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# Investigating the Impacts of Conventional and Advanced Treatment Technologies on Energy Consumption at Satellite Water Reuse Plants

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# INVESTIGATING THE IMPACTS OF CONVENTIONAL AND ADVANCED TREATMENT TECHNOLOGIES ON ENERGY CONSUMPTION AT SATELLITE WATER REUSE PLANTS


By

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# ABSTRACT

## INVESTIGATING THE IMPACTS OF CONVENTIONAL AND ADVANCED TREATMENT TECHNOLOGIES ON ENERGY CONSUMPTION AT SATELLITE WATER REUSE PLANTS

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With the ever increasing world population and the resulting increase in industrialization and agricultural practices, depletion of two of the world's most important natural resources, water and fossil fuels, is inevitable. Water reclamation and reuse is the key to protecting these natural resources. Water reclamation using smaller decentralized wastewater treatment plants, known as satellite water reuse plants (WRP), have become popular in the last decade. With stricter standards and regulations on effluent quality and requirements for a smaller land footprint (i.e. real estate area), additional treatment processes and advanced technologies are needed. This greatly increases the energy consumption of an already energy intensive process. With growing concerns over the use of nonrenewable energy sources and the resulting greenhouse gas (GHG) emissions, WRPs are in need of energy evaluations. This research investigated the energy consumption of both conventional and advanced treatment processes in satellite WRPs with average flows varying from 1 to 11 MGD and was calculated using accepted industry design criteria and equations. The associated carbon footprint from energy consumption at these facilities was determined in carbon dioxide equivalents on a per MG treated basis. Renewable energy sources, solar and anaerobic digestion, were incorporated into the WRPs in an attempt to offset the energy consumption and GHGs emitted. Results of this research provide a means for engineers and operators to evaluate unit processes based on energy consumption and provide a foundation for decision making regarding sustainability of using advanced treatment technologies at the reuse facility.

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# CHAPTER 1

# INTRODUCTION

With the ever growing increase in the world's population and the resulting increase in industrialization and agricultural practices, the depletion of two of the world's most important natural resources, water and fossil fuels, is inevitable. Water is the most abundant resource in the world but with only one percent of the world's water resources being fresh water, this abundant resource needs to be protected (Urkiaga, et al., 2008). Water and wastewater collection, distribution, and treatment consumes two to four percent of the total power consumed in the United States (McMahon, et al., 2011; Daigger, 2009; U.S. EPA, 2010; Metcalf & Eddy, Inc, 2003; EPRI, 2002; WEF, 2010b); making the water and wastewater industry the third largest energy consumer, behind primary metals and chemicals. (McMahon, et al., 2011; EPRI, 2009). Thus, water and energy are intertwined resources. This current usage of energy requires between 100 and 123.45 billion kWh each year (U.S. EPA, 2010; EPRI, 2009) and emits roughly 116 billion lbs (52 million metric tonnes) of carbon dioxide ($CO_2$) into the atmosphere (McMahon, et al., 2011; NRDC, 2009). Due to the increase in population, higher levels of treatment mandated by regulations, and the employment of advanced technologies to treat to higher treatment levels, it has been estimated that during the next 15 years wastewater loads are expected to increase by 20% (U.S. EPA, 2008); resulting in an increase of 30 to 40% in energy consumption for wastewater treatment facilities during the next 20 to 30 years in the country (Metcalf & Eddy, Inc, 2003).

Ways to curb the large energy consumption in wastewater treatment plants (WWTP) has been an upcoming topic of interest. There are at least two ways to decrease energy use within an existing WWTP: (1) the increase of efficiencies in plant equipment; (2) and the optimization of plant processes and equipment. There is however a limit to how much energy within an existing plant can be curbed, because current design requires a minimum amount of energy to run installed processes and equipment. As a result, new approaches are needed to curb electrical energy consumption, not only for existing WWTPs but also for future planned plants.

Fossil fuels represent between 80-84% of the world's electrical energy supply today (Demirbas, 2009; Gude, et al., 2010). At this current consumption rate, known petroleum reserves are projected to be depleted in less than 50 years (Demirbas, 2009; Gude, et al., 2010). There are two main downsides for the use of fossil fuels as energy: all types of fossils fuels are finite resources; and the production of energy from fuels produce large amounts of greenhouse gas (GHG) emissions. In WWTPs, consumption of electric power accounts for about 90% of the total energy consumption in a plant (Mizuta, et al., 2010). Thus, the increasingly large amount of energy consumption from WWTPs greatly contributes to the production of GHG emissions. These emissions are subsequently resulting in crucial environmental problems worldwide, including acid rain and global warming (Gude, et al., 2010). One way to help curb GHG emissions is to conserve energy consumed in WWTPs, as mentioned. Additionally, GHG emissions can be minimized by implementing renewable energy resources in WWTPs.

Currently, renewable energy only represents a 14-16% total of the world's energy. This number has been projected to reach 48-50% by the year 2040 (Demirbas, 2009;

Gude, et al., 2010). There are a number of WWTPs that have integrated renewable energy sources (i.e. solar energy and biosolids digestion) as a part of their power grid (Bernier, et al., 2011; Gude, et al., 2010). Most of these plants incorporated these sources of energy as part of their renewable energy portfolio that was established by state regulations. To increase the percent of total energy that plants can use from renewable sources, energy considerations must be introduced during the design phase. With the cost and depletion of fossil fuels rapidly rising (Mizuta, et al., 2010; Brandt, et al., 2011), the need to conserve energy and transition from fossil fuels to renewable energy has now become a necessity not a luxury.

It is expected that by the year 2025, the percentage of the world population that lives in water short/stressed environments will increase by 45% (Daigger, 2009). Water reclamation and reuse is the key to protecting this natural resource. Water reclamation and reuse has been practiced in the form of wastewater treatment by the use of WWTPs. Reuse water can be used for a variety of applications including irrigation, recreational uses, groundwater recharge, nonpotable reuse, and potable reuse (Metcalf & Eddy, Inc, 2007; Tchobanoglous, et al., 2004; Metcalf & Eddy, Inc, 2003). WWTPs are generally centralized plants that treat wastewater collected from the entire community. Typically, wastewater treated in centralized facilities is discharged into a receiving water body (e.g. river or lake). In recent decades, smaller decentralized wastewater treatment plants, termed satellite water reuse plants (WRP) or scalping plants, have become very prevalent. WRPs are satellite treatment facilities that treat wastewater from a specific part of a community and reuse the effluent in or around the location where the wastewater was collected. This practice allows for conservation of freshwater because reuse water is utilized instead. Because of the close proximity and/or potential direct contact of reclaimed water with the general public, regulations and effluent standards for reuse water are strict and are becoming stricter (Crook, 2011). To achieve these stricter standards on effluent quality and a smaller real estate area, additional treatment processes along with advanced technologies are needed (Bennett, 2007; EPRI, 2002; Brandt, et al., 2011; Urkiaga, et al., 2008).

The use of advanced treatment technologies to treat reuse water requires a large increase in energy consumption compared to conventional unit processes. In the past, energy consumption and GHG generation has not been a concern in reuse plant design. However, the current efforts to minimize GHG emissions and related energy footprint challenges the actual benefits of reuse plants. With the increase in WRPs and the use of advanced treatment technologies rising, energy consumption within these facilities must be evaluated. Research on energy consumption has been performed for many centralized WWTPs in specific sites (Sobhani, et al., 2011) and for whole regions (Mizuta, et al., 2010; Yang, et al., 2010). In addition, energy consumption research has been performed on specific individual unit processes and equipment (Messenger, et al., 2011; Pellegrin, et al., 2011; Brandt, et al., 2011). However, a complete evaluation of energy consumption in WRPs has not been reported to date, as compared to centralized plants. In this research, a typical WRP is designed and its associated energy consumption was estimated based on major energy consuming units. In addition, associated GHG emissions from electrical energy consumption and the renewable energy potential of the WRP is determined to evaluate the savings in GHG emissions.

## 1. Objectives and Hypotheses

The specific objectives and hypotheses of this research are:

1. To design a satellite WRP for varying flowrates and determine the associated energy consumption and carbon footprint for each individual unit process of the entire plant. To determine the impact on energy consumption when replacing advanced treatment processes with conventional treatment processes. It is expected that advanced treatment units will consume more energy; however, the magnitude of the difference remains to be determined.
2. To determine the associated renewable energy benefit from incorporating renewable energy sources (e.g. solar and biosolids digestion) into the previously designed WRPs. This involves incorporating renewable energy sources onto the existing real estate acreage of the WRP. WRPs are compact and do not have extensive space for photovoltaic (PV) solar system installation, however it is expected that at least some fraction of the energy consumption can be met by implementing renewable sources. Sludge digestion is also expected to contribute to meeting some of the energy consumption.
3. To compare energy footprint and associated real estate area needed of advanced treatment technologies versus conventional treatment technologies required for conventional activated sludge (CAS) and membrane bioreactor (MBR) as treatment processes in WRPs. Advanced treatment with MBRs are generally more compact, therefore savings in real estate area needed is expected.

# CHAPTER 2

# ENERGY IMPACTS OF CONVENTIONAL AND ADVANCED TREATMENT TECHNOLOGIES AT SATELLITE WATER REUSE PLANTS AS A FUNCTION OF FLOW

## 1. Introduction

The depletion of two of the world's most important natural resources, water and fossil fuels, has become difficult to control due to population growth that has resulted in increased industrialization and agricultural practices. Currently, with only one percent of the world's water resources being fresh water, this abundant resource needs to be protected (Urkiaga, et al., 2008). Water reclamation and reuse is the key to protecting this natural resource. Water reclamation has been practiced in the form of wastewater treatment plants (WWTP) using centralized treatment facilities located at low elevations to allow gravity collection of wastewater from the metropolitan area. In the United States, applications of water reuse in order of descending water volumes are: agricultural irrigation, industrial recycling and reuse, landscape irrigation, groundwater recharge, recreational and environmental uses, nonpotable urban uses, and finally potable reuse (Leverenz, et al., 2011; Metcalf & Eddy, Inc, 2007; Tchobanoglous, et al., 2004).

Direct potable reuse is not practiced in the United States, except for reuse after groundwater recharge. An example is Orange County, California, where treated wastewater effluent discharges into aquifer recharge basins into the county's groundwater basin that is used for potable purposes (Metcalf & Eddy, Inc, 2007; Orange County Water District, 2012; Tchobanoglous, et al., 2011). Internationally, water reuse is being practiced in a similar fashion as in the United States, including in China (Yi, et al., 2011), Japan (Kazmi, 2005; Asano, et al., 1996), Europe (Bixio, et al., 2006; Angelakis, et al., 2008), and Africa (Ilemobade, et al., 2008).

Two areas leading the way in water reuse worldly are Singapore and Windhoek, Namibia. In Singapore, high-grade reclaimed water (NEWater), is used for several nonpotable reuse applications, but most importantly for planned indirect potable reuse (Public Utilities Board, 2012; Daigger, 2009). This is accomplished by mixing NEWater with raw water before sending through a drinking water treatment facility (Public Utilities Board, 2012; Onn, 2005). In Windhoek, Namibia direct potable reuse has been practiced since 1968, due to arid desert climate, lack of nearby rivers, and low groundwater (Metcalf & Eddy, Inc, 2007; Tchobanoglous, et al., 2011; du Pisani, 2006). The highly-treated reclaimed water is blended directly into the potable pipeline that feeds to the water distribution network of the city (Metcalf & Eddy, Inc, 2007; Tchobanoglous, et al., 2011). Windhoek is the only area in the world that operates and practices direct potable reuse of reclaimed wastewater (du Pisani, 2006; Metcalf & Eddy, Inc, 2007).

The reuse of water has been limited through time due to the lack of risk assessment, incentives, and public perception (Urkiaga, et al., 2008; Hartley, 2006). Public perception has been a major obstacle in the progression of water reuse, primarily because of the “yuck factor” (Hartley, 2006). The “yuck factor” can be avoided if reuse water does not come in direct contact with the public (Hartley, 2003; Toze, 2006). Thus, reuse applications today are limited to noncontact, non-potable use. Risk assessment has been a continuous research topic since the beginning of water reclamation, and especially recently with developing concerns over endocrine disrupting compounds and pharmaceutically-active compounds (Toze, 2006; Salgot, et al., 2006; Cleary, et al., 2011; Huertas, et al., 2008). Through each study, new progress has been made requiring stricter standards (Crook, 2011) by governing bodies (e.g. World Health Organization (WHO) (WHO, 2006), United States Environmental Protection Agency (USEPA) (U.S. EPA, 2004a), and state regulatory agencies (U.S. EPA, 2004a)). To achieve these stricter standards of effluent quality, additional treatment processes along with new technologies are needed (Bennett, 2007; EPRI, 2002; Brandt, et al., 2011; Urkiaga, et al., 2008). This factor has led the use of high performance advanced treatment processes, which in turn drive up the energy consumption and price of reuse water.

In the last decade, to overcome the obstacle of cost, decentralized wastewater management (DWM) has become the norm. DWM is defined by Tchobanoglous, et al. (2004) as “the collection, treatment, and reuse of wastewater from individual homes, cluster of homes, subdivisions, and isolated commercial facilities at or near the point of waste generation”. By means of using DWM, development of small WWTPs known as water reuse plants (WRP) have become popular, especially in the last decade (Metcalf & Eddy, Inc, 2007). WRPs are satellite treatment facilities typically located near potential reuse applications in urban areas and integrated with a centralized treatment facility. This allows WRPs to be strategically placed throughout an urban community where reuse demand is needed (Daigger, 2009).

WRPs are small in stature as their effluent is treated to non-potable reclamation grade water and all solids/residuals produced during the biological treatment are discharged back into the collection system for processing at the centralized treatment facility (Metcalf & Eddy, Inc, 2007; Daigger, 2009; Tchobanoglous, et al., 2004).

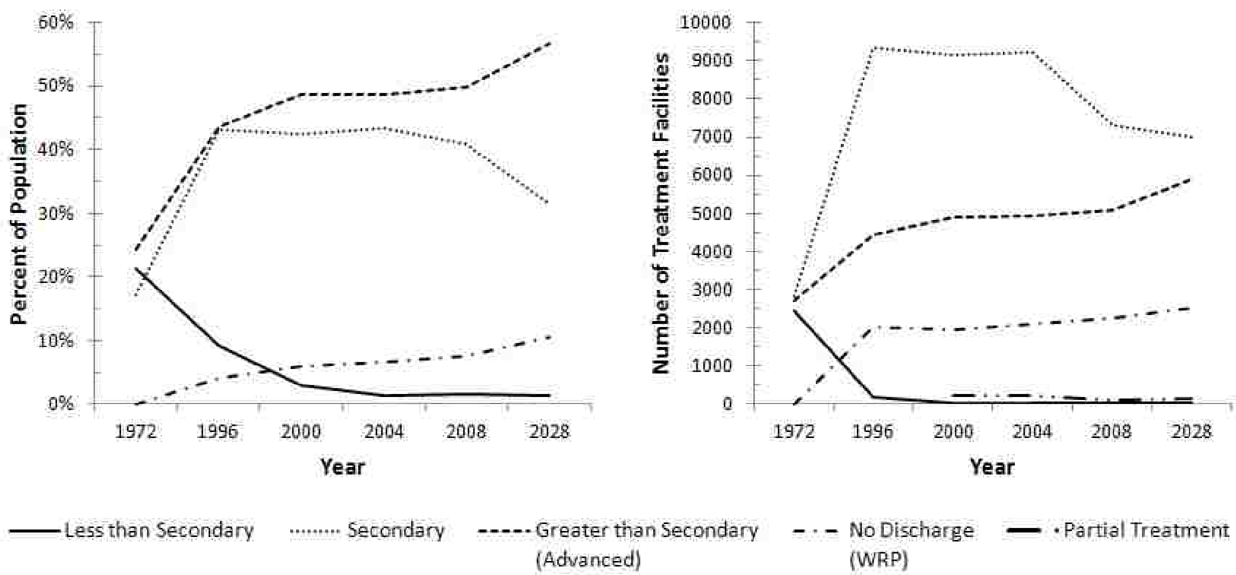
Therefore, reuse plants do not include thickening and dewatering units for solids handling. An extraction type collection system can provide a steady state flow throughout a WRP (Metcalf & Eddy, Inc, 2007; Crites, et al., 1998; Daigger, 2009). This flow is obtained by diverting a specific amount of flow from an adjacent collection system. This is known as sewer mining (Daigger, 2009; Fane, et al., 2005; WEF, 2006). All these factors help keep the land footprint (i.e. real estate area) of WRPs as minimal as possible. As a result of these advantages of WRPs and the use of high performance advanced treatment technologies, many water-short urban communities worldwide have incorporated these facilities in their municipality.

For WRPs to achieve the strict effluent standards and regulations, as well as keeping the real estate area of the facility to a minimal, advanced treatment technologies are needed throughout the plant. These advanced technologies replace traditional treatment processes and are only a fraction of the size using a much smaller real estate area, but achieve the same, or higher, removal rates (Metcalf & Eddy, Inc, 2007; Metcalf & Eddy, Inc, 2003; Davis, 2010; WEF, 2008; WEF, 2006). With the use of DWM and the employment of high-performance treatment technologies, WRPs are helping to further the transition from large centralized WWTPs (Daigger, 2009).

In 2010, prime energy consumption in the world was 153 trillion kWh (522 quadrillion Btu) per year (U.S. EIA, 2011a). Of this consumption, the United States used 28.7 trillion kWh (97.8 quadrillion Btu) (U.S. EIA, 2011a), roughly 18.7% of the world's consumption. Electrical energy consumption in the United States accounted for 4.15 trillion kWh (U.S. EIA, 2011b), 14.5% of their total energy consumption. Two to four percent of this consumption, roughly 83 to 166 billion kWh, is processed through collecting, distributing, and treating wastewater and drinking water (McMahon, et al., 2011; Daigger, 2009; U.S. EPA, 2010; Metcalf & Eddy, Inc, 2003; EPRI, 2002; WEF, 2010b). The combination of both municipal wastewater treatment and water supply systems makeup an average of 35% of the total energy consumed by municipalities (McMahon, et al., 2011; U.S. EPA, 2008; NRDC, 2009), but can be as much as 60% (WEF, 2010b). The USEPA reports that in 1996 the water and wastewater industry used 75 billion kWh of energy (U.S. EPA, 2008; U.S. EPA, 2010) and is estimated to consume between 100 and 123.45 billion kWh of energy in 2010 (U.S. EPA, 2010; EPRI, 2009). This consumption of energy currently emits roughly 116 billion lbs (52 million metric tonnes) of carbon dioxide ($CO_2$) into the atmosphere (McMahon, et al., 2011; NRDC, 2009). Current data show and supports this increase in energy consumption with the number of facilities and the percent of population served by secondary treatment are decreasing while the use of advanced wastewater treatment is increasing (Figure 1). Due to the increase in population, more stringent water quality regulations, and the development of advanced treatment technologies to treat to the desired level of treatment, it has been estimated that during the next 15 years wastewater loads are expected to increase by 20% (U.S. EPA, 2008) and during the next 20 to 30 years energy consumption for wastewater treatment facilities are expected to increase by 30 to 40% in the United States (Metcalf & Eddy, Inc, 2003).

The use of advanced treatment technologies to treat reuse water requires a large increase in energy consumption compared to conventional treatment. In the past, energy consumption has not been a concern in reuse plant design. However, the current efforts to minimize energy footprint challenge the actual benefits of reuse plants. With the increase in WRPs and the use of advanced treatment technologies rising, energy consumption within these facilities must be evaluated. In this research, a typical WRP located in the Southwestern United States was designed and an evaluation of the facility's associated energy consumption was performed based on major energy consuming units for both advanced and conventional treatment processes. The plant produces reuse water that is used for golf course irrigation. In this research, the impacts of advanced treatment processes and varying wastewater flowrates on the energy consumption in a typical satellite water reuse plant were investigated.

**Figure 1 – Population and Corresponding Number of Wastewater Treatment Facilities in the United States (U.S. EPA, 2008; U.S. EPA, 2004b)**




*For 1972 and 1996, partial treatment facilities are included in less than secondary

## 2. Methodology

To estimate the potential energy consumed in the WRP, a typical satellite WRP in the Southwestern United States was designed with focus on the energy consuming units of each process. The process flow diagram of the WRP is shown in Figure 2 and includes, in order of treatment: coarse screen, aerated grit chambers, fine screen, bioreactor system, membranes, and UV disinfection. Since there are no solids processing on site, all screenings, grit, and biosolids are discharged back into the collection sewer trunk. In the design, a five-stage modified Bardenpho CAS system is provided for the removal of the nutrients phosphorous and nitrogen (WEF, 2012; WEF, 2011). The design provides for carbonaceous BOD removal, $NH_3$ oxidation, denitrification through endogenous respiration, and biological phosphorous removal through PAOs. For this reuse plant, stringent nutrient removal is required because during winter, when golf course irrigation needs are less, the effluent could be discharged into an environmentally sensitive lake, where algal blooms avoidance is a goal. The WRP was designed using design recommendations and WWTP design equations from various sources (Metcalf & Eddy, Inc, 2003; WEF, 2010a; Qasim, 1999; Davis, 2010; Lin, 2007; WEF, 2012). The size of each unit process was determined using Microsoft Excel spreadsheet for the various scenarios under consideration. Once designed, the energy consuming unit of every unit process was identified and the expected energy consumption for each unit was computed. Next, advanced treatment processes were replaced with more traditional unit processes to evaluate the changes in energy consumption. The MBR system was redesigned to include a traditional CAS bioreactor with secondary clarification and dual media filters. Then UV disinfection was replaced with traditional chlorination.

