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# The Impact of Advanced Wastewater Treatment Technologies and Wastewater Strength on the Energy Consumption of Large Wastewater Treatment Plants

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# THE IMPACT OF ADVANCED WASTEWATER TREATMENT TECHNOLOGIES AND WASTEWATER STRENGTH ON THE ENERGY CONSUMPTION OF LARGE WASTEWATER TREATMENT PLANTS

By

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ABSTRACT

**The Impact of Advanced Wastewater Treatment Technologies and Wastewater Strength on the Energy Consumption of Large Wastewater Treatment Plants**

by

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Wastewater treatment is an energy intensive process often requiring the use of advanced treatment technologies. Stricter effluent standards have resulted in an increase in the number of wastewater treatment plants (WWTPs) with advanced treatment over time. Accordingly, associated energy consumption has also increased. Concerns about lowering operating costs for WWTPs and reducing associated greenhouse gas generation present an incentive to investigate energy use in WWTPs. This research investigated the impact of wastewater strength and the introduction of advanced treatment technologies, to replace traditional technologies on energy use to treat wastewater in WWTPs. Major unit processes were designed for a 100 MGD plant and variables controlling energy were identified and used to compute energy consumption.

Except for primary clarification and plate and frame press dewatering, energy consumption computed using fundamental equations are within values in the literature. Results show that energy consumption for dissolved air flotation thickeners, centrifuges, gravity thickeners, and aeration basins are heavily influence by wastewater strength.

Secondary treatment and tertiary treatment require a significant amount of energy. Secondary treatment requires 104 times the energy of preliminary treatment, 17 times the energy of solids processing, and 2.5 times the energy of tertiary treatment. Secondary treatment requires 41 times the energy of preliminary treatment, and 7 times the energy of solids processing.

The results of this research provide a means of estimating energy consumption in the design and operation phase of a WWTP. By using the fundamental equations and methodology presented, alternative technologies can be compared or targeted for future energy savings implementation. Limitations of the methodology include design assumptions having to be made carefully, as well as assumptions of motor and equipment efficiencies.

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DEDICATION

To my parents and grandparents who have always

believed in me even when I did not.

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# CHAPTER 1

# INTRODUCTION AND OBJECTIVES

Wastewater treatment (WWT) is an energy-intensive process, and the need to meet stricter effluent standards often requires the use of advanced treatment technologies such as biological nutrient removal (BNR), ultraviolet (UV) disinfection, and membrane filtration (Metcalf and Eddy, 2003; U.S. EPA, 2010b). Figure 1.1 shows the percentage of the US population utilizing different levels of WWT between 1996 and 2028. The population served by advanced treatment (i.e., greater than secondary treatment) increased by 36.3% while the population served by secondary treatment increased by only 13.2% (U.S. EPA, 2008; U.S. EPA, 2010a). It is projected that that the population served by secondary treatment will decrease by 4.0% between 2008 and 2028, while the population served by advanced treatment will increase by 42.7% (U.S. EPA, 2010a). During 2008, roughly 50% of the U.S. population was served by advanced treatment plants (U.S. EPA, 2010a). Energy consumption for WWT is also estimated to increase another 30 to 40% over the next 20 to 30 years (Metcalf and Eddy, 2003). The water and wastewater industries combined are estimated to consume roughly two to four percent of the total energy consumption in the United States (Metcalf and Eddy, 2003; U.S. EPA, 2010b; WEF, 2009). Over a 14-year period, this consumption has increased 33% from 75 billion kWh per year in 1996 to 100 billion kWh per year in 2010 (U.S. EPA, 2010b). Energy consumption in WWTPs also represents 18% (Molinos-Senante, et al., 2010) to 30% (Metcalf and Eddy, 2003) of a WWTP's total operations costs. Rising energy costs are an incentive for WWTPs to investigate ways to lower their overall energy consumption (Metcalf and Eddy, 2003; U.S. EPA, 2010b; WEF, 2009).

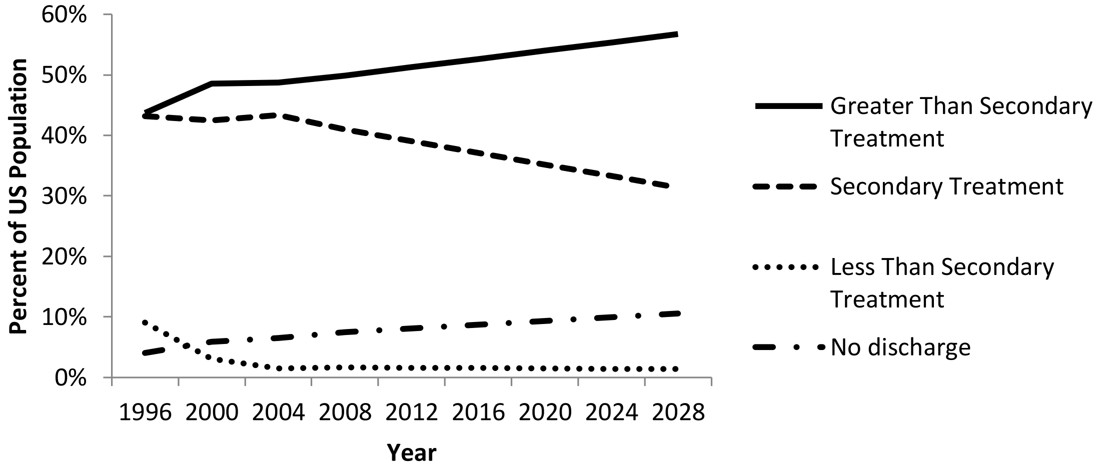


**Figure 1.1- Percent of US Population Served by Varying Levels of WWT Modified From (U.S. EPA, 2008; U.S. EPA, 2010a)**

Greenhouse gas (GHG) minimization in WWTPs provides another incentive to reduce overall energy consumption. The Kyoto Protocol proposed that countries worldwide to reduce their GHG production by 5.2% between 2008 and 2012 (UNFCCC, 1998) as compared to 1990 GHG levels. While the United States never adopted the Kyoto protocol, it calls on the United States to reduce its GHG production by 7% as compared to 1990 GHG levels (UNFCCC, 1998). WWT is listed as one of the sources of GHG in the Kyoto Protocol (UNFCCC, 1998). Between 1990 and 2009, emissions of GHG in the United States increased 7% from 6,181.8 to 6,633.2 Tg $CO_2$ equivalents (U.S. EPA, 2011). For WWT, the GHG production increased by 8.3% during the same period from 27.2 to 29.5 Tg $CO_2$ equivalents which represents a larger increase than total US GHG emissions (U.S. EPA, 2011). In response to concerns about GHG, various major cities including Los Angeles and New York have documented their GHG production (Moke, et al., 2011; Planyc, 2011).

The effect of wastewater strength on energy consumption in WWTP has not been fully investigated (Gori, et al., 2011). There is also a need to investigate the contribution of advanced treatment technologies such as UV disinfection and centrifugation to the overall energy consumption of a WWTP.

Given their high energy use and GHG concern, there is a need to evaluate energy use in WWTPs so that individual areas of high energy consumption can be identified and targeted for energy reduction and GHG curbing. The research reported herein contributes to this area of knowledge and includes the following specific objectives:

1. Evaluate whether design and fundamental energy consumption equations can be used to evaluate energy consumption in a WWTPs.
2. Evaluate the impact of wastewater strength on energy consumption for a large (100 MGD) advanced WWTP based upon energy consuming units of individual unit operations.
3. Investigate the impact on energy consumption for a large WWTP resulting from switching traditional treatment technologies to advanced treatment technologies.

It is hypothesized that:

1. Aeration of activated sludge will be largest energy-consuming operation in the WWTP regardless of wastewater strength. This is based upon previous work where aeration is almost always the largest consumer (Metcalf and Eddy, 2003; WEF, 2009).

2. Energy requirements are directly proportional to wastewater strength. This is due to Total Kjeldahl Nitrogen (TKN) having a high impact on aeration costs (Rittman, et al., 2000) and increased solids production.
3. Incorporating full biological nutrient removal (BNR) to remove total nitrogen (ammonia and nitrate) and phosphorous will not have a significant impact on energy requirements as compared to partial BNR to remove ammonia and phosphorous (Foley, et al., 2010).

# CHAPTER 2

# EVALUATION OF ENERGY USE IN WASTEWATER TREATMENT PLANTS USING DESIGN EQUATIONS AND ENERGY CONSUMING UNITS AS A FUNCTION OF WASTEWATER STRENGTH


## Abstract

Wastewater treatment is energy intensive and concerns about lowering operating energy costs and reducing associated greenhouse gas generation present an incentive to investigate WWTP energy use. This research investigated the impact of wastewater strength (low, average, and high strength) on energy consumption in advanced WWTPs. Major unit processes were designed for a 100 MGD plant and fundamental variables controlling energy usage were identified. Energy consuming units were then identified and energy usage was estimated. Unit processes evaluated include bar racks, aerated grit chambers, primary and secondary clarifiers, aeration basins, dual-media filtration, ultraviolet disinfection, gravity thickening, dissolved air flotation (DAFTs), and centrifugation. The results of this research came close to previous literature estimates in all cases but primary clarification. Processes heavily influenced by wastewater strength include DAFTs, centrifuges, gravity thickeners, and aeration basins. This research is most useful for designers as it provides a means of estimating energy consumption in the design phase of a wastewater treatment plant.


## 1 Introduction

Wastewater treatment (WWT) is an energy intensive process, and the need to meet stricter effluent standards often requires the use of advanced treatment technologies such as biological nutrient removal (BNR), ultraviolet (UV) disinfection, and membrane filtration (Metcalf and Eddy, 2003; U.S. EPA, 2010b). The percentage of the US population served by different levels of WWT between 1996 and 2028 is shown in Figure 2.1. The population served by advanced treatment (i.e greater than secondary treatment) increased by 36.3% while the population served by secondary treatment increased by only 13.2% (U.S. EPA, 2008; U.S. EPA, 2010a). It is projected that the population served by secondary treatment will decrease by 4.0% between 2008 and 2028, while the population served by advanced treatment will increase by 42.7% (U.S. EPA, 2010a). During 2008, roughly 50% of the U.S. population was served by advanced treatment plants (U.S. EPA, 2010a). Energy consumption for WWT is also estimated to increase another 30 to 40% over the next 20 to 30 years (Metcalf and Eddy, 2003). The water and wastewater industries combined are estimated to represent roughly two to four percent of the total energy consumption in the United States (Metcalf and Eddy, 2003; WEF, 2009; U.S. EPA, 2010b). Over a 14 year period, this consumption has increased 33% from 75 billion kWh per year in 1996 to 100 billion kWh per year in 2010 (U.S. EPA, 2010b). Energy consumption in WWTPs is estimated to represent 18% (Molinos-Senante, et al., 2010) to 30% (Metcalf and Eddy, 2003) of a WWTP's total operations costs. Rising energy costs are an incentive for WWTPs to investigate ways to lower their overall energy consumption (Metcalf and Eddy, 2003; WEF, 2009; U.S. EPA, 2010b).

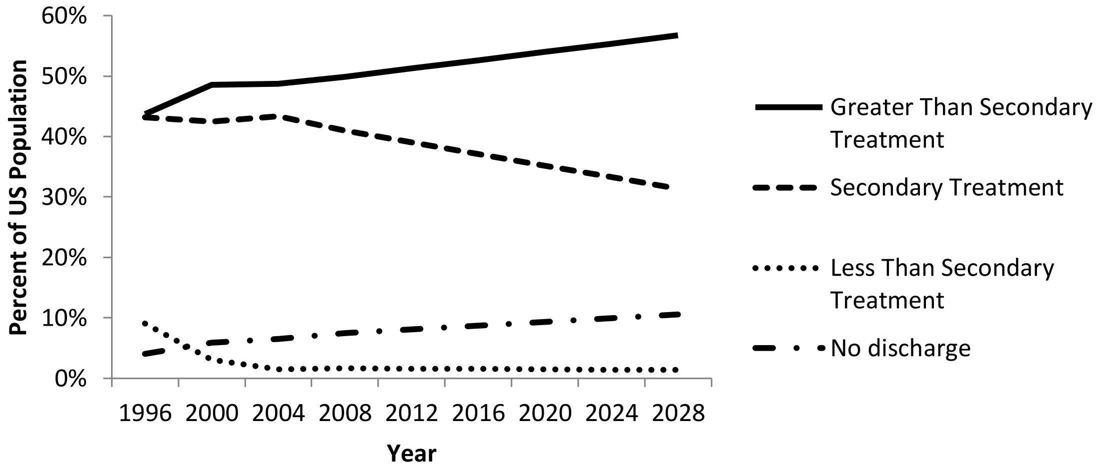


**Figure 2.1 - Percent of US Population Served by Varying Levels of WWT Modified From (U.S. EPA, 2008; U.S. EPA, 2010a)**

Greenhouse gas (GHG) generation from WWTPs provides another incentive to reduce overall energy consumption. Between 1990 and 2009, emissions of GHG in the United States increased 7% from 6,181.8 to 6,633.2 Tg $CO_2$ equivalents (U.S. EPA, 2011). For WWT, the GHG production increased by 8.3% during the same period from 27.2 to 29.5 Tg $CO_2$, equivalents which represents a larger increase than total US GHG emissions (U.S. EPA, 2011).

As a result of energy costs and GHG concerns, there is a need to evaluate energy usage in WWTPs so that individual areas of high energy consumption can be identified and targeted for energy reduction and GHG curbing.

Activated sludge aeration is the most significant process often identified for energy reduction due to its very high energy consumption (Metcalf and Eddy, 2003; WEF, 2009). Various studies have investigated ways to reduce energy consumption and associated GHG. For instance, improving aeration efficiency was found to reduce GHG by as much as 10.5% in winery wastewaters (Rosso, et al., 2009). Another study found that controllers on aeration for activated sludge reduce GHG and energy consumption and that a low solids retention time (SRT) reduces GHG and energy consumption (Flores-Alsina, 2011). Yet another way a WWTP can lower its overall GHG generation is through the use of anaerobic digestion (Poulsen, et al., 2009), which can generate part of the energy consumed by the plant. Also, where advanced treatment is not required, anaerobic processes have been found to be more energy efficient and generate less GHG than aerobic processes while producing acceptable effluent standards (Keller, et al., 2003). In certain wastewaters, however, aerobic treatment processes actually produce less GHG than anaerobic treatment processes with regards to chemical oxygen demand (COD) removal (Cakir, et al., 2005; Shahabadi, et al., 2009). For at least one WWTP in China anaerobic digestion and aerobic digestion of sludge were found to generate similar GHG (Wei, et al., 2008).

In addition to aeration, BNR processes for nitrogen removal were found to require more energy and produce more GHG while BNR processes for phosphorous removal do not significantly impact energy requirements or GHG generation (Vidal, et al., 2002; Shahabadi, et al., 2009; Foley, et al., 2010). It has also been determined that nitrogen removal can increase energy consumption and GHG production by as much as 150% and 146%, respectively (Vidal, et al., 2002). The previous studies conflict with another study which found that full WWT with BNR has the ability to reduce GHG on a global scale due to carbon sequestration (Rosso, et al., 2008).

Effluent discharge requirements also affect energy usage in WWTPs. For example, for only primary treatment and anaerobic digestion, energy requirements in one study were estimated as 397.7 kWh/MGal (Foley, et al., 2010). Incorporating activated sludge to further remove BOD increased energy consumption to 1,530.3 kWh/MGal

(Foley, et al., 2010). Incorporating nitrification increased energy consumption to approximately 2,575 kWh/MGal (Foley, et al., 2010). Incorporating denitrification (approximately 20 mg/L N) in a Modified Ludzack-Ettinger (MLE) setup decreased energy consumption to approximately 2,400 kWh/MGal due to oxygen credits from denitrification (Foley, et al., 2010). Incorporating complete denitrification (approximately 5 mg/L N) in a five stage Bardenpho setup decreased energy consumption to approximately 2,200 kWh/MGal due to oxygen credits from denitrification (Foley, et al., 2010).

Variable frequency drives (VFDs) present a means of reducing energy consumption for WWTPs (Pacific Gas and Electric Company, 2003; Europump and Hydraulic Institute, 2004; WEF, 2009; U.S. EPA, 2010b). VFDs are electronic controllers that adjust the output of energy to a process component which allows for the speed of process components such as pumps to be controlled (Pacific Gas and Electric Company, 2003; U.S. EPA, 2010b). This is accomplished by a converter in the controller varying voltages to create a magnetic flux in a motor (Europump and Hydraulic Institute, 2004). It has been reported that 75% of pumps are oversized and VFDs provide a means to better match system conditions (Europump and Hydraulic Institute, 2004). VFDs have also been used in situations where valves were used to control flowrates (Europump and Hydraulic Institute, 2004). Valve controls are much less energy efficient than VFDs (Europump and Hydraulic Institute, 2004). VFDs are able decrease energy consumption by as much as 30 to 50% (Pacific Gas and Electric Company, 2003; Europump and Hydraulic Institute, 2004).

Energy recovery technologies such as wind, solar, microturbines, and fuel cells also provide a means for lowering energy consumption for WWTPs (WEF, 2009). Wind turbines operate by converting mechanical energy (wind) to electrical energy (U.S. EPA, 2007). Wind turbines have efficiencies between 20 and 40 percent and have been used successfully in several WWTPs (U.S. EPA, 2007). Solar panels operate by absorbing light and transferring the energy to a semiconductor where electrons are allowed to flow and form a circuit with an electrical current that provides external energy (U.S. EPA, 2007). Typical efficiencies for solar panels are 5 to 17 percent (U.S. EPA, 2007). Solar panels have been able to provide as much as 30% of a WWTPs total energy requirements or about 4,100 kWh/day (Collingwood, et al., 2011). Fuel cells operate similar to a battery (U.S. EPA, 2011). Hydrogen flows in on the anode side and oxygen flows in on the cathode side with an electrolyte separating the cathode and anode (U.S. EPA, 2011). The hydrogen diffuses into protons and electrons where electrons pass through a circuit and provide energy (U.S. EPA, 2011). Water is then created and exits the fuel cell (U.S. EPA, 2011). Microturbines operate by combusting gas and spinning turbine fans at high speeds to rotate copper coils and create energy (U.S. EPA, 2011). Microturbines are relatively inexpensive compared to other gas generators and require little maintenance (U.S. EPA, 2011). Biogas from anaerobic digestion can be used with microturbines (U.S. EPA, 2011). For an 11 MGD WWTP, ten 30 kW microturbines were installed that provide energy savings of 2300 MW per year (U.S. EPA, 2011).

Some studies have also addressed the impact of plant size on energy consumption; in general, larger plants were found to be more energy efficient than smaller plants in terms of kWh of energy per volume of water treated (Hernández-Sancho, et al., 2009; Hernández-Sancho, et al., 2011).

There have been studies that have addressed some aspects of energy usage in WWTPs. For example, the effect of particulate and soluble matter on energy consumption in wastewater treatment has been evaluated (Gori, et al., 2011). Similarly, the effect of different BOD strengths of wastewater on energy consumption and GHG generation for aerobic and anaerobic systems has been investigated (Cakir, et al., 2005). The effect of high strength biochemical oxygen demand (BOD) wastewater has also been addressed (Shahabadi, et al., 2009; Shahabadi, et al., 2010).

A comprehensive evaluation of energy consumption in wastewater treatment plants, based on energy consuming units of specific unit processes, has not been performed to date. Furthermore, the effect of wastewater strength on energy consumption has not yet been addressed in depth.

The objective of this research is to evaluate energy consumption in WWTPs using as a basis design equations to size energy consuming units for all unit operations of the WWTP. Another major objective is to explore the impact of wastewater on strength consumption. It is expected that the results from the research will help identify unit processes that can be targeted for energy savings in WWTPs. In addition, the methodology and equations presented here can be used to obtain baseline estimates of energy consumption as well as estimated GHG generation potential from energy use.

## 2 Methodology

In order to evaluate energy consumption in an advanced WWTP, major unit processes were designed for a 100 MGD plant. Next, the energy consuming units of every process were identified and the energy use was computed for individual units. Only the energy associated with treatment (unit operations) was computed in this research. Other energy requirements such as those for building heating/cooling, lighting, chemical manufacturing and transport were not computed in this research. The design was based on an existing WWTP in the arid southwestern United States. The design of the plant as a whole focused on sizing the unit processes and on identifying fundamental variables in each unit that control energy usage. Actual design criteria from the full-scale plant were used to validate designs used in the research. The plant is designed to remove biochemical oxygen demand (BOD), total suspended solids (TSS), ammonia, and phosphorous.

A process flow diagram of the example WWTP is shown in Figure 2.2. Solid lines in represent liquid flows, and dashed lines represent solids flows. Influent enters the plant and passes through the bar racks where large solids are removed, and grit is removed in the aerated grit chambers. BOD and TSS are then partially removed in the primary clarifiers. Aeration basins and secondary clarifiers then remove BOD, TSS, ammonia and phosphorous. Some of the TSS remaining is also removed in the dual media filters and the effluent is disinfected with ultraviolet irradiation (UV). Primary solids are thickened in gravity thickeners while secondary solids are thickened in dissolved air flotation thickeners (DAFTS). Primary and secondary solids are then combined and dewatered in centrifuges with the cake being sent to a landfill. The centrate from the centrifuges is recycled back to the primary influent. As filter backwashing is not a constant process, it is not shown in Figure 2.2. The effects of the backwash were addressed in the design of the dual media filters and UV disinfection, however.

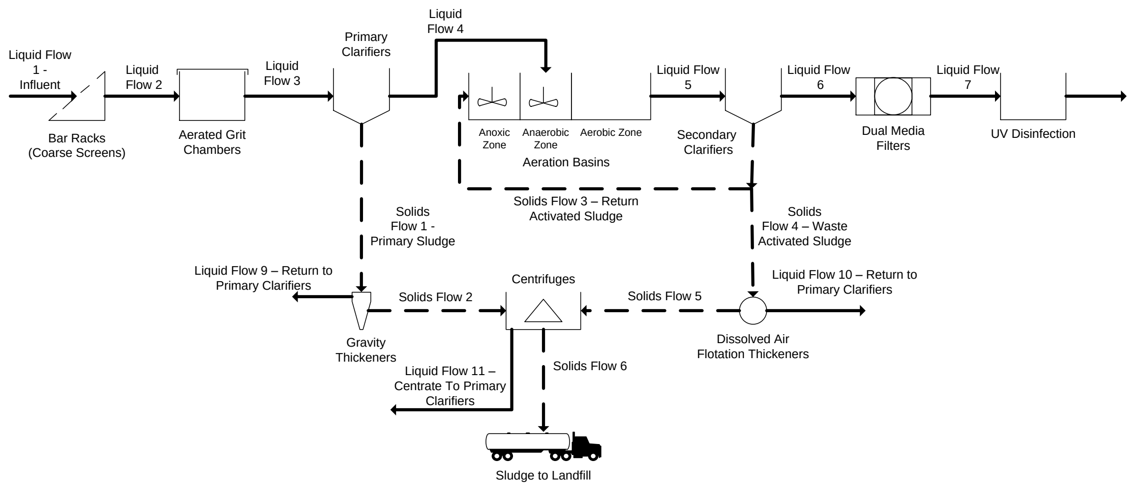


**Figure 2.2 - Process Flow Diagram of Waste Water Treatment Plant for Which Energy Consumption Was Evaluated**

## 2.1 Wastewater Influent and Effluent Standards

The wastewater effluent standard and the three wastewater strengths for the influent assumed in this research are shown in Table 2.1 and were based on data in Metcalf and Eddy (2003). It should be noted that the values in Table 2.1 represent municipal wastewater strengths and not industrial wastewater strengths. Different wastewater strengths were selected to examine the effect of wastewater strength on energy consumption. The low strength wastewater has an influent total suspended solids (TSS), biochemical oxygen demand (BOD), total phosphorous (TP), and total Kjeldahl Nitrogen (TKN) of 120 mg/L, 110 mg/L, 4 mg/L, and 20 mg/L respectively (Table 2.1). The average strength wastewater has a TSS, BOD, TP, and TKN of 210 mg/L, 190 mg/L, 7 mg/L, and 40 mg/L respectively (Table 2.1). The high strength wastewater has an TSS, BOD, TP, and TKN of 400 mg/L, 350 mg/L, 12 mg/L, and 70 mg/L respectively (Table 2.1). The average strength had an increase of 1.8 times for TSS and TP as compared to the low strength wastewater. The average strength also had an increase of 1.7 times for BOD as compared to the low strength wastewater and a 2 times increase for TKN as compared to the low strength wastewater. The high strength had an increase of 1.9 times for TSS and TP as compared to the low strength wastewater. The high strength also had an increase of 1.8 times for BOD and TKN as compared to the low strength wastewater and a 1.7 times increase for TP as compared to the average strength wastewater. Between the low and high strength wastewaters, the high strength wastewater has an increase of 3.6 times for TSS, 3.2 times for BOD, 3 times for TP, and 3.5 times for TKN. The effluent requirements for TSS and BOD are based upon national standards for wastewater discharge (National Archives and Records Administration, 2012). The effluent standards for TP, TKN, total coliform were set to the effluent standards for an actual WWTP in the arid southwestern United States. The TP level is also standard among National Pollution Discharge Elimination System (NPDES) permits requiring nutrient removal (U.S. EPA, 2010c). The peak flow factors are also shown in Table 2.1. The dual media filters and UV (both low pressure high output UV and medium pressure high output UV) have a higher peak flow factor due to backwash off the dual media filters. While the processes were designed including the peak flows, only the average flow was used to calculate energy usage.

**Table 2.1 - Wastewater Characteristics and Effluent Criteria Partially Reproduced from (Metcalf and Eddy, 2003)**

| Contaminant | Influent Low Strength | Influent Average Strength | Influent High Strength | Effluent | Units |
|---|---|---|---|---|---|
| Influent Flow | 100 | 100 | 100 | N/A | MGD |
| Total Suspended Solids (TSS) | 120 | 210 | 400 | ≤30 | mg/L |
| Volatile Portion of TSS | 80 | 80 | 80 | N/A | % |
| 5 Day Biochemical Oxygen Demand (BOD) | 110 | 190 | 350 | ≤30 | mg/L |
| Total Phosphorous (TP) | 4 | 7 | 12 | ≤0.2 | mg/L |
| Total Kjeldahl Nitrogen (TKN)/Ammonia | 20 | 40 | 70 | ≤0.5 | mg/L |
| Total Coliform | $10^6$-$10^8$ | $10^7$-$10^9$ | $10^7$-$10^{10}$ | ≤200 | MPN/100 mL |
| Peak Flow Factor for All Liquid Units but Dual Media Filters, low pressure high output UV, and medium pressure high output UV | 1.5 | 1.5 | 1.5 | N/A | N/A |
| Peak Flow Factor for Dual Media Filters, low pressure high output UV, and medium pressure high output UV | 1.9 | 1.9 | 1.9 | N/A | N/A |

## 2.2 Physical Treatment Units Design

Table 2.2 shows major design criteria for the physical treatment processes. The number of units in operation for the average design flow is shown after the unit name in parenthesis in Table 2.2. The design of all units followed typical design parameters as established in references such as Metcalf and Eddy (2003), GLUMRB (2004), WEF (2010a), WEF (2010b).

**Table 2.2 - Physical Treatment Units Design**

| | Parameter | Value | Units | Reference |
|---|---|---|---|---|
| **Bar Racks (2abc)** | Bar Spacing/Bar Width | 0.5 | in | [3] |
| | Headloss at Average Flow | 0.23 | ft | [3,7] |
| | Headloss at Average Flow (Clogged) | 1.53 | ft | [3,7] |
| **Aerated Grit Chambers (4abc)** | Length, Width, Depth | 58, 16, and 16 | ft | [3,7] |
| | Hydraulic Retention Time (HRT) at Peak Flow | 4.3 | min | [3,4,7] |
| | Airflow Requirements (Increases with strength) | 3.75 – 7.5 | $ft^3/ft$-min | [3,7] |
| **Primary Clarifiers (10abc)** | Diameter | 120 | ft | [3] |
| | Sidewater Depth | 12.5 | ft | [3] |
| | Overflow Rate (OFR) at Average Flow | 884 | $gal/ft^2$-d | [3] |
| | Weir Loading at Average Flow | 26,500 | gal/ft-d | [3] |
| **Secondary Clarifiers (10 ab, 12c)** | Diameter | 140 | ft | [6] |
| | Sidewater Depth | 14 | ft | [6] |
| | Secondary Clarifier OFR at Average Flow | 650 | $gal/ft^2$-d | [7] |
| | Secondary Clarifier Solids Loading Rate (SLR) at Average Flow (Increases with strength) | 1.02 - 1.19 | $lb/ft^2$-hr | [7] |
| **Dual Media Filters (28abc)** | Filtration Rate With One Filter Out of Service at Peak Flow | 5 | $gpm/ft^2$ | [4] |
| | Filter Backwash Rate With Air Scour | 8.9 | $gpm/ft^2$ | [3,7] |
| | Filter Air Scour Flow Rate | 4 | $ft^3/ft^2$-min | [3,7] |
| **UV (5abc)** | Design UV Dosage | 30 | $mW{\cdot}s/cm^2$ | [4] |
| | Low Pressure High Output Lamps | 352 | # of Lamps | N/A |
| | Medium Pressure High Output Lamps | 160 | # of Lamps | N/A |
| **Gravity Thickeners (1a, 2b, 3c)** | Diameter | 65 | ft | [8] |
| | Sidewater Depth | 10.5 | ft | [8] |
| | SLR (Increases with strength) | 0.96 - 1.2 | $lb/ft^2$-h | [1,3,8] |
| **DAFTS (1a, 2b, 3c)** | Diameter | 60 | ft | N/A |
| | Sidewater Depth | 10 | ft | N/A |
| | Recycle Rate | 300% | N/A | [2] |
| | SLR (Increases with strength) | 0.5 - 0.8 | $lb/ft^2$-h | [1,3,8] |
| | DAFT Air to Solids Ratio | 0.034 | Unitless | [3] |
| **Centrifuges (1a, 2b, 4c)** | SLR for Combined Sludge | 4,200 | lb/hr | [5] |
| a = low strength, b = average strength, c = high strength | | | | |
| [1] = (U.S. EPA, 1979), [2] = (WEF, 1982), [3] = (Metcalf and Eddy, 2003), [4] = (GLUMRB, 2004), [5]= (Sieger, et al., 2006), [6] = (WEF, 2005), [7] = (WEF, 2010a), [8] = (WEF, 2010b) | | | | |

## 2.3 Bar Racks

The bar racks channel design utilized the Manning's equation (Mays, 2005) with a Manning's number of 0.015 for concrete. Headloss through the bar rack was determined using two equations for a clean screen: the Kirschmer's equation (WEF, 2010a) and the headloss equation for bar racks (Metcalf and Eddy, 2003) with the equation providing higher headloss being utilized. When calculating the headloss for a 50% clogged screen, only the headloss equation for bar racks can be used (Metcalf and Eddy, 2003). At average flow for a clean screen, both headloss equations provided nearly identical headloss values. At peak flow for a clean screen, the Kirschmer equation provided a higher headloss value. For the Kirschmer equation a bar rack angle of 80° angle and a K value of 2.42 were used while the headloss equation had C values of 0.7 and 0.6 for clean and clogged bar racks. Pertinent design parameters are shown in Table 2.2.

Energy consumption for the bar racks is dictated by the size of the motor driving the rake and rake cleaning frequencies. The increase in timing was assumed to be linear, which created a linear relationship between wastewater strength and energy consumption. Cleaning frequencies of 20, 15, and 10 minutes were assumed for the low, average, and high strength wastewaters, respectively (WEF, 2008).

## 2.4 Aerated Grit Chambers

Airflow was assumed based upon typical values as shown in Table 2.2. The airflow requirements in Table 2.2 were assumed to increase with increasing wastewater strength. The hydraulic retention time (HRT) falls within typical recommended design parameters as shown in Table 2.2.

The energy consuming units for the grit chambers are the blowers. Blower energy consumption in the grit chambers was determined as per (U.S. EPA, 1989; WEF, 2010a). The blower energy equation is defined as follows:

$$\mathrm{WP} = \left(4.28 \times \frac{10^{-4} q_s T_a}{e}\right)[(P_d/P_b)^{0.283} - 1]$$

Where WP = wire power consumption (HP, multiply by 0.746 for kW), $q_s$ = airflow rate (scfm), $T_a$ = intake temperature (°R), e = combined efficiency, $P_d$ = blower discharge pressure (psia), $P_b$ = blower intake pressure (always assumed as 14.7 psia for all blowers).

## 2.5 Primary Clarifiers

The main equations used in the design were the HRT, overflow rate (OFR), and weir loading rate as shown in Table 2.2. The OFR at average flow is slightly above typical range given in Metcalf and Eddy (2003). TSS and BOD removals in the primary clarifiers were assumed to follow (Metcalf and Eddy, 2003). The sludge solids concentration was assumed to increase with wastewater strength and varied between 4 and 5%.

Energy consumers for the primary clarifiers include sludge pumping, and torque to power the rake arms. The brake horsepower (BHP) equation (Jones, et al., 2008) was used to compute the energy requirements of the pumps. The BHP equation is as follows:

$$\mathrm{BHP} = \frac{\mathrm{QH}}{3960\mathrm{e}}$$

Where BHP = brake horsepower (HP multiply by 0.746 for kW), Q = flow rate (gpm), H = pump head (ft), and e = efficiency. The time between pumping cycles was assumed as 20, 15, and 10 minutes for the low, average, and high strength cases, respectively. The pumping time was assumed as three minutes for all strength cases. The energy required to drive the rake arms (WEF, 1982; WEF, 2005) was calculated as follows:

$$\mathrm{P} = \mathrm{Wr}^2\omega/550\mathrm{e}$$

Where P = power required for rake arms (HP multiply by 0.746 for kW), W = arm loading factor (lb/ft), r = radius of tank (ft), ω = angular velocity (rad/s), and e = efficiency.

## 2.6 Secondary Clarifiers

For the secondary clarifier design, the return activated sludge (RAS) concentration was assumed as 8,000 mg/L and typical RAS ratios varied between 0.6Q and 0.7Q based upon wastewater strength (Metcalf and Eddy, 2003). Effluent TSS was assumed to be 5, 10, and 20 mg/L for the low, average, and high strength wastewaters. The waste activated sludge (WAS) flow rate was calculated using the SRT equation (Metcalf and Eddy, 2003). Design parameters included OFR and solids loading rate (SLR) (WEF, 2010a) as shown in Table 2.2.

Energy consuming units for the secondary clarifiers include RAS pumping, WAS pumping, and torque to power the rake arms.

## 2.7 Dual Media Filtration

Dual media filters consisting of 1.84 ft of anthracite and 1.18 ft of anthracite with effective sizes of 1.29 mm and 0.49 mm, respectively were designed. The filtration rate at peak flow is shown in Table 2.2. Cleanbed headloss was predicted using the Rose equation (Metcalf and Eddy, 2003) with a headloss of 1.5 ft at the peak filtration rate. The Rajagopalan and Tien model was used to predict TSS removal (MWH, 2005) in lieu of pilot study data. Solids storage capability data (WEF, 2010a) were used to predict clogged headloss buildup (Metcalf and Eddy, 2003). Backwashing frequencies were estimated at 68, 42, and 32 hours for the low, average, and high strength wastewaters.

Energy consuming units for the dual media filters include the filter influent pumps (FIPS), backwash pumps, and backwash blowers. The backwash pumps were assumed to run for eight minutes (WEF, 2008) while the air scour cycle was assumed to run for four minutes (Chen, et al., 2003).

## 2.8 **UV Disinfection**

Two types of UV disinfection were evaluated: low pressure high output (LPHO), and medium pressure high output (MPHO). The dosage was estimated using the point source summation (PSS) method (U.S. EPA, 1986; WEF, 2010a) in lieu of bioassay data with the Emerick and Darby model used to predict effluent coliform values (WEF, 2010a). The transmittance was assumed as 78, 72, and 68% for the low, average, and high strength cases, respectively. The design dosage and number of lamps for LPHO and MPHO are shown in Table 2.2.

The main energy consuming units for UV are the lamps. The maximum input of the lamps is 250 W and 3,200 W for LPHO and MPHO (Trojan UV, 2007; Trojan UV, 2008). Turndown capabilities were 60% and 30% for LPHO and MPHO (Trojan UV, 2007; Trojan UV, 2008).

## 2.9 **Gravity Thickeners**

Gravity thickeners were designed as per (U.S. EPA, 1979; Metcalf and Eddy, 2003; WEF, 2010b) and the design is shown in Table 2.2. The thickened sludge was assumed to be 8, 9%, and 10% solids for the low, average, and high wastewater strengths, respectively. The solids capture efficiency was assumed to be 90% (Qasim, 1999). The gravity thickeners each had a surface area of approximately 3300 $ft^2$ and one, two, and three gravity thickeners were required for the low, average, and high strength wastewaters. Solids loading in the gravity thickeners were 29, 52, and 99 tons of solids/day entering the process for the low, average and high strength wastewaters. Energy consuming units for the gravity thickeners include the sludge pumps, overflow pumps, and torque to drive the rake arms.

## 2.10 **Dissolved Air Flotation Thickeners (DAFTs)**

The main design parameters for the DAFTs are the air to solids ratio, recycle rate, and SLR (Metcalf and Eddy, 2003) as shown in Table 2.2. The solids capture efficiency was assumed to be 95% (WEF, 2010a). The thickened sludge percent solids were assumed to be 4% for all three wastewater strengths. The DAFTs each had a surface area of approximately 2800 $ft^2$ and required one, two, and three DAFTs for the low, average, and high strength wastewaters. The DAFT loadings were 14, 27, and 53 tons of solids/day for the low, average and high strength wastewaters.

Energy consuming units for the DAFTS include recycle pumps, air compressors, sludge pumps, subnatant pumps, and torque to drive the rake arms.

## 2.11 **Centrifuge Dewatering**

The solids concentration of the combined sludge before centrifugation was 6, 6.5, and 7% solids for the low, average, and strength wastewater, respectively. The main design equations for the centrifuges are the volumetric and solids loading criteria as presented in Sieger, et al. (2006) and are shown in Table 2.2. The cake solids concentration was assumed to be 20% with 95% solids capture efficiency (Metcalf and Eddy, 2003).

Energy consumption in the centrifuges comes from feed acceleration and cake conveyance as presented in Maloney, et al. (2008). Feed acceleration energy requirements are as follows:

$$P_{acc}=5.984\times10^{-10}SGQ(\Omega r_p)^2/e$$

Where $P_{acc}$ = feed acceleration energy (HP multiply by 0.746 for kW), SG = specific gravity, Q = feed flow rate (gpm), $\Omega$ = speed (rpm), $r_p$ = pool radius (in), and e = efficiency. The cake conveyance energy requirements are as follows:

$$P_{con}=1.587\times10^{-5}T\Delta/e$$

Where $P_{con}$ = cake conveyance energy requirements, T = torque (lb-in), $\Delta$ = differential speed (rpm), and e = efficiency.

## 2.12 **Biochemical Treatment Processes**

Table 2.3 shows the microbiological parameters used in the design of the aeration basins. The HRT of the anoxic and anaerobic zones was one hour and the HRT of the aerobic zone is four hours (WEF, 2010c). The aeration basins had 73, 127, 232 mg/L BOD and 20, 40, 70 mg/L TKN entering the process after primary clarification. This amounts to 61,200, 106,600, 197,400 lb BOD and 16,700, 33,700, 59,500 lb TKN entering the aeration basins after primary clarification. The design of the aeration basins was based around plug flow reactor kinetics (Rittman, et al., 2000), biological phosphorous removal assumptions published in Metcalf and Eddy (2002), and BioWin (EnviroSim Associates LTD., 2003) modeling. VFAs for the PAOs are assumed to be generated in the primary clarifiers. For the high strength case, the TKN and BOD were high enough such that the hydraulic capacity of the aeration basins decreased and an additional two aeration basins had to be brought in operation. SRTS were 20, 10, and 8 days for low, average, and high strength cases, respectively (WEF, 2010a). Total phosphorous left after biological activity was 0.73, 2.02, and 0.81 mg/L for the low, average, and high strength cases, respectively. The high strength is lower than the average case due to influent entering the anaerobic zone. Alum was assumed to be added in the aeration basins for phosphorous polishing. Full nitrification and BOD removal occurred in all three cases.

Energy consuming units for the aeration basins include the blowers, chemical pumps, and mixers for the anaerobic/anoxic zones.

**Table 2.3 - Microbial Parameters at 20°C for the Design of the Aeration Basins**

| Parameter | BOD Microbes | Nitritation Microbes | Nitratation Microbes | Phosphorous Accumulating Organisms (PAOs) | Denitrification Microbes |
|---|---|---|---|---|---|
| Half-Velocity constant, K (mg donor/L) | 10 [1] | 1 [1] | 1.3 [1] | 1 [1] | 12.6 [1] |
| Yield, Y (g VSS/g donor) | 0.4 [1] | 0.33 [1] | 0.083 [1] | 0.3 [2] | 0.26 [1] |
| Maximum Specific Growth Rate, $\mu_m$ (g VSS/g VSS -d) | 9 [1] | 0.76 [1] | 0.81 [1] | 0.95 [2,3] | 3.12 [1] |
| Endogenous Decay Coefficient, $k_d$ (g VSS/g VSS-d) | 0.15 [1] | 0.11 [1] | 0.11 [1] | 0.04 [3] | 0.05 [1] |
| $f_d$ | 0.8 [1] | | | | |
| [1] = (Rittman, et al., 2000), [2] = (Metcalf and Eddy, 2003), [3] = (WEF, 2010c) | | | | | |

# 3 Results and Discussion

Motor size requirements for individual unit processes in the plant are shown in APPENDIX A. Typical efficiency values and equations from literature (Metcalf and Eddy, 2003; U.S. EPA, 1989; WEF, 2005; WEF, 2010a; WEF, 2010b) and various manufacturer literature were used to calculate the motor size for each energy consuming unit (APPENDIX A). The efficiency values given are combined motor and equipment efficiency also known as "wire to water". For the rake arms for the primary and secondary clarifiers, and the gravity thickener rake arms the efficiency was assumed as 75%. The blowers needed for the aeration basins require the largest motors with energy requirements of 1,878, 3,526, and 5,032 HP total for the low, average and high strength wastewaters, respectively. The second largest motors were the MPHO UV lamps which require 1,030, 1,125, and 1,420 HP total for the low, average and high strength wastewaters, respectively. This amounts to 55, 32, 28% of the aeration basin blower requirements for the low, average, and high strength wastewaters. After MPHO UV, the backwash pumps for the dual media filters have the largest motors with requirements of 702 HP total. This amounts to 37, 20, 14% of the aeration basin blower requirements for the low, average, and high strength wastewaters. For comparison, preliminary and primary treatment combined (bar racks, aerated grit chambers, and primary clarifiers) have motor requirements of 62, 79, and 95 HP for the low, average and high strength wastewaters, respectively. This amounts to only 3, 2, and 2% of the aeration basin blower requirements for the low, average, and high strength wastewaters. Solids processing (gravity thickeners, DAFTs, and centrifuges) combined require 136, 245, and 442 HP for the low, average, and high strength wastewaters, respectively. Solids processing amounts to only 9% of the aeration basin blower requirements for the low, average, and high strength wastewaters. Secondary clarification requires 569, 563.2, and 689 HP for the low, average, and high strength wastewaters. Secondary clarification requires 30, 16, and 15% of the aeration basin blower requirements for the low, average, and high strength wastewaters.

The energy consumption for each energy consuming unit was found by determining energy requirements in kWh/day and dividing by the 100 MGD flow which yields kWh/MGal (Table 2.4). The energy consumption for secondary treatment to remove BOD and TKN on a mass basis was computed by dividing the energy for each energy consuming unit (Table 2.4) by the BOD and TKN entering the secondary treatment process (Table 2.5). The energy consumption for sludge processing on a mass basis was computed by dividing the energy for each energy consuming unit (Table 2.4) by the tons of sludge entering the process (Table 2.6). It should be noted that only the energy associated with treatment was computed in this research. Other energy requirements such as those for building heating/cooling, lighting, chemical generation, and transport energy were not computed. Two types of estimates from the literature were used for comparison: volumetric based estimates (Table 2.7) and mass based estimates (Table 2.8).

