BOD and total suspended solids be reduced to a desirable level. Recently, the removal of nutrients such as nitrogen and phosphorus has become a concern. NPDES permits not only require acceptable levels of BOD and total suspended solids (TSS) but also require reductions in various forms of nitrogen and phosphorus. Advanced treatment or tertiary treatment options refer to treatment methods for the removal of nutrients beyond the capability of conventional biological processes. Both conventional and advanced treatment methods rely on biological processes that are mediated by a variety of microorganisms. Microorganisms effectively treat wastewater by accomplishing the following: oxidizing dissolved and particulate biodegradable organic matter to simple end products, capturing and incorporating suspended and nonsettleable colloidal solids into biomass, and transforming and/or removing nutrients (Metcalf & Eddy, 2014). Complete removal of organic matter incorporated into biomass is accomplished by settling and disposing of biomass.

NAS Fallon uses an activated sludge process known as a sequencing batch reactor (SBR). The SBR is a fill and drain type of system that incorporates all steps of the activated sludge process into a single reactor. NAS Fallon uses 2 SBRs that alternatively fill up at 3-hour intervals. The influent first enters the SBR and is allowed to fill to a certain depth. Next the reactor is allowed to react with the introduction of air. After the reactor theoretically achieves a desired removal, the reactor is then given time to settle, and the effluent is decanted off the top. This system was designed to achieve a standard BOD and TSS removal and advanced treatment was not initially planned. The existing SBRs have proven to do a fair job of removing nutrients on average, but existing data from December 2007 to April 2008 shows that NAS Fallon experienced 7 total-N concentrations that exceeded the threshold set by the NPDES permit (10 mg/L total-N). In order to improve wastewater treatment and ensure the NPDES permit discharge limit is met for nitrogen, construction of a new advanced treatment process is proposed.

6.1 Alternative Assessment

Several suspended-growth biological processes were analyzed and compared before a variation on the Bardenpho process was selected. The extent to which nitrogen is removed from the wastewater is dependent on the specific treatment method used. Thus, the capability of the treatment method to remove nitrogen and meet effluent discharge limits was an important aspect of design selection. Since it is desirable to reuse the treated effluent in the Fallon area, it was desired that the process selected be able to treat the wastewater to the standards listed in Table 9

Table 9: Desirable effluent wastewater characteristics.

	NH_4^+-N	TN-N	BOD	TSS
	(mg/L)	(mg/L)	(mg/L)	(mg/L)
Target effluent concentrations	≤2.5	≤7.5	≤15	≤15

Other factors such as operational problems, effect of reaction kinetics, process control, reliability, future treatment needs, ease of future renovations, space, and cost were also considered for the selection of the biological process. Four alternatives were assessed and compared against the chosen step feed removal process. The four alternatives include: oxidation ditch, 2-stage nitrification process, 3-stage denitrification process, and Ludzack-Ettinger process. Each alternative will be addressed separately with basic operation and various advantages and disadvantages mentioned.

Oxidation Ditch

An oxidation ditch is an activated sludge process that typically consists of a "racetrack" design that uses long channels with looped configuration. Oxidation ditches typically use long aeration periods in order to achieve BOD and nitrification. The oxidation ditch can be modified to perform nitrification and denitrification by installing aerobic and anoxic zones. Modifications include means of controlling mixing and aeration in order to control the dissolved oxygen concentration and homogeneity of substrates. This process typically has long SRTs and requires large tank volumes in order to ensure that sufficient capacity is available to accommodate nitrification and denitrification zones (Metcalf & Eddy, 2014). Treatment efficiency is largely a function of dissolved oxygen control and the length of the oxidation ditch channel. While the oxidation ditch channel is a reliable process with simple operation that is adaptable for nutrient removal, the large structure and greater space requirement is an important limiting factor. Also, the aeration energy required for an oxidation ditch is typically more than that required by a plug flow or staged reactor (Metcalf & Eddy, 2014).

2-Stage Nitrification Process

A 2-stage nitrification process is typically used to achieve ammonium removal. The process is a two-sludge system that utilizes a high-rate activated sludge process and the second stage is designed to enhance nitrification. The first stage is designed primarily to achieve BOD removal, while the second stage focuses on the transformation of ammonium to nitrate. A 2-stage nitrification process is a more efficient method than an oxidation ditch due to the separate systems for BOD and ammonium removal and the advanced control options in each stage. Separation of the stages also reduces the aeration energy demands. This process also has additional control features for return activated sludge and bypass options (Metcalf & Eddy, 2014). The 2-stage nitrification process typically requires three clarifiers: primary clarifier, secondary clarifier, and a nitrification clarifier. While the 2-stage nitrification process offers more advanced controls and less air requirements than the oxidation ditch, the three clarifiers and two basins require a large amount of space. The main disadvantage is the inability of the process to completely remove nitrogen. An additional denitrification basin would have to be constructed in order to improve nitrogen removal. The number of reactors and structures complicates the operation of this process.

3-Stage Denitrification Process

A 3-stage denitrification process is identical to the 2-stage nitrification process except removal of nitrate in a denitrification reactor is addressed. This process is a 3-sludge system in which the first two stages focuses on BOD and transformation of ammonium to nitrate. The third stage converts nitrate to nitrogen gas which effectively removes nitrogen from the system. The third stage is operated under anoxic conditions. Often the third stage requires the addition of an external carbon source for microbial growth. The 3-stage process requires even more control and maintenance than the 2-stage process.

Ludzack-Ettinger Process

The Ludzack-Ettinger process uses an anoxic zone and an aerobic zone to achieve nitrification and partial denitrification. The process relies heavily on the return activated sludge off of the secondary clarifier to the anoxic zone to achieve denitrification. The return activated sludge supplies the anoxic zone with nitrate to be used as an oxidant, while the influent raw wastewater supplies a source of organic matter. Modification to the process can be made to achieve greater denitrification rates. With modification, the average effluent concentration can range from 4-7 mg/L NO_3^--N (Metcalf & Eddy, 2014). With modification, the process can also achieve effluent standards less than 10 mg/L TN-N.

Step-Feed Nitrogen Removal Process

The step-feed nitrogen removal process takes the concept of preanoxic zones and replicates the stages in order to achieve more advanced nitrogen removal. The step-feed nitrogen removal process is composed of four passes with each pass comprised of an anoxic and aerobic zone. The influent flow is distributed to each pass. A typical splitting percent distribution to a 4-pass system is 15/35/30/20 (Metcalf & Eddy, 2014). Non-symmetrical designs of the anoxic and aerobic compartments and adjustments of flow splitting can lead to advanced treatment efficiency and greater treatment capacity (Metcalf & Eddy, 2014). In addition, the step-feed nitrogen removal process is a single sludge system. The return activated sludge is returned to the first pass. Optional external carbon sources can be added to the last two passes (anoxic basin) in order to improve treatment efficiency. This process is designed to achieve more nitrogen removal than a 2-stage or 3-stage process. Effluent concentrations of 2-4 mg/L TN-N are possible with this system. In addition, this process can be modified to achieve phosphorus removal with the addition of an anaerobic basin at the front end of the process.

The step-feed nitrogen removal process was selected to perform biological treatment. The biological removal process is a single-sludge system comprised of four passes where each pass is comprised of an anoxic and aerobic zone. The passes are arranged in series. A schematic of the selected design is shown in Figure 12. The mixed liquor is recycled from the bottom of the secondary clarifier back to the beginning of the anoxic section of the first pass. In summary, this process was chosen over the others because it is designed to achieve more total nitrogen removal than the alternative designs. This process was also favored, because it can be modified by installing an anaerobic reactor at the front of the treatment chain in order to achieve phosphorus removal. NPDES permit discharge limits may require enhanced phosphorus removal in the future, so the selected process is capable of addressing future treatment needs. Unlike other alternatives, the alternating anoxic and aerobic zones prevent operational problems such as bulking sludge by providing “selector” zones (Metcalf & Eddy, 2014). The staged process which includes several compartments in series also allows for more process control. Staged reactor designs also exploit reaction kinetic advantages that result in less volume than single reactors; thus the spatial site constraints are better addressed with staged reactors.

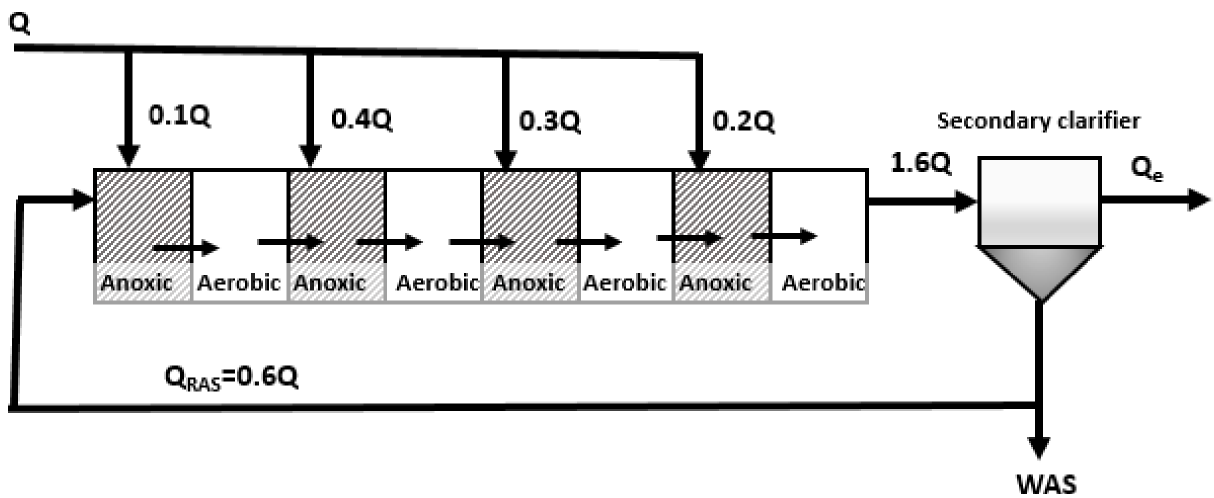


Figure 12: Schematic of step feed nitrogen removal process

6.2 Step Feed Biological Nutrient Removal Process

In order to design for a step-feed biological nutrient removal process (step-feed BNR), the following variables were initially considered: flow distribution between passes, relative volumes of anoxic and aerobic stages, and the final pass MLSS concentration (Metcalf & Eddy, 2014). The final pass MLSS concentration was determined by assuming an acceptable solids loading for the secondary clarifier. Selection of the influent flow distribution, RAS ratio, final pass MLSS concentration, and wastewater characteristics were used to find the solids retention time (SRT). Various wastewater characteristics were assumed while few were derived from existing data. The primary design assumptions are shown in Table 10; other design assumptions are shown in the calculations located in Appendix A. Table 11 shows design parameters derived from water quality data

Table 10: Design assumption used for process design calculations (MLSS is the assumed value for the last pass).

	Q (m^3/d)	1,161 (0.475 cfs)
	MLSS (mg/L)	3,000
	Q_{RAS}/Q	0.6
Flow Split	**Pass 1**	0.1
	Pass 2	0.4
	Pass 3	0.3
	Pass 4	0.2
	DO (mg/L)	2
	Fraction Anoxic Volume/ Pass	0.15
	Temperature (°C)	12

Table 11: Design parameters derived from water quality (data values are averages).

	Influent	**Effluent**
BOD (mg/L)	123	-
TSS (mg/L)	156	-
TDS (mg/L)	-	3074
TP (mg/L)	22.9	-
TKN (mg/L)	35.3	2.42
NO_2^-/NO_3^--N (mg/L)	0.822	1.88
TN-N (mg/L)	27.3	4.84

Determination of the SRT led to the calculation of mixed liquor components such as biomass and nitrifying bacteria concentration. After the biomass and nitrifying bacteria concentrations were calculated, the nitrification and denitrification capacity was calculated for the system. The oxygen, chemical addition, and mixing requirements for the anoxic/aerobic stages were estimated. The design involved multiple iterations on reactor volumes in order to optimize for treatment efficiency and SRT. All calculations were done at a ‘winter’ temperature of 12°C in which microbial kinetics are slower in order to design for the worst case scenario. Also, two identical step-feed BNR reactors will be designed to handle the average daily flow. An additional step-feed BNR reactor will be constructed for purposes of redundancy.

6.3 Design Parameters

The basin geometry of the anoxic and aerobic zones was made unsymmetrical in order to improve the treatment efficiency. By decreasing the size of the anoxic zone relative to the aerobic zone, the mixed-liquor (MLSS) concentration increases and improves the removal of nutrients. The total anoxic volume made up 15% of the total volume while the aerobic volume made up 85%. The step-feed BNR will be made up of 4 passes and each pass will include an anoxic and aerobic zone. Figure 13 shows the overall dimensions of the step-feed BNR process as well as the flow distributions across the system. The anoxic zone dimensions were designed to be as symmetrical as possible to enhance mixing efficiency. The width and height of the anoxic and aerobic zones were made to match in order to allow flow from one zone to the next. The flow distribution chosen (10-40-30-20) is a generally acceptable distribution (Metcalf & Eddy, 2014). No further modification of the flow distribution was performed because effluent concentrations met target design criteria.

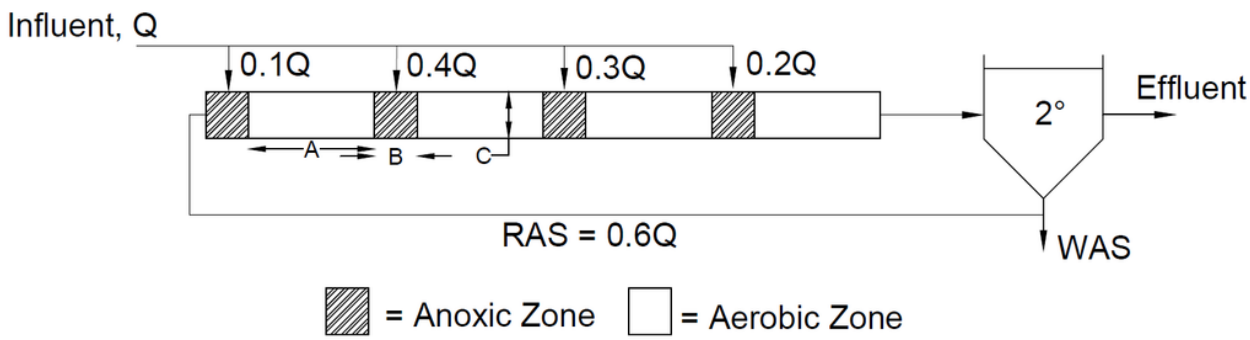


Figure 13: General schematic of the step feed BNR process with the RAS ratio and flow distribution to each anoxic basin. The schematic is not drawn to scale: (A)=36’ (B)=6’ (C)=7’.

Figure 14 shows a profile view of a single pass of the step feed BNR. The height of the reactor will be constructed with 2 feet of freeboard in order to prevent overflow and interference of wind velocities. The anoxic zone and aerobic zones will be separated by a diffuser wall in order to minimize dead zones and promote even flow across the reactor. The mixing and aeration equipment will be discussed in another section.

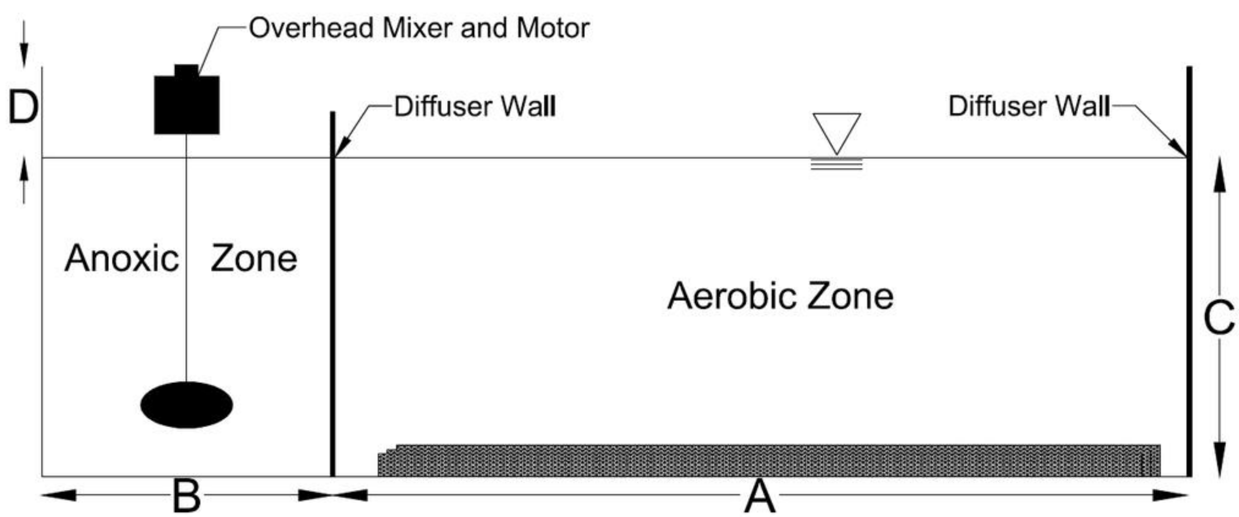


Figure 14: Profile view of a single pass of the step feed BNR process. The schematic is not drawn to scale: (A)=36’, (B)=6’, (C)=7’, (D)=2’.