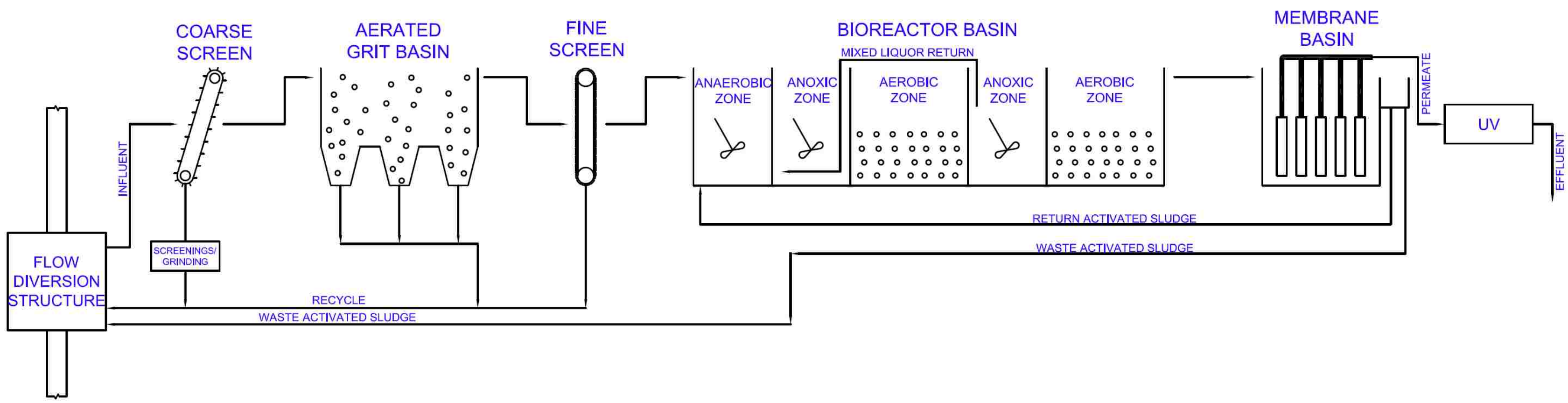



**Figure 2 – Process Flow Diagram of the Water Reuse Plant for Which Energy Consumption is Evaluated**

### *2.1 Influent and Effluent Quality*

The influent characteristics and effluent requirements for the WRP are depicted in Table 1. The requirements are typical water reuse standards found in California and Florida (U.S. EPA, 2004a), with the exception for the need to remove nutrients.

**Table 1 – Plant Influent and Effluent Process Characteristics Used in the Design**

| Parameter | Influent Characteristics | Effluent Requirements |
|---|---|---|
| **BOD (mg/L)** | 250 | 30 |
| **TSS (mg/L)** | 309 | 30 |
| **TKN (mg/L as N)** | 42 | – |
| **$NH_3$ (mg/L as N)** | 34 | 0.5 |
| **TN (mg/L as N)** | – | 10 |
| **TP (mg/L as P)** | 8 | 0.2 |
| **TC (MPN/100 mL)** | – | 2.2 |
| **TC, daily max (MPN/100 mL)** | – | 23 |
| **Minimum Temp (°C)** | 18.3 | 18.3 |

### *2.2 Design Parameters and Considerations*

Typical design criteria used to size each unit process are shown in Table 2. Unit processes included reported in the table include those shown in the process diagram (Figure 2) and additional ones used for energy consumption comparison. Design values in the table are typical of values reported in the design literature. All process were designed taking peak flows into consideration, however, energy consumption computations are for monthly average flow conditions. A maximum day and peak hour factors of 1.09 and 1.49 were used in the design, respectively. Peak flows in the facility are to allow for extra capacity during mid day when irrigation cycles happen more frequently. The designs for each unit process are discussed below. Complete design methodology and details are found in Appendix A.

**Table 2 – WRP Design Parameters**

| | Parameter | Value | Unit | Reference(s) |
|---|---|---|---|---|
| ***Coarse Screens*** | Bar Width | 0.375 (9.53) | in (mm) | 1, 2, 3 |
| | Bar Spacing | 0.75 (19.05) | in (mm) | 1, 2, 3 |
| | Headloss at Peak Flow – Clean | 0.15 (45.72) | ft (mm) | 4 |
| | Headloss at Peak Flow – Clogged | 1.13 (344.42) | ft (mm) | 4 |
| ***Grit Chamber*** | HRT at Peak Flow | 4.5 | min | 1, 2, 3, 4 |
| | Air Supply per Unit Length | 8 (0.74) | cfm/ft ($m^3$/m/min) | 1, 3, 4 |
| ***Fine Screens*** | Perforation Size | 7.87E-2 (2) | in (mm) | 4, 5, 6 |
| | Headloss at Peak Flow – Clean | 2.17 (661.42) | ft (mm) | 1, 4 |
| ***Activated Sludge*** | Solids Retention Time | 10 | day | 7 |
| | Internal Mixed Liquor Recycle (IMLR) | 200 | % | 4, 5, 8 |
| ***Membrane*** | RAS Recycle Ratio | 400 | % | 4, 5, 6 |
| | Net Flux at Peak-day | 13.5 (22.9) | gal/$ft^2$/day (L/$m^2$·hr) | 7 |
| | Air Scour Flowrate at Peak-day | 11.77 (20) | scf/min ($Nm^3$/hr) | 6 |
| ***Secondary Clarifiers*** | Surface Overflow Rate at Average Flow | 698 (1.19) | gpd/$ft^2$ ($m^3$/$m^2$·hr) | 1, 4, 9 |
| | Solids Loading Rate at Average Flow | 21.8 (106.4) | lb/day·$ft^2$ (kg/$m^2$·day) | 1, 4, 9 |
| ***Dual Media Filters*** | Dual Media Filtration Rate | 5 (0.2) | gpm/$ft^2$ ($m^3$/$m^2$·min) | 10 |
| | Dual Media Backwash Rate w/Air Scour | 9.4 (0.38) | gpm/$ft^2$ ($m^3$/$m^2$·min) | 1, 4 |
| | Dual Media Backwash Air Flow Rate | 3.5 (1.07) | $ft^3$/$ft^2$·min ($m^3$/$m^2$·min) | 1, 4 |
| ***UV Disinfection*** | Minimum UV Dosage – Membrane Effluent | 80 | mW·s/$cm^2$ | 1, 4, 11, 12, 13 |
| | Minimum UV Dosage – Filter Effluent | 100 | mW·s/$cm^2$ | 13 |
| ***Chlorination*** | Minimum Chlorine Contact Time | 450 | mg·min/L | 4 |
| | HRT at Peak Flow | 30 | min | 1 |

1 ~ (Metcalf & Eddy, Inc, 2003); 2 ~ (Davis, 2010); 3 ~ (Qasim, 1999); 4 ~ (WEF, 2010a); 5 ~ (WEF, 2006); 6 ~ (WEF, 2012); 7 ~ (Menniti, et al., 2011); 8 ~ (WEF, 2011); 9 ~ (WEF, 2005); 10 ~ (GLUMRB, 2004); 11 ~ (Metcalf & Eddy, Inc, 2007); 12 ~ (U.S. EPA, 2004a); 13 ~ (NWRI, 2012)

### 2.2.1 *Influent Channel and Coarse Screens*

The design of the rectangular open channel leading to the coarse screens was based on the Manning's equation, with a Manning's coefficient of 0.015 (Sturm, 2010). Velocity in the designed channel exceeds 1.3 ft/sec (0.4 m/s) during minimum flow to avoid grit deposition or 3 ft/sec (0.9 m/s) was maintained during peak flows to ensure resuspension of solids (WEF, 2010a). Key parameters used in the design of the coarse screens are shown in Table 2. The headloss through the screens was calculated using both the modified minor loss headloss equation and the Kirshmer's equation (Metcalf & Eddy, Inc, 2003; WEF, 2010a). The higher headloss value governed the design. Energy consumption for the coarse screens is driven by the size of the motor that powers the rake and the rake cleaning frequency. Based on channel and screen dimensions, a motor size for the rake was obtained using a graphical method provided by a screen manufacturer (Vulcan Industries, Inc, 2011).

### 2.2.2 *Aerated Grit Chamber*

Parameters used in the design of the aerated grit chamber can be found in Table 2. The hydraulic retention time (HRT) was determined for the desired peak flowrate with a depth, width-depth ratio, and length-width ratio chosen in the range of design criteria (Metcalf & Eddy, Inc, 2003; WEF, 2010a; Qasim, 1999). Energy consumption for the aerated grit chamber is driven by the air blower capacity used to maintain discrete particle sedimentation and can be estimated by the following equation (U.S. EPA, 1989):

$$BHP = \left[(4.28E-4)q_s T_a / e\right] * \left[(P_d / P_b)^{0.283} - 1\right] \quad \textbf{(1)}$$

where $BHP$ = brake horsepower, hp; $q_s$ = required flow rate, scfm; $T_a$ = blower inlet air temperature, °R; $e$ = blower and motor combined efficiency; $P_d$ = blower discharge pressure, psia (the addition of atmospheric pressure and the system head); and $P_b$ = field atmospheric pressure, psia. System head was estimated as per (U.S. EPA, 1989) using headloss values for diffuser (0.70 psi; 4.826 kPa), piping (0.15 psi; 1.034 kPa), and inlet valve and filter headloss (0.30 psi; 2.068 kPa). Atmospheric pressure at 2,000 feet (609.6 meters) elevation was used and a combined blower and motor efficiency of 80% were assumed (Metcalf & Eddy, Inc, 2003; Davis, 2010).

### *2.2.3 Fine Screens*

Design considerations for the open channel preceding the fine screens are the same as for the open channel before the coarse screens. Parameters used in the design for the fine screen can be found in Table 2. The headloss across the screen was determined using the modified orifice headloss equation (Metcalf & Eddy, Inc, 2003; WEF, 2010a). A blinding factor of up to 50% was applied to determined clogged screen headloss (WEF, 2010a). Typical effective open areas for fine screens and their corresponding solid removal rates are shown in Table 3. Energy consumption for the fine screens was computed using the same procedure as for the coarse screens, except that the raking is continuous.

**Table 3 – Fine Screen Effective Open Areas and Removal Rates**

| Hole Spacing (mm) | Open Area (%) | Percent Solids Removal (%) | Reference(s) |
|---|---|---|---|
| **9** | 55 | – | (Davis, 2010) |
| **6** | 40-51 | 73-81 | (Davis, 2010; Cluin, 2011; Mackie, et al., 2007) |
| **3** | 35-40 | 84-93 | (Davis, 2010; Cluin, 2011; Mackie, et al., 2007) |
| **2** | 30 | – | (Cluin, 2011) |
| **1** | 31 | – | (Davis, 2010) |

### *2.2.4 Activated Sludge*

Both a CAS system and a MBR system were considered in this study. A five-stage modified Bardenpho CAS system is provided for nutrient removal of both phosphorous and nitrogen (WEF, 2012; WEF, 2011). The BOD and solids removal by the coarse and fine screens were based on data provided by manufacturers (Table 4) (Huber Technology, 2008; Mackie, et al., 2007). Design and biological treatment parameters used in the activated sludge design are depicted in Table 2 and Table 5, respectively. Design equations used for the activated sludge process are those provided by Rittmann, et al. (2001).

**Table 4 – Characteristics of Effluent from Fine Screening**

| Parameter | Value | Unit |
|---|---|---|
| **BOD** | 125 | mg/L |
| **$BOD_L$** | 187.5 | mg/L |
| **TSS** | 61.8 | mg/L |
| **Volatile portion of TSS** | 81 | % |
| **VFA** | 43 | mg/L |
| **TKN** | 42 | mg/L as N |
| **TP** | 4.68 | mg/L as P |

**Table 5 – Microbiological Parameters in Activated Sludge Process**

| Parameter (Unit) | BOD Heterotrophic Microorganisms | Nitrification Microorganisms (Nitrosomonas) | Nitrification Microorganisms (Nitrobacter) | Denitrification Microorganisms (Pseudomonas) | Phosphorous Accumulating Organisms |
|---|---|---|---|---|---|
| $K$ (mg $BOD_L$/L) | 10[1] | 1[1] | 1.3[1] | 12.6[2] | 1[1] |
| $Y$ (mg VSS/mg $BOD_L$) | 0.45[1] | 0.33[1] | 0.083[1] | 0.26[1] | 0.3[3] |
| $\hat{q}$ (mg $BOD_L$/mg VSS-day) | 20[1] | 2.3[1] | 9.8[1] | 12[1] | 3.17[1] |
| $\hat{\mu}$ (mg VSS/mg VSS-day) | 9[1] | 0.76[1] | 0.81[1] | 3.12[1] | 0.95[3, 4] |
| $b$ (mg VSS/mg VSS-day) | 0.15[1] | 0.11[1] | 0.11[1] | 0.05[1] | 0.04[3] |
| $f_d$ (–) | 0.8[1] | 0.8[1] | 0.8[1] | 0.8[1] | 0.8[1] |
| $\left[\theta_x^{\min}\right]_{\lim}$ (day) | 0.11299 | 1.54083 | 1.42167 | 0.32573 | 1.09890 |

Parameters: $K$ = concentration giving one-half the maximum rate; $Y$ = true yield for cell synthesis; $\hat{q}$ = maximum specific rate of substrate utilization; $\hat{\mu}$ = maximum specific growth rate; b = endogenous-decay coefficient; $f_d$ = fraction of active biomass that is biodegradable; $[\theta_x^{min}]_{lim}$ = absolute minimum SRT for steady-state biomass

References: 1 ~ (Rittmann, et al., 2001); 2 ~ (U.S. EPA, 1993); 3 ~ (WEF, 2011); 4 ~ (Metcalf & Eddy, Inc, 2003)

In an activated sludge MBR system, return activated sludge (RAS) rates are typically higher compared to CAS process. For a MBR system, RAS rates are typically 200 to 500% of the average influent flow, versus 50 to 100% in CAS systems (WEF, 2012; WEF, 2010a; WEF, 2006). These systems also require a higher MLSS concentration compared to CAS systems. For a MBR system, the MLSS concentration inside the bioreactor tank can be between 4,000 to 10,000 mg/L and inside the membrane tank 8,000 to 18,000 mg/L, versus 1,500 to 3,500 mg/L in CAS systems (WEF, 2012; WEF, 2006; WEF, 2010a). Due to these higher MLSS concentrations (Fabiyi, et al., 2008), a decreased alpha factor, or oxygen transfer efficiency of diffused air, of 0.5 results for MBR facilities with MLSS concentrations around 10,000 mg/L (Germain, et al., 2007). For CAS facilities with nitrification and denitrification, an alpha factor of 0.7 was used (Rosso, et al., 2006). The alpha factor is not only affected by solid concentrations inside the basin but also the type of treatment, due to low molecular weight surfactant uptake in the anoxic zone (Rosso, et al., 2006). Energy consumption for the activated sludge process is driven by mixers used to maintain particles suspension in the anaerobic and anoxic zones of the biological nutrient removal system, and blowers used to provide oxygen and particle suspension in the aerated zones. In addition, energy is required to operate the IMLR pumps and RAS pumps. Mixer energy requirement was determined based on the basin volume and the type of mixer. For horizontal mixers the required energy used was 7 $W/m^3$ (WEF, 2010a). Blower energy was determined using equation 1 and a combined blower and motor efficiency of 80% (Metcalf & Eddy, Inc, 2003; Davis, 2010). Energy requirements for pumps after they have been sized were determined as (Jones, et al., 2008):

$$BHP = \frac{qH}{3960E_p} \quad \textbf{(2)}$$

where $BHP$ = brake horsepower, hp; $q$ = required flow rate, gal/min; H = total dynamic head, ft; and $E_p$ = pump efficiency. Efficiencies for both the IMLR and RAS pumps were chosen in ranges from pump data and curves. A pump efficiency of 80% was used for both pumps (Goulds Pumps, 2012).

*2.2.5 Membranes*

Parameters used in the design of the membrane portion of the MBR system can be found in Table 2. MLSS concentration inside the membrane tank was determined as per (WEF, 2012). The required membrane area needed inside the tank was determined using the net flux concept (WEF, 2012). Typical membrane parameters including membrane area per small subunit, number of small subunits per large subunit, and volume required per large subunits (WEF, 2012). The air scour cycle rates during average and peak-day flowrates were 10 sec on/30 sec off and 10/10, respectively (WEF, 2012). An online factor of 95% percent was also used to allow for relaxation intervals and maintenance cleaning (WEF, 2012). Energy consumption for the membranes is driven by air scour blowers, permeate pumps, backpulse pumps, and WAS pumps. The consumption of energy was calculated for the blower and pumps using equations 1 and 2, respectively. The combined and pump efficiencies used for both the blower and WAS pumps, respectively, were 80% (Metcalf & Eddy, Inc, 2003; Davis, 2010), and the pump efficiencies used for permeate and backpulse pumps were 70% (Goulds Pumps, 2012).

*2.2.6 Secondary Clarifier*

The alternative biological process used to contrast a MBR system was a traditional CAS system. The biological portion of the design is the same as for the MBR system, except for the MLSS concentration, RAS ratio, and alpha factor as discussed above. This would require a doubling in aeration volume compared to the MBR system's biological process. The membranes are replaced with secondary clarification and filtration to provide solid separation. Parameters used in the design of the secondary clarifier can be found in Table 2. The clarifier was sized using recommended overflow rates and solids loading rates as per (Metcalf & Eddy, Inc, 2003; WEF, 2010a; WEF, 2005). Design was performed for both peak and average flow, with the highest value governing the design. Weir loading was checked for both average and peak flows to ensure the loadings were under recommended limits (WEF, 2005; WEF, 2010a). Energy consumption for the secondary clarifier is driven by the size of the motor that provides the torque for the rake arm and the WAS pump. The required power to move the rake arm was calculated using (WEF, 2005):

$$P = T\omega \qquad \textbf{(3)}$$

where P = power required by the motor, W; T = required torque, J, $T = Wr^2$ where $W$ = rake arm loading, N/m and $r$ = radius of rake arm, m; and $\omega$ = angular velocity, rad/s. A rake arm loading value of 95 N/m was used and fell within the recommended range for secondary sludge (WEF, 2005). The energy requirement for the WAS pump was determined using equation 2. A pump efficiency of 80% was used (Goulds Pumps, 2012).

### *2.2.7 Dual Media Filters*

Parameters used in the design of the dual media filters can be found in Table 2. The number and size of the filters were determined using (WEF, 2010a) and the filtration rate (GLUMRB, 2004). Filter sizes were rounded to the nearest increment of 25 square feet to allow for ease of construction. The filters were designed with one filter out of service for backwashing cycles. The cleanwater headlosses were determined to be 0.81 and 1.45 feet for average and peak filtration rates, respectively, using the Rose equation (Metcalf & Eddy, Inc, 2003). Backwash cycles were design to be 36 hours, determined using solids holding capacity for clogged headloss determination (Metcalf & Eddy, Inc, 2003; WEF, 2010a). Energy consumption for the dual media filters is driven by the backwash blower and backwash pump, equations 1 and 2. A combined blower and motor efficiency of 80% was used for the backwash blower (Metcalf & Eddy, Inc, 2003) and a pump efficiency of 78% was used for the backwash pump (Goulds Pumps, 2012).

*2.2.8 UV Disinfection*

The parameters used in the design of the UV disinfection process can be found in Table 2. Two UV disinfection system designs were considered, low and medium-pressure. When designing the UV disinfection system with low-pressure UV lamps, a graphical point-source-summation method was used to determine the water quality factor and the effluent coliform number, using suspended solids concentrations and UV dosage, respectively (WEF, 2010a; U.S. EPA, 1986). Low-pressure high intensity lamps were assumed to have a maximum input power of 260 W with an efficiency of 33% (Metcalf & Eddy, Inc, 2003; Trojan Technologies, 2008). The variable output (dimming) capabilities of this lamp are from 60 to 100% (Trojan Technologies, 2008). For medium-pressure UV lamps, an equation based point-source-summation was performed for estimating the UV intensity (U.S. EPA, 1986). The required UV dose was determined as per (WEF, 2010a). To determine the effluent coliform number after exposure, a variation of the Chick-Watson first-order model was used (Metcalf & Eddy, Inc, 2003; WEF, 2010a; U.S. EPA, 1986). Medium-pressure high intensity were assumed having a maximum input power of 3,200 W with an efficiency of 12% (Metcalf & Eddy, Inc, 2003; Trojan Technologies, 2007). The variable output capabilities of this lamp are from 30 to 100% (Trojan Technologies, 2007). The headloss through the UV channel was determined using the energy equation from (Metcalf & Eddy, Inc, 2003; Qasim, 1999).

### *2.2.9 Chlorination*

The alternative disinfection process used to contrast UV disinfection was chlorination. Chlorination would follow membranes in the MBR facility and the dual media filters in the CAS facility. Parameters used in the design of the chlorination contact basin are depicted in Table 2. The chlorine dosage was determined using a modification of the Collins-Selleck model found in (Metcalf & Eddy, Inc, 2003). Membrane effluent total coliform bacterium has a typical range of 10 to 1000 MPN/100mL (Metcalf & Eddy, Inc, 2003; DeCarolis Jr, et al., 2007) and filter effluent total coliform bacterium has a typical range of $10^4$ to $10^6$ MPN/100mL (Metcalf & Eddy, Inc, 2003). The design assumed a chlorine residual of 3 mg/L leaving the facility. Dechlorination was not considered in this design because water is to be used for golf course irrigation. With a design scheme layout of the chlorine contact basin determined and sized, proper dispersion was evaluated using the axial dispersion equations found in Metcalf & Eddy, Inc (2003). Energy consumption for chlorination is driven by the size of the diaphragm pump used to inject chlorine before the contact basin. This energy requirement can be calculated using equation 2. A pump efficiency of 70% was used (Goulds Pumps, 2012).

## 3. Results and Analysis

Estimated energy consumption for major energy driving units of each process and for varying WRP flowrates are shown in Table 6.