**Table 2.4 – Energy Consumption per Unit Operation for a 100 MGD Wastewater Treatment Plant Treating Different Wastewater Strengths**

| | | kWh/MGal | | |
|---|---|---|---|---|
| | **Component** | **Low Strength** | **Average Strength** | **High Strength** |
| **Bar Racks** | Rakes | 0.07 | 0.09 | 0.13 |
| **Grit Chambers** | Blowers | 4.10 | 6.29 | 8.21 |
| **Primary Clarifiers** | Sludge Pumping | 0.86 | 1.17 | 1.85 |
| | Torque | 0.46 | 0.47 | 0.53 |
| | Total | 1.32 | 1.64 | 2.37 |
| **Aeration Basins** | Blowers | 336.2 | 631.3 | 1,081 |
| | Chemical Pumps | 0.54 | 1.24 | 1.61 |
| | Mixers | 104.8 | 104.8 | 125.8 |
| | Total | 441.5 | 737.3 | 1,208.4 |
| **Secondary Clarifiers** | RAS | 100.3 | 101.8 | 160.5 |
| | Torque | 0.7 | 0.8 | 0.80 |
| | WAS | 1 | 2 | 3.8 |
| | Total | 102 | 104.6 | 165.1 |
| **Dual Media Filters** | Filter Influent Pump Station (FIPS) | 125.7 | 125.7 | 125.7 |
| | Backwash Pump Energy | 1.7 | 2.9 | 3.8 |
| | Backwash Blower Energy | 0.7 | 1.2 | 2.0 |
| | Total | 128.1 | 130.2 | 133.1 |
| **UV** | MPHO UV | 184.3 | 201.6 | 254.4 |
| **Gravity Thickeners** | Rake Arm | 0.12 | 0.25 | 0.37 |
| | Overflow Pumps | 0.16 | 0.31 | 0.63 |
| | Sludge Pumps | 0.22 | 0.34 | 0.58 |
| | Total | 0.5 | 0.9 | 1.58 |
| **DAFTS** | Recycle Pumps | 9.3 | 18.5 | 35.4 |
| | Rake Arms | $2 \times 10^{-2}$ | $4 \times 10^{-2}$ | $7 \times 10^{-2}$ |
| | Sludge Pumps | 0.2 | 0.3 | 0.7 |
| | Air Compressors | $8 \times 10^{-2}$ | 0.2 | 0.3 |
| | Overflow Pumps | 0.7 | 1.3 | 2.5 |
| | Total | 10.2 | 20.3 | 39.0 |
| **Centrifuges** | Feed Acceleration | 14.4 | 24.5 | 43.9 |
| | Cake Conveyance | 1.7 | 3.3 | 5.0 |
| | Total | 16.1 | 27.8 | 48.9 |
| | Total Energy | 888.3 | 1,230.8 | 1,861.1 |

**Table 2.5 - Secondary Treatment Energy Requirements per Pound of BOD or TKN Removed**

| | | kWh/lb BOD (kWh/lb TKN) | | |
|---|---|---|---|---|
| | **Component** | **Low Strength** | **Average Strength** | **High Strength** |
| **Aeration Basins** | **Blowers** | 0.55 (2.02) | 0.59 (1.87) | 0.55 (1.82) |
| | **Chemical Pumps** | 8.82E-04 (3.24E-03) | 1.16E-03 (3.68E-03) | 8.16E-04 (2.70E-03) |
| | **Mixers** | 0.17 (0.63) | 0.1 (0.31) | 0.06 (0.21) |
| | **Total** | 0.72 (2.65) | 0.69 (2.19) | 0.61 (2.03) |
| **Secondary Clarifiers** | **RAS** | 0.16 (0.60) | 0.1 (0.29) | 0.08 (0.27) |
| | **Torque** | 1.18E-03 (4.32E-03) | 7.13E-04 (2.26E-03) | 4.05E-04 (1.34E-03) |
| | **WAS** | 1.63E-03 (6.00E-03) | 1.88E-03 (5.94E-03) | 1.93E-03 (6.38E-03) |
| | **Total** | 0.17 (0.61) | 0.1 (0.31) | 0.08 (0.28) |
| **Secondary Treatment Total** | | 0.89 (3.26) | 0.79 (2.49) | 0.69 (2.31) |

**Table 2.6 - Solids Processing Energy Requirements per Ton of Solids**

| | | kWh/ton of solids | | |
|---|---|---|---|---|
| | **Component** | **Low Strength** | **Average Strength** | **High Strength** |
| **Gravity Thickeners** | **Rake Arm** | 0.42 | 0.48 | 0.37 |
| | **Overflow Pumps** | 0.55 | 0.60 | 0.64 |
| | **Sludge Pumps** | 0.75 | 0.66 | 0.59 |
| | **Total** | 1.72 | 1.74 | 1.6 |
| **DAFTS** | **Recycle Pumps** | 67.42 | 67.42 | 67.42 |
| | **Rake Arm** | 0.16 | 0.16 | 0.13 |
| | **Subnate Pumps** | 1.24 | 1.24 | 1.24 |
| | **Air Compressors** | 0.59 | 0.59 | 0.59 |
| | **Overflow Pumps** | 4.84 | 4.84 | 4.84 |
| | **Total** | 74.25 | 74.25 | 74.22 |
| **Centrifuges** | **Feed Acceleration** | 33.4 | 31.0 | 29.1 |
| | **Cake Conveyance** | 3.9 | 4.2 | 3.3 |
| | **Total** | 37.3 | 35.2 | 32.4 |

**Table 2.7 – Energy Consumption Estimates on a Volumetric Basis for Wastewater Treatment**

| | Volumetric Literature Estimates (kWh/MGal) | Reference |
|---|---|---|
| **Bar Racks** | 0.1 | [1] |
| | 0.11 | [4] |
| **Grit Chambers** | 6.58 | [1] |
| | 12.00 | [4] |
| **Primary Clarifiers** | 4.38 | [1] |
| | 15.5 | [4] |
| **Aeration Basins** | 532 for aeration (BOD removal only) | [4] |
| | 338 for aeration (ammonia removal only) | [4] |
| | 870 for aeration (BOD and ammonia removal) | [4] |
| | 1191.8 for aeration (BOD and ammonia removal) | [1] |
| | 2400 | [5] |
| **Secondary Clarifiers** | 24.7 | [1] |
| | 53.1 | [4] |
| | 118.3 | [2] |
| | 256.7 | [2] |
| | 302.9 | [2] |
| **Dual Media Filters** | 138.1 | [1] |
| | 100.1 | [4] |
| **UV** | 253.3 | [3] |
| | 250 | [2] |
| **Gravity Thickeners** | 1.38 | [4] |
| | 0.31 for rake arms only | (U.S. EPA, 1978) |
| **DAFTS** | 49.3 per daft | [1] |
| | 132 | [4] |
| **Centrifuges** | 10 to 75 | [4] |
| [1] = (U.S. EPA, 1978), [2] = (Pacific Gas and Electric Company, 2003), [3] = (URS Corporation, 2004), [4] = (WEF, 2009), [5] = (Foley, et al., 2010) | | |

**Table 2.8 - Energy Consumption Estimates on a Mass Basis for Wastewater Treatment**

| | Mass Based Literature Estimates | Reference |
|---|---|---|
| Aeration Basins | 0.6 (BOD removal only) | [2] |
| | 1.9 (BOD removal only) | [2] |
| | 2.6 (BOD removal only) | [2] |
| | 0.9 (BOD removal and nitrification/denitrification) | [2] |
| | 2.2 (BOD removal and nitrification/denitrification) | [2] |
| DAFTS | 52 to 75 | [1] |
| Centrifuges | 36 | [1] |
| [1] = (WEF, 1982), [2] = (Pacific Gas and Electric Company, 2003) | | |

## 3.1 Estimated Energy Consumption for Preliminary and Primary Treatment Units

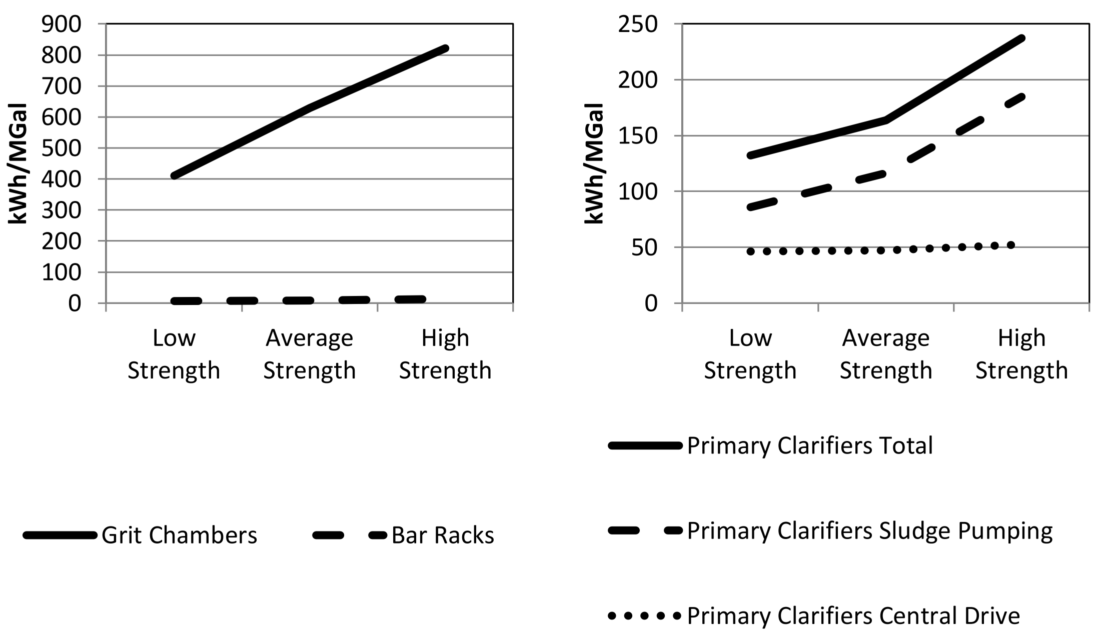


**Figure 2.3 (a) Energy Requirements For Bar Racks, and Grit Chambers, (b) Primary Clarifiers Energy Requirements**

The bar racks represent a very small energy consumption for the WWTP. The bar racks require 0.07, 0.09, and 0.13 kWh/MGal for the low, average, and high strength cases [Table 2.4 and Figure 2.3 (a)]. The energy requirements are less than 0.01% of the total WWTP energy consumption for all three wastewater cases. Stronger wastewaters often contain more screenings. Therefore, the rake needs to be activated more times. Energy consumption increased by 25% between the low and average strength wastewaters, 36% between the average and high strength wastewaters, and 60% between the low and high strength wastewaters (Table 2.4). Reported estimates of bar rack energy consumption are 0.11 kWh/MGal (WEF, 2009) and 0.10 kWh/MGal (U.S. EPA, 1978) which are close to the estimates found in this research.

The grit chambers require the most energy of the preliminary and primary treatment units. The grit chambers require about 75, 78, and 77% of the total preliminary and primary treatment units energy consumption. Requirements were 4.10, 6.29, and 8.21 kWh/MGal for the low, average, and high strength wastewaters, respectively [Table 2.4 and Figure 2.3 (a)]. Similar to the bar racks, the grit chambers had a linear increase in energy consumption with wastewater strength as a result of higher solids concentration present [Figure 2.3 (a)]. Energy consumption increased by 42% between the low and average strength wastewaters, 27% between the average and high strength wastewaters, and 67% between the low and high strength wastewaters (Table 2.4). Reported literature values for energy consumption in aerated grit chambers include 12.00 kWh/MGal (WEF, 2009) and 6.58 kWh/MGal (U.S. EPA, 1978). In this research, the energy consumption estimate for grit chambers treating medium strength wastewater is close to the value reported by EPA (1978), but it is about half of that reported recently by WEF (2009). While the methodology for estimation in the WEF (2009) was not given, the difference is most likely due to a higher design HRT or a higher airflow rate.

The primary clarifiers required 1.32, 1.64, and 2.37 kWh/MGal [Table 2.4 and

Figure 2.3 (b)]. The primary clarifiers' central drive torque varies very little with wastewater strength [Figure 2.3 (b)]. Energy consumption in primary clarifiers is mainly dictated by the size of the primary sludge pumps. The largest increase in energy consumption occurs between the average and high strength wastewaters. Energy consumption increased by 22% between the low and average strength wastewaters, 36% between the average and high strength wastewaters, and 57% between the low and high strength wastewaters (Table 2.4). Reported energy consumption of primary clarification are 4.38 kWh/MGal (U.S. EPA, 1978) and 15.50 kWh/MGal (WEF, 2009). Differences between the estimates in this article and (U.S. EPA, 1978) and (WEF, 2009) can most likely be explained by this study having a lower TDH for the sludge than the other estimates. No assumptions of TDH were stated in the other estimates.

## 3.2 Secondary Treatment Energy

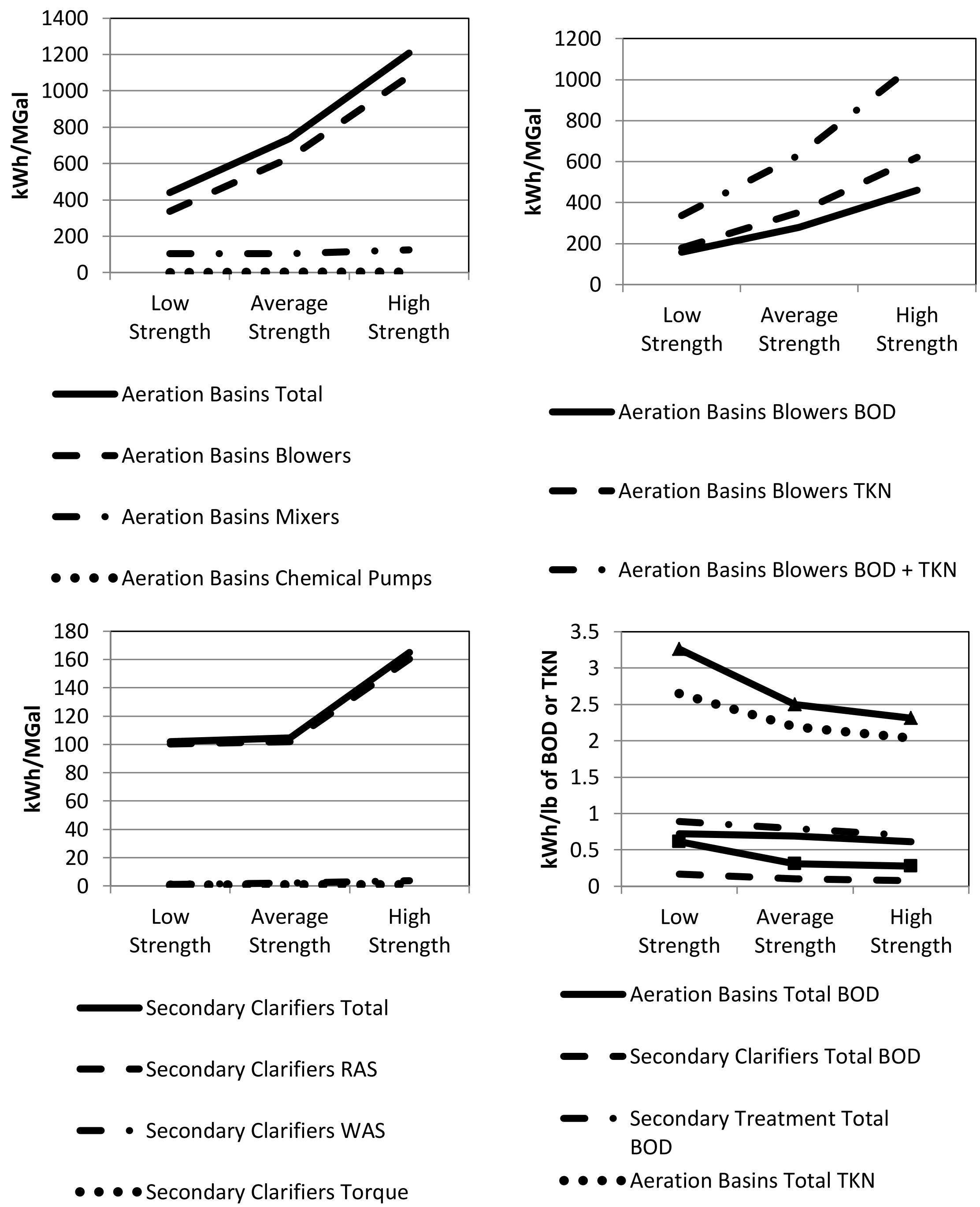


**Figure 2.4 (a) Energy Requirements for Aeration Basins, (b) Aeration Basins Blowers Energy requirements for BOD and TKN Removal Based Upon Airflow Requirements, (c) Energy Requirements for Secondary Clarifiers, (d) Secondary Treatment Energy Requirements for BOD and TKN Removal on a Mass Basis**

The aeration basins are the largest energy consuming units in the WWTP. On a volumetric basis, the aeration basins require a total of 441.5, 737.3, and 1,208.4 kWh/MGal for the low, average, and high strength wastewaters [Table 2.4 and Figure 2.4 (a)] on a volumetric basis. The aeration basin blower requirements for BOD and TKN removal [Figure 2.4 (b)] were calculated using oxygen demand requirement equations (Rittman, et al., 2000). For BOD removal, the aeration basin blowers require 158.2, 279.1, and 460.6 kWh/MGal for the low, average, and high strength wastewaters. For ammonia removal, the aeration basin blowers require 178.0, 352.2, and 620.4 kWh/MGal for the low, average, and high strength wastewaters. The total aeration basin energy consumption represents 50%, 61%, and 65% of the total energy consumption for the WWTP, respectively on a volumetric basis. Increasing TKN has the largest impact on energy consumption for aeration with increasing BOD having a smaller impact [Figure 2.4 (b)]. Total energy consumption on a volumetric basis increased by 50% between the low and average strength wastewaters, 48% between the average and high strength wastewaters, and 93% between the low and high strength wastewaters (Table 2.4).

In addition to calculating energy on a volumetric basis, the energy of the aeration basins was also calculated on a mass basis by dividing the energy requirements (Table 2.4) by the mass of BOD and TKN entering the aeration basins. For BOD removal, the secondary treatment process requires 0.89, 0.79, and 0.69 kWh/lb BOD for the low, average, and high strength wastewaters (Table 2.5 and Figure 2.4 (d)). For TKN removal, the secondary treatment process requires 3.26, 2.49, and 2.31 kWh/lb TKN for the low, average, and high strength wastewater [Table 2.5 and Figure 2.4 (d)]. The biggest decrease in energy consumption on a mass basis for both BOD and TKN was between the low and average strength wastewaters (Table 2.5). Total energy consumption on a mass basis for BOD decreased by 12% between the low and average strength wastewaters, 14% between the average and high strength wastewaters, and 25% between the low and high strength wastewaters (Table 2.5). Total energy consumption on a mass basis for TKN decreased by 27% between the low and average strength wastewaters, 8% between the average and high strength wastewaters, and 34% between the low and high strength wastewaters (Table 2.5).

Volumetric basis estimates reported 532.0 kWh/MGal for aeration (BOD removal only), 338.0 kWh/MGal (ammonia removal), 870.0 kWh/MGal for aeration (BOD and ammonia removal) only (WEF, 2009), 1191.8 kWh/MGal for aeration (BOD and ammonia removal) only (U.S. EPA, 1978), and approximately 2400.0 kWh/MGal (Foley, et al., 2010) total for partial BNR. The WEF (2009) estimate for BOD removal is higher than the estimate for the high strength wastewater, while the estimate for ammonia removal is similar to the average strength wastewater estimate. The WEF (2009) estimate for BOD and ammonia removal also falls within the estimates of average and high strength wastewaters in this study. The estimate in U.S. EPA (1978) assumed an influent BOD of 136 mg/L and influent ammonia of 25 mg/L. The BOD in U.S. EPA (1978) is closest to the average strength BOD in this research. The TKN in U.S. EPA (1978) is closest to the low strength TKN in this research. The U.S. EPA (1978) estimate also falls within the estimates of average and high strength wastewaters. The high energy consumption found in U.S. EPA (1978) can most likely be explained with advances in aeration technology since the article was published. There are reports that energy consumption as a function of wastewater flow decreases with increasing wastewater strength (Hernández-Sancho, et al., 2011; WEF, 2009). The estimate in Foley, et al. (2010) had a low flow (approximately 2.6 MGD) which is the most likely why the estimate in Foley, et al. (2010) is higher than the estimate in this research.

Mass basis estimates for secondary process requirements are 0.60, 1.90, and 2.60 kWh/lb BOD for BOD removal only (Pacific Gas and Electric Company, 2003). The estimates in Pacific Gas and Electric Company (2003) were computed using data from operating plants. The three estimates had influent BOD estimates of 175, 165, and 80 mg/L with flow rates of 11.5, 2.4, and 1.7 MGD. The 0.60 kWh/lb BOD has an influent BOD in between the average and high strength wastewater estimates in this research. While the estimate in this research is higher than the 0.60 kWh/lb BOD estimate (this research found 0.69 and 0.61 kWh/lb BOD for the average and high strength wastewaters) the estimate 0.60 kWh/lb BOD can be considered similar to this research. The 0.60 kWh/lb BOD estimate is for BOD removal only. In contrast, this research assumed nitrification and partial denitrification. When the estimates of the aeration basin blowers and total secondary clarifier energy consumption are added together the estimate of energy consumption for the secondary treatment process becomes 0.79 and 0.69 kWh/lb BOD for the average and high strength wastewaters. This situation also more closely mimics the 0.60 kWh/lb BOD estimate in terms of treatment and the minor difference in this case between the estimates in this study and the 0.60 kWh/lb BOD estimate can then be explained by this research assuming nitrification takes place in the aeration basins; without nitrification, the estimates would be even closer. The 1.90 and 2.60 kWh/lb BOD estimates are most likely higher than the estimates in this research due to their low flows. Other estimates for secondary process requirements were 0.90, and

2.20 kWh/lb BOD for BOD removal and nitrification/denitrification (Pacific Gas and Electric Company, 2003). These two estimates had influent BOD estimates of 180, and 85 mg/L with flow rates of 19.4, and 5.4 MGD. The influent TKN values for these estimates were not reported. The 0.90 kWh/lb BOD estimate has an influent BOD in between the average and high strength wastewater estimates in this research. The 0.90 kWh/lb BOD estimate is closest to the low strength estimate in this research in terms of energy consumption. The estimates of average and high strength wastewater in this research are both within 30% of the 0.90 kWh/lb BOD estimate. Differences between the estimates in this research and Pacific Gas and Electric Company (2003) can most likely be explained by a lack of mixed liquor recycle pumping that is required for denitrification. Finally, the 2.20 kWh/lb BOD estimate is most likely higher than the estimates in this research due to the low flow.

The energy consumption for the secondary clarifiers did not change significantly due to wastewater strength except for the high strength wastewater [Table 2.4 and Figure 2.4 (c)]. The small change in energy consumption between the low and average strength is due to increased WAS flow. The major increase came with the high strength wastewater where 12 aeration basins were required to treat the wastewater. The energy requirements for RAS and WAS were 101.3, 103.8, and 164.3 kWh/MGal for the low, average, and high strength wastewaters [Table 2.4 and Figure 2.4 (c)]. The total energy requirements were 102, 104.6, and 165.1 kWh/MGal for the low, average, and high strength wastewaters (Table 2.4 and Figure 2.4 (c)). Reported estimates of the secondary clarifiers are 24.7 kWh/MGal (U.S. EPA, 1978) and 53.1 kWh/MGal (WEF, 2009). Differences between the estimate in this study and (U.S. EPA, 1978) and (WEF, 2009) can most likely be explained by having a high RAS TDH in this study. No assumptions for RAS TDH are shown in either U.S. EPA (1978) or WEF (2009), however. Other estimates of secondary clarifier energy consumption are 118.3, 256.7, and 302.9 kWh/MGal with flow rates of 11.5, 2.4, and 1.7 MGD (Pacific Gas and Electric Company, 2003). The 118.3 kWh/MGal estimate is close to the estimates presented in this research. The 256.7 and 302.9 kWh/MGal estimates are likely higher than the estimates in this research due to their low flows.

### 3.3 Tertiary Treatment Energy

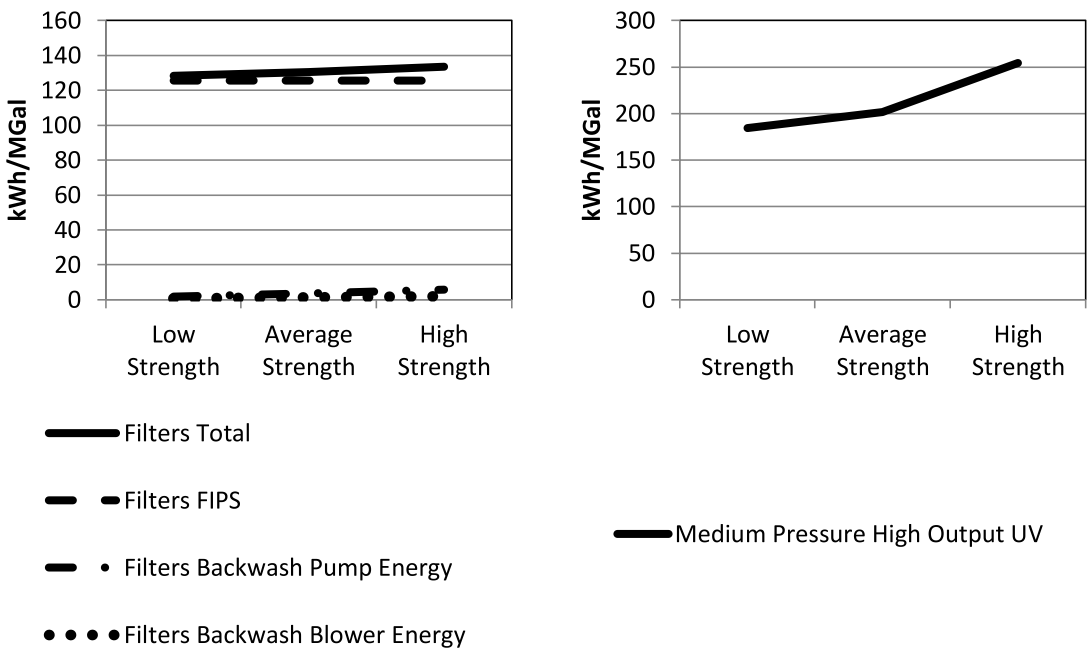


**Figure 2.5 (a) Energy Requirements for Dual Media Filters, (b) Energy Requirements for MPHO UV Disinfection**

The dual media filters (aside from FIPS) are a relatively small energy consuming unit requiring only 0.27%, 0.37%, and 0.39% of the total energy consumption for the entire WWTP for the low, average, and high strength wastewaters. Without FIPS, the dual media filters require 2.4, 4.5, and 7.4 kWh/MGal for the low, average, and high strength wastewaters. With FIPS, the dual media filters require 128.1, 130.2, and 133.1 kWh/MGal. FIPS is not dependent on wastewater characteristics, so energy consumption does not change for wastewater strength [Table 2.4 and Figure 2.5 (a)]. The backwashes, however, are affected significantly by wastewater strength [Table 2.4 and Figure 2.5 (a)]. Without FIPS, the energy consumption increased by 62% between the low and average strength wastewaters, 48% between the average and high strength wastewaters, and 103% between the low and high strength wastewaters (Table 2.4). With FIPS, the energy consumption increased by 1.7% between the low and average strength wastewaters, 2% between the average and high strength wastewaters, and 4% between the low and high strength wastewaters (Table 2.4). Other estimates including FIPS predicted 138.1 kWh/MGal (U.S. EPA, 1978) and 100.1 kWh/MGal (WEF, 2009). The estimate of this research is close to the one presented in U.S. EPA (1978). Differences between the estimate in this study and the estimate in WEF (2009) can most likely be explained by differences in TDH values.

MPHO UV disinfection is a significant energy consumer for the 100 MGD WWTP with UV disinfection requiring 184.3, 201.6, and 254.4 kWh/MGal for the low, average, and high wastewater strengths. This energy consumption amounts to 26%, 20%, and 16% of the total energy consumption for the low, average, and high wastewater strengths. MPHO UV disinfection also amounts to 42%, 27%, and 20% of the total aeration basin energy requirements in the WWTP. Disinfection is affected by transmittance which was assumed to decrease with increasing wastewater strength. As a result, the energy consumption increased significantly with regards to wastewater strength [Figure 2.5 (b)]. Energy consumption increased by 9% between the low and average strength wastewaters, 23% between the average and high strength wastewaters, and 32% between the low and high strength wastewaters (Table 2.4). Previous estimates for MPHO UV were 253.3 kWh/MGal (URS Corporation, 2004) and 250 kWh/MGal (Pacific Gas and Electric Company, 2003). These estimates are close to the estimate for the high strength wastewater in this research. The estimate in URS Corporation (2004) and the estimate in this study are close because the transmittances used were similar. High strength wastewater in this study was assumed to have a transmittance of 68% and (URS Corporation, 2004) had transmittances of around 65%. The estimate in Pacific Gas and Electric Company (2003) is close because a similar effluent coliform requirement (i.e. 200 MPN/100 mL) and high flow rate (43 MGD) were used.

## 3.4 Solids Processing Energy

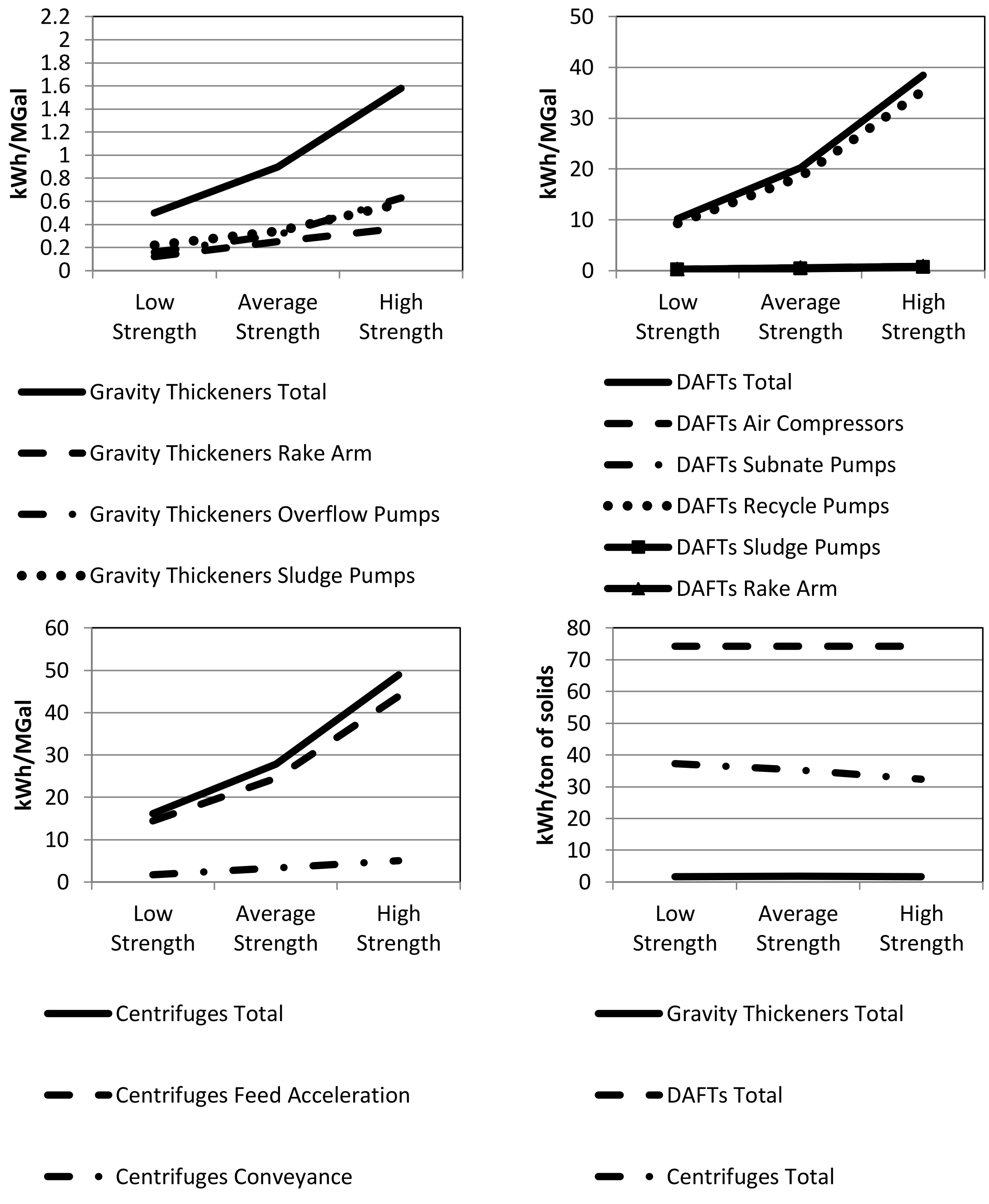


**Figure 2.6 (a) Energy Requirements for Gravity Thickeners, (b) Energy Requirements DAFTs, (c) Energy Requirements for Centrifuges**

Similar to secondary treatment, there are two different ways for reporting solids processing energy requirements: on a volumetric basis (kWh/MGal) or a mass basis (kWh/ton of sludge). To calculate the mass basis estimate of energy, the energy requirements (Table 2.4) were divided by the mass of sludge entering the solids process. Gravity thickening consumes a small amount of energy in the WWTP, requiring only 0.1% of the total energy consumption for the entire WWTP for the low, average, and high strength wastewaters. On a volumetric basis, total energy requirements for the gravity thickeners were only 0.5, 0.9, and 1.58 kWh/MGal for the low, average, and high strength wastewaters, respectively [Table 2.4 and Figure 2.6 (a)]. For the rake arms alone, energy requirements were 0.12, 0.25, and 0.37 kWh/MGal for the low, average, and high strength wastewaters, respectively [Table 2.4 and Figure 2.6 (a)]. Total energy requirements for gravity thickening are 38%, 55%, and 67% for the low, average, and high strength wastewaters of the total primary clarification energy requirements in the WWTP on a volumetric basis. The gravity thickeners are affected by wastewater strength as a result of increasing wastewater solids [Figure 2.6 (a)]. Energy consumption increased by 57% between the low and average strength wastewaters, 55% between the average and high strength wastewaters, and 104% between the low and high strength wastewaters (Table 2.4) on a volumetric basis. On a mass basis, the total energy requirements are 1.72, 1.74, and 1.6 kWh/ton for the gravity thickeners [Table 2.6 and Figure 2.6 (d)] for the low, average, and high strength wastewaters, respectively.

One volumetric estimate of gravity thickening energy consumption is 1.38 kWh/MGal (WEF, 2009) which is in between the average and high strength estimates in this research. Another volumetric estimate is for the rake arms only and predicts three gravity thickeners for a 100 MGD flow (U.S. EPA, 1978). The estimate in U.S. EPA

(1978) predicts 0.31 kWh/MGal for the rake arms only which is close to the estimate in this research for the high strength wastewater.

DAFTs also consume a relatively small amount of energy in the plant. On a volumetric basis, total energy requirements for the DAFTs were 10.2, 20.3, and 39.0 kWh/MGal for the low, average, and high strength wastewaters, respectively (Table 2.4 and Figure 2.6 (b)). Like the gravity thickeners, the DAFTs energy consumption is affected by the higher amounts of solids present in high strength wastewaters (Figure 2.6 (b)). Energy consumption increased by 66% between the low and average strength wastewaters, 63% between the average and high strength wastewaters, and 117% between the low and high strength wastewaters, on a volumetric basis (Table 2.4). On a mass basis, the total energy requirements are 74.25, 74.25, and 74.22 kWh/ton for the gravity thickeners (Table 2.6 and Figure 2.6 (d)). On a mass basis, there is not a significant difference in energy consumption between different wastewater strengths.

A previous reported volumetric estimate of DAFT energy consumption is 49.3 kWh/MGal/DAFT (U.S. EPA, 1978). The estimate in U.S. EPA (1978) provides a figure that gives an energy estimate for each DAFT. The estimate in U.S. EPA (1978) is based upon surface area of the DAFT which was taken as the surface area of one DAFT in this research. To compare the estimate in U.S. EPA (1978) to the estimates in this research, the estimate per DAFT must be multiplied by the number of operating DAFTs found in this research for each wastewater strength. Thus, the estimate in U.S. EPA (1978) provides estimates for low, average, and high strength wastewaters of 49.3, 98.6, 148.0 kWh/MGal for the DAFTs. The estimates in U.S. EPA (1978) is higher than the estimates in this research as U.S. EPA assumed an air to solids ratio of 0.20 while the estimate in this research uses 0.03. Another volumetric energy estimate for DAFTs has been reported as 132 kWh/MGal (WEF, 2009) which is three times higher than the estimate for the high strength wastewater in this study. While no assumptions are given in WEF (2009), the estimate is most likely higher due to having a higher air to solids ratio. A mass based estimate of DAFT energy consumption is 52 to 75 kWh/ton of solids (WEF, 1982). The estimates in this research for all three wastewater strengths fall within this range.

The centrifuges constitute a relatively small energy consuming processes in the WWTP. On a volumetric basis, energy requirements for the centrifuges were 16.1, 27.8, and 48.9 kWh/MGal, respectively. [Table 2.4 and Figure 2.6 (c)]. Energy consumption in the centrifuges is highly affected by wastewater strength as a result of increasing wastewater solids [Figure 2.6 (c)]. Energy consumption increased by 56.5% between the low and average strength wastewaters, 51.1% between the average and high strength wastewaters, and 100.4% between the low and high strength wastewaters on a volumetric basis (Table 2.4). On a mass basis, the total energy requirements are 37.3, 35.2, and 32.4 kWh/ton for the centrifuges [Table 2.6 and Figure 2.6 (d)]. A previous volumetric estimate of centrifuge energy is 10 to 75 kWh/MGal (WEF, 2009). The estimates in this research all fall within this range. A previous mass estimate of centrifuge energy predicts 36 kWh/ton (WEF, 1982). The estimate in WEF (1982) is similar to the estimates in this research.

Energy use in the many treatment categories is summarized in Figure 2.7. Treatment processes that do not require a significant amount of energy are preliminary and primary treatment, and sludge processing (Figure 2.7). Treatment processes that require a significant amount of energy are secondary treatment, and tertiary treatment (Figure 2.7). Secondary treatment requires 99, 104, and 128 times the energy of preliminary treatment for the low, average, and high strength wastewaters (Figure 2.7). Secondary treatment also requires 1.7, 2.5, and 3.5 times the energy of tertiary treatment for the low, average, and high strength wastewaters (Figure 2.7). Secondary treatment also requires 20, 17, and 15 times the energy of solids processing for the low, average, and high strength wastewaters (Figure 2.7). The next largest energy consumer tertiary treatment requires 57, 41, and 36 times the energy of preliminary treatment for the low, average, and high strength wastewaters (Figure 2.7). Tertiary treatment requires 12, 7, and 4 times the energy of solids processing for the low, average, and high strength wastewaters (Figure 2.7). The third largest energy consumer solids requires 5, 6, and 8 times the energy of preliminary treatment for low, average, and high strength wastewaters (Figure 2.7).

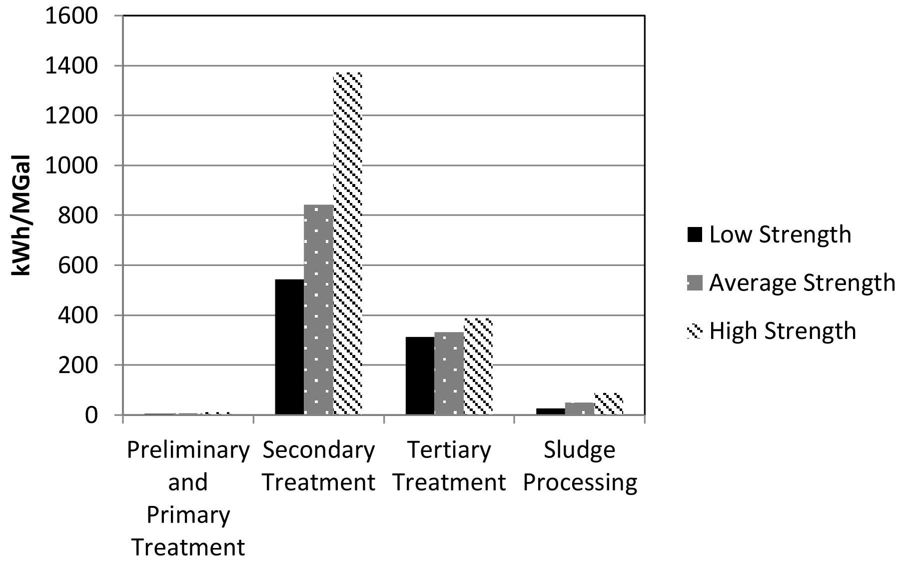


**Figure 2.7 – Comparison of Energy Consumption in Wastewater Treatment Plants for Different Treatment Levels**

## 4 Conclusions

While wastewater treatment is paramount for health and environmental protection the energy consumption associated with it should be scrutinized to conserve resources and minimize greenhouse gas generation. Evaluation of energy consumption in the various unit operations provided insight into areas to be targeted for conservation and/or process changes. The major objective of this research was to evaluate whether design and fundamental equations can be used to evaluate energy consumption in a WWTP. The energy use found in this study are comparable to existing reported studies in all cases, but primary clarification. There is, however, wide variation in the existing studies. Many factors may contribute to this variation including influent to a process and operating parameters can change energy estimates. In addition, a large amount of data available is from the 1970s and 1980s before the advent of more advanced technologies such as VFDs. The fundamental and design equations used to estimate energy consumption in this research are useful, but must be used carefully. Assumptions must be made carefully and verified against existing WWTPs to obtain useful results.

The other major objective of this research was to evaluate how wastewater strength affects energy consumption in a WWTP and identify treatment processes that are susceptible to wastewater strength. Overall, this research was able to identify treatment processes that are influenced by wastewater strength. In terms of overall impact to the WWTP total energy consumption, aeration basins, UV, DAFTs, and centrifuges are the most influenced by wastewater strength (Table 2.4 and Figure 2.7). In terms of percent increase in individual processes with increasing strength, DAFTs, centrifuges, gravity thickeners, and aeration basins are the most influenced by wastewater strength (Table 2.4 and Figure 2.7). Processes that were not significantly affected by wastewater strength included the bar racks, secondary clarifiers, and dual media filters (Table 2.4 and Figure 2.7). The aeration basins were affected by increasing wastewater strength as a result of increasing TKN. UV is susceptible to increasing wastewater strength due to decreasing transmittance. The DAFTs, gravity thickeners and centrifuges are affected by increasing wastewater strength as a result of increased solids. Bar racks are not susceptible to increasing wastewater strength, because they are such a small energy consuming process in the WWTP. Secondary clarifiers were not significantly affected by wastewater strength due to increased wasting. The dual media filters were not significantly affected by wastewater strength due to FIPS being the largest energy consumer.

Aeration, UV disinfection, and pumping should be targeted for energy reduction. Optimizing aeration and UV disinfection to achieve the best effluent at the lowest energy cost should be evaluated in individual plants. Ways to optimize aeration include: 1) operating plug flow systems as tapered aeration systems (Rittman, et al., 2000; Metcalf and Eddy, 2003), 2) operating blowers near the best efficiency point, 3) install DO meters in aeration basins to control aeration (BASE Energy, 2006) and 4) improving the oxygen transfer efficiency of diffusers (WEF, 2009). One way to optimize UV that is already in use on many systems is to dim lamps during periods of low flow. This option requires advanced control systems, however (WEF, 2009). Optimization of pumping could take place by: 1) operating variable flow drive pumps, 2) operating the pumps near the best efficiency point, and 3) operating sludge removal processes in clarifiers and thickeners at intermittent times (WEF, 2009).

The hypothesis from Chapter 1 that aeration for the aeration basins would have the highest energy consumption no matter the wastewater strength is validated by the results found. In Table 2.4, the aeration basin blowers had the highest single energy consumption independent of the wastewater strength. Another hypothesis from Chapter 1 was that wastewater strength would have a significant impact on energy consumption due to increasing TKN. This hypothesis was also validated; total energy consumption increased by 32% between the low and average strengths, 41% between the average and high strengths, and 71% between the low and high strengths (Table 2.4). Total energy consumption increased for the aeration basins by 50% between the low and average strength wastewaters, 48% between the average and high strength wastewaters, and 93% between the low and high strength wastewaters (Table 2.4). Within the blowers on the aeration basins, TKN energy requirements were 53, 56, and 57% of the total energy consumption for the blowers [Figure 2.4 (b)]. While not addressed in this research, if advanced treatment is necessary for discharge then energy recovery and alternative forms of energy should be considered. Energy recovery includes anaerobic digestion and alternative forms of energy include solar, wind, and geothermal. Anaerobic digestion is a common form of energy recovery and one estimate of its production for a 100 MGD plant is 350 kWh/MGal (WEF, 2009) which would have reduced the energy consumption in this article by 39%, 28%, and 21% for the low, average, and high strength wastewater cases. Solar and geothermal technologies have already been used successfully at several WWTPs (Bernier, et al., 2011; Collingwood, et al., 2011; Seeta, et al., 2011) and in one case reduced net energy consumption by 90% (Bernier, et al., 2011). This, along with the energy consumption of additional treatment technologies such as membrane filtration, membrane bioreactors, and other treatment technologies are areas that need more research.