The overall dimensions for the anoxic and aerobic zones for a single pass are summarized in Table 12.

Table 12: Step feed BNR dimensions for anoxic and aerobic zones.

	Anoxic	Aerobic
W/pass (ft)	7.00	7.00
H/pass (ft)	9.00	9.00
L/pass (ft)	6.38	36.4
Total L (ft)	170	

In order to determine the aerobic volume, several iterations were done comparing the effects DO and SRT have on the effluent ammonium concentration. The iterations on the aerobic volume per pass for a DO concentration of 2.0 mg/L O_2 is shown in Table 13. It was found that an aerobic volume for a single pass of 2,175 ft^3 provided excessive treatment with the ammonium concentration from the fourth pass at 1.6 mg/L NH_4^+-N. Decreasing the aerobic volume to 1,591 ft^3 led to the effluent ammonium concentration exceeding the target design concentration of 2.5 mg/L NH_4^+-N. A middle range aerobic volume of 1,771 ft^3 was chosen, and the effluent ammonium concentration was just below the target effluent concentration at 2.41 mg/L NH_4^+-N. The final SRT value for the aerobic zone was calculated to be 6.71 days. The overall SRT including the anoxic zone was estimated to be 7.89 days. The overall SRT was calculated using a ratio of the total volume to the aerobic volume multiplied by the aerobic SRT (Appendix A).

Table 13: Iterations on aerobic volume for a single pass in order to optimize for DO, SRT, and ammonium effluent.

Pass	Aerobic Volume/Pass	Aerobic SRT (days)	NH_4^+-N (mg/L)
1	2175	8.77	0.129
2	2175	8.77	1.85
3	2175	8.77	2.14
4	2175	8.77	1.6
1	1771	6.71	0.184
2	1771	6.71	2.36
3	1771	6.71	2.86
4	1771	6.71	2.41
1	1591	5.86	0.229
2	1591	5.86	2.66
3	1591	5.86	3.28
4	1591	5.86	2.9

After the biomass and nitrifying bacteria concentrations were calculated and the nitrification and denitrification capacity was calculated for the system, the final nitrogen effluent, BOD, and TSS were calculated. Table 14 shows the design effluent concentrations compared to the target effluent concentrations. The BOD was calculated using best and worst case scenarios for settling characteristics in the secondary clarifier. Assuming the secondary clarifier performed properly and achieved good settling sludge, the effluent TSS was assumed to fall in the range of 4 to 10 mg/L TSS (Metcalf & Eddy, 2014). The best case settling scenario (4 mg/L TSS) would result in an effluent BOD concentration of 5.73 mg/L BOD while the worst case scenario (10 mg/L TSS) results in a concentration of 9.83 mg/L BOD. The worst case scenario met the target design concentration of 15.0 mg/L BOD, so the design was considered adequate. All design concentrations met the target effluent concentrations. The total nitrogen concentration included contributions due to ammonium as N and nitrate as N. A nitrate balance was done on the anoxic reactors, and it was found that an effluent concentration of 0.560 mg/L NO_3^--N was expected from the fourth pass of the anoxic zone. Due to the very small nitrate concentration, the TN concentration fell far below the target effluent concentration. Summer conditions of greater than 25°C will result in even lower effluent concentration due to the enhanced effects on microbial kinetics.

Table 14: Comparison of design and target effluent concentration for the step feed BNR process.

	NH_4^+-N	TN-N	BOD	TSS
	(mg/L)	(mg/L)	(mg/L)	(mg/L)
Target effluent concentrations	≤2.5	≤7.5	≤15	≤15
Design effluent concentration	2.41	2.97	9.83	10

The corresponding mass balance for MLSS, MLVSS, nitrifiers, and biomass are shown in Table 15.

Table 15: Mass balance for each pass on the substrate concentrations and biomass.

Pass	**MLSS**	**MLVSS**	**Nitrifiers**	**Biomass**
	g/m^3	kg/d	g VSS/ m^3	g VSS/ m^3
1	6857	5497	117	3823
2	4364	3498	74.5	2433
3	3429	2748	58.5	1912
4	3000	2405	51.2	1673

6.4 Mixing Requirements

For the anoxic zones in the step feed BNR, mixing is required to homogenize the substrates and biomass. Without proper mixing, the efficiency of nutrient removal will decrease dramatically. A small diameter, submersible mixer was chosen because it could provide intensive mixing. Calculations showed that top entry agitators would not be needed, and a single submersible motor will be adequate for each anoxic zone (Appendix A). The mixers will be positioned according to Table 16 instructions. A single guide bar system will be installed to lower or raise the mixers. The mixers will need top and bottom anchorage to the wall and floor of the reactor with the appropriate brackets. Power required for the mixers were estimated using a minimum and maximum mixing energy. Table 17 depicts the power required by the mixers with an assumed electric motor efficiency of 85%. A standard motor size was selected for each scenario.

Table 16: Placement of mixers from center of propeller relative to the anoxic basin.

Distance from bottom to propeller tip (ft)	1.64
Distance from inlet (ft)	3.19
Distance from side (ft)	3.50

Table 17: Power requirements for anoxic mixers.

	Mixing Energy= 8 kW/ 10^3 m^3	**Mixing Energy= 3 kW/ 10^3 m^3**
Power required (kW)	0.283 (0.375 hp)	0.106 (.125 hp)
Power/pass (W)	70.8 (0.098 hp)	26.6 (0.0368 hp)
Standard Motor Size	0.17 hp	0.08 hp

6.5 Provisions for Inputs

The step feed BNR process requires various provisions; this includes air and alkali adding chemicals.

Air

Air will be needed for the aerobic zones within the step feed BNR process. A fine bubble diffuser ensemble will be installed. An air transfer efficiency of 35% was assumed. At the given atmospheric pressure (~4000 feet) and cold, winter temperature of 12°C, the air flowrate required is 105 cfs.

Chemicals

Due to the acidifying nature of nitrification, alkalinity amending chemicals such as calcium carbonate or sodium bicarbonate will need to be added to pass 4 of the step feed BNR. The influent wastewater was assumed to have an alkalinity of 140 g/m^3 as $CaCO_3$. In order to maintain an effluent pH of 6.8 to 7.0, 123 kg/d as $CaCO_3$ will need to be added. Since sodium bicarbonate is typically used due to the ease of handling and fewer scaling problems, the amount of sodium bicarbonate needed was calculated to be 206 kg/d as $NaHCO_3$.

6.6 Secondary Clarification

Liquid-solids separation is a crucial step in the activated sludge process and to the overall success in the treatment of wastewater. Secondary clarifiers have been designed and chosen to fulfill this need. In this process, the secondary clarifiers will act to remove solids via gravity, and provide concentrated activated sludge to recycle to the step feed biological nutrient removal processes (BNR). The design process for secondary clarification is anchored by the surface overflow rate (SOR) and solids loading rate (SLR) parameters. Optimizing these two parameters is essential for implementing an effective clarification process. The design outlined below results in a surface overflow rate of approximately 970 gal/ft^2*d and a solids loading rate of approximately 1.25 lb TSS/ft^2*hr; both of which are excellent values for healthy sedimentation. Each clarifier will have a diameter of 23 feet and a side water depth of 13 feet (depth of water near exterior wall). The floor of each clarifier will be sloped towards the center to enhance sludge collection and removal. Influent water will be fed through a center well, fitted with diffuser ports to promote even, distributed flow and enhance particle settling. There will be rotating rakes along the floor of the clarifiers to facilitate sludge removal through a buried pipeline. Collected sludge will then be moved to either the solids handling facility for dewatering or recycled to the step feed BNR. An effluent weir and collection trough will encompass the sedimentation zone. The collection trough will have a width of one foot and will be sloped to direct water to a singular effluent exit. Each clarifier will be fitted with a walkway above the water surface. The walkway will hold the motor required to rotate the feed well and sludge rakes. Tables 18 and 19 hold a summary of important design criteria regarding each clarifier. Figures 15 and 16 show plan and profiles views of the physical design.

Table 18: Summary of calculated design parameters for each secondary clarifier.

	$Q_{Average}$	Q_{Peak}	Typical
SOR (gal/ft^2*d)	740	970	600 - 1600
SLR (lb TSS/ft^2*hr)	1.25	1.64	1.00 - 2.00
HRT (hr)	3.18	2.43	2.50 - 4.50
Weir Loading Rate (gal/ft*d)	4,251.43	5,573.30	10,000 max

Table 19: Summary of physical design parameters for each secondary clarifier.

	Design	Typical
Diameter (ft)	23	40 - 150
Depth (ft)	13	10-16
Bottom Slope (ft/ft)	1/10	1/6 – 1/16
Trough Width (ft)	1	-

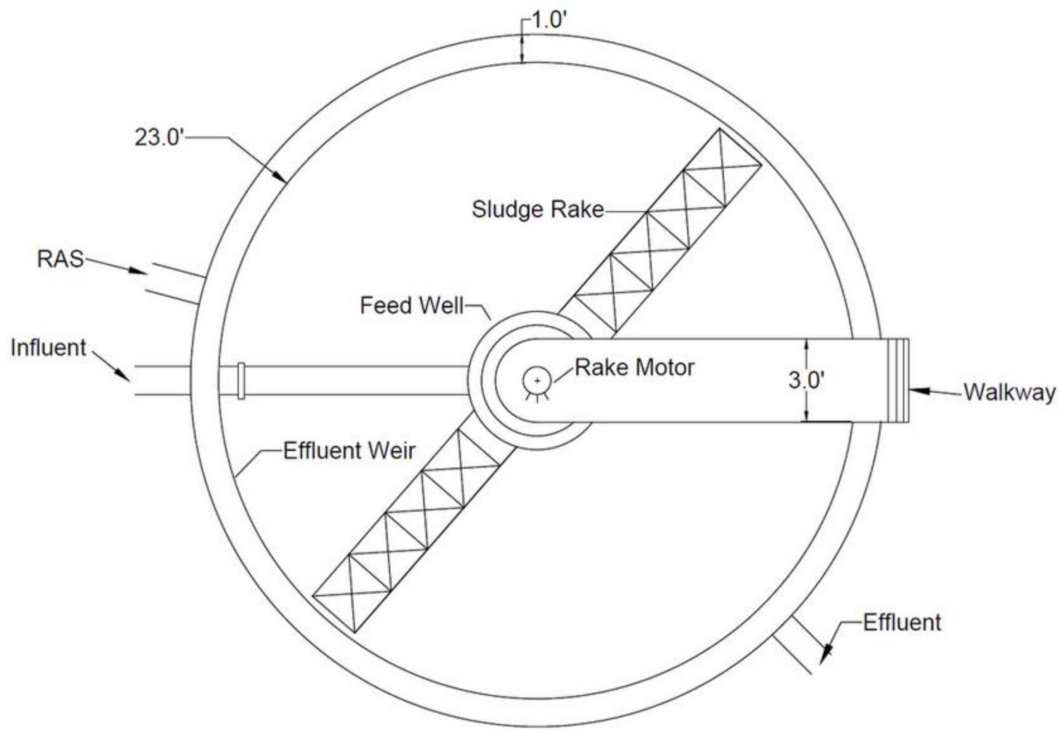


Figure 15: Plan view of secondary clarifier.

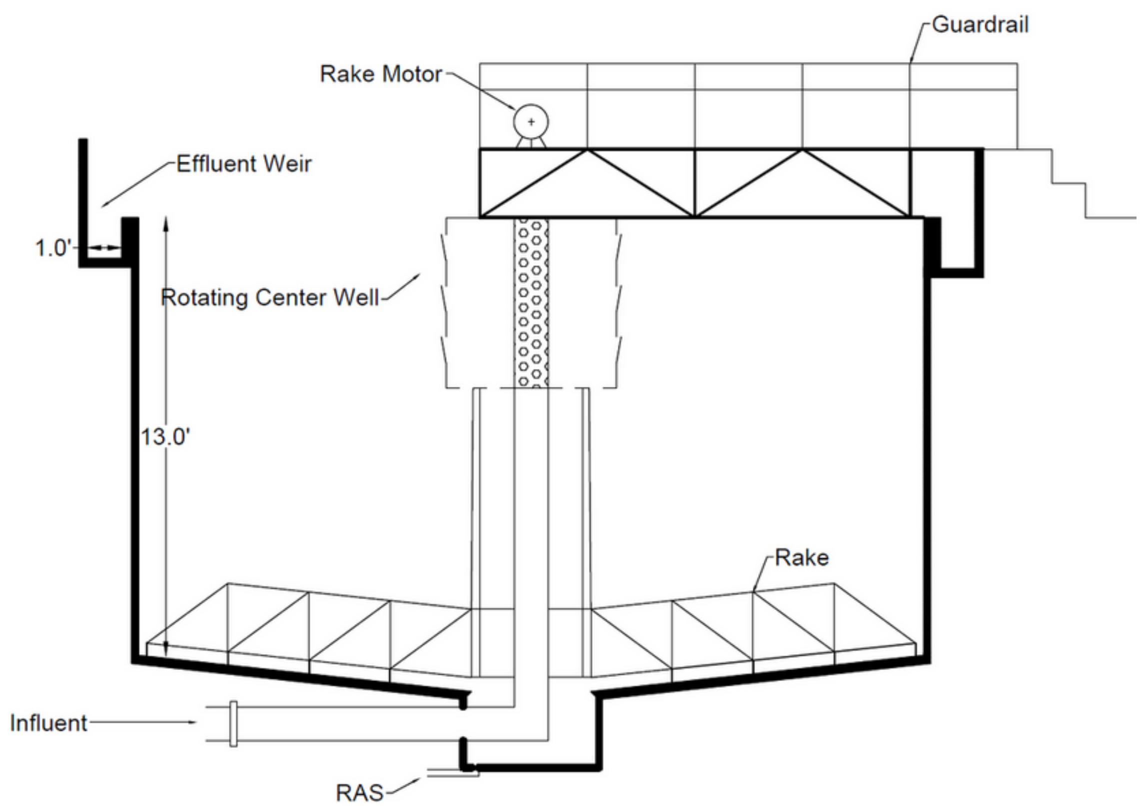


Figure 16: Profile view of secondary clarifier.

Upon construction and activation of each secondary clarifier, certain actions should be taken to monitor, assess and improve process performance. A sludge thickening assessment should be performed on a regular basis. This assessment general requires the use a ‘sludge judge’ to measure the sludge blanket depth. In addition, a Sludge Volume Index (SVI) test may be performed to assess particle settling performance. Solids Flux Analysis and State Point Analysis are good testing procedures as well. In addition to sludge monitoring, operators and engineers should monitor the surface of the clarifiers for excess accumulation of floating scum.

7.0 Disinfection

The disinfection process for wastewater and reuse applications is important in order to ensure the water is safe for dispersal into the environment. The presence of waterborne microorganisms in water can cause various diseases. Most importantly the presence of bacteria, viruses, and protozoa are of concern. Disinfection achieves inactivation or destruction of these organisms. Disinfection of wastewater can be accomplished several ways; including, chemical agents, non-ionizing radiation, ionizing radiation, and through mechanical means (Metcalf & Eddy, 2014).

Currently NAS Fallon WWTP uses a single 100,000 gallon steel tank as a chlorine contact basin. The basin does not contain any amendments to improve the mixing efficiency of the disinfectant. NAS Fallon is currently in need of a new disinfection process that is designed for efficient chemical dosage, contact time, mixing, and flexibility. In the next section, a few alternatives for disinfection will be compared.

7.1 Alternative Assessment

The disinfectants commonly employed in wastewater are chemical agents such as chlorine and ozone or non-ionizing radiation such as UV. Several factors are considered when choosing a disinfectant. These factors include: toxicity to microorganism, availability, safety, solubility, stability, and alteration of solution characteristics (Metcalf & Eddy, 2014). These factors and associated issues with various disinfectants will be discussed in order to determine the ideal disinfectant for the NAS Fallon WWTP.