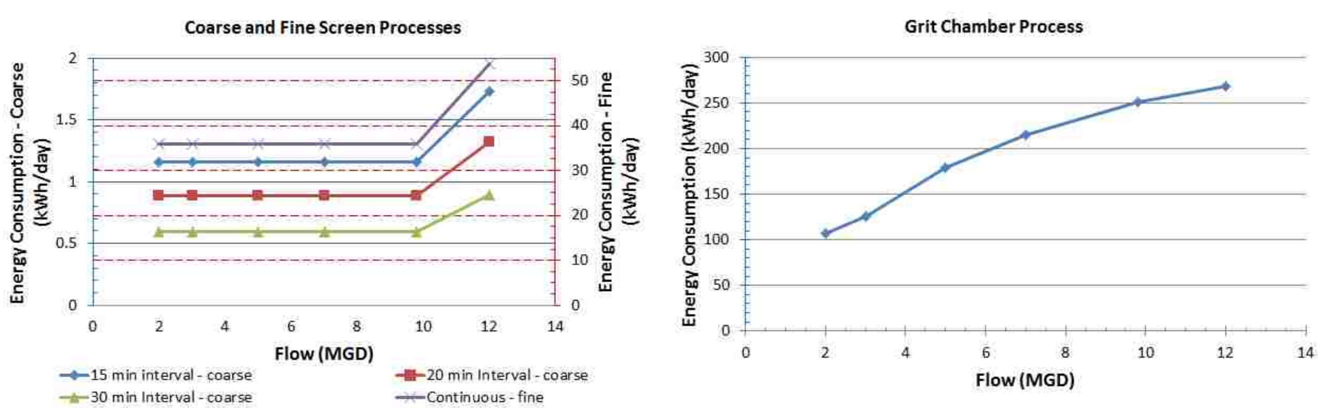
**Table 6 – Estimated Energy Consumption of Energy Driving Units in Water Reuse Plants of Varying Sizes**

| | Energy Driving Units | 1 MGD Plant (kWh/day) | 2 MGD Plant (kWh/day) | 4 MGD Plant (kWh/day) | 6 MGD Plant (kWh/day) | 8.8 MGD Plant (kWh/day) | 11 MGD Plant (kWh/day) |
|---|---|---|---|---|---|---|---|
| ***Coarse Screens*** | Rake Motor | 1.16 | 1.16 | 1.16 | 1.16 | 1.16 | 1.73 |
| ***Grit Chamber*** | Air Blowers | 107.42 | 125.36 | 179.04 | 214.85 | 250.66 | 268.56 |
| ***Fine Screens*** | Screen Motor | 35.81 | 35.81 | 35.81 | 35.81 | 35.81 | 53.71 |
| ***Bioreactor*** | Mixers | 69.78 | 143.93 | 287.85 | 431.78 | 680.38 | 850.48 |
| | Air Blowers | 1038.43 | 2076.86 | 4153.73 | 6230.59 | 9166.85 | 11458.56 |
| | IMLR Pumps | 196.94 | 393.89 | 787.78 | 1181.66 | 1790.40 | 2238.00 |
| | RAS Pumps | 286.46 | 572.93 | 1145.86 | 1718.78 | 2506.56 | 3133.20 |
| | ***Total*** | 1591.61 | 3187.61 | 6375.22 | 9562.81 | 14144.19 | 17680.24 |
| ***Membranes*** | Air Scour Blowers | 646.33 | 1292.67 | 2585.34 | 3878.01 | 5170.68 | 6463.34 |
| | Permeate Pumps | 238.12 | 476.25 | 952.49 | 1428.74 | 2041.06 | 2551.32 |
| | Backpulse Pumps | 15.22 | 30.44 | 60.87 | 91.31 | 136.07 | 170.09 |
| | WAS Pumps | 4.48 | 8.95 | 17.90 | 26.86 | 35.81 | 44.76 |
| | ***Total*** | 904.15 | 1808.31 | 3616.60 | 5424.92 | 7383.62 | 9229.51 |
| ***Conventional Activated Sludge*** | Mixers | 69.78 | 143.93 | 287.85 | 431.78 | 680.38 | 850.48 |
| | Air Blowers | 751.97 | 1486.03 | 2972.06 | 4458.10 | 6517.06 | 8146.32 |
| | IMLR Pumps | 196.94 | 393.89 | 787.78 | 1181.66 | 1790.40 | 2238.00 |
| | RAS Pumps | 161.14 | 304.37 | 608.74 | 913.10 | 1360.70 | 1700.88 |
| | ***Total*** | 1179.83 | 2328.22 | 4656.43 | 6984.65 | 10348.54 | 12935.68 |
| ***Secondary Clarifier*** | Rake Arm Torque | 4.48 | 4.48 | 8.95 | 13.43 | 17.90 | 22.38 |
| | WAS Pumps | 4.48 | 8.95 | 17.90 | 26.86 | 35.81 | 44.76 |
| | ***Total*** | 8.96 | 13.43 | 26.85 | 40.29 | 53.71 | 67.14 |
| ***Dual Media Filters*** | Backwash Blower | 1.46 | 2.92 | 4.38 | 5.84 | 8.22 | 10.28 |
| | Backwash Pump | 3.58 | 7.10 | 10.64 | 14.19 | 20.16 | 25.20 |
| | ***Total*** | 5.04 | 10.02 | 15.02 | 20.03 | 28.38 | 35.48 |
| ***UV Disinfection – Membrane Effluent*** | Low-Pressure, High Intensity | 98 | 210 | 404 | 584 | 839 | 1078 |
| | Medium-Pressure, High Intensity | 590 | 1181 | 2362 | 3542 | 5184 | 6480 |
| ***UV Disinfection – Filter Effluent*** | Low-Pressure, High Intensity | 138 | 276 | 539 | 832 | 1229 | 1475 |
| | Medium-Pressure, High Intensity | 960 | 1920 | 3816 | 5760 | 8496 | 10685 |
| ***Chlorination – Membrane Effluent*** | Pump | 4.48 | 4.48 | 4.48 | 4.48 | 4.48 | 4.48 |
| ***Chlorination – Filter Effluent*** | Pump | 4.48 | 4.48 | 4.48 | 4.48 | 4.48 | 4.48 |

*Preliminary and Primary Treatment Units*

Preliminary and primary treatment units include coarse screens, aerated grit chamber, and fine screens. The energy consumption by the fine screens in the reuse plants are about thirty-one times that consumed by the coarse screens, due to the fine screens being continuously run. However, the energy consumed by both screens is small relative to that consumed by other unit processes. On average, both screens together require 0.72% of the plant's total energy consumption. For flowrates varying from 1 to 8.8 MGD (Figure 3a), energy consumption for both processes are constant until a flowrate above 8.8 MGD is reached. This is the case because in order to remove large debris from screens a minimum motor size must be used, independent of the flowrate (Vulcan Industries, Inc, 2011). The Water Environment Federation (WEF, 2010b) reports energy consumption for coarse screens are equal to 2 kWh/d for flows between 1 to 10 MGD and increases at larger flows (WEF, 2010b). In this research, values of 1.16 to 1.73 kWh/day were found and are similar to the values and pattern reported by WEF, 2010b. Malcolm Pirnie (1995) reports that a 0.39 MGD facility uses 17.53 kWh/day for fine screens and 96.89 kWh/day for a 2.85 MGD facility. A value of 35.81 kWh/day was found in this research at 1 MGD, which is roughly two times the value found at the 0.39 MGD facility.

The energy consumption in the aerated grit chamber is a function of flowrate treated and it increases initially and tapers down resulting in a decreasing slope as flow increases (Figure 3b). This behavior occurs due to the chosen design depth used in the chamber. Design depth increases rapidly at lower flow ranges, 1 – 4 MGD, and begins to steady at flow ranges above 6 MGD; indicating depth is directly related to the energy consumption. The increase in energy is directly proportional to the required air flowrate needed by the air blower and it reflects the amount of air supply needed per unit length of the grit chamber. It has been reported that energy consumption in aerated grit chambers is about 77.5 (WEF, 2010b) and 56.2 (Smith & Loveless, 2007) kWh/day at 4 MGD. In this research, the estimated consumption is 174.04 kWh/day, which is 2.2 to 3.1 times greater than the reported values. These differences can be due to variations in the amount of air supply per unit length used in the design. In this design, a recommended high air flowrate of 8 cfm/ft (0.74 $m^3$/m/min) was used. If the system were designed for the recommended lower range of air flowrate (3 cfm/ft), the energy consumption would be 71.62 kWh/day, falling within the values reported above. There is no theoretical way to determine the exact blower output required thus, variations will be observed for different designs (WEF, 2010b).




*Flows through the screening and aerated grit chamber processes are 1 MGD higher than indicated in the text, as this flow is assumed to be wasted due to screenings and grit removal

**Figure 3 – Energy Consumption for Preliminary and Primary Unit Processes: (a) Coarse and Fine Screen Energy Consumption Versus Flow; and (b) Aerated Grit Chamber Energy Consumption Versus Flow**

*Secondary and Tertiary Treatment Units*

Secondary and tertiary treatment units include: bioreactor and membrane filters for a MBR facility; and CAS bioreactor process, secondary clarifier, and dual media filters for a CAS facility. The energy consumption for secondary and tertiary treatment unit processes are shown in Figure 4a to 3d. The air requirements in the basins were estimated as 1,038.43 kWh/day at 1 MGD for a MBR facility and 751.97 kWh/day for a CAS facility. At this same flowrate WEF (2010b) reports a value of 878 kWh/day, which is about 15.4% lower than the value estimated by this research for the MBR facility and 14.4% higher for the CAS facility. It is known that energy consumption in biological treatment units is affected by wastewater strength (i.e. BOD and ammonia loadings). However, in this research, the impacts of wastewater loading on energy consumption in the bioreactors were not evaluated. Therefore, comparisons with reported literature are based on flowrates only. For flowrates between 1 – 11 MGD, the energy consumption of the air blowers on average was 65.1% of the total biological process energy consumption for MBR facilities and 63.5% for CAS facilities. IMLR and RAS pumps required 12.5 and 17.9% of the total biological energy consumption for the MBR facilities and 17.0 and 13.2% for CAS facilities. Aerobic and anoxic mixers were on average 4.6% for MBR and 6.7% for CAS. In comparing the biological bioreactor process and CAS bioreactor process, the difference in energy consumptions relates mainly to the RAS pumps and air blowers. It was estimated that the RAS pumps required 2,506.56 and 1,360.70 kWh/day of energy for MBR and CAS facilities at 8.8 MGD, respectively (Table 6). The higher energy consumption for MBR facilities is due to the high recycle rates needed in the

MBR process. In CAS facilities, the high energy consumption is dependent of the TDH difference related to the position of the RAS pumps and the clarifiers; however, the impact of the recycle in the MBR process is much greater. For flowrates between 1 – 11 MGD, the MBR facilities were found on average to require 1.85 times more consumption of energy in RAS pumping. The air blowers at 8.8 MGD required 9,166.85 kWh/day for MBR facilities and 6,517.06 kWh/day for CAS facilities. This increase for MBR facilities was on average 1.4 times the amount of energy needed at CAS facilities. This is a result of a decreased alpha factor (oxygen transfer efficiency of diffused air) of 0.5 (Germain, et al., 2007) in MBR facilities, as compared to 0.7 (Rosso, et al., 2006) in CAS facilities. The different alpha factor is a result of the higher solids concentrations maintained in MBRs (Fabiyi, et al., 2008).

Comparing the membrane with the secondary clarifier and dual media filter for secondary filtration, it can be observed that the membrane process requires a very large amount of energy (7,383.61 kWh/day), while the secondary clarifier and dual media filter processes requires less (82.09 kWh/day) at 8.8 MGD, which comprises about 1% of that consumed by the membrane process for flowrates between 1 – 11 MGD on average. The reason for this is due to the required pumping and blowers needed to run and maintain the membrane system (Figure 4b and 3d). Air scour blowers and permeate and backpulse pumps require 71.0 and 28.5%, respectively, of the total membrane energy consumption. WAS pumps only require a consumption of 0.5%. For secondary clarifier and dual media filter energy consumption, the largest contributor was the WAS pumps requiring an average of 40.8% of the total consumption across all flows. The secondary clarifier rake arm and the dual media filter backwash pumps both require about 25% of the total energy consumption.

The overall energy consumption for each system train, MBR and CAS is depicted in Figure 4e. It is observed that the MBR process, on average for the flow ranges investigated, is 2.10 times more energy intensive than the traditional CAS process. Reports on MBR energy consumption say MBR energy may be twice that of CAS (WEF, 2010b; U.S. EPA, 2010) to as much as three times (Wallis-Lage, et al., 2011). As observed in Figure 4e, energy consumption is directly proportional to the influent flowrate for both MBR and CAS with secondary filtration processes. For instance at 2 MGD, energy consumption is 4,996 kWh/day while at 6 MGD the consumption of energy is 14,988 kWh/day, which is three times more energy intensive. The largest energy consuming unit in the MBR process is air scouring and it accounts for 23.7% of the total energy demand of the entire plant across all flows. This is contrasted to 35 to 40% found in (WEF, 2010b; U.S. EPA, 2010; DeCarolis, et al., 2008).

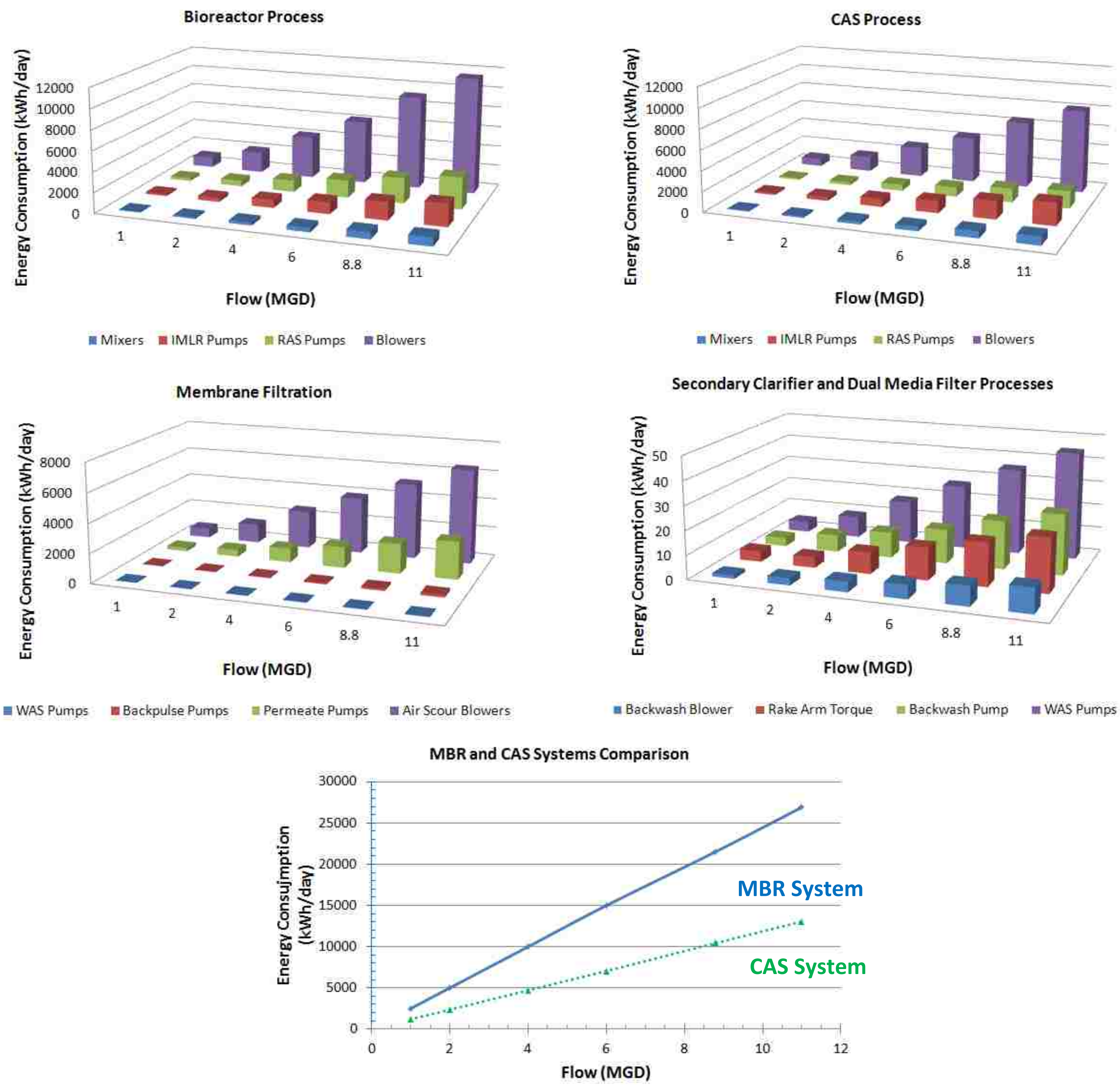



**Figure 4 – Energy Consumption for Secondary and Tertiary Unit Processes: (a) Bioreactor Process Energy Consumption Versus Flow; (b) Membrane Energy Consumption Versus Flow; (c) CAS Bioreactor Process Energy Consumption Versus Flow; (d) Secondary Clarifier and Dual Media Filter Energy Consumption Versus Flow; and (e) MBR and CAS Energy Consumption Comparison Versus Flow**

*Disinfection Units*

Disinfection methods considered include UV disinfection and chlorination. The energy consumption in terms of flowrate for the UV disinfection process for both low and medium pressure lamps is shown in Figure 5. For low and medium-pressure high intensity lamps, energy consumption increases with flowrate. This increase is directly proportional to the flow. Slight variations in energy consumption of both low and medium-pressure lamps is due to the number of lamps that can be in a module and the number of modules that can be in a bank per UV channel (Trojan Technologies, 2007; Trojan Technologies, 2008). Therefore, the exact dosage varied slightly at different flowrates. Studies have found that UV disinfection can take up approximately 10 to 25% of a facility's total energy consumption (U.S. EPA, 2010). In this research, it was found that UV disinfection for all flows had on average a 3.7 and 9.9% total energy consumption for MBR and CAS treatment facilities with low pressure lamps, respectively; and 18.8 and 43.6% with medium pressure lamps. It is observed that filter effluent requires more energy for disinfection compared to membrane effluent due to the higher MPN and TSS levels in the filter effluent, as well as the higher dosage requirement. For instance at 6 MGD, membrane effluent requires 584 and 3,542 kWh/day for low-pressure and medium-pressure lamps, respectively, while the filter effluent requires 832 and 5,760 kWh/day. On average across all flows, filter effluent requires a 38.6% increase in energy consumption for low-pressure lamps and 63.0% increase for medium-pressure lamps.

In the research, results indicate that medium-pressure high intensity lamps required more energy to disinfect compared to low-pressure high intensity lamps. On average for membrane effluent, medium-pressure lamps required a 5.96 times increase in energy consumption compared to low-pressure. For filter effluent, a 7.01 times increase is also observed. These results are consistent with reports on low-pressure lamps requiring less energy to deliver the same UV dose compared to medium-pressure lamps (WEF, 2010b). As energy consumption is directly proportional to the flowrate being treated, the energy gap between the low and medium pressure lamps stays constant as flows change. URS (2004) reports that at a 18 MGD facility, low-pressure high intensity lamps require 1,080 kWh/day and medium-pressure high intensity lamps require 4,560 kWh/day; resulting in an energy gap of 4.22 times between low and medium-pressure lamps. For this reason, in this study total facility energy calculations incorporate low-pressure lamps. It is well known that UV disinfection is an energy intensive process, especially when compared to chlorination (WEF, 2010b; Metcalf & Eddy, Inc, 2003). Chlorination energy consumption stayed constant for both MBR and CAS facility flows. This occurs because the pump motor size used stayed the same (0.25 hp) to allow sufficient power to overcome greater pressure heads at higher flows. This additional power allows for sufficient mixing energy. Chlorination on average was only 1% of the total energy consumed when compared to UV disinfection.

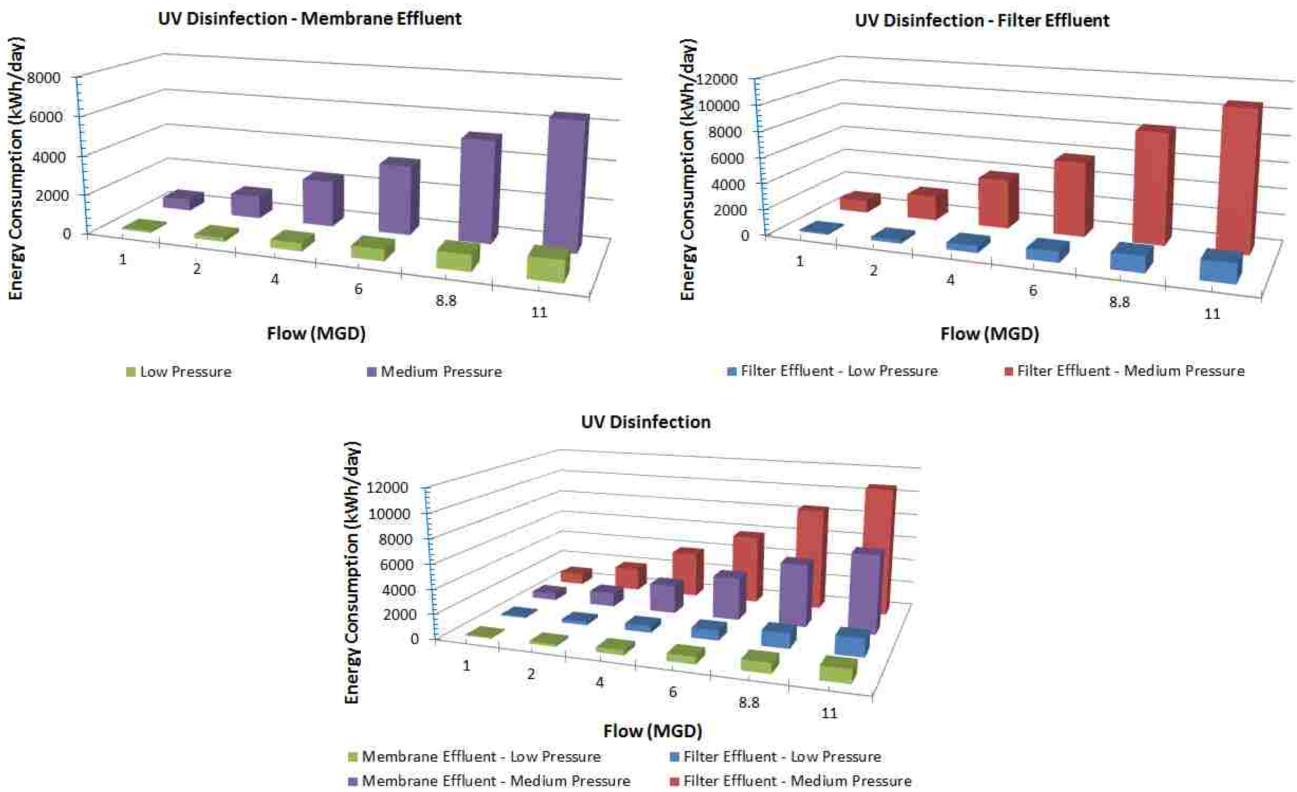



**Figure 5 – Energy Consumption for the Disinfection Unit Process: (a) UV Disinfection of Membrane Effluent Energy Consumption Versus Flow; (b) UV Disinfection of Dual Media Filter Effluent Energy Consumption Versus Flow; and (c) UV Disinfection Comparison of Membrane and Dual Media Filter Effluent**

*Unit Flow Energy Consumption*

Table 7 summarizes the energy consumption of each unit process per unit flow in terms of kWh/MG. These values were derived by dividing the energy consumption per day (kWh/day) by the unit flow (MGD), resulting in energy consumption per million gallon (kWh/MG). The results show that as WRPs increase in the treatment capacity, energy consumption per million gallons treated decreases, as also seen in WEF (2010b). This decreasing in energy consumption can be directly related to cost savings. Assuming a commercial electrical energy rate of $0.08 USD/kWh for low rates, $0.10 USD/kWh for average rates, and $0.12 USD/kWh for high rates at a CAS WRP, when flow is five times as large compared to a 1 MGD facility, the savings in energy costs is $7.44/MG treated at low rates, $9.30/MG treated at average rates, and $11.16/MG treated at high rates; and $13.12/MG, $16.40/MG, $19.68/MG treated at ten times the flow for low, average, and high energy rates. Table 7 can be used in targeting unit processes that are in need of minimizing energy consumption. In addition, the table can be used as a basis for decision making regarding sustainability of using advanced treatment technologies in reuse plants.