This research is most useful for designers as it provides a means of estimating energy consumption in the design and operating phase of WWTPs. By using the fundamental equations and methodology in this research, alternative technologies can be compared for energy consumption. In operating plants, similar methodology can be used to target high energy consuming units for energy savings. Limitations of the methodology include assumptions made in the design. These must be carefully evaluated when using the proposed methodology, and equipment efficiencies must match the equipment efficiencies for a WWTP. Efficiencies in particular can be hard to locate for certain equipment such as chemical pumps.

CHAPTER 3

# INFLUENCE OF ADVANCED TREATMENT TECHNOLOGIES ON THE ENERGY CONSUMPTION IN WASTEWATER TREATMENT PLANTS

## Abstract

Stricter effluent standards have fueled an increase of the number of advanced wastewater treatment plants over time. With this, associated energy consumption has also increased. In this research, the impacts on energy consumption, caused by switching from traditional to advanced treatment technologies, for a large (100 MGD) wastewater treatment plant were investigated. Commonly used design equations were used to size the unit operations, the energy consuming units of each unit operation were identified, and electrical motor sizes for energy consuming unit were calculated. Four alternatives were evaluated in this research: 1) medium pressure high output (MPHO) ultraviolet (UV) disinfection versus low pressure high output (LPHO) UV disinfection, 2) MPHO UV and LPHO UV disinfection versus chlorine disinfection followed by dechlorination, 3) plate and frame press dewatering versus centrifugation, and 4) partial biological nutrient removal (BNR) for ammonia oxidation and phosphorous removal was replaced with full BNR for total nitrogen removal and phosphorous removal. The results of this research came close to previous literature estimates in all cases but plate and frame press dewatering. By using the fundamental equations and methodology in this research, alternatives can be compared through energy consumption.

# 1 Introduction

Due to concerns about disinfection byproducts and harm to aquatic life, the U.S. EPA discouraged the use of chlorination for disinfection in wastewater during the 1980s and began researching alternative disinfection methods such as UV disinfection and dechlorination (Black and Veatch, 2010). During the 1980s, the installed UV systems were typically small with flows less than 5 million gallons per day (MGD) and were prone to failures which caused them to not be readily accepted by regulatory agencies until the 1990s (Black and Veatch, 2010). The first UV lamps were low pressure low output which limited the applicability of UV due to higher flows requiring a high number of lamps. The introduction of medium pressure high output UV lamps and low pressure high output UV lamps during the 1990s allowed for WWTPs with larger flows to adopt the use of UV disinfection (Black and Veatch, 2010). By the year 2005, approximately 20% of municipal WWTPs in the US utilized UV disinfection (Black and Veatch, 2010; Trojan UV, 2005).

The use of membranes in wastewater treatment has increased due to the need to provide cleaner discharge and for water reuse (WEF, 2006). Membranes have been used in the past to filter wastewater for advanced treatment and, recently, to provide a combination of filtration and activated sludge treatment through membrane bioreactors (MBRS) (WEF, 2006; WEF, 2012). In 1996, 43.7% of the US population was served by advanced treatment and this value is expected to increase to 56.7% in the year 2028 (U.S. EPA, 2008; U.S. EPA, 2010a). Between 2005 and 2010, the average price of energy increased roughly 18.5% in the United States from \$0.0573 per kWh to \$0.0679 per kWh (U.S. Energy Information Association, 2011). Energy consumption in WWTPs also represents anywhere from 18 to 30% of a WWTP's total operating costs (Molinos-Senante, et al., 2010; Metcalf and Eddy, 2003). Rising energy costs and the need to curb greenhouse gases generation (Foley, et al., 2010; Shahabadi, et al., 2009) are strong incentives for WWTPs to investigate ways to lower their overall energy consumption (Metcalf and Eddy, 2003; U.S. EPA, 2010b; WEF, 2009). Although advanced treatment technologies have several operational and water quality advantages, their sustainability as it relates to energy consumption and associated greenhouse gases generation has not been evaluated.

The sustainability of WWTPs related to energy can be increased by using technologies that reduce electricity consumption (e.g. variable frequency drives), using less advanced technologies that produce similar effluent quality, and using energy recovery technologies within WWTPs.

Variable frequency drives (VFDs) present a means of reducing energy consumption for WWTPs (Pacific Gas and Electric Company, 2003; Europump and Hydraulic Institute, 2004; WEF, 2009; U.S. EPA, 2010b). VFDs are electronic controllers that adjust the output of energy to a process component which allows for the speed of process components such as pumps to be controlled (Pacific Gas and Electric Company, 2003; U.S. EPA, 2010b). This is accomplished by a converter in the controller varying voltages to create a magnetic flux in a motor (Europump and Hydraulic Institute, 2004). It has been reported that 75% of pumps are oversized and VFDs provide a means to better match system conditions (Europump and Hydraulic Institute, 2004). VFDs have also been used in situations where valves were used to control flowrates (Europump and Hydraulic Institute, 2004). Valve controls are much less energy efficient than VFDs

(Europump and Hydraulic Institute, 2004). VFDs are able decrease energy consumption by as much as 30 to 50% (Pacific Gas and Electric Company, 2003; Europump and Hydraulic Institute, 2004).

Energy recovery technologies also provide a means for lowering energy consumption for WWTPs. These technologies include wind, solar, microturbines, and fuel cells (WEF, 2009). Wind turbines operate by converting mechanical energy (wind) to electrical energy (U.S. EPA, 2007). Wind turbines have efficiencies between 20 and 40 percent and have been used successfully in several WWTPs (U.S. EPA, 2007). Solar panels operate by absorbing light and transferring the energy to a semiconductor where electrons are allowed to flow and form a circuit with an electrical current that provides external energy (U.S. EPA, 2007). Typical efficiencies for solar panels are 5 to 17 percent (U.S. EPA, 2007). Solar panels have been able to provide as much as 30% of a WWTPs total energy requirements or about 4,100 kWh/day (Collingwood, et al., 2011). Fuel cells operate similar to a battery (U.S. EPA, 2011). Hydrogen flows in on the anode side and oxygen flows in on the cathode side with an electrolyte separating the cathode and anode (U.S. EPA, 2011). The hydrogen diffuses into protons and electrons where electrons pass through a circuit and provide energy (U.S. EPA, 2011). Water is then created and exits the fuel cell (U.S. EPA, 2011). Microturbines operate by combusting gas and spinning turbine fans at high speeds to rotate copper coils and create energy (U.S. EPA, 2011). Microturbines are relatively inexpensive compared to other gas generators and require little maintenance (U.S. EPA, 2011). Biogas from anaerobic digestion can be used with microturbines (U.S. EPA, 2011). For an 11 MGD WWTP, ten 30 kW microturbines were installed that provide energy savings of 2300 MW per year (U.S. EPA, 2011).

In this research, the impacts on energy consumption, caused by switching from traditional treatment technologies to advanced treatment technologies, for a large example (100 MGD) wastewater treatment plant were investigated. In order to investigate the effect of switching treatment technologies, commonly used design equations were used to size the unit operations. Once designed, the energy consuming units of each unit operation were identified and the electrical motor sizes to power those units were computed. It is anticipated that the results of this research will provide additional means to evaluate the impact on energy consumption by wastewater treatment plants.

## 2 Methodology

To estimate the amount of energy consumed in advanced plants as compared to traditional wastewater treatment plants, a 100 MGD plant was designed using typical design equations, such as those found in Metcalf and Eddy (2003), GLUMRB (2004), WEF (2010a), WEF (2010b). The design was based on an existing WWTP in the arid southwestern United States. The design focused on sizing the unit processes and on identifying fundamental variables in each unit that control energy use. Only the energy associated with treatment (unit operations) was computed in this research. Other energy requirements such as those for building heating/cooling, lighting, chemical manufacturing and transport energy were not computed in this research. Actual design criteria from an existing plant were used to validate designs used in the research. The plant is designed to remove biochemical oxygen demand (BOD), total suspended solids (TSS), ammonia, and phosphorous.

A process flow diagram of the designed WWTP is shown in Figure 3.1. Solid lines represent liquid flows, and dashed lines represent solids flows. Influent enters the plant and passes through the bar racks and the grit chamber where large solids and grit are removed in aerated grit chambers. BOD and TSS are partially removed in the primary clarifiers. Activated sludge aeration basins and secondary clarifiers remove BOD, TSS, ammonia and phosphorous. Phosphorus is removed biologically. Dual media filters are used to remove remaining suspended solids and the effluent is disinfected with UV. Primary solids are thickened in gravity thickeners while secondary solids are thickened in dissolved air flotation thickeners (DAFTS). Primary and secondary solids are then combined and dewatered in centrifuges with the cake being sent to a landfill. The centrate from the centrifuge is recycled back to the primary clarifiers. In the design, the impacts of recycle streams flows from solids handling were addressed using mass balances. Because filter backwashing is not a constant process, it is not shown in Figure 3.1. The impacts of filter backwashing were addressed in the design of the dual media filters and UV disinfection.

Four alternative treatment train designs were considered to evaluate the impact of advanced technologies on energy consumption:

Alternative Design I: The design train shown in Figure 3.1 with medium pressure high output UV (MPHO). In this scenario, MPHO UV is switched to low pressure high output (LPHO) UV disinfection.

Alternative Design II: Design train of Figure 3.1 where both MPHO and LPHO UV disinfection are switched to chlorine disinfection followed by dechlorination.

Alternative Design III: Design train of Figure 3.1 focusing on sludge dewatering. In this scenario, traditional plate and frame press dewatering is replaced with centrifugation.

Alternative Design IV: Design train of Figure 3.1 focusing on advanced nutrient removal in the aeration basins. It replaces partial BNR (ammonia oxidation and phosphorous removal) with full BNR (total nitrogen removal and phosphorous removal).

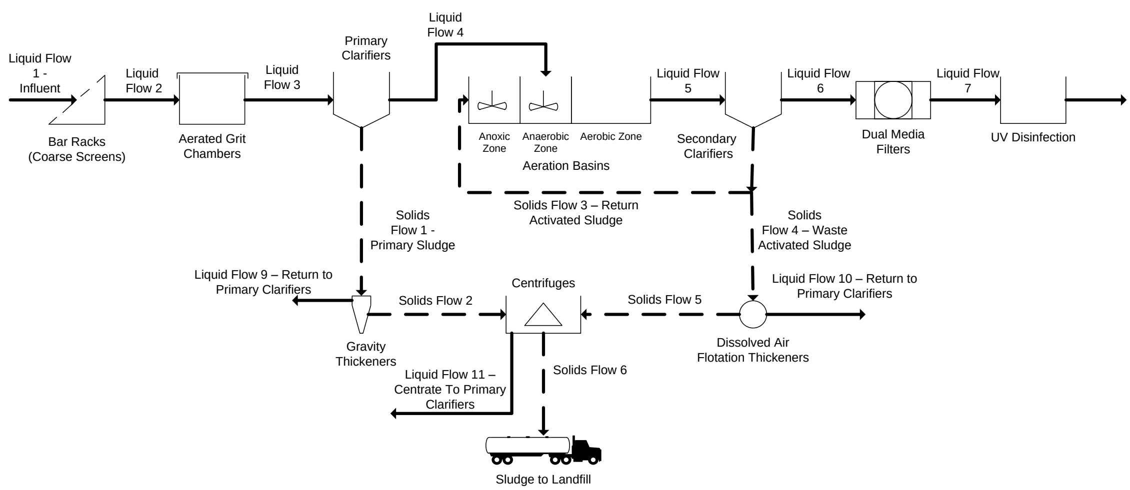


**Figure 3.1 - Process Flow Diagram for the Wastewater Treatment Plant Used in the Energy computations**

## 2.1 Wastewater Influent and Effluent Standards

Influent and effluent wastewater characteristics and peak factors used in the design are shown in Table 3.1 (Metcalf and Eddy, 2003). The influent characteristics are based upon the average strength characteristics presented in Chapter 2. The effluent requirements for total suspended solids (TSS) and biochemical oxygen demand (BOD) are based upon national standards for wastewater discharge (National Archives and Records Administration, 2012). The effluent standards for total phosphorous (TP), total

Kjeldahl Nitrogen (TKN), total coliform, and total nitrogen (TN) were set to the effluent standards of a WWTP in the arid southwestern United States. The TP and TN levels are also common National Pollution Discharge Elimination System (NPDES) for WWTPs requiring nutrient removal (U.S. EPA, 2010c). The dual media filters and UV have a higher peak flow factor because of the backwash from the dual media filters. The unit processes were designed taking the peak flows into consideration. Energy use in each unit was computed by dividing the energy in kWh/day by the average flowrate of the WWTP.

**Table 3.1 - Wastewater Characteristics and Effluent Criteria Partially Reproduced from (Metcalf and Eddy, 2003)**

| Contaminant | Influent | Effluent | Units |
|---|---|---|---|
| Influent Flow | 100 | N/A | MGD |
| Total Suspended Solids (TSS) | 210 | ≤30 | mg/L |
| Volatile Portion of TSS | 80 | N/A | % |
| 5 Day Biochemical Oxygen Demand (BOD) | 190 | ≤30 | mg/L |
| Total Phosphorous (TP) | 7 | ≤0.2 | mg/L |
| Total Kjeldahl Nitrogen (TKN)/Ammonia | 40 | ≤0.5 | mg/L |
| Total Nitrogen (TN) for Full Biological Nutrient Removal Case | 40 | ≤12 | mg/L |
| Total Coliform | $10^7$-$10^9$ | ≤200 | MPN/100 mL |
| Peak Flow Factor for All Liquid Units but Dual Media Filters and UV | 1.5 | N/A | N/A |
| Peak Flow Factor for Dual Media Filters and UV | 1.9 | N/A | N/A |

## 2.2 **Design**

Major design criteria for all the treatment processes are depicted in Table 3.2, where the number of units in operation for the average design flow is shown in parenthesis in the first column. The design of all units followed typical design parameters as established in references such as Tchobanoglous, et al. (2003), GLUMRB (2004), WEF (2010a), WEF (2010b).

**Table 3.2 - Treatment Process Design Parameters**

| Unit Process (number of units) | Parameter | Value | Units | Reference |
|---|---|---|---|---|
| UV (5) | Design UV Dosage | 30 | mW·s/cm$^2$ | [2] |
| | Low Pressure High Output Lamps | 352 | # of Lamps | N/A |
| | Medium Pressure High Output Lamps | 160 | # of Lamps | N/A |
| Chlorination (10) | Length, Width, Depth | 480, 12.5, 12.5 | ft | [1,4,5] |
| | Hydraulic Retention Time (HRT) at Average Flow | 80 | min | [1] |
| | Hydraulic Retention Time (HRT) at Peak Flow | 42 | min | [1] |
| | Average Flow Dosage | 7.8 | mg/L | [1] |
| | Peak Flow Dosage | 9.2 | mg/L | [1] |
| Dechlorination (10) | Average Flow Dosage | 1.6 | mg/L | [1] |
| | Peak Flow Dosage | 3.2 | mg/L | [1] |
| Centrifuges (2) | SLR for Combined Sludge | 4,200 | lb/hr | [3] |
| Plate and Frame Presses (2) | Volume | 250 | ft$^3$ | [6] |
| | Number of Chambers | 91 | N/A | [6] |
| | Height, Width of Press | 11.5, 8.9 | ft | [6] |
| | Length of Filter Press | 31.2 | ft | [6] |
| [1] = (Metcalf and Eddy, 2003), [2] = (GLUMRB, 2004), [3]= (Sieger, et al., 2006), [4] = (Black and Veatch, 2010), [5] = (WEF, 2010a), [6] = (WEF, 2010b) | | | | |

## 2.3 Alternative Design I – Switching MPHO UV to LPHO UV for Disinfection

This case examined the effect of switching MPHO UV to LPHO UV in the design. The UV dosage was estimated using the point source summation (PSS) method (U.S. EPA, 1986; WEF, 2010a) in lieu of bioassay data with the Emerick and Darby model used to predict effluent coliform values (WEF, 2010a). The transmittance was assumed as 72% for both MPHO and LPHO. The design dosage and number of lamps for LPHO and MPHO are shown in Table 3.2.

The main energy-consuming units for UV are the lamps. The maximum input of the lamps is 250 W and 3,200 W for LPHO and MPHO (Trojan UV, 2007; Trojan UV, 2008). Turndown capabilities were 60% and 30% for LPHO and MPHO (Trojan UV, 2007; Trojan UV, 2008).

## 2.4 **Alternative Design II – Switching UV to Chlorination/Dechlorination for Disinfection**

This case examined the effect of switching from either LPHO or MPHO UV to chlorination/dechlorination as the disinfection method for the plant's effluent. The chlorine source for the design was liquid sodium hypochlorite at 12.5% free chlorine (Metcalf and Eddy, 2003). The dechlorination source used in the design was gaseous sulfur dioxide (Metcalf and Eddy, 2003). Both the sodium hypochlorite and sulfur dioxide were assumed to be shipped in. The hydraulic retention time (HRT) of the chlorine contact chambers fall within typical recommended design parameters as shown in Table 3.2.

The energy consuming units for chlorination/dechlorination are the chemical feed system.

## 2.5 **Alternative Design III – Switching Centrifugation to Plate and Frame Press Dewatering for Solids Dewatering**

This case examined the effect of switching from centrifuges to plate and frame press for dewatering combined primary and secondary sludges. The main design equations used for the centrifuges were the volumetric and solids loading criteria as presented in Sieger, et al. (2006) (Table 3.2). The design of the plate and presses is presented in Table 3.2 following design procedures in (Davis, 2010). The pumping cycle used is that recommended by U.S. Army Corps of Engineers (2003). The cake solids concentration for the centrifuges was assumed to be assumed to be 20% with 95% solids capture efficiency (Metcalf and Eddy, 2003). The cake solids concentration for the plate and frame presses was assumed to be 36% with 95% solids capture efficiency (Metcalf and Eddy, 2003).

Energy consumption in the centrifuges comes from feed acceleration and cake conveyance as presented in Maloney, et al. (2008). Feed acceleration energy requirements were computed as follows:

$$P_{acc}=5.984\times10^{-10}SGQ(\Omega r_p)^2/e$$

Where $P_{acc}$ = feed acceleration energy (HP multiply by 0.746 for kW), SG = specific gravity, Q = feed flow rate (gpm), $\Omega$ = speed (rpm), $r_p$ = pool radius (in), and e = efficiency. The cake conveyance energy requirements are as follows:

$$P_{con}=1.587\times10^{-5}T\Delta/e$$

Where $P_{con}$ = cake conveyance energy requirements, T = torque (lb-in), $\Delta$ = differential speed between the bowl and conveyor (rpm), and e = efficiency.

Energy consuming units in the plate and frame presses are the pumps required to pressurize the sludge for dewatering. The brake horsepower (BHP) equation (Jones, et al., 2008) was used to compute the energy requirements of the pumps. The BHP equation is as follows:

$$BHP=\frac{QH}{3{,}960e}$$

Where BHP = brake horsepower (HP multiply by 0.746 for kW), Q = flow rate (gpm), H = pump head (ft), and e = efficiency.

### 2.6 Alternative Design IV – Switching Partial BNR to Full BNR in the Secondary Treatment

This case examined the effect of switching partial BNR to full BNR. Partial BNR includes removal of BOD, TSS, ammonia, and phosphorous. Partial BNR required polishing with alum for additional phosphorous removal. Full BNR removes BOD, TSS, phosphorous, and total nitrogen. Full BNR also required polishing with alum for additional phosphorous removal and methanol addition for denitrification at a rate of approximately 700 gpd (2650 lpd) per basin. Table 3.3 shows the microbial parameters used in the design of the aeration basins. The assumed HRT of the anoxic and anaerobic zones was one hour each. The HRT of the aerobic zone was four hours (WEF, 2010c). The design of the aeration basins was based on plug flow reactor kinetics (Rittman, et al., 2000), biological phosphorous removal assumptions published in Metcalf and Eddy (2003), and BioWin modeling (EnviroSim Associates LTD., 2003). The aeration basin blower requirements for BOD and TKN removal were calculated using oxygen demand requirement equations (Rittman, et al., 2000). VFAs for the PAOs are assumed to be generated in the primary clarifiers. The aeration basins had 127 mg/L BOD and 40 mg/L TKN entering the process after primary clarification. Total phosphorous remaining after biological treatment was 2.0 and 0.6 mg/L for the partial BNR and full BNR, respectively. In this study, an oxygen credit for denitrification oxygen credit was not included in the computation of the aeration needs.

Energy-consuming units for the aeration basins include the blowers, chemical pumps, and mixers for the anaerobic/anoxic zones. For processes such as the aeration basins, gravity thickeners, dissolved air flotation thickeners (DAFTS), and centrifuges, the kWh/day were divided by the amount of BOD, TKN, or solids processed.

**Table 3.3 - Microbial Parameters at 20°C for the Design of the Aeration Basins**

| Parameter | BOD Microbes | Nitritation Microbes | Nitratation Microbes | Phosphorous Accumulating Organisms (PAOs) | Denitrification Microbes |
|---|---|---|---|---|---|
| Half-Velocity constant, K (mg donor/L) | 10 [1] | 1 [1] | 1.3 [1] | 1 [1] | 12.6 [1] |
| Yield, Y (g VSS/g donor) | 0.4 [1] | 0.33 [1] | 0.083 [1] | 0.3 [2] | 0.26 [1] |
| Maximum Specific Growth Rate, $\mu_m$ (g VSS/g VSS -d) | 9 [1] | 0.76 [1] | 0.81 [1] | 0.95 [2,3] | 3.12 [1] |
| Endogenous Decay Coefficient, $k_d$ (g VSS/g VSS-d) | 0.15 [1] | 0.11 [1] | 0.11 [1] | 0.04 [3] | 0.05 [1] |
| $f_d$ | 0.8 [1] | | | | |
| [1] = (Rittman, et al., 2000), [2] = (Metcalf and Eddy, 2003), [3] = (WEF, 2010c) | | | | | |

# 3 Results and Discussion

Design equations for the various unit processes included in the WWTP allowed for the computation of electrical motor size requirements for individual unit processes (APPENDIX B). Typical efficiency values were acquired from the literature (Metcalf and Eddy, 2003; U.S. EPA, 1989; WEF, 2005; WEF, 2010a; WEF, 2010b) and equipment manufacturer data, and were used to calculate the motor size for each energy-consuming unit. The efficiency values given are combined motor and equipment efficiency.

The largest motors are those that power the blowers for the aeration basins for both the partial BNR and full BNR cases with requirements of 3,526 HP. The second largest motors are those for the mixed liquor recycle pumps for the full BNR aeration basins and require 2,106 HP. The mixed liquor recycle pumps energy requirement is roughly 60% of that of the aeration basin blowers. The third largest motors are the

MPHO UV lamps which require 1,125 HP. The MPHO UV lamps energy requirement is roughly 32% of the requirements of the aeration basin blowers. The fourth largest motors are for the aeration basin mixers with requirements of 436 HP. LPHO UV motor requirements are 368 HP, which is 10% of the requirements for the aeration basins blowers and 33% of the requirements for MPHO UV.

Centrifuge dewatering has total energy requirements of 82 HP which is only 2% of the requirements for the aeration basins blowers. Plate and frame press dewatering requires 17 HP for 15 min, 33 HP for 30 min, 50 HP for 30 min, 66 HP for 15 min which is 0.5, 0.9, 1.4, and 1.9% of the requirements for the aeration basins blowers, respectively. Chlorination/dechlorination combined has motor requirements of 15 HP which is only 0.4% of the requirements for the aeration basins blowers, 1.4% of the requirements for MPHO UV, and 4.2% of the requirements for LPHO UV.

The energy for each energy-consuming unit was found by determining energy requirements in kWh/day and dividing by the 100 MGD average flow which yields kWh/MGal (Table 3.4). The energy consumption for sludge processing on a mass basis was computed by dividing the energy for each energy-consuming unit (Table 3.4) by the tons of sludge entering the process (Table 3.5). The energy consumption for secondary treatment to remove BOD and TKN on a mass basis was computed by dividing the energy for each energy consuming unit (Table 3.4) by the BOD and TKN entering the secondary treatment process (Table 3.6). It should be noted that only the energy associated with treatment was computed in this research. Other energy requirements such as those for building heating/cooling, lighting, chemical generation, and transport energy were not computed in this research. Two types of estimates from the literature were used for comparison: volumetric based estimates (Table 3.7) and mass based estimates (Table 3.8).

**Table 3.4 - Energy Consumption for Selected Unit Operations for a 100 MGD Wastewater Treatment**

| | Component | kWh/MGal |
|---|---|---|
| **UV** | Low Pressure High Output UV | 65.9 |
| | Medium Pressure High Output UV | 201.6 |
| **Chlorination/ Dechlorination** | Chemical Pumps | 2.7 |
| | Chemical Pumps | 0.1 |
| | Total | 2.8 |
| **Centrifuges** | Feed Acceleration | 24.5 |
| | Cake Conveyance | 3.3 |
| | Total | 27.8 |
| **Plate and Frame Press** | Sludge Feed Pump | 6.6 |
| **Aeration Basins (Partial BNR)** | Blowers | 631.3 |
| | Chemical Pumps | 1.2 |
| | Mixers | 104.8 |
| | Total | 737.3 |
| **Aeration Basins (Full BNR)** | Blowers | 631.3 |
| | Chemical Pumps | 1.2 |
| | Mixers | 104.8 |
| | Internal Recycle Pumps | 37.7 |
| | Total | 775.0 |

**Table 3.5 - Solids Processing Energy Requirements per Ton of Solids**

| | Component | kWh/ton of solids |
|---|---|---|
| **Centrifuges** | Feed Acceleration | 31 |
| | Cake Conveyance | 4.2 |
| | Total | 35.2 |
| **Plate and Frame Press** | Sludge Feed Pump | 8.3 |

**Table 3.6 – Aeration Basins Energy Requirements per Pound of BOD or TKN Removed**

| | Component | kWh/lb BOD (kWh/lb TKN) |
|---|---|---|
| **Aeration Basins (Partial BNR)** | Blowers | 0.59 (1.87) |
| | Chemical Pumps | 1.16E-03 (3.68E-03) |
| | Mixers | 0.1 (0.31) |
| | Total | 0.69 (2.19) |
| **Aeration Basins (Full BNR)** | Blowers | 0.59 (1.87) |
| | Chemical Pumps | 1.16E-03 (3.68E-03) |
| | Mixers | 0.1 (0.31) |
| | Internal Recycle Pumps | 0.04 (0.11) |
| | Total | 0.73 (2.30) |

**Table 3.7 – Energy Consumption Estimates on a Volumetric Basis for Wastewater Treatment**

| | Volumetric Literature Estimates (kWh/MGal) | Reference |
|---|---|---|
| **Low Pressure High Output UV** | 60 | (URS Corporation, 2004) |
| | 76.8 | (WEF, 2009) |
| **Medium Pressure High Output UV** | 253.3 | [3] |
| | 250 | [2] |
| **Chlorination/Dechlorination** | 2.66 (chlorination only) | (WEF, 2009) |
| | 8 (chlorination/dechlorination) | (URS Corporation, 2004) |
| **Centrifuges** | 10 to 75 | [4] |
| **Plate and Frame Press** | 10.4 kWh/MGal | (U.S. EPA, 1978) |
| **Aeration Basins (Partial BNR)** | 532 for aeration (BOD removal only) | [4] |
| | 338 for aeration (ammonia removal only) | [4] |
| | 870 for aeration (BOD and ammonia removal) | [4] |
| | 1191.8 for aeration (BOD and ammonia removal) | [1] |
| | 2400 | [5] |
| **Aeration Basins (Total BNR)** | 2200 | (Foley, et al., 2010) |
| [1] = (U.S. EPA, 1978), [2] = (Pacific Gas and Electric Company, 2003), [3] = (URS Corporation, 2004), [4] = (WEF, 2009), [5] = (Foley, et al., 2010) | | |

**Table 3.8 - Energy Consumption Estimates on a Mass Basis for Wastewater Treatment**

| | Mass Based Literature Estimates | Reference |
|---|---|---|
| **Aeration Basins** | 0.6 (BOD removal only) | [2] |
| | 1.9 (BOD removal only) | [2] |
| | 2.6 (BOD removal only) | [2] |
| | 0.9 (BOD removal and nitrification/denitrification) | [2] |
| | 2.2 (BOD removal and nitrification/denitrification) | [2] |
| **Centrifuges** | 36 | [1] |
| **Plate and Frame Press** | 30 – 50 | (WEF, 1982) |
| [1] = (WEF, 1982), [2] = (Pacific Gas and Electric Company, 2003) | | |

## 3.1 Alternative Design I – Switching MPHO UV to LPHO UV for Disinfection

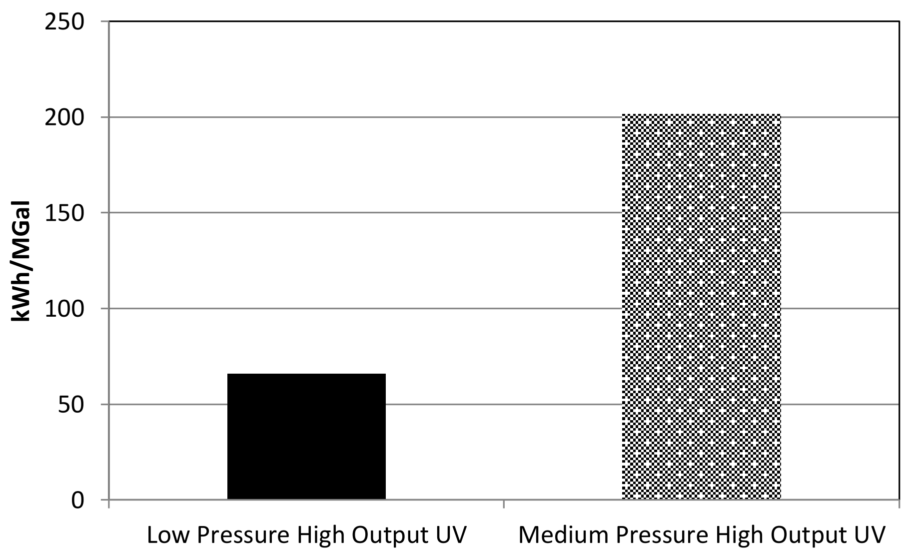


**Figure 3.2 - LPHO UV and MPHO UV Energy Consumption**

UV disinfection consumes a significant amount of the total energy with LPHO and MPHO consuming roughly 65.9 kWh/MGal and 201.6 kWh/MGal, respectively (Table 3.4 and Figure 3.2). LPHO UV is more energy-efficient than MPHO UV as it requires 3.1 times less energy to disinfect the same wastewater. Previous estimates for MPHO UV have been reported as 253.3 kWh/MGal (URS Corporation, 2004) and 250 kWh/MGal (Pacific Gas and Electric Company, 2003). In its estimate, URS (2004) had a transmittance of 65% while in this study 72% transmittance was assumed and is likely the reason for the difference between the values. The estimate by Pacific Gas and Electric Company (2003) did not report the transmittance used. At a transmittance of 65%, the estimate of this research would be 296.6 kWh/MGal, which is 15% higher than the estimates of URS (2004) and Pacific Gas and Electric Company (2003).

Compared to the aeration basins, which are the largest users of energy in the plant, LPHO uses about 9% and 6% of the total energy consumed by the partial and full BNR systems, respectively. MPHO UV requires 27% and 18% of the total energy required by partial and full BNR aeration basins, respectively. Published energy use for LPHO UV are 60 kWh/MGal (URS Corporation, 2004) and 76.8 kWh/MGal (WEF, 2009), which are both in the same other of magnitude as the value computed in this research, 66 kWh/MGal.

While LPHO UV requires much less energy than MPHO UV, MPHO UV still has some advantages for WWTPs over LPHO UV. Advantages of MPHO UV are MPHO UV has a lower construction cost than LPHO UV, takes up less room than LPHO UV, requires less lamps, has lower cleaning costs, and has lower labor costs for lamp replacement (WEF, 2009; WEF, 2010a). The chief disadvantage of MPHO UV is that the energy consumption is three to four times higher than LPHO UV at the same dosage and flow rate (WEF, 2009). Another disadvantage of MPHO UV are that the lamps are more expensive to purchase than LPHO UV (WEF, 2009).

## 3.2 Alternative Design II – Switching from UV to Chlorination/Dechlorination for Disinfection

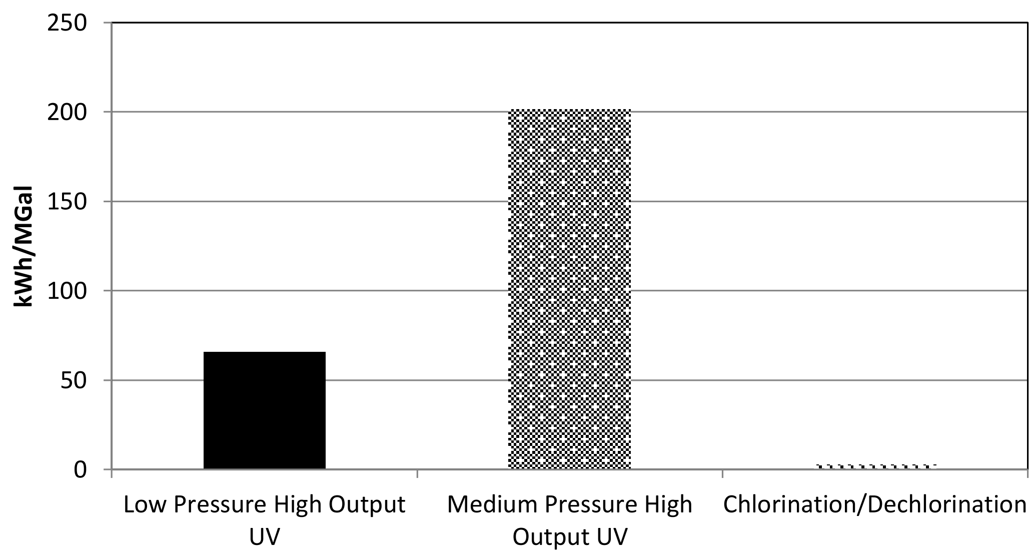


**Figure 3.3 - LPHO UV, MPHO UV, and Chlorination/Dechlorination Energy Consumption**

Chlorination/dechlorination is a small energy-consuming process, requiring only 2.76 kWh/MGal (Table 3.4 and Figure 3.3), which is about 24 times and 76 times less energy than that required for LPHO UV and MPHO UV, respectively. WEF (2009) has reported the energy estimate for chlorination as 2.66 kWh/MGal (WEF, 2009), which is very similar to the value found in this research. An estimate of chlorination/dechlorination energy requirements combined has been reported as 8 kWh/MGal (URS Corporation, 2004), which is significantly higher than the estimate of this research.

Compared to energy requirements for the aeration basins, chlorination/dechlorination requires only 0.4% and 0.3% of the total energy of partial BNR and full BNR, respectively.

While chlorination/dechlorination requires less energy than UV disinfection, the effects of disinfection byproducts and harm to aquatic life have to be taken into consideration (WEF, 2009; Black and Veatch, 2010). When disinfection byproducts are an issue in effluent discharge, then UV disinfection should be considered. However, as WWTP move towards sustainability and curbing of GHGs, the high energy consumption of UV has to be weighed against the disadvantages of chlorination/dechlorination for individual plants.

## 3.3 Alternative Design III – Switching Centrifugation to Plate and Frame Press Dewatering for Solids Dewatering

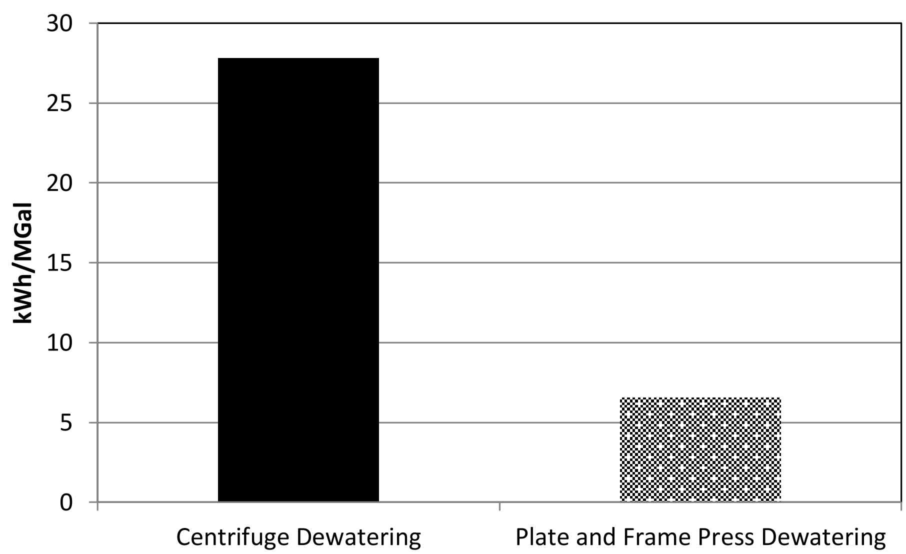


**Figure 3.4 - Centrifuge Dewatering and Plate and Frame Press Dewatering Energy Consumption**

While solids dewatering is not typically thought of as a large energy-consuming unit, there are differences in energy consumption among dewatering alternatives. Energy requirements for sludge processing can be reported on a volumetric basis (kWh/MGal of wastewater treated) or on a mass basis (kWh/ton of sludge dewatered). The results of this research reveal that centrifuge dewatering requires 27.8 kWh/MGal, while plate and frame press dewatering requires 6.6 kWh/MGal (Table 3.4 and Figure 3.4). Thus, centrifuging is about 2.2 more energy intensive that plate and frame press dewatering. In this research, it was found that centrifuge dewatering requires 35.2 kWh/ton of solids, while plate and frame press dewatering requires 8.3 kWh/ton of solids (Table 3.5).

Reported values for energy consumption for centrifugation vary from 10 to 75 kWh/MGal (WEF, 2009). The estimate in this research falls in this range (Table 3.4). An older estimate of centrifugation energy consumption reports 36 kWh/ton of sludge processed (WEF, 1982). The estimate in WEF (1982) is similar to the estimates in this research.

A reported estimate for plate and frame press dewatering energy consumption is 30 to 50 kWh/ton (WEF, 1982), which is higher than the 6.6 kWh/ton estimated in this research (Table 3.5). A US EPA report (U.S. EPA, 1978) estimates an energy consumption of 10.4 kWh/MGal (U.S. EPA, 1978) for plate and frame press dewatering. Both estimates are higher than the estimate in this research.

While plate and frame press dewatering requires less energy than centrifuge dewatering, a few key factors must be taken into consideration when considering these units. Plate and frame press dewatering is a batch operation, and therefore it requires more operator attention than centrifuge dewatering, which is a continuous process (WEF, 2009). In general, plate and frame press dewatering, makes a drier cake that translates to cost savings in further processing or transportation of the sludge for final disposal (Metcalf and Eddy, 2003; WEF, 2010b). Energy consumption, operator attention, and dewatered cake moisture content should be evaluated when choosing between centrifuge dewatering and plate and frame press dewatering.

## 3.4 Alternative Design IV – Switching Partial BNR to Full BNR in the Secondary Treatment

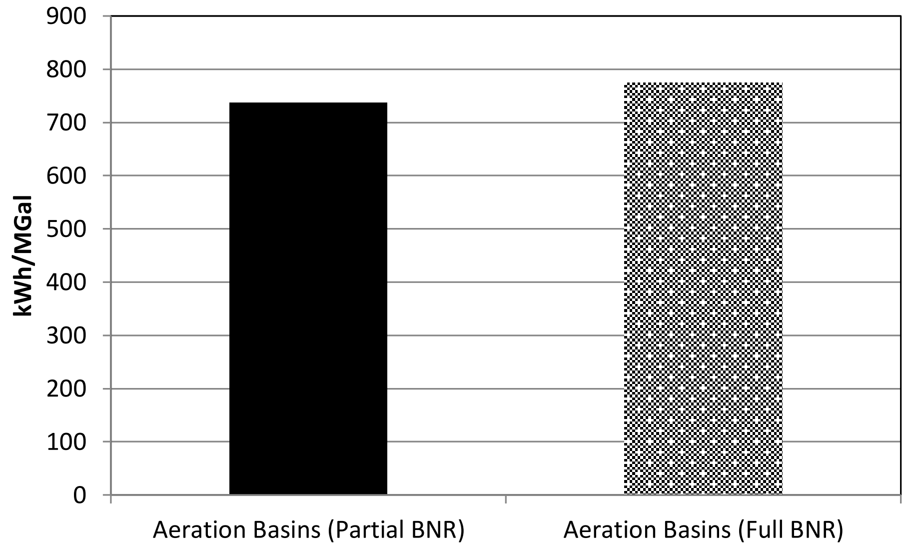


**Figure 3.5 – Partial BNR and Full BNR Total Energy Consumption**

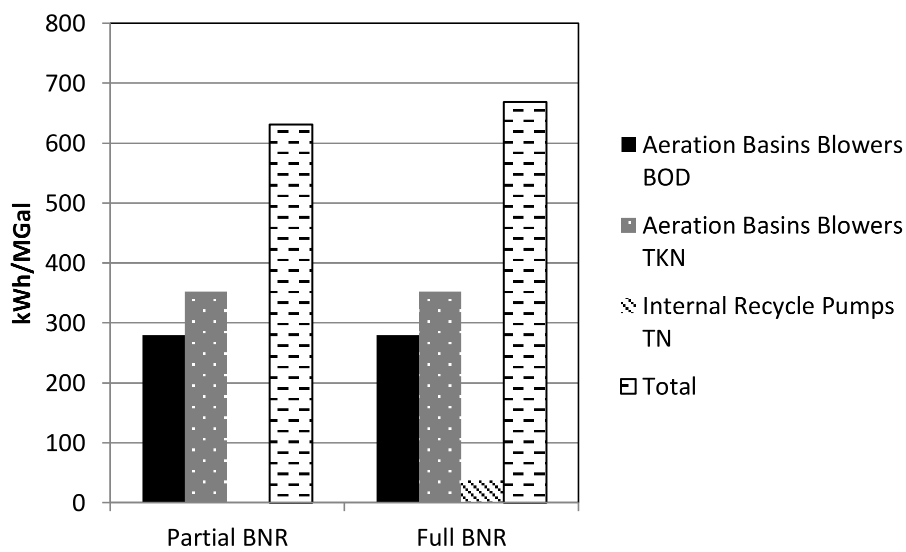


**Figure 3.6 - Energy Consumption for Partial and Full BNR Based Upon BOD, TKN and TN**

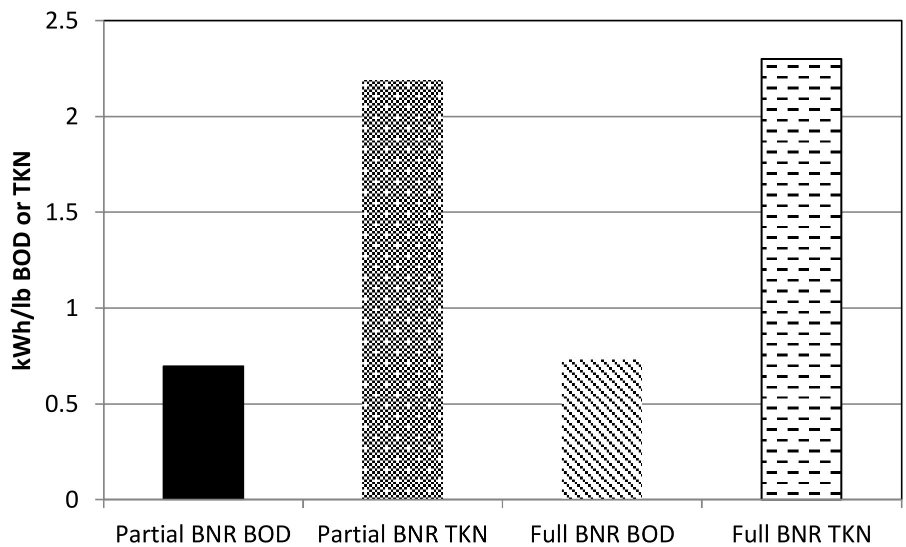


**Figure 3.7 – Aeration Basins Requirements for BOD and TKN Removal on a Mass Basis**

On a volumetric basis, partial, the results of this research show that BNR requires 737.3 kWh/MGal while full BNR requires 775 kWh/MGal (Table 3.4 and Figure 3.5), which is about 1.05 times larger. In other words, partial BNR requires 5% less energy than he full BNR. The difference in energy consumption between the partial and full BNR cases comes from the internal recycle pumps used in full BNR to remove total nitrogen (Table 3.4 and Figure 3.6).