Chlorine

Chlorine and related compounds such as sodium hypochlorite (NaOCl), calcium hypochlorite ($Ca(OCl)_2$), combined chlorine, and chlorine dioxide are commonly used as disinfectants for wastewater treatment. The cost is relatively low compared to other alternatives while efficiency of disinfection is highly dependent on basin design, mixing efficiency, and contact time. While all chlorine disinfectants are highly corrosive and toxic, the degree of stability and safety of concern varies for each. While chlorine gas is stable, the compound needs a high degree of containment and additional safety measures have to be considered. Sodium hypochlorite and calcium hypochlorite are slightly unstable, but the level of containment and safety concerns are significantly less. One of the main concerns with using chlorine as a disinfectant is the formation of byproducts such as trihalomethanes (THMs) and haloacetic acids (HAAs) that are known to be toxic. Overall, chlorine as a disinfectant is commonly used and reliable if the basin is properly designed and chlorine dosage, chlorine residual, and the coliform removal are monitored (Metcalf & Eddy, 2014).

Ozone

Ozone is a chemical disinfectant that introduces ozone (O_3) gas into the water. This disinfectant is highly dependent on ozone transfer efficiency into the water. Efficiency can be improved by using fine bubble diffusers and minimizing dead zones in the contact chamber by installing baffles. The use of ozone is increasingly steadily, but the cost is moderately high. Ozone, like chlorine, is highly corrosive, toxic, and unstable. Ozone typically has to be generated on site due to its instability; this requirement increases the costs. Unlike chlorine, ozone does not create THMs and HAAs, but the process can produce bromate. Ozone is an effective disinfectant for bacteria and viruses, and unlike chlorine it is a good disinfectant for protozoa. Ozone also does not increase the total dissolved solids (TDS) of the water (Metcalf & Eddy, 2014).

UV

The use of ultraviolet (UV) radiation photochemically damages the RNA and DNA of problematic organisms and inactivates the ability of the organisms to reproduce. UV disinfection uses open or closed contact chambers. These two reactors differ in UV lamps. The contact time (seconds) is shorter than the contact time typically required by chlorine and ozone. The design of the contact chambers is crucial for proper disinfection. Similar to ozone, the costs of UV disinfection are high. Unlike chlorine and ozone, UV is not corrosive and the safety of concern is low. The use of UV as a disinfectant is increasing rapidly, and one of the major benefits of using UV is no byproduct formation and no increase in TDS (Metcalf & Eddy, 2014).

Table 20 summarizes the comparisons of the disinfectant processes using chlorine, ozone, and UV.

Table 20: Comparison of various disinfection technologies (Metcalf & Eddy, 2014).

Characteristics	Chlorine gas	Sodium hypochlorite	Ozone	UV
Availability/cost	Low	Moderately Low	Moderately high	Moderately high
Interaction with OM	Oxidizes OM	Oxidizes OM	Oxidizes OM	Absorbance of UV
Corrosiveness	Highly corrosive	Corrosive	Highly Corrosive	na
Toxic to higher life forms	Highly toxic	Highly toxic	Toxic	Toxic
Safety concern	High	Moderate to low	Moderate	Low
Solubility	Moderate	High	Moderate	na
Stability	Stable	Slightly Unstable	Unstable	na
Effectiveness for:				
Bacteria	Excellent	Excellent	Excellent	Good
Protozoa	Fair to poor	Fair to poor	Good	Excellent
Viruses	Excellent	Excellent	Excellent	Good
Byproduct formation	THMS and HAAs	THMS and HAAs	Bromate	No
Increases TDS	Yes	Yes	No	No
Use as a disinfectant	Common	Common	Increasing slowly	Increasing rapidly

Of the alternatives discusses, chlorine in the form of liquid sodium hypochlorite (NaOCl) was selected because of the low cost, effective disinfectant properties, availability, and ease of use. The corrosive, toxic, and unstable nature of sodium hypochlorite will be addressed by designing for proper containment and ensuring proper storage and handling.

7.2 Estimated Coliform Removal Upstream

Upstream treatment processes influence disinfection due to incidental removal of problematic microorganisms during the removal of NOM, organic matter, and suspended. Biological processes can also influence the removal of microorganisms (Metcalf & Eddy, 2014). The range of estimated percent removal of various upstream processes is summarized in Table 21.

Table 21: Removal of total coliform by upstream treatment processes (Metcalf & Eddy, 2014).

	Percent Removal		
Process	**Low**	**Average**	**High**
Coarse screens	0	2.5	5
Fine screens	10	15	20
Grit chambers	10	17.5	25
Total Primary	20	35	50
Biological process	90	94	98
Total Secondary	90	94	98

In order to determine the Ct values for reactor design, the initial and final concentration of total coliform has to be known or assumed. Unfortunately, data for initial total coliform concentrations at the NAS Fallon WWTP were not available. Instead, a high range total coliform concentration was estimated for the influent. A value of 1.00×10^9 MPN TC/100 mL was assumed (Metcalf & Eddy, 2014). The current design expects a high degree of percent removal for primary and biological processes. Thus it was estimated that 50% and 98% removal of total coliform is to be expected for primary processes and biological processes. For various effluent standards, removal of total coliform (N/N_0) was calculated for effluent at different stages of the waste water treatment process (Metcalf & Eddy, 2014). Table 22 summarizes this data. The target total coliform (TC) count represents various degrees of treatment. The 200 MPN TC/ 100 mL effluent standard is the allotted Nevada Division of Environmental Protection (NDEP) permit value. However, the 2.2 MPN TC/ 100 mL value represents a higher degree of treatment and allows water to be reused. Both values were analyzed and considered in the design in order to allow plant operators to make adjustments for chlorine dosage in the future if the wastewater effluent is to be reused on the NAS base.

Table 22: Fraction removal of total coliform for various stages of the waste water treatment process assuming different effluent standards.

Target TC count	**Raw Wastewater**	**Primary Effluent**	**Secondary Effluent**
(MPN/100 mL)	**N/N_0**	**N/N_0**	**N/N_0**
1,000	1.00E-06	2.00E-06	5.00E-03
200	2.00E-07	4.00E-07	1.00E-03
23	2.30E-08	4.60E-08	1.15E-04
2.2	2.20E-09	4.40E-09	1.10E-05

7.3 Chlorine Dosage

In order to determine sodium hypochlorite dosage, appropriate Ct values have to be developed and the influence of contact time, chlorine residual, and the breakpoint chlorination point have to be considered (Metcalf & Eddy, 2014). Using the fraction of total coliform (N/N_0), Ct values using the Collins-Selleck model were calculated. The Collins-Selleck model corrects for shouldering and tailing effects associated with no reduction in the number of organisms despite chlorine addition and the shielding of microorganisms. The Collins-Selleck model assumes a pH of 7.5 and a water temperature of 20˚C. Next, the initial chlorine demand to the presence of nitrogen containing compounds were analyzed by calculating the breakpoint chlorination. Chlorine not only reacts with problematic microorganisms, but it also oxidizes various inorganic substances and ammonium. The breakpoint chlorination value due to ammonium was determined to be 9.1 mg/L Cl (Appendix A). A safety factor of 1.5 was used to account for initial chlorine demand due to inorganics and chlorine decay. Lastly, the chlorine dosage was calculated for several flow and contact time scenarios. Table 23 summarizes the required chlorine dosage for a standard effluent value of 200 MPN/ 100 mL. Table 24 summarizes the required chlorine dosage for a standard effluent value of 2.2 MPN/ 100 mL. The chlorine demand includes the 1.5 safety factor.

Table 23: Total required chlorine dosage for secondary effluent at different contact times for an effluent standard of 200 MPN/ 100 mL effluent standard.

	t=90 min.	**t= 30 min.**	**t= 15 min.**
	NaOCl-Cl	**NaOCl-Cl**	**NaOCl-Cl**
	(kg/d)	(kg/d)	(kg/d)
PHF	66.6	72.7	81.8
ADF	16.7	18.2	20.4

Table 24: Total required chlorine dosage for secondary effluent at different contact times for an effluent standard of 2.2 MPN/ 100 mL.

	t=90 min.	**t= 30 min.**	**t= 15 min.**
	NaOCl-Cl	**NaOCl-Cl**	**NaOCl-Cl**
	(kg/d)	(kg/d)	(kg/d)
PHF	78.4	108	152
ADF	19.6	27.0	38.1

To design the disinfection contact basin, the Ct values for PHF at an effluent standard of 200 MPN TC/ 100 mL at a 90 minute contact time was chosen. This design is suitable for the worst-case-scenario. The following parameters for the chosen design are summarized in Table 25. In addition, values for the 2.2 MPN TC/ 100 mL scenario are shown in Table 25 to illustrate the change in dosage and chlorine residual for higher treatment. The chlorine dosage is within the typically expected range based on a 90 minute contact time (Metcalf & Eddy, 2014).

Table 25: Chlorine load and dosage for PHF and ADF for 90 minute contact period for different effluent standards.

200 MPN/100 mL	**Q**	**NaOCl-Cl**	**Cl Dose**	**C**
	(m^3/d)	(kg/d)	(mg/L)	(mg/L)
PHF	4642	66.6	9.57	0.435
ADF	1161	16.7	9.57	-
2.2 MPN/ 100 mL				
PHF	4642	78.4	11.3	2.12
ADF	1161	19.6	11.3	-

Alkalinity Requirements

The addition of chlorine to water generates acids when chlorite (HOCl) reacts with ammonium present in effluent from the secondary clarifier. The amount of acid generated and the required addition of alkalinity to buffer the pH drop was calculated. The amount of alkalinity represented as calcium carbonate ($CaCO_3$) at different flow regimes is summarized in Table 26. Maintaining a proper pH and alkalinity in solution is crucial for disinfection. Since chlorite (HOCl) is a much more efficient disinfectant than its conjugate base hypochlorite (OCl-) the pH should be maintained above the pKa value of 7.52.

Table 26: Alkalinity required for different flow regimes.

	Q	**ALK required**
	(m^3/d)	(kg/d $CaCO_3$)
PHF	4642	159.7
ADF	1161	39.92

7.4 Basin Design

Chlorination capacities for disinfection were chosen to meet the design criteria of 200 MPN TC/ 100 mL as required by the NPDES permit. The chosen design will be able to meet the 2.2 MPN TC/ 100 mL treatment standard by adjusting the dose of chlorine supplied to the basin. The chlorine contact basin will be a serpentine basin that functions as a long plug-flow reactor. A serpentine basin was chosen to conserve space. Additional amendments such as baffle walls were included in order to ensure proper mixing and eliminate dead zones. The main objective addressed when designing the chlorine contact basin was to assure that the flow remained in the contact basin for the chosen detention time of 90 minutes. Figure 17 shows the configurations and dimensions for the designed chlorine contact basin.

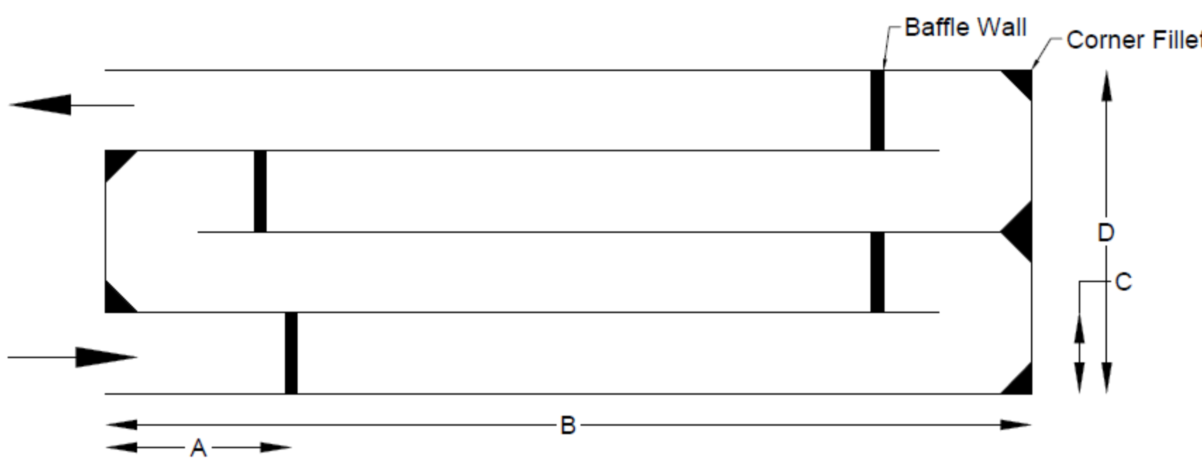


Figure 17: Chlorine contact basin plan view (A)=6', (B)=46.5', (C)= 1.31', (D)= 5.25'.

Two basins will be constructed in parallel for redundancy requirements. Each basin will be able to handle half the PHF. A single basin will be in operation during ADF events, but in the event that PHF occurs, the second basin will be put online. The design adhered to typical design criteria such as a L:W ratio between 20:1 and 40:1 and acceptable dispersion numbers. In addition, the dispersion number for the design was calculated and the efficiency of the chlorine contact basins was analyzed at different flow conditions. Table 27 shows the chosen design dimensions compared with typical design criteria.

Table 27: Proposed design parameters for chlorine contact basin compared to design criteria (Metcalf & Eddy, 2014).

	Design Criteria	**Proposed Design**
W (ft)	-	5.25
W/pass (ft)	-	1.31
D (ft)	-	5.35
freeboard (ft)	-	2.00
Basin L (ft)	-	46.5
Path L (m)	-	186
L:W	20:1-40:1	35.4:1
# passes	4-5	4
t (min.)	-	90

The dispersion number for a PHF and ADF were both less than the typically accepted 0.015 signifying the approach of ideal plug flow (Appendix A). Also, the expected velocities under PHF and ADF were analyzed to check for solids deposition. The velocities were both lower than typically acceptable velocity of 6.5 ft/min to 15 ft/min to avoid solids deposition (Metcalf & Eddy, 2014). The basins will have to be shut down and cleaned out for maintenance when this occurs. Table 28 compares the velocities of the proposed design against design criteria.

Table 28: Velocities in chlorine contact basin for different flow regimes and typically accepted design criteria to limit solids deposition (Metcalf & Eddy, 2014).

	Design Criteria	Proposed Design
vPHF (fpm)	6.5-15	2.07
vADF (fpm)	6.5-15	0.516

Baffles will be installed at the beginning of every pass. A total of four baffles will be needed. These will be located 6 feet from the entrance of the channel pass, and 3 feet from the beginning of each subsequent pass (Figure 17). The chlorine contact basin will also be equipped with corner fillets in order to minimize dead zones.

7.5 Injection and Initial Mixing

Efficient mixing of the disinfectant is essential for proper disinfection. Liquid sodium hypochlorite will be added into the beginning of the chlorine contact basin by a hangling nozzle type chlorine diffuser typically used for open channels (Metcalf & Eddy). A PVC pipe will span the width of the chlorine contact basin positioned 0.5 feet from the entrance. Flexible hoses, hose clamps, and diffuser nozzles will run along the PVC pipe, perpendicular to water flow. The diffuser nozzles will be submerged in the water and will reside 1.5 feet from the floor of the basin to ensure submergence for all flow regimes. The PVC pipe will be fed 15% sodium hypochlorite solution by a variable pump. The rate of the pump will be controlled by analysis of chlorine residual and flow measuring devices such as a v-notched weir placed at the outlet. Dosing will vary depending on the flow; Table 25 illustrates the expected range of dosing needed for PHF and ADF.

7.6 Outlet Control

The flow at the end of the basin will be metered by means of a v-notch weir. The flow control device will correlate with the control devices for sodium hypochlorite dosing. In addition, samples will be taken at the end of the last pass of the chlorine contact basin. The samples will be analyzed for chlorine residual and total coliform count in order to gage the performance of the basin. The dosage can also be manually changed in order to meet compliance with the NPDES permit.

After the effluent leaves the chlorine contact basins, the water will enter into a steel pipe and will be conveyed to the lower diagonal drain system to a free outfall.

7.7 Chemical Storage and Handling

Several safety devices and precautions must be taken when storing and handling chlorine. Though the procedures vary depending on the type of chlorine related compounded being used, chlorine in general is corrosive and toxic. The 15% liquid sodium hypochlorite solution will be stored in a separate, enclosed, temperature controlled room. The building will be accessible by road in order to ensure efficient delivery of chemical supplies. Chlorine solution for a two month period (assuming PHF) will be stored on site in six, 800-gallon drums. Delivery by truck can ensure up to 4,800 gallons of liquid sodium hypochlorite at any one time. Backman's Inc. will supply bulk 15% sodium hypochlorite; the specification sheet is located in Appendix B. The volume of disinfectant needed and delivery schedule will be adjusted depending on typical usage. The disinfectant will be handed according to MSDS specification. All liquid containing chlorine will be handled in Schedule 80 PVC piping due to the chemicals highly corrosive nature (Metcalf & Eddy, 2014).