The resulting values for the CAS and MBR facilities along with published values for energy consumption in WWTPs are shown in Table 8. For the 1 MGD CAS facility, the energy consumed was found to be 1,476 kWh/MG in this research. This value is 50.0% smaller than values for the same flowrate reported by EPRI (2002), 2,951 kWh/MG, and it is 12.8% greater than values reported by WEF (2010b), 1,308 kWh/MG. On average, energy consumption for the designed MBR facilities were determined to be 2,643 kWh/MG and is comparable to reported values for typical MBR facilities with an energy consumption of 3,000 kWh/MG (Livingston, 2010). This research found that the MBR WRP is on average 1.91 times more energy intensive than the CAS WRP.

**Table 7 – Energy Consumption of Each Unit Process per Unit Flow**

| | Unit Process | Energy Consumption per Unit Flow (kWh/MG) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 1 MGD | 2 MGD | 4 MGD | 6 MGD | 8.8 MGD | 11 MGD |
| MBR Facility | Coarse Screens | 1.156 | 0.578 | 0.289 | 0.193 | 0.131 | 0.158 |
| | Grit Chamber | 107.42 | 62.68 | 44.76 | 35.81 | 28.48 | 24.41 |
| | Fine Screens | 35.81 | 17.90 | 8.95 | 5.97 | 4.07 | 4.88 |
| | Bioreactor | 1591.6 | 1593.8 | 1593.8 | 1593.8 | 1607.3 | 1607.3 |
| | Membranes | 904.2 | 904.2 | 904.2 | 904.2 | 839.0 | 839.0 |
| | UV Disinfection | 98.0 | 105.0 | 101.0 | 97.3 | 95.3 | 98.0 |
| | **Total w/UV** | 2738 | 2684 | 2653 | 2637 | 2574 | 2574 |
| | Chlorination | 4.48 | 2.24 | 1.12 | 0.75 | 0.51 | 0.41 |
| | **Total w/Chlorination** | 2645 | 2581 | 2553 | 2541 | 2479 | 2476 |
| CAS Facility | Coarse Screens | 1.156 | 0.578 | 0.289 | 0.193 | 0.131 | 0.158 |
| | Grit Chamber | 107.42 | 62.68 | 44.76 | 35.81 | 28.48 | 24.41 |
| | Fine Screens | 35.81 | 17.90 | 8.95 | 5.97 | 4.07 | 4.88 |
| | CAS | 1179.8 | 1164.1 | 1164.1 | 1164.1 | 1176.0 | 1176.0 |
| | Secondary Clarifier | 8.95 | 6.71 | 6.71 | 6.71 | 6.10 | 6.10 |
| | Dual Media Filters | 5.04 | 5.01 | 3.75 | 3.34 | 3.23 | 3.23 |
| | UV Disinfection | 138.0 | 138.0 | 134.8 | 138.7 | 139.7 | 134.1 |
| | **Total w/UV** | 1476 | 1395 | 1363 | 1355 | 1358 | 1349 |
| | Chlorination | 4.48 | 2.24 | 1.12 | 0.75 | 0.51 | 0.41 |
| | **Total w/Chlorination** | 1342 | 1259 | 1229 | 1217 | 1219 | 1215 |

**Table 8 – Comparison of Energy Consumption per Unit Flow**

| Water Reuse Plant Size | Energy Consumption per Unit Flow (kWh/MG) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | CAS Facilities | | | | | MBR Facilities | | | |
| | This Study | (WEF, 2010b)[1] | (EPRI, 2002) | (Mizuta, et al., 2010) | (Yang, et al., 2010) | This Study | (Verrecht, et al., 2010) | (Yang, et al., 2010) | (Livingston, 2010) |
| 1 MGD | 1476 | 1308 | 2951 | – | – | 2738 | – | – | – |
| 2 MGD | 1395 | 1271.55[2] | 2694.75[2] | – | – | 2684 | – | – | – |
| 4 MGD | 1363 | 1198.65[2] | 2182.25[2] | – | – | 2653 | – | – | – |
| 5 MGD | – | 1162.20 | 1926 | – | – | – | – | – | – |
| 6 MGD | 1355 | 1158.80[2] | 1899[2] | – | – | 2637 | – | – | – |
| 8.8 MGD | 1358 | 1149.28[2] | 1823.40[2] | – | – | 2574 | – | – | – |
| 10 MGD | – | 1145.20 | 1791 | – | – | – | – | – | – |
| 11 MGD | 1349 | 1142.94[2] | 1779.50[2] | – | – | 2574 | – | – | – |
| 20 MGD | – | 1122.60 | 1676 | – | – | – | – | – | – |
| Average | 1382.7 | 1204.87 | 2221.65 | 1135.62 – 7154.43 | 1010.71 | 2643.3 | 2271.25 – 7570.82 | 1249.19 | 3000 |

1: Total energy consumption was determined based off the addition of similar unit processes
2: Values were interpolated from corresponding reference literature

The efficiencies considered in the energy computations are a combined motor and equipment efficiency, also known in the water industry as 'wire-to-water' efficiency. This efficiency is affected by several factors including type and age of motors, age of equipment (e.g. belts, pulleys, and bearings), and operating conditions (e.g. partial load operation, valve and pipe maintenance, and equipment maintenance) (Kaya, et al., 2008). To evaluate the impact of efficiency on energy computations, a sensitivity analysis was performed to evaluate the impacts of efficiency variations on energy consumption. Table 9 and Table 10 provide the results of this analysis at 8.8 MGD for each energy consuming unit and their respective totals for low-end and high-end efficiencies, respectively. The pump efficiencies were increased by 3 and 5% for the high efficiency range as it has been reported that a 3 to 5% increase has been seen in efficiencies when converted from average to high efficiency motors (Liu, et al., 2005). A low efficiency range for pumps had a decrease of 3 and 5% as it was the mean of a range up to 10-12.5% decrease due to unmaintained pumps (Kaya, et al., 2008). For blowers, efficiencies were increased by 5 and 10% for the high efficiency range and decreased by 5 and 10% for the low efficiency range. These increments were chosen as they covered the typical range of blower efficiencies of 70 to 90% (Metcalf & Eddy, Inc, 2003; Davis, 2010). The sensitivity analysis found that when efficiencies are the lowest compared to average, a 10.9 and 11.3% increase in energy consumption occurs for MBR WRPs with UV radiation and MBR WRPs with chlorination, respectively. A 9.3 and 10.4% increase in energy consumption was found for CAS WRPs with UV radiation and chlorination, respectively. When the highest efficiencies are compared to average efficiencies, a 9.1 and 9.4% decrease in energy consumption occurs for MBR WRPs with UV radiation and chlorination, respectively. An 8.1 and 9.0% decrease is observed for CAS WRPs with UV radiation and chlorination, respectively. Overall, this analysis has shown that even with a slight increase or decrease in efficiencies, the total energy consumption of the entire plant can be greatly affected, by as much as an 11.3% increase or 9.4% decrease.

**Table 9 – Sensitivity Table of Low-End Combined Motor and Wire Efficiencies for Energy Driving Units at an 8.8 MGD Water Reuse Plant**

| | Energy Driving Units | Low Efficiency | | | | Average Efficiency | |
|---|---|---|---|---|---|---|---|
| | | Energy Consumption (kWh/day) | Efficiency; Pump -5, Blower -10 (%) | Energy Consumption (kWh/day) | Efficiency; Pump -3, Blower -5 (%) | Energy Consumption (kWh/day) | Efficiency (%) |
| *Coarse Screens* | Rake Motor | 1.16 | - | 1.16 | - | 1.16 | - |
| *Grit Chamber* | Air Blowers | 286.46 | 70 | 268.56 | 75 | 250.66 | 80 |
| *Fine Screens* | Screen Motor | 35.81 | - | 35.81 | - | 35.81 | - |
| *Bioreactor* | Mixers | 680.38 | - | 680.38 | - | 680.38 | - |
| *Bioreactor* | Air Blowers | 10455.94 | 70 | 9739.78 | 75 | 9166.85 | 80 |
| *Bioreactor* | IMLR Pumps | 1862.02 | 75 | 1862.02 | 77 | 1790.40 | 80 |
| *Bioreactor* | RAS Pumps | 2721.41 | 75 | 2649.79 | 77 | 2506.56 | 80 |
| *Bioreactor* | Total | 15719.75 | - | 14931.97 | - | 14144.19 | - |
| *Membranes* | Air Scour Blowers | 5885.05 | 70 | 5510.85 | 75 | 5170.68 | 80 |
| *Membranes* | Permeate Pumps | 2177.13 | 65 | 2041.06 | 67 | 2041.06 | 70 |
| *Membranes* | Backpulse Pumps | 143.23 | 65 | 136.07 | 67 | 136.07 | 70 |
| *Membranes* | WAS Pumps | 35.81 | 75 | 35.81 | 77 | 35.81 | 80 |
| *Membranes* | Total | 8241.22 | - | 7723.79 | - | 7383.62 | - |
| *Conventional Activated Sludge* | Mixers | 680.38 | - | 680.38 | - | 680.38 | - |
| *Conventional Activated Sludge* | Air Blowers | 7448.06 | 70 | 6946.75 | 75 | 6517.06 | 80 |
| *Conventional Activated Sludge* | IMLR Pumps | 1862.02 | 75 | 1862.02 | 77 | 1790.40 | 80 |
| *Conventional Activated Sludge* | RAS Pumps | 1432.32 | 75 | 1360.70 | 77 | 1360.70 | 80 |
| *Conventional Activated Sludge* | Total | 11422.78 | - | 10849.85 | - | 10348.54 | - |
| *Secondary Clarifier* | Rake Arm Torque | 17.90 | - | 17.90 | - | 17.90 | - |
| *Secondary Clarifier* | WAS Pumps | 35.81 | 75 | 35.81 | 77 | 35.81 | 80 |
| *Secondary Clarifier* | Total | 53.71 | - | 53.71 | - | 53.71 | - |
| *Dual Media Filters* | Backwash Blower | 9.42 | 70 | 8.75 | 75 | 8.22 | 80 |
| *Dual Media Filters* | Backwash Pump | 21.62 | 73 | 20.95 | 75 | 20.16 | 78 |
| *Dual Media Filters* | Total | 31.04 | - | 29.70 | - | 28.38 | - |
| *UV Disinfection – Membrane Effluent* | Low-Pressure | 839 | - | 839 | - | 839 | - |
| *UV Disinfection – Membrane Effluent* | Medium-Pressure | 5184 | - | 5184 | - | 5184 | - |
| *UV Disinfection – Filter Effluent* | Low-Pressure | 1229 | - | 1229 | - | 1229 | - |
| *UV Disinfection – Filter Effluent* | Medium-Pressure | 8496 | - | 8496 | - | 8496 | - |
| *Chlorination – Membrane Effluent* | Pump | 4.48 | 65 | 4.48 | 67 | 4.48 | 70 |
| *Chlorination – Filter Effluent* | Pump | 4.48 | 65 | 4.48 | 67 | 4.48 | 70 |
| *MBR WRP* | With Low-Pressure UV Radiation | 25123.40 | - | 23800.29 | - | 22654.44 | - |
| *MBR WRP* | With Chlorination | 24288.88 | - | 22965.77 | - | 21819.92 | - |
| *CAS WRP* | With Low-Pressure UV Radiation | 13059.96 | - | 12467.79 | - | 11947.26 | - |
| *CAS WRP* | With Chlorination | 11835.44 | - | 11243.27 | - | 10722.74 | - |

**Table 10 – Sensitivity Table of High-End Combined Motor and Wire Efficiencies for Energy Driving Units at an 8.8 MGD Water Reuse Plant**

| | Energy Driving Units | Average Efficiency | | High Efficiency | | | |
|---|---|---|---|---|---|---|---|
| | | Energy Consumption (kWh/day) | Efficiency (%) | Energy Consumption (kWh/day) | Efficiency; Pump +3, Blower +5 (%) | Energy Consumption (kWh/day) | Efficiency; Pump +5, Blower +10 (%) |
| *Coarse Screens* | Rake Motor | 1.16 | - | 1.16 | - | 1.16 | - |
| *Grit Chamber* | Air Blowers | 250.66 | 80 | 232.75 | 85 | 214.85 | 90 |
| *Fine Screens* | Screen Motor | 35.81 | - | 35.81 | - | 35.81 | - |
| *Bioreactor* | Mixers | 680.38 | - | 680.38 | - | 680.38 | - |
| | Air Blowers | 9166.85 | 80 | 8593.92 | 85 | 8164.22 | 90 |
| | IMLR Pumps | 1790.40 | 80 | 1718.78 | 83 | 1647.17 | 85 |
| | RAS Pumps | 2506.56 | 80 | 2434.94 | 83 | 2363.33 | 85 |
| | Total | 14144.19 | - | 13428.02 | - | 12855.10 | - |
| *Membranes* | Air Scour Blowers | 5170.68 | 80 | 4864.52 | 85 | 4592.38 | 90 |
| | Permeate Pumps | 2041.06 | 70 | 1904.99 | 73 | 1904.99 | 75 |
| | Backpulse Pumps | 136.07 | 70 | 128.91 | 73 | 121.75 | 75 |
| | WAS Pumps | 35.81 | 80 | 35.81 | 83 | 35.81 | 85 |
| | Total | 7383.62 | - | 6934.23 | - | 6654.93 | - |
| *Conventional Activated Sludge* | Mixers | 680.38 | - | 680.38 | - | 680.38 | - |
| | Air Blowers | 6517.06 | 80 | 6158.98 | 85 | 5800.90 | 90 |
| | IMLR Pumps | 1790.40 | 80 | 1790.40 | 83 | 1647.17 | 85 |
| | RAS Pumps | 1360.70 | 80 | 1360.70 | 83 | 1289.09 | 85 |
| | Total | 10348.54 | - | 9990.46 | - | 9417.54 | - |
| *Secondary Clarifier* | Rake Arm Torque | 17.90 | - | 17.90 | - | 17.90 | - |
| | WAS Pumps | 35.81 | 80 | 35.81 | 83 | 35.81 | 85 |
| | Total | 53.71 | - | 53.71 | - | 53.71 | - |
| *Dual Media Filters* | Backwash Blower | 8.22 | 80 | 7.82 | 85 | 7.29 | 90 |
| | Backwash Pump | 20.16 | 78 | 19.50 | 81 | 18.96 | 83 |
| | Total | 28.38 | - | 27.32 | - | 26.25 | - |
| *UV Disinfection – Membrane Effluent* | Low-Pressure | 839 | - | 839 | - | 839 | - |
| | Medium-Pressure | 5184 | - | 5184 | - | 5184 | - |
| *UV Disinfection – Filter Effluent* | Low-Pressure | 1229 | - | 1229 | - | 1229 | - |
| | Medium-Pressure | 8496 | - | 8496 | - | 8496 | - |
| *Chlorination – Membrane Effluent* | Pump | 4.48 | 70 | 4.48 | 73 | 4.48 | 75 |
| *Chlorination – Filter Effluent* | Pump | 4.48 | 70 | 4.48 | 73 | 4.48 | 75 |
| *MBR WRP* | With Low-Pressure UV Radiation | 22654.44 | - | 21470.97 | - | 20600.85 | - |
| | With Chlorination | 21819.92 | - | 20636.45 | - | 19766.33 | - |
| *CAS WRP* | With Low-Pressure UV Radiation | 11947.26 | - | 11570.21 | - | 10978.32 | - |
| | With Chlorination | 10722.74 | - | 10345.69 | - | 9753.80 | - |

Figure 6 aides in visualizing the breakdown of the percentage of energy consumption corresponding to each unit process of the 8.8 MGD WRPs found in this research. It should be noted that RAS pump energy is included in the Bioreactor/CAS energy, not the Membranes/Secondary Clarifier energy. The percentages for each unit process inside the CAS WRP correlates to percentages calculated using WEF (2010b) values. For instance using WEF (2010b) values, at 8.8 MGD the aerated grit chamber at a CAS facility uses 122.72 kWh/day and a total plant energy consumption of 10,113.66 kWh/day; resulting in the aerated grit chamber using 1.21% of the total plant's energy consumption. This is comparable to the 1.84% at a CAS WRP found in this research. WEF (2010b) breaks down the energy consumption for each unit process per flow, therefore comparisons can be made with this research. However, comparisons with individual unit processes with EPRI (2002) is not possible because the report provides energy consumption of only the entire facility per flow.

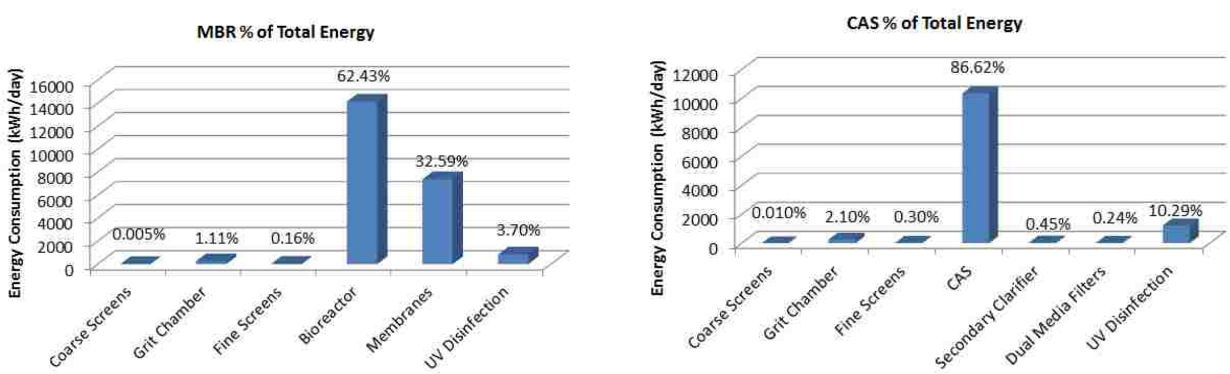



**Figure 6 – Percentage of Total Energy Consumption of the Plant per Unit Process: (a) Energy Consumption of 8.8 MGD MBR WRP; and (b) Energy Consumption of 8.8 MGD CAS WRP**

## 4. Conclusion and Discussion

This research explored the energy consumption of conventional and advanced treatment processes used in reuse wastewater treatment inside satellite WRPs. Both conventional and advanced unit processes were chosen to provide the same treatment level so comparisons were equivalent. For conventional treatment with flowrates varying from 1 to 11 MGD, a CAS process with phosphorous and nitrogen removal averaged an energy footprint of 1382.7 kWh/MG. For advanced treatment, a MBR process with phosphorous and nitrogen removal averaged and energy footprint of 2643.3 kWh/MG. This demonstrates that MBR WRPs are 1.91 times more energy intensive than CAS WRPs, costing an additional $126.06/MG treated at an average energy rate of $0.10/kWh or 1260.6 kWh/MG. The higher cost of MBR systems is associated with air scouring of the membranes for cleaning, which consumes an average of 23.4% of the facility's total energy. In addition higher cost is also related to the higher blower requirements inside the bioreactor, as a result of a lower oxygen transfer efficiency associated with the high solids concentrations. Disinfection of reuse plant effluent using UV radiation was shown to be on average 100 times more energy intensive than chlorination. UV radiation was shown to have an increase of 38.6% in the consumption of energy for disinfection of filter effluent versus membrane effluent with low-pressure lamps, and 63.0% with medium-pressure. Based on these results, energy savings could be realized by: using chlorination as the disinfectant for membrane effluent, if land area permits, as MPN and TSS levels are already minimal; and UV disinfection with low-pressure lamps would still be advisable for filter effluent because higher concentration of microorganisms and TSS is observed. However, if chlorination were used for filter effluent, a 2.8 times increase in sodium hypochlorite per day will be expected, compared to disinfection of membrane effluent.

The results of this study clearly show that advanced treatment processes, typically used in reuse facilities, have a significantly larger energy footprint compared to that of conventional processes. However, there are tradeoffs if conventional treatment processes were to be selected over advanced treatment processes to save energy. For example, if CAS was selected versus a MBR process a doubling in aeration volume and corresponding land area would be needed. Furthermore, more land would be needed for clarifiers and dual media filters to achieve comparable effluent quality. In addition, capital costs of material (e.g. concrete) to provide for additional conventional treatment units also need to be taken into consideration and compared to capital costs of the advanced treatment units (e.g. membranes and UV systems). Chlorination used in place of UV disinfection requires area for a chlorine contact basin and for chemical storage. Therefore overall, advanced treatment processes greatly reduce the real estate area needed but greatly increases the energy consumption of the facility.

The term ‘energy hog’ has been used for satellite WRPs to describe their high energy consumption. This research shows if satellite WRPs are designed using conventional treatment technologies their energy consumption is comparable to that of non-satellite WWTPs. However, when advanced treatment technologies are implemented in satellite WRPs, especially MBRs, ‘energy hog’ can be an adequate term. As of late, large improvements have been made at MBR facilities in energy consumption with the scheduling of air scour timings but more improvement is still needed. With the ever growing increase in satellite WRPs, evaluations on the consumption of energy in these facilities need to be a part of the design process. In this matter, pros and cons of the increase in energy consumption associated with advanced treatment technologies can be evaluated to determine which treatment processes are more suitable for the facility.

# CHAPTER 3

# IMPACTS OF ON-SITE RENEWABLE ENERGY GENERATION ON TOTAL ENERGY CONSUMPTION AND GREENHOUSE GAS EMISSIONS OF SATELLITE WATER REUSE PLANTS

## 1. Introduction

In recent decades, smaller decentralized wastewater treatment plants (WWTP), termed satellite water reuse plants (WRP), have become very prevalent. WRPs are satellite treatment facilities that treat wastewater from a specific part of community and reuse the effluent in or around the location where the wastewater was collected. Due to the close proximity and/or potential direct contact of reclaimed water with the general public, regulations and effluent standards for reuse water are strict and are becoming stricter (Crook, 2011). To achieve these stricter standards on effluent quality and smaller land footprint (i.e. real estate area), additional treatment processes along with advanced technologies are needed (Bennett, 2007; EPRI, 2002; Brandt, et al., 2011; Urkiaga, et al., 2008). Despite the obvious benefits of water reuse and recycle, the application of advanced treatments technologies in WRPs coupled with stringent effluent discharge standards greater energy consumption is likely to result.