Results show that power requirements for BOD and TKN removals are 279.1 kWh/MGal and 352.2 kWh/MGal, respectively (Figure 3.6) for both the partial and full BNR. That is, TKN removal is 1.26 times more energy consuming than BOD removal alone. On a volumetric basis, the internal recycle pumps for full BNR require 37.7 kWh/MGal (Table 3.4 and Figure 3.6).

On a TKN and BOD mass basis and for loadings of 106,600 lb BOD/day and 33,700 lb TKN/day entering the aeration basins after primary clarification, energy consumptions in the aeration basins of 0.69 kWh/lb BOD and 0.73 kWh/lb BOD for partial and full BNR, respectively, were computed (Table 3.6 and Figure 3.7). For TKN removal, requirements of 2.19 kWh/lb and 2.30 kWh/lb TKN for partial and full BNR were estimated, respectively (Table 3.6 and Figure 3.7).

Total energy consumption on a mass basis for both BOD and TKN increased by 5% between the partial and full BNR cases (Table 3.6). In Chapter 2, energy consumption for secondary clarification was estimated as 0.10 kWh/lb BOD and 0.31 kWh/lb TKN. This consumption must be added to the energy consumption in the aeration basis since secondary clarifiers are an integral part of the activated sludge system. The total consumption is then computed as 0.79 kWh/lb BOD and 2.49 kWh/lb TKN for partial BNR. For full BNR, the energy consumption is calculated as 0.83 kWh/lb BOD and 2.61 kWh/lb TKN.

Volumetric basis estimates reported consumptions of 532 kWh/MGal for aeration (BOD removal only), 338 kWh/MGal (ammonia removal only), 870 kWh/MGal for aeration (BOD and ammonia removal) only (WEF, 2009), 1191.8 kWh/MGal for aeration (BOD and ammonia removal) only (U.S. EPA, 1978), and approximately 2200 kWh/MGal (Foley, et al., 2010) total for full BNR. It is significant to note that the estimates in WEF (2009) for aeration (BOD removal only) and aeration (BOD and ammonia removal) are higher than the values found in this research for partial and full BNR An explanation of differences between the estimates in WEF (2009) and this research could be due to differences in influent wastewater quality values. Previous estimates reported in Chapter 2 show values similar to the ones reported by WEF (2009) with higher influent BOD. The estimate in WEF (2009) for aeration (ammonia removal only) only more closely matches the estimates in this research for partial and full BNR.

The energy estimate in U.S. EPA (1978) involved aeration only (BOD and ammonia). The EPA estimate assumed an influent BOD of 136 mg/L, a concentration similar to that assumed in this research paper. Furthermore, the EPA estimate assumed influent ammonia of 25 mg/L which is significantly lower than what was assumed in this research. The EPA energy estimate is much higher than that in this research which considered partial and full BNR. The relatively high energy consumption was cited by U.S. EPA (1978) at a time that predated advances in aeration technology made after the article was published. Advances such as automated DO monitoring have improved aeration technology greatly in the time after the article was published (U.S. EPA, 2010b).

The energy consumption estimate in Foley, et al, (2010) was based on a flow rate of approximately 2.6 MGD, which is much lower than the flow rate used in this research study to estimate energy requirements. Previous research has found that energy consumption as a function of wastewater flow decreased with higher flow wastewater flow rates (Hernández-Sancho, et al., 2011; WEF, 2009). Thus the estimate in Foley, et al, (2010) is likely higher due to the small flow rate.

A mass based estimate for secondary process requirements to remove BOD only is 0.6 kWh/lb BOD (Pacific Gas and Electric Company, 2003). The estimates in Pacific Gas and Electric Company (2003) were computed using data from operating plants and also include the requirements for secondary clarification. The 0.6 kWh/lb BOD estimate had influent a BOD estimate of 175 mg/L with a flow rate of 11.5 MGD. In comparison to this research, the 175 mg/L BOD estimate is about 1.4 times higher than the estimate in this research while the flow 11.5 MGD flow rate is 8.7 times lower than the flow rate used in this research paper. Moreover, the 0.6 kWh/lb BOD estimate is for BOD removal only, while this research paper included nitrification and partial denitrification. As a result, the (Pacific Gas and Electric, 2003) energy estimate is lower than that in this research paper for partial and full BNR.

A more accurate comparison to the previous estimate (0.6 kWh/lb BOD) is produced if the estimates of the aeration basins blowers and total secondary clarifier consumption (Chapter 2) are added together. This then makes the estimate for the secondary treatment 0.69 kWh/lb BOD. This evaluation also more closely mimics the 0.6 kWh/lb BOD estimate in terms of treatment. In this case, there is only a minor difference between this study's 0.69 kWh/lb BOD energy estimate and the 0.6 kWh/lb BOD estimate of (Pacific Gas and Electric, 2003). This minor difference can be largely attributed to the fact that this study's energy estimate included nitrification. Nitrification was not included in the 0.60 kWh/lb BOD estimate. Without nitrification, the difference between the two estimates would be smaller.

Additional energy consumption estimates, on a mass basis, for BOD removal only in secondary treatment are 1.90, and 2.60 kWh/lb BOD for BOD removal only (Pacific Gas and Electric Company, 2003). These estimates were for wastewaters with influent BOD of 165, and 80 mg/L with flow rates of 2.4, and 1.7 MGD, respectively. These estimates are higher those in this research because of their low flow rates.

Additional energy consumption estimates for BOD removal and nitrification/denitrification in secondary processes are 0.90, and 2.20 kWh/lb BOD for BOD removal and nitrification/denitrification (Pacific Gas and Electric Company, 2003). These two estimates were for wastewaters with influent BOD estimates of 180, and 85 mg/L with flow rates of 19.4, and 5.4 MGD, respectively. The influent TKN values for these estimates were not reported. The 0.9 kWh/lb BOD is within 21% of the estimate in this research for full BNR. As influent TKN and other operational parameters for denitrification were not reported, it is difficult to ascertain why the estimate is not closer. Finally, the 2.20 kWh/lb BOD estimate is most likely higher than the estimates in this research due to the low flow.

# 4 Conclusions and Discussion

When considering the use of different treatment alternatives in a WWTP, there are many criteria that go into weighing alternatives. This research explored the change in energy consumption by switching traditional technologies for advanced technologies. The change in energy consumption as a result of using more advanced technology provides other criteria to consider. Most of the energy consumption values found in this research were close to values found in the literature, but some differ. Plate and frame press dewatering, specifically was different. There is, however, wide variation in the existing studies. Parameters like influent to a process and operating parameters can change energy estimates. The fundamental and design equations used to estimate energy consumption in this research are useful, but must be used carefully. Assumptions must be made carefully and verified against existing WWTPs to obtain useful results.

In the case of secondary treatment additional energy is required for the implementation of denitrification, but not many plants require denitrification currently. A previous estimate of total energy for wastewater treatment without denitrification is 1,230.8 kWh/MGal (Chapter 2). To incorporate denitrification, the estimate of mixed liquor recycle pumps must be added to the total energy. Adding these two estimates yields 1,268.5 kWh/MGal which is a 3% increase in overall energy consumption for the entire WWTP. In this regard, the hypothesis from Chapter 1 that incorporating full BNR (phosphorous and total nitrogren removal) would not increase energy consumption for the WWTP significantly is confirmed. Another study found that incorporating denitrification in a WWTP using a modified Ludzack-Ettinger configuration increased energy consumption 40% for the aeration basins over traditional activated sludge (Vidal, et al., 2002). Incorporating denitrification in a WWTP using an oxidation ditch configuration increased energy consumption 12% for the aeration basins over traditional activated sludge (Vidal, et al., 2002). One study found that incorporating denitrification decreased overall energy consumption by 5% (Foley, et al., 2010) by assuming denitrification provides an oxygen credit (Metcalf and Eddy, 2003). Based upon this and previous research, the cost of incorporating denitrification in existing WWTPs is still an area that needs to be explored because the added benefits must be weighed against the higher energy cost. Denitrification may not be an option for some WWTPs due to discharge permit requirements.

In the case of solids dewatering, both centrifuges and plate and frame press dewatering have several advantages and disadvantages. Advantages of centrifuges include that centrifugation is a continuous process that requires less operator attention, and has a low landuse footprint; disadvantages of centrifuges include that centrifugation is noisy, produces high suspended solids in the centrate, is energy intensive, and requires skilled maintenance personnel (Metcalf and Eddy, 2003; WEF, 2010b). Advantages of plate and frame press dewatering include that it produces a very dry cake and a filtrate with low suspended solids; disadvantages of plate and frame press dewatering include that the process is a batch operation that requires more operator attention, and requires a significant amount of chemicals (Metcalf and Eddy, 2003; WEF, 2010b). WWTPs may choose to use centrifuge dewatering if they do not have the land footprint available for other dewatering alternatives, or desire a continuous process that requires less operator attention. As mentioned previously, plate and frame press dewatering produces a filtrate with low suspended solids while centrifugation produces a centrate with high suspended solids. The filtrate and centrate are typically returned to the liquid treatment portion of a treatment plant to recover the liquid. The effect of this return liquid stream on energy consumption of downstream processes (in particular aeration) is another factor that must be included in comparing dewatering alternatives.

In the case of disinfection, UV disinfection is appropriate compared to chlorination/dechlorination when disinfection byproducts in wastewater effluent are an issue. A situation where wastewater effluent having disinfection byproducts would be an issue is when wastewater effluent discharges into an area such as a lake or river that eventually provides water for a drinking water treatment plant. One study found that when chloramine was used as a disinfectant trihalomethanes (THM) in wastewater effluent was as high as 112 µg/L (Rebhun, et al., 1997). The same study found that when chlorine was used as a disinfectant (THM) in wastewater effluent was high as 4,570 µg/L (Rebhun, et al., 1997). Another study found a median THM value of 2 µg/L for non-nitrified plant effluent and a median value of 57 µg/L for nitrified plant effluent for multiple WWTPs in the US utilizing chlorination (Krasner, et al., 2009). For comparative purposes, the current federal regulatory limit on THM on drinking water in the United States is 80 µg/L (U.S. EPA, 2012). There are no federal discharge standards for THM in wastewater. Nevertheless, elevated THM in wastewater that directly or indirectly discharges into source of drinking water may adversely impact public health.

Some of the previous studies have found THMs that are near or well above the regulatory limit. UV disinfection should be considered in cases where DBP formation is high and effluent can impact drinking water sources. In these cases, LPHO UV should be considered as it consumes three to four times less energy than MPHO UV (WEF, 2010a). This research is most useful for designers as it provides a means of estimating energy consumption in the design phase of a wastewater treatment plant. By using the fundamental equations and methodology in this research, alternatives can be compared through energy consumption. Limitations of the methodology include assumptions made in the design. These must be carefully evaluated when using the proposed methodology, and equipment efficiencies must match the equipment efficiencies for a WWTP. Efficiencies in particular can be hard to locate for certain equipment such as chemical pumps. It should be noted that this research does not address all treatment technologies for energy consumption. There are still other treatment alternatives that are open for energy computation including membrane bioreactors (MBRs), ozonation, and oxidation ditches.

# CHAPTER 4

# CONCLUSIONS

Evaluation of energy consumption in the various unit operations provided insight into areas to be targeted for conservation and/or process changes. One of the objectives of this research was to evaluate whether design and fundamental equations can be used to evaluate energy consumption in a WWTP. The results of this research came close to existing studies in all cases but primary clarification (Chapter 2) and plate and frame press dewatering (Chapter 3). There is, however, wide variation in the existing studies. Parameters like influent water quality to a process and operating parameters can change energy estimates. The fundamental and design equations used to estimate energy consumption in this research are useful, but must be used carefully. Assumptions must be made carefully and verified against existing WWTPs to obtain useful results.

Another objective of this research was to evaluate how wastewater strength affects energy consumption in a WWTP and identify treatment processes that are susceptible to wastewater strength. Overall, this research was able to identify treatment processes that are susceptible to wastewater strength (Chapter 2). In terms of overall impact to the WWTP total energy consumption, aeration basins, UV, DAFTs, and centrifuges are the most susceptible to wastewater strength (Table 2.4 and Figure 2.7). The impact of aeration basins regardless of wastewater strength confirms the hypothesis in this research (Chapter 1). In terms of percent increase in individual processes with increasing strength, DAFTs, centrifuges, gravity thickeners, and aeration basins are the most susceptible to wastewater strength (Table 2.4 and Figure 2.7). Processes that were not significantly affected by wastewater strength included the bar racks, secondary clarifiers, and dual media filters (Table 2.4 and Figure 2.7). The aeration basins were susceptible to increasing wastewater strength as a result of increasing TKN. UV is susceptible to increasing wastewater strength due to decreasing transmittance. The DAFTs, gravity thickeners and centrifuges are susceptible to increasing wastewater strength as a result of increased solids. Bar racks are not susceptible to increasing wastewater strength, because they are such a small energy consuming process in the WWTP. Secondary clarifiers were not significantly affected by wastewater strength due to increased wasting. The dual media filters were not significantly affected by wastewater strength due to FIPS being the largest energy consumer. The hypothesis in this research that as wastewater strength increases the overall energy also increases is also confirmed (Chapter 1). TKN had a high impact on aeration costs and increased TSS increased the need for solids processing which had a large impact on overall energy consumption.

Another objective was to investigate the impact of advanced WWT technologies on the overall energy consumption for a large WWTP. The change in energy consumption as a result of using more advanced technology provides other criteria to consider for WWTP design and operation.

In the case of secondary treatment additional energy is required for the implementation of denitrification, but not many plants require denitrification currently. A previous estimate of total energy for wastewater treatment without denitrification is 1,230.8 kWh/MGal (Chapter 2). To incorporate denitrification, the 37.7 kWh/MGal estimate of mixed liquor recycle pumps (Chapter 3) must be added to the total energy. Adding these two estimates yields 1,268.5 kWh/MGal which is a 3% increase in overall energy consumption for the entire WWTP. Another study found that incorporating denitrification in a WWTP using a modified Ludzack-Ettinger configuration increased energy consumption 40% for the aeration basins over traditional activated sludge (Vidal, et al., 2002). Incorporating denitrification in a WWTP using an oxidation ditch configuration increased energy consumption 12% for the aeration basins over traditional activated sludge (Vidal, et al., 2002). One study found that incorporating denitrification decreased overall energy consumption by 5% (Foley, et al., 2010) by assuming denitrification provides an oxygen credit (Metcalf and Eddy, 2003). Based upon this and previous research, the cost of incorporating denitrification in existing WWTPs is still an area that needs to be explored because the added benefits must be weighed against the higher energy cost. Denitrification may not be an option for some WWTPs due to discharge permit requirements.

In the case of solids dewatering, both centrifuges and plate and frame press dewatering have several advantages and disadvantages. Advantages of centrifuges include that centrifugation is a continuous process that requires less operator attention, and has a low land footprint (Metcalf and Eddy, 2003; WEF, 2010b). Disadvantages of centrifuges include that centrifugation is noisy, produces high suspended solids in the centrate, is energy intensive, and requires skilled maintenance personnel (Metcalf and Eddy, 2003; WEF, 2010b). Advantages of plate and frame press dewatering include that it produces a very dry cake and a filtrate with low suspended solids (Metcalf and Eddy, 2003; WEF, 2010b). Disadvantages of plate and frame press dewatering include that the process is a batch operation that requires more operator attention, and requires a significant amount of chemicals (Metcalf and Eddy, 2003; WEF, 2010b). WWTPs have good reason to use centrifuge dewatering if they do not have the land footprint available for other dewatering alternatives, desire a continuous process that requires less operator attention, or simply do not have the funding for other alternatives that require a higher capital cost. As mentioned previously, plate and frame press dewatering produces a filtrate with low suspended solids while centrifugation produces a centrate with high suspended solids. The filtrate and centrate are typically returned to the liquid treatment portion of a treatment plant to recover the liquid. The effect of this return liquid stream on energy consumption of downstream processes (in particular aeration) is another way to compare dewatering alternatives.

In the case of disinfection, UV disinfection is appropriate compared to chlorination/dechlorination when disinfection byproducts in wastewater effluent are an issue. A situation where wastewater effluent having disinfection byproducts would be an issue is when wastewater effluent discharges into an area such as a lake or river that eventually provides water for a drinking water treatment plant. One study found that when chloramine was used as a disinfectant trihalomethanes (THM) in wastewater effluent was as high as 112 µg/L (Rebhun, et al., 1997). The same study found that when chlorine was used as a disinfectant (THM) in wastewater effluent was high as 4,570 µg/L (Rebhun, et al., 1997). Another study found a median THM value of 2 µg/L for non-nitrified plant effluent and a median value of 57 µg/L for nitrified plant effluent for multiple WWTPs in the US utilizing chlorination (Krasner, et al., 2009). For comparative purposes, the current federal regulatory limit on THM on drinking water in the United States is 80 µg/L (U.S. EPA, 2012). Some of the previous studies have found THMs that are near or well above the regulatory limit. UV disinfection should be considered in cases where DBP formation is high and effluent can impact drinking water. In these cases, LPHO UV should be considered as it consumes three to four times less energy than MPHO UV (WEF, 2010a).

Aeration, UV disinfection, and pumping should be targeted for energy reduction. Optimizing aeration and UV disinfection to achieve the best effluent at the lowest energy cost should be evaluated in individual plants. Ways to optimize aeration include: 1) operating plug flow systems as tapered aeration systems (Rittman, et al., 2000; Metcalf and Eddy, 2003), 2) operating blowers near the best efficiency point, 3) install DO meters in aeration basins to control aeration (BASE Energy, 2006) and 4) improving the oxygen transfer efficiency of diffusers (WEF, 2009). One way to optimize UV that is already in use on many systems is to dim lamps during periods of low flow. This option requires advanced control systems, however (WEF, 2009). Optimization of pumping could take place by: 1) operating variable flow drive pumps, 2) operating the pumps near the best efficiency point, and 3) operating sludge removal processes in clarifiers and thickeners at intermittent times (WEF, 2009).

This research is most useful for designers as it provides a means of estimating energy consumption in the design phase. By using the fundamental equations and methodology in this research, alternatives can be compared for energy consumption.

The following is recommended as follow up research in this area:

a) Investigate the impact of energy recovery and alternative forms of energy on overall energy consumption in a WWTP. Energy recovery includes technologies like anaerobic digestion. Alternative forms of energy include solar, wind, and geothermal. An estimate of reduction in overall energy for a WWTP as a result of energy recovery is illustrated in the following example. An estimate of anaerobic digestion energy production for a 100 MGD plant is 350 kWh/MGal (WEF, 2009). This amount would have reduced the energy consumption in Chapter 2 by 39%, 28%, and 21% for the low, average, and high strength wastewater cases. In addition, for one WWTP using alternative forms of energy, a 90% reduction in energy consumption was achieved (Bernier, et al., 2011).

b) Evaluate the impact of temperature conditions on WWTP energy consumption. Microbes for example are directly affected by temperature conditions. In general, as temperature decreases microbial kinetics also decrease (Rittman, et al., 2000). This leads to an impact on the aeration basins as a longer hydraulic retention time is necessary at colder temperatures than warmer temperatures to achieve the same treatment. As a result, larger aeration basins are required. Another example of temperature impacting WWTP units is in aeration. As temperature increases, the energy requirements of aeration also increase (U.S. EPA, 1989).

c) Weigh the impact of solids processing return streams on overall WWTP energy consumption. Parameters like TSS, BOD, and TKN in return streams directly affect energy consumption of subsequent processes such as clarification and secondary treatment.

d) Investigate the impact of operating parameters on overall WWTP energy consumption. Operating parameters include factors such as differentials in centrifuges (the difference between the scroll and bowl speed) and hydraulic retention times in treatment units.

e) Further investigate the impact of advanced treatment units on overall energy consumption. Advanced treatment units not addressed in this research include membrane bioreactors, membrane filtration, oxidation ditches, and sequencing batch reactors.

APPENDIX A - COMPUTED MOTOR SIZES FOR SPECIFIC UNIT PROCESSES FOR A 100 MGD WASTEWATER TREATMENT PLANT, ENERGY REQUIREMENTS AND EFFICIENCIES BASED UPON WASTEWATER STRENGTH

| | Energy Component (Number components in Operation) | Equation Type | Equation Parameters | Efficiency | Motor Size | Total Motor Size | Reference |
|---|---|---|---|---|---|---|---|
| **Bar Racks** | Rakes (2 bar racks for a, b, and c) | N/A | N/A | N/A | 5 HP (3.73 kW) per bar rack | 10 HP | [13] |
| **Grit Chambers** | Blowers (4 blowers for a, b, and c) | Blower | $q_s$ = 218, 334, 435 (scfm), $T_a$ = 570(°R), $P_b$ = 19.7 psia | 80% | 5.7a, 8.8b, 11.5c HP per blower | 23, 35, 46 HP | [1,3,9] |
| **Primary Clarifiers** | Sludge Pumping (10 pumps for a, b, and c) | Pump | H = 60 (ft) | 50% | 2.7a, 3.1b, 3.6c HP per pump | 27, 31, 36 HP | [4,8] |
| | Torque (10 rake arms for a, b, and c) | Torque | K = 8, 9, 10 (lb/ft), ω = 0.0033 (rad/s) | 75% | 0.24a, 0.26b, 0.29c HP Per rake arm | 2.4, 2.6, 2.9 HP | [5] |
| **Aeration Basins** | Blowers (10 blowers for a and b, and 12 for c) | Blower | $q_s$ = 4,200, 79,00, 11,300, (scfm), $T_a$ = 528(°R), $P_b$ = 24.7 psia | 80% | 187.8a, 352.6b, 503.2c HP per blower | 1,878, 3,526, 5,032 HP | [1,3,9] |
| | Chemical Pumps (10 pumps for a and b, and 12 for c) | N/A | N/A | N/A | 1abc HP per pump | 10, 12 HP | [12] |
| | Mixers (10 mixers for a and b, and 12 for c) | N/A | 0.53 HP/1000 $ft^3$ | N/A | 43.6abc HP per mixer | 436, 523 HP | [9] |
| **Secondary Clarifiers** | RAS (10 pumps for a and b, and 12 for c) | Pump | H = 40 ft | 75% | 58a,56b, 75c HP | 580, 560, 900 HP | [4,11] |
| | Torque (10 rake arms for a and b, and 12 for c) | Torque | K = 6, 6.3, 6.6 (lb/ft), ω = 0.0055 (rad/s) | 75% | 0.4a, 0.42b, 0.44c HP per rake arm | 4, 4.2, 5.3 HP | [5] |
| | WAS (10 pumps for a and b, and 12 for c) | Pump | H = 50 ft | 75% | 0.6a, 1.1b, 1.8c HP per pump | 6, 11, 22HP | [2,4] |
| **Dual Media Filters** | Filter Influent Pump Station (FIPS) (6 pumps | Pump | H = 30 ft | 75% | 117abc HP per pump | 702 HP | [4,11] |

| | Energy Component (Number components in Operation) | Equation Type | Equation Parameters | Efficiency | Motor Size | Total Motor Size | Reference |
|---|---|---|---|---|---|---|---|
| | for a, b, and c) | | | | | | |
| | Backwash Pump Energy (1 pump for a, b, and c) | Pump | H = 60 ft | 75% | 188abc HP per pump | 188 HP | [4,11] |
| | Backwash Blower Energy (1 blower for a, b, and c) | Blower | $q_s$ = 4,000 (scfm), $T_a$ = 528(°R), $P_b$ = 21.7 psia | 80% | 132abc HP per blower | 132 HP | [1,3,9] |
| **UV** | MPHO UV (160 lamps per channel for a, b, and c; 5 channels for a, b, and c) | N/A | N/A | 12% | 206a, 225b, 284c HP per channel | 1,030, 1,125, 1,420 HP | [7] |
| **Gravity Thickeners** | Rake Arm (1 rake arm for a, 2 for b, and 3 for c) | Torque | K = 30 (lb/ft), ω = 0.0089 (rad/s) | 75% | 0.69abc HP per rake arm | 0.69, 1.38, 2.07 HP | [10] |
| | Overflow Pumps (1 pump for a, 2 for b, and 3 for c) | Pump | H = 30 ft | 50% | 0.87a, 0.85b, 1.17c HP per pump | 0.9, 1.7, 3.5 HP | [4,8] |
| | Sludge Pumps (1 pump for a, 2 for b, and 3 for c) | Pump | H = 50 ft | 50% | 6.1a, 4.7b, 5.4c HP Per pump | 6.1, 9.4, 16.2 HP | [4,8] |
| **DAFTS** | Recycle Pumps (1 for a, 2 for b, and 3 for c pumps) | Pump | H = 170 ft | 75% | 31a, 30.9b, 39.6c HP Per pump | 31, 62, 119 HP | [2,4] |
| | Rake arms (1 collector for a, 2 for b, and 3 for c) | Torque | K = 4 (lb/ft), ω = 0.014 (rad/s) | 75% | 0.12abc HP per collector | 0.12, 0.24, 0.36 HP | [10] |
| | Sludge Pumps (1 pump for a, 2 for b, and 3 for c) | Pump | H = 50 ft | 50% | 2.8ab, 3.6c HP Per pump | 2.8, 5.6, 10.8 HP | [2,4] |
| | Air Compressors (1 air compressor for a, 2 for b, and 3 for c) | Blower | $q_s$ = 14, 14, 17 (scfm), $T_a$ = 528(°R), $P_b$ = 21.7 psia | 80% | 0.45ab, 0.58c HP per compressor | 0.45, 0.9, 1.74 HP | [1,3,9] |
| | Overflow Pumps (1 pump for a, 2 for b, and 3 for c) | Pump | H = 30 ft | 50% | 3.7ab, 4.7c HP per pump | 3.7, 7.4, 14.1 HP | [2,4] |
| **Centrifuges** | Feed Acceleration (1 centrifuge for a, 2 for b, and 3 for c) | Feed | Q = 145, 110, 126 (gpm), Ω = 1780 (rpm), $r_p$ = 15 (in) | 90% | 81a, 69b, 82c HP Per centrifuge | 81, 138, 246 HP | [6] |

| Energy Component (Number components in Operation) | Equation Type | Equation Parameters | Efficiency | Motor Size | Total Motor Size | Reference |
|---|---|---|---|---|---|---|
| Cake Conveyance (1 centrifuge for a, 2 for b, and 3 for c) | Cake | T = 265,000 (lb-in), Δ = 2 rpm | 90% | 9.3abc HP Per centrifuge | 9.3, 18.6, 27.9 HP | [6] |
| a = low strength, b = average strength, c = high strength | | | | | | |
| [1] = (U.S. EPA, 1989), [2] = (Moyno, 1999), [3] = (Metcalf and Eddy, 2003), [4] = (Mays, 2005), [5] = (WEF, 2005), [6] = (Sieger, et al., 2006), [7] = (Trojan UV, 2007), [8] = (Vaughan, 2009) , [9] = (WEF, 2010a), [10] = (WEF, 2010b), [11] = (Goulds Pumps), [12] = (Madden Manufacturing), [13] = (Vulcan Industries, Inc.) | | | | | | |

APPENDIX B – COMPUTED MOTOR SIZES AND ASSUMED EFFICIENCIES FOR SPECIFIC UNIT PROCESSES IN A 100 MGD WASTEWATER TREATMENT PLANT BASED UPON SWITCHING TRADITIONAL TECHNOLOGIES TO ADVANCED TECHNOLOGIES

| | Energy Consuming Unit | Equation | Equation Parameters | Efficiency | Motor Size | Total Motor Size | Reference for Design |
|---|---|---|---|---|---|---|---|
| **UV** | Low Pressure High Output UV (528 lamps per channel including standby, 352 lamps per channel in operation, 5 channels) | N/A | N/A | 34% | 73.6 HP per channel | 368 HP | [8] |
| | Medium Pressure High Output UV (160 lamps per channel, 5 channels) | N/A | N/A | 12% | 225 HP per channel | 1,125 HP | [7] |
| **Chlorination/ Dechlorination** | Chlorination Chemical Pumps (10 pumps) | N/A | N/A | N/A | 1.5 HP per pump | 15 HP | [11] |
| | Dechlorinator (10 dechlorinators) | N/A | N/A | N/A | 0.04 HP per dechlorinator | 0.4 HP | [1] |
| **Centrifuges** | Feed Acceleration (2 centrifuges) | Feed | Q = 145, 110, 126 (gpm), Ω = 1780 (rpm), $r_p$ = 10 (in) | 75% | 27 HP Per centrifuge | 54 HP | [6] |
| | Cake Conveyance (2 centrifuges) | Cake | T = 132,500 (lb-in), Δ = 5 rpm | 75% | 14 HP Per centrifuge | 28 HP | [6] |
| **Plate and Frame Press** | Sludge Feed Pump (2 pumps) | Pump | Q = 202 gpm, H = 130 ft for 15 min, 260 ft for 30 min, 390 ft for 30 min, 520 ft for 15 min, | 40% | 17 HP for 15 min, 33 HP for 30 min, 50 HP for 30 min, 66 HP for 15 min | 17 HP for 15 min, 33 HP for 30 min, 50 HP for 30 min, 66 HP for 15 min | [3,5] |
| **Aeration Basins (Partial BNR)** | Blowers (10 blowers) | Blower | $q_s$ = 4,200, 79,00, 11,300, (scfm), $T_a$ = 528(°R), $P_b$ = 24.7 psia | 80% | 352.6 HP per blower | 3,526 HP | [2,4,9] |
| | Chemical Pumps (10 pumps) | N/A | N/A | N/A | 1 HP | 10 HP | [11] |

| | Energy Consuming Unit | Equation | Equation Parameters | Efficiency | Motor Size | Total Motor Size | Reference for Design |
|---|---|---|---|---|---|---|---|
| | Mixers (10 mixers) | N/A | 0.53 HP/1000 $ft^3$ | N/A | 43.6 HP | 436 HP | [9] |
| **Aeration Basins (Full BNR)** | Blowers (10 blowers) | Blower | $q_s$ = 4,200, 79,00, 11,300, (scfm), $T_a$ = 528(°R), $P_b$ = 24.7 psia | 80% | 352.6 HP per blower | 3,526 HP | [2,4,9] |
| | Chemical Pumps (10 pumps) | N/A | N/A | N/A | 1 HP | 10 HP | [11] |
| | Mixers (10 mixers) | N/A | 0.53 HP/1000 $ft^3$ | N/A | 43.6 HP | 436 HP | [9] |
| | Mixed Liquor Recycle Pumps (10 pumps) | Pump | H = 30 ft | 75% | 210.6 HP | 2,106 HP | [4,9,10] |

[1] = (WEF, 1982), [2] = (U.S. EPA, 1989), [3] = (Moyno, 1999), [4] = (Metcalf and Eddy, 2003), [5] = (U.S. Army Corps of Engineers, 2003), [6] = (Sieger, et al., 2006), [7] = (Trojan UV, 2007), [8] = (Trojan UV, 2008), [9] = (WEF, 2010a), [10] = (Goulds Pumps), [11] = (Madden Manufacturing)

APPENDIX C – UNIT OPERATION DESIGN METHODS AND ENERGY COMPUTATION EQUATIONS USED

## 1 Bar Racks

Table C.1 summarizes the design criteria used for the bar racks. Relevant design equations for the bar racks include the Manning equation, headloss equation for bar racks, and Kirschmer's equation. The equation that provided the higher headloss was used for the design. The Manning equation is as follows (Mays, 2005):

$$Q=\frac{K}{n}AR^{2/3}S_0^{1/2}$$

where Q is the flow rate, K is 1.49 for US units and 1 for SI units, n is the Manning's roughness coefficient (assumed as 0.015), A is the area, R is the hydraulic radius, and $S_0$ is the slope (assumed as 1%). The bar rack headloss equation is as follows and is valid for both clean screens and clogged screen headloss (Metcalf and Eddy, 2003; WEF, 2010a):

$$h_L=\frac{1}{C}\left(\frac{V^2-v^2}{2g}\right)$$

where $h_L$ is the headloss, C is the discharge coefficient (0.7 for a clean screen and 0.6 for a screen that is clogged), V is velocity through the bar openings, v is the velocity upstream of the bar racks, and g is the gravitational acceleration. The Kirschmer's equation is as follows and is valid only for calculating clean screen headloss (Metcalf and Eddy, 2003; WEF, 2010a):

$$h_L=\beta\left(\frac{w}{b}\right)^{4/3}h\sin\phi$$

where $h_L$ is the headloss, $\beta$ is the bar shape factor, w is the width of the bars, b is the clear spacing of the bars, h is upstream velocity head, and $\varphi$ is the angle from the horizontal.

The effects of different wastewater strength on the bar racks was assumed in the screenings production (low, average, and high) for the bar racks by reviewing data provided by WWTP surveys in literature (WEF, 2010a). Table C.2 shows the screenings for the bar racks. Table C.3 shows the bar rack design. The reason that the peak clogged headloss is less than the average clogged headloss is due to the use of a hydraulic control structure to control the depth of flow during peak (WEF, 2010a). The use of a hydraulic control structure modifies the velocity and the headloss. The energy parameters for the bar racks are shown in Table C.4. The motor size and rake speeds were found from manufacturer literature (Vulcan Industries, Inc.). The time for one raking was found by using the screen length and rake speed. The time between rakings was assumed based upon wastewater strength with guidance from operations literature where a typical time between rakings is given as 15 to 30 minutes (WEF, 2008). Two bar racks were assumed to be in operation for the full average flow.

**Table C.1 - Bar Racks Design Criteria**

| Parameter | Range | Units | Reference |
|---|---|---|---|
| Bar Width | 0.2 – 0.6 in | in | (Metcalf and Eddy, 2003) |
| Bar Depth | 1.0 – 1.5 in | in | (Metcalf and Eddy, 2003) |
| Clear Spacing Between Bars | 0.6 – 3.0 in | in | (Metcalf and Eddy, 2003) |
| | 0.25 - 1.5 in | in | (WEF, 2010a) |
| Slope from vertical | 0 – 30 º | º | (Metcalf and Eddy, 2003) |
| Approach Velocity Maximum | ≤ 3 at Average Flow | ft/s | (GLUMRB, 2004) |
| | 2.0 – 3.25 | ft/s | (Metcalf and Eddy, 2003) |
| Approach Velocity Minimum | ≥ 1.25 at Average Flow | ft/s | (GLUMRB, 2004) |
| | 1.0 – 1.6 | ft/s | (Metcalf and Eddy, 2003) |
| | ≥ 1.3 | ft/s | (WEF, 2010a) |
| Velocity Through Screens | 2 - 4 | ft/s | (WEF, 2010a) |
| Allowable Headloss | 0.5 – 2 | ft | (Metcalf and Eddy, 2003; WEF, 2010a) |

**Table C.2 - Bar Racks Screenings**

| Parameter | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| Wastewater Screenings at average flow | 3.25 | 5.8 | 8 | $ft^3/10^6$ gal |
| Average Screenings | 325 | 580 | 800 | $ft^3$/day |

**Table C.3 - Bar Racks Design**

| Parameter | Value | Units |
|---|---|---|
| Average Flow per Bar Rack | 50 | MGD |
| Peak Flow per Bar Rack | 75 | MGD |
| Number of Bar Racks | 4 | N/A |
| Total Average Flow Capability | 200 | MGD |
| Total Peak Flow Capability | 300 | MGD |
| Screen Length in Channel | 12 | ft |
| Channel Width | 8 | ft |
| Channel Slope | 0.0001 | ft/ft |
| Bar Clear Spacing | 0.5 | in |
| Bar Width | 0.5 | in |
| Angle From Horizontal | 80 | ° |
| Upstream Velocity at Average Flow | 2.19 | ft/s |
| Velocity Through Screen at Average Flow | 3.5 | ft/s |
| Upstream Velocity at Peak Flow | 2.47 | ft/s |
| Velocity Through Screen at Peak Flow | 3.77 | ft/s |
| Headloss at Average Flow | 0.18 | ft |
| Headloss at Average Flow (50% clogged) | 1.13 | ft |
| Headloss at Peak Flow | 0.23 | ft |
| Headloss at Peak Flow (50% clogged) | 1.2 | ft |

**Table C.4 - Bar Racks Energy Parameters**

| Parameters | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| Number of Bar Racks in Service | 2 | 2 | 2 | N/A |
| Required Motor Size | 5 | 5 | 5 | HP |
| Required Motor Size | 3.7 | 3.7 | 3.7 | kW |
| Rake Speed | 30 | 30 | 30 | ft/min |
| Time for One Raking | 0.8 | 0.8 | 0.8 | min |
| Time Between Rakings | 20 | 15 | 10 | min |
| Number of Cycles Per Day | 69 | 91.1 | 133.3 | N/A |
| Total Time For Rakings Per Bar Rack | 0.9 | 1.2 | 1.8 | hours/day |
| Bar Racks Total Energy | 6.66 | 8.88 | 13.32 | kWh/day |
| Bar Racks Total Energy | 0.07 | 0.09 | 0.13 | kWh/MGal |

A summary of the bar rack design procedure is as follows:

1. Determine the number of bars in the bar rack channel. This is calculated as follows (Davis, 2010):

$$N_{\text{bars}} = \frac{\text{width of channel - bar spacing}}{\text{bar width+bar spacing}} = \frac{8-\frac{0.5}{12}}{\frac{0.5}{12}+\frac{0.5}{12}} = 96 \text{ bars}$$

2. Determine the number of bar spacings in the bar rack channel. This is the number of bars plus one (Davis, 2010) or 97 spaces.

3. Calculate the width available for flow through the bar racks:

$$\text{Width} = \text{bar spacing} \times \text{number of bar spacings} = \frac{0.5}{12} \times 97 = 4 \text{ ft.}$$

4. Use the Manning equation to estimate the depth of flow for the average flowrate. Use an iterative process to guess and check the depth of flow until the Manning equation is equal to the flowrate. At the average flowrate, the 50 MGD flowrate is equal to 77.4 cfs. The area of a rectangular channel is equal to BY where B is the width of the channel and Y is the depth of flow. The hydraulic radius is equal to $\frac{BY}{B+2Y}$ (Mays, 2005). Using the iterative process yielded a depth of flow of 5.55 ft at the average flowrate.

$$Q = \frac{K}{n}(BY)\left(\frac{BY}{B+2Y}\right)^{2/3} S_0^{1/2} = \frac{1.49}{0.015}(8\times5.55)\left(\frac{8\times5.55}{8+2\times5.55}\right)^{2/3} 0.0001^{1/2}$$

$$Q = 77.4 \text{ cfs}$$

5. Repeat number 4 for the peak flowrate. This yields a depth of flow of 7.7 ft.

6. Calculate the upstream velocity at the average flowrate using the Manning equation (Mays, 2005).

$$V = \frac{K}{n} R^{2/3} S_0^{1/2} = \frac{K}{n} \left(\frac{BY}{B+2Y}\right)^{2/3} S_0^{1/2}$$

$$V = \frac{1.49}{0.015} \left(\frac{8\times5.55}{8+2\times5.55}\right)^{2/3} 0.0001^{1/2} = 2.2 \text{ ft/s}$$

This velocity is acceptable.

7. Repeat number 6 for the peak flowrate. This yields an upstream velocity of 2.5 ft/s. This velocity is acceptable.
8. Calculate the area available for flow through the bar racks at average flow.

   $A=5.55\times4=22.2\text{ft}^2$

9. Repeat number 8 for the peak flowrate. This yields 30.8 $\text{ft}^2$.
10. Calculate the velocity through the bar racks at the average flowrate.

    $Q=AV\text{->}V=\frac{Q}{A} = \frac{77.4}{22.2} = 3.5\text{ft/s}$

    This velocity is acceptable.

11. Repeat number 10 for the peak flowrate. This yields 3.8 ft/s. This velocity is acceptable.
12. Calculate the headloss for the average flowrate. Use the Kirschmer equation and the headloss equation. Use the equation that provides the higher headloss. Assume $\beta$=2.42 for the Kirschmer equation.

    For the Kirschmer equation: $h_L=2.42 \left(\frac{0.5/12}{0.5/12}\right)^{4/3} \frac{2.5^2}{2\times32.2} \sin 80 = 0.18 \text{ ft}$

    For the headloss equation: $h_L=\frac{1}{C}\left(\frac{V^2\text{-}v^2}{2g}\right) = \frac{1}{0.7}\left(\frac{3.5^2\text{-}2.2^2}{2\times32.2}\right) = 0.16 \text{ ft}$

    The Kirschmer equation controls and provides an acceptable headloss.

13. Repeat number 12 for the peak flowrate. This provides a headloss of 0.23 ft which is acceptable.

14. Calculate the 50% clogged headloss for the average flowrate. Use the headloss equation only as the Kirschmer equation is only valid for clean screens.

$$h_L=\frac{1}{C}\left(\frac{V^2\text{-}v^2}{2g}\right)=\frac{1}{0.6}\left(\frac{(2\times 3.5)^2\text{-}2.2^2}{2\times 32.2}\right)=1.13\text{ ft}$$

This headloss is acceptable.

15. Repeat 14 for the peak flowrate. This yields an acceptable headloss value of 1.2 ft.

The energy was calculated for the bar racks as follows:

1. The motor sizing was provided as 5 HP (3.73 kW) by manufacturer literature and is a function of channel size (Vulcan Industries, Inc.). The rake speed was also provided by manufacturer literature (Vulcan Industries, Inc.).
2. The time between rakings was assumed using ranges provided in operations literature (WEF, 2008).
3. The time for one raking was calculated using the screen length and rake speed. The rake must move twice.

$$\text{Time}=\frac{\text{screen length}}{\text{rake speed}}\times 2=\frac{12}{30}\times 2=0.8\text{ min}$$

4. The total number of rake cycles for the average flow assuming a 15 minute interval is $\frac{24\times 60}{15+0.8}=91$ cycles.
5. The total raking time per day for the average strength is $91\ \times 0.8=\ 72.8$ min = 1.2 hours/day.
6. The energy consumption per day for one bar rack at average strength is $3.73\ \times 1.2=\ 4.5$ kWh/day.

7. The energy consumption for the two operating bar racks per is 9 kWh/day for the average strength.

## 2 Aerated Grit Chambers

Table C.5 summarizes the design criteria used for the aerated grit chambers. The main design equation for the aerated grit chambers is the hydraulic retention time which is as follows (Metcalf and Eddy, 2003):

$$\text{HRT} = \frac{V}{Q}$$

Where HRT = hydraulic retention time (min), V = tank volume ($ft^3$), and Q = the influent flow ($ft^3$/min).

The effects of different wastewater strength on the aerated grit chamber was assumed in the grit production (low, average, and high) for the aerated grit chamber by reviewing data provided in literature (Metcalf and Eddy, 2003). It was assumed that more grit production would require more air. Table C.6 shows the grit production and air requirements assumed for the aerated grit chambers. Table C.7 shows the aerated grit chamber design. The energy parameters for the aerated grit chambers (blowers) are shown in Table C.8.

The energy requirements for the aerated grit chambers come from the blowers. The blower energy equation is as follows (U.S. EPA, 1989):

$$\text{WP} = \left(4.28 \times \frac{10^{-4} q_s T_a}{e}\right) [(P_d/P_b)^{0.283} - 1]$$

Where WP = wire power consumption (HP, multiply by 0.746 for kW), $q_s$ = airflow rate (scfm), $T_a$ = intake temperature (°R), e = combined efficiency, $P_d$ = blower discharge pressure (psia), $P_b$ = blower intake pressure (always assumed as 14.7 psia for all blowers).