The manufacturer, Grundfos, will supply all dosing pumps and measurement and control devices (Appendix B). Digital diaphragm dosing pumps equipped with a variable-speed stepper motor will be used for chlorine dosage (Appendix B). Model D250 was selected to provide the maximum dosage rate of 4 gallons/hour (Appendix A and Appendix B). Table 29 summarizes the expected sodium hypochlorite pumping rates.

Table 29: The expected disinfectant pumping rates for various flow regimes and effluent standards.

For 200 MPN/ 100 mL

	Q	**NaOCl-Cl**
	(m^3/d)	(gal/hr)
PHF	4642	3.26
ADF	1161	0.815

For 2.2 MPN/ 100 mL

	Q	**NaOCl-Cl**
	(m^3/d)	(gal/hr)
PHF	4642	3.84
ADF	1161	0.959

In addition, the Aquacell and single rod probe will be installed at the outlet of the basin in order to obtain chlorine and pH/redox values (Appendix B). In conjunction with the sensors, the DIA-2 system for controls will be installed in order to vary dosage depending on the chlorine residual (Appendix B).

8.0 Solids Handling

It was determined that approximately 10 m^3/s (353 cfs) should leave the secondary clarifier as waste activated sludge (WAS). This WAS is estimated to have solids concentration between 0.6% and 1.2%. The WAS will then enter a dewatering process to increase removal efficiency and limit disposal costs. Dewatering will be carried out by continuous belt filter press. A belt-filter press works on two main mechanisms; gravity and pressure. First water is passed through a 'gravity drainage' zone where free standing water is allowed to drain from the WAS, reducing the total volume. Then the WAS is passed through a series of overhead rollers which squeeze water content out of the solids. Ultimately a dewatered sludge cake is produced which was easily and cost-effectively disposed of. The belt-filter press will operate for five hours each day with a sludge flow rate of $2m^3/hr$. WAS storage will be provided to collect daily WAS flows. The five hour operating time will completely drain the day's stored WAS. A $20m^3$ (5000 gal) storage container will be provided to accommodate for times of high WAS production or belt-filter press down time. A water soluble polymer will be applied prior to entering the filter press to promote solid coagulation and enhance dewatering. The design calls for approximately 4300 g of polymer per five hour cycle, however results should be monitored to assess a correct dosage. At the estimated WAS solids concentration, there will be approximately $23m^3$ of dry cake solids produced per week. A weekly or bi-weekly (twice per week) disposal schedule should be arranged to take wasted solids to the Lockwood landfill. Recommended manufactures for storage containment and belt-filter press modules are located in Appendix B.

9.0 Hydraulic Profile

In order to ensure that the wastewater will travel through the proposed treatment facility by gravity flow a water surface profile was created (Figure 18). The wet well and dry well system will provide enough additional head at the front end of the treatment facility that wastewater will flow through the plant due to gravity (Appendix A).

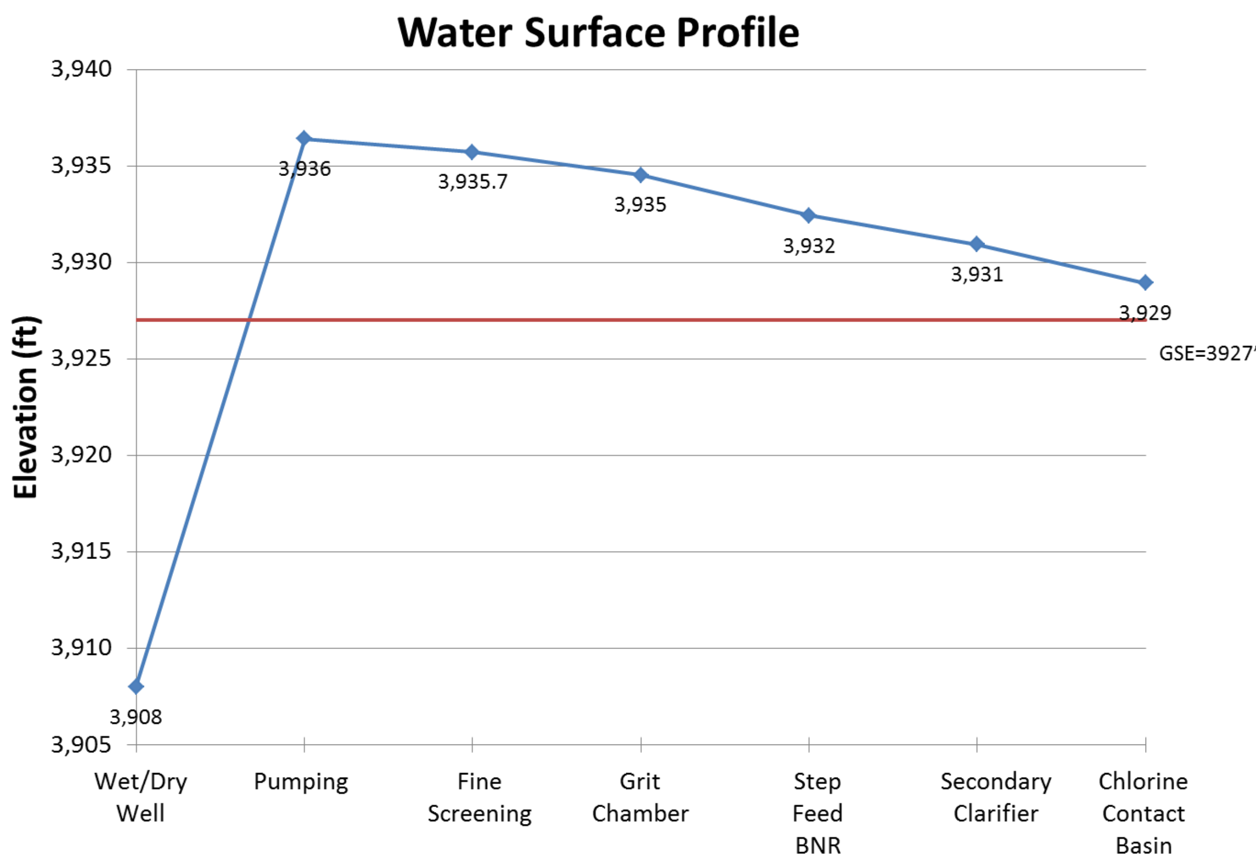


Figure 18: Water surface profile of proposed NAS Fallon WWTP.

10.0 Cost Analysis

A complete cost analysis has been performed for the proposed design. Capital costs, operation and maintenance costs (O&M), and power costs (where applicable) were considered for each suggested treatment process. Costs were estimated using an ENR index value of approximately 2475 from the year 1980. Values were adjusted using a May, 2014 ENR Index value of 9796. Table 30 shows a general overview of associated costs (power costs included in O&M). Appendix A includes a complete list of costs per each proposed process.

Table 30. Summary of associated cost estimates for proposed treatment facility

	Capital Cost ($)	O&M Cost ($/yr)
Pipes	17,550	-
Coarse/Fine Screening	156,000	23,400
BNR	4,632,619	191,841
Secondary Sedimentation	2,340,000	81,584
Belt Filter Press	378,300	7,410
Solids Disposal	-	624,000
Disinfection	2,340,000	780,000
TOTAL	**10,011,120**	**1,773,210**

11.0 Plant Schematic

The treatment process outlined in this document has been designed to provide complete treatment of wastewater flow from Fallon Air Force Base. The existing facility processes, including the headworks, grit separators, sequencing batch reactors and sludge lagoons, will be phased out as the new process train comes online for operation. The new process train will include a dry and wet well headworks system, fine screening, aerated grit chambers, step feed BNR, secondary sedimentation, and disinfection. The new facility will have an onsite solids handling facility to dewater and export collected sludges. Figures 19 and 20 show the proposed layout for the new treatment facility.

Figure 19: Overall plant schematic with existing NAS Fallon WWTP.

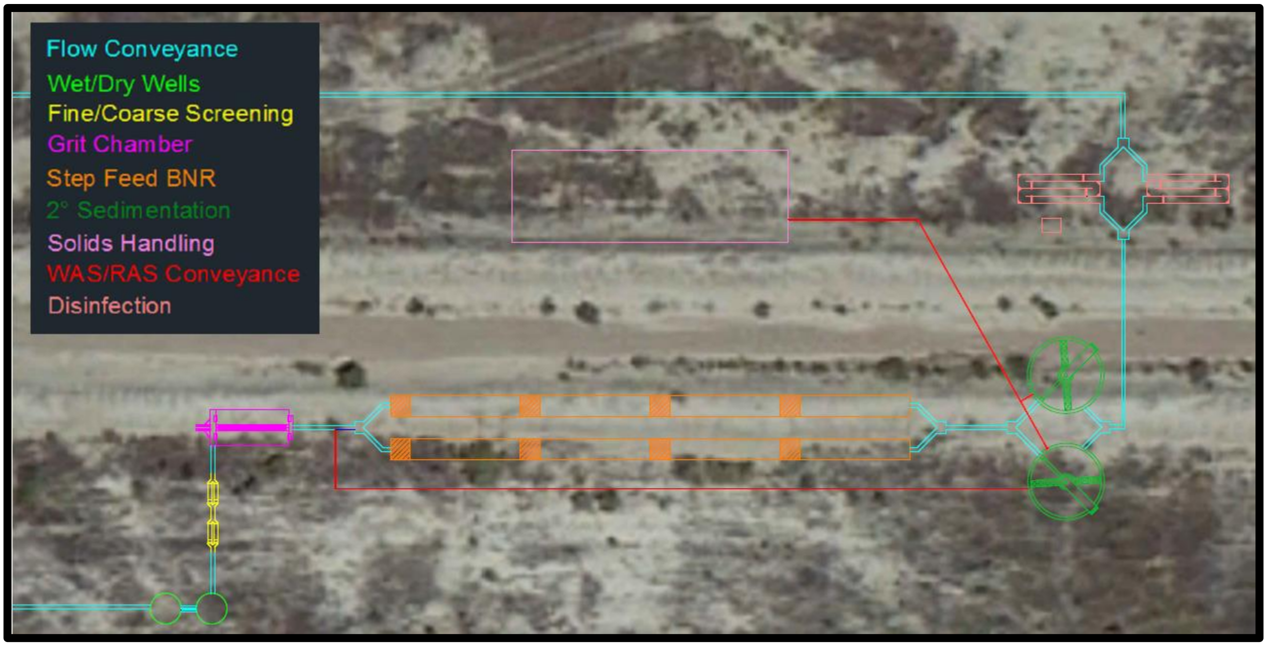


Figure 20: Proposed WWTP and plant layout of primary, secondary, and tertiary treatment.

References

Metcalf & Eddy AECOM, Wastewater Engineering Treatment and Resource Recovery, McGraw-Hill Education, New York. 2014.

Mackenzie L. Davis, Water and Wastewater Engineering: Design Principles and Practice, McGraw-Hill Education, New York. 2010

Appendix A

WASTEWATER INFLUENT

	DESIGN
NAS FALLON	
INFLUENT FLOW, ADF, MGD.	0.307
INFLUENT FLOW, MAX ADF, M	0.668
INFLUENT FLOW, PHF, MGD.	1.23

EXISITING NAS FALLON WWTP DESIGN

DESIGN FLOW	
PLANT CAPACITY, MGD	0.75

NEW NAS FALLON WASTEWATER TREATMENT PLANT

DESIGN FLOW	
PLANT CAPACITY, MGD	1
WET/DRY WELL INTAKE STRUCTURE	
CAPACITY, GAL	12,000
PUMP HEAD, FT	28.4
COARSE BAR SCREEN	
DIMENSIONS, LxWxD, FT	5x0.88x3
NUMBER OF BARS	5
BAR WIDTH, IN	0.6
FINE STEP SCREEN	
NUMBER	2
CHANNEL DIM., LxWxD, FT	7x2.5x3
SLOPE, %	0.5
SCREEN OPENING, IN.	0.35
AERATED GRIT CHAMBER	
NUMBER	2
DIMENSIONS, LxWxD, FT	25x8x6
DETENTION TIME, MIN	5.06
AIR SUPPLY, FT^3/FT*MIN	13.2
BIOLOGICAL NUTRIENT REMOVAL	
TYPE	STEP FEED
NUMBER	2
VOLUME, FT^3	10,700
AEROBIC SRT/PASS, DAYS	6.71
OVERALL SRT/PASS, DAYS	7.89
EFF. TOTAL NITROGEN, MG/L	2.97
EFFLUENT BOD, MG/L	9.83
EFFLUENT TSS, MG/L	10
MIXING MOTOR, HP	0.17
CALCIUM CARBONATE, KG/DA	123

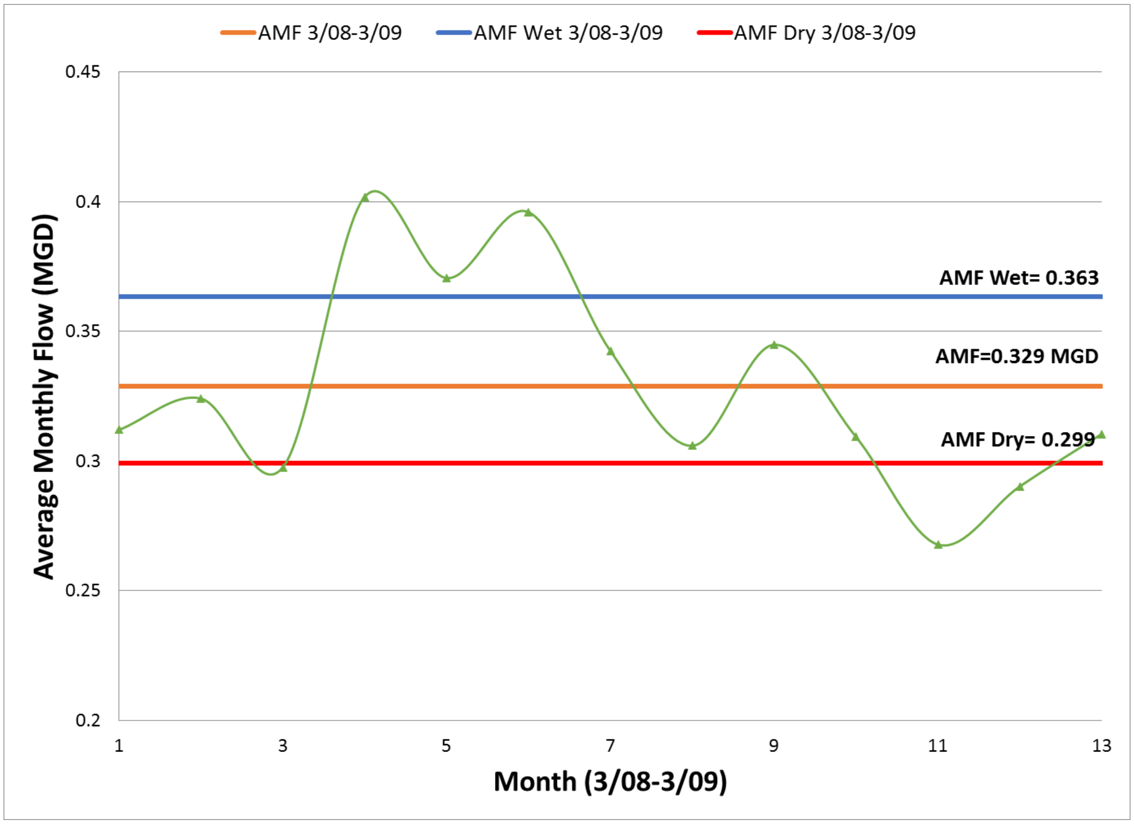
SECONDARY SEDIMENTATION	
TYPE	CIRCULAR
NUMBER	2
VOLUME, FT^3	5,400
FLOW, MAX, MGD	0.75
RETENTION TIME, HR	3.18
OVERFLOW RATE, GPD/FT^2	740
WEIR LOADING RATE, GPD/FT	4,252
SLR, LB TSS/HR*FT^2	1.25
RETURN ACTIVATED SLUDGE, CI	0.3
SOLIDS HANDLING AND DISPOSAL	
TYPE	BELT-FILTER PRESS
NUMBER	1
WASTE FLOW, M^3/D	10
SOLIDS CONC., %	0.7
OPERATION CYCLE, HR	5
OPERATION LOADING, M^3/D	2
CAKE SOLIDS PRODUCED, M^3/WK	23
DISPOSAL, PER WEEK	2
DISINFECTION	
TYPE	PLUG FLOW
METHOD	CHLORINE
NUMBER	2
DIMENSIONS, LxWxD, FT	186x1.31x5.25
HRT, MIN.	90