Wastewater treatment is a very energy intensive process; over recent years ways to curb this large consumption of energy has been pursued. Energy can be curbed within an existing WWTP by increase of efficiencies in plant equipment and the optimization of plant processes and equipment. To achieve this reduction in energy a WWTP can undergo a benchmarking evaluation, where energy usage for the whole plant and individual processes can be computed and compared to published values (WEF, 2010b). Another way to implement energy saving measures is with the use of supervisory control and data acquisition (SCADA) systems. With a SCADA system, a plant can monitor their own facility's operational data and obtain useful energy measuring units (e.g. kW, kWh, kWh/gal, kWh/ft$^3$, etc.) (WEF, 2010b). However, in order to perform such an evaluation, the necessary sensors must be installed in the plant for the unit operations of interest. For example, plants may choose to monitor air flowrate use in the plant and the respective energy use associated with the motors that fuel the air blowers for varying wastewater flowrates entering the plant. With this type of analysis, it has been found that pumping can represent up to 30% of energy consumption for wastewater treatment and 80% for clean water (Brandt, et al., 2011). To help curb this consumption the use of modern variable speed drives in pumps can result in 83% of energy savings (Brandt, et al., 2011). This is due to the included power factor management on these pumps, but this does require a 4 - 5% increase in rated motor power to control the variable speed (Brandt, et al., 2011). To assist the management of pump efficiency in WWTPs, effective screening must be maintained because grit, rags, debris, and other solids can contribute to higher wear rate (Brandt, et al., 2011). SCADA system energy analyses have shown that aeration typically represents 50 to 60% of the total energy consumption in WWTPs, however with a variety of measures, including checking control set-points, check rates with metered electrical input, equipment performance optimization, and overall routine maintenance, savings in energy can be up to 40% (Brandt, et al., 2011).

A ranking of energy consumption for treatment units has been developed in wastewater treatment. In order of low energy consumption to high: biological filters;

anaerobic membrane bioreactor; bio-aerated flooded filter; step fed activated sludge; nutrient removal activated sludge; and conventional membrane bioreactor (Brandt, et al., 2011). Potential savings in energy consumption in any WWTP will be system-specific and requires a site-specific analysis (Daigger, 2009). There is however a limit to how much energy use within an existing plant can be curbed, because current design requires a minimum amount of energy to run installed processes and equipment. As a result, new approaches are needed to curb (minimize) energy consumption, not only for existing WWTPs but also for future planned plants.

Fossil fuels, oil, coal, and gas, currently are providing over two-thirds of the world's energy (Demirbas, 2009). At this consumption, known petroleum reserves are projected to be depleted in less than 50 years (Demirbas, 2009; Gude, et al., 2010). With the world's energy growth rate of 2% a year and the resulting energy consumption doubling by the year 2035 relative to 1998 and tripling by 2055 (Demirbas, 2009), this depletion rate will only get worse. Thus, the use of these fuels produces enormous amounts of greenhouse gas (GHG) emissions, which are subsequently resulting in crucial environmental problems worldwide including acid rain and global warming (Gude, et al., 2010). To help maintain the reduction in GHG emissions protocols such as the Kyoto Protocol have been adopted by the United Nations Framework Convention on Climate Change. Kyoto Protocol was adopted by countries to help reduce GHG emissions by an average of 5%, against 1990 levels, over a period of five years, 2008 – 2012 (UNFCCC, 2012). But since levels of GHG emissions had increased by 25% since 1990 (The World Bank, 2010); the protocol has only had a slight effect on emissions. Even though the protocol did not prove to be as successful as planned it contributes to the beginning of change.

In the United States, 2 - 4 % of the total energy consumed is for the collection, distribution, and treatment of wastewater and drinking water (McMahon, et al., 2011; Daigger, 2009; U.S. EPA, 2010; Metcalf & Eddy, Inc, 2003; EPRI, 2002; WEF, 2010b). In all WWTPs, consumption of electric power accounts for about 90% of the total energy consumption in a plant (Mizuta, et al., 2010). This current usage of energy translated to 75 billion kWh in 1996 (U.S. EPA, 2008; U.S. EPA, 2010) and was estimated to increase to 100 - 123.45 billion kWh in 2010 (U.S. EPA, 2010; EPRI, 2009). This consumption emits roughly 116 billion lbs (52 million metric tonnes) of carbon dioxide ($CO_2$) into the atmosphere (McMahon, et al., 2011; NRDC, 2009). In order to decrease this production of GHG emissions and dependency on fossil fuels, the use of renewable energy in wastewater treatment has become popular in replacing grid connect as a supply for energy. In addition, efforts have been spent towards wastewater treatment generating some of the energy they consumed via methane generation from anaerobic sludge digestion and the installation of photovoltaic (PV) solar panels throughout or around the plant (Palmer, 2009; Seeta, et al., 2011).

Anaerobic sludge digestion, is generally not found in satellite WRPs, due to the lack of solids handling at the facility to achieve a smaller real estate area. However, introduction of membrane bioreactors (MBR) into satellite reuse plants is expected to significantly reduce land acreage needed. Therefore, application of anaerobic digesters at these facilities should now be re-evaluated. With the increase of pretreatment requirements before the use of a MBR, solids screening removal has become more stringent; thus a richer thicker primary sludge is obtained that can be processed directly in a digester without the need for thickening. For this reason, only one additional unit process is needed to have an energy producing unit at the facility, the digester itself.

In using digesters at a WRP, only primary (screened) sludge can be selected to be diverted to the digester to allow for more energy production and less energy consumption. This is because if even a small amount of waste activated sludge (WAS) were to be blended into the process, the rate of biological reaction in the digester would decrease (Metcalf & Eddy, Inc, 2003). With the WAS being directly discharged back into the collection trunk without processing through the digester, the volume and the overall acreage of the digester will be smaller. Using a single-stage high-rate mesophilic anaerobic digester also provides a small acreage for the digester. With the digestion of the primary sludge, odors can be greatly reduced when compared to discharging undigested primary sludge back into the collection system for further processing.

The use of renewable energy in industry as a whole has had a slow start. Renewable energy only represents a 14-16% total of the world's energy (Demirbas, 2009; Gude, et al., 2010). This number has been projected to reach 48-50% by the year 2040 (Demirbas, 2009; Gude, et al., 2010). This projection has been paralleled with the recent growth rate in renewable energy application such as wind and solar energy during 2009 - 2010 (Trabish, 2012). With the development of new technology, renewable energy has become more cost effective, comparable to grid connect using fossil fuels. Renewable energies, geothermal, solar, and wind, cost 0.07, 0.05-0.09, and 0.05/kWh, respectively; while grid connected electricity costs 0.05-0.09/kWh (Gude, et al., 2010). With the cost of fossil fuels rapidly rising (Mizuta, et al., 2010; Brandt, et al., 2011), the need to conserve energy and transition from fossil fuels to renewable energy has now become a necessity over a luxury.

The use of advanced treatment technologies to treat reuse water requires a large increase in energy consumption compared to conventional unit processes (Chapter 2). In the past, energy consumption and GHG generation has not been a concern in WWTP design and especially in reuse plant design. However, the current efforts to minimize GHG emissions and related energy footprint challenges the actual benefits of reuse plants with advanced treatment. Previous work has been done on the energy consumption in satellite WRPs and was found that with advanced treatment technologies, such as a MBR, requires on average a 1.67 increase in energy consumption compared to a conventional activated sludge system (CAS) (Chapter 2). However, a complete evaluation on GHG emissions and the renewable energy potential of a WRP have not been investigated to date. In this research, a WRP was evaluated to determine the GHG emissions associated with conventional and advanced treatment units. In addition, the renewable energy potential inside the plant was investigated based on acreage available from basin and membrane/clarifier area for a PV solar system, and from biosolids digestion from fine screened (primary) sludge.

## 2. Methodology

The flow diagram for the WRP considered in this study is presented in Figure 7. In order of treatment, the unit processes include: coarse screen, aerated grit chambers, fine screen, conventional activated sludge (CAS) system, membranes, and UV disinfection. When comparing conventional versus advanced unit processes, the membranes were replaced by the combination of secondary clarification and dual media filtration, and UV disinfection by chlorination.

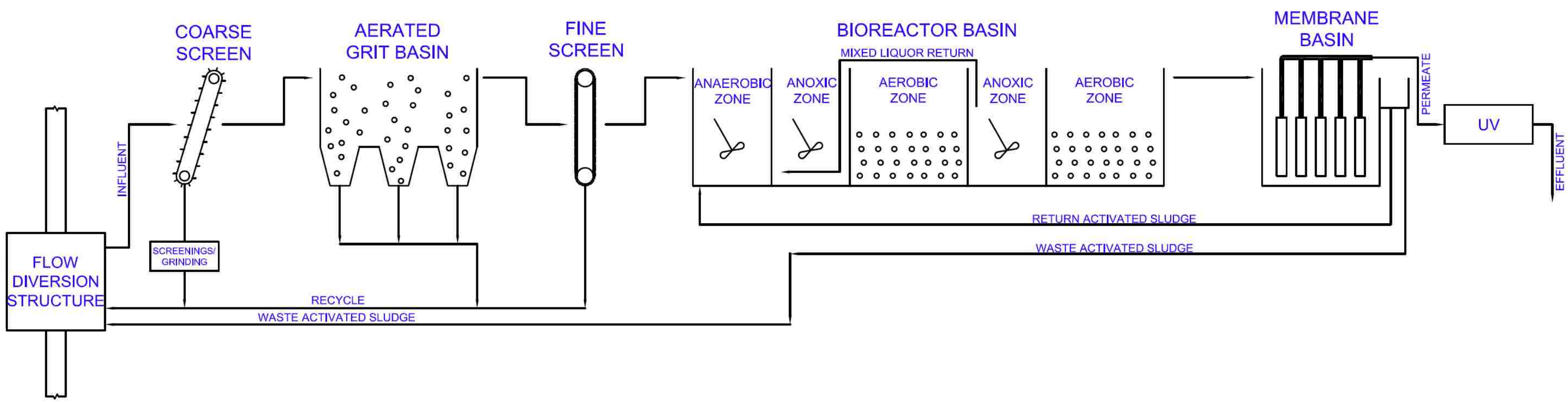



**Figure 7 – Process Flow Diagram of the Water Reuse Plant for Which Greenhouse Gas Emissions are Evaluated**

### *2.1 Influent and Effluent Quality*

The influent characteristics and effluent requirements for the WRP are presented in Table 11. The requirements are typical water reuse standards found in California and Florida, with the exception for the need to remove nutrients. A five-stage modified Bardenpho CAS system is provided at the facility for the removal of the nutrients phosphorous and nitrogen (WEF, 2012; WEF, 2011).

**Table 11 – Plant Influent and Effluent Process Characteristics Found in the Water Reuse Plant**

| Parameter | Influent Characteristics | Effluent Requirements |
|---|---|---|
| **BOD (mg/L)** | 250 | 30 |
| **TSS (mg/L)** | 309 | 30 |
| **TKN (mg/L as N)** | 42 | – |
| **$NH_3$ (mg/L as N)** | 34 | 0.5 |
| **TN (mg/L as N)** | – | 10 |
| **TP (mg/L as P)** | 8 | 0.2 |
| **TC (MPN/100 mL)** | – | 2.2 |
| **TC, daily max (MPN/100 mL)** | – | 23 |
| **Minimum Temp (°C)** | 18.3 | 18.3 |

### *2.2 Energy Consumption in Unit Processes of the Water Reuse Plant*

To determine the energy consumption associated with the reuse plant, the energy driving unit from each process was identified and the energy associated with it was computed (computations shown in Chapter 2). These computations were done using typical design equations available in reference literature (Metcalf & Eddy, Inc, 2003; WEF, 2010a; Qasim, 1999; Davis, 2010; Lin, 2007; WEF, 2012). Energy consumption levels for advanced treatment processes and comparable conventional treatment processes in the satellite WRPs were computed. The energy consumption levels obtained are found in Chapter 2 and are repeated in Table 12 for convenience.

**Table 12 – Energy Consumption per Unit Flow of Each Unit Process in a Satellite Reuse Plant**

| | Unit Process | Energy Consumption per Unit Flow (kWh/MG) | | | | | |
|---|---|---|---|---|---|---|---|
| | | *1 MGD* | *2 MGD* | *4 MGD* | *6 MGD* | *8.8 MGD* | *11 MGD* |
| **MBR Facility** | *Coarse Screens* | 1.156 | 0.578 | 0.289 | 0.193 | 0.131 | 0.158 |
| | *Grit Chamber* | 107.42 | 62.68 | 44.76 | 35.81 | 28.48 | 24.41 |
| | *Fine Screens* | 35.81 | 17.90 | 8.95 | 5.97 | 4.07 | 4.88 |
| | *Bioreactor* | 1591.6 | 1593.8 | 1593.8 | 1593.8 | 1607.3 | 1607.3 |
| | *Membranes* | 904.2 | 904.2 | 904.2 | 904.2 | 839.0 | 839.0 |
| | *UV Disinfection* | 98.0 | 105.0 | 101.0 | 97.3 | 95.3 | 98.0 |
| | ***Total*** | 2738 | 2684 | 2653 | 2637 | 2574 | 2574 |
| **CAS Facility** | *Coarse Screens* | 1.156 | 0.578 | 0.289 | 0.193 | 0.131 | 0.158 |
| | *Grit Chamber* | 107.42 | 62.68 | 44.76 | 35.81 | 28.48 | 24.41 |
| | *Fine Screens* | 35.81 | 17.90 | 8.95 | 5.97 | 4.07 | 4.88 |
| | *CAS* | 1179.8 | 1164.1 | 1164.1 | 1164.1 | 1176.0 | 1176.0 |
| | *Secondary Clarifier* | 8.95 | 6.71 | 6.71 | 6.71 | 6.10 | 6.10 |
| | *Dual Media Filters* | 5.04 | 5.01 | 3.75 | 3.34 | 3.23 | 3.23 |
| | *UV Disinfection* | 138.0 | 138.0 | 134.8 | 138.7 | 139.7 | 134.1 |
| | ***Total*** | 1476 | 1395 | 1363 | 1355 | 1358 | 1349 |

### *2.3 Greenhouse Gas Production*

To compute the GHG production, equivalent carbon dioxide generation potential was used. Carbon dioxide equivalent ($CO_2e$) is the conversion of all GHG (most contributing: carbon dioxide, methane, nitrous oxide, and fluorinated gases) into a common unit for ease of computing and reporting. The GHG emitted from energy consumption in the unit processes was determined based on fuel type from an average of three to eleven separate studies (Shrestha, et al., 2012; Shrestha, et al., 2011). An energy fuel mix found in the southwestern United States was used. The energy fuel mix includes: 60% natural gas, 25% coal, 7% hydroelectric, 7% geothermal, and 1% solar (U.S. EIA, 2010). The emissions rates for these fuel types are as followed: natural gas = 605.9 g $CO_2$e/kWh; coal = 1022.9 g $CO_2$e/kWh; hydroelectric = 25.4 g $CO_2$e/kWh; geothermal = 66.7 g $CO_2$e/kWh; and solar = 70.8 g $CO_2$e/kWh (Shrestha, et al., 2012; Shrestha, et al., 2011). The resulting GHG emission rate used in this research for electrical energy is 626.4 g $CO_2$e/kWh consumed.

### *2.4 Design Parameters and Considerations*

Typical design criteria used to size the PV solar systems and anaerobic digesters are shown in Table 13. Design values in the table are typical of values reported in the design literature. All energy consumption computations for the anaerobic digester are for monthly average flow conditions. Details of the design for each process are discussed below.

**Table 13 – Photovoltaic Solar System and Anaerobic Digester Design Parameters**

| | Parameter | Value | Unit | Reference(s) |
|---|---|---|---|---|
| *Photovoltaic Solar System* | Average Solar Insolation | 0.59 (6.31) | kWh/ft$^2$/day (kWh/m$^2$/day) | 1, 2, 3 |
| | Total Efficiency | 70-80 | % | 2, 3, 4, 5 |
| | Power Generated per Panel Area | 10-16.7 (107.6-179.8) | W/ft$^2$ (W/m$^2$) | 5, 6, 7, 8, 9 |
| *Anaerobic Digester* | Solids Retention Time (SRT) | 15 | day | 10, 11 |
| | Temperature | 95 (35) | ºF (ºC) | 10, 11, 12 |
| | Methanogenic Bacterial Yield for Cell Synthesis | 0.08 | kg VSS/kg bCOD | 10, 12 |
| | Bacterial Endogenous Decay Coefficient | 0.03 | day$^{-1}$ | 10, 12 |
| | Waste Utilization Efficiency | 70 | % | 10 |
| | Percentage of Methane in Digester Gas | 65 | % | 10, 11, 12 |

1 ~ (NREL, 2011); 2 ~ (Energy Matters, 2012); 3 ~ (Find Solar, 2012); 4 ~ (Leonics, 2009); 5 ~ (California Energy Commission, 2001); 6 ~ (Dryden, et al., 1961); 7 ~ (Green, 2005); 8 ~ (Burkart, et al., 2012); 9 ~ (Mandalaki, et al., 2012); 10 ~ (Metcalf & Eddy, Inc, 2003); 11 ~ (WEF, 2010b); 12 ~ (Davis, 2010)

### *2.4.1 Anaerobic Digester*

Key parameters used in the design of the single-stage high-rate mesophilic anaerobic digester can be found in Table 13. The HRT, equivalent to the SRT, was used in the determination of the volume required for the digester (Metcalf & Eddy, Inc, 2003). The amount of methane-forming volatile solids synthesized per day was determined using the complete-mix high-rate digester equation, followed by the calculation of the volume of methane gas using kinetic equations (Metcalf & Eddy, Inc, 2003; Davis, 2010). These were done taking into account the volume of methane gas at the operating temperature of 35ºC. An egg-shaped digester was used in the design to provide a higher mixing efficiency, improved homogeneous biomass, and most importantly, a smaller real estate area in the WRP (Metcalf & Eddy, Inc, 2003; WEF, 2010b).

The anaerobic digestion process produces methane gas that can be used for energy generation; however, digestion itself consumes energy. Energy consumption for the anaerobic digester is driven by the mixers providing a homogeneous biomass mixture and by the heat-exchanger providing heating for the sludge and heat losses through the digester walls. Mixer energy requirements were determined based on the volume of the digester, using an average energy consumption of 6.5 W/$m^3$ (WEF, 2010b). The energy requirement to heat the sludge was determined using (Metcalf & Eddy, Inc, 2003; Davis, 2010; WEF, 2010b):

$$q = M_s C_s \left(T - T_i\right) \qquad \textbf{(1)}$$

where $q$ = heat required, J/day; $M_s$ = mass flow of sludge, kg/day; $C_s$ = specific heat of sludge, J/kg·ºC; $T$ = digestion temperature, ºC; and $T_i$ = influent sludge temperature, ºC. For purposes of this research, 4200 J/kg·ºC was used for the specific heat of sludge

(Metcalf & Eddy, Inc, 2003). The energy required to compensate for the loss of heat through the walls of the digester were determined as (Metcalf & Eddy, Inc, 2003; Davis, 2010; WEF, 2010b):

$$q = UA\Delta T \qquad \textbf{(2)}$$

where q = heat loss, J/sec; U = overall coefficient of heat transfer, J/m$^2$·sec·ºC; A = cross-sectional area perpendicular to heat flow, m$^2$; and $\Delta$T = change in temperature between digestion and surface in question. Coefficients of heat transfer used in the research are 0.68, 0.85, and 0.91 W/m$^2$·ºC for the walls, floor, and roof, respectively (Metcalf & Eddy, Inc, 2003; Davis, 2010; WEF, 2010b). Energy production from the combustion of digester gas was determined using:

$$E = HVe \qquad \textbf{(3)}$$

where $E$ = energy generated, kJ/day; $H$ = heat of combustion, kJ/m$^3$; $V$ = volume of gas produced per day, m$^3$/day; and $e$ = electrical efficiency. In this research, 37,000 kJ/m$^3$ was used for the heat of combustion of methane (WEF, 2010b). An electrical efficiency of 33% was used based off the efficiency for an internal combustion engine (ICE) (WEF, 2010b).

*2.4.2 Photovoltaic Solar System*

Parameters used in the design of the PV solar system can be found in Table 13. Real estate area available for the PV system was determined based off basin and membrane/clarifier area in the form of a shaded structure with tilt single-axis panels. The system size was determined by multiplying the available area by the amount of power that can be generated per solar area. In this research, a radiative efficiency (i.e. panel efficiency) of 15% was used, which provides 13.9 W of power generated per square foot of solar paneling. The energy production from this system size was calculated using:

$$E = P_s I_s e / I_p \tag{4}$$

Where $E$ = energy generated, kWh/day; $P_s$ = PV system size, kW; $I_s$ = solar insolation, kWh/m$^2$/day; $e$ = combined efficiency; and $I_p$ = panel irradiance, kW/m$^2$. The combined efficiency takes into account manufacture rating, wiring and power point tracking losses, and the inverter efficiency (Energy Matters, 2012; California Energy Commission, 2001). A combined efficiency of 80% was used. A panel irradiance of 1000 W/m$^2$ was used for the PV systems per ASTM G173-03 (ASTM International, 2012).

A sensitivity analysis was performed on solar panel efficiency. If a low radiative efficiency of 10.8% (10 W/ft$^2$) were used (California Energy Commission, 2001; Dryden, et al., 1961), this would be a reduction of 28.2% of the energy generated by the panels. If a high radiative efficiency of 18% (16.7 W/ft$^2$) were used (Green, 2005), an increase in energy generation of 20.0% would result.

## 3. Results and Analysis

Estimated energy consumption of the major energy driving and producing units for the anaerobic digester and PV solar system for varying flowrates in the WRP are presented in Table 14. Overall net totals of the energy consumption and generation are also provided.