**Table C.5 - Aerated Grit Chambers Design Criteria**

| Parameter | Range | Units | Reference |
|---|---|---|---|
| Hydraulic Retention Time (HRT) at Peak Flow | 2 - 5 | min | (Metcalf and Eddy, 2003) |
| | 3 - 5 | min | (GLUMRB, 2004) |
| | 3 - 10 | min | (WEF, 2010a) |
| Depth | 7 – 16 | ft | (Metcalf and Eddy, 2003) |
| | 12 – 16 | ft | (WEF, 2010a) |
| Length | 25 – 65 | ft | (Metcalf and Eddy, 2003) |
| Width | 8 – 23 | ft | (Metcalf and Eddy, 2003) |
| Width-Depth Ratio | 1:1 – 5:1 | N/A | (Metcalf and Eddy, 2003) |
| | 0.8:1 - 1:1 | N/A | (WEF, 2010a) |
| Length-Width Ratio | 3:1 – 5:1 | N/A | (Metcalf and Eddy, 2003) |
| | 3:1 - 8:1 | N/A | (WEF, 2010a) |
| Air Supply | 3 – 8 | $ft^3$/ft-min | (Metcalf and Eddy, 2003; GLUMRB, 2004; WEF, 2010a) |
| Grit Volume | 0.5 – 27 | $ft^3$/MGal | (Metcalf and Eddy, 2003) |
| | 0.5 - 20 | $ft^3$/MGal | (WEF, 2010a) |

**Table C.6 - Grit Chamber Grit**

| Parameter | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| Wastewater Grit at Average Flow | 4.48 | 15.1 | 24.4 | $ft^3/10^6$ gal |
| Average Grit | 448 | 1,510 | 2,440 | $ft^3$/day |
| Airflow Requirements | 3.75 | 5.75 | 7.5 | $ft^3$/ft-min |

### Table C.7 - Aerated Grit Chamber Design

| Parameter | Value | Units |
|---|---|---|
| Average Flow per Aerated Grit Chamber | 25 | MGD |
| Peak Flow per Aerated Grit Chamber | 37.5 | MGD |
| Number of Grit Chambers | 8 | N/A |
| Total Average Flow Capability | 200 | MGD |
| Total Peak Flow Capability | 300 | MGD |
| Length | 58 | ft |
| Width | 16 | ft |
| Depth | 16 | ft |
| HRT at Peak Flow | 4.27 | min |
| Length to Width Ratio | 3.625 | N/A |
| Width to Depth Ratio | 1 | N/A |

### Table C.8 - Aerated Grit Chambers Energy Parameters

| Parameters | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| Number of Aerated Grit Chambers in Service | 4 | 4 | 4 | N/A |
| Air Supply per Unit Length | 3.75 | 5.75 | 7.5 | $ft^3$/ft-min |
| Air Requirements | 217.5 | 333.5 | 435 | $ft^3$/min |
| Blower Inlet Pressure | 1 | 1 | 1 | atm |
| Blower Outlet Pressure | 1.34 | 1.34 | 1.34 | atm |
| Blower Efficiency | 80% | 80% | 80% | N/A |
| Blower Temperature | 110 | 110 | 110 | °F |
| Blower Energy Requirements | 5.7 (4.3) | 8.8 (6.6) | 11.5 (8.6) | HP (kW) |
| Blower Power Requirement Per Channel | 102.6 | 157.3 | 205.2 | kWh/day |
| Total Blower Power Requirement | 410.4 | 629.3 | 820.9 | kWh/day |
| Total Blower Power Requirement | 4.1 | 6.29 | 8.21 | kWh/MGal |

A summary of the aerated grit chamber design procedure is as follows:

1. Calculate the HRT of the aerated grit chamber at peak flow:

$$\mathrm{HRT} = \frac{V}{Q} = \frac{58 \times 16 \times 16}{37.5 \times 1.547} = 256\ \mathrm{s} = 4.27\ \mathrm{min}$$

This is an acceptable value.

2. Calculate the length to width ratio:

$$\text{length to width ratio} = \frac{\text{length}}{\text{width}} = \frac{58}{16} = 3.625$$

This is an acceptable value.

3. Calculate the width to depth ratio:

$$\text{width to depth ratio}=\frac{\text{width}}{\text{depth}}=\frac{16}{16}=1$$

This is an acceptable value.

The energy was calculated for the aerated grit chambers as follows:

1. The air supply per unit length for the average flowrate was 5.75$ft^3$/ft-min. At a length of 58 feet for the grit chamber, this is a total air requirement of 333.5 $ft^3$/min for each grit chamber.
2. A 5 psi drop was assumed between the blower and coarse bubble diffuser.
3. An efficiency of 80% was assumed for the blowers which is typical (Metcalf and Eddy, 2003).
4. A temperature of 110°F was assumed for the blower temperature which is a typical summer temperature in the southwestern United States. This also assumes that the blower is outside.

5. The blower energy equation is used to calculate the energy requirements for a grit chamber:

$$WP = \left(4.28 \times \frac{10^{-4} q_s T_a}{e}\right)[(P_d/P_b)^{0.283} - 1]$$

$$WP = \left(\frac{4.28 \times 10^{-4} \times 333.5 \times 570}{0.8}\right)[(1.34/1)^{0.283} - 1] = 8.79\ HP = 6.6\ kW$$

This amounts to 157.3 kWh/day per grit chamber for the average strength. With four operating grit chambers the total energy requirements would be 629.3 kWh/day for the grit chambers.

## 3 Primary Clarifiers

Table C.9 summarizes the design criteria used for the primary clarifiers. The main equations used for the design of the primary clarifiers were the HRT, the OFR, and weir loading. Recommendations from (WEF, 2005) were used to determine the diameter of the clarifier based upon the depth. Energy consumers for the primary clarifiers include sludge pumping, and torque to power the rake arms. The brake horsepower (BHP) equation (Jones, et al., 2008) was used to compute the energy requirements of the pumps. The BHP equation is as follows:

$$BHP = \frac{QH}{3{,}960e}$$

Where BHP = brake horsepower (HP multiply by 0.746 for kW), Q = flow rate (gpm), H = pump head (ft), and e = efficiency. The time between pumping cycles was assumed as 20, 15, and 10 minutes for the low, average, and high strength cases, respectively. The pumping time was assumed as three minutes for all strength cases. The energy required to drive the rake arms (WEF, 1982; WEF, 2005) was calculated as follows:

$$P = Wr^2\omega/550e$$

Where P = power required for rake arms (HP multiply by 0.746 for kW), W = arm loading factor (lb/ft), r = radius of tank (ft), ω = angular velocity (rad/s), and e = efficiency.

The effects of different wastewater strength on the primary clarifiers come from increased sludge as wastewater strength increases. TSS and BOD removals were calculated from (Metcalf and Eddy, 2003). Table C.10 shows the primary sludge production calculated. The TSS and BOD values in the influent are higher than the influent values in Table C.11 due to solids stream recycles. Table C.11 shows the primary clarifier design. The energy parameters for the primary clarifiers are shown in Table C.12.

**Table C.9 - Primary Clarifiers Design Criteria**

| **Parameter** | **Range** | **Units** | **Reference** |
|---|---|---|---|
| Hydraulic Retention Time (HRT) | 1.5 - 2.5 | hours | (Metcalf and Eddy, 2003) |
| Overflow Rate (OFR) at Average Flow | 800 – 1,200 | gal/ft$^2$-day | (Metcalf and Eddy, 2003) |
| Overflow Rate (OFR) at Peak Flow | 2,000 – 3,000 | gal/ft$^2$-day | (Metcalf and Eddy, 2003) |
| Weir Loading | 10,000 – 40,000 | gal/ft-day | (Metcalf and Eddy, 2003) |
| Depth | 10 - 16 | ft | (Metcalf and Eddy, 2003) |
| Diameter | 10 - 200 | ft | (Metcalf and Eddy, 2003) |
| Bottom Slope | 3/4 - 2 | in/ft | (Metcalf and Eddy, 2003) |
| Rake Arm Speed | 0.02 - 0.05 | rpm | (Metcalf and Eddy, 2003) |
| | 6 - 12 | ft/min | (WEF, 2010a) |

**Table C.10 - Primary Sludge Production**

| Parameter | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| TSS in Primary Clarifier Influent due to Recycle Off Other Processes including Solids Recycle | 133 | 231 | 438 | mg/L |
| BOD in Primary Clarifier Influent due to Recycle Off Other Processes including Solids Recycle | 116 | 200 | 367 | mg/L |
| TSS in Primary Clarifier Effluent at Average Flowrate | 55 | 95 | 181 | mg/L |
| TSS in Primary Clarifier Effluent at Peak Flowrate | 61 | 106 | 202 | mg/L |
| BOD in Primary Clarifier Effluent at Average Flowrate | 73 | 127 | 232 | mg/L |
| BOD in Primary Clarifier Effluent at Peak Flowrate | 78 | 135 | 248 | mg/L |
| TSS Percent Removal At Average Flow | 59 | 59 | 59 | % |
| TSS Percent Removal at Peak Flow | 54 | 54 | 54 | % |
| BOD Percent Removal at Average Flow | 37 | 37 | 37 | % |
| BOD Percent Removal at Peak Flow | 33 | 33 | 33 | % |
| Primary Sludge per Clarifier | 6,530 | 11,470 | 21,900 | lb/day |
| Total Primary Sludge | 65,300 | 114,700 | 219,000 | lb/day |

**Table C.11 - Primary Clarifier Design**

| Parameter | Value | Units |
|---|---|---|
| Average Flow per Primary Clarifier | 10 | MGD |
| Peak Flow per Primary Clarifier | 15 | MGD |
| Number of Primary Clarifiers | 14 | N/A |
| Total Average Flow Capability | 140 | MGD |
| Total Peak Flow Capability | 210 | MGD |
| Diameter | 120 | ft |
| Depth | 12.5 | ft |
| HRT at Average Flow | 2.5 | hours |
| HRT at Peak Flow | 1.7 | hours |
| OFR at Average Flow | 884 | gal/ft$^2$-day |
| OFR at Peak Flow | 1,330 | gal/ft$^2$-day |
| Weir Type | V-Notch | N/A |
| Weir Center to Center Spacing | 8 | in |
| Individual Weir Length | 6 | in |
| Weir Face to Face Spacing | 2 | in |
| Weir Depth | 3 | in |
| Number of V-Notches W | 565.00 | N/A |
| Weir Length | 376.7 | ft |
| Weir Loading at Average Flow | 26,550 | gal/ft-day |
| Weir Loading at Peak Flow | 39,800 | gal/ft-day |
| Rake Arm Speed | 0.03 | rpm |

**Table C.12 - Primary Clarifiers Energy Parameters**

| Parameters | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| Total Number of Operating Primary Clarifiers | 10 | 10 | 10 | N/A |
| Pump Total Dynamic Head (TDH) | 60 | 60 | 60 | ft |
| Pump Flow Rate | 88 | 103 | 118 | gpm |
| Pump Cycle | 20 mins off, 3 mins on | 15 mins off, 3 mins on | 10 mins off, 3 mins on | N/A |
| Pump Efficiency | 50% | 50% | 50% | N/A |
| Pump Energy Per Clarifier | 2.7 (2.0) | 3.1 (2.3) | 3.6 (2.7) | HP (kW) |
| Total Energy For Pumping | 68.5 | 93.2 | 147.9 | kWh/day |
| Rake Arm Loading Factor, W | 8 | 9 | 10 | lb/ft |
| Rake Arm Angular Velocity, ω | 0.0033 | 0.0033 | 0.0033 | rad/s |
| Rake Arm Efficiency | 75% | 75% | 75% | N/A |
| Rake Arm Energy Per Clarifier | 0.24 (0.18) | 0.26 (0.2) | 0.29 (0.22) | HP (kW) |
| Total Rake Arm Energy | 46.3 | 47.4 | 52.7 | kWh/day |
| Total Energy | 114.9 | 140.6 | 200.5 | kWh/day |
| Total Energy | 1.1 | 1.4 | 2.0 | kWh/MGal |

A summary of the aerated girt chamber design procedure is as follows:

1. Calculate the HRT (Metcalf and Eddy, 2003) of the primary clarifiers at the average flowrate:

$$\mathrm{HRT}=\frac{\mathrm{V}}{\mathrm{Q}}=\frac{\frac{\pi}{4}\times 120^2\times 12.5}{10\times 1.547}=9140\ \mathrm{s}=2.5\ \mathrm{hours}$$

   This value is acceptable.

2. Repeat number 1 for the peak flowrate. This yields a value of 1.7 hours which is acceptable.

3. Calculate the overflow rate for the average flowrate (Metcalf and Eddy, 2003):

$$\mathrm{OFR}=\frac{\mathrm{Q}}{\mathrm{A}}=\frac{10\times 10^6}{\frac{\pi}{4}\times 120^2}=884\ \mathrm{gal/ft^2\text{-}day}$$

   This value is acceptable.

4. Repeat number 3 for the peak flowrate. This yields a value of 1,330 gal/ft$^2$-day which is acceptable.

5. Calculate the number of v-notches for the outlet weir on the primary clarifier:

$$\text{V-notches}=\frac{\pi\times 120}{\frac{8}{12}}=565$$

6. Calculate the weir length:

$$\text{Weir length}=565\times\frac{8}{12}=376.7\text{ ft}$$

7. Calculate the weir loading at the average flowrate (Metcalf and Eddy, 2003):

$$\text{WL}=\frac{Q}{L}=\frac{10\times 10^6}{376.7}=26{,}550\text{ gal/ft-day}$$

This value is acceptable.

8. Repeat number 7 for the peak flowrate. This yields a value of 39,800 gal/ft-day which is acceptable.

9. Calculate the TSS removed at average flow for the average strength (Metcalf and Eddy, 2003):

$$\text{TSS Removed}=231\times 0.59=141\text{ mg/L}$$

For the average flow rate and average strength, 95 mg/L TSS leaves the primary clarifiers.

10. Repeat number 9 for the low and high strengths at the average flow. This yields 55 mg/L for the low strength and 181 mg/L for the high strength.

11. Repeat numbers 9 and 10 for the peak flow for the low, average, and high strengths. This yields 61, 106, and 202 mg/L for the low, average, and high strengths at the peak flow.

12. Repeat 9 – 11 for BOD at average and peak flow for the low, average, and high strength wastewaters. For the average flow rate: 73, 127, 232 mg/L for the low, average, and high strength wastewater cases. For the peak flow rate: 78, 135, and 248 mg/L for the low, average, and high strength wastewater cases.
13. Calculate the sludge production rate per primary clarifier for the average strength at the average flowrate. This is based around TSS removed (Qasim, 1999).

    Sludge production rate$= (10\times231\times8.34)\times0.59=11{,}400\ \mathrm{lb/day}$ per primary clarifier
14. Repeat 13 for the average flow rate for the low and high strengths. This yields 6,500 and 21,900 lb/day per primary clarifier for the low and high strengths.

The energy for the pumps in the primary clarifiers was calculated as follows:

1. Calculate the sludge volume at average flow for the average strength (Qasim, 1999):

    $$\text{Sludge volume}=\frac{11{,}400}{(1.03\times62.4\times0.045\times60\times24)}=2.8\ \mathrm{ft}^3/\mathrm{min}$$
2. Repeat number 1 for the low and high strengths. This yields 1.8 and 4.7 ft3/min for the low and high strengths.
3. Calculate the required sludge flow rate for the primary clarifiers at average strength (Qasim, 1999):

    $$\text{Sludge flow rate: }\frac{2.8\times15}{3}\times7.48=103\ \mathrm{gpm}$$
4. Repeat 3 for the low and high strengths. This yields 88 and 118 gpm for the low and high strengths.

5. Calculate the energy per pump for the average strength using the brake horsepower equation (Jones, et al., 2008). Assume an efficiency of 40% that is typical for a primary clarifier pump (Vaughan, 2009). Assume a TDH of 60 feet.

$$\mathrm{BHP}=\frac{\mathrm{QH}}{3{,}960\mathrm{e}}=\frac{103\times 60}{3{,}960\times 0.4}=3.9\ \mathrm{HP}=2.9\ \mathrm{kW}$$

6. Repeat number 5 for the low and high strengths. This yields 2.5 kW and 3.3 kW.

7. Calculate the number of pumping cycles per day for the average strength:

$$\text{Pumping cycles} = \frac{24 \times 60}{15+3} = 80$$

8. Repeat number 7 for the low and high strengths. This yields 63 and 111 cycles per day for the low and high strengths.

9. Calculate how many hours the pumps pump for the average strength:

$$\text{Pumping hours}=80\times 3=240\ \text{minutes}=4\ \text{hours}$$

10. Repeat number 9 for the low, and high strengths. This yields 3.1 and 5.5 hours.

11. Calculate the total pumping energy per day for the average strength:

$$\text{Pumping energy}=10\times 2.9\times 4=116\ \mathrm{kWh/day}$$

12. Repeat number 11 for the low, and high strengths. This yields 86 and 185 kWh/day for the low and high strengths.

The calculation of the rake arm energy is as follows:

1. Calculate the rake arm energy using the rake arm energy equation for the average strength (WEF, 1982; WEF, 2005). Assume an efficiency of 75%.

$$\mathrm{P}=\frac{\mathrm{Wr}^2\omega}{550\mathrm{e}}=\frac{9\times\left(\frac{120}{2}\right)^2\times 0.0033}{550\times 0.75}=0.26\ \mathrm{HP} = 0.20\ \mathrm{kW}$$

2. Repeat number 1 for the low and high strengths. This yields 0.18 and 0.22 kW for the low and high strengths.
3. Calculate the total energy requirements per day for the rake arms at average strength:

   $\text{Rake arm energy} = 0.20 \times 10 \times 24 = 47.4 \text{ kWh/day}$

4. Repeat number 3 for the low and high strengths. This yields 46.3 and 52.7 kWh/day.

# 4 Aeration Basins

Table C.13 shows the microbial parameters for the aeration basins. Table C.14 shows the aeration basins design. Each aeration basin had a design flow of 10 MGD with the exception of the high strength wastewater in Chapter 2 that have a design flow of 8.5 MGD due to very high substrate loadings. The major difference between the aeration basins in Chapter 2 and Chapter 3 is the inclusion of an internal recycle pump in Chapter 3. The inclusion of an internal recycle pump allows for full biological nutrient removal (BNR).

There are many design equations and procedures described in references such as Metcalf and Eddy (2003), Rittman, et al. (2000) and WEF (2010a). Some of the equations are described below.

The hydraulic retention time is as follows (Metcalf and Eddy, 2003):

$$\text{HRT} = \frac{V}{Q}$$

Where HRT = hydraulic retention time, V = volume of aeration basin, and Q = flow rate.

The solids retention time is as follows (Rittman, et al., 2000):

$$\mathrm{SRT} = \frac{X_a V}{X_a^e Q^e + X_a^w Q^w}$$

Where SRT = solids retention time, $X_a$ = concentration of active biomass, $X_a^e$ = effluent concentration of active biomass, $Q^e$ = effluent flowrate, $X_a^w$ = waste concentration of active biomass, $Q^w$ = wastage flowrate.

The minimum solids retention time to prevent washout is as follows (Rittman, et al., 2000):

$$\mathrm{SRT}_{min} = \frac{K + S_0}{S_0(Y\mu_m - k_d) - Kk_d} \qquad S \rightarrow S_0$$

Where K = half-velocity constant, Y = yield, $S_0$ = influent concentration of substrate, $\mu_m$ = maximum specific growth rate.

The remaining substrate after treatment is as follows (Rittman, et al., 2000):

$$S = K\frac{1 + b\mathrm{SRT}}{\mathrm{SRT}(Y\mu_m - b) - 1}$$

Where S = substrate remaining after treatment.

The lowest concentration the substrate can reach after treatment is as follows (Rittman, et al., 2000):

$$S_{min} = K\frac{b}{Y\mu_m - b} \qquad \mathrm{SRT} \rightarrow \infty$$

Where $S_{min}$ = lowest theoretical substrate remaining after treatment.

The concentration of active biomass is as follows (Rittman, et al., 2000):

$$X_a = \frac{\mathrm{SRT}[Y(S^0 - S)]}{\theta(1 + b\mathrm{SRT})}$$

The concentration of inert biomass is as follows (Rittman, et al., 2000):

$$X_i = \frac{\mathrm{SRT}}{\theta}[X_i^0 + X_a(1 - f_d)b\theta]$$

Where $X_i$ = concentration of inert biomass, $X_i^0$ = initial concentration of inert biomass.

The total mixed liquor suspended solids in the aeration basin is as follows (Rittman, et al., 2000):

$$X_v = \frac{SRT}{\theta}\left[X_i^0 + \frac{Y(S^0 - S)(1 + (1 - f_d)bSRT)}{1 + bSRT}\right]$$

Where $X_v$ = mixed liquor suspended solids.

**Table C.13 - Aeration Basins Microbial Parameters**

| Parameter | BOD Microbes | Nitritation Microbes | Nitratation Microbes | Phosphorous Accumulating Organisms (PAOs) | Denitrification Microbes |
|---|---|---|---|---|---|
| Half-Velocity constant, K (mg donor/L) | 10 [1] | 1 [1] | 1.3 [1] | 1 [1] | 12.6 [1] |
| Yield, Y (g VSS/g donor) | 0.4 [1] | 0.33 [1] | 0.083 [1] | 0.3 [2] | 0.26 [1] |
| Maximum Specific Growth Rate, $\mu_m$ (g VSS/g VSS -d) | 9 [1] | 0.76 [1] | 0.81 [1] | 0.95 [2,3] | 3.12 [1] |
| Endogenous Decay Coefficient, $k_d$ (g VSS/g VSS-d) | 0.15 [1] | 0.11 [1] | 0.11 [1] | 0.04 [3] | 0.05 [1] |
| $f_d$ | 0.8 [1] | | | | |
| [1] = (Rittman, et al., 2000), [2] = (Metcalf and Eddy, 2003), [3] = (WEF, 2010c) | | | | | |

**Table C.14 - Aeration Basins Design**

| | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| Number of Aeration Basins | 10 | 10 | 12 | N/A |
| Anoxic/Anaerobic Zone Length | 102 | 102 | 102 | ft |
| Anoxic/Anaerobic Zone Depth | 18 | 18 | 18 | ft |
| Anoxic/Anaerobic Zone Width | 30 | 30 | 30 | ft |
| Aerobic Zone Length | 420 | 420 | 420 | ft |
| Aerobic Zone Depth | 18 | 18 | 18 | ft |
| Aerobic Zone Width | 30 | 30 | 30 | ft |

| | | | | |
|---|---|---|---|---|
| Solids Retention Time (SRT) | 20 | 10 | 8 | days |
| Mixed Liquor Suspended Solids | 2,957 | 3,035 | 3,880 | mg/L |
| Mixed Liquor Suspended Solids for Full BNR | N/A | 3,408 | N/A | mg/L |
| Internal Recycle Rate for Full BNR Case | N/A | 300 | N/A | % |

As the procedure for describing activated sludge design is very complex, a general outline is provided instead:

1. Assume an HRT for the anoxic and anaerobic zones of the aeration basins. In the case of this research, the HRT is roughly one hour.
2. Assume an SRT. This value can be modified as necessary.
3. Determine the amount of volatile fatty acids entering the aeration basin. This is an important factor for the polyphosphate accumulating organisms that utilize phosphorous.
4. Determine the growth of the PAOs using the above equations as described in (Rittman, et al., 2000) and growth kinetics. Keep in mind that the above equations are for a complete mix reactor. This research assumed a plug flow reactor. The values for the amount of biomass that form are not significantly different between plug flow and complete mix reactors. The substrate value can however go below the minimum substrate value described above for a plug flow reactor. Also keep in mind that the PAOs take up phosphorous in the aerobic zone.
5. Determine the growth of the BOD, nitritation, and nitration organisms using the procedure described in (Metcalf and Eddy, 2003; Rittman, et al., 2000). These

procedures also describe how to determine the aerobic zone volume. Make sure to take into account nutrient utilization by microbes.

6. Determine the growth of the denitrifying organisms using the procedure described in (Metcalf and Eddy, 2003). Make sure to properly address issues such as internal recycle.
7. Repeat the process until a satisfactory design is reached.
8. Determine air flow requirements using the procedure described in (Metcalf and Eddy, 2003; Rittman, et al., 2000).
9. Calculate the alum requirements for further phosphorous removal (Metcalf and Eddy, 2003).

The blower energy requirements are calculated as follows. Table C.15 summarizes the aeration blower energy parameters.

1. The oxygen requirements and airflow requirements were calculated using the procedure in Rittman, et al. (2000) and Metcalf and Eddy (2002). The oxygen requirements are based around microbial oxygen requirements. An α value of 0.5 was assumed which is a conservative value; the range of α values vary from 0.5 to 0.75 (Rosso, et al., 2007).
2. Calculate the blower energy requirements for the average strength (U.S. EPA, 1986). Assume a headloss of 10 psi.

$$\mathrm{WP} = \left(\frac{4.28 \times 10^{-4} \times 7{,}888 \times 570}{0.8}\right)\left[(1.68/1)^{0.283} - 1\right] = 352.6\ \mathrm{HP} = 263\ \mathrm{kW}$$

3. Repeat number 2 for the low and high strengths. This yields 140.1 and 398.1 kW.
4. Calculate the energy requirements for the average strength:

$263 \times 24 \times 10 = 63{,}128.9$ kWh/day

5. Repeat number 4 for the low and high strengths. This yields 336.2 and 1,081 kWh/day.

**Table C.15 – Aeration Basin Blower Energy Parameters**

| | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| Input $O_2$ Requirements | 8,213.5 | 15,276.2 | 23,144.3 | kg OD/day |
| Soluble Output $O_2$ Equivalents | 45.3 | 45.8 | 38.5 | kg OD/day |
| Solid Output $O_2$ Equivalents | 2,148.8 | 3,929.0 | 6,978.6 | kg OD/day |
| Oxygen Requirements | 6,019.4 | 11,301.4 | 16,127.2 | kg OD/day |
| Oxygen Requirements | 250.8 | 470.9 | 672.0 | kg OD/hour |
| $C_{20}$ | 9.08 | 9.08 | 9.08 | mg/L |
| $P_b/P_a$ | 0.93 | 0.93 | 0.93 | N/A |
| $C_{s,T,H}$ | 8.46 | 8.46 | 8.46 | mg/L |
| $P_{atm,H}$ | 9.64 | 9.64 | 9.64 | m |
| Diffuser Height From Bottom of Tank | 0.61 | 0.61 | 0.61 | m |
| Tank Height | 5.49 | 5.49 | 5.49 | m |
| Assumed Oxygen Percentage | 0.19 | 0.19 | 0.19 | N/A |
| DO In Aeration Basins | 2 | 2 | 2 | mg/L |
| $C_{s,T,H}$ | 10.19 | 10.19 | 10.19 | mg/L |
| $\alpha$ | 0.5 | 0.5 | 0.5 | N/A |
| $\beta$ | 0.95 | 0.95 | 0.95 | N/A |
| F | 0.9 | 0.9 | 0.9 | N/A |
| SOTR | 674 | 1,266 | 1,807 | kg/hour |
| Assumed $O_2$ Transfer Efficiency | 0.35 | 0.35 | 0.35 | N/A |
| Inlet Pressure | 1 | 1 | 1 | atm |
| Assumed Headloss | 10 | 10 | 10 | psi |
| Outlet Pressure | 1.68 | 1.68 | 1.68 | atm |
| Assumed Blower Temperature | 68 | 68 | 68 | °F |
| Assumed Blower Efficiency | 0.8 | 0.8 | 0.8 | N/A |
| Air Density | 1.204 | 1.204 | 1.204 | $kg/m^3$ |
| Air Flowrate | 7,137 | 13,401 | 19,123 | $m^3$/hour |
| Air Flowrate | 119 | 223 | 319 | $m^3$/minute |
| Air Flowrate | 4,202 | 7,888 | 11,257 | $ft^3$/minute |
| Blower Power Requirement | 187.8 | 352.6 | 533.7 | HP |
| Blower Power Requirement | 140.1 | 263.0 | 398.1 | kW |
| Blower Power Requirement | 33,623.7 | 63,128.9 | 108,102.1 | kWh/day |
| Blower Power Requirement | 336.2 | 631.3 | 1,081.0 | kWh/day/MGD |

The mixer energy requirements are calculated as follows. Table C.16 summarizes the aeration basins mixers requirements.

1. An estimate of mixing energy requirements is 14 W/m$^3$ (0.4 W/ft$^3$) (WEF, 2010a).
2. Multiplying by the volume of the anaerobic and anoxic zones yields 104.8 kWh/MGal for the low and average strengths, and 125.76 kWh/MGal.

**Table C.16 - Aeration Basin Mixer Energy Parameters**

| | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| Mixer Power Requirements per Volume | 14 | 14 | 14 | W/m$^3$ |
| Mixer Power Requirements per Volume | 0.40 | 0.40 | 0.40 | W/ft$^3$ |
| Mixer Power Energy Requirements | 10,479.6 | 10,479.6 | 12,575.5 | kWh/day |
| Mixer Power Energy Requirements | 104.8 | 104.8 | 125.8 | kWh/day |

The chemical pump energy requirements were calculated as follows. Table C.17 summarizes the pump energy requirements.

1. Calculate the pump energy requirments for the average strength. The motor size is 1 HP (Madden Manufacturing).
2. The energy requirements were calculated as follows for the average strength:

   $1 \times 0.746 \times 24 \times 7 = 124 \text{ kWh/day}$
3. Repeat number 2 for the low and high strength. This yields 54 and 161 kWh/day.

**Table C.17 - Aeration Basin Chemical Pump Energy Requirements**

| | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| Power Input per Chemical Pump | 1 | 1 | 1 | HP |
| Power Input per Chemical Pump | 0.7 | 0.7 | 0.7 | kW |
| Required Number of Chemical Pumps | 3 | 7 | 9 | N/A |
| Pumping Energy Requirements | 54 | 124 | 161 | kWh/day |

The internal recycle pump energy requirements were calculated as follows for the full BNR case. Table C.18 summarizes the internal recycle pump energy parameters.

1. Calculate the energy requirements for the internal recycle pumps:

$$\mathrm{BHP}=\frac{\mathrm{QH}}{3{,}960\mathrm{e}}=\frac{20{,}833\times 3}{3{,}960\times 0.75}=21\ \mathrm{HP}=15.7\ \mathrm{kW}$$

2. Calculate the energy requirements per day for the internal recycle pumps:

$$15.7\times 24\times 10=3{,}771\ \mathrm{kWh/day}$$

**Table C.18 - Aeration Basins Full BNR Internal Recycle Pumps Energy Parameters**

| | | |
|---|---|---|
| Internal Recycle Flowrate | 20833 | gpm |
| Pump Specific Weight γ | 62.40 | $lb/ft^3$ |
| Provided TDH for Pump | 3 | ft |
| Provided Efficiency for Pump | 75% | N/A |
| Power Input for Pumps Per Clarifier | 21 | HP |
| Power Input for Pumps Per Clarifier | 15.7 | kW |
| Total Power Input | 3,771 | kWh/day |

## 5 Secondary Clarifiers

Table C.19 summarizes the design criteria used for the secondary clarifiers. The main equations used for the design of the secondary clarifiers were the OFR and solids loading rate. Recommendations from (WEF, 2005) were used to determine the diameter of the clarifier based upon the depth. Energy consumers for the secondary clarifiers include return activated sludge (RAS) pumping, waste activated sludge (WAS) pumping, and torque to power the rake arms. The solids loading rate equation is as follows (Metcalf and Eddy, 2003):

$$SLR=\frac{(1+R)QX}{A}$$

Where SLR = solids loading rate (lb/ft$^2$-hr), Q = flowrate (ft$^3$/hr), R = recycle rate, X = MLSS concentration (lb/ft$^3$), A = secondary clarifier area.

The waste activated sludge (WAS) pumps are a function of the solids retention time (SRT). Using the simplified SRT equation provides a way to calculate the required WAS flow rate (Metcalf and Eddy, 2003):

$$Q_w=\frac{VX}{X_R SRT}$$

Where $Q_W$ = WAS flowrate, X = mixed liquor suspended solids, $X_R$ = RAS suspended solids, V = aeration basin volume, SRT = solids retention time.

The return activated sludge (RAS) pumps are a function of the solids retention time and sludge concentrations. Use the following equation to size the RAS pumps (Metcalf and Eddy, 2003):

$$Q_R=Q\frac{X\left(1-\frac{V}{QSRT}\right)}{X_R-X}$$

Where $Q_R$ = RAS flowrate.

Table C.20 summarizes the design of the secondary clarifiers. The energy parameters for the secondary clarifiers are shown in Table C.21.

**Table C.19 - Secondary Clarifiers Design Criteria**

| Parameter | Range | Units | Reference |
|---|---|---|---|
| Overflow Rate (OFR) at Average Flow | 400 - 700 | gal/ft$^2$-d | (WEF, 2010a) |
| Overflow Rate (OFR) at Peak Flow | 1,000-1,600 | gal/ft$^2$-d | (WEF, 2010a) |
| Weir Loading | ≤30,000 | gpd/ft | (WEF, 2010a) |
| Solids Loading at Average Flow | 0.8 - 1.2 | lb/ft$^2$-hr | (WEF, 2010a) |
| Solids Loading at Peak Flow | 1.6 | lb/ft$^2$-hr | (WEF, 2010a) |

**Table C.20 - Secondary Clarifier Design**

| Parameter | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| Average Flow per Secondary Clarifier | 10 | 10 | 8.5 | MGD |
| Peak Flow per Secondary Clarifier | 15 | 15 | 12.75 | MGD |
| Number of Primary Clarifiers | 10 | 10 | 12 | N/A |
| Total Average Flow Capability | 100 | 100 | 102 | MGD |
| Total Peak Flow Capability | 150 | 150 | 153 | MGD |
| Diameter | 140 | 140 | 140 | ft |
| Depth | 14 | 14 | 14 | ft |
| OFR at Average Flow | 650 | 650 | 552 | gal/ft$^2$-day |
| OFR at Peak Flow | 974 | 974 | 828 | gal/ft$^2$-day |
| Weir Type | V-notch | V-notch | V-Notch | N/A |
| Weir Center to Center Spacing | 8 | 8 | 8 | in |
| Individual Weir Length | 6 | 6 | 6 | in |
| Weir Face to Face Spacing | 2 | 2 | 2 | in |
| Weir Depth | 3 | 3 | 3 | in |
| Number of V-Notches | 659 | 659 | 659 | N/A |
| Weir Length | 439.3 | 439.3 | 439.3 | ft |
| Rake Arm Speed | 0.05 | 0.05 | 0.05 | rpm |
| Mixed Liquor Suspended Solids (MLSS) | 3,070 | 2,980 | 3,900 | mg/L |
| Minimum RAS Ratio | 0.5 | 0.5 | 0.5 | N/A |
| Maximum RAS Ratio | 1.5 | 1.5 | 1.5 | N/A |
| Solids Loading Rate at average flowrate and Minimum RAS Ratio | 1.0 | 1.0 | 1.1 | lb/ft$^2$-hr |
| Solids Loading Rate at average flowrate and Maximum RAS Ratio | 1.6 | 1.5 | 1.7 | lb/ft$^2$-hr |
| Solids Loading Rate at peak flowrate and Minimum RAS Ratio | 1.6 | 1.5 | 1.7 | lb/ft$^2$-hr |

**Table C.21 - Secondary Clarifier Energy Parameters**

| Parameters | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| Total Number of Operating Primary Clarifiers | 10 | 10 | 12 | N/A |
| Solids Retention Time | 20 | 10 | 8 | days |
| Waste Activated Sludge (WAS) Pump Flow Rate | 33 | 65 | 106 | gpm |
| WAS Pump Head | 50 | 50 | 50 | ft |
| WAS Pump Efficiency | 75% | 75% | 75% | N/A |
| WAS Pump Energy Requirements per Secondary Clarifier | 0.4 | 0.8 | 1.3 | kW |
| Total WAS Pump Energy Requirements | 101 | 197 | 384 | kWh/Day |
| Assumed MLSS in Return Activated Sludge (RAS) | 8,000 | 8,000 | 8,000 | mg/L |
| RAS Pump Flow Rate | 4,160 | 4,100 | 5,500 | gpm |
| RAS Pump Efficiency | 75% | 75% | 75% | N/A |
| Total RAS Pump Energy Requirements | 10,030 | 10,180 | 16,050 | kWh/Day |
| Rake Arm Loading Factor, W | 6.2 | 6.5 | 6.9 | lb/ft |
| Rake Arm Angular Velocity, ω | 0.0055 | 0.0055 | 0.0055 | rad/s |
| Rake Arm Efficiency | 75% | 75% | 75% | N/A |
| Total Rake Arm Energy | 71.7 | 75.7 | 95.6 | kWh/day |
| Total Energy | 10,202.7 | 10,452.7 | 16,529.6 | kWh/day |

A summary of the secondary clarifier design procedure is as follows:

1. Calculate the overflow rate for the average strength and average flowrate (Metcalf and Eddy, 2003):

   $$\mathrm{OFR}=\frac{\mathrm{Q}}{\mathrm{A}}=\frac{10\times10^6}{\frac{\pi}{4}\times140^2}=650\ \mathrm{gal/ft^2\text{-}day}$$

   This value is acceptable.

2. Repeat number 1 for the low and high strengths for the average flowrate. This yields 650 and 552 gal/ft$^2$-day. These values are acceptable.

3. Repeat numbers 1 and 2 for the peak flowrate. This yields 974, 974, and 828 gal/ft$^2$-day for the low, average, and high strengths.

4. Calculate the number of v-notches for the outlet weir on the primary clarifier:

   $$\text{V-notches}=\frac{\pi\times140}{\frac{8}{12}}=659$$

5. Calculate the weir length:

   $$\text{Weir length}=659\times\frac{8}{12}=439.3\ \mathrm{ft}$$

6. Calculate the weir loading at the average flowrate (Metcalf and Eddy, 2003):

   $$\mathrm{WL}=\frac{\mathrm{Q}}{\mathrm{L}}=\frac{10\times10^6}{439.3}=22{,}763\ \mathrm{gal/ft\text{-}day}$$

   This value is acceptable.

7. Calculate the solids loading rate for the average strength for the average flowrate and the minimum RAS ratio:

   $$\mathrm{SLR}=\frac{(1+\mathrm{R})\mathrm{QX}}{\mathrm{A}}=\frac{(1+0.5)\times10\times1.547\times60\times60\times2{,}980\times6.243\times10^{-5}}{\frac{\pi}{4}\times140^2}$$

$SLR = 1\ lb/ft^2\text{-}hr$

This value is acceptable.

8. Repeat number 4 for the low and high strengths. This yields 1.0 and 1.1 $lb/ft^2$-hr for the low and high strengths. These values are acceptable.
9. Repeat numbers 4 and 5 for the maximum RAS ratio at the average flowrate for the low, average, and high strengths. This yields 1.6, 1.5, and 1.7 $lb/ft^2$-hr for the low, average, and high strengths. The high strength value is above the value recommended in the design parameter table.
10. Repeat numbers 4 and 5 for the minimum RAS ratio at the peak flowrate for the low, average, and high strengths. This yields 1.6, 1.5, and 1.7 $lb/ft^2$-hr for the low, average, and high strengths. The high strength value is above the value recommended in the design parameter table.

The energy for the waste activated sludge (WAS) pumps in the secondary clarifiers was calculated as follows:

1. Calculate the required WAS flowrate for the average strength using the simplified SRT equation (Metcalf and Eddy, 2003):

$$Q_w = \frac{VX}{X_R SRT} = \frac{337{,}000 \times 7.48 \times 2{,}984}{10 \times 24 \times 60 \times 8{,}000} = 65\ \text{gpm}$$

2. Repeat number 1 for the low and high strengths. This yields 33 and 106 gpm for the low and high strengths.
3. Calculate the energy requirements for the low, average and high strengths for each WAS pump using the brake horsepower equation. This yields 0.4, 0.8, and 1.3 kW.

4. Calculate the total WAS pump energy requirements for the low, average, and high strengths. This yields 101, 197, and 384 kWh/day.

The energy for the return activated sludge (RAS) pumps in the secondary clarifiers was calculated as follows:

1. Calculate the required RAS flowrate for the average flowrate using the RAS equation:

$$Q_R = Q\frac{X\left(1-\frac{V}{QSRT}\right)}{X_R - X}$$

$$Q_R = 10.1\times10^6 \times \frac{1}{24\times60} \times \frac{2{,}984\left(1-\frac{337{,}000}{10.1\times10^6\times\frac{1}{24\times60}\times10\times24\times60}\right)}{8{,}000-2{,}984} =$$

$Q_R = 4{,}159$ gpm

2. Repeat number 1 for the low and high strengths. This yields 4,272 and 5,531 gpm for the low and high strengths.

3. Calculate the BHP of a RAS pump for the low, average, and high strengths. This yields 43, 41.8, and 55.6 kW for the low, average, and high strengths.

4. Calculate the total RAS pump energy requirements for the low, average, and high strengths. This yields 10,311, 10,038, and 16,117 kWh/day.

The calculation of the rake arm energy is as follows:

1. Calculate the rake arm energy using the rake arm energy equation for the average strength (WEF, 1982; WEF, 2005). Assume an efficiency of 75%.

$$P = \frac{Wr^2\omega}{550e} = \frac{6.5\times\left(\frac{140}{2}\right)^2\times0.0055}{550\times0.75} = 0.42\text{ HP} = 0.32\text{ kW}$$

2. Repeat number 1 for the low and high strengths. This yields 0.30 and 0.33 kW for the low and high strengths.
3. Calculate the total energy requirements per day for the rake arms at average strength:

   Rake arm energy$=0.32 \times 10 \times 24=75.7$ kWh/day
4. Repeat number 3 for the low and high strengths. This yields 71.7 and 95.6 kWh/day.

# 6 Dual Media Filters

Table C.22 summarizes the design criteria used for the dual media filters. Table C.23 summarizes the dual media filter design. The main equations used for the design of the dual media filters are the filtration rate, cleanbed headloss, and uniformity coefficient. The filtration rate is calculated as follows (Davis, 2010):

$$q = \frac{Q}{NA}$$

Where q = filtration rate (gpm/ft$^2$), Q = flowrate (gpm), N = number of filters, A = area of one filter (ft$^2$).

The Rose headloss equation is commonly used to describe headloss in a clean filter (Metcalf and Eddy, 2003):

$$h_L = \frac{1.067}{\emptyset}\frac{Lv_s^2}{\alpha^4 g}\sum C_d \frac{P}{d_g}$$

$h_L$ = headloss, $\emptyset$ = particle shape factor, L = filter depth, $v_s$ = filtration velocity, α = porosity, g = gravitational acceleration, $C_d$ = drag coefficient, P = fraction of particles within adjacent sieve sizes, $d_g$ = geometric mean diameter.

The uniformity coefficient is calculated as follows (Metcalf and Eddy, 2003):

$$UC = \frac{d_{60}}{d_{10}}$$

Where UC = uniformity coefficient, $d_{60}$ = diameter of particles at which 60% of material is finer by weight, $d_{10}$ = diameter of particles at which 10% of material is finer by weight.