University of Nevada, Reno

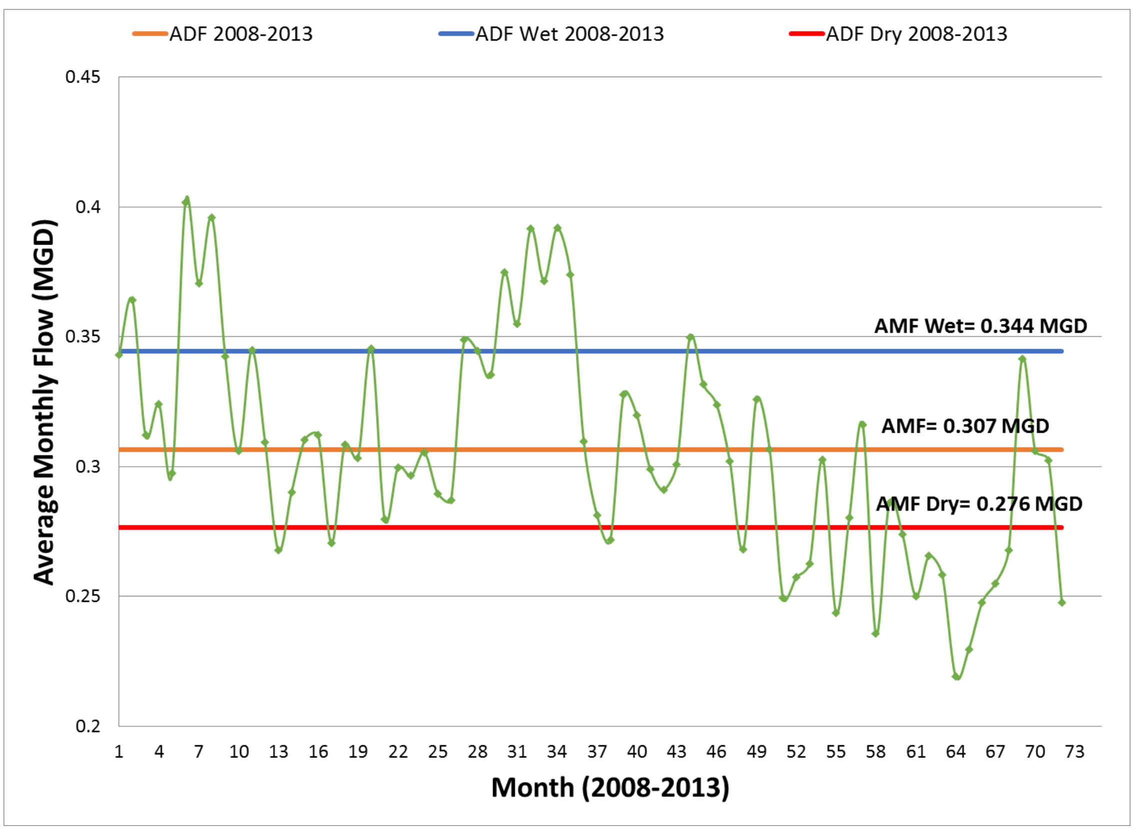
NAS FALLON NEVADA BIGGEST LITTLE AIR STATION IN THE WORLD

T&K Designs

DSGN	K.KIMBROUGH	CEE 457	GENERAL
DSGN	T. STROUD	UNIVERSITY OF NEVADA, RENO	
APVD		WASTERWATER TREATMENT PLANT	DESIGN DATA SUMMARY
DATE	5 MAY. 2014	ORIGINAL DESIGN	



AMF 3/08-3/09
AMF Wet 3/08-3/09
AMF Dry 3/08-3/09
0.45
0.4
0.35
0.3
0.25
0.2
Average Monthly Flow (MGD)
AMF Wet= 0.363
AMF=0.329 MGD
AMF Dry= 0.299
1
3
5
7
9
11
13
Month (3/08-3/09)



ADF 2008-2013
ADF Wet 2008-2013
ADF Dry 2008-2013
0.45
0.4
0.35
0.3
0.25
0.2
Average Monthly Flow (MGD)
AMF Wet= 0.344 MGD
AMF= 0.307 MGD
AMF Dry= 0.276 MGD
1 4 7 10 13 16 19 22 25 28 31 34 37 40 43 46 49 52 55 58 61 64 67 70 73
Month (2008-2013)

Dry Well/ Wet Well

Water	7	ft			
Well	15	ft		Drainage Volume	Drainage Time
Total Depth	22	ft		(gallons)	(min.)
				7637	12.7
Diameter	10	ft			
Total Volume	1728	ft3			
	12924	gallons	(TOTAL CAPACITY)		
Drainage Volume	1021	ft3			
	7637	gallons	(CAPACITY WET WELL)		
Flows					
PHF	852	gpm			
ADF	213	gpm			
Drainage Time					
PHF	9	minutes			
ADF	36	minutes			
Sensor On/Off Controls					
Off Sensor	3	ft	from bottom		
On Sensor	6	ft	from top		
Alarm Sensor	2	ft	from top		
System Curve					
z1	1	ft	(intake pipes placed 1 feet above bottom of wet well		
y1	0	ft	open to atmosphere		
v1	9.66	fps	PHF		
v1	2.42	fps	ADF		
z2	28		(pumped to 6 feet above grade)		
y2	0	ft	open to atmosphere at aerated grit chamber		
v2	9.66	fps	PHF		
v2	2.42	fps	ADF		

Dry Well/ Wet Well continued

Head Loss from Pipe			
Hazen William			
C=	140	assuming ductile iron piping	
Q=	852	gpm PHF	
	213	gpm ADF	
D=	6	inches	
hL/100ft=	4.8841531	ft	PHF
	0.37477807	ft	ADF
length of piping	53	ft	
	0.53		
hL=	2.58860114	ft	PHF
	0.19863237	ft	ADF
hpump=	29.5886011	ft	PHF
hpump=	27.1986324	ft	ADF

hpump	Q	Q
ft	gpm	cfs
27.0000	0	0
27.0007	10	0.022283
27.0025	20	0.044567
27.0053	30	0.06685
27.0090	40	0.089133
27.0136	50	0.111417
27.0190	60	0.1337
27.0253	70	0.155983
27.0324	80	0.178267
27.0403	90	0.20055
27.0490	100	0.222833
27.0585	110	0.245117
27.0687	120	0.2674

Aerated Grit Chamber (Metcalf & Eddy pg. 373)

Typical Design Criteria

	Design Criteria					
	Low	High	Average	Proposed Design		Meet Criteria?
t_d @ PHF (min.)	2.00	5.00	3.00	5.06		OK
Depth (ft)	7.00	16.0	11.5	4.00		LOW
Length (ft)	25.0	65.0	45.0	24.0		OK
Width (ft)	8.00	23.0	15.5	6.00		LOW
W:D	1:1	5:1	1.5:1	1.5:1		OK
L:W	3:1	5:1	4:1	4.5:1		OK
Air supply/L (ft^3/ft-min)	3	8	5.5	132	ft^3/min	
Grit Quantity (ft^3/Mgal)	0.5	27	2	2.45	ft^3/min	

Proposed Design

Volume(grit chamber)=Q(PHF)*td

assume td=	5	(chose high detention time to increase volume of reactor needed)
PHF=	1.23	MGD
V=	569	ft3

Assume smallest D, W, L possible

D=	7	ft	
W=	8	ft	
L=	25	ft	
V=	1400	ft3	(lowest dimension require too high of a volume)

try lowering dimensions and keeping the design criteria ratios good

D=	5	ft	W:D	1	OK...
W=	5	ft	L:W	4.8	OK
L=	24	ft			

D=	6	ft	W:D	1.5	OK
W=	9	ft	L:W	1.222222	<<<<
L=	11	ft			

D=	4	ft	W:D	1.50	OK
W=	6	ft	L:W	4.00	OK
L=	24	ft			
V_{new}=	576	ft3			
td_{new}=	5.06	min	(slightly higher than 5 OK)		

How does it perform at all expected flows?

	Q	t_d	V
	MGD	**min**	ft^3
ADF	0.307	20.2	576
Wet ADF	0.344	18.0	576
Dry ADF	0.275	22.6	576
Min. ADF	0.001	4308	576
Max. ADF	0.668	9.28	576
Min. MDF	0.219	28.3	576
Max. MDF	0.402	15.4	576
PHF	1.226	5.06	576

University of Nevada, Reno

T&K Designs

Step Feed Biological Nitrogen Removal Process Design (pg. 819)

Assumptions:

Q (ADF)=	1161	m3/d
# passes=	4	per (anoxic/aerobic)
Q split=	0.1	pass 1
	0.4	pass 2
	0.3	pass 3
	0.2	pass 4
MLSS	3000	mg/L in final aerobic zone
Qras/Q	0.6	
Anoxic Volume=	0.15	
DO=	2	mg/L
Total aeration tar	236	m3
Temp=	12	C
Effluent NH4-N	2.5	mg/L

Typical Design Parameters (pg. 793 Metcalf & Eddy)

SRT	d	3-15
F/M	kg BOD/ kg MLVSS-d	0.2-0.4
lb BOD	lb BOD/ 1000 ft3-d	40-60
kg BOD	kg BOD/ m3-d	0.7-1.0
MLSS	mg/L	1500-4000
Total tau	h	41703

WW characteristics (partial data/ Example 8-6 assumed values)

	(mg/L)			
BOD	123	(data)	bCOD/BOD=	1.8
COD	300	(assumed)		
nbCOD	76	(calculated from assumed)		
rbCOD	80	(assumed)		
bCOD	224	(assumed)		
sCOD	132	(assumed)		
sBOD	70	(assumed)		
nbsCODe	20	(calculated from assumed)		
nbpCOD	56	(calculated from assumed)		
VSS	60	(assumed)		
VSSCOD	2.80	(calculated from assumed)		
nbVSS	20	(assumed)		
iTSS	96	(calculated from data/assumed)		
TSS	156	(data)		
TKN	35.3	(data)		
ALK	140.0	(assumed)		
TSS0-VSS0	10	(assumed)		

University of Nevada, Reno

T&K Designs

Step Feed Biological Nitrogen Removal Process Design continued

Aeration/Anoxic Zone Volumes				
Total Volume=	236	m3		
Anoxic Volume=	35.4	m3		
Aerobic Volume	200.6	m3		
Anoxic/pass	8.85	m3	312.5348305	ft3
Aerobic/pass	50.15	m3	1771.030706	ft3
RAS concentration				
assume that effluent TSS concentration from secondary clarifier is negligible when doing MB				
XR=	8000	mg/L or g/m3		
MLSS concentration in each pass				
Pass 1 MB				
Assume solids production for single pass is negligible				
X1=	6857	g/m3		
Pass 2 MB				
X2=	4364	g/m3		
Pass 3 MB				
X3=	3429	g/m3		
Pass 4 MB				
X4=	3000	g/m3		
Aerobic SRT				
Solids Balance				
XV=	885	kg		
A	138			
B	1.82			
C	5.55			
D	23.2	nbVSS		
E	11.6	inert solids		
EQN	0.00011997	(USED SOLVER TO FIND SRT)		
Aerobic SRT	6.71	days		
Overall SRT	7.89	days		

Step Feed Biological Nitrogen Removal Process Design continued

Composition of MLSS and MLVSS

Assume MLVSS is 78% of MLSS

Item	MLVSS	Fraction of total MLVSS	MLSS
	kg		kg
Heterotrophs	494	0.696	581
Cell Debris	43.7	0.062	51.4
Nitrifiers	16.6	0.0234	19.5
nbVSS	156	0.219	156
Inert Organics			77.9
Total	710		885

MLVSS/MLSS	0.802	

Fraction of biomass solids

Heterotrophs Bio	0.696	MLVSS	
Nitrifiers Biomas	0.0234	MLVSS	(updated below after correction)

Nitrogen for nitrifier growth (all done to correct MB on MLVSS for nitrifier biomass)

Biomass production

Biomass=	537	kg VSS/d
Daily Wasting=	80.1	kg VSS/d

assuming 0.12 g N/ g VSS biomass

N used for synthe	9.61	kg N/ d

N used for synthesis based on influent flow

N used for synthe	8.28	g N/m3 or mg N/L

assume effluent ammonia-N concentration is 2.41 mg/L

NOx=TKN-N(syn)-NH4asN

NOx=	24.6	mg/L
Mass nitrifiers=	15.1	kg VSS attributed to nitrifiers
Corrected MLVSS	708	kg MLVSS
Nitrifier as mass	0.0213	(as compared to 0.0163)

Step Feed Biological Nitrogen Removal Process Design continued

Summary Table

Pass	MLSS	MLVSS	Nitrifiers	Biomass
	g/m3	kg/d	g VSS/ m3	g VSS/ m3
1	6857	5497	117	3823
2	4364	3498	74.5	2433
3	3429	2748	58.5	1912
4	3000	2405	51.2	1673

Nitrification Rates

assume SNH4=	2.41	mg NH3-N/L
EQN 1=	0.00335957	
SNH1=	0.1841	mg/L NH4-N
Rn1=	773	g/d
EQN 2=	-0.00055219	
SNH2=	2.36418564	mg/L NH4-N
Rn2=	1509	g/d
EQN 3=	-0.00032265	
SNH3=	2.86	mg/L NH4-N
Rn3=	1223	g/d
EQN 4=	-3.0013E-11	
SNH4=	2.41	mg/L NH4-N
Rn4=	1040	g/d

Step Feed Biological Nitrogen Removal Process Design continued

Determine Amount of Nitrate Removal in Anoxic Zones

Calculate F/Mb

F/Mb= 0.424 g BOD/g V <u>total applied substrate rate</u> 0.2-0.4 range slightly higher

total microbial biomass

Obtain SDNRb

% rbCOD= rbCOD/bCOD

% rbCOD= 35.7 %

SDNR equation coefficients (Table 8-22 Metcalf & Eddy)		<u>linear interpolation</u> % rbCOD	bo	b1
bo=	0.239	30	0.235	0.141
b1=	0.147	40	0.242	0.152

SDNRb (20C)= 0.112 g NO3-/MLVSS,biomass-d

SDNRb (12C)= 0.0916 g NO3-/MLVSS,biomass-d

Nitrate Removed= SDNRb(Vnox)(Xb)

NO3- removed= 3099 g NO3-/d

Pass	Xb	Q Influent	Anoxic Volume/pass	F/Mb	SDNR12	NO3-N Removal
	g VSS/ m3	m3/d	m3	g BOD/ g VSS-d	g NO3--N/g MLVSS-d	g NO3--N/d
1	3823	116	8.85	0.424	0.0916	3099
2	2433	464	8.85	2.66	0.356	7675
3	1912	348	8.85	2.54	0.351	5936
4	1673	232	8.85	1.94	0.318	4711

University of Nevada, Reno

T & K Designs

Step Feed Biological Nitrogen Removal Process Design continued

Nitrate Balance

Pass	Total NO3--N to pass	Anoxic Removal Capacity	NO3--N remaining after anoxic	NO3--N produced (Rn) in pass	Effluent NO3--N
	g/d	g/d	g/d	g/d	g/d
1	390	3099	0	773	773
2	773	7675	0	1509	1509
3	1509	5936	0	1223	1223
4	1223	4711	0	1040	1040

Determine Effluent NO3-N Concentration

Assume the effluent concentration is 0.50 g/m3 nitrate to calculate NO3--N fed to pass 1 then perform iterations till pass 4 effluent concentration matches with assumed value.