**Table 14 – Estimated Energy Consumption and Generation of Anaerobic Digester and Photovoltaic Solar System in a Water Reuse Plant**

| | Energy Driving & Producing Equipment | 1 MGD Plant (kWh/day) | 2 MGD Plant (kWh/day) | 4 MGD Plant (kWh/day) | 6 MGD Plant (kWh/day) | 8.8 MGD Plant (kWh/day) | 11 MGD Plant (kWh/day) |
|---|---|---|---|---|---|---|---|
| *Anaerobic Digester* | Mixers | 32.67 | 64.69 | 96.63 | 188.19 | 277.88 | 343.06 |
| | Heat-Exchanger | 252.09 | 478.34 | 708.41 | 1345.50 | 1944.15 | 2343.96 |
| | **Total Consumption** | 284.76 | 543.04 | 805.04 | 1533.69 | 2222.03 | 2687.02 |
| | ICE – Generation | 404.71 | 809.42 | 1214.13 | 2428.25 | 3561.44 | 4451.8 |
| | **Net Total** | 119.95 | 266.38 | 409.09 | 894.56 | 1339.41 | 1764.78 |
| *Photovoltaic Solar System* | Panel Generation – MBR Plant | 116.47 | 235.20 | 470.40 | 705.60 | 1028.58 | 1285.73 |
| | Panel Generation – CAS Plant | 347.46 | 630.46 | 1260.92 | 1891.38 | 2825.71 | 3532.14 |

For flowrates between 1 and 11 MGD, the heat-exchanger consumed on average 87.8% of the total energy consumed by the anaerobic digester for both MBR and CAS facilities. The mixers used to avoid stratification inside the digester only required on average 12.2% of the total energy consumption. Assuming a specific gravity of 1.01 for primary sludge, an average of 653 kWh/ton (0.72 kWh/kg) of sludge digested is generated by the anaerobic digester for both MBR and CAS facilities across all flows. This is the result for all flows as the volume of primary sludge increase proportionally to the flow. Energy consumption in the anaerobic digestion process was found to be higher than values found in WEF (2010b) and Malcolm Pirnie (1995). The energy consumption for an anaerobic digester of an 11 MGD facility was reported as 1850 (WEF, 2010b) and 236.35 (Malcolm Pirnie, 1995) kWh/day, compared to 2687.02 kWh/day found in this research. This difference can be due to the combination of primary and secondary sludge per WEF (2010b). At 11 MGD, the energy generated by the digester was 4451.8 kWh/day. WEF (2010b) reports a value of 3850 kWh/day. This is a 13.5% decrease in energy consumption compared to the value reported in this research. This difference can be due to the type of energy generator used, as different generators have different efficiencies. If microturbines with an efficiency of 27% were used, the energy generated would be 3642.4 kWh/day, making a difference of only 5.4% less comparing to WEF (2010b). In addition, a pattern is seen in the anaerobic digester, as flow increases the fraction of energy generated over energy consumed by the digester increases by an average of 3.1% across all flows.

For flowrates between 1 and 11 MGD, energy generation of the PV solar system in CAS facilities was proven to be on average 2.75 times higher than MBR facilities due to the large real estate size. The real estate size is directly proportional to the amount of energy generated as CAS facilities were on average 2.75 higher in real estate area compared to MBR facilities. The real estate sizes and their corresponding PV system sizes can be found in Table 15. For both MBR and CAS facilities, 0.07 kWh/day is generated per square foot of solar paneling. Future improvements in PV solar cell performance will only make this energy generation even greater. Since 1954, PV solar cells have increased from a two percent radiative efficiency to percentages of twenty-five plus in laboratory settings (Green, 2005; Spanggaard, et al., 2004; Green, 2012; Hecht, 2010). This is compared to the average 15% radiative efficiency (13.9 W/ft$^2$) used in this research.

**Table 15 – Estimated Areas and System Size for PV Installation in Reuse Facilities with Advanced and Conventional Treatment Units**

| | Parameter | *1 MGD* | *2 MGD* | *4 MGD* | *6 MGD* | *8.8 MGD* | *11 MGD* |
|---|---|---|---|---|---|---|---|
| **MBR Facility** | *Area* ($ft^2$) | 1655.78 | 3343.55 | 6687.10 | 10030.66 | 14622.21 | 18277.76 |
| | *System size* (kW) | 23.07 | 46.59 | 93.18 | 139.78 | 203.76 | 254.70 |
| **CAS Facility** | *Area* ($ft^2$) | 4939.43 | 8962.55 | 17925.10 | 26887.64 | 40170.01 | 50212.51 |
| | *System size* (kW) | 68.83 | 124.89 | 249.79 | 374.68 | 559.77 | 699.71 |

Table 16 summarizes energy generation from advanced and traditional treatment facilities incorporating anaerobic digestion and solar power individually and in conjunction per unit flow. These values were derived by dividing the energy consumption/generation per day (kWh/day) by the unit flow (MGD), resulting in energy consumption/generation per million gallon (kWh/MG). Energy consumption patterns are as expected, with the consumption of energy per million gallon decreasing as treatment capacity increases (WEF, 2010b). In addition, energy production patterns are also similar to WEF, 2010b as energy recovery in anaerobic digestion stays constant on a per million gallon basis. This is the result of primary sludge increasing proportionally as flow increases. For both MBR and CAS facilities at flowrates between 1 and 11MGD, an average net total of 136.19 kWh/MG is generated by the anaerobic digester. This correlates to an average of 5.2% of the MBR facility's total energy consumption and 9.9% for the CAS facility. Assuming an average commercial electrical energy rate of $0.10 USD/kWh, the savings in energy costs by the anaerobic digester is $13.62/MG treated. PV solar energy however only generates 117 kWh/MG for MBR facilities and 323 kWh/MG for CAS facilities. This produces on average of 4.4% of the total energy consumption for MBR facilities and 23.3% for CAS facilities; resulting in a savings of

$11.70/MG treated at MBR facilities and $32.30/MG treated at CAS facilities, not including the capital costs of the anaerobic digester.

The low energy generation observed with solar energy is due to panels only being incorporated over basin and membrane/clarifier area. The solar energy generation can be greatly increased if panels were to be placed on top of building structures, parking shade structures, or around the facility itself. The size of the PV systems at WWTPs is not proportional to the treatment capacity of the facility. For instance an 819 kW PV system was installed at a 4.2 MGD facility (Drainville, et al., 2011) while 1000 kW PV systems were installed at 25 and 32 MGD facilities (Seeta, et al., 2011; City of Boulder, 2012). For this reason, comparing energy generation potential by PV systems at WWTPs is impracticable. In this research however, incorporating solar energy on structures was not evaluated because facility layout and design was not developed in this research. For MBR facilities, an average energy savings of 9.6% is accomplished when both anaerobic digestion and solar energy are incorporated in the WRP. While for CAS facilities, an average energy savings of 33.2% is obtained. This is a total savings of 253.36 kWh/MG ($25.34/MG) for MBR facilities and 458.75 kWh/MG ($45.88/MG) for CAS facilities, not including the capital costs of both the anaerobic digester and solar system. If 100% of the energy consumption were to be offset at each facility by solar generation, a 21.6 times increase of available real estate area on average would be required at the MBR facilities and 3.3 times increase at the CAS facilities. If 50% of the energy consumption were to be offset, a 10.3 times increase in available real estate area is required at MBR facilities and 1.1 times increase at CAS facilities.

**Table 16 – Energy Consumption and Generation per Unit Flow of the Anaerobic Digester and Photovoltaic Solar System**

| | Unit Process | Energy Consumption per Unit Flow (kWh/MG) | | | | | |
|---|---|---|---|---|---|---|---|
| | | *1 MGD* | *2 MGD* | *4 MGD* | *6 MGD* | *8.8 MGD* | *11 MGD* |
| **MBR Facility** | ***Wastewater Treatment Total*** | 2738 | 2684 | 2653 | 2637 | 2574 | 2574 |
| | *Anaerobic Digester* | 284.8 | 271.5 | 201.3 | 255.6 | 252.5 | 244.3 |
| | *Anaerobic Digester Generation* | 405 | 405 | 304 | 405 | 405 | 405 |
| | ***Net Total w/Digester*** | 2618 | 2551 | 2551 | 2488 | 2422 | 2413 |
| | *Photovoltaic System Generation* | 116 | 118 | 118 | 118 | 117 | 117 |
| | ***Net Total w/PV*** | 2622 | 2567 | 2535 | 2520 | 2457 | 2457 |
| | ***Net Total w/Digester and PV*** | 2502 | 2433 | 2433 | 2371 | 2305 | 2296 |
| **CAS Facility** | ***Wastewater Treatment Total*** | 1476 | 1395 | 1363 | 1355 | 1358 | 1349 |
| | *Anaerobic Digester* | 284.8 | 271.5 | 201.3 | 255.6 | 252.5 | 244.3 |
| | *Anaerobic Digester Generation* | 405 | 405 | 304 | 405 | 405 | 405 |
| | ***Net Total w/Digester*** | 1356 | 1262 | 1261 | 1206 | 1206 | 1188 |
| | *Photovoltaic System Generation* | 347 | 315 | 315 | 315 | 321 | 321 |
| | ***Net Total w/PV*** | 1129 | 1080 | 1048 | 1040 | 1037 | 1028 |
| | ***Net Total w/Digester and PV*** | 1009 | 947 | 946 | 890 | 884 | 867 |

The costs for both anaerobic digesters and PV solar systems are only for operational energy consumption. Capital costs to install PV systems and digesters were evaluated based off current literature, but were not extensively explored. For PV solar systems, ranges vary widely based on the size and type of system, from \$1.99 - \$7.40/W generated (Barbose, et al., 2011; Goodrich, et al., 2012). In this research an average value of \$4.00/W was assumed. A wide range in capital costs was also found for anaerobic digesters, from \$2574 - \$7000/kWh generated (Navaratnasamy, et al., 2008; IRENA, 2012). An average value of \$5,000/kWh was assumed in this research. Table 17 shows the payback period in years for both energy generating systems. A low, medium, and high energy price as well as municipality and/or government incentives are incorporated in the table. For all cases in the table, PV solar systems will take 40% longer to pay back compared to anaerobic digesters. However in recent decades due to the growth in technologies, capitals costs for PV systems have declined rapidly and PV system capacity has increased. For instance, capital costs for PV systems have decreased by a factor of six and the installed capacity has increased from 100 MW to 2,000 MW in 2000 (Gude, et al., 2010), to now over 5,700 MW in 2012 (SEIA, 2012). These benefits have resulted in a 30% growth in PV systems per year and are estimated to be the largest renewable energy source providing a production of 25.1% of the total global power generation by 2040 (Demirbas, 2009).

**Table 17 – Cost Evaluation of Photovoltaic System and Anaerobic Digester with and without Incentives**

| | Energy Price ($/kWh) | Payback (years) | Payback w/25% Incentive (years) | Payback w/50% incentive (years) |
|---|---|---|---|---|
| ***Photovoltaic System*** | 0.08 | 27.1 | 20.4 | 13.6 |
| | 0.10 | 21.7 | 16.3 | 10.9 |
| | 0.12 | 18.1 | 13.6 | 9.0 |
| ***Anaerobic Digester*** | 0.08 | 19.4 | 14.5 | 9.7 |
| | 0.10 | 15.5 | 11.6 | 7.7 |
| | 0.12 | 12.9 | 9.7 | 6.5 |

Figure 8 helps visualize energy saving trends when comparing advanced and conventional treatment facilities with and without energy generating units. The MBR WRP with energy generating units is on average 2.59 times more energy intensive than the CAS WRP. This is an even greater increase in energy consumption difference compared to MBR WRPs being 1.91 times more energy intensive than CAS WRPs without energy generating units (Chapter 2).

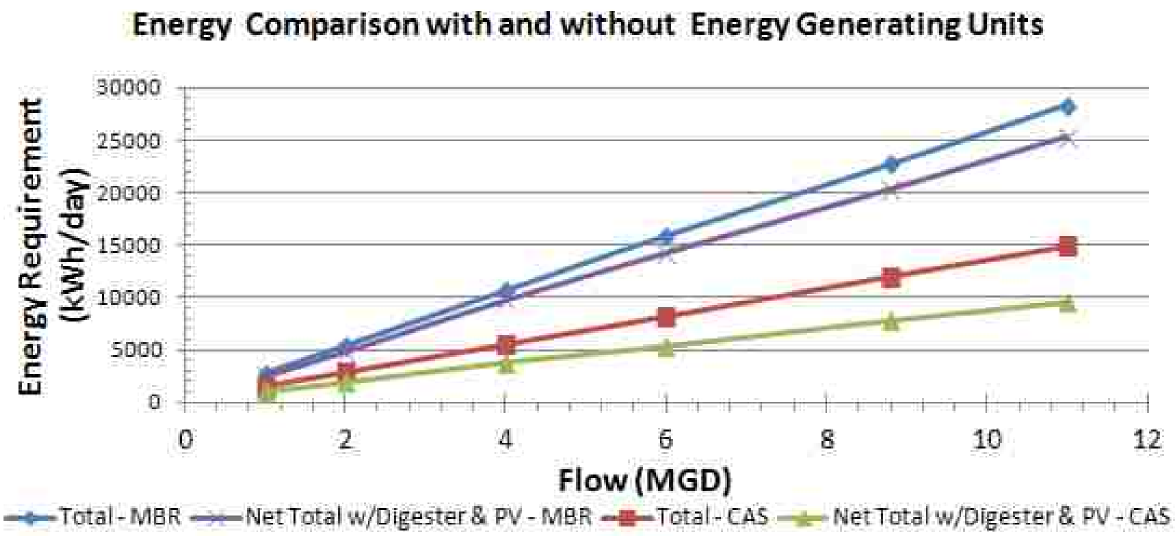



**Figure 8 – Energy Comparison of Advanced and Conventional Treatment Facilities with and without Incorporating Energy Generating Units**

Table 18 summarizes the GHG emissions of each unit process per unit flow in terms of g $CO_2$/MG. Totals are also provided for each scenario with energy generating units. As with energy consumption, GHG emissions with MBRs are 1.91 and 2.59 times more intensive without and with energy generating units at the facilities, respectively, compared to CAS facilities. In MBR WRPs, an average decrease of 9.6% in emissions is observed when energy generating units are used; and 33.2% for CAS WRPs. Even with energy generating units at advanced and conventional treatment WRPs, GHG emissions are still relatively large. For instance at the 8.8 MGD MBR WRP, GHG emissions without energy generating units are 14,190 kg $CO_2$e/day and with energy generating units the emissions are 12,707 kg $CO_2$e/day, as shown in Figure 9. This however is a reduction of 1,483 kg $CO_2$e/day, which is equivalent to: the burning of 3.4 barrels of oil a day, the use of 106 passenger vehicles a day, or the electricity for 68 single-family homes a day (U.S. EPA, 2012).

**Table 18 – Greenhouse Gas Emissions of Each Unit Process per Unit Flow**

| | Unit Process | GHG Emissions per Unit Flow (kg $CO_2e$/MG) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 1 MGD | 2 MGD | 4 MGD | 6 MGD | 8.8 MGD | 11 MGD |
| MBR Facility | Coarse Screens | 0.72 | 0.36 | 0.18 | 0.12 | 0.08 | 0.10 |
| | Grit Chamber | 67.29 | 39.26 | 28.04 | 22.43 | 17.84 | 15.29 |
| | Fine Screens | 22.43 | 11.21 | 5.61 | 3.74 | 2.55 | 3.06 |
| | Bioreactor | 996.98 | 998.36 | 998.36 | 998.36 | 1006.81 | 1006.81 |
| | Membranes | 566.39 | 566.39 | 566.39 | 566.39 | 525.55 | 525.55 |
| | UV Disinfection | 61.39 | 65.77 | 63.27 | 60.95 | 59.70 | 61.39 |
| | ***Total*** | 1715.20 | 1681.36 | 1661.84 | 1651.99 | 1612.53 | 1612.20 |
| | Anaerobic Digester | 178.37 | 170.08 | 126.07 | 160.12 | 158.17 | 153.01 |
| | Anaerobic Digester GHG Savings | 253.51 | 253.51 | 190.13 | 253.51 | 253.51 | 253.51 |
| | ***Net Total w/Digester*** | 1640.06 | 1597.93 | 1597.78 | 1558.60 | 1517.19 | 1511.70 |
| | Photovoltaic System GHG Savings | 72.96 | 73.66 | 73.66 | 73.66 | 73.21 | 73.21 |
| | ***Net Total w/PV*** | 1642.24 | 1607.69 | 1588.17 | 1578.32 | 1539.32 | 1538.98 |
| | ***Net Total w/Digester and PV*** | 1567.11 | 1524.26 | 1524.11 | 1484.93 | 1443.97 | 1438.49 |
| CAS Facility | Coarse Screens | 0.72 | 0.36 | 0.18 | 0.12 | 0.08 | 0.10 |
| | Grit Chamber | 67.29 | 39.26 | 28.04 | 22.43 | 17.84 | 15.29 |
| | Fine Screens | 22.43 | 11.21 | 5.61 | 3.74 | 2.55 | 3.06 |
| | CAS | 739.03 | 729.19 | 729.19 | 729.19 | 736.65 | 736.65 |
| | Secondary Clarifier | 5.61 | 4.20 | 4.20 | 4.20 | 3.82 | 3.82 |
| | Dual Media Filters | 3.16 | 3.14 | 2.35 | 2.09 | 2.02 | 2.02 |
| | UV Disinfection | 86.44 | 86.44 | 84.44 | 86.88 | 87.51 | 84.00 |
| | ***Total*** | 924.68 | 873.81 | 854.01 | 848.66 | 850.47 | 844.94 |
| | Anaerobic Digester | 178.37 | 170.08 | 126.07 | 160.12 | 158.17 | 153.01 |
| | Anaerobic Digester GHG Savings | 253.51 | 253.51 | 190.13 | 253.51 | 253.51 | 253.51 |
| | ***Net Total w/Digester*** | 849.54 | 790.38 | 789.94 | 755.27 | 755.13 | 744.44 |
| | Photovoltaic System GHG Savings | 217.65 | 197.46 | 197.46 | 197.46 | 201.14 | 201.14 |
| | ***Net Total w/PV*** | 707.03 | 676.35 | 656.55 | 651.20 | 649.33 | 643.80 |
| | ***Net Total w/Digester and PV*** | 631.89 | 592.92 | 592.48 | 557.81 | 553.99 | 543.30 |

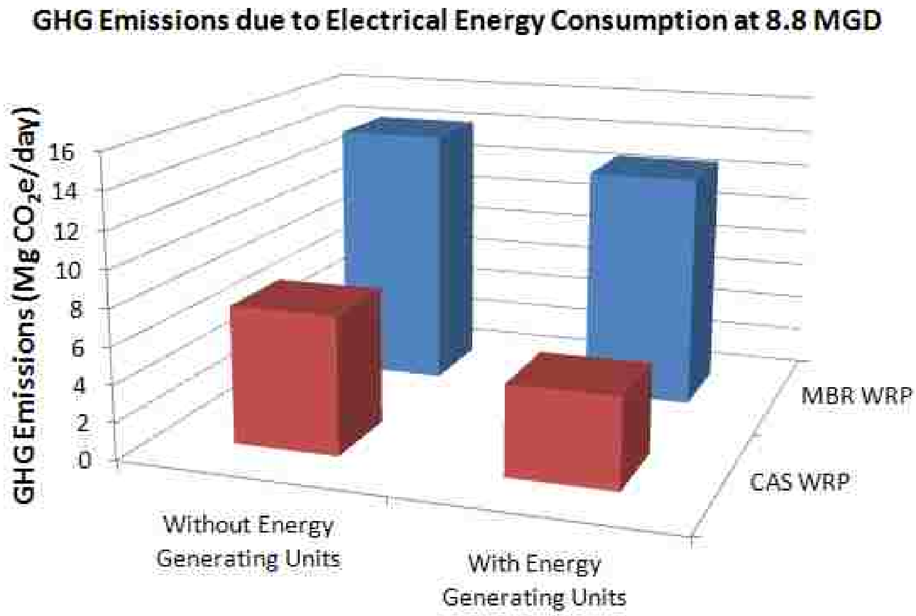



**Figure 9 – Greenhouse Gas Emissions due to Electrical Energy Consumption with and without Energy Generating Units at 8.8 MGD**

## 4. Conclusion and Discussion

This research explored the renewable energy generation potential of a satellite WRP with the addition of a PV solar system and anaerobic digestion. This was performed for two types of facilities: conventional (CAS bioreactor with secondary clarifiers and dual media filtration) and advanced (bioreactor with membrane filtration) treatment satellite WRPs. In addition, the associated GHG emissions for both conventional and advanced treatment processes were evaluated. For conventional treatment, it was found that 9.9% and 23.3% of the facility's total energy consumption can be generated by anaerobic digestion and solar energy, respectively. For advanced treatment, 5.2% and 4.4% of the facility's total energy consumption can be generated by anaerobic digestion and solar energy, respectively. It was observed that energy recovery generation for both anaerobic digestion and PV systems is constant on a per million gallon basis. When both energy generating units are incorporated in satellite WRPs, an average energy savings of 33.2% is accomplished in a CAS facility and 9.6% in a MBR facility, resulting in MBR WRPs averaging 1.86 times more energy intensive than CAS WRPs. This translates to a cost savings in electricity of $25.34/MG treated for MBR facilities and $45.88/MG treated at CAS facilities using an average commercial energy rate of $0.10/kWh. The payback periods for both anaerobic digestion and solar energy were investigated and it was found that no matter the energy rate or the incentive, solar energy requires on average 40% longer to pay back compared to anaerobic digestion.

Furthermore, the results of this research showed that in terms of GHG emissions, MBR WRPs without energy generating units are 1.91 times more intensive than CAS WRPs and 2.59 times more intensive with energy generating units. With or without energy generating units, GHG emissions are still very large at WRPs. For MBR WRPs, 1,656 kg $CO_2e$/MG treated is emitted without energy generating units at the facilities, while 1,497 kg $CO_2e$/MG treated is emitted at the facilities with energy generating units. For CAS WRPs, 866 kg $CO_2e$/MG treated is emitted at facilities without energy generating units, while 579 kg $CO_2e$/MG treated is emitted at facilities with energy generating units. This research has shown that with the addition of energy generating units the energy consumption of the facility can have the potential to be greatly decreased. Performing such energy analyses will provide a means for engineers and operators in the decision making process regarding sustainability of using advanced or conventional treatment technologies at a reuse facility. The term ‘energy hog’ is often used for satellite WRPs. With time, as more energy saving and producing measures are implemented, satellite WRPs will have the prospective to be termed ‘energy neutral’ facilities, in replacement of ‘energy hog’.