**Table C.22 - Dual Media Filters Design Criteria**

| **Parameter** | **Range** | **Units** | **Reference** |
|---|---|---|---|
| Filtration Rate at Peak Flow With One Filter out of Service | ≤5 | $gpm/ft^2$ | (GLUMRB, 2004) |
| Anthracite Depth | 14.2 - 35.4 | in | (Metcalf and Eddy, 2003) |
| Anthracite Effective Size | 0.031 - 0.079 | in | (Metcalf and Eddy, 2003) |
| Anthracite Uniformity Coefficient | 1.3 - 1.6 | unitless | (Metcalf and Eddy, 2003) |
| Sand Depth | 7.1 - 14.2 | in | (Metcalf and Eddy, 2003) |
| Sand Effective Size | 0.016 - 0.031 | in | (Metcalf and Eddy, 2003) |
| Sand Uniformity Coefficient | 1.2 - 1.6 | unitless | (Metcalf and Eddy, 2003) |
| Filter Bed Expansion During Backwash | 30 - 50% | unitless | (WEF, 2010a) |
| Water Backwash Time | 5 - 8 | min | (WEF, 2008) |
| Air Scour Backwash Time | 2 - 5 | min | (Chen, et al., 2003) |
| Air Scour Rate | 3 - 5 | $ft^3/min\text{-}ft^2$ | (WEF, 2010a) |
| Anthracite Effective Size | 0.8 – 2.0 | mm | (Metcalf and Eddy, 2003) |
| Sand Effective Size | 0.4 – 0.8 | mm | (Metcalf and Eddy, 2003) |
| Anthracite Depth | 1.2 – 3.0 | ft | (Metcalf and Eddy, 2003) |
| Sand Depth | 0.60 – 1.2 | ft | (Metcalf and Eddy, 2003) |

**Table C.23 - Dual Media Filter Design Table**

| Parameter | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| Total Average Flow for All Filters | 100 | 100 | 100 | MGD |
| Total Peak Flow for All Filters | 190 | 190 | 190 | MGD |
| Area per Filter | 1,000 | 1,000 | 1,000 | $ft^2$ |
| Dual Media Filter Materials | Sand and Anthracite | Sand and Anthracite | Sand and Anthracite | N/A |
| Anthracite Depth | 1.8 | 1.8 | 1.8 | ft |
| Sand Depth | 1.2 | 1.2 | 1.2 | ft |
| Number of Filters | 28 | 28 | 28 | N/A |
| Calculated Filtration Rate at Peak Flow With One Filter Out of Service | 4.9 | 4.9 | 4.9 | $gpm/ft^2$ |
| Cleanwater Headloss at Average Flow | 0.72 | 0.72 | 0.72 | ft |
| Cleanwater Headloss at Peak Flow | 1.44 | 1.44 | 1.44 | ft |
| Airflow Rate for Air Scour | 4 | 4 | 4 | $ft^3/ft^2 \cdot min$ |
| Required Blower Sizing | 4,000 | 4,000 | 4,000 | scfm |
| Backwash Rate With Air Scour | 8.9 | 8.9 | 8.9 | $gpm/ft^2$ |

| Parameter | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| Backwash Rate With Air Scour | 8,900 | 8,900 | 8,900 | gpm |
| Backwash Cycle For Air | 4 | 4 | 4 | min |
| Backwash Cycle for Water | 8 | 8 | 8 | min |
| Time to reach Terminal Headloss | 72 | 36 | 22 | hours |
| Recovery Rate | 95.2 | 95.2 | 95.2 | % |
| $d_{10}$ of anthracite | 1.29 | 1.29 | 1.29 | mm |
| $d_{60}$ of anthracite | 1.8 | 1.8 | 1.8 | mm |
| Uniformity Coefficient of anthracite | 1.4 | 1.4 | 1.4 | N/A |
| Anthracite Porosity | 0.55 | 0.55 | 0.55 | N/A |
| Anthracite Specific Gravity | 1.65 | 1.65 | 1.65 | N/A |
| $d_{10}$ of sand | 0.49 | 0.49 | 0.49 | mm |
| $d_{60}$ of sand | 0.72 | 0.72 | 0.72 | mm |
| Uniformity Coefficient of sand | 1.47 | 1.47 | 1.47 | N/A |
| Sand Porosity | 0.44 | 0.44 | 0.44 | N/A |
| Sand Specific Gravity | 2.6 | 2.6 | 2.6 | N/A |

| Parameter | Low Strength | Average Strength | High Strength | Units |
|---|---|---|---|---|
| TSS Percent Removal at Average Flow in Anthracite | 24 | 24 | 24 | % |
| TSS Percent Removal at Average Flow in Sand | 55 | 55 | 55 | % |

The first step in evaluating the design is to perform a sieve analysis of the anthracite and sand. Table C.24 summarizes the anthracite sand sieve parameters at average flow. Table C.25 summarizes anthracite sand sieve parameters at peak flow. Table C.26 summarizes the filter sand sieve parameters at average flow. Table C.27 summarizes the filter sand sieve parameters at peak flow. Figure C.1 shows the anthracite sand grain distribution. Figure C.2 shows the filter sand grain distribution. The anthracite filter media and sand filter media sieve analysis were found in manufacturer literature (Carbon Enterprises, Inc., 2006; Red Flint Sand and Gravel). Additional information was calculated as follows:

1. Determine the $d_{10}$ and $d_{60}$ for the sand and anthracite using the sieve analysis presented in Figure C.1 and Figure C.2. For the sand, the $d_{10}$ and $d_{60}$ are 0.49 and 0.72 mm, respectively. For the anthracite, the $d_{10}$ and $d_{60}$ are 1.29 and 1.8 mm, respectively.
2. Calculate the uniformity coefficient (Metcalf and Eddy, 2003) for the anthracite:

   $$UC = \frac{d_{60}}{d_{10}} = \frac{1.8}{1.29} = 1.4$$

   This value is acceptable.

3. Repeat number 2 for the sand. This yields 1.47 which is acceptable.

4. Calculate the Reynolds number (Metcalf and Eddy, 2003) for the anthracite value shown in the Sieve number 7 row in Table C.24:

$$\mathrm{N_R} = \frac{\emptyset \mathrm{d} \mathrm{v_s}}{v}$$

Where $N_R$ = Reynolds number, $\emptyset$ = particle shape factor (0.75 for sand, 0.73 for anthracite), d = geometric mean size diameter (m), $v_s$ = filtration velocity (m/s), $v$ = kinematic viscosity ($m^2/s$).

$$\mathrm{N_R} = \frac{0.73 \times \frac{3.07}{1000} \times 1.75 \times 10^{-3}}{1.003 \times 10^{-6}} = 3.90$$

5. Repeat number 4 for the rest of the anthracite values and sand values for the average and peak flowrates in Table C.25, Table C.26, and Table C.27.

6. Calculate the drag coefficient (Metcalf and Eddy, 2003) for the anthracite value shown in the Sieve number 7 row in Table C.24:

$$\mathrm{C_d} = \frac{24}{\mathrm{N_R}} + \frac{3}{\sqrt{\mathrm{N_R}}} + 0.34 = \frac{24}{3.90} + \frac{3}{\sqrt{3.90}} + 0.34 = 8.02$$

7. Repeat number 6 for the rest of the anthracite values and sand vales for the average and peak flowrates in Table C.25, Table C.26, and Table C.27.

8. Calculate $C_d$(p/d) for the anthracite value shown in the Sieve number 7 row in Table C.24:

$(8.02 \times 0.9/100)/(3.07/1000) = 23.53\ m^{-1}$

9. Repeat number 8 for the rest of the anthracite values and sand vales for the average and peak flowrates in Table C.25, Table C.26, and Table C.27.

10. Add up the $C_d$(p/d) values for the anthracite and sand. This yields 7932.62 and 4480.72 for the anthracite for the average and peak flowrates as shown in Table C.24 and Table C.25. The values are 46697.41 and 25706.57 for the sand for the average and peak flowrates as shown in Table C.26 and Table C.27.

11. Calculate the clean filter headloss for the anthracite using the Rose equation (Metcalf and Eddy, 2003):

$$h_L = \frac{1.067}{0.73} \times \frac{1.8 \times \frac{1}{3.281} \times (1.75 \times 10^{-3})^2}{0.55^4 \times 9.81} \times 7932.62 = 0.022 \text{ m} = 0.072 \text{ ft}$$

12. Repeat number 11 for the sand using the Rose equation. This yields 0.65 ft. The total headloss is then 0.72 ft.

**Table C.24 - Anthracite Sand Sieve Parameters at Average Flow**

| Sieve Number | Nominal Sieve Size (mm) | Percent Finer | Percent Retained | Geometric Mean Size (mm) | $N_R$ | $C_d$ | $C_d$(p/d) ($m^{-1}$) |
|---|---|---|---|---|---|---|---|
| 6 | 3.36 | 100 | - | - | - | - | - |
| 7 | 2.8 | 99.1 | 0.9 | 3.07 | 3.90 | 8.02 | 23.53 |
| 8 | 2.38 | 96.2 | 2.9 | 2.58 | 3.28 | 9.32 | 104.65 |
| 10 | 2 | 81.8 | 14.4 | 2.18 | 2.77 | 10.80 | 712.97 |
| 12 | 1.68 | 54.5 | 27.3 | 1.83 | 2.33 | 12.61 | 1,878.59 |
| 14 | 1.41 | 16.9 | 37.6 | 1.54 | 1.96 | 14.76 | 3,606.28 |
| 16 | 1.19 | 6.4 | 10.5 | 1.30 | 1.65 | 17.26 | 1,399.49 |
| 18 | 1 | 1.3 | 5.1 | 1.09 | 1.39 | 20.21 | 944.79 |
| 20 | 0.841 | 0.9 | 0.4 | 0.92 | 1.16 | 23.72 | 103.47 |
| | | | | | | **Sum** | 7,932.62 |

**Table C.25 - Anthracite Sand Sieve Parameters at Peak Flow**

| Sieve Number | Nominal Sieve Size (mm) | Percent Finer | Percent Retained | Geometric Mean Size (mm) | $N_R$ | $C_d$ | $C_d(p/d)$ ($m^{-1}$) |
|---|---|---|---|---|---|---|---|
| 6 | 3.36 | 100 | - | - | - | - | - |
| 7 | 2.8 | 99.1 | 0.9 | 3.07 | 7.41 | 4.68 | 13.73 |
| 8 | 2.38 | 96.2 | 2.9 | 2.58 | 6.24 | 5.39 | 60.52 |
| 10 | 2 | 81.8 | 14.4 | 2.18 | 5.27 | 6.20 | 409.07 |
| 12 | 1.68 | 54.5 | 27.3 | 1.83 | 4.43 | 7.18 | 1,069.72 |
| 14 | 1.41 | 16.9 | 37.6 | 1.54 | 3.72 | 8.35 | 2,039.25 |
| 16 | 1.19 | 6.4 | 10.5 | 1.30 | 3.13 | 9.70 | 786.40 |
| 18 | 1 | 1.3 | 5.1 | 1.09 | 2.64 | 11.29 | 527.84 |
| 20 | 0.841 | 0.9 | 0.4 | 0.92 | 2.22 | 13.18 | 57.50 |
| | | | | | | **Sum** | 4,480.72 |

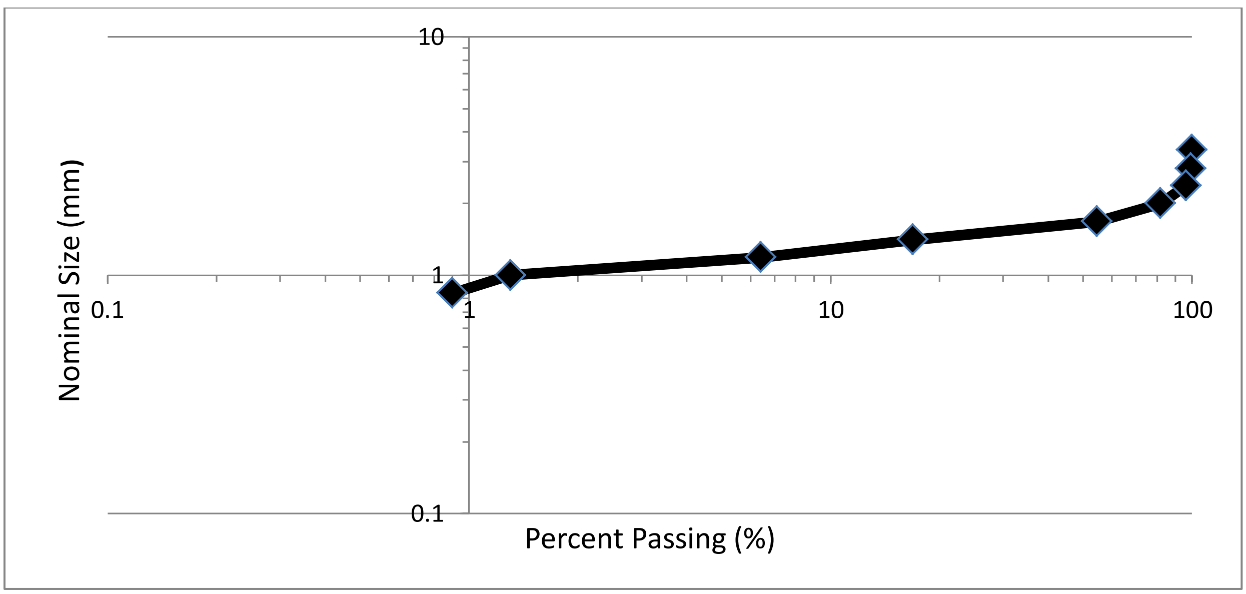


**Figure C.1 – Anthracite Sand Grain Distribution**

**Table C.26 – Filter Sand Sieve Parameters at Average Flow**

| Sieve Number | Nominal Sieve Size (mm) | Percent Finer | Percent Retained | Geometric Mean Size (mm) | $N_R$ | $C_d$ | $C_d(p/d)$ ($m^{-1}$) |
|---|---|---|---|---|---|---|---|
| 14 | 1.41 | 100 | - | - | - | - | - |
| 16 | 1.19 | 98 | 2 | 1.30 | 1.69 | 16.84 | 260.08 |
| 18 | 1 | 95 | 3 | 1.09 | 1.42 | 19.71 | 542.11 |
| 20 | 0.841 | 83 | 12 | 0.92 | 1.20 | 23.14 | 3,027.37 |
| 25 | 0.71 | 59 | 24 | 0.77 | 1.01 | 27.13 | 8,425.13 |
| 30 | 0.595 | 33 | 26 | 0.65 | 0.85 | 31.89 | 12,757.40 |
| 35 | 0.5 | 11 | 22 | 0.55 | 0.71 | 37.61 | 15,170.82 |
| 40 | 0.42 | 4 | 7 | 0.46 | 0.60 | 44.35 | 6,774.57 |
| | | | | | | **Sum** | 46,697.41 |

**Table C.27 – Filter Sand Sieve Parameters at Peak Flow**

| Sieve Number | Nominal Sieve Size (mm) | Percent Finer | Percent Retained | Geometric Mean Size (mm) | $N_R$ | $C_d$ | $C_d(p/d)$ ($m^{-1}$) |
|---|---|---|---|---|---|---|---|
| 14 | 1.41 | 100 | - | - | - | - | - |
| 16 | 1.19 | 98 | 2 | 1.30 | 3.22 | 9.47 | 146.28 |
| 18 | 1 | 95 | 3 | 1.09 | 2.71 | 11.02 | 303.14 |
| 20 | 0.841 | 83 | 12 | 0.92 | 2.28 | 12.87 | 1,683.73 |
| 25 | 0.71 | 59 | 24 | 0.77 | 1.92 | 15.01 | 4,663.04 |
| 30 | 0.595 | 33 | 26 | 0.65 | 1.61 | 17.57 | 7,029.15 |
| 35 | 0.5 | 11 | 22 | 0.55 | 1.35 | 20.64 | 8,324.25 |
| 40 | 0.42 | 4 | 7 | 0.46 | 1.14 | 24.24 | 3,703.26 |
| | | | | | | **Sum** | 25,706.57 |

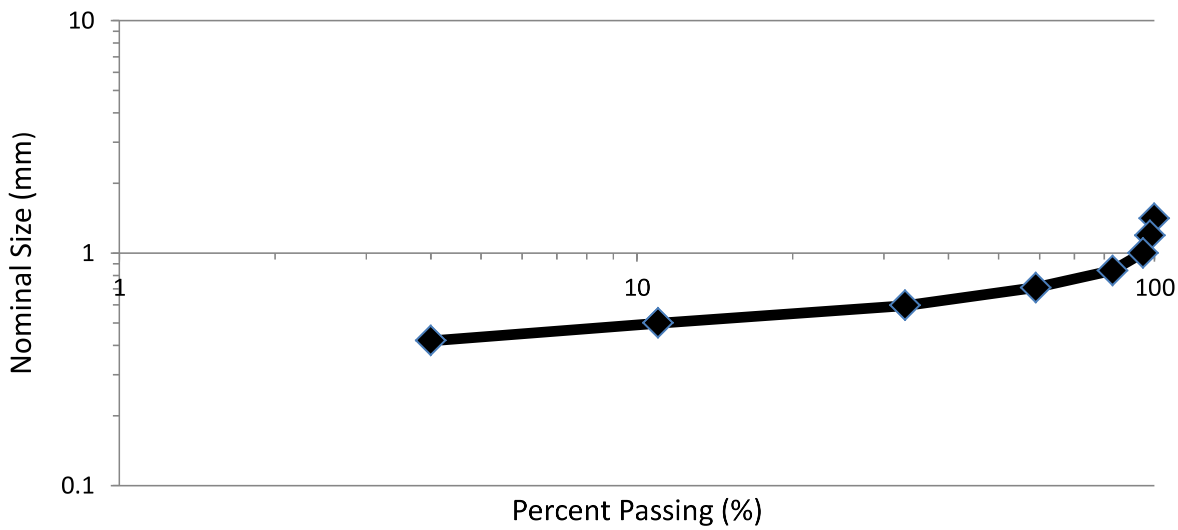


**Figure C.2 – Filter Sand Grain Distribution**

The second step in evaluating the design requires predicting TSS a function of filter depth using the Rajagopalan and Tien model (MWH, 2005). The parameters for the Rajagopalan and Tien model for the anthracite are shown in Table C.28 and for the sand are shown in Table C.29 with the assumptions of Hamaker constant, Boltzmann constant, particle density, water density, and porosity coming from MWH (2005). The Rajagopalan and Tien model is dependent upon the infiltration rate and changes from infiltration rate to infiltration rate. Table C.30 shows one set of calculations for a 5 m/h (2.05 gpm/ft$^2$) rate. The first layer (between 0 and 56 cm) in Table C.30 is anthracite and the second layer (56 to 92 cm) in Table C.30 is sand. Table C.31 and Figure C.3 show the completed model.

1. Calculate the porosity function, γ, for anthracite (MWH, 2005):

   $\gamma = (1 - \varepsilon)^{1/3}$

   Where $\gamma$ = porosity function, ε = porosity.

   $\gamma = (1 - 0.55)^{1/3} = 0.77$

2. Repeat number 1 for the sand. This yields 0.82.

3. Calculate the porosity function, $A_S$, for anthracite (MWH, 2005):

$$A_S = \frac{2(1-\gamma^3)}{2-3\gamma+3\gamma^5-2\gamma^6}$$

Where $A_S$ = porosity function.

$$A_S = \frac{2(1-\gamma^3)}{2-3\gamma+3\gamma^5-2\gamma^6} = 16.56$$

4. Repeat number 3 for the sand. This yields 29.91.

5. Calculate the transport efficiency due to gravity for anthracite (MWH, 2005):

$$N_G = \frac{g(\rho_P-\rho_W)d_p^2}{18\mu v_F}$$

Where $N_G$= transport efficiency due to gravity, g = gravitational acceleration (9.81 m/s$^2$), $\rho_P$ = particle density (1,050 kg/m3), $\rho_W$= water density (998.2 kg/m$^3$), $d_p$= particle diameter (0.1 µm), µ = viscosity (0.001 kg/m-s), and $v_f$ = filtration rate (m/h).

$$N_G = \frac{9.81(1{,}050-998.2)(0.1\times10^{-6})^2\times3600}{18\times0.001\times5} = 2.033\times10^{-7}$$

6. Repeat number 5 for sand. This yields $2.033\times10^{-7}$.

7. Calculate the London group for anthracite (MWH, 2005):

$$N_{Lo} = \frac{4Ha}{9\pi\mu d_p^2 v}$$

Where $N_{Lo}$ = London group, and Ha = Hamaker constant ($1.00\times10^{-19}$ J).

$$N_{Lo} = \frac{4\times1.00\times10^{-19}}{9\times\pi\times0.001\times(0.1\times10^{-6})^2\times5} = 1.019$$

8. Repeat number 7 for sand. This yields 1.019.

9. Calculate the Peclet number for anthracite (MWH, 2005) for the 1.25 mm row in Table C.30:

$$Pe = \frac{3\pi\mu d_P d_c \upsilon}{k_B T}$$

Where Pe = Peclet number, $k_B$ = Boltzmann constant (1.381 x $10^{-23}$ J/K), and T = absolute temperature (°K).

$$Pe = \frac{3 \times \pi \times 0.001 \times 0.1 \times 10^{-6} \times 1.25 \times 10^{-3} \times 5}{1.381 \times 10^{-23} \times 3{,}600 \times (20 + 273.15)} = 404{,}170.9$$

10. Repeat number 9 for the rest of the sand and anthracite values in Table C.30.

11. Calculate the relative-size group for anthracite (MWH, 2005) for the 1.25 mm row in Table C.30:

$$N_R = \frac{d_p}{d_c}$$

Where $N_R$= relative-size group, $d_p$= particle diameter (m), $d_c$ = diameter of collector (m).

$$N_R = \frac{0.1 \times 10^{-6}}{1.25 \times 10^{-3}} = 8.00 \times 10^{-5}$$

12. Repeat number 11 for the rest of the sand and anthracite values in Table C.30.

13. Calculate the total transport efficiency for anthracite (MWH, 2005) for the 1.25 mm row in Table C.30:

$$\eta = 4A_S^{1/3}Pe^{-2/3} + A_S N_{Lo}^{1/8} N_R^{15/8} + 3.38 \times 10^{-3} A_S N_G^{6/5} N_R^{-2/5}$$

Where η = total transport efficiency.

$$\eta = 4 \times 16.56^{1/3} \times 404170.9^{-2/3} + 16.56 \times 1.019^{1/8} \times (8.00 \times 10^{-5})^{15/8} + 3.38 \times 10^{-3} \times 16.56 \times (2.033 \times 10^{-7})^{6/5} \times (8.00 \times 10^{-5})^{-2/5} = 1.87 \times 10^{-3}$$

14. Repeat number 13 for the rest of the sand and anthracite values in Table C.30.

15. Calculate $\frac{C}{C_0}$ for anthracite (MWH, 2005) for the 1.25 mm row in Table C.30:

$$\frac{C}{C_0} = \exp\left[\frac{-3(1-\varepsilon)\eta\alpha L}{2d_C}\right]$$

Where C = remaining TSS concentration (mg/L), $C_0$ = initial TSS concentration (mg/L), L = filter depth (m).

$$\frac{C}{C_0} = \exp\left[\frac{-3(1-0.55)\times 1.87\times 10^{-3}\times 1\times \frac{4}{100}}{2\times \frac{1.25}{100}}\right] = 0.96$$

For subsequent calculations, make sure to add the previous decrease in TSS.

16. Repeat number 15 for the rest of the sand and anthracite values in Table C.30.

**Table C.28 - Anthracite Sand Rajagopalan and Tien Parameters**

| | | |
|---|---|---|
| Assumed attachment efficiency α = | 1 | N/A |
| Particle Size = | 0.1 | μm |
| Assumed Particle Density = | 1050 | $kg/m^3$ |
| $\rho_w$ = | 998.2 | $kg/m^3$ |
| Temperature = | 20 | °C |
| Hamaker Constant = | 1.00E-19 | $kg{\cdot}m^2/s^2$ |
| Boltzmann Constant = | 1.381E-23 | $kg{\cdot}m^2/s^2{\cdot}K$ |
| ε = | 0.55 | N/A |
| γ = | 0.77 | N/A |
| $A_s$ = | 16.56 | N/A |
| μ = | 0.001 | kg/m·s |
| $V_F$ = | 5 | m/h |
| $N_G$ = | 2.033E-07 | N/A |
| $N_{LO}$ = | 1.019E+00 | N/A |
| Depth Increments | 4 | cm |

**Table C.29 - Filter Sand Rajagopalan and Tien Parameters**

| | | |
|---|---|---|
| Assumed attachment efficiency α = | 1 | N/A |
| Particle Size = | 0.1 | µm |
| Assumed Particle Density = | 1050 | $kg/m^3$ |
| $\rho_w$ = | 998.2 | $kg/m^3$ |
| Temperature = | 20 | °C |
| Hamaker Constant = | 1.00E-19 | $kg{\cdot}m^2/s^2$ |
| Boltzmann Constant = | 1.381E-23 | $kg{\cdot}m^2/s^2{\cdot}K$ |
| ε = | 0.44 | N/A |
| γ = | 0.82 | N/A |
| $A_s$ = | 29.91 | N/A |
| μ = | 0.001 | kg/m·s |
| $V_F$ = | 5 | m/h |
| $N_G$ = | 2.033E-07 | N/A |
| $N_{L0}$ = | 1.019E+00 | N/A |
| Depth Increments | 4 | cm |

**Table C.30 - Rajagopalan and Tien Filtration Model for 5 m/h (2.05 gpm/ft$^2$)**

| Depth (cm) | Percent Passing (%) | Diameter (mm) | Pe | $N_R$ | η | $C/C_0$ |
|---|---|---|---|---|---|---|
| 0 | 0.00 | 0 | 0 | - | - | 1 |
| 4 | 7.14 | 1.25 | 404,170.9 | 8.00E-05 | 1.87E-03 | 0.96 |
| 8 | 14.29 | 1.4 | 452,671.4 | 7.14E-05 | 1.73E-03 | 0.93 |
| 12 | 21.43 | 1.6 | 517,338.8 | 6.25E-05 | 1.58E-03 | 0.90 |
| 16 | 28.57 | 1.61 | 520,572.1 | 6.21E-05 | 1.58E-03 | 0.88 |
| 20 | 35.71 | 1.66 | 536,739 | 6.02E-05 | 1.54E-03 | 0.86 |
| 24 | 42.86 | 1.68 | 543,205.7 | 5.95E-05 | 1.53E-03 | 0.84 |
| 28 | 50.00 | 1.7 | 549,672.5 | 5.88E-05 | 1.52E-03 | 0.82 |
| 32 | 57.14 | 1.78 | 575,539.4 | 5.62E-05 | 1.47E-03 | 0.80 |
| 36 | 64.29 | 1.9 | 614,339.8 | 5.26E-05 | 1.41E-03 | 0.78 |
| 40 | 71.43 | 1.95 | 630,506.6 | 5.13E-05 | 1.39E-03 | 0.77 |
| 44 | 78.57 | 1.98 | 640,206.7 | 5.05E-05 | 1.37E-03 | 0.76 |
| 48 | 85.71 | 2.05 | 662,840.3 | 4.88E-05 | 1.34E-03 | 0.74 |
| 52 | 92.86 | 2.3 | 743,674.5 | 4.35E-05 | 1.24E-03 | 0.73 |
| 56 | 100.00 | 2.9 | 937,676.5 | 3.45E-05 | 1.06E-03 | 0.72 |
| 60 | 11.11 | 0.49 | 158,435 | 2.04E-04 | 4.24E-03 | 0.54 |
| 64 | 22.22 | 0.56 | 181,068.6 | 1.79E-04 | 3.88E-03 | 0.43 |
| 68 | 33.33 | 0.6 | 194,002 | 1.67E-04 | 3.71E-03 | 0.35 |
| 72 | 44.44 | 0.65 | 210,168.9 | 1.54E-04 | 3.51E-03 | 0.29 |
| 76 | 55.56 | 0.7 | 226,335.7 | 1.43E-04 | 3.35E-03 | 0.25 |
| 80 | 66.67 | 0.78 | 252,202.7 | 1.28E-04 | 3.11E-03 | 0.22 |
| 84 | 77.78 | 0.83 | 268,369.5 | 1.20E-04 | 2.99E-03 | 0.19 |
| 88 | 88.89 | 0.9 | 291,003.1 | 1.11E-04 | 2.83E-03 | 0.17 |
| 92 | 100.00 | 1.41 | 455,904.8 | 7.09E-05 | 2.10E-03 | 0.16 |

**Table C.31 - Rajagopalan and Tien Model Summary**

| m/h | 5 | 9.75 | 14.5 | 19.25 | 24 |
|---|---|---|---|---|---|
| gal/ft$^2$·min | 2.05 | 3.99 | 5.93 | 7.87 | 9.82 |
| Depth (cm) | $C/C_0$ | $C/C_0$ | $C/C_0$ | $C/C_0$ | $C/C_0$ |
| 0 | 1 | 1.00 | 1.00 | 1.00 | 1.00 |
| 4 | 0.96 | 0.97 | 0.98 | 0.98 | 0.99 |
| 8 | 0.93 | 0.95 | 0.96 | 0.97 | 0.97 |
| 12 | 0.90 | 0.94 | 0.95 | 0.96 | 0.97 |
| 16 | 0.88 | 0.92 | 0.94 | 0.95 | 0.96 |
| 20 | 0.86 | 0.91 | 0.93 | 0.94 | 0.95 |
| 24 | 0.84 | 0.89 | 0.92 | 0.93 | 0.94 |
| 28 | 0.82 | 0.88 | 0.91 | 0.92 | 0.93 |
| 32 | 0.80 | 0.87 | 0.90 | 0.91 | 0.92 |
| 36 | 0.78 | 0.86 | 0.89 | 0.91 | 0.92 |
| 40 | 0.77 | 0.85 | 0.88 | 0.90 | 0.91 |
| 44 | 0.76 | 0.84 | 0.87 | 0.89 | 0.91 |
| 48 | 0.74 | 0.83 | 0.86 | 0.89 | 0.90 |
| 52 | 0.73 | 0.82 | 0.86 | 0.88 | 0.90 |
| 56 | 0.72 | 0.81 | 0.85 | 0.88 | 0.89 |
| 60 | 0.54 | 0.67 | 0.74 | 0.78 | 0.81 |
| 64 | 0.43 | 0.58 | 0.66 | 0.71 | 0.74 |
| 68 | 0.35 | 0.51 | 0.60 | 0.65 | 0.69 |
| 72 | 0.29 | 0.45 | 0.54 | 0.60 | 0.65 |
| 76 | 0.25 | 0.41 | 0.50 | 0.57 | 0.61 |
| 80 | 0.22 | 0.37 | 0.47 | 0.54 | 0.58 |
| 84 | 0.19 | 0.35 | 0.44 | 0.51 | 0.56 |
| 88 | 0.17 | 0.32 | 0.42 | 0.49 | 0.54 |
| 92 | 0.16 | 0.31 | 0.41 | 0.48 | 0.53 |

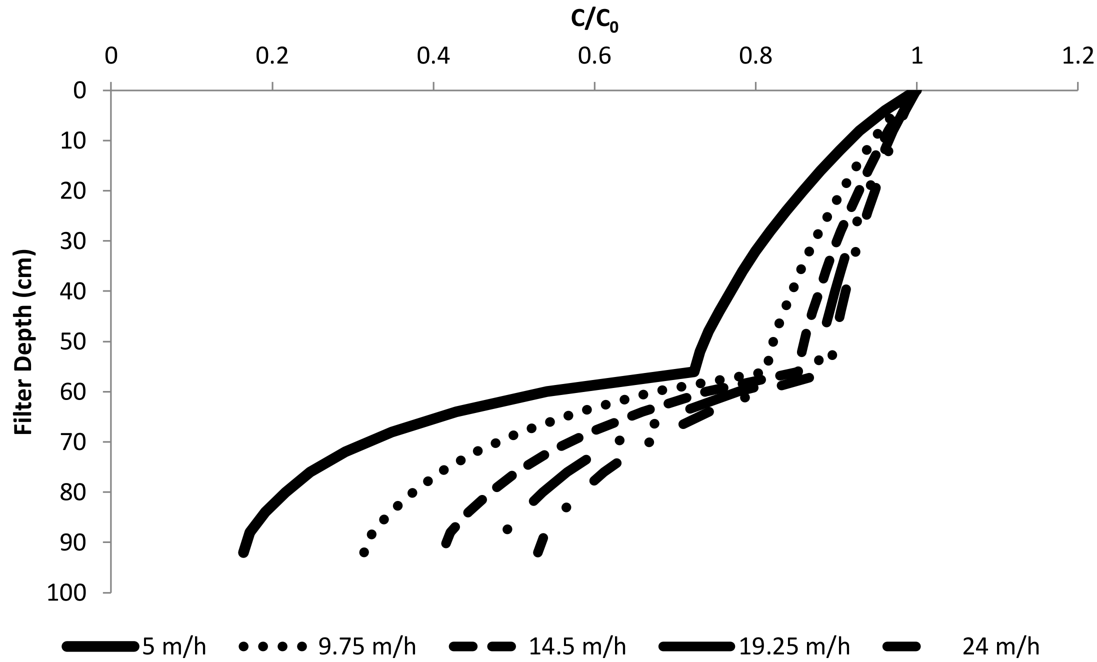


**Figure C.3 - Rajagopalan and Tien Model**

The third step in evaluating the design requires determining the headloss buildup using the headloss development method (Metcalf and Eddy, 2003) and Figure 11-10 from Metcalf and Eddy (2003). Table C.32 presents the headloss buildup for the low strength. Table C.33 presents the headloss buildup for the average strength. Table C.34 presents the headloss buildup for the high strength.

1. Figure 11-10 of Metcalf and Eddy (2003) provides a prediction of headloss versus suspended solids removed.
2. Assume an influent TSS value of 10, 15, and 20 mg/L for the low, average, and high strength wastewaters. Using this information and the percent removals provided in Table C.31 yields how much TSS is removed. TSS removed in the anthracite is 2.4, 3.6, and 4.8 mg/L for the low, average, and high strength wastewaters. TSS removed in the sand is 5.5, 8.25, and 11 mg/L for the low,

average, and high strength wastewaters. These values are ΔC in Metcalf and Eddy (2003) and are repeated in Table C.32, Table C.33, and Table C.34.

3. Calculate the suspended solids removed (Metcalf and Eddy, 2003) for the anthracite at 10 hours for the low strength:

   $$\Delta q = (-v_F)\left(\frac{\Delta C}{-L}\right)\Delta T$$

   Where Δq = suspended solids removed (mg/cm$^3$), and ΔT = elapsed time (min).

   $$\Delta q = (-0.0105)\left(\frac{2.4}{-\frac{56}{100}}\right) \times 10 \times 60 = 0.27 \text{ mg/cm}^3$$

4. Repeat step 3 for the rest of the anthracite and sand values in Table C.32, Table C.33, and Table C.34.
5. Use Figure 11-10 of Metcalf and Eddy (2003) to predict the change in headloss Δh due to the buildup in the sand layer at 10 hours for the low strength. This yields a value of 0.02 m (0.07 ft).
6. Repeat step 5 for the rest of the anthracite and sand values in Table C.32, Table C.33, and Table C.34.
7. Calculate the Total Headloss Δh at 10 hours for the low strength. The Total Headloss is equal to the Δh of the sand and the anthracite added to the clean filter headloss. This yields 0.79 ft.
8. Repeat number 7 for the rest of the anthracite and sand values in Table C.32, Table C.33, and Table C.34.
9. Using a terminal headloss of 10 ft, the time to completion of the filter cycles is approximately 68, 42, and 32 hours for the low, average, and high strength wastewaters.

### Table C.32 - Low Strength Headloss Buildup

| Time (hours) | Anthracite ΔC (mg/L) | Sand ΔC (mg/L) | Anthracite Δq ($mg/cm^3$) | Sand Δq ($mg/cm^3$) | Anthracite Δh (ft) | Sand Δh (ft) | Total Δh (ft) |
|---|---|---|---|---|---|---|---|
| 10 | 2.4 | 5.5 | 0.27 | 0.96 | 0 | 0.07 | 0.79 |
| 14 | 2.4 | 5.5 | 0.38 | 1.34 | 0 | 0.20 | 0.92 |
| 18 | 2.4 | 5.5 | 0.48 | 1.73 | 0 | 0.43 | 1.15 |
| 22 | 2.4 | 5.5 | 0.59 | 2.11 | 0 | 0.75 | 1.47 |
| 26 | 2.4 | 5.5 | 0.70 | 2.50 | 0 | 1.05 | 1.77 |
| 30 | 2.4 | 5.5 | 0.81 | 2.88 | 0 | 1.18 | 1.90 |
| 34 | 2.4 | 5.5 | 0.92 | 3.26 | 0 | 1.80 | 2.52 |
| 38 | 2.4 | 5.5 | 1.02 | 3.65 | 0 | 2.30 | 3.02 |
| 42 | 2.4 | 5.5 | 1.13 | 4.03 | 0 | 3.28 | 4.00 |
| 46 | 2.4 | 5.5 | 1.24 | 4.42 | 0 | 3.61 | 4.33 |
| 50 | 2.4 | 5.5 | 1.35 | 4.80 | 0 | 4.27 | 4.99 |
| 54 | 2.4 | 5.5 | 1.45 | 5.18 | 0 | 4.92 | 5.64 |
| 58 | 2.4 | 5.5 | 1.56 | 5.57 | 0 | 6.56 | 7.28 |
| 62 | 2.4 | 5.5 | 1.67 | 5.95 | 0 | 7.55 | 8.27 |
| 66 | 2.4 | 5.5 | 1.78 | 6.34 | 0 | 8.53 | 9.25 |
| 70 | 2.4 | 5.5 | 1.88 | 6.72 | 0 | 9.84 | 10.56 |

### Table C.33 - Average Strength Headloss Buildup

| Time (hours) | Anthracite ΔC (mg/L) | Sand ΔC (mg/L) | Anthracite Δq ($mg/cm^3$) | Sand Δq ($mg/cm^3$) | Anthracite Δh (ft) | Sand Δh (ft) | Total Δh (ft) |
|---|---|---|---|---|---|---|---|
| 10 | 3.6 | 8.25 | 0.40 | 1.44 | 0 | 0.33 | 1.26 |
| 14 | 3.6 | 8.25 | 0.57 | 2.02 | 0 | 0.66 | 1.59 |
| 18 | 3.6 | 8.25 | 0.73 | 2.59 | 0 | 0.98 | 1.92 |
| 22 | 3.6 | 8.25 | 0.89 | 3.17 | 0 | 1.97 | 2.90 |
| 26 | 3.6 | 8.25 | 1.05 | 3.74 | 0 | 2.62 | 3.56 |
| 30 | 3.6 | 8.25 | 1.21 | 4.32 | 0 | 3.28 | 4.22 |
| 34 | 3.6 | 8.25 | 1.37 | 4.90 | 0 | 4.92 | 5.86 |
| 38 | 3.6 | 8.25 | 1.53 | 5.47 | 0 | 6.56 | 7.50 |
| 42 | 3.6 | 8.25 | 1.70 | 6.05 | 0 | 9.02 | 9.96 |

**Table C.34 - High Strength Headloss Buildup**

| Time (hours) | Anthracite ΔC (mg/L) | Sand ΔC (mg/L) | Anthracite Δq (mg/cm$^3$) | Sand Δq (mg/cm$^3$) | Anthracite Δh (ft) | Sand Δh (ft) | Total Δh (ft) |
|---|---|---|---|---|---|---|---|
| 10 | 4.8 | 11 | 0.54 | 1.92 | 0 | 0.82 | 1.76 |
| 14 | 4.8 | 11 | 0.75 | 2.69 | 0 | 1.31 | 2.25 |
| 18 | 4.8 | 11 | 0.97 | 3.46 | 0 | 2.62 | 3.56 |
| 22 | 4.8 | 11 | 1.18 | 4.22 | 0 | 3.28 | 4.22 |
| 26 | 4.8 | 11 | 1.40 | 4.99 | 0 | 4.92 | 5.86 |
| 30 | 4.8 | 11 | 1.62 | 5.76 | 0 | 8.20 | 9.14 |
| 34 | 4.8 | 11 | 1.83 | 6.53 | 0 | 9.84 | 10.78 |

The next step is to determine the backwash rate required to fluidize the $d_{60}$ of anthracite using a backwash rate of 1.05 m/min. Table C.35 presents the anthracite backwash parameters.

1. Obtain $V_S$ values from Figure 5-21 in Metcalf and Eddy (2003). For sieve number 7 in Table C.35 the value is 0.365 m/s.
2. Repeat number 2 for the rest of the values in Table C.35.
3. Calculate $\alpha_e$ (Metcalf and Eddy, 2003) for sieve number 7 in Table C.35:

   $$\alpha_e = \left(\frac{v}{v_s}\right)^{0.22}$$

   Where $\alpha_e$ = expanded porosity, $v$ = backwash rate (m/s), $v_s$ = settling velocity (m/s).

   $$\alpha_e = \left(\frac{1.05}{0.365}\right)^{0.22} = 0.513$$

4. Repeat number 3 and calculate the rest of the anthracite values. The backwash rate of 1.05 m/min is sufficient to fluidize the $d_{60}$ of anthracite.

Table C.35 - Anthracite Backwash Parameters

| Sieve Number | Nominal Sieve Size (mm) | Percent Finer | Geometric Mean Size (mm) | $V_s$ (m/s) | $\alpha_e = (v/V_s)^{0.22}$ | Greater than Normal Porosity? |
|---|---|---|---|---|---|---|
| 6 | 3.36 | 100 | - | - | - | - |
| 7 | 2.8 | 99.1 | 3.07 | 0.365 | 0.513 | NO |
| 8 | 2.38 | 96.2 | 2.58 | 0.33 | 0.52 | NO |
| 10 | 2 | 81.8 | 2.18 | 0.29 | 0.54 | NO |
| 12 | 1.68 | 54.5 | 1.83 | 0.26 | 0.55 | YES |
| 14 | 1.41 | 16.9 | 1.54 | 0.23 | 0.57 | YES |
| 16 | 1.19 | 6.4 | 1.30 | 0.2 | 0.59 | YES |
| 18 | 1 | 1.3 | 1.09 | 0.17 | 0.61 | YES |
| 20 | 0.841 | 0.9 | 0.92 | 0.15 | 0.62 | YES |
| | | | | | | Sum |

A summary of the design procedure for the dual media filters is as follows:

1. Determine the optimal backwash water flowrate assuming a backwash air flowrate of 4 $ft^3/ft^2$-min (WEF, 2010a). The Amirtharajah equation provides a means of estimating the backwash water flowrate during an air scour backwash cycle (Chen, et al., 2003). Using a guess and check process yielded a value of 8.9 gpm/$ft^2$ (0.36 m/min).

   $$0.45Q_a^2 + 100\left(\frac{U_b}{U_{mf}}\right) = 41.9$$

   Where $Q_a$ = backwash air flowrate ($ft^3/ft^2$-min), $U_b$ = backwash water flowrate ($ft^3/ft^2$-min), $U_{mf}$ = minimum fluidization velocity for $d_{60}$ ($ft^3/ft^2$-min).

   $$0.45 \times (4 \times 1{,}000)^2 + 100 \times \left(\frac{0.36}{1.05}\right) = 41.9 \quad 41.9 = 41.9$$

2. Calculate the filtration rate for the peak flow rate using the filtration rate equation with one filter out of service (Davis, 2010):

   $$q = \frac{Q}{NA} = \frac{190\times10^6}{(28-1)\times1{,}000\times24\times60} = 4.9 \text{ gpm/ft}^{2.}$$

This value is acceptable.

The calculation of the filter influent pump station (FIPS) energy is as follows. Table C.36 summarizes the FIPS energy parameters.

1. FIPS has to move all the influent wastewater to the filters which in this case is 100 MGD (69,444 gpm).
2. Calculate the energy of FIPS using the brake horsepower equation (Jones, et al., 2008). Assume a typical efficiency of 75% for centrifugal pumps. Assume a TDH of 30 feet.

$$\mathrm{BHP} = \frac{\mathrm{QH}}{3{,}960\mathrm{e}} = \frac{69{,}444 \times 30}{3{,}960 \times 0.75} = 702\ \mathrm{HP} = 524\ \mathrm{kW}$$

3. Calculate the energy requirements of FIPS per day.

$$524\ \mathrm{kW}\ \times \frac{24\ \mathrm{hrs}}{\mathrm{day}} = 12{,}571.5\ \mathrm{kWh/day}$$

**Table C.36 - FIPS Energy Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Pump Specific Weight γ | 62.4 | 62.4 | 62.4 | $lb/ft^3$ |
| Pump Flow Rate | 69,444 | 69,444 | 69,444 | gpm |
| Total Dynamic Head of Pump | 30 | 30 | 30 | ft |
| Provided Efficiency for Pump | 0.75 | 0.75 | 0.75 | N/A |
| Power Input for Pumps Per Clarifier | 702 | 702 | 702 | HP |
| Power Input for Pumps Per Clarifier | 524 | 524 | 524 | kW |
| Number of Pumping Hours Per Day | 24 | 24 | 24 | hours |
| Pumping Power Requirement | 12,571.5 | 12,571.5 | 12,571.5 | kWh/day |
| Pumping Power Requirement | 125.7 | 125.7 | 125.7 | kWh/Mgal |

The calculation of the backwash pump energy is as follows. Table C.37 summarizes the backwash pump energy parameters.

1. Calculate the energy of backwashing pumps using the brake horsepower equation (Jones, et al., 2008). Assume a typical efficiency of 78% for centrifugal pumps. Assume a TDH of 60 feet.

$$\mathrm{BHP} = \frac{\mathrm{QH}}{3{,}960\mathrm{e}} = \frac{8933 \times 60}{3{,}960 \times 0.78} = 173.7\ \mathrm{HP} = 129.6\ \mathrm{kW}$$

2. Calculate the energy requirements of backwashing pumps per day for the average strength:

$$129.6\ \mathrm{kW} \times 2.3\ \mathrm{hrs} = 293.7\ \mathrm{kWh/day}$$

3. Repeat number 2 for the low and high strengths. This yields 172.8 and 380.1 kWh/day as shown in Table C.37.

**Table C.37 – Backwash Pump Energy Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Pump Specific Weight γ | 62.4 | 62.4 | 62.4 | $lb/ft^3$ |
| Pump Flow Rate | 8,933 | 8,933 | 8,933 | gpm |
| Total Dynamic Head of Pump | 60 | 60 | 60 | ft |
| Provided Efficiency for Pump | 0.78 | 0.78 | 0.78 | N/A |
| Power Input for Pumps Per Clarifier | 173.7 | 173.7 | 173.7 | HP |
| Power Input for Pumps Per Clarifier | 129.6 | 129.6 | 129.6 | kW |
| Number of Pumping Hours Per Day | 1.3 | 2.3 | 2.9 | hours |
| Pumping Power Requirement | 172.8 | 293.7 | 380.1 | kWh/day |
| Pumping Power Requirement | 1.7 | 2.9 | 3.8 | kWh/Mgal |

The calculation of the backwash blower energy is as follows. Table C.38 summarizes the backwash blower energy parameters.