Nitrate effluent=	0.5603279	g/m3
NO3--N=	0.560	mgNO3--N/L

Total N Concentration

TN=	2.97	mg N/L

BOD Removal (pg. 756 Example 8-6 w/ Nitrification Metcalf & Eddy)

Mass Balances for VSS and TSS have already been computed, just to summarize

PX,VSS=	106	kg VSS/ day
PX,TSS=	132	kg TSS/ day
X,VSS=	708	kg VSS
V,TSS=	885	kg TSS
Fraction VSS=	0.802	kg VSS/ kg TSS
assuming MLSS= 3000 g/m3		
MLVSS=	2405	g VSS/m3

Step Feed Biological Nitrogen Removal Process Design continued

Calculate F/M and BOD loading for aerobic regions			Typical Range	
F/M=	0.2971	kg BOD/ kg VSS-d	0.2-0.4	OK
BOD loading=	0.71	kg BOD/ m3-d	0.7-1.0	OK
Determine observed yield based on TSS and VSS				
bCOD removed= Q(bCOD-BODloading*1.8bCOD/BOD)=Q(S0-S)				
bCOD removed=	258	kg bCOD/d	468.86	kg BOD/d
Yobs,TSS=	0.51	g TSS/g bCOD		
Yobs,TSS=	0.93	g TSS/g BOD		
Observed Yield based on VSS				
Yobs,VSS	VSS/TSS=	0.802	kg VSS/ kg TSS	
Yobs,VSS=	0.41	g VSS/g bCOD		
	0.74	g VSS/g BOD		
Assume TSSe=	10.0	mg/L		
gBOD/gUBOD	0.6	g BOD/gUBOD		
gUBOD/g VSS	1.4	gUBOD/gVSS		
Assume sBODe=	3.0	mg/L		
BODe=	9.83	mg/L	(Using worse case scenario for settling characteristics in 2nd clarifier)	
BODe=	5.73	mg/L	(Using best case scenario for settling charicteristics in 2nd clarifier)	
Oxygen Demand				
R0=	239	kg O2/h		

Step Feed Biological Nitrogen Removal Process Design continued

Oxygen Demand						
R0=	239	kg O2/h				
Fine Bubble Aeration Design (Assumed 35% efficiency)						
Oxy. Xfer Rate	239	kg O2/h			Pb/Pa	0.654592
alpha	0.65		(assumed)			
Beta	0.95		(assumed)			
Fouling Factor	0.9		(assumed)			
Sat. SeaLvl DO and Ope	10.78	mg/L				
Sat. SeaLvl DO and 20 C	9.09	mg/L				
Sat. SeaLevel DO @ 20 f	9.66373475	mg/L				
Sea Level Pressure	10.33	m				
Elevation Pressure	6.76193272	m				
Diffuser Depth	1.63	m				
Operation DO	2	mg/L				
Basin Temp	12	celcius				
Correction Factor	0.4					
Std. Oxy. Xfer Rate	931.0	kg/h				
Density of Air						
@ 12 C	1.07037602					
Air Flowrate	178.7	m3/min				

University of Nevada, Reno

T&K Designs

Step Feed Biological Nitrogen Removal Process Design continued

Check Alkalinity

Assumed Alkalinity needed to maintain pH 6.8-7.0 is 70g/m3 as CaCO3

Influent ALK	140	g/m3 as CaCO3
ALK used	176	g/m3 as CaCO3
ALK to be added=	106	g/m3 as CaCO3
ALK load added=	123	kg/d as CaCO3

sodium carbonate preffered due to ease of handlig and fewer scaling problems

NaHCO3 load added=	206	kg/d as NaHCO3	**as sodium bicarbonate

Anoxic Zone Mixing Energy

**choose highest mixing energy for worst case scenario

mixing energy (max)=	8	kW/103m3	typical range	
mixing energy (max)=	3	kW/103m4	3-8	kW/103m3
			0.1-0.3	hp/103ft3

University of Nevada, Reno

T&K Designs

Secondary Clarification Design

Flow Characteristics			
Q-ADF (m3/d)	1161		
Q-PK (m3/d)	1522		
Q-RAS (m3/d)	697		
[MLSS] (g/m3)	3000		
Physical Specs			
Side Water Depth (m)	4	13.12336	
Diameter (m)	7	22.96588	
Surface Area (m2)	38.465		
Volume (m3)	153.86		
Bottom Slope	1/10		
Average Flow	SI	English	
SOR (m/d)	30.18	740.6665	gal/ft2*d
HRT (hr)	3.18		
SLR (kg TSS/m2*hr)	6.04	1.251172	lb TSS/ft2*d
WLR (m3/lin m*d)	52.82	4251.426	ft3/ft*d
Peak Flow			
SOR (m/d)	39.57	970.9685	gal/ft2*d
HRT (hr)	2.43		
SLR (kg TSS/m2*hr)	7.91	1.64021	lb TSS/ft2*d
WLR (m3/lin m*d)	69.24	5573.359	ft3/ft*d
Influent/Effluent Pipe Sizing			
Pipe Radius (m)	0.0597166	0.137348	1.64817922 "
Flow Velocity (m/s)	1.2000466		
Chosen Radius (m)	0.1016	4"	
Actual Velocity (m/s)	0.4145734	(1.3 ft/s)	
RAS Pipe Sizing	Goal: 0.6-1.2 m/s		
Pipe Radius (m)	0.0654751	0.2 ft.	2.4"
Flow Velocity (m/s)	0.5992899		
Chosen Radius (m)	0.05	2"	
Actual Velocity (m/s)	1.0276598		

Typical Design Parameters (pg. 890 Metcalf & Eddy)

SOR (avg)	m/d	24-32
SOR (pk)	m/d	40-64
SLR (avg)	kg TSS/ m^2*hr	5-8
SLR (pk)	kg TSS/ m^2*hr	10
Side Water Depth	m	4.0-55
WLR (avg)	m^3/lin m*d	125 max
WLR (pk)	m^3/lin m*d	250 max

BOD Removal	
Avg. Flow	38.97%
Pk. Flow	36.47%
TSS Removal	
Avg. Flow	61.13%
Pk. Flow	58.51%

University of Nevada, Reno

T&K Designs

Disinfection

Estimated Coliform Removal

**highlighted values represent values used for further calculations

	Percent Removal		
Process	**Low**	**Average**	**High**
Coarse screens	0	2.5	5
Fine screens	10	15	20
Grit chambers	10	17.5	25
Total Primary	20	35	50
Activated sludge	90	94	98
Total Secondary	90	94	98

Effluent Fecal Coliform Data

Average (FC/100mL)	31
Minimum (FC/100mL)	1
Maximum (FC/100mL)	6200

Influent Fecal Coliform Estimate (Metcalf & Eddy, 2014)

	Low	**Average**	**High**
Total Coliform (MPN/100 mL)	1.00E+07	5.05E+08	1.00E+09
Fecal Coliform (MPN/ 100 mL)	1.00E+06	5.05E+07	1.00E+08

University of Nevada, Reno

T & K Designs

Disinfection continued

Effluent Standard (MPN/100 mL)

Target TC count	Raw Wastewater	Primary Effluent	Secondary Effluent
(MPN/100 mL)	N/N_0	N/N_0	N/N_0
1,000	1.00E-06	2.00E-06	5.00E-03
200	2.00E-07	4.00E-07	1.00E-03
23	2.30E-08	4.60E-08	1.15E-04
2.2	2.20E-09	4.40E-09	1.10E-05

Target TC count	Raw Wastewater	Primary Effluent	Secondary Effluent
(MPN/100 mL)	Ct	Ct	Ct
1,000	430.44	340.74	21.08
200	739.12	585.74	39.13
23	1524.50	1209.10	85.06
2.2	3338.61	2648.96	191.15

**using Colins-Selleck Model

15 minute contact time

Target TC count	Raw Wastewater	Primary Effluent	Secondary Effluent
(MPN/100 mL)	C	C	C
1,000	28.7	22.7	1.41
200	49.3	39.0	2.61
23	102	80.6	5.67
2.2	223	177	12.7

30 minute contact time

Target TC count	Raw Wastewater	Primary Effluent	Secondary Effluent
(MPN/100 mL)	C	C	C
1,000	14.3	11.4	0.7
200	24.6	19.5	1.3
23	50.8	40.3	2.8
2.2	111.3	88.3	6.4

University of Nevada, Reno

T&K Designs

Disinfection continued

90 minute contact time

Target TC count	Raw Wastewater	Primary Effluent	Secondary Effluent
(MPN/100 mL)	C	C	C
1,000	4.8	3.8	0.2
200	8.2	6.5	0.4
23	16.9	13.4	0.9
2.2	37.1	29.4	2.1

45 minute contact time

Target TC count	Raw Wastewater	Primary Effluent	Secondary Effluent
(MPN/100 mL)	C	C	C
1,000	9.6	7.6	0.5
200	16.4	13.0	0.9
23	33.9	26.9	1.9
2.2	74.2	58.9	4.2

Required chlorine dosage

considering breakpoint chlorination

$NaOCl\text{-}Cl/NH_4^+\text{-}N$	3.80	
$NH_4^+\text{-}N$ effluent=	2.41	mg/L or g/m^3
Breakpoint Cl demand=	9.1	g Cl/m^3

University of Nevada, Reno

Disinfection continued

Total Required Cl dosage for Secondary Effluent at 30 minute contact time

For 200 MPN/ 100 mL

	Q	NaOCl-Cl	NaOCl-Cl	NaOCl-Cl
		t=90 min.	t= 30 min.	t= 15 min.
	(m^3/d)	(kg/d)	(kg/d)	(kg/d)
PHF	4642	44.4	48.5	54.5
ADF	1161	11.1	12.1	13.6

For 2.2 MPN/ 100 mL

	Q	NaOCl-Cl	NaOCl-Cl	NaOCl-Cl
		t=90 min.	t= 30 min.	t= 15 min.
	(m^3/d)	(kg/d)	(kg/d)	(kg/d)
PHF	4642	52.3	72	102
ADF	1161	13.1	18.0	25.4

Alkalinity Requirements

ALK $CaCO_3$/ NH_4^+-N= 14.3

ALK required= 34.4 mg/L or g/m^3 $CaCO_3$

ALK load= 159.7 kg/d $CaCO_3$ PHF

39.92 kg/d $CaCO_3$ ADF

Chosen Design Parameters

t=90 min.

200 MPN/100 ml	Q	NaOCl-Cl	Cl Dose	C_{200}
	(m^3/d)	(kg/d)	(mg/L)	mg/L
PHF	4642	66.6	9.57	0.435
ADF	1161	16.7	9.57	-
2.2 MPN/ 100 ml	**Q**	**NaOCl-Cl**	**Cl Dose**	**C_{200}**
	(m^3/d)	(kg/d)	(mg/L)	mg/L
PHF	4642	78.4	11.3	2.12
ADF	1161	19.6	11.3	-

T & K Designs

Disinfection continued

Dosage Control for NaOCl

low capacity proportioning pumps

max. cap.= 120 gal/d

pumps constant rate or variable speed with analog signals for varying feed rate?

injection and initial mixing

typical hanging nozzle type chlorine diffuser for open channels

plug flow chlorine contact basin w/ addins to reduce hydraulic dead zones

>submerged baffles

L:W 20:1 40:1

>Design a chlorine contact basin for secondary effluent with the given ADF and PHF values. The PHF will be used for calculations and the detention time will be 90 min. Two channels will be constructed in parallel for redundancy requirements. Designed to achieve disperion number equal to or less than 0.015 at PHF. Efficienct of the chlorine contact basins when low flow periods occur (solids, etc.)

** During a PHF event, the basins will run simultaneously (designed so flow splits)

Assume...

# passes=	4			
W=	1.6	m	5.25	ft
W/pass=	0.4		1.31	ft
D=	1.6	m	5.25	ft
t_d=	90	min		
Determine length				
Total L=	56.7		185.92	ft
L basin=	14.2		46.48	ft
L:W=	35.4	In the range of 20 and 40, OK		
v_{PHF}=	0.01049	m/s		
v_{ADF}=	0.002624	m/s		

	Design Criteria	Proposed Design
W (m)	-	1.60
W/pass (m)	-	0.400
D (m)	-	1.60
freeboard (m)	-	0.610
L (m)	-	56.7
L:W	20:1-40:1	35.4:1
# passes	4-5	4

Disinfection continued

v	0.010494	m/s			
R	0.533333	m			
viscosity=	1.003E-06	m^2/s	(at 20C)		
Reynolds # (PHF)=	22,320	R=4vR/vis.			
coefficient of dispersion=	6.47E-03	D=1.01vR^0.875			
dispersion #=	0.01088	<0.015 design OK, good mixing			
# reactors in series=	46				
Reynolds # (ADF)=	5,580	R=4vR/vis.			
coefficient of dispersion=	1.92E-03	D=1.01vR^0.875			
dispersion #=	0.00323	<0.015 design OK, good mixing			
# reactors in series=	155				
Under ADF, velocity is small and solids deposition is expected to occur, basins will have to be shut down and cleaned out occassionally.					
baffles					
Total Ac=	0.64	m^2			
Q=	2321	m^3/d			
Cd=	0.7000	m/s			
Orf D=	50	mm			
D=	0.05	m			
Orf A=	0.001963495	m^2			
# orifices=	25	orifices			
Q/orifice=	-	m3/s			
v=	0.2500	m/s	(0.25-0.55)		
hL=	0.0065244	m		6.5244	mm
hL/orifice=	6.524	mm	(3-9mm/orifice)		
total hL=	162.5	mm			
total hL=	0.53	ft			

University of Nevada, Reno

T&K Designs

Disinfection continued

Chlorine Storage

>15% NaOCL soluiton chosen

Density= 150 g/L

For 200 MPN/ 100 mL

	Q	NaOCl-Cl
	(m3/d)	(gal/hr)
PHF	4642	3.26
ADF	1161	0.815

For 2.2 MPN/ 100 mL

	Q	NaOCl-Cl
	(m3/d)	(gal/hr)
PHF	4642	3.84
ADF	1161	0.959

For conservative storage estimate assume chlorine demand for PHF at 200 MPN/ 100 mL for 2 month period

Volume Chlorine/ 2 months= 4695 gallons

>Order 4800 gallons of NaOCl 15% every 2 months

Volume of Drums= 800 gallons

Number of Drums= 6 drums

Drums will be handled acccording to MSDS specifications and will be stored in a separate room in a cool, dark place with appropriate corrosion resistant container (concrete).

Sludge Handling

1 ft^3 = 0.028 m^3

Solids Retention Time

Volume	236.0488	m3
X bio	3000	mg/l
X Eff.	15	mg/l
Q Eff.	1160.376	m3/d
SRT	33.48284	day
Qw	3743.895	mg/l

Typical Design Parameters and Assumptions

X_e	mg/l	10-20
X_w	mg/l	6000 - 12000

X_w (mg/l)	Q_w (m3/d)	Q_wX_w
6000	0.6239825	3743.895
7000	0.5348421	3743.895
8000	0.4679869	3743.895
9000	0.4159883	3743.895
10000	0.3743895	3743.895
11000	0.3403541	3743.895
12000	0.3119912	3743.895

Volume	X bio	X eff	Q	Q eff	X_w	SRT	Q_w (m3/d)
236.0488	3000	15	1161	1148.741	6000	7.8	12.259482
236.0488	3000	15	1161	1150.496	7000	7.8	10.5043665
236.0488	3000	15	1161	1151.811	8000	7.8	9.1888541
236.0488	3000	15	1161	1152.834	9000	7.8	8.16616583
236.0488	3000	15	1161	1153.652	10000	7.8	7.34832248
236.0488	3000	15	1161	1154.321	11000	7.8	6.67938097
236.0488	3000	15	1161	1154.878	12000	7.8	6.12206925

Continuous Belt Filter Press

Design Parameters

	Range	Typical
Belt Width (m)	0.5-3.5	2
Belt Speed (m/min)	1.0-2.5	
Loading (L/m*min Width)	40-150	
Polymer (g/kg Dry Solids)	4-10	6

SLR Check

5 Hour loading cycle

Q, Sludge = 10m3/d / 5hrs/day = 2m3/hr

Q, Sludge	**ρ**	**Spec Grav.**	**% Solids**	**Mass Sldge.**	
2	1000	1.02	0.07	142.8	kg/hr

Hydraulic Loading

Q, Sludge	Belt Width			
2	1	33.3	L/m*min	Slightly Low!