# CHAPTER 4

# CONCLUSIONS

Wastewater treatment is a very energy intensive process and with the continued increase in satellite water reuse plants (WRPs), and the associated advanced treatment processes with these plants, this energy consumption will only increase.  In the arid southwestern United States where nutrient requirements must be met, along with the strict standards and regulations on reuse water, increased energy consumption is inevitable.  This research investigated the intertwined resources of wastewater and energy, along with the associated greenhouse gas (GHG) emissions from the treatment of wastewater at satellite WRPs.  With the growing concerns of GHG emissions and linked crucial environmental problems, implementation of renewable energy resources was used to minimize these emissions.  Objectives of this research were:  (1) to investigate the impact of conventional and advanced treatment technologies on energy consumption at satellite WRPs; (2) to evaluate the impact of renewable technologies implementation on energy consumption and associated GHG generation at satellite WRPs; and (3) to compare energy footprint and associated real estate area required for advanced and conventional treatment technologies.  The conclusions of this research are as follows:

- When comparing advanced treatment processes, membrane bioreactor (MBR), with conventional treatment processes, conventional activated sludge (CAS) with secondary clarifiers and dual media filters, the MBR requires on average 2.10 times more energy to treat to the same effluent quality for flowrates between 1 and 11 MGD.

- Comparing advanced disinfection, ultraviolet (UV) radiation, with conventional disinfection, chlorination, resulted in UV disinfection being 100 times more energy intensive for both MBR and CAS WRPs. When comparing the energy consumption of disinfecting membrane effluent against filter effluent with UV disinfection, it was found that an increase of 38.6% in energy is required to treat filter effluent with low-pressure lamps, and 63.0% with medium-pressure. Comparing energy consumption with low-pressure lamps versus medium-pressure lamps, it was found that medium-pressure lamps required an increase of 5.96 and 7.01 times in energy consumption for MBR and CAS WRPs, respectively. When disinfecting with chlorination, CAS WRPs require 2.8 times the amount of sodium hypochlorite needed compared to MBR WRPs.
- For flowrates between 1 to 11 MGD, MBR and CAS WRPs with low-pressure UV disinfection required on average 2643.3 and 1382.7 kWh/MG, respectively. This demonstrates the MBR WRPs are 1.91 times more energy intensive than CAS WRPs, costing and additional $126.06/MG treated in energy consumption using an average commercial energy rate of $0.10/kWh.
- The highest energy consuming unit in the MBR WRP contributing to the large energy footprint is the air scour blowers; requiring on average 23.7% of the facility's total energy consumption.
- A sensitivity analysis on 'wire-to-water' efficiencies has shown that even with a slight increase or decrease in efficiencies (±5% for pumps and ±10% for blowers), the total energy consumption of the entire plant can be greatly affected, by as much as an 11.3% increase or 9.4% decrease.

- Comparing basin real estate area between the MBR and CAS WRPs, it was found that the CAS WRPs required a doubling in aeration volume, resulting in a doubling in the acreage. Total real estate area (the addition of secondary treatment and filtration units) for MBR WRPs was on average 1666 $ft^2$/MG, while CAS WRPs was 4585 $ft^2$/MG. Comparing total real estate yields an increase of 2.75 times in acreage for CAS WRPs. This means it costs MBR WRPs an increase in energy consumption of 0.43 kWh/MG per square foot of real estate saved or $21.50/MG per 500 $ft^2$ of real estate saved.
- Using the real estate area for photovoltaic (PV) solar systems, 4.4% of the facility's total energy consumption can be generated for MBR WRPs and 23.3% can be generated for CAS WRPs.
- If anaerobic digesters were to be added to a plant's unit processes for energy generation by primary sludge digestion, 5.2 and 9.9% of the total facility's energy consumption can be generated for MBR and CAS WRPs, respectively.
- When both PV solar systems and anaerobic digesters are incorporated at a WRP, savings in energy can be 9.6 and 33.2% of the total facilities energy consumption for MBR and CAS WRPs, respectively. This translates to a cost savings in electricity of $25.34/MG treated for MBR facilities and $45.88/MG treated at CAS facilities.
- With or without the use of energy generating units, GHG emissions due to electrical energy consumption are still very large at WRPs. Considering an energy fuel mix of 60% natural gas, 25% coal, 7% hydroelectric, 7% geothermal, and 1% solar, emissions for MBR WRPs are 1,656 kg $CO_2$e/MG treated without

energy generating units at the facilities, while 1,497 kg $CO_2$e/MG treated is emitted at the facilities with energy generating units. This is a reduction of 9.6%. For CAS WRPs, 866 kg $CO_2$e/MG treated is emitted at facilities without energy generating units, while 579 kg $CO_2$e/MG treated is emitted at facilities with energy generating units. Achieving a 33.2% reduction.

This research has shown that with the use of design criteria and equations for unit processes, along with their associated fundamental energy equations, engineers can determine a very accurate estimate of energy consumption for individual unit processes of an entire WRP. The values found closely match actual energy consumption reported by varies literature. This approach highlights a means for engineers and operators to target unit processes that are candidates for reduction in energy consumption and provide a basis for decision making regarding sustainability of using advanced treatment technologies at a reuse facility. In addition, with the increase in satellite WRPs and the overall increase in advanced treatment technologies at wastewater treatment plants in general, evaluations on the consumption of energy at these facilities needs to be a part of the design process; providing pros and cons to determine the need for certain unit process and the overall sustainability of the facility.

This research has provided a beginning in the determination of energy consumption and the corresponding GHG emissions inside satellite WRPs. Both advanced and conventional treatment processes commonly used at these facilities have been evaluated in this study. Two forms of renewable energy generation were also evaluated to determine the energy savings and GHG reduction that can be achieved at these facilities. However, this study is not comprehensive and much work remains to be performed. Below are suggestions for future research:

- To investigate the impact on energy consumption for different advanced and conventional treatment processes other than the ones investigated in this research.
- Investigate the potential energy generation for other renewable technologies.
- To design and evaluate a total facility layout for which actual drawings are available. In this matter a complete facility energy calculation can be done as pumping stations will now be included (e.g. influent pumping station, primary effluent pumping station, and filter influent pumping station).
- To evaluate renewable energy implementation in facilities for which actual drawings are available. This will provide a total acreage of the facility giving the ability to increase solar area to the tops of structures (e.g. rooftops and parking structures).
- To perform equipment energy audits as equipment ages to determine if energy consumption of the reuse facility increases with age.
- To perform life cycle analysis to compare GHG emissions of various unit processes.
- To evaluate total GHG emissions from WRPs, this includes emissions from the unit processes themselves (e.g. carbon dioxide, methane, and nitrous oxide from biological treatment with activated sludge).

# APPENDIX A

# DESIGN PARAMETERS AND EQUATIONS FOR UNIT OPERATIONS AND ENERGY COMPUTATION EQUATIONS USED

### *A-1 Coarse Screens*

The initial open channel leading into the coarse screens was designed using Manning's equation:

$$Q = \frac{K_n}{n} A R^{2/3} S^{1/2}$$ (Sturm, 2010; Mays, 2010)

where $Q$ = flow rate; $K_n$ = 1.0 with $R$ in m and $Q$ in $m^3/s$, and 1.49 for $R$ in ft and $Q$ in $ft^3/s$; $n$ = Manning's coefficient; $R$ = hydraulic radius; and $S$ = bed slope. For purposes of this research, a value of 0.015 (Sturm, 2010; Mays, 2010) was used for the Manning's coefficient in determination of channel properties. Velocity in this channel should exceed 1.3 ft/sec (0.4 m/s) during minimum flows to ensure grit deposition is avoided (WEF, 2010a). If this is impossible due to diurnal flows, a velocity of 3 ft/sec (0.9 m/s) should be used during peak flows to ensure resuspension of solids (WEF, 2010a). The maximum approach velocity was in the desired range at 2.35 ft/sec (Metcalf & Eddy, Inc, 2003; Qasim, 1999; WEF, 2010a). Key parameters used in the design can be found in Table A 1.

**Table A 1 – Coarse Screen Design Parameters**

| Parameter | Value | Unit | Reference(s) |
|---|---|---|---|
| **Bar width** | 5-15 | mm | (Metcalf & Eddy, Inc, 2003; Davis, 2010) |
| | 8-10 | mm | (Qasim, 1999) |
| **Clear spacing between bars** | 10-50 | mm | (Metcalf & Eddy, Inc, 2003) |
| | 15-75 | mm | (Qasim, 1999) |
| | 6-75 | mm | (Davis, 2010) |
| **Bar angle from vertical** | 0-30 | ° | (Metcalf & Eddy, Inc, 2003; Davis, 2010) |
| | 5-15 | ° | (Qasim, 1999) |
| **Bar shape factor for sharp-edged rectangular bars** | 2.42 | – | (Qasim, 1999; Lin, 2007; WEF, 2010a) |
| **Maximum approach velocity** | 0.6-1.0 | m/s | (Metcalf & Eddy, Inc, 2003; Qasim, 1999) |
| | 0.6-1.2 | m/s | (WEF, 2010a) |
| **Minimum headloss** | 6 | in | (WEF, 2010a) |
| **Average headloss** | 2-24 | in | (WEF, 2010a) |
| **Maximum headloss** | 36 | in | (WEF, 2010a) |

To determine the headloss across the screen the following equations were used:

$$h_L = \frac{1}{C}\left(\frac{V^2 - v^2}{2g}\right)$$ (Metcalf & Eddy, Inc, 2003; Qasim, 1999; Lin, 2007) or

$$h_L = \frac{k\left(V^2 - v^2\right)}{2g}$$ (Davis, 2010; WEF, 2010a); and

$$h_L = \beta\left(\frac{W}{b}\right)^{4/3} h_v \sin\theta$$ (Qasim, 1999; Lin, 2007; WEF, 2010a)

where $h_L$ = headloss; $C$ = empirical discharge coefficient to account for turbulence and losses; $k$ = friction coefficient; $V$ = velocity of flow through the openings of the screen; $v$ = approach velocity upstream of the screen; $g$ = gravitational acceleration; $\beta$ = bar shape factor; $W$ = maximum cross-sectional width of screen in the direction facing the flow; $b$ = minimum clear space of the screen; $h_v$ = velocity head upstream of the screen; and $\theta$ = angle of bars from horizontal. The values for $C$ are dependent on whether a headloss for clean or clogged screen is desired. For clean screens the value is typically

0.7 and for 50% clogged screens the value is 0.6 (Metcalf & Eddy, Inc, 2003; Qasim, 1999; Lin, 2007). The same goes for the values of *k*. For clean screens the value is typically 1.4 and for partially blinded screens the value is 1.7 (Davis, 2010; WEF, 2010a). For calculating the headloss including the coarse screen angle, a value of 70° (Metcalf & Eddy, Inc, 2003; Davis, 2010; Qasim, 1999) was used and a value of 2.42 (Qasim, 1999; Lin, 2007; WEF, 2010a) was used for the bar shape factor for sharp-edged rectangular bars. Both the modified minor loss headloss equation and the Kirshmer's equation were used to calculate headloss; the higher of the two governed for the design. Energy consumption for the coarse screens is driven by the size of the motor that powers the rake and the rake cleaning frequency. Based on channel and screen dimensions, a motor size for the rake was obtained using a graphical method provided by a screen manufacturer (Vulcan Industries, Inc, 2011). Using this motor size, energy consumption was determined for cleaning intervals 15, 20, and 30 minutes using a manufacture raking speed of 20 ft/min. Table A 2 shows the design for the coarse screens for the 8.8 MGD WRP facility.

**Table A 2 – Coarse Screen Design at 8.8 MGD**

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| **Influent Channel Design Parameters** | | |
| Channel Width | 4 | ft |
| Channel Slope | 0.05 | % |
| Manning's Coefficient | 0.015 | |
| **Influent Channel Calculations** | | |
| Average Flow Height | 1.78 | ft |
| Average Flow Velocity | 2.13 | ft/sec |
| Peak Flow Height | 2.40 | ft |
| Peak Flow Velocity | 2.35 | ft/sec |
| **Coarse Screen Design Parameters** | | |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| Bar Width | 5-15 | mm |
| | 8-10 | mm |
| Clear spacing between bars | 10-50 | mm |
| | 15-75 | mm |
| | 6-75 | mm |
| Bar angle for vertical | 0-30 | ° |
| | 5-15 | ° |
| Bar Shape Factor for sharp-edged rectangular bars | 2.42 | |
| **Coarse Screen Calculations** | | |
| Bar Spacing | 0.75 | in |
| Bar Width | 0.375 | in |
| Bar Angle from horizontal | 70 | ° |
| Number of bars | 43 | |
| Available space through bars | 2.69 | ft |
| Maximum cross section width | 1.34 | ft |
| Average Flow Velocity through bars | 3.17 | ft/sec |
| Peak Flow Velocity through bars | 3.50 | ft/sec |
| *Clean w/angle* | | |
| Average Flow Headloss | 0.063748 | ft |
| Peak Flow Headloss | 0.077624 | ft |
| *Clean w/o angle* | | |
| Average Flow Headloss | 0.122633 | ft |
| Peak Flow Headloss | 0.149325 | ft |
| *Clogged w/o angle* | | |
| Average Flow Headloss | 0.925478 | ft |
| Peak Flow Headloss | 1.126918 | ft |
| **Power Requirements - 15 min Interval** | | |
| Motor size | 2 | Hp |
| Motor size | 1.492 | kW |
| Rake speed | 20 | ft/min |
| Screen length | 10 | ft |
| Time to complete one rake | 0.5 | min |
| Time between cleanings | 15 | min |
| Number of cleanings | 93 | |
| Time spent cleaning | 46.5 | min/day |
| Total energy consumption | 1.1563 | kWh/day |
| **Power Requirements - 20 min Interval** | | |
| Motor size | 2 | Hp |
| Motor size | 1.492 | kW |

| *Parameter* | *Value* | *Unit* |
| --- | --- | --- |
| Rake speed | 20 | ft/min |
| Screen length | 10 | ft |
| Time to complete one rake | 0.5 | min |
| Time between cleanings | 20 | min |
| Number of cleanings | 71 | |
| Time spent cleaning | 35.5 | min/day |
| Total energy consumption | 0.882767 | kWh/day |
| **Power Requirements - 30 min Interval** | | |
| Motor size | 2 | Hp |
| Motor size | 1.492 | kW |
| Rake speed | 20 | ft/min |
| Screen length | 10 | ft |
| Time to complete one rake | 0.5 | min |
| Time between cleanings | 30 | min |
| Number of cleanings | 48 | |
| Time spent cleaning | 24 | min/day |
| Total energy consumption | 0.5968 | kWh/day |

*A-2 Aerated Grit Chamber*

Parameters used in the design of the aerated grit chamber can be found in Table A 3.

**Table A 3 – Aerated Grit Chamber Design Parameters**

| Parameter | Value | Unit | Reference(s) |
|---|---|---|---|
| **Detention time at peak flow** | 2-5 | min | (Metcalf & Eddy, Inc, 2003; Qasim, 1999; Davis, 2010) |
| | 3-10 | min | (WEF, 2010a) |
| **Air supply per unit length** | 3-8 | cfm/ft | (Metcalf & Eddy, Inc, 2003; Qasim, 1999; WEF, 2010a) |
| **Depth** | 2-5 | m | (Metcalf & Eddy, Inc, 2003; Qasim, 1999; Davis, 2010) |
| | 3.7-5 | m | (WEF, 2010a) |
| **Length** | 7.5-20 | m | (Metcalf & Eddy, Inc, 2003; Qasim, 1999) |
| | 7.5-27.5 | m | (Davis, 2010) |
| **Width** | 2.5-7 | m | (Metcalf & Eddy, Inc, 2003; Qasim, 1999; Davis, 2010) |
| **Width-depth ratio** | 1:1-5:1 | – | (Metcalf & Eddy, Inc, 2003; Qasim, 1999; Davis, 2010) |
| | 0.8:1-1:1 | – | (WEF, 2010a) |
| **Length-width ratio** | 2.5:1-5:1 | – | (Qasim, 1999; Davis, 2010) |
| | 3:1-5:1 | – | (Metcalf & Eddy, Inc, 2003) |
| | 3:1-8:1 | – | (WEF, 2010a) |

To determine the hydraulic retention time (HRT) inside the aerated grit chamber the following equation was used:

$\theta = \frac{V}{Q}$ (Metcalf & Eddy, Inc, 2003; Davis, 2010; Qasim, 1999; WEF, 2010a)

where $\theta$ = HRT; $V$ = volume of the tank; and $Q$ = flow rate flowing through the tank. To determine the volume of the grit basin a depth of 10 feet (Metcalf & Eddy, Inc, 2003; Davis, 2010; Qasim, 1999), width-depth ratio of 1.6 (Metcalf & Eddy, Inc, 2003; Davis, 2010; Qasim, 1999), and a length-width ratio of 2.5 (Davis, 2010; Qasim, 1999) were chosen. Energy consumption for the aerated grit chamber is driven by the air blower capacity used to maintain discrete particle sedimentation and can be estimated by the following equation:

$$BHP = \left[(4.28E-4)q_s T_a / e\right] * \left[(P_d / P_b)^{0.283} - 1\right] \quad \text{(U.S. EPA, 1989)}$$

where *BHP* = brake horsepower, hp; $q_s$ = required flow rate, scfm; $T_a$ = blower inlet air temperature, °R; *e* = blower and motor combined efficiency; $P_d$ = blower discharge pressure, psia (the addition of atmospheric pressure and the system head); and $P_b$ = field atmospheric pressure, psia. System head was estimated as per (U.S. EPA, 1989) using headloss values for diffuser (0.70 psi; 4.826 kPa), piping (0.15 psi; 1.034 kPa), and inlet valve and filter headloss (0.30 psi; 2.068 kPa). Atmospheric pressure at 2,000 feet (609.6 meters) elevation was used and a combined blower and motor efficiency of 80% were assumed (Metcalf & Eddy, Inc, 2003; Davis, 2010). Table A 4 shows the design for the aerated grit chamber for the 8.8 MGD WRP facility.

**Table A 4 – Aerated Grit Chamber Design at 8.8 MGD**

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| **Aerated Grit Chamber Design Parameters** | | |
| Number of hoppers | 3 | |
| Detention time at peak flow | 2-5 | min |
| | 3-10 | min |
| Air supply per unit length | 3-8 | cfm/ft |
| *Dimensions* | | |
| Depth | 2-5 | m |
| | 3.7-5 | m |
| Length | 7.5-20 | m |
| | 7.5-27.5 | m |
| Width | 2.5-7 | m |
| Width-depth ratio | 1:1-5:1 | |
| | 0.8:1-1:1 | |
| Length-width ratio | 2.5:1-5:1 | |
| | 3:1-5:1 | |

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| | 3:1-8:1 | |
| **Aerated Grit Chamber Calculations** | | |
| *Dimensions* | | |
| Detention time required at peak flow | 4.5 | min |
| Volume required | 6098 | $ft^3$ |
| Depth | 10 | ft |
| Width-depth ratio | 1.6 | :1 |
| Width | 16 | ft |
| Length-width ratio | 2.5 | :1 |
| Length | 40 | ft |
| Volume provided | 6400 | $ft^3$ |
| Detention time provided | 4.72 | min |
| *Air Requirement* | | |
| Air criteria requirement per unit length | 8 | cfm/ft |
| Air required | 320 | $ft^3$/min |
| *Blower and Diffuser Design* | | |
| Blower peaking capacity factor | 1.5 | |
| Blower capacity requirement | 480 | $ft^3$/min |
| Diffuser Capacity | 30 | $ft^3$/min |
| Number of diffusers | 16 | |
| **Energy Requirement** | | |
| Static head | 4.335 | psi |
| Diffuser headloss | 0.70 | psi |
| Piping headloss | 0.15 | psi |
| Inlet valve and filter headloss | 0.30 | psi |
| System head | 5.485 | psig |
| Atmospheric pressure (2000 ft) | 13.779 | psia |
| Discharge pressure | 19.264 | psia |
| Efficiency (blower & motor combined) | 0.8 | |
| Brake horsepower | 14 | Hp |
| Motor size | 10.444 | kW |
| Total energy consumption | 250.656 | kWh/day |

*A-3 Fine Screens*

Design and considerations for the open channel before the fine screens are the same as for the open channel before the coarse screens. Parameters used in the design for the fine screen can be found in Table A 5.

**Table A 5 – Fine Screen Design Parameters**

| Parameter | Value | Unit | Reference |
|---|---|---|---|
| **Perforation size** | 1-3 | mm | (WEF, 2010a) |
| **Average headloss** | 2-24 | in | (WEF, 2010a) |
| | 30-54 | In | (Metcalf & Eddy, Inc, 2003) |
| **Maximum headloss** | ≥36 | in | (WEF, 2010a) |

To determine the headloss across the screen the following equation was used

$h_L = \frac{1}{2g}\left(\frac{v}{C}\right)^2 = \frac{1}{2g}\left(\frac{Q}{CA}\right)^2$ (Metcalf & Eddy, Inc, 2003; Davis, 2010; Lin, 2007; WEF, 2010a)

where $h_L$ = headloss; $v$ = approach velocity; $C$ = discharge coefficient; $g$ = gravitational acceleration; $Q$ = discharge through screen; and $A$ = effective open area of submerged screen. For the headloss of a clean screen, the value for $C$ is 0.60 to 0.61 (Metcalf & Eddy, Inc, 2003; Qasim, 1999; Davis, 2010; WEF, 2010a). If headloss is desired for other than clean screens, a blinding factor of up to 50% can be applied by reducing the open area of the submerged screen by the same percentage (WEF, 2010a). Typical effective open areas for fine screens and their corresponding solid removal rates are shown in Table 3. Energy consumption for the fine screens was computed using the same procedure as for the coarse screens, except the motor for the screen is ran continuously. Table A 6 shows the design for the fine screen for the 8.8 MGD WRP facility.

**Table A 6 – Fine Screen Design at 8.8 MGD**

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| **Influent Channel Design Parameters** | | |
| Channel Width | 2.5 | ft |
| Channel Slope | 0.05 | % |
| Manning's Coefficient | 0.015 | |
| **Influent Channel Calculations** | | |
| Average Flow Height | 2.97 | ft |
| Average Flow Velocity | 2.04 | ft/sec |
| Peak Flow Height | 4.17 | ft |
| Peak Flow Velocity | 2.16 | ft/sec |
| **Fine Screen Design Parameters** | | |
| Perforation Size | 2 | mm |
| Effective Open Area | 30 | % |
| Coefficient of Discharge | 0.61 | |
| **Fine Screen Calulations** | | |
| Average Flow Cross-sectional Area | 7.43 | $ft^2$ |
| Peak Flow Cross-sectional Area | 10.44 | $ft^2$ |
| *Clean* | | |
| Average Flow Headloss | 1.93 | ft |
| Peak Flow Headloss | 2.17 | ft |
| **Energy Requirement** | | |
| Motor size | 2 | Hp |
| Motor size | 1.492 | kW |
| Screen speed | 18 | ft/sec |
| Screen length | 10 | ft |
| Time to complete one band | 0.555556 | min |
| Time spent cleaning | 1440 | min/day |
| Total energy consumption | 35.808 | kWh/day |

### *A-4 Activated Sludge (Bioreactor)*

A five-stage modified Bardenpho CAS system is provided for nutrient removal of both phosphorous and nitrogen (WEF, 2012; WEF, 2011). The BOD and solids removal by the coarse and fine screens were based on data provided by manufacturers (Table 4). Microbiological parameters of the activated sludge process can be found in Table 5. Key design parameters for the activated sludge system are found in Table A 7 and design equations are found in Table A 8.