1. Calculate the energy of backwashing for the blowers (U.S. EPA, 1989). Assume headloss of 7 psi, an efficiency of 80%, and a blower temperature of 68°F (assuming the blower is indoors).

$$\mathrm{WP} = \left(\frac{4.28 \times 10^{-4} \mathrm{q_s T_a}}{\mathrm{e}}\right)[(\mathrm{P_d/P_b})^{0.283} - 1]$$

$$= \left(\frac{4.28 \times 10^{-4} \times 4{,}000 \times (460 + 68)}{0.8}\right)[(1.5/1)^{0.283} - 1]$$

$$= 131.7\ \mathrm{HP} = 98.2\ \mathrm{kW}$$

2. Calculate the energy requirements of backwash blower energy per day for the average strength:

   $98.2\ \mathrm{kW} \times 1.1\ \mathrm{hrs} = 111.3\ \mathrm{kWh/day}$

3. Repeat number 2 for the low and high strengths. This yields 0.7 and 1.4 kWh/day as shown in Table C.38.

**Table C.38 - Backwash Blower Energy Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Air required | 4,000 | 4,000 | 4,000 | $ft^3$/min |
| Inlet Pressure | 1 | 1 | 1 | atm |
| Assumed Headloss | 7 | 7 | 7 | psi |
| Outlet Pressure | 1.5 | 1.5 | 1.5 | atm |
| Blower Efficiency | 0.8 | 0.8 | 0.8 | N/A |
| Blower Temperature In Summer | 68 | 68 | 68 | °F |
| Blower Power Requirement | 131.7 | 131.7 | 131.7 | HP |
| Blower Power Requirement | 98.2 | 98.2 | 98.2 | kW |
| Total Blower Hours | 0.7 | 1.1 | 1.5 | hours |
| Blower Power Requirement | 65.5 | 111.3 | 144.1 | kWh/day |
| Blower Power Requirement | 0.7 | 1.1 | 1.4 | kWh/Mgal |

# 7 Gravity Thickeners

Table C.39 summarizes the design criteria used for the gravity thickeners. The main equation used for the design of the gravity thickeners was the solids loading rate. Energy consumers for the gravity thickeners include the rake arms, overflow pumps, and sludge pumps. The solids loading rate equation is as follows (Metcalf and Eddy, 2003):

$$SLR = \frac{\dot{m}}{A}$$

Where SLR = solids loading rate (lb/ft$^2$-d), $\dot{m}$ = mass loading (lb/day), A = area of gravity thickener (ft$^2$).

Table C.40 shows the sludge parameters and Table C.41 shows the design parameters for the gravity thickeners.

**Table C.39 - Gravity Thickeners Design Criteria**

| Parameter | Range | Units | Reference |
|---|---|---|---|
| Solids Loading | 20 - 30 | lb/ft$^2$-d | (U.S. EPA, 1979; Metcalf and Eddy, 2003; WEF, 2010a) |

**Table C.40 - Gravity Thickener Sludge Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Assumed Initial Solids % | 4 | 4.5 | 5 | % |
| Settled Solids % | 8 | 9 | 10 | % |
| Amount of Solids produced at Average Flow | 65,303.4 | 114,682.4 | 218,948.7 | lb/day |
| Amount of Solids produced at Peak Flow | 90,161.7 | 158,232.6 | 301,895.1 | lb/day |
| Sludge Volume at Average Flow per minute | 15.7 | 27.5 | 52.6 | ft$^3$/min |
| Sludge Volume at Peak Flow per minute | 21.6 | 38.0 | 72.5 | ft$^3$/min |
| Sludge Volume at Average Flow | 168,889.7 | 296,595.0 | 566,251.9 | gal/day |
| Sludge Volume at Peak Flow | 233,178.9 | 409,226.1 | 780,770.5 | gal/day |
| Assumed Solids Capture Efficiency | 90 | 90 | 90 | % |
| Quantity of Sludge Withdrawn in Underflow | 5,8773.1 | 10,3214 | 19,7054 | lb/day |
| Sludge in Overflow | 5,877.31 | 10,321.4 | 19,705.4 | lb/day |

**Table C.41 - Gravity Thickener Design**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Number of Gravity Sludge Thickeners | 3 | 3 | 3 | N/A |
| Number of Gravity Sludge Thickeners In Service | 1 | 2 | 3 | N/A |
| Diameter | 65 | 65 | 65 | ft |
| Surface Area Provided | 3,318 | 3,318 | 3,318 | $ft^2$ |
| Solids Loading Rate at Peak Flow | 27.2 | 23.8 | 30.3 | $lb/ft^2$-day |
| Pump Sizing Assuming 5 Minute Pumping Time with 20 minutes off | 240 | 187 | 214 | gpm |

A summary of the gravity thickener design procedure is as follows:

1. Calculate the solids loading rate at the peak flowrate for the average strength in Table C.41 (Metcalf and Eddy, 2003):

$$\mathrm{SLR} = \frac{\dot{\mathrm{m}}}{\mathrm{A}} = \frac{158{,}232.6}{3{,}318 \times 2} = 23.8\ \mathrm{lb/ft^2 - d}$$

This value is acceptable.

2. Repeat number 1 for the low and high strength wastewaters in Table C.41. This yields 27.2 and 30.3 $lb/ft^2$-d. The low strength value is acceptable. The high strength value is just above the recommended 30 $lb/ft^2$-d.

3. Calculate the volume of sludge settling in gpm for the average strength wastewater.

$$103{,}214\,\frac{\mathrm{lb}}{\mathrm{day}} \times \frac{7.48}{\frac{9}{100} \times 1.02 \times 62.4 \times 24 \times 60} = 93.6\ \mathrm{gpm}$$

4. Repeat number 3 for the low and high strengths. This yields 60 and 160.8 gpm.

5. Calculate the settled pump sizing assuming a 20 minute interval with 5 minute pumping cycle:

$$\frac{93.6 \times 20}{5 \times 2\ \mathrm{pumps}} = 187.2\ \mathrm{gpm}$$

6. Repeat number 5 for the low and high strengths. This yields 239.8 and 214.4 gpm.
7. Calculate the number of settled pumping cycles.

$$\frac{24 * 60}{(20 + 5)} = 57.6 \text{ cycles}$$

The calculation of the sludge pump energy is as follows. Table C.42 presents the sludge pump energy parameters.

1. Calculate the energy of sludge pumping per pump for the average strength using the brake horsepower equation (Jones, et al., 2008). Assume an efficiency of 50%. Assume a TDH of 50 feet.

$$\text{BHP} = \frac{\text{QH}}{3960\text{e}} = \frac{187 \times 50}{3960 \times 0.5} = 4.7 \text{ HP} = 3.5 \text{ kW}$$

2. Repeat number 1 for the low and high strengths. This yields 4.5 kW and 4.0 kW.
3. Calculate the number of pumping hours for the average strength.

$$57.6 \times \frac{5}{60} \times 2 \text{ pumps} = 9.6 \text{ hrs}$$

4. Repeat number 3 for the low and high strengths. This yields 4.8 and 14.4 hours.
5. Calculate the energy requirements per day for the average strength.

$$3.5 \text{ kW} \times 9.6 \text{ hrs} = 34 \text{ kWh/d}$$

6. Repeat number 5 for the low and high strengths. This yields 22 and 58 kWh/day.
7. Calculate the energy requirements per ton of sludge processed for the average strength.

$$\frac{34 \text{ kWh/day}}{\frac{103{,}214}{2{,}000} \text{ ton/day}} = 0.66 \text{ kWh/ton}$$

8. Repeat number 7 for the low and high strengths. This yields 0.75 and 0.59 kWh/ton.

**Table C.42 - Sludge Pump Energy Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Pump Specific Weight γ | 62.4 | 62.4 | 62.4 | $lb/ft^3$ |
| Flow Rate | 240 | 187 | 214 | gpm |
| Provided Efficiency for Pump | 0.5 | 0.5 | 0.5 | N/A |
| Provided TDH for Pump | 50 | 50 | 50 | ft |
| Number of Pumps | 1 | 2 | 3 | N/A |
| Power Input for Pumps Per Thickener | 6.1 | 4.7 | 5.4 | HP |
| Power Input for Pumps Per Thickener | 4.5 | 3.5 | 4.0 | kW |
| Number of Pumping Cycle | 57.6 | 57.6 | 57.6 | N/A |
| Number of Pumping Hours | 4.8 | 9.6 | 14.4 | hours |
| Total Energy Required For Pumping | 22 | 34 | 58 | kWh/day |
| Total Energy Required For Pumping | 0.75 | 0.66 | 0.59 | kWh/ton |

The calculation of the overflow pump energy is as follows. Table C.43 presents the overflow pump energy parameters.

1. Calculate the energy of overflow pumping per pump for the average strength using the brake horsepower equation (Jones, et al., 2008). Assume an efficiency of 50%. Assume a TDH of 30 feet.

$$\mathrm{BHP} = \frac{\mathrm{QH}}{3{,}960\mathrm{e}} = \frac{56 \times 30}{3{,}960 \times 0.5} = 0.85\ \mathrm{HP} = 0.64\ \mathrm{kW}$$

2. Repeat number 1 for the low and high strengths. This yields 0.65 kW and 0.88 kW.

3. Calculate the energy requirements per day for the average strength.

$$0.64\ \mathrm{kW}\ \times 24\ \mathrm{hrs}\ \times 2\ \mathrm{pumps} = 31\ \mathrm{kWh/d}$$

4. Repeat number 5 for the low and high strengths. This yields 16 and 63 kWh/day.

5. Calculate the energy requirements per ton of sludge processed for the average strength.

$$\frac{31\ \text{kWh/day}}{\frac{103{,}214}{2{,}000}\ \text{ton/day}} = 0.60\ \text{kWh/ton}$$

6. Repeat number 5 for the low and high strengths. This yields 0.55 and 0.64 kWh/ton.

**Table C.43 - Overflow Pump Energy Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Pump Specific Weight γ | 62.4 | 62.4 | 62.4 | lb/ft$^3$ |
| Flow Rate | 57 | 56 | 77 | gpm |
| Provided Efficiency for Pump | 0.5 | 0.5 | 0.5 | N/A |
| Provided TDH for Pump | 30 | 30 | 30 | ft |
| Number of Pumps | 1 | 2 | 3 | N/A |
| Power Input for Pumps Per Thickener | 0.87 | 0.85 | 1.17 | HP |
| Power Input for Pumps Per Thickener | 0.65 | 0.64 | 0.88 | kW |
| Number of Pumping Hours | 24 | 48 | 72 | hours |
| Total Energy Required For Pumping | 16 | 31 | 63 | kWh/day |
| Total Energy Required For Pumping | 0.55 | 0.60 | 0.64 | kWh/lb |

The calculation of the rake arm energy is as follows. Table C.44 presents the rake arm energy parameters.

1. Calculate the rake arm energy using the rake arm energy equation for the average strength (WEF, 1982; WEF, 2005). Assume an efficiency of 75%.

$$\text{P} = \frac{\text{Wr}^2\omega}{550\text{e}} = \frac{44.7 \times \frac{1}{3.281} \times \frac{2.2}{1} \times \left(\frac{65}{2}\right)^2 \times 0.00892}{550 \times 0.75} = 0.69\ \text{HP} = 0.51\ \text{kW}$$

2. Repeat number 1 for the low and high strengths. This yields 0.69 and 0.69 kW for the low and high strengths.

3. Calculate the total energy requirements per day for the rake arms at average strength:

   $$\text{Rake arm energy} = 0.51 \times 2 \times 24 = 24.6\ \text{kWh/day}$$

4. Repeat number 3 for the low and high strengths. This yields 12.3 and 36.8 kWh/day.

**Table C.44 - Rake Arm Energy Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| K Value For Torque | 44.7 | 44.7 | 44.7 | kg/m |
| K Value For Torque | 438.5 | 438.5 | 438.5 | N/m |
| Torque Required Using Equation 20.10 of WEF 1998 | 43,026 | 43,026 | 43,026 | J |
| Alarm Torque | 51,631 | 51,631 | 51,631 | J |
| Shut off Torque | 60,236 | 60,236 | 60,236 | J |
| Failure Torque | 86,052 | 86,052 | 86,052 | J |
| Typical Peripheral Velocity | 5.3 | 5.3 | 5.3 | m/min |
| Typical Peripheral Velocity | 17.4 | 17.4 | 17.4 | ft/min |
| Typical Peripheral Velocity | 0.3 | 0.3 | 0.3 | ft/s |
| Angular Velocity | 8.92E-03 | 8.92E-03 | 8.92E-03 | rad/s |
| Required Motor Size | 0.51 | 0.51 | 0.51 | kW |
| Required Motor Size | 0.69 | 0.69 | 0.69 | HP |
| Energy Usage Per Day | 12.3 | 24.6 | 36.8 | kWh/day |
| Energy Usage Per Ton | 4.18E-01 | 4.76E-01 | 3.74E-01 | kWh/ton |

## 8 Dissolved Air Flotation Thickeners

Table C.45 summarizes the design criteria used for the dissolved air flotation thickeners (DAFTs). The main equations used for the design of the DAFTs were the solids loading rate, recycle ratio, and air to solids ratio. Energy consumers for the DAFTs include recycle pumps, sludge collectors, overflow pumps, air compressors, and sludge pumps. The air to solids ratio is calculated as follows (Metcalf and Eddy, 2003):

$$\frac{A}{S} = \frac{1.3 s_a (fP - 1) R}{S_a Q}$$

Where $\frac{A}{S}$ = air to solids ratio (mL of air/mg of solids), $s_a$ = air solubility (18.7 mL/L at 68°F), $f$ = fraction of air dissolved at pressure P, P = pressure (atm), Sa = influent suspended solids (mg/L), R = pressurized recycled rate (MGD), Q = influent rate (MGD or $m^3$/d).

Table C.46 shows the sludge parameters and Table C.47 shows the design for the DAFTs.

**Table C.45 - DAFTs Design Criteria**

| **Parameter** | **Range** | **Units** | **Reference** |
|---|---|---|---|
| Solids Loading | 0.5 - 0.8 | lb/$ft^2$-h | (U.S. EPA, 1979; Metcalf and Eddy, 2003) |
| | 0.4 - 1 | lb/$ft^2$-h | (WEF, 2010a) |
| Recycle Ratio | 200 – 300 | % | (WEF, 1982) |
| | 15 - 120 | % | (Metcalf and Eddy, 2003) |
| Air to Solids Ratio | 0.005 - 0.06 | Unitless | (Metcalf and Eddy, 2003) |
| | 0.02 - 0.06 | Unitless | (WEF, 2010a) |

**Table C.46 - DAFTs Sludge Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Assumed Initial Solids % | 0.8 | 0.8 | 0.8 | **%** |
| Sludge Solids Concentration Goal | 4 | 4 | 4 | % |
| Number of DAFTs | 3 | 3 | 3 | N/A |
| Number of DAFTs in Service | 1 | 2 | 3 | N/A |
| Sludge Volume at Average Flow | 1,640 | 3,270 | 6,276 | $m^3$/day |
| Sludge Volume at Average Flow | 57,925 | 115,496 | 22,1667 | $ft^3$/day |
| Sludge Volume at Average Flow | 301 | 600 | 1,151 | gpm |
| Sludge Mass at Average Flow | 28,916 | 57,656 | 11,0656 | lb/day |
| Sludge Volume at Average Flow Per DAFT | 1,640 | 1,635 | 2,092 | $m^3$/day |
| Sludge Volume at Average Flow Per DAFT | 301 | 300 | 384 | gpm |
| Sludge Volume at Average Flow Per DAFT | 57,916 | 57,739 | 73,878 | $ft^3$/day |
| Sludge Mass at Average Flow Per DAFT | 28,912 | 28,824 | 36,880 | lb/day |
| | | | | |
| Influent Suspended Solids | 8,000 | 8,000 | 8,000 | mg/L |
| Maximum Effluent Suspended Solids | 3,000 | 3,000 | 3,000 | mg/L |
| Assumed Solids Capture Efficiency | 95 | 95 | 95 | % |
| Quantity of Sludge Withdrawn | 27,466 | 54,765 | 105,108.1 | lb/day |

**Table C.47 - DAFTs Design**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Pressure in Atmospheres | 4.80 | 4.80 | 4.80 | atm |
| Recycle Rate | 4,920 | 4,905 | 6,276 | $m^3$/day |
| Recycle Rate | 903 | 900 | 1,151 | gpm |
| Air to Solids Ratio From Recycle Rate | 0.034 | 0.034 | 0.034 | mg (air)/mg (solids) |
| Actual Diameter | 60 | 60 | 60 | ft |
| Assumed Float TSS | 400 | 400 | 400 | mg/L |
| Recovery | 95 | 95 | 95 | % |
| Solids Loading Rate | 2.08 | 2.07 | 2.65 | kg/$m^2$·h |
| DAFT Solids | 1.31E+10 | 1.31E+10 | 1.67E+10 | mg/day |
| Air Required | 1,180 | 1,177 | 1,506 | kg/day |
| Air Required | 2,602 | 2,594 | 3,319 | lb/day |
| Air Density | 0.075 | 0.075 | 0.075 | lb/$ft^3$ |
| Air Required | 19,660 | 19,600 | 25,080 | $ft^3$/day |
| Air Required | 14 | 14 | 17 | $ft^3$/min |

A summary of the DAFTs design procedure is as follows:

1. Assume a typical recycle pressure in atmospheres of 4.8 which is typical (WEF, 2010b).
2. Calculate the recycle rate per DAFT assuming a rate of 300% which is typical (WEF, 1982):
   $300 \times 3 = 900 \text{ gpm}$
3. Calculate the air to solids ratio (Metcalf and Eddy, 2003):
   $$\frac{\mathrm{A}}{\mathrm{S}} = \frac{1.3 \times 18.7 \times (0.5 \times 4.8 - 1) \times 1{,}635 \times 3}{3{,}000 \times 1{,}635} = 0.034$$
   This is an acceptable value.
4. Calculate the solids loading rate for the average strength (Metcalf and Eddy, 2003):
   $$\mathrm{SLR} = \frac{\dot{\mathrm{m}}}{\mathrm{A}} = \frac{28{,}824 \times \frac{1}{2.2 \times 24}}{\frac{\pi}{4} \times 60^2 \times \left(\frac{1}{3.281}\right)^2} = 2.07 \text{ kg/m}^2 - \text{h}$$
   This value is acceptable.

5. Repeat number 4 for the low and high strengths. This yields 2.08 and 2.65 kg/m$^2$-h which is acceptable.
6. Calculate the DAFT solids in mg/day for the average strength per DAFT:

$$28{,}824 \times \frac{1}{2.2} \times 1{,}000 \times 1{,}000 = 1.31 \times 10^{10} \text{mg/day}$$

7. Repeat number 6 for the low and high strengths. This yields $1.31 \times 10^{10}$ and $1.67 \times 10^{10}$ mg/day.
8. Calculate the air requirements per day for the average strength:

$$1.31 \times 10^{10} \times 0.034 \times 1.5 \times \frac{1}{1{,}000 \times 1{,}000} = 667 \text{ kg/d} = 1{,}470 \text{ lb/d}$$

9. Repeat number 8 for the low and high strengths. This yields 1,474 and 1,881 lb/d.
10. Calculate the air requirements for the average strength. The density of air is 0.075 lb/ft$^3$:

$$\frac{1{,}470}{0.075} = 19{,}600 \frac{\text{ft}^3}{\text{d}} = 14 \text{ ft}^3/\text{min}$$

11. Repeat number 10 for the low and high strengths. This yields 14 and 17 ft$^3$/min.

The calculation of the air compressor energy is as follows. Table C.48 summarizes the DAFTs air compressor energy parameters.

1. Calculate the air compressor energy for the average strength (U.S. EPA, 1989). Assume headloss of 7 psi, an efficiency of 80%, and a blower temperature of 68°F (assuming the blower is indoors).

$$\text{WP} = \left(\frac{4.28 \times 10^{-4} \text{q}_s \text{T}_a}{\text{e}}\right) [(\text{P}_d/\text{P}_b)^{0.283} - 1]$$

$$= \left(\frac{4.28 \times 10^{-4} \times 14 \times (460 + 68)}{0.8}\right) [(1.48/1)^{0.283} - 1]$$

$$= 0.5 \text{ HP} = 0.3 \text{ kW}$$

2. Repeat number 1 for the low and high strengths. This yields 0.3 and 0.4 kW.
3. Calculate the energy requirements of the air compressor energy per day for the average strength:

   $0.3 \text{ kW} \times 24 \text{ hrs } \times 2 \text{ DAFTs} = 16.2 \text{ kWh/day}$
4. Repeat number 3 for the low and high strengths. This yields 8.1 and 31 kWh/day as shown in Table C.48.
5. Calculate the energy requirements per ton of sludge processed for the average strength.

   $$\frac{16.2 \text{ kWh/day}}{\frac{54765}{2{,}000} \text{ ton/day}} = 0.59 \text{ kWh/ton}$$
6. Repeat number 5 for the low and high strengths. This yields 0.59 and 0.59 kWh/ton as shown in Table C.48.

**Table C.48 - DAFTs Air Compressor Energy Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Air required | 14 | 14 | 17 | $ft^3$/min |
| Inlet Pressure | 1 | 1 | 1 | atm |
| Outlet Pressure | 1.48 | 1.48 | 1.48 | atm |
| Compressor Efficiency | 0.8 | 0.8 | 0.8 | N/A |
| Compressor Temperature In Summer | 68 | 68 | 68 | °F |
| Compressor Power Requirement | 0.5 | 0.5 | 0.6 | HP |
| Compressor Power Requirement | 0.3 | 0.3 | 0.4 | kW |
| Compressor Power Requirement Per DAFT | 8.1 | 8.1 | 10.3 | kWh/day |
| Number of Operational DAFTs | 1 | 2 | 3 | N/A |
| Total Compressor Power Requirement | 8.1 | 16.2 | 31.0 | kWh/day |
| Total Compressor Power Requirement | 0.59 | 0.59 | 0.59 | kWh/ton |

The calculation of the recycle pump energy is as follows. Table C.49 summarizes the recycle pump energy parameters.

1. Calculate the energy of recycle pumping per pump for the average strength using the brake horsepower equation (Jones, et al., 2008). Assume an efficiency of 75%. Assume a TDH of 50 feet.

   $$\mathrm{BHP} = \frac{\mathrm{QH}}{3{,}960\mathrm{e}} = \frac{900 \times 170}{3{,}960 \times 0.75} = 51.6\ \mathrm{HP} = 38.5\ \mathrm{kW}$$

2. Repeat number 1 for the low and high strengths. This yields 38.6 kW and 49.2 kW as shown in Table C.49.

3. Calculate the energy requirements per day for the average strength.

   $$38.5\ \mathrm{kW}\ \times 24\ \mathrm{hrs}\ \times 2\ \mathrm{DAFTs} = 1{,}846.2\ \mathrm{kWh/d}$$

4. Repeat number 3 for the low and high strengths. This yields 925.9 and 3,543.3 kWh/day as shown in Table C.49.

5. Calculate the energy requirements per ton of sludge processed for the average strength.

$$\frac{1,846.2 \text{ kWh/day}}{\frac{54,765}{2,000}\text{ton/day}} = 67.42 \text{ kWh/ton}$$

Repeat number 5 for the low and high strengths. This yields 67.42 and 67.42 kWh/ton.

**Table C.49 - DAFTs Recycle Pump Energy Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Pump Specific Weight γ | 62.4 | 62.4 | 62.4 | $lb/ft^3$ |
| Flow Rate For Pump | 903 | 900 | 1,151 | gpm |
| Provided Efficiency for Pump | 0.75 | 0.75 | 0.75 | N/A |
| Provided TDH for Pump | 170 | 170 | 170 | ft |
| Power Input for Pumps Per DAFT | 51.7 | 51.6 | 66.0 | HP |
| Power Input for Pumps Per DAFT | 38.6 | 38.5 | 49.2 | kW |
| Number of Pumping Hours Per DAFT | 24 | 24 | 24 | hours |
| Number of Operational DAFTs | 1 | 2 | 3 | N/A |
| Total Energy For Pumping | 925.9 | 1,846.2 | 3,543.3 | kWh/day |
| Total Energy For Pumping | 67.42 | 67.42 | 67.42 | kWh/ton |

The calculation of the sludge pump energy is as follows. Table C.49 and Table C.50 summarizes the sludge pump energy parameters.

1. Calculate the energy of sludge pumping per pump for the average strength using the brake horsepower equation (Jones, et al., 2008). Assume an efficiency of 50%. Assume a TDH of 50 feet.

$$\text{BHP} = \frac{\text{QH}}{3,960\text{e}} = \frac{112 \times 50}{3,960 \times 0.5} = 2.8 \text{ HP} = 2.1 \text{ kW}$$

2. Repeat number 1 for the low and high strengths. This yields 2.1 kW and 2.7 kW as shown in Table C.50.

3. Calculate the number of pumping hours for the average strength.

$$96 \times \frac{5}{60} \times 2 \text{ pumps} = 16 \text{ hrs}$$

4. Repeat number 3 for the low and high strengths. This yields 8 and 24 hours.

5. Calculate the energy requirements per day for the average strength.

   $2.1 \text{ kW } \times 16 \text{ hrs} = 33.7 \text{ kWh/d}$

6. Repeat number 5 for the low and high strengths. This yields 16.9 and 64.7 kWh/day.

7. Calculate the energy requirements per ton of sludge processed for the average strength.

   $$\frac{33.7 \text{ kWh/day}}{\frac{54{,}765}{2{,}000} \text{ton/day}} = 1.23 \text{ kWh/ton}$$

8. Repeat number 7 for the low and high strengths. This yields 1.23 and 1.23 kWh/ton as shown in Table C.50.

**Table C.50 - DAFTs Sludge Pumps Energy Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Pump Specific Weight γ | 62.4 | 62.4 | 62.4 | $lb/ft^3$ |
| Flow Rate For Pump | 112 | 112 | 143 | gpm |
| Provided Efficiency for Pump | 0.5 | 0.5 | 0.5 | N/A |
| Provided TDH for Pump | 50 | 50 | 50 | ft |
| Power Input for Pumps Per DAFT | 2.8 | 2.8 | 3.6 | HP |
| Power Input for Pumps Per DAFT | 2.1 | 2.1 | 2.7 | kW |
| Number of Cycles Per Day | 96 | 96 | 96 | N/A |
| Number of Pumping Hours Per DAFT | 8 | 16 | 24 | hours |
| Total Energy For Pumping | 16.9 | 33.7 | 64.7 | kWh/day |
| Total Energy For Pumping | 1.23 | 1.23 | 1.23 | kWh/ton |

The calculation of the overflow pump energy is as follows. Table C.51 summarizes the overflow pump energy parameters.

1. Calculate the energy of recycle pumping per pump for the average strength using the brake horsepower equation (Jones, et al., 2008). Assume an efficiency of 50%. Assume a TDH of 30 feet.

$$\text{BHP} = \frac{\text{QH}}{3{,}960\text{e}} = \frac{244 \times 30}{3{,}960 \times 0.5} = 3.7 \text{ HP} = 2.8 \text{ kW}$$

2. Repeat number 1 for the low and high strengths. This yields 2.8 kW and 3.5 kW as shown in Table C.51.

3. Calculate the energy requirements per day for the average strength.

   $$2.8 \text{ kW } \times 24 \text{ hrs } \times 2 \text{ DAFTs} = 132.5 \text{ kWh/d}$$

4. Repeat number 3 for the low and high strengths. This yields 66.5 and 254.4 kWh/day as shown .

5. Calculate the energy requirements per ton of sludge processed for the average strength.

   $$\frac{132.5 \text{ kWh/day}}{\frac{54{,}765}{2{,}000} \text{ ton/day}} = 4.84 \text{ kWh/ton}$$

6. Repeat number 5 for the low and high strengths. This yields 4.84 and 4.84 kWh/ton.

**Table C.51 - DAFTs Overflow Pump Energy Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Pump Specific Weight γ | 62.4 | 62.4 | 62.4 | $lb/ft^3$ |
| Flow Rate | 245 | 244 | 312 | gpm |
| Provided Efficiency for Pump | 0.5 | 0.5 | 0.5 | N/A |
| Provided TDH for Pump | 30 | 30 | 30 | ft |
| Number of Pumps | 1 | 2 | 3 | N/A |
| Power Input for Pumps Per Clarifier | 3.7 | 3.7 | 4.7 | HP |
| Power Input for Pumps Per Clarifier | 2.8 | 2.8 | 3.5 | kW |
| Number of Pumping Hours | 24 | 48 | 72 | hours |
| Total Energy Required For Pumping | 66.5 | 132.5 | 254.4 | kWh/day |
| Total Energy Required For Pumping | 4.84 | 4.84 | 4.84 | kWh/ton |

The calculation of the rake arm energy is as follows. Table C.52 presents the rake arm energy parameters.

1. Calculate the rake arm energy using the rake arm energy equation for the average strength (WEF, 1982; WEF, 2005). Assume an efficiency of 75%.

$$\mathrm{P} = \frac{\mathrm{Wr^2}\omega}{550\mathrm{e}} = \frac{6 \times \frac{1}{3.281} \times \frac{2.2}{1} \times \left(\frac{60}{2}\right)^2 \times 0.0139}{550 \times 0.75} = 0.12\ \mathrm{HP} = 0.091\ \mathrm{kW}$$

2. Repeat number 1 for the low and high strengths. This yields 0.091 and 0.091 kW for the low and high strengths.

3. Calculate the total energy requirements per day for the rake arms at average strength:

   $\text{Rake arm energy} = 0.091\ \times 2\ \times 24 = 4.4\ \mathrm{kWh/day}$

4. Repeat number 3 for the low and high strengths. This yields 2.2 and 6.5 kWh/day.

**Table C.52 - DAFTs Rake Arms Energy Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| K Value For Torque | 6 | 6 | 6 | kg/m |
| K Value For Torque | 58.9 | 58.9 | 58.9 | N/m |
| Torque Required Using Equation 20.10 of WEF 1998 | 4,921 | 4,921 | 4,921 | J |
| Alarm Torque | 5,905 | 5,905 | 5,905 | J |
| Shut off Torque | 6,889 | 6,889 | 6,889 | J |
| Failure Torque | 9,842 | 9,842 | 9,842 | J |
| Typical Peripheral Velocity | 7.6 | 7.6 | 7.6 | m/min |
| Typical Peripheral Velocity | 24.9 | 24.9 | 24.9 | ft/min |
| Typical Peripheral Velocity | 0.4 | 0.4 | 0.4 | ft/s |
| Angular Velocity | 1.39E-02 | 1.39E-02 | 1.39E-02 | rad/s |
| Required Motor Size | 0.091 | 0.091 | 0.091 | kW |
| Required Motor Size | 0.12 | 0.12 | 0.12 | HP |
| Energy Usage Per Day | 2.2 | 4.4 | 6.5 | kWh/day |
| Energy Usage Per Ton | 1.59E-01 | 1.59E-01 | 1.25E-01 | kWh/ton |

## 9 Centrifuge Dewatering

Table C.53 summarizes the design criteria used for the centrifuges. The main equation used for the design of the centrifuges was the solids loading rate. Energy consumers for the centrigues include the feed acceleration, and cake conveyance. The design of the centrifuge is unique in that there is only one major design criteria: solids loading. The design selected is based upon centrifuge criteria in Sieger, et al. (2006). Sieger, et al. (2006) includes the centrifuges currently in use at the WWTP used in this research. Feed acceleration energy requirements are as follows (Maloney, et al., 2008):

$$P_{acc}=5.984\times10^{-10}SGQ(\Omega r_p)^2/e$$

Where $P_{acc}$ = feed acceleration energy (HP multiply by 0.746 for kW), SG = specific gravity, Q = feed flow rate (gpm), $\Omega$ = speed (rpm), $r_p$ = pool radius (in), and e = efficiency. The cake conveyance energy requirements are as follows (Maloney, et al., 2008):

$$P_{con}=1.587\times10^{-5}T\Delta/e$$

Where $P_{con}$ = cake conveyace energy requirements, T = torque (lb-in), $\Delta$ = differential speed (rpm), and e = efficiency.

Table C.54 shows the sludge influent parameters and Table C.55 shows the sludge cake parameters.

**Table C.53 - Centrifuge Design Parameters**

| Parameter | Range | Units | Reference |
|---|---|---|---|
| Solids Loading (Blended Primary and Secondary Sludge) | 4,200 | lb/hour | (Sieger, et al., 2006) |

**Table C.54 - Centrifuge Sludge Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Amount of Secondary Sludge Solids produced at Average Flow In All Secondary Clarifiers | 28,916 | 57,656 | 110,656 | lb/day |
| Amount of Secondary Sludge Solids Assuming 95% Solids Capture Efficiency in DAFTS | 27,470 | 54,773 | 105,123 | lb/day |
| Amount of Secondary Sludge Solids Assuming 95% Solids Capture Efficiency in DAFTS | 14 | 27 | 53 | ton/day |
| Amount of Primary Sludge Solids produced at Average Flow In All Primary Clarifiers | 65,303 | 114,682 | 218,949 | lb/day |
| Amount of Primary Sludge Solids Assuming 90% Solids Capture Efficiency in Gravity Thickeners | 58,773 | 103,214 | 197,054 | lb/day |
| Amount of Primary Sludge Solids Assuming 90% Solids Capture Efficiency in Gravity Thickeners | 29 | 52 | 99 | ton/day |
| Total Solids | 86,243 | 157,987 | 302,177 | lb/day |
| Total Solids | 43 | 79 | 151 | ton/day |
| Polymer Dosage | 7 | 7 | 7 | lb/ton |
| Polymer Dosage | 302 | 553 | 1,058 | lb/day |
| Polymer Flow Rate | 967 | 1,772 | 3,390 | $ft^3$/day |
| Concentration of Active Solids in Diluted Polymer | 0.31 | 0.31 | 0.31 | lb/$ft^3$ |
| Polymer Flow Rate | 5 | 9 | 18 | gpm |
| Assumed Cake Solids | 20 | 20 | 20 | % Dry Solids |
| Assumed Solids Capture Efficiency | 95 | 95 | 95 | % Capture |
| Influent Sludge % Solids | 6 | 6.5 | 7 | % Solids |
| Total Volume of Sludge | 23,035 | 38,951 | 69,180 | $ft^3$/day |
| Total Volume of Sludge | 120 | 202 | 359 | gpm |
| Max Flow for Solids per centrifuge | 1,122 | 1,036 | 962 | $ft^3$/hr |
| Max Flow for Solids per centrifuge | 140 | 129 | 120 | gpm |
| Total Flow Into Centrifuge per centrifuge | 145 | 110 | 46 | gpm |
| Centrifuges Required | 1 | 2 | 3 | N/A |

**Table C.55 - Centrifuge Sludge Cake Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Mass of Cake (lb/day) | 81,931 | 150,088 | 287,068 | lb/day |
| Volume of Cake ($ft^3$/day) | 6,565 | 12,026 | 23,002 | $ft^3$/day |
| Volume of Cake (gpm) | 34 | 62 | 119 | gpm |

A summary of the centrifuge design procedure is as follows:

1. Calculate the required polymer dosage for the average flow (WEF, 2010b):

   $$7 \times 79 = 553 \text{ lb/day}$$

2. Repeat number 1 for the low and high strengths. This yields 302 and 1,058 lb/day as shown in Table C.54.

3. Calculate the polymer flowrate for the average flow based upon a diluted polymer concentration of 0.5% and a polymer specific gravity of 1.02 (WEF, 2010b). The density of the polymer is 0.31 lb/$ft^3$.

   $$553 \times \frac{1}{0.31 \times 60 \times 24} \times 7.48 = 9 \text{ gpm}$$

4. Repeat number 3 for the low and high strengths. This yields 5 and 18 gpm as shown in Table C.54.

5. Calculate the flow rate of the sludge for the average strength:

   $$\frac{157{,}987 \times 7.48}{0.065 \times 62.4 \times 24 \times 60} = 202 \text{ gpm}$$

6. Repeat number 5 for the low and high strength. This yields 120 and 359 gpm as shown in Table C.54.

7. Calculate the number of centrifuges required for the average strength based upon the solids loading rate:

$$\frac{157{,}987 \times \frac{1}{24}}{4{,}200} = 2 \text{ centrifuges}$$

8. Repeat number 7 for the low and high strength. This yields 1 and 3 centrifuges as shown in Table C.54.

9. Calculate the cake mass for the average strength:

   $157{,}987 \times 0.95 = 150{,}088 \text{ lb/day}$

10. Repeat number 9 for the low and high strengths. This yields 81,931 and 287,068 lb/day.

The calculation of acceleration energy is as follows. Table C.56 summarizes both the acceleration energy and the conveyance energy parameters.

1. Calculate the acceleration energy per centrifuge using the feed acceleration energy equation (Maloney, et al., 2008):

   $$\mathrm{P_{acc}} = 5.984 \times 10^{-10} \times \frac{\left(\frac{202}{2} + \frac{9}{2}\right)(2{,}500 \times 12.5)^2}{0.9} = 68.7\ \mathrm{HP} = 51.2\ \mathrm{kW}$$

2. Repeat number 1 for the low and high strengths. This yields 60.4 and 60.8 kW as shown in Table C.56.

The calculation of the conveyance energy is as follows. Table C.56 summarizes both the acceleration energy and the conveyance energy parameters.

1. Calculate the conveyance energy per centrifuge using the cake conveyance energy equation (Maloney, et al., 2008):

   $\mathrm{P_{con}}=1.587\times10^{-5}\times265{,}000\times2/0.9=9.3\ \mathrm{HP} = 7.0\ \mathrm{KW}$

2. Repeat number 1 for the low and high strengths. This yields 7.0 and 7.0 kW as shown in Table C.56.

3. Calculate the total energy requirements per day for the average strength:

4. $(7 + 51.2) \times 24 \times 2 \text{ centrifuges} = 2{,}793 \text{ kWh/day}$

**Table C.56 - Centrifuge Energy Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Solids Diameter | 15 | 15 | 15 | in |
| Pool Depth | 12.5 | 12.5 | 12.5 | in |
| Operating Torque | 265,000 | 265,000 | 265,000 | lb-in |
| Operating Torque | 29,941 | 29,941 | 29,941 | N-m |
| Operating Speed | 2,500 | 2,500 | 2,500 | rpm |
| Operating Speed | 262 | 262 | 262 | rad/s |
| Operating Solids Flow | 7.87E-03 | 1.33E-02 | 2.38E-02 | $m^3/s$ |
| Solids Flow Per Centrifuge | 7.87E-03 | 6.67E-03 | 7.93E-03 | $m^3/s$ |
| Assumed Differential | 2 | 2 | 2 | rpm |
| Assumed Differential | 0.21 | 0.21 | 0.21 | rad/s |
| Efficiency Acceleration | 0.9 | 0.9 | 0.9 | % |
| Efficiency Conveyance | 0.9 | 0.9 | 0.9 | % |
| Acceleration Power Per Centrifuge | 60.4 | 51.2 | 60.8 | kW |
| Acceleration Power Per Centrifuge | 80.9 | 68.7 | 81.6 | HP |
| Conveyance Power Per Centrifuge | 7.0 | 7.0 | 7.0 | kW |
| Conveyance Power Per Centrifuge | 9.3 | 9.3 | 9.3 | HP |
| Total Power | 67.3 | 116.4 | 203.4 | kW |
| Total Power | 90.3 | 78.0 | 90.9 | HP |
| Centrifuge Energy | 1,616.3 | 2,793.0 | 4,882.7 | kWh/day |

## 10 Plate and Frame Press Dewatering

The design of the plate and presses is presented in Table C.57 following design procedures in (Davis, 2010). The plate and frame press chosen is the 7,000 L model with a height of 4.2 m and width of 2.7 m from (Davis, 2010). The pumping cycle used is that recommended by U.S. Army Corps of Engineers (2003). The cake solids concentration for the plate and frame presses was assumed to be 36% with 95% solids capture efficiency (Metcalf and Eddy, 2003). The sludge parameters are presented in Table C.58 and the cake parameters are presented in Table C.59.

**Table C.57 - Plate and Frame Press Design**

| | | |
|---|---|---|
| Influent % Solids | 6.5 | % |
| Volume of Sludge | 38,951 | $ft^3$/day |
| Volume of Sludge | 1.10E+06 | L/day |
| Mass of Dewatered Sludge | 150,088 | lb/day |
| Volume of Dewatered Sludge | 6,681 | $ft^3$/day |
| Volume of Dewatered Sludge | 189,164 | L/day |
| Volume Required for Filter Press | 6,700 | L/cycle |
| Plate and Frame Press Height | 4.2 | m |
| Plate and Frame Press Width | 2.7 | m |
| Plate and Frame Press Length | 9.6 | m |
| Plate and Frame Press Volume | 7,000 | L |
| Operating Filter Presses | 2 | N/A |
| Standby Filter Presses | 1 | N/A |
| Total Cycle Time | 6,120 | s |

**Table C.58 - Plate and Frame Press Sludge Parameters**

| | | |
|---|---|---|
| Amount of Secondary Sludge Solids Assuming 95% Solids Capture Efficiency in DAFTS | 54,773 | lb/day |
| Amount of Secondary Sludge Solids Assuming 95% Solids Capture Efficiency in DAFTS | 27 | ton/day |
| Amount of Primary Sludge Solids produced at Average Flow In All Primary Clarifiers | 114,682 | lb/day |
| Amount of Primary Sludge Solids Assuming 90% Solids Capture Efficiency in Gravity Thickeners | 103,214 | lb/day |
| Amount of Primary Sludge Solids Assuming 90% Solids Capture Efficiency in Gravity Thickeners | 52 | ton/day |
| Total Solids | 157,987 | lb/day |
| Total Solids | 79 | ton/day |
| Polymer Dosage | 4.7 | lb/ton |
| Polymer Dosage | 371 | lb/day |
| Polymer Flow Rate | 1,190 | $ft^3$/day |
| Polymer Flow Rate | 6.2 | gpm |
| Assumed Cake Solids | 36 | % Dry Solids |
| Assumed Solids Capture Efficiency | 95 | % Capture |

**Table C.59 - Plate and Frame Press Sludge Cake Parameters**

| | |
|---|---|
| Mass of Cake (lb/day) | 150,088 |
| Volume of Cake ($ft^3$/day) | 6,681.2 |
| Volume of Cake (gpm) | 34.7 |

A summary of the plate and frame press design procedure is as follows:

1. Calculate the required polymer dosage (Metcalf and Eddy, 2003):

$$4.7 \times 79 = 371 \text{ lb/day}$$

2. Calculate the polymer flowrate for the average flow based upon a diluted polymer concentration of 0.5% and a polymer specific gravity of 1.02 (WEF, 2010b). The density of the polymer is 0.31 lb/ft$^3$.

$$31 \times \frac{1}{0.31 \times 60 \times 24} \times 7.48 = 6.2 \text{ gpm}$$

3. Calculate the mass of dewatered sludge:

$$157{,}987 \times 0.95 = 150{,}087 \text{ lb/day}$$

4. Calculate the volume of dewatered sludge:

$$150{,}087 \times \frac{1}{0.36 \times 62.4} = 6{,}681 \text{ ft}^3/\text{d} = 189{,}164 \text{L/d}$$

5. Calculate the volume requirements for the plate and frame press assuming a 6,120 s cycle (Davis, 2010):

$$\frac{189{,}164 \times 6{,}120}{60 \times 60 \times 24 \times 2} = 6{,}700 \text{ L/cycle}$$

To calculate the energy requirements of the plate and frame press. The plate and frame press energy parameters are presented in Table C.60.

1. Calculate the number of filter press cycles:

$$\frac{24}{6{,}120 \times \frac{1}{60} \times \frac{1}{60}} = 14.1$$

2. Calculate the energy requirements based upon the brake horsepower equation. The initial fill energy is as follows:

$$\text{BHP} = \frac{\text{QH}}{3{,}960\text{e}} = \frac{101 \times 129.9}{3{,}960 \times 0.4} = 8.3 \text{ HP} = 6.2 \text{ kW}$$

**Table C.60 - Plate and Frame Press Energy Parameters**

| | | |
|---|---|---|
| Number of Cycles Per Day Per Filter Press | 14.1 | N/A |
| Number of Plate and Frame Presses In Operation | 2 | N/A |
| Pump Efficiency | 40 | % |
| Flow Rate | 6 | lps |
| Flow Rate | 101 | gpm |
| Total Pumping Cycle Time | 5400 | s |
| Total Cycle Time | 6120 | s |
| Initial Fill | 15 | min |
| Initial Fill | 56.3 | psi |
| Initial Fill | 129.9 | ft |
| Initial Fill Energy | 8.3 | HP |
| Initial Fill Energy | 6.2 | kW |
| Initial Fill Energy | 1.5 | kWh/cycle |
| Filtration Stage 1 | 30 | min |
| Filtration Stage 1 | 112.5 | psi |
| Filtration Stage 1 | 259.9 | ft |
| Filtration Stage 1 Energy (HP) | 16.6 | HP |
| Filtration Stage 1 Energy | 12.4 | kW |
| Filtration Stage 1 Energy | 6.2 | kWh/cycle |
| Filtration Stage 2 | 30 | min |
| Filtration Stage 2 | 168.8 | psi |
| Filtration Stage 2 | 389.8 | ft |
| Filtration Stage 2 Energy | 24.9 | HP |
| Filtration Stage 2 Energy | 18.6 | kW |
| Filtration Stage 2 Energy | 9.3 | kWh/cycle |
| Terminate Filtrate | 15 | min |
| Terminate Filtration | 225 | psi |
| Terminate Filtration | 519.8 | ft |
| Terminate Filtration | 33.2 | HP |
| Terminate Filtration | 24.8 | kW |
| Terminate Filtration | 6.2 | kWh/cycle |
| Energy Per Cycle Per Plate and Frame Press | 23.2 | kWh/cycle |
| Energy Per Day | 655.4 | kWh/day |

## 11 Low Pressure High Output UV Disinfection

The low pressure high output UV has an average design flow of 20 MGD and a peak flow of 30 MGD each. The dosage was estimated using the point source summation (PSS) method (U.S. EPA, 1986; WEF, 2010a) in lieu of bioassay data with the Emerick and Darby model used to predict effluent coliform values (WEF, 2010a). The transmittance was assumed as 78, 72, and 68% for the low, average, and high strength cases, respectively.