Solids Produced Per Week

23990.4 kg/week

assumed SG of 1.02 => Density = 1020 kg/m3

23.52 m3/wk

Polymer

@ 6g Poly per kg Dry Solids

=> 4284 g Poly/day

Disposal

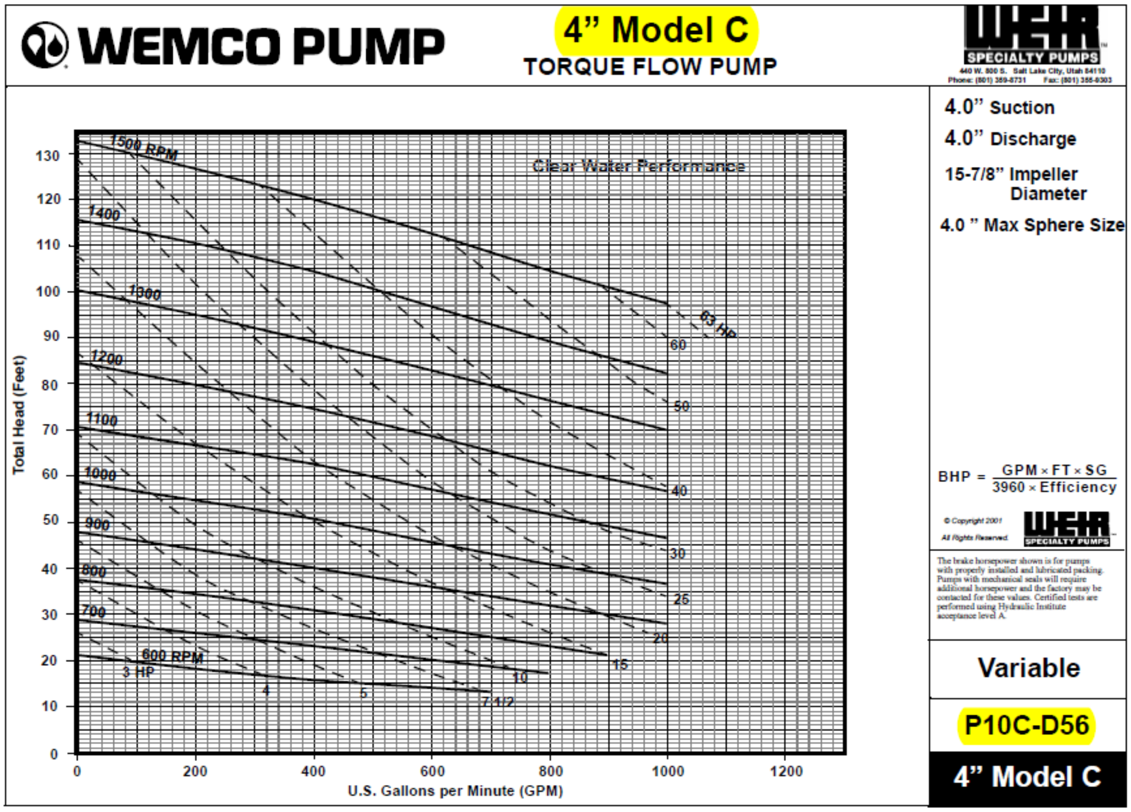
Sludge collected in hopper

Delivered to landfill weekly or bi-weekly

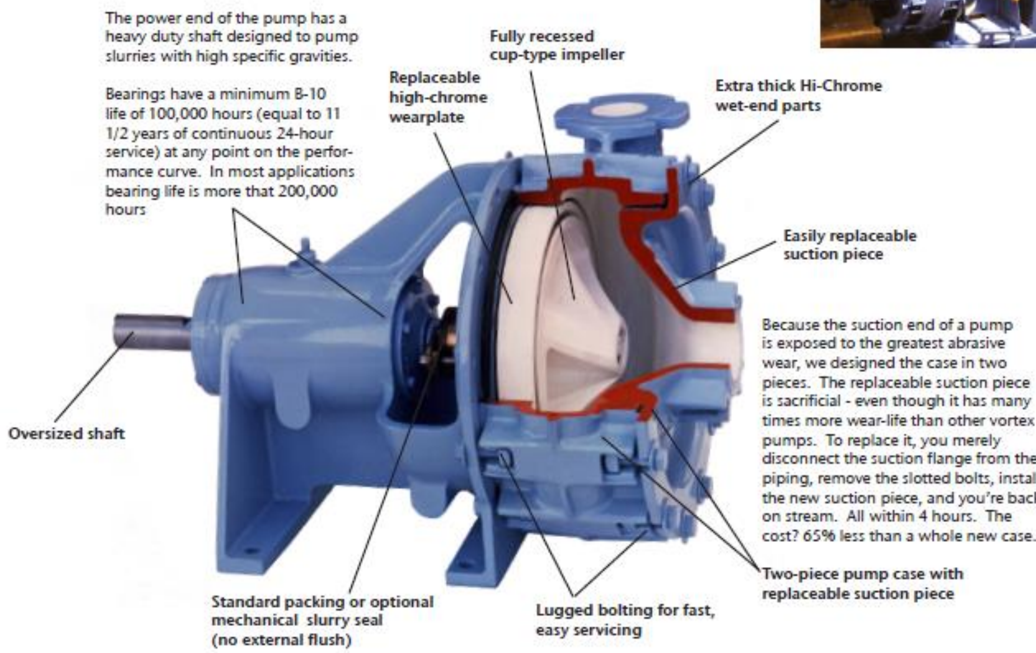
University of Nevada, Reno

Design Cost Analysis					**Year**	**ENR**
					1980	2475
(EPA Operation and Maitenence Costs for MWWTPs)					2014	9796
Administrative Costs (Fig. 3.1)						
	$10,000.00	/yr				3.95798
Costs (1980 Estimates)						
Piping/Water Conveyance			**BNR** (BNR Processes and Costs 2007)			
	$15.00	/ft	*Construction*			
Total	$4,500.00		*Based on Celanese Step-Feed BNR w/ 1.25 MGD capacity*			
			ea.	$593,925.40		
Fine Screening (Fact Sheet 3.1.17)			Total	$1,187,850.80		
Construction			*Electricity*			
ea.	$20,000.00			$100,000.00	kWh/yr	
Total	$40,000.00		Total	$9,190.00	/yr	
O & M			*O & M*			
ea.	$3,000.00	/yr	ea.	$20,000.00	/yr	
Total	$6,000.00	/yr	Total	$40,000.00	/yr	
Secondary Clarifier (Fact Sheet 3.1.3)			**Belt Filter Press**			
Electricity (per two units)			*Electricity (per two units)*			
	$10,000.00	kWh/yr				
Total	$919.00	/yr	Total	$500.00	/yr	
Construction			*Construction*			
ea.	$300,000.00		ea.	$48,500.00		
Total	$600,000.00		Total	$97,000.00		
O & M			*O & M*			
ea.	$10,000.00	/yr	ea.	$700.00	/yr	
Total	$20,000.00	/yr	Total	$1,400.00	/yr	
Solids Disposal			**Totals (1980)**			
O & M			Capital Costs		$2,529,350.80	
ea.	$80,000.00	/yr	Annual Costs		$448,009.00	
Total	$160,000.00	/yr	*O & M*			
			Power			
Disinfection						
Construction			**Totals (2014 Adjusted)**			
ea.	$300,000.00		Capital Costs		$10,011,119.37	
Total	$600,000.00		Annual Costs		$1,773,210.57	
O & M			*O & M*			
ea.	$100,000.00	/yr	*Power*			
Total	$200,000.00	/yr				

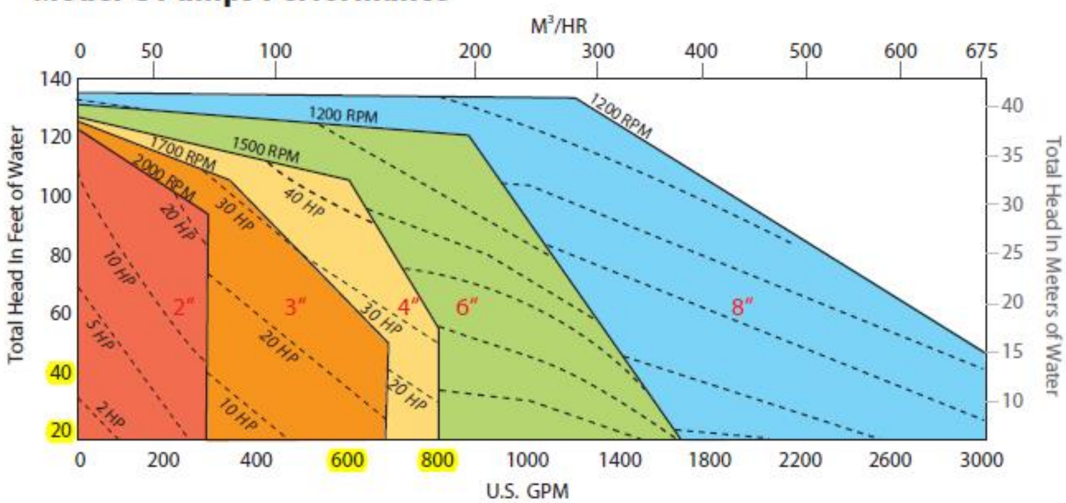
Appendix B


WEMCO PUMP
4" Model C
TORQUE FLOW PUMP
WEIR SPECIALTY PUMPS
4.0" Suction
4.0" Discharge
15-7/8" Impeller Diameter
4.0 " Max Sphere Size
Clear Water Performance
1500 RPM
1400
1300
1200
1100
1000
900
800
700
600 RPM
3 HP
4
5
7 1/2
10
15
20
25
30
40
50
60
63 HP
Total Head (Feet)
U.S. Gallons per Minute (GPM)
BHP = GPM × FT × SG / 3960 × Efficiency
© Copyright 2001 All Rights Reserved.
The brake horsepower shown is for pumps with properly installed and lubricated packing. Pumps with mechanical seals will require additional horsepower and the factory may be contacted for these values. Certified tests are performed using Hydraulic Institute acceptance level A.
Variable
P10C-D56
4" Model C

Advantages of Model C Mechanical Design



Model C Pumps Performance

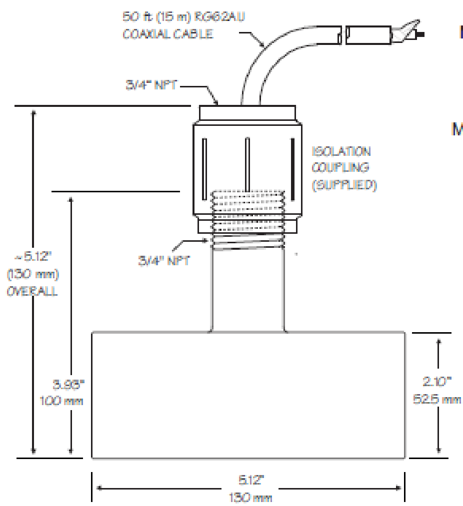


GREYLINE instruments inc.

DATA SHEET

SENSOR MOUNTING

Ultrasonic Sensor Model PZ56



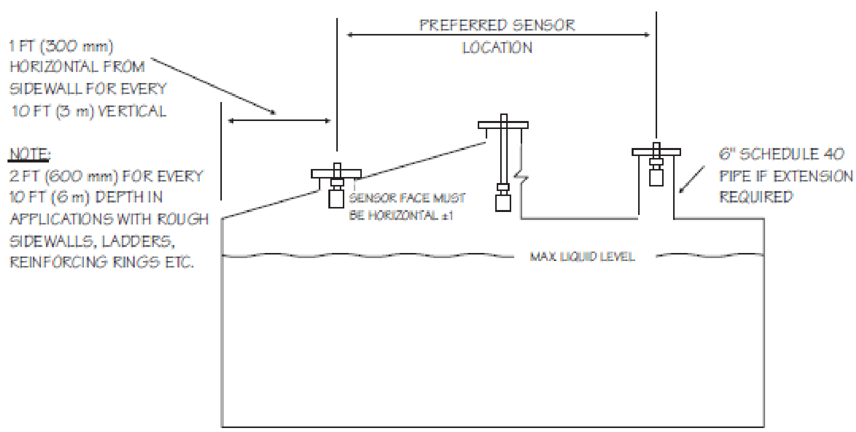
General Specifications

Maximum Range:	50 ft. (15.25 m)
Minimum Range (Deadband):	16" (406.4 mm)
Operating Frequency:	34 KHz
Beam Angle:	6°
Temperature Compensation:	Automatic, continuous
Operating Temperature:	-40° to 150° F (-40° to 65° C)
Maximum Operating Pressure:	20 psi (1.35 bar)
Exposed Materials:	PVC
Sensor Mounting:	3/4" NPT
Maximum Cable Length:	500 ft (150 m)
Optional Hazardous Rating:	CSA rated Intrinsically Safe Class I, Groups C,D, Class II, Groups E,F,G with optional Intrinsic Safety Barrier. Note: Max Range reduced to 25 ft. (7.8 m) with ISB option.

Notes:

1. Shield from direct sunlight (temperature probe in sensor)
2. Use the "Isolation Coupling" supplied for mounting.
3. Use with Option ISB Intrinsic Safety Barrier for installation in hazardous locations.
4. Run Sensor cable in plastic or grounded metal conduit with no other conductors.
5. Extend Sensor cable up to 500 ft (150 m) with RG62AU coaxial only.
6. Refer to the Instruction Manual for complete installation and wiring instructions.

Recommended Sensor Mounting



PROJECT PROFILE

15410.08.09

LOW FLOW TRIDEN™ SCREEN

MODEL
LFS-11-95-6L and WCP-6-6S-28

LOCATION
Avila Beach, CA USA

APPLICATION
Municipal Wastewater Treatment Plant

PEAK FLOW
1 MGD (44 l/s)

GRID OPENINGS
6 mm - Stainless Steel Laced Links

SCREEN HEIGHT & WIDTH
95" x 11" (2,413 mm x 279 mm)

COMPACTOR LENGTH
28" (711 mm)

COMPACTOR DIAMETER
6" (152 mm)

CONTROLS
NEMA 4X 508A UL listed control panel with stainless enclosure, recycle timer, and ultrasonic level sensor and controller

Top of screen from rear with covers removed.

Screen in channel with neoprene seals.

401 E. Douglas Rd.
Oldsmar, FL USA

HYDRO-DYNE® Engineering, Inc.
www.hydro-dyne.com

Phone (813) 818-0777
Fax (813) 818-0770

buckman's inc.
BuckmansInc.com

SPECIFICATIONS: SODIUM HYPOCHLORITE SOLUTION - 15%

SPECIFICATIONS

Component	Basis	Specification
Available Chlorine	grams per liter	150 minimum
Sodium Carbonate	grams per liter	1.0 maximum
Sodium Hydroxide	grams per liter	.4-10
Heavy Metals	ppm by weight	<1
Temperature	Fahrenheit	<90
Specific Gravity	H20 = 1	1.190-1.210
Color		Yellow- Green

NOTES:
STORE IN A COOL DRY PLACE TO MAINTAIN PRODUCT QUALITY.
MEETS ANSI/NSF STANDARD 60 FOR DRINKING WATER ADDITIVE.

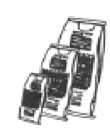
SAFETY:
HANDLE PRODUCT WITH CARE. IT IS SUGGESTED THAT ANYONE INVOLVED IN HANDLING OF THIS PRODUCT SHOULD BE FAMILIAR WITH THE APPROPRIATE SAFETY PRECAUTIONS.

All statements, information and data presented are believed to be accurate and reliable but are presented without guarantee, warranty, or responsibility.

ver: Oct 2008

Sodium Hypochlorite – Muriatic Acid
ONE GALLON - 2 1/2 GALLON - 5 GALLON
15 GALLON - 55 GALLON - BULK

buckman's inc.
105 Airport Road
Pottstown, PA 19464
610-495-7495
FAX 610-495-7229

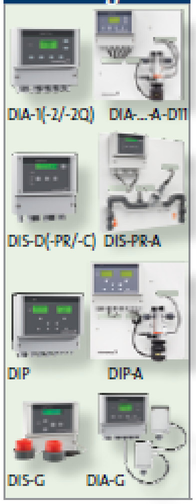


Bagged Products

Diatomaceous Earth	Aluminum Sulfate
Calcium Chloride	Soda Ash Briquettes
Light Soda Ash	Sodium Bisulfate
Sodium Bicarbonate	Sodium Thiosulfate

Measurement and control

Measuring units



Measuring cells and sensors

AquaCell – for Cl_2 / ClO_2 / O_3; single rod probe – pH, redox (ORP); membrane covered – H_2O_2, PAA; conductivity – conductive, inductive; gas sensor – amperom., potentiostatic

Parameters

1 1 1 1 – 1. parameter
2 2 – 2. parameter
3 – 3. parameter
A – amperometric
P – potentiostatic
AP – amperom. or potentiostatic

Measuring units		AQC-D11: cleaning motor, pressure-proof	AQC-D12: hydromechanical cleaning, pressure-proof, with water sensor	AQC-D13: hydromechanical cleaning, pressureless	pH-value	redox-potential (ORP)	hydrogen peroxide	peracetic acid	conductive	inductive	amperometric	potentiostatic	chlorine Cl_2	chlorine dioxide ClO_2	ozone O_3	hydrogen peroxide H_2O_2	peracetic acid PAA	conductivity	pH value	redox potential (ORP)	temperature	ammonia	hydrochloric acid
Conex® DIA / DIS and DIP measuring amplifiers and controllers																							
DIA-1	(1 parameter)	●	●	●	●	●	●	●					1	1	1	1	1		1	1	●)		
DIA-2	(2 parameters)	●	●	●	●		●						1	1	1	1			2		●)		
DIA-2Q *)	(2 parameters)	●	●	●	●	●	●	●					1	1	1	1	1		2	2	●)		
DIS-D	(1 parameter)	●	●	●									1	1	1								
DIS-PR	(1 parameter)				●	●													1	1	●)		
DIS-C	(1 parameter)								●	●								1			●)		
DIP	(3 parameters)	●	●	●	●	●							1	1	1				2	3	●)		
Conex® DIA-x-A / DIS-x-A and DIP-A preassembled compact systems																							
DIA-1-A-*D11/-D12/-D13*		●	●	●	●								1	1	1				●		●)		
DIA-2-A -*D11/-D12/-D13*		●	●	●	●								1	1	1				2		●)		
DIA-2Q-A -*D11/-D12/-D13* *)		●	●	●	●	●							1	1	1				2	2	●)		
DIA-1-A-PR					●	●													1	1	●)		
DIA-2-A-PR					●	●													1		●)		
DIA-1-A-HP							●									1							
DIA-1-A-PA								●									1						
DIS-D-A -*D11/-D12/-D13*		●	●	●									1	1	1								
DIS-PR-A					●	●													1	1			
DIP-A -*D11/-D12/-D13*		●	●	●	●	●							1	1	1				2	3	●)		
Conex® DIA-G / DIS-G gas warning units control 2 gases simultaneously																							
DIA-G											●	●	AP	AP	AP							P	P
DIS-G											●		A	A	A								

*) Conex® DIA-2Q with additional 4-20 mA input to compensate variable flow rates ●) only for compensation in case of temperature fluctuations

*) Conex® DIA-2Q-A with additional 4-20 mA input to compensate variable flow rates ●) only for compensation in case of temperature fluctuations

DIT mobile photometer measures all important parameters in water chemistry

DIT-M
DIT-L

DIT-M: aluminium, bromine, chlorine (free, combined, total), chlorine dioxide, chloride, chlorite, cyanuric acid, iron, fluoride, manganese, ozone, phosphate, pH, acid demand to pH 4.3, hydrogen peroxide

DIT-L: chlorine (free, combined, total), chlorine dioxide, chlorite, ozone, pH

Features and benefits

DMH

Hydraulic diaphragm dosing from 0.50 to 2 x 278 GPH.