The main energy consuming units for UV are the lamps. The maximum input of the lamps is 250 W (Trojan UV, 2008). Turndown capabilities were 60% (Trojan UV, 2008).

The low pressure high output UV design is summarized in Table C.61. The low pressure high output UV hydraulics are summarized in Table C.62. The low pressure high output UV headloss is summarized in Table C.63. The low pressure high output UV energy parameters are summarized in Table C.64.

**Table C.61 - Low Pressure High Output UV Design**

| | | |
|---|---|---|
| Lamp Length | 4.92 | ft |
| Lamp and Sleeve Diameter | 0.91 | in |
| Lamp and Sleeve Area | 0.0045 | $ft^2$ |
| Lamp Spacing (center to center) | 4 | in |
| Lamps Per Module | 8 | N/A |
| Modules Per Bank | 22 | N/A |
| Banks Per Channel | 2 | N/A |
| Standby Banks Per Channel | 1 | N/A |
| Lamps Per Channel Not Including Standby | 352 | N/A |
| Lamps Per Channel Including Standby | 528 | N/A |
| UV Input/Output Range | 60-100% | N/A |
| Maximum UV Input | 250 | W |
| Minimum UV Input | 150 | W |
| Maximum UV Output | 85 | W |
| Minimum UV Output | 51 | W |
| Minimum UV Dosage According To Ten States Standards | 30 | $mW \cdot s/cm^2$ |

**Table C.62 - Low Pressure High Output UV Hydraulics**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Width of Channel | 7.4 | 7.4 | 7.4 | ft |
| Depth of Channel | 2.7 | 2.7 | 2.7 | ft |
| Freeboard | 2 | 2 | 2 | ft |
| Area of Channel | 19.7 | 19.7 | 19.7 | $ft^2$ |
| Cross Sectional Area of Channel | 18.9 | 18.9 | 18.9 | $ft^2$ |
| Volume of Liquid Per Lamp (Vv) | 15.0 | 15.0 | 15.0 | L |
| Assumed Transmittance (Metcalf and Eddy Figure 12-41) | 78 | 72 | 68 | % |
| au/cm | 0.11 | 0.14 | 0.17 | au/cm |
| Absorbance coefficient (α) | 0.25 | 0.32 | 0.39 | 1/cm |
| Average Flow Contact Time Per Bank | 3.0 | 3.0 | 3.0 | s |
| Average Flow Contact Time Per Channel | 6.0 | 6.0 | 6.0 | s |

**Table C.63 - Low Pressure High Output UV Headloss**

| | | |
|---|---|---|
| Velocity at Average | 0.50 | m/s |
| Velocity at Average | 49.85 | cm/s |
| Headloss at Average (Metcalf and Eddy) | 0.07 | m |
| Headloss at Average (Metcalf and Eddy) | 0.22 | ft |

**Table C.64 - Low Pressure High Output UV Energy Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| UV Input Required | 156 | 156 | 156 | W |
| UV Output Required | 53.04 | 53.04 | 53.04 | W |
| Percent Illuminated | 62.4 | 62.4 | 62.4 | % |
| UV Density | 3.5 | 3.5 | 3.5 | W/L |
| Nominal Average Intensity (Iavg) From Figure 7-28 of EPA 1986 | 16.5 | 12.2 | 11 | $mW/cm^2$ |
| Adjusted Average Intensity (Iavg) | 9.2 | 6.8 | 6.2 | $mW/cm^2$ |
| Dosage at Average Flow | 55.6 | 41.0 | 37.1 | $mW{\cdot}s/cm^2$ |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Average Flow | 18 | 25 | 50 | MPN/100 mL |
| Energy Per Channel At Average Flow | 54.9 | 54.9 | 54.9 | kW |
| Energy Per Channel Per Day at Average Flow | 1,317.9 | 1,317.9 | 1,317.9 | kWh/day |

An evaluation of the low pressure high output UV design and energy consumption is as follows:

1. Calculate the volume of liquid per lamp (Qasim, 1999):

$$V_V/\text{lamp} = \; (S^2Z) - \left(\frac{\pi d_q^2}{4}\right) Z$$

Where $V_V/\text{lamp}$ = volume of liquid per lamp (L), S = center to center spacing between lamps (cm), Z = arc length of lamp (cm), $d_q$ = diameter of quartz sleeve (cm).

$V_V/\text{lamp}$

$$= \left(\left(4 \times 0.0254 \times \frac{1}{100}\right)^2 \times 4.92 \times \frac{1}{3.281} \times \frac{1}{100}\right)$$

$$- \left(\frac{\pi \times \left(0.91 \times 0.0254 \times \frac{1}{100}\right)^2}{4}\right) \times 4.92 \times \frac{1}{3.281} \times \frac{1}{100}$$

$$= 15.0 \text{ L}$$

2. Calculate the UV density (Qasim, 1999):

   UV density = total UV output per lamp/liquid volume per lamp = 53.04/15 = 3.5 W/L

3. Assume a transmittance of 72% for the average strength and calculate the absorbance unit (Qasim, 1999):

   $70 = 100 \times 10^{-\text{au/cm}}$

   $\text{au/cm} = 0.14$

4. Repeat number 3 for the low and high strengths. This yields 0.11 and 0.17 au/cm.
5. Calculate the absorbance coefficient (Metcalf and Eddy, 2003):

   $\alpha = 2.3 \times 0.14 = 0.32 \text{ cm}^{-1}$

6. Repeat number 5 for the low and high strengths. This yields 0.25 and 0.39 $\text{cm}^{-1}$.
7. Use Figure 7.28 of U.S. EPA (1986) to find the nominal average intensity, $I_{avg,}$ for the average strength:

   Nominal $I_{avg}$ = 12.2 $\text{mW/cm}^2$

8. Repeat number 7 for the low and high strengths. This yields 16.5 and 11 $\text{mW/cm}^2$.

9. Calculate the adjusted nominal adjusted average intensity, $I_{avg,}$ for the average strength (Qasim, 1999; U.S. EPA, 1986):

$$I_{avg} = (\text{Nominal } I_{avg})F_pF_t$$

Where $I_{avg}$ = adjusted average intensity (mW/cm$^2$), Nominal $I_{avg}$ = nominal average intensity (mW/cm$^2$), $F_p$ = ratio of the actual output to the nominal output of the lamps, $F_t$ = ratio of the actual transparency of the quartz sleeve to the nominal transparency.

$$I_{avg} = 12.2 \times 0.8 \times 0.7 = 6.8 \text{ mW/cm}^2$$

10. Repeat number 9 for the low and high strengths. This yields 9.2 and 6.8 mW/cm$^2$.
11. Calculate the cross sectional area of the channel:

$$(7.4 \times 2.7) - (22 \times 8 \times 0.0045) = 18.9 \text{ ft}^2$$

12. Calculate the contact time per bank:

$$\frac{18.9 \times 4.92}{20 \times 1.547} = 3 \text{ s}$$

13. Calculate the contact time per channel:

$$3 \times 2 = 6 \text{ s}$$

14. Calculate the dosage at average flow for the average strength:

$$6.8 \times 6 = 41 \text{ mW} \cdot \text{s/cm}^2$$

15. Repeat number 14 for the low and high strengths. This yields 55.6 and 37.1 mW·s/cm$^2$.
16. Estimate the effluent total coliform count using Figure 19.37 of WEF (2010a) for the average flowrate. This yields 25 MPN/100 mL.
17. Repeat number 16 for the low and high strengths. This yields 18 and 50 MPN/100 mL.

18. Calculate the velocity at average flow:

$$\frac{20 \times 1.54 \times \left(\frac{1}{3.281}\right)^3}{18.9 \times \left(\frac{1}{3.281}\right)^2} = 0.5 \text{ m/s}$$

19. Calculate the headloss at the average flow rate (Metcalf and Eddy, 2003):

$$h_L = k\frac{V^2}{2g}$$

Where $h_L$ = headloss (m), k = headloss coefficient (0.8), V = velocity (m/s), g = gravitational acceleration (9.81 m/s$^2$).

$$h_L = 0.8\frac{0.5^2}{2 \times 9.81} = 0.07 \text{ m} = 0.22 \text{ ft}$$

20. Calculate the energy requirements for the average flowrate:

$$156 \times 352 \times \frac{1}{1{,}000} = 54.9 \text{ kW}$$

21. Calculate the energy requirements for the average flowrate per day per channel:

$$54.9 \times 24 = 1{,}317.6 \text{ kWh/day}$$

## 12 Medium Pressure High Output UV Disinfection

The medium pressure high output UV has an average design flow of 20 MGD and a peak flow of 30 MGD each. The dosage was estimated using the point source summation (PSS) method (U.S. EPA, 1986; WEF, 2010a) in lieu of bioassay data with the Emerick and Darby model used to predict effluent coliform values (WEF, 2010a). The transmittance was assumed as 78, 72, and 68% for the low, average, and high strength cases, respectively.

The main energy consuming units for UV are the lamps. The maximum input of the lamps is 3,200 W MPHO (Trojan UV, 2007). Turndown capabilities were 60% and 30% for LPHO and MPHO (Trojan UV, 2007).

The medium pressure high output UV design is summarized in Table C.65. The medium pressure high output UV hydraulics are summarized in Table C.66. The medium pressure high output UV headloss is summarized in Table C.67. The medium pressure high output UV energy parameters are summarized in Table C.68.

**Table C.65 - Medium Pressure High Output UV Design**

| | | |
|---|---|---|
| Lamp Length | 0.82 | ft |
| Lamp and Sleeve Diameter | 3 | in |
| Lamp and Sleeve Area | 0.049 | $ft^2$ |
| Lamp Spacing (center to center) | 5 | in |
| Lamps Per Module | 16 | N/A |
| Modules Per Bank | 5 | N/A |
| Banks Per Channel | 2 | N/A |
| Standby Banks Per Channel | 0 | N/A |
| Lamps Per Channel Not Including Standby | 160 | N/A |
| Lamps Per Channel Including Standby | 160 | N/A |
| UV Input/Output Range | 30-100% | N/A |
| Maximum UV Input | 3200 | W |
| Minimum UV Input | 960 | W |
| Maximum UV Output | 384 | W |
| Minimum UV Output | 115.2 | W |
| Minimum UV Dosage According To Ten States Standards | 30 | $mW \cdot s/cm^2$ |

**Table C.66 - Medium Pressure High Output UV Hydraulics**

| | Low | Average | High | Units |
|---|---|---|---|---|
| Width of Channel | 5.2 | 5.2 | 5.2 | ft |
| Depth of Channel | 3.2 | 3.2 | 3.2 | ft |
| Freeboard | 2 | 2 | 2 | ft |
| Area of Channel | 16.4 | 16.4 | 16.4 | $ft^2$ |
| Cross Sectional Area of Channel | 12.4 | 12.4 | 12.4 | $ft^2$ |
| Volume of Liquid Per Lamp (Vv) | 2.9 | 2.9 | 2.9 | L |
| Assumed Transmittance (Metcalf and Eddy Figure 12-41) | 78 | 72 | 68 | % |
| au/cm | 0.11 | 0.14 | 0.17 | au/cm |
| Absorbance coefficient (α) | 0.253 | 0.322 | 0.391 | 1/cm |
| Average Flow Contact Time Per Bank | 0.33 | 0.33 | 0.33 | s |
| Average Flow Contact Time Per Channel | 0.66 | 0.66 | 0.66 | s |

**Table C.67 - Medium Pressure High Output UV Headloss**

| | | |
|---|---|---|
| Velocity at Average | 0.76 | m/s |
| Velocity at Average | 75.88 | cm/s |
| Headloss at Average (Metcalf and Eddy) | 0.07 | m |
| Headloss at Average (Metcalf and Eddy) | 0.24 | ft |
| Velocity at Peak | 1.44 | m/s |
| Velocity at Peak | 144.17 | cm/s |
| Headloss at Peak (Metcalf and Eddy) | 0.26 | m |
| Headloss at Peak (Metcalf and Eddy) | 0.87 | ft |

**Table C.68 - Medium Pressure High Output UV Energy Parameters**

| | Low | Average | High | Units |
|---|---|---|---|---|
| UV Input Required | 960 | 1050 | 1325 | W |
| UV Output Required | 115.2 | 126 | 159 | W |
| Percent Illuminated | 30.0 | 32.8 | 41.4 | % |
| UV Density | 39.8 | 43.6 | 55.0 | W/L |
| Nominal Average Intensity (Iavg) From Point Source Summation | 75.5 | 64.1 | 63.8 | $mW/cm^2$ |
| Adjusted Average Intensity (Iavg) | 54.4 | 46.2 | 45.9 | $mW/cm^2$ |
| Dosage at Average Flow | 35.8 | 30.4 | 30.3 | $mW{\cdot}s/cm^2$ |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Average Flow | 60 | 35 | 50 | MPN/100 mL |
| Energy Per Channel At Average Flow | 153.6 | 168 | 212 | kW |
| Energy Per Channel Per Day at Average Flow | 3,686.4 | 4,032 | 5,088 | kWh/day |

An evaluation of the low pressure high output UV design and energy consumption is as follows:

1. Calculate the volume of liquid per lamp (Qasim, 1999):

$$V_V/\text{lamp} = (S^2Z) - \left(\frac{\pi d_q^2}{4}\right)Z$$

Where $V_V$/lamp = volume of liquid per lamp (L), S = center to center spacing between lamps (cm), Z = arc length of lamp (cm), $d_q$ = diameter of quartz sleeve (cm).

$V_V/\text{lamp}$

$$= \left(\left(5 \times 0.0254 \times \frac{1}{100}\right)^2 \times 0.82 \times \frac{1}{3.281} \times \frac{1}{100}\right)$$

$$- \left(\frac{\pi \times \left(3 \times 0.0254 \times \frac{1}{100}\right)^2}{4}\right) \times 0.82 \times \frac{1}{3.281} \times \frac{1}{100} = 2.9\text{ L}$$

2. Calculate the UV density (Qasim, 1999):

   UV density = total UV output per lamp/liquid volume per lamp = 126/2.9 = 43.6 W/L

3. Assume a transmittance of 72% for the average strength and calculate the absorbance unit (Qasim, 1999):

   $70 = 100 \times 10^{-\text{au/cm}}$

   $\text{au/cm} = 0.14$

4. Repeat number 3 for the low and high strengths. This yields 0.11 and 0.17 au/cm.

5. Calculate the absorbance coefficient (Metcalf and Eddy, 2003):

   $\alpha = 2.3 \times 0.14 = 0.32\text{ cm}^{-1}$

6. Repeat number 5 for the low and high strengths. This yields 0.25 and 0.39 $\text{cm}^{-1}$.

7. Figure 7.28 of U.S. EPA (1986) cannot be used to find the nominal average intensity, $I_{avg,}$ for the average strength as the medium pressure lamps produce values that are out of the range of the figure. To calculate the nominal average intensity, $I_{avg}$, apply the PSS method as shown in U.S. EPA (1986).

   Nominal $I_{avg}$ = 64.1 $\text{mW/cm}^2$

8. Repeat number 7 for the low and high strengths. This yields 75.5 and 63.8 $\text{mW/cm}^2$.

9. Calculate the adjusted nominal adjusted average intensity, $I_{avg,}$ for the average strength (Qasim, 1999; U.S. EPA, 1986):

$$I_{avg} = (\text{Nominal } I_{avg})F_pF_t$$

Where $I_{avg}$ = adjusted average intensity ($mW/cm^2$), Nominal $I_{avg}$ = nominal average intensity ($mW/cm^2$), $F_p$ = ratio of the actual output to the nominal output of the lamps, $F_t$ = ratio of the actual transparency of the quartz sleeve to the nominal transparency.

$$I_{avg} = 64.1 \times 0.8 \times 0.7 = 46.2 \text{ mW/cm}^2$$

10. Repeat number 9 for the low and high strengths. This yields 54.4 and 45.9 $mW/cm^2$.

11. Calculate the cross sectional area of the channel:

$$(5.2 \times 3.2) - (16 \times 5 \times 0.049) = 12.4 \text{ ft}^2$$

12. Calculate the contact time per bank:

$$\frac{12.4 \times 0.82}{20 \times 1.547} = 0.33 \text{ s}$$

13. Calculate the contact time per channel:

$$3 \times 2 = 0.66 \text{ s}$$

14. Calculate the dosage at average flow for the average strength:

$$46.2 \times 0.66 = 30.4 \text{ mW} \cdot \text{s/cm}^2$$

15. Repeat number 14 for the low and high strengths. This yields 35.8 and 30.3 $mW\cdot s/cm^2$.

16. Estimate the effluent total coliform count using Figure 19.37 of WEF (2010a) for the average flowrate. This yields 35 MPN/100 mL.

17. Repeat number 16 for the low and high strengths. This yields 60 and 50 MPN/100 mL.

18. Calculate the velocity at average flow:

$$\frac{20 \times 1.54 \times \left(\frac{1}{3.281}\right)^3}{12.4 \times \left(\frac{1}{3.281}\right)^2} = 0.76 \text{ m/s}$$

19. Calculate the headloss at the average flow rate (Metcalf and Eddy, 2003):

$$h_L = k\frac{V^2}{2g}$$

Where $h_L$ = headloss (m), k = headloss coefficient (1.25), V = velocity (m/s), g = gravitational acceleration (9.81 $m/s^2$).

$$h_L = 1.25\frac{0.76^2}{2 \times 9.81} = 0.07 \text{ m} = 0.24 \text{ ft}$$

20. Calculate the energy requirements for the average flowrate:

$$2{,}000 \times 160 \times \frac{1}{1{,}000} = 168 \text{ kW}$$

21. Calculate the energy requirements for the average flowrate per day per channel:

$$168 \times 24 = 4{,}032 \text{ kWh/day}$$

22. Repeat number 21 for the low and high strengths. This yields 3,686.4 and 5,088 kWh/day.

## 13 Chlorination/Dechlorination

The chlorine source for the design was liquid sodium hypochlorite at 12.5% free chlorine (Metcalf and Eddy, 2003). The dechlorination source used in the design was gaseous sulfur dioxide (Metcalf and Eddy, 2003). Both the sodium hypochlorite and sulfur dioxide were assumed to be shipped in. The main design equation for the chlorination is the contact time or hydraulic retention time (HRT). The design flow for the chlorine contact chamber and dechlorination facility is a 10 MGD average flow with a 20 MGD peak flow. The initial coliform bacteria count is $1.0 \times 10^6$ MPN/100 mL and the effluent requirement is less than 200 MPN/100 mL.

Energy consumption for chlorination/dechlorination constitutes the energy needed to power the chemical feed system.

Table C.69 summarizes the chlorination design criteria. Table C.70 summarizes the chlorination design. Table C.71 summarizes the dechlorination design. Table C.72 summarizes the chlorination energy parameters. Table C.73 summarizes the dechlorination energy parameters.

**Table C.69 - Chlorination Design Criteria**

| Parameter | Range | Units | Reference |
|---|---|---|---|
| Typical chlorine dosage for filtered nitrification effluent for ≤200 MPN/100 mL | ≥ 6 and ≤ 12 | mg/L | (Metcalf and Eddy, 2003) |
| Contact time at average flow | 30 – 120 | min | (Metcalf and Eddy, 2003) |
| Contact time at peak flow | 15 – 90 | min | (Metcalf and Eddy, 2003) |
| Length to Width Ratio | 20:1 preferrably 40:1 | N/A | (Metcalf and Eddy, 2003) |
| Depth to Width Ratio | 1:1 or less | N/A | (Black and Veatch, 2010) |
| Minimum Depth | 10 | ft | (Black and Veatch, 2010) |
| Dispersion Number | 0.02-0.004 | N/A | (Metcalf and Eddy, 2003) |
| Velocity through channel | 6.5 – 15 | ft/min | (Metcalf and Eddy, 2003) |

**Table C.70 - Chlorination Design**

| Parameter | Value | Units |
|---|---|---|
| Initial Effluent Chlorine Demand | 4 | mg/L |
| Demand due to decay during chlorine contact | 2.5 | mg/L |
| Required Chlorine Contact Time at Average Flow | 80 | min |
| Required Chlorine Contact Time at Peak Flow | 40 | min |
| Collins-Sellick b | 4 | N/A |
| Collins-Sellick n | 2.8 | N/A |
| Required Chlorine Residual Required at Average Flow | 1.3 | mg/L |
| Required Chlorine Residual Required at Peak Flow | 2.7 | mg/L |
| Chlorine Dosage Required at Average Flow | 7.8 | mg/L |
| Chlorine Dosage Required at Peak Flow | 9.2 | mg/L |
| Length | 480 | ft |
| Width | 12.5 | ft |
| Depth | 12.5 | ft |
| Volume | 75,000 | $ft^3$ |
| Freeboard | 2 | ft |
| Length to Width Ratio (Preferably At Least 40 to 1) | 38.4 | N/A |
| Width to Depth Ratio | 1 | N/A |
| HRT At Average Flow | 80.8 | min |
| Velocity at Average Flow Conditions | 0.1 | ft/s |
| Kinematic Viscosity | 1.06E-05 | $ft^2/s$ |
| Hydraulic Radius | 4.2 | ft |
| Reynolds Number | 1.56E+05 | N/A |
| Dispersion Coefficient | 3.74E-01 | $ft^2/s$ |
| Dispersion Number | 0.0078 | N/A |
| HRT At Peak Flow | 40.4 | min |
| Velocity at Peak Flow Conditions | 0.2 | ft/s |
| Reynolds Number | 3.12E+05 | N/A |
| Dispersion Coefficient | 6.9E-01 | $ft^2/s$ |
| Dispersion Number | 0.014 | N/A |
| Chlorine Type | Sodium Hypochlorite | N/A |
| Chemical Formula | NaOCl | N/A |

| Parameter | Value | Units |
|---|---|---|
| Chlorine Type | Liquid | N/A |
| Percent $Cl_2$ Available | 12.5 | % |
| Concentration of $Cl_2$ | 125,000 | mg/L |
| Concentration of $Cl_2$ | 7.8 | $lb/ft^3$ |
| Average Dosage Required | 654.0 | lb/day |
| Peak Flow Dosage Required | 1,531.7 | lb/day |
| Average Flow Rate Required | 83.8 | $ft^3/day$ |
| Average Flow Rate Required for Chemical Pump | 0.4 | gpm |
| Peak Flow Rate Required | 196.3 | $ft^3/day$ |
| Peak Flow Rate Required For Chemical Pump | 1.0 | gpm |

An evaluation of the chlorination design is follows:

1. Calculate the length to width ratio (Metcalf and Eddy, 2003):

$$\frac{480}{12.5} = 38.4$$

This value is acceptable.

2. Calculate the depth to width ratio (Black and Veatch, 2010):

$$\frac{12.5}{12.5} = 1$$

This value is acceptable.

3. Calculate the HRT at average flow (Metcalf and Eddy, 2003):

$$\mathrm{HRT} = \frac{\mathrm{V}}{\mathrm{Q}} = \frac{480 \times 12.5 \times 12.5}{10 \times 1.547 \times \frac{1}{60}} = 80.8\ \mathrm{min}$$

This value is acceptable.

4. Repeat number 3 for the peak flow. This yields 40.4 min which is acceptable.

5. Calculate the velocity at average flow (Metcalf and Eddy, 2003):

$$V = \frac{Q}{A} = \frac{10 \times 1.547 \times \frac{1}{60}}{12.5 \times 12.5} = 6\ \mathrm{ft/min}$$

This value is just below the range given, however, it is hard to achieve a proper velocity at all flows.

6. Repeat number 5 for the peak flow. This yields 12 ft/min which is acceptable.
7. Use the Collins-Selleck model (Metcalf and Eddy, 2003) with a b value of 5.0 and a n value of 2.8 to calculate the required chlorine residual at average flow. Assume an effluent residual coliform count of 100 MPN/100 mL.

$$\frac{N}{N_0} = (C_R t/b)^{-n}$$

Where N = number of organisms remaining after disinfection at time t, $N_0$ = number of organisms present before disinfection, $C_R$ = chlorine residual remaining at the end of time t, t = contact time, b = value of x-intercept where $N/N_0 = 1$ or $\log N/N_0 = 0$.

$$\frac{100}{1 \times 10^6} = (C_R \times 80/4)^{-2.8}$$

$$C_R = 1.3\ \mathrm{mg/L}$$

8. Repeat number 7 for the peak flow. This yields 2.7 mg/L.
9. Calculate the required chlorine dosage for the average flow (Metcalf and Eddy, 2003). Assume an initial demand of 4 mg/L, a demand due to decay during contact time of 2.5 mg/L (Metcalf and Eddy, 2003).

1.3 + 2.5 + 4 = 7.8 mg/L

This value is acceptable.

10. Repeat number 9 for the peak flow. This yields 9.2 mg/L. This value is acceptable.

11. Calculate the Reynolds number for the average flow (Metcalf and Eddy, 2003):

$$N_R = \frac{4VR}{v}$$

Where $N_R$ = Reynolds number, V = velocity (m/s), R = hydraulic radius (m), $v$ = kinematic viscosity ($1.06 \times 10^{-5}$ ft$^2$/s).

$$N_R = \frac{4 \times 0.1 \times \frac{1}{3.281} \times \left( \frac{12.5 \times 12.5 \times \left(\frac{1}{3.281}\right)^2}{(2 \times 12.5 + 12.5) \times \frac{1}{3.281}} \right)}{1.06 \times 10^{-5}} = 1.56 \times 10^5$$

12. Repeat number 11 for the peak flow. This yields $3.12 \times 10^5$.

13. Calculate the coefficient of dispersion (Metcalf and Eddy, 2003) for the average flow:

$$D = 1.01v(N_R)^{0.875}$$

Where D = coefficient of dispersion.

$$D = 1.01 \times 1.06 \times 10^{-5}(1.56 \times 10^5)^{0.875} = 0.374 \text{ ft}^2/\text{s}$$

14. Repeat number 13 for the peak flow. This yields 0.686 ft$^2$/s.

15. Calculate the dispersion number (Metcalf and Eddy, 2003) for the average flow:

$$d = \frac{Dt}{L^2}$$

Where d = dispersion number, L = length (m).

$$d = \frac{0.374 \times 60}{\left(480 \times \frac{1}{3.281}\right)^2} = 0.0078$$

This value is acceptable.

16. Repeat number 15 for the peak flow. This yields 0.014.

17. Calculate the required pounds per day of chlorine for the average flow (Metcalf and Eddy, 2003):

$$10 \times 7.8 \times 8.34 = 654 \text{ lb/day}$$

18. Repeat number 17 for the peak flow. This yields 1531.7 lb/day.

19. Calculate the required flowrate of chlorine for the average flow:

$$654 \times \frac{1}{7.8} \times 7.48 \times \frac{1}{24} \times \frac{1}{60} = 0.4 \text{ gpm}$$

20. Repeat number 19 for the peak flow. This yields 1 gpm.

**Table C.71 - Dechlorination Design**

| | | |
|---|---|---|
| Dechlorination Choice | Sulfur Dioxide | N/A |
| Chemical Formula | $SO_2$ | N/A |
| Sulfur Type | Gas | N/A |
| Stoichiometric Ratio Needed | 1.2 | N/A |
| Sulfur Dosage Needed at Average Flow | 1.6 | mg/L |
| Sulfur Dosage Needed at Average Flow | 1.0E-04 | $lb/ft^3$ |
| Sulfur Dosage Needed at Average Flow | 134.2 | lb/day |
| Sulfur Dosage Needed at Peak Flow | 3.2 | mg/L |
| Sulfur Dosage Needed at Peak Flow | 2.0E-04 | $lb/ft^3$ |
| Sulfur Dosage Needed at Peak Flow | 537.0 | lb/day |
| Average Flow Rate Required | 134.3 | $ft^3/day$ |
| Average Flow Rate Required for Chemical Pump | 0.7 | gpm |
| Peak Flow Rate Required | 537.3 | $ft^3/day$ |
| Peak Flow Rate Required For Chemical Pump | 2.8 | gpm |

An evaluation of the dechlorination design is as follows:

1. Calculate the sulfur dioxide required at average flow (Metcalf and Eddy, 2003):

   $1.2 \times 1.3 = 1.6 \text{ mg/L}$

2. Repeat number 1 for the peak flow. This yields 3.2 mg/L.
3. Calculate the number of pounds required per day for sulfur dioxide at average flow.

   $10 \times 8.34 \times 1.6 = 134.2 \text{ lb/day}$

4. Repeat number 3 for the peak flow. This yields 537 lb/day.
5. Calculate the required flowrate at average flow:
6. Repeat number 5 for the peak flow. This yields 2.8 gpm.

**Table C.72 – Chlorination Energy Parameters**

| | | |
|---|---|---|
| Actual Power Input | 1.5 | HP |
| Actual Power Input | 1.12 | kW |
| Total Power Input | 268.6 | kWh/day |

The calculation of the energy for chlorination is as follows:

1. A suitable pump was found to have a motor size of 1.5 HP (1.12 kW) (Madden Manufacturing).
2. Calculate the energy requirements per day:

   $1.12 \times 24 \times 10 = 268.6 \text{ kWh/day}$

**Table C.73 – Dechlorination Energy Parameters**

| | | |
|---|---|---|
| Actual Power Input | 0.04 | HP |
| Actual Power Input | 0.03 | kW |
| Total Power Input | 7.2 | kWh/day |

The calculation of the energy for dechlorination is as follows:

1. A suitable dechlorinator was found to have a motor size of 0.04 HP (0.03 kW) (WEF, 1982).
2. Calculate the energy requirements per day:

   $0.03 \times 24 \times 10 = 7.2 \text{ kWh/day}$

REFERENCES

**BASE Energy** Energy Baseline Study for Municipal Wastewater Treatment Plants [Book]. - San Fransisco, CA : Pacific Gas and Electric Company, 2006.
**Bernier D. and et al.** Using Green to Get to Blue: How Alternative Energy Technologies Reduce the Carbon Footprint of Wastewater Treatment [Conference] // WEFTEC. - Los Angeles : 2011.
**Black and Veatch** White's Handbook of Chlorination and Alternative Disinfectants [Book]. - Hoboken, New Jersey : Wiley, 2010.
**Cakir F. and Stenstrom M.** Greenhouse gas production: A comparison between aerobic and anaerobic wastewater treatment technology [Journal]: Water Research, 2005. - 17 : Vol. 39.
**Carbon Enterprises, Inc.** CEI Anthracite [Online] // Carbon Enterprises, Inc. 2006. November 12, 2006. - http://www.ceifiltration.com/anthracite.asp?p=technical.
**Chen W. and et al.** The Civil Engineering Handbook [Book]. - Boca Raton : CRC Press, 2003.
**Collingwood B. and et al.** Going off the Grid: Pittsfield, MA Wastewater Treatment Plant Renewable Energy and Efficiency Program [Conference] // WEFTEC. - Los Angeles : 2011.
**Davis M.** Water and Wastewater Engineering: Design Principles and Practice [Book]. - New York : McGraw-Hill, 2010.
**EnviroSim Associates LTD.** BioWin Information [Online] // Envirosim Associates LTD. 2003. September 04, 2012. - http://www.envirosim.com/products/bw32/bw32intro.php.
**European Commission Eurostat** Electricity prices for industrial consumers [Online] // European Commission Eurostat. - December 20, 2011. - February 2010, 2012. - http://epp.eurostat.ec.europa.eu/tgm/table.do?tab=table&init=1&plugin=1&language=en&pcode=ten00114.
**Europump and Hydraulic Institute** Variable Speed Pumping: A Guide to Successful Applications [Book]. - Oxford : Elsevier, 2004.
**Flores-Alsina X.** Including greenhouse gas emissions during benchmarking of wastewater treatment plant control strategies [Journal]. - : Water Research, 2011. - 16 : Vol. 45.
**Foley J. and et al.** Comprehensive life cycle inventories of alternative wastewater treatment systems [Journal]. - : Water Research, 2010. - 5 : Vol. 44.
**GLUMRB** Recommended Standards for Wastewater Facilities [Book]. - Albany, NY : Health Education Services Division, 2004.
**Gori R. and et al.** Effects of soluble and particulate substrate on the carbon and energy footrpint on wastewater treatment processes [Journal]. - : Water Research, 2011. - 18 : Vol. 45.
**Goulds Pumps** Pump Selection System [Online] // ITT. - May 3, 2012. - http://www.gouldspumps.com/pss.html.
**Hernández-Sancho F. and et al.** Energy efficiency in Spanish wastewater treatment plants: A non-radial DEA approach [Journal]. - : Science of The Total Environment, 2011. - 14 : Vol. 409.

**Hernández-Sancho F. and Sala-Garrido R.** Technical efficiency and cost analysis in wastewater treatment processes: A DEA approach [Journal]. - : Desalination, 2009. - 1 : Vol. 249.
**JO TECHNOLOGIES, LLC** Energy Efficient Sludge Management Evaluation For Reduced Energy Consumption [Book]. - Albany, NY : NYSERDA, 2008. - Vol. .
**Jones G [et al.]** Pumping Station Design [Book]. - Ipswich, MA : Elsevier/Butterworth-Heinemann, 2008.
**Keller J. and Hartley K.** Greenhouse gas production in wastewater treatment: process selection is the major factor [Journal]. - : Water Science and Technology, 2003. - 12 : Vol. 47.
**Krasner St. and et al.** Occurrence of Disinfection Byproducts in United States Wastewater Treatment Plant Effluents [Journal]. - [s.l.] : Environmental Science & Technology, 2009. - 21 : Vol. 43.
**Madden Manufacturing** MetriFlow Diaphragm Metering Pumps [Online] // Madden Manufacturing. - March 29, 2012. - http://maddenmfg.thomasnet.com/item/diaphragm-metering-pumps/metriflow-diaphragm-metering-pumps/mh1180ef?.
**Madden Manufacturing** MH Series Diaphragm Metering Pumps [Online] // Madden Manufacturing. - March 29, 2012. - http://www.maddenmfg.com/metering-pumps/pumps-mh-series.cfm.
**Maloney J. and et al.** Perry's Chemical Engineers' Handbook [Book]. - New York : McGraw-Hill, 2008.
**Mays L.** Water Resources Engineergin [Book]. - Hoboken, NJ : Wiley, 2005.
**Metcalf and Eddy** Wastewater Engineering: Treatment and Reuse [Book]. - New York : McGraw-Hill, 2003.
**Moke A. and et al.** Quantifying the Greenhouse Gas Emissions at the City of Los Angeles, Bureau of Sanitation [Conference] // WEFTEC. - Los Angeles : WEF, 2011.
**Molinos-Senante M., Hernández-Sancho F. and Sala-Garrido R.** Economic feasibility study for wastewater treatment: A cost–benefit analysis [Journal] // Science of the Total Environment. - 2010. - 20 : Vol. 408. - pp. 4396-4402.
**Moyno** Robbins & Myers, Inc. [Online]. - March 15, 1999. - May 3, 2012. - http://www.moyno.com/pdfs/sani_curve2402.pdf.
**MWH** Water Treatment: Principles and Design [Book]. - Hoboken, New Jersey : Wiley, 2005.
**National Archives and Records Administration** Code of Federal Regulations Title 40 Part 133 - Secondary Treatment Regulation [Online]. - September 27, 2012. - September 29, 2012. - http://ecfr.gpoaccess.gov/cgi/t/text/text-idx?c=ecfr&sid=6886fabf25f45bd9544d5ce233e02c14&rgn=div5&view=text&node=40:23.0.1.1.20&idno=40.
**Ortiz M. and et al.** Life cycle assessment of water treatment technologies: wastewater and water-reuse in a small town [Journal]. - : Desalination, 2006. - 1-3 : Vol. 204.
**Pacific Gas and Electric Company** Municipal Wastewater Treatment Plant Energy Baseline Study [Book]. - San Francisco, Ca : , 2003. - Vol. .
**Planyc** Inventory of New York City Greenhouse Gas Emissions [Report]. - New York : Planyc, 2011.

**Poulsen T. and Hansen J.** Assessing the impacts of changes in treatment technology on energy and greenhouse gas balances for organic waste and wastewater treatment using historical data [Journal]. - : Waste management & research, 2009. - 9 : Vol. 27.
**Qasim S.** Wastewater treatment plants: planning, design, and operation [Book]. - Boca Raton, Florida : CRC Press, 1999.
**Rebhun M. and et al.** Formation of Disinfection Byproducts during Chlorination of Secondary Effluent and Renovated Water [Journal]. - [s.l.] : Water Environment Research, 1997. - 6 : Vol. 69.
**Red Flint Sand and Gravel** Sand and Gravel Water Filtration Media - Red Flint [Online] // Red Flint Sand and Gravel Web Site. - November 12, 2012. - http://www.redflint.com/filter_sand.htm.
**Rittman B. and McCarty P.** Environmental Biotechnology: Principles and Applications [Book]. - New York : McGraw-Hill, 2000.
**Rosso D. and Bolzonella D.** Carbon footprint of aerobic biological treatment of winery wastewater [Journal]. - : Water Science & Technolgy, 2009. - 5 : Vol. 60.
**Rosso D. and Stenstrom M.** Energy-Saving Benefits of Denitrification [Journal]. - Annapolis, Maryland : American Academy of Environmental Engineers, 2007. - Vol. 3.
**Rosso D. and Stenstrom M.** The carbon-sequestration potential of municipal wastewater treatment [Journal]. - [s.l.] : Chemosphere, 2008. - 8 : Vol. 70.
**Seeta V. and et al.** "Free" Solar Power – A Big Leap Towards Energy Self-Sufficiency at WWTPs [Conference] // WEFTEC. - Los Angeles : [s.n.], 2011.
**Shahabadi M. and et al.** Estimation of greenhouse gas: generation in wastewater treatment plants model development and application [Journal]. - : Chemosphere, 2010. - 9 : Vol. 78.
**Shahabadi M. and et al.** Impact of process design on greenhouse gas (ghg) generation by wastewater treatment plants [Journal]. - : Water Research, 2009. - 10 : Vol. 43.
**Sieger R. and et al.** Centrifuge selection and specification for the clark county water reclamation district [Conference] // Residuals and Biosolids Management Conference. - : WEF, 2006.
**Sterman J.** Business Dynamics: Systems Thinking and Modeling for a Complex World [Book]. - New York : McGraw-Hill, 2000.
**Trojan UV** Public Health & Environmental Advantages of UV [Report]. - London, Ontario, Canada : Trojan UV, 2005.
**Trojan UV** Trojan UV 3000 Plus [Online] // Trojan UV. - 2008. - March 29, 2012. - http://trojanuv.com/resources/trojanuv/Products/TrojanUV3000Plus/TROW1040_3000Plus_Detailer_LR.pdf.
**Trojan UV** Trojan UV 4000 Plus [Online]. - 2007. - March 29, 2012. - http://trojanuv.com/resources/trojanuv/Products/TrojanUV4000Plus/TROJANUV_4000Plus.pdf.
**U.S. Army Corps of Engineers** Army Plate and Frame Filter Press [Book]. - [s.l.] : U.S. Army Corps of Engineers, 2003.
**U.S. Energy Information Association** Total Energy [Online] // U.S. Energy Information Association. - October 19, 2011. - February 9, 2012. - http://www.eia.gov/totalenergy/data/annual/showtext.cfm?t=ptb0810.
**U.S. EPA** Auxiliary and Supplemental Power Fact Sheet: Fuel Cells [Book]. - Washington, D.C. : U.S. EPA, 2011.

**U.S. EPA** Auxiliary and Supplemental Power Fact Sheet: Microturbines [Book]. - Washington, D.C. : U.S. EPA, 2011.
**U.S. EPA** Auxiliary and Supplemental Power Fact Sheet: Solar Power [Book]. - Washington, D.C. : U.S. EPA, 2007.
**U.S. EPA** Auxiliary and Supplemental Power Fact Sheet: Wind Turbines [Book]. - Washington, D.C. : U.S. EPA, 2007.
**U.S. EPA** Clean Watersheds Needs Survey 2004: Report to Congress [Book]. - Washington, D.C : U.S. EPA, 2008.
**U.S. EPA** Clean Watersheds Needs Survey 2008: Report to Congress [Book]. - Washington, D.C : U.S. EPA, 2010a.
**U.S. EPA** Design Manual: Fine Pore Aeration Systems [Book]. - Washington, D.C. : U.S. EPA, 1989.
**U.S. EPA** Design Manual: Municipal Wastewater Disinfection [Book]. - Washington D.C. : U.S. EPA, 1986.
**U.S. EPA** Drinking Water Contaminants [Online] // U.S. EPA Website. - June 5, 2012. - September 10, 2012. - http://water.epa.gov/drink/contaminants/index.cfm.
**U.S. EPA** Energy Conservation in Municipal Wastewater Treatment [Book]. - Washington, D.C. : U.S. EPA, 1978.
**U.S. EPA** Evaluation of Energy Conservation for Wastewater Treatment Facilities [Book]. - Washington, D.C. : U.S. EPA, 2010b.
**U.S. EPA** Inventory of U.S. Greenhouse Gas Emissions and Sinks: 1990 - 2009 [Book]. - Washington, D.C. : U.S. EPA, 2011.
**U.S. EPA** Nutrient Control Design Manual [Book]. - Washingon, D.C. : U.S. EPA, 2010c.
**U.S. EPA** Process Design Manual for Sludge Treatment and Disposal [Book]. - Washington, D.C. : U.S. EPA, 1979.
**UNFCCC** Kyoto Protocol To The United Nations Framework [Conference] // Convention On Climate Change. - Kyoto, Japan : UNFCCC, 1998.
**URS Corporation** Evaluation of Ultraviolet (UV) Radiation Disinfection Technologies for Wastewater Treatment Plant Effluent [Book]. - Albany, New York : New York State Energy and Research Development Authority, 2004.
**URS Corporation** Evaluation of Ultraviolet (UV) Radiation Disinfection Technologies for Wastewater Treatment Plant Effluent [Book]. - Albany, New York : New York State Energy and Research Development Authority, 2004.
**Vaughan** Vaughan E Series [Online] // Vaughan. - 2009. - March 28, 2012. - http://www.chopperpumps.com/pdfs/curves/e3pvari.pdf.
**Vidal N. and et al.** Evaluation of the Environmental Implications to Include Structural Changes in a Wastewater Treatment Plant [Journal]. - : Journal of Chemical Technology and Biotechnology, 2002. - 11 : Vol. 77.
**Vulcan Industries, Inc.** Mensch Crawler Bar Screen [Book]. - Missouri Valley, Iowa : Vulcan Industries, Inc..
**WEF** Clarifier Design [Book]. - New York : McGraw-Hill, 2005.
**WEF** Design of Municipal Wastewater Treatment Plants [Book]. - New York : McGraw-Hill, 2010a. - Vol. 2.
**WEF** Design of Municipal Wastewater Treatment Plants [Book]. - New York : McGraw-Hill, 2010b. - Vol. 3.

**WEF** Energy Conservation in the Design and Operation of Wastewater Treatment Facilities [Book]. - Washington, D.C. : Lancaster Press, 1982.
**WEF** Energy Conservation in Water and Wastewater Facilities [Book]. - New York : McGraw-Hill, 2009.
**WEF** Membrane Bioreactors [Book]. - New York : McGraw Hill, 2012.
**WEF** Membrane Systems for Wastewater Treatment [Book]. - New York : McGraw-Hill, 2006.
**WEF** Nutrient Removal [Book]. - New York : McGraw-Hill, 2010c.
**WEF** Operation of Municipal Wastewater Treatment Plants [Book]. - New York : McGraw-Hill, 2008. - Vol. 2.
**Wei Y. and et al.** Estimation of Greenhouse Gas Emissions by the Wastewater Treatment Plant of a Locomotive Repair Factory in China [Journal]. - : Water Environment Research, 2008. - 12 : Vol. 80.

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