Fig. 1 DMH

Fig. 2 DMH 283 duplex version

Fig. 3 DMH 251 with stroke frequency controller, leak detector, and remote stroke length control

The preferred choice for demanding applications
The Grundfos DMH range is a series of hydraulic pumps for applications demanding a high-quality pump. With its high accuracy and modern interface possibilities, the DMH is ideal for many applications in water treatment and industrial processing.

Prepared for performance and safety in extreme situations
The DMH 250 series of pumps is available with PVC, PVDF, polypropylene, stainless steel and Hastelloy C wetted components. For high-pressure requirements, select from the series of stainless steel or Hastelloy C DMH 280 pumps, rated up to 2900 PSI. All models are fitted with a PTFE diaphragm, with the AMS diaphragm protection system and internal relief valve for pump protection.

Get the pump configuration you need
Control the capacity manually by adjusting the stroke length from 0 to 100 %, with a +/- 1 % repeatable accuracy.

Additional options:

- Duplex version offers twice the capacity of a simplex pump. It is also used for blending applications.
- Electric 4-20 mA servomotor (electric actuator) or pneumatic stroke length control
- Variable speed drive controller
- Integrated stroke sensor and electronic counter
- Leak detection with double diaphragm and alarm contact.

Typical applications:

- Municipal and industrial water treatment
- Wastewater treatment
- Chemical industry
- Boiler feed
- Petroleum industry
- Filtration systems
- pH control
- Demineralizers
- Pulp & paper
- Textile
- Food & beverage.

API 675 models
DMH 250 and 280 series pumps are available in API 675 compatible versions. This is commonly used in petroleum, chemical refineries, and transmission pipeline applications. Contact Grundfos for available models.

Performance range

Performance range by max. pressure

DMH 250

Model	Size	Max. capacity [GPH]	Max. pressure [PSI]	stokes/min.
251	2.4-10	0.77	145	17
251	5-10	1.58	145	35
251	13-10	4.22	145	75
251	19-10	6.07	145	115
252	11-10	3.45	145	35
252	24-10	7.66	145	75
252	37-10	11.62	145	115
253	21-10	6.60	145	35
253	43-10	13.70	145	75
253	67-10	20.60	145	115
253	83-10	26.10	145	144
254	50-10	15.80	145	32
254	102-10	32.20	145	65
254	143-10	45.40	145	90
254	175-10	55.40	145	110
254	213-10	67.30	145	134
255	194-10	61.50	145	65
255	270-10	85.50	145	90
255	332-10	105.00	145	110
255	403-10	128.00	145	134
257	220-10	69.70	145	33
257	440-10	139.40	145	65
257	575-10	182.20	145	90
257	770-10	244.00	145	110
257	880-10	278.00	145	134
251	2.3-16	0.74	232	17
251	4.9-16	1.55	232	35
251	12-16	3.70	232	75
251	18-16	5.81	232	115
252	10-16	3.17	232	35
252	23-16	7.13	232	75
252	36-16	11.35	232	115
254	97-16	30.60	232	65
254	136-16	43.00	232	90
254	166-16	52.80	232	110
254	202-16	63.90	232	134
251	2.2-25	0.69	363	17
251	4.5-25	1.43	363	35
251	11-25	3.43	363	75
251	17-25	5.28	363	115

DMH 280

Model	Size	Max. capacity [GPH]	Max. pressure [PSI]	stokes/min.
286	85-50	26.90	725	67
286	111-50	35.10	725	88
286	170-50	53.90	725	134
281	2-100	0.60	1450	35
281	4.2-100	1.30	1450	76
281	6.4-100	2.00	1450	115
281	8-100	2.50	1450	144
283	19-100	6.10	1450	65
283	27-100	8.40	1450	90
283	33-100	10.60	1450	110
283	40-100	12.70	1450	134
285	20-100	6.30	1450	33
285	40-100	12.70	1450	67
285	52-100	16.60	1450	88
285	70-100	22.20	1450	118
285	80-100	25.30	1450	134
280	1.3-200	0.50	2900	76
280	2.2-200	0.70	2900	115
280	2.5-200	0.90	2900	144
287	18-200	5.80	2900	67
287	23-200	7.40	2900	88
287	31-200	9.80	2900	118
287	36-200	11.40	2900	134
288	7.5-200	2.40	2900	67
288	10-200	3.30	2900	88
288	13-200	4.10	2900	118
288	15-200	4.90	2900	134

Note: Double GPH with duplex versions.

Performance range by model

DMH 251

Size	Max. capacity [GPH]	Max. pressure [PSI]	str./min.
2.2-25	0.69	363	17
2.3-16	0.74	232	17
2.4-10	0.77	145	17
4.5-25	1.43	363	35
4.9-16	1.55	232	35
5-10	1.58	145	35
11-25	3.43	363	75
12-16	3.70	232	75
13-10	4.22	145	75
17-25	5.28	363	115
18-16	5.81	232	115
19-10	6.07	145	115

Dimensions, DMH 251-257

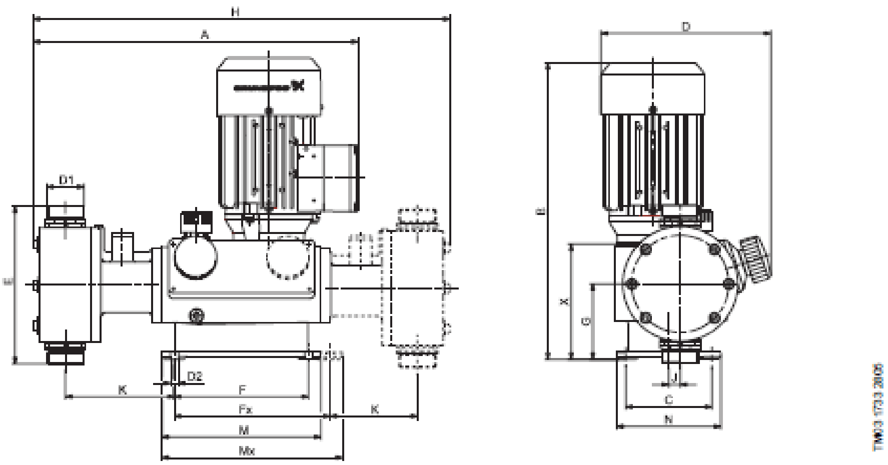


Fig. 28 Dimensions, DMH 251-257

Dimensions in inches (mm)

Pump	Model	A	B	C	D	D1*	D2	E	F	Fx	G	H	J	K	M	Mx	N	X
DMH 2.2-25	251	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 2.3-16	251	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 2.4-10	251	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 4.5-25	251	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 4.9-16	251	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 5-10	251	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 10-16	252	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 11-10	252	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 11-25	251	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 12-16	251	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 13-10	251	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 17-25	251	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 18-16	251	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 19-10	251	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 21-10	253	14.49 (368)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 1 1/4	0.35 (9)	7.05 (179)	5.98 (152)	5.98 (152)	3.37 (85.5)	18.58 (472)	0.51 (13)	4.88 (124)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 23-16	252	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)
DMH 24-10	252	13.58 (345)	13.23 (336)	3.84 (97.5)	7.56 (192)	R 5/8	0.35 (9)	6.30 (160)	5.98 (152)	5.98 (152)	3.37 (85.5)	17.01 (432)	0.63 (16)	4.57 (116)	7.09 (180)	7.09 (180)	4.63 (117.5)	5.14 (130.5)

2 Cubic Yard Standard Duty Dumping Hopper

Safety chain secures hopper to forklift.

Enlarge

Made for really tough jobs. Safely transport and dump heavy loads.

- Welded seams. Handles wet or dry materials.
- Easy to unload. Simply pull cable from the seat of your forklift to release hopper.
- Automatically returns to locked position after dumping.
- Lid sold separately.
- 6" steel caster option provides mobility.

MODEL NO.	DESC.	SUGGESTED USE	CUBIC YARD	CAP. (LBS.)	FORK POCKET CENTERS	OVERALL L x W x H	WT. (LBS)	PRICE EACH 1	PRICE EACH 2+	ADD TO CART
H-2109	Standard	Wood, Corrugate	2	2,000	28" W	69 x 57 x 52"	692	$999	$979	1 ADD

DROP SHIPS IN 3 DAYS FROM IN

Additional Info Email Page

DIMENSIONS:
* Fork Pockets: 7 1/2 x 2 1/2" (wxh)
* Actual Dimensions: 69 x 56 1/2 x 52"
* Usable Chute Size: 53 x 64 x 42" (wxlxh)
* Safety Chain: 3ft.

SIZE:
* Will not fit through a standard doorway.
* 2 Cubic Yard= 403 gallons
54 Cubic feet
43.4 Bushels

MATERIAL:
* Body: 12 gauge
* Base: 7 gauge

COMPATIBLE CASTERS:
6" Polyolefin: H-3322
6" Rubber: H-3326
6" Steel: H-2112
8" Polyolefin: H-3323
8" Rubber: H-3327
8" Pneumatic: H-3328
10" Pneumatic: H-3329

Ships Via Motor Freight

Availability: Drop Ship
Unit Weight: 692 lbs.

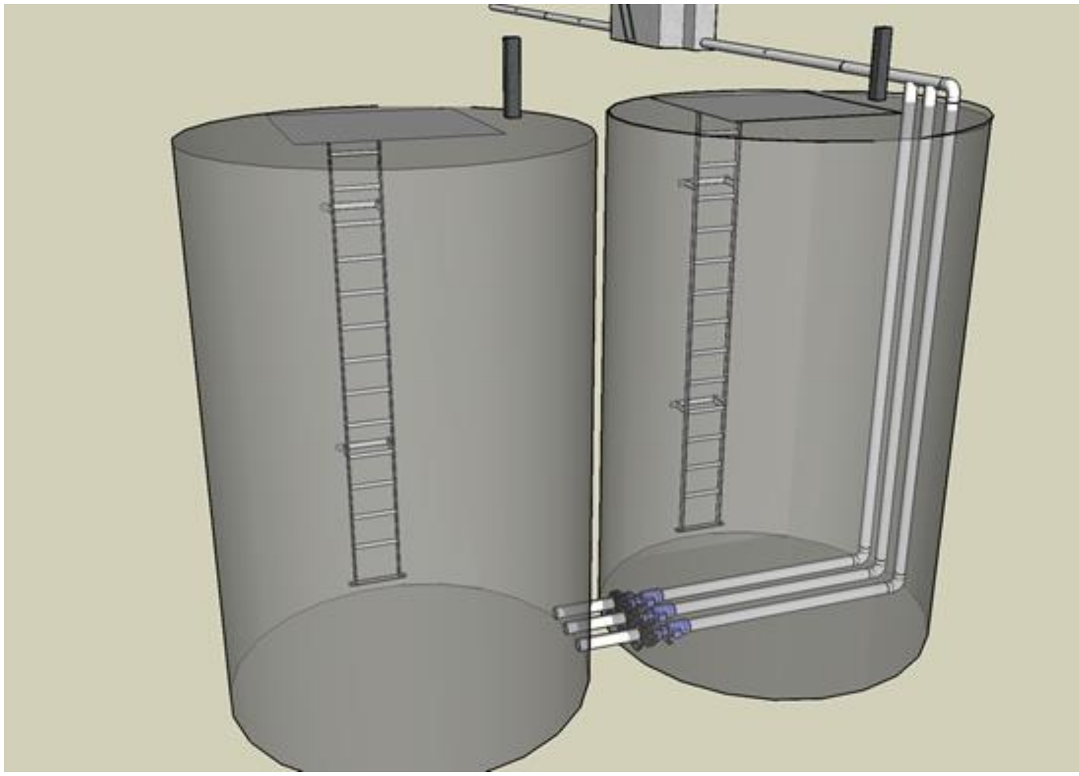
Instructions

Parts List

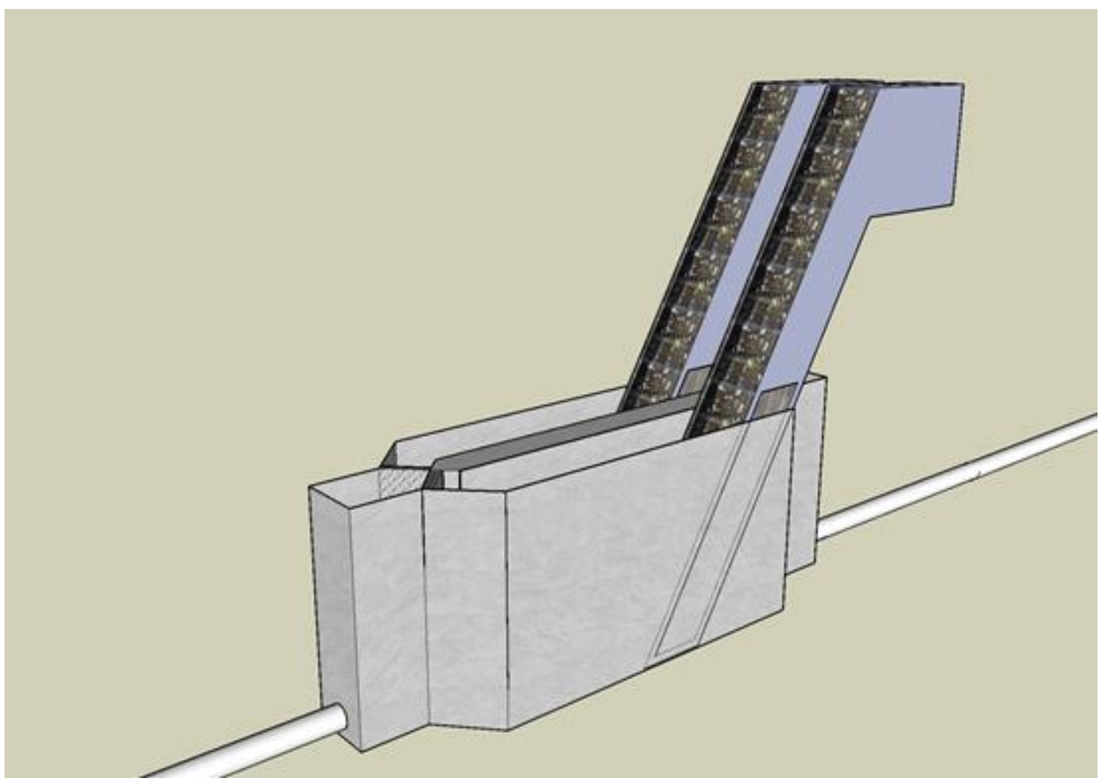
Catalog Page 337

Country of Origin: USA

Appendix C



Wet well and dry well combined unit



Fine screening unit

Secondary clarifier