**Table A 7 – Activated Sludge Design Parameters**

| Parameter | Value | Unit | Reference |
|---|---|---|---|
| **Solids Retention Time** | 10 | day | (Menniti, et al., 2011) |
| **Internal Mixed Liquor Recycle (IMLR)** | 200 | % | (WEF, 2010a; WEF, 2006; WEF, 2011) |
| **Return Activated Sludge (RAS) Recycle Ratio – MBR** | 400 | % | (WEF, 2010a; WEF, 2006; WEF, 2012) |
| **RAS Recycle Ratio – CAS** | 100 | % | (WEF, 2010a; Metcalf & Eddy, Inc, 2003) |
| **VFA** | 43 | mg/L | |

**Table A 8 – Design Equations for Activated Sludge Process**

| Parameter | Equation | Reference |
|---|---|---|
| **Hydraulic Retention Time (HRT)** | $\theta_x = \frac{V}{Q^0} = \frac{V(1+R)}{Q}$ | (Rittmann, et al., 2001) |
| **HRT in Reactor** | $\theta_r = \frac{1}{\hat{q}X_a}\left[K\ln\left(\frac{S^i}{S}\right)+\left(S^i-S\right)\right]$ | (Rittmann, et al., 2001) |
| **Solids retention time (SRT)** | $\theta_x = \frac{X_a V}{X_a^e Q^e + X_a^w Q^w}$ | (Rittmann, et al., 2001) |
| **SRT at which microorganisms washout** | $\theta_x^{\min} = \frac{K+S^0}{S^0(Y\hat{q}-b)-Kb} \quad S \to S^0$ <br> $\left[\theta_x^{\min}\right]_{\lim} = \frac{1}{Y\hat{q}-b} \quad S \to \infty$ | (Rittmann, et al., 2001) |
| **Reactor substrate concentration** | $S = K\frac{1+b\theta_x}{\theta_x(Y\hat{q}-b)-1}$ | (Rittmann, et al., 2001) |
| **Reactor minimum substrate concentration** | $S_{\min} = K\frac{b}{Y\hat{q}-b} \quad \theta_x \to \infty$ | (Rittmann, et al., 2001) |

| Parameter | Equation | Reference |
|---|---|---|
| **Reactor active microorganism concentration** | $X_a = \theta_x \dfrac{X(-r_{ut})}{1+b\theta_x}$ <br> $X_a = \dfrac{\theta_x}{\theta}\dfrac{Y(S^0-S)}{1+b\theta_x}$ | (Rittmann, et al., 2001) |
| **Reactor inert microorganism concentration** | $X_i = \dfrac{\theta_x}{\theta}\left[X_i^0 + X_a(1-f_d)b\theta\right]$ | (Rittmann, et al., 2001) |
| **Reactor volatile suspended solids concentration** | $X_v = X_i + X_a$ <br> $X_v = \dfrac{\theta_x}{\theta}\left[X_i^0 + \dfrac{Y(S^0-S)(1+(1-f_d)b\theta_x)}{1+b\theta_x}\right]$ | (Rittmann, et al., 2001) |
| **Active biological sludge production rate** | $r_{abp} = \dfrac{X_a V}{\theta_x}$ | (Rittmann, et al., 2001) |
| **Total biological solids production rate** | $r_{tbp} = \dfrac{X_v V}{\theta_x}$ | (Rittmann, et al., 2001) |
| **Substrate-utilization-associated products** | $UAP = -\dfrac{(\hat{q}_{UAP}X_a\theta + K_{UAP} + k_1 r_{ut}\theta)}{2} + \dfrac{\sqrt{(\hat{q}_{UAP}X_a\theta + K_{UAP} + k_1 r_{ut}\theta)^2 - 4K_{UAP}k_1 r_{ut}\theta}}{2}$ | (Rittmann, et al., 2001) |
| **Biomass-associated products** | $BAP = \dfrac{-(K_{BAP} + (\hat{q}_{BAP} - k_2)X_a\theta)}{2} + \dfrac{\sqrt{(K_{BAP} + (\hat{q}_{BAP} - k_2)X_a\theta)^2 + 4K_{BAP}k_2 X_a\theta}}{2}$ | (Rittmann, et al., 2001) |
| **Soluble microbial products** | $SMP = UAP + BAP$ | (Rittmann, et al., 2001) |

The internal mixed liquor recycle (IMLR) needed for denitrification is typically between 200 and 500% of the average influent flow for a five-stage modified Bardenpho CAS system (WEF, 2012; WEF, 2011; WEF, 2010a). For purposes of this research 200% was used. In an MBR system, return activated sludge (RAS) rates are required to be higher compared to CAS process. For a MBR system, RAS rates are typically 200 to 500% of the average influent flow, versus 50 to 100% in CAS systems (WEF, 2012; WEF, 2006; WEF, 2010a). A MBR system also requires a higher MLSS concentration compared to CAS systems. For a MBR system, the MLSS concentration inside the bioreactor tank can be between 4,000 to 10,000 mg/L and inside the membrane tank 8,000 to 18,000 mg/L, versus 1,500 to 3,500 mg/L in CAS systems (WEF, 2012; WEF, 2006; WEF, 2010a). Energy consumption for the activated sludge process is driven by mixers used to maintain particles suspension in the anaerobic and anoxic zones of the biological nutrient removal system, and blowers used to provide oxygen and particle suspension in the aerated zones. In addition, energy is required to operate the IMLR pumps and RAS pumps. Mixer energy requirement was determined based on the basin volume and the type of mixer. For horizontal mixers the required energy used was 7 $W/m^3$ (WEF, 2010a). Blower energy was determined using the blower equation and a combined blower and motor efficiency of 80% (Metcalf & Eddy, Inc, 2003; Davis, 2010). Energy requirements for pumps after they have been sized were determined as (Jones, et al., 2008):

$$BHP = \frac{qH}{3960E_p}$$

where $BHP$ = brake horsepower, hp; $q$ = required flow rate, gal/min; H = total dynamic head, ft; and $E_p$ = pump efficiency. Efficiencies for both the IMLR and RAS pumps were chosen in ranges from pump data and curves. A pump efficiency of 80% was used for both pumps (Goulds Pumps, 2012). Table A 9, Table A 10, and Table A 11 show the design for the anaerobic, anoxic, and aerobic tanks for the 8.8 MGD MBR WRP facility, respectively. Table A 12, Table A 13, and Table A 14 show the design for the anaerobic, anoxic, and aerobic tanks for the 8.8 MGD CAS WRP facility, respectively.

**Table A 9– Anaerobic Tank Design at 8.8 MGD for MBR WRP**

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| **Flow Parameters** | | |
| Maximum Month | 2.2 | MGD |
| Maximum Month | 3.4034 | cfs |
| Flow Type | Complete Mix Assumed | |
| 4 Trains | | |
| **Tank Sizing** | | |
| Length | 24 | ft |
| Width | 26 | ft |
| Depth | 19.1 | ft |
| Volume | 11918.4 | $ft^3$ |
| **Influent Parameters** | | |
| Assume 50% BOD Removal in Fine Screens | 0.5 | |
| Assume 80% TSS Removal in Fine Screens | 0.8 | |
| VFA | 43 | mg/L |
| TSS | 61.8 | mg/L |
| BOD | 125 | mg/L |
| $BOD_L$ | 187.5 | mg/L |
| Acetate to COD Conversion | 1.048 | g COD/g Acetate |
| **Microbiological Parameters – PAOs** | | |
| K | 1 | mg VFA/L |
| Y | 0.3 | g VSS/g VFA |
| $\hat{q}$ | 3.17 | g VFA/g VSS-day |
| $\hat{\mu}$ | 0.95 | g VSS/g VSS-day |
| b | 0.04 | g VSS/g VSS-day |
| $f_d$ | 0.8 | |
| **Anaerobic Zone Design** | | |
| SRT | 10 | days |
| HRT | 0.041 | days |
| HRT | 58 | minutes |
| $\mathbf{S_{min}}$ | 0.044 | mg VFA/L |
| S | -1.0400 | mg VFA/L |
| $X_{a\text{ Generated In An Zone}}$ | 9.2049 | mg VSS/L |
| $X_{i\text{ Generated In An Zone}}$ | 0.0104 | mg VSS/L |
| VFAs Remaining | 0.044 | mg VFA/L |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| VFAs Remaining to $BOD_L$ | 0.046 | mg $BOD_L$/L |
| VFAs Removed | 42.956 | mg VFA/L |
| VFAs Removed to $BOD_L$ | 45.02 | mg $BOD_L$/L |
| $BOD_L$ to Anoxic Zone | 142 | mg $BOD_L$/L |

**Table A 10 – Anoxic Tank Design at 8.8 MGD for MBR WRP**

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| **Flow Parameters** | | |
| Maximum Month | 2.2 | MGD |
| Maximum Month | 3.4034 | cfs |
| Flow Regime | Complete Mix Assumed | |
| 4 Trains | | |
| **Tank Sizing** | | |
| Length | 24 | ft |
| Width | 26 | ft |
| Depth | 19.1 | ft |
| Volume | 11918.4 | $ft^3$ |
| **Influent Parameters** | | |
| TSS | 102 | mg/L |
| BOD | 95 | mg/L |
| $BOD_L$ | 142 | mg/L |
| **Microbiological Parameters – Denitrifiers (Pseudomonas)** | | |
| K | 12.6 | mg $BOD_L$/L |
| Y | 0.26 | g VSS/g $BOD_L$ |
| $\hat{q}$ | 12.00 | g $BOD_L$/g VSS-day |
| $\hat{\mu}$ | 3.12 | g VSS/g VSS-day |
| b | 0.05 | g VSS/g VSS-day |
| $f_d$ | 0.8 | |
| **Anoxic Zone Design** | | |
| $S^o$ | 36 | mg $NO_3^-$-N/L |
| SRT | 10 | days |
| HRT | 0.041 | days |
| HRT | 58 | minutes |
| IR Actual | 2 | N/A |
| Estimated $N_e$ For IR | 4.95 | mg $NO_3^-$-N/L |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| IR Equation | 2.17 | N/A |
| $X_b$ | 1805.9 | mg/L |
| Flow Rate to Anoxic Tank | 49967 | $m^3$/day |
| $NO_3$ Estimate for $NO_X$ Feed | 5.80 | mg $NO_3^-$-N/L |
| $NO_X$ Feed | 289811 | g $NO_3^-$-N/day |
| $V_{nox}$ | 337.49 | $m^3$ |
| $F/M_b$ | 1.30 | g/g-day |
| Assumed rbCOD/COD | 0.30 | N/A |
| SDNR | 0.34 | g $NO_3^-$-N/g VSS-day |
| $SDNR_{adj}$ | 0.32 | g $NO_3^-$-N/g VSS-day |
| $NO_r$ | 195303 | g $NO_3^-$-N/day |
| SDNR (MLSS) | 0.20 | g/g-day |
| $BOD_L$ Consumed | 110.0 | mg/L |
| $X_a$ | 1343.8 | mg VSS/L |
| $X_i$ | 134.4 | mg VSS/L |
| $X_v$ | 1478.2 | mg VSS/L |
| $\Delta \bar{X}_V / \Delta t$ | 47.2 | kg VSS/day |
| Oxygen Credit | 708 | kg/d |
| Oxygen Credit | 29.5 | kg/hour |
| Alkalinity Produced | 106.1 | mg/L as CaCO3 |
| Phosphorous Removed | 0.11 | mg P/L |

**Table A 11 – Aerobic Tank Design at 8.8 MGD for MBR WRP**

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| **Flow Parameters** | | |
| Maximum Month | 2.2 | MGD |
| Maximum Month | 3.4034 | cfs |
| Maximum Month | 8328 | $m^3$/day |
| Flow Regime | Plug Flow Assumed | |
| 4 Trains | | |
| **Tank Sizing** | | |
| Length | 60 | ft |
| Width | 26 | ft |

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| Depth | 19.1 | ft |
| Volume | 29796 | $ft^3$ |
| **Influent Parameters** | | |
| TSS | 102 | mg/L |
| Assumed Volatile Portion of TSS | 0.81 | NA |
| VSS | 82.62 | mg/L |
| BOD | 95 | mg/L |
| $BOD_L$ | 142 | mg/L |
| TKN | 42 | mg/L |
| P | 4.68 | mg/L |
| **BOD Microbiological Parameters – Heterotrophic Microorganisms** | | |
| K | 10 | mg BODL/L |
| Y | 0.45 | mg VSS/mg $BOD_L$ |
| $\hat{q}$ | 20.00 | mg BODL/mg VSS |
| $\hat{\mu}$ | 9 | mg VSS/mg VSS-day |
| b | 0.15 | mg VSS/mg VSS-day |
| $f_d$ | 0.8 | |
| $[\theta_x^{min}]_{lim}$ | 0.113 | day |
| **Nitrification Microbiological Parameters – Nitrosomonas (Ammonia Donor)** | | |
| K | 1 | mg $NH_4^+$-N/L |
| Y | 0.33 | g VSS/g $NH_4^+$-N |
| $\hat{q}$ | 2.30 | g $NH_4^+$-N/g VSS-day |
| $\hat{\mu}$ | 0.76 | g VSS/g VSS-day |
| b | 0.11 | g VSS/g VSS-day |
| $f_d$ | 0.8 | |
| $[\theta_x^{min}]_{lim}$ | 1.541 | day |
| **Nitrification Microbiological Parameters – Nitrobacter (Nitrite Donor)** | | |
| K | 1.3 | mg $NO_2^-$ -N/L |
| Y | 0.083 | g VSS/g $NO_2^-$ -N |
| $\hat{q}$ | 9.80 | g $NO_2^-$ -N/g VSS-day |
| $\hat{\mu}$ | 0.81 | g VSS /g VSS-day |
| b | 0.11 | g VSS/g VSS-day |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| $f_d$ | 0.8 | |
| $[\theta_x^{min}]_{lim}$ | 1.422 | day |
| **Effluent Criteria** | | |
| BOD | 30 | mg BOD/L |
| TSS | 30 | mg TSS/L |
| Ammonia | 0.4 | mg $NH_4^+$-N/L |
| Total P | 0.2 | mg P/L |
| Total N | 10 | mg N/L |
| **Operational Assumptions** | | |
| SRT | 10 | day |
| R[1] | 4 | N/A |
| $\theta$ | 0.10 | day |
| $\theta_r$ | 0.02 | day |
| $\theta_{totalsystem}$ | 0.14 | day |
| **BOD** | | |
| Left side of Equation 5.54, 5.55, 5.57 | 0.100 | $day^{-1}$ |
| S | 1.00E-200 | mg $BOD_L$/L |
| e for Equation 5.55 | 2319.3 | |
| Right Side of Equation 5.55 | -0.10 | $day^{-1}$ |
| Right Side of Equation 5.57 | 0.12 | $day^{-1}$ |
| $S_{min}$ | 0.17 | mg $BOD_L$/L |
| $S^i$ | 29 | mg $BOD_L$/L |
| $\bar{X}_a$ | 1805.9 | mg VSS/L |
| Right Side of Equation 5.54 | 0.20 | $day^{-1}$ |
| $\theta_r$ left side of Equation 5.53 | 0.02 | day |
| Right side of Equation 5.53 | 0.13 | day |
| $\bar{X}_i$ | 386.94 | mg VSS/L |
| $\bar{X}_v$ | 2192.9 | mg VSS/L |
| $\Delta\bar{X}_v/\Delta t$ | 277.3 | kg VSS/day |
| Ammonia Removed by BOD Bacteria | 4.1 | mg $NH_4^+$-N/L |
| Phosphorous removed by BOD Bacteria | 0.67 | mg P/L |
| **Nitrosomonas** | | |
| $S^0$ | 37.9 | mg $NH_4^+$-N/L |
| Left side of Equation 5.54, 5.55, 5.57 | 0.100 | $day^{-1}$ |

| *Parameter* | *Value* | *Unit* |
| --- | --- | --- |
| S | 1.00E-16 | mg $NH_4^+$-N/L |
| e for Equation 5.55 | 194.3 | |
| Right Side of Equation 5.55 | 0.01 | $day^{-1}$ |
| Right Side of Equation 5.57 | 0.23 | $day^{-1}$ |
| $S_{min}$ | 0.17 | mg $NH_4^+$-N/L |
| $S^i$ | 8 | mg $NH_4^+$-N/L |
| $\bar{X}_a$ | 417.7 | mg VSS/L |
| Right Side of Equation 5.54 | 0.18 | $day^{-1}$ |
| $\theta_r$ left side of Equation 5.53 | 0.02 | day |
| Right side of Equation 5.53 | 0.05 | day |
| $\bar{X}_i$ | 65.6 | mg VSS/L |
| $\bar{X}_v$ | 483.3 | mg VSS/L |
| $\Delta\bar{X}_v/\Delta t$ | 60.19 | kg VSS/day |
| Nitrite Removed by Ammonia Bacteria | 0.9 | mg $NO_2^-$-N/L |
| Phosphorous removed by Ammonia Bacteria | 0.14 | mg P/L |
| **Nitrobacter** | | |
| $S^0$ | 37.0 | mg $NO_2^-$-N/L |
| Left side of Equation 5.54, 5.55, 5.57 | 0.100 | $day^{-1}$ |
| S | 1.00E-10 | mg $NO_2^-$-N/L |
| e for Equation 5.55 | 125.1 | |
| Right Side of Equation 5.55 | 0.04 | $day^{-1}$ |
| Right Side of Equation 5.57 | 0.31 | $day^{-1}$ |
| $S_{min}$ | 0.20 | mg $BOD_L$/L |
| $S^i$ | 8 | mg $NO_2^-$-N/L |
| $\bar{X}_a$ | 102.5 | mg VSS/L |
| Right Side of Equation 5.54 | 0.18 | $day^{-1}$ |
| $\theta_r$ (left side of Equation 5.53) | 0.02 | day |
| Right side of Equation 5.53 | 0.04 | day |
| $\bar{X}_i$ | 16.1 | mg VSS/L |
| $\bar{X}_v$ | 118.6 | mg VSS/L |
| $\Delta\bar{X}_v/\Delta t$ | 14.77 | kg VSS/day |
| Nitrate Removed by Nitrite Bacteria | 0.2 | mg $NO_3^-$-N/L |
| Phosphorous removed by Nitrite Bacteria | 0.04 | mg P/L |
| **PAOs** | | |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| $\bar{X}_a$ – aerobic zone due to recycle | 649 | mg VSS/L |
| $\bar{X}_i$ – aerobic zone due to recycle | 51.9 | mg VSS/L |
| $\bar{X}_v$ | 700.9 | mg VSS/L |
| $\Delta\bar{X}_v/\Delta t$ | 82.8 | kg VSS/day |
| Phosphorous removed by PAO Bacteria | 2.98 | mg P/L |
| Nitrate Removed by PAO Bacteria | 1.2 | mg $NO_3^-$-N/L |
| **Effluent** | | |
| Phosphorous Effluent Estimated (Rittmann, et al., 2001) | 0.74 | mg P/L |
| Total Influent COD | 250.00 | mg COD/L |
| $f_{us}$ | 0.05 | mg/mg COD |
| $f_{up}$ | 0.13 | mg/mg COD |
| $f_{cv}$ | 1.48 | mg COD/mg VSS |
| $b_{hT}$ | 0.24 | g VSS/g VSS-day |
| SRT | 10.00 | days |
| $Y_h$ | 0.45 | mg VSS/mg COD |
| $\gamma$ | -0.21 | mg P/mg VSS |
| $P_f$ | 2.70 | |
| $S_{bsa}$ | 43.00 | mg/L |
| $f_{xa}$ | 0.15 | |
| $f_p$ | 0.015 | mg P/mg VSS |
| $f$ | 0.2 | mg/mg VSS |
| Part 1 of Equation 15.15 (WEF, 2010a) | -2.2E-02 | |
| Part 2 of Equation 15.15 | 1.3E-03 | |
| Phosphorous Removal Estimated by Equation 15.15 | -5.10 | mg P/L |
| Effluent P by Equation 15.15 | 0 | mg P/L |
| P Removal by Figure 15.45 (WEF, 2010a) | 0.017 | mg P/mg COD |
| P Removal by Figure 15.45 | 4.250 | mg P/L |
| Effluent P by Figure 15.45 | 0 | mg P/L |
| Reactive Nitrate In Influent for P Reactor | 1.208 | mg/L |
| rbCOD/nitrate ratio | 6.6 | g rbCOD/g Nitrate |
| rbCOD (VFA) | 43 | mg/L |
| rbCOD Equivalent | 7.98 | mg/L |
| rbCOD available for P Removal | 35.03 | mg/L |
| rbCOD/P Ratio | 10 | g rbCOD/g P |
| Biological P Removal | 3.50 | mg/L |
| P Removal by other Bacteria | 0.96 | mg/L |

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| Total P Removal | 4.46 | mg/L |
| P in Effluent | 0.22 | mg P/L |
| Inert VSS pass through | 19.38 | mg VSS/L |
| Inert VSS Recycled pass through | 1366 | mg VSS/L |
| $\Delta\bar{X}_i/\Delta t$ pass through | 161.4 | kg VSSi/day |
| MLSS Total | 6340.1 | mg TSS/L |
| **Oxygen Requirements** | | |
| Input $O_2$ Requirements | 3014 | kg OD/day |
| Soluble Output $O_2$ Equivalents | 9.79 | kg OD/day |
| Solid Output $O_2$ Equivalents | 926.7 | kg OD/day |
| Oxygen Requirements | 2077.71 | kg OD/day |
| Oxygen Requirements | 86.57 | kg OD/hour |
| **Oxygen Requirements w/Oxygen Credit** | | |
| Input $O_2$ Requirements | 2306 | kg OD/day |
| Soluble Output $O_2$ Equivalents | 9.79 | kg OD/day |
| Solid Output $O_2$ Equivalents | 926.7 | kg OD/day |
| Oxygen Requirements | 1369.80 | kg OD/day |
| Oxygen Requirements | 57.08 | kg OD/hour |
| **Fine Bubble Diffuser Design** | | |
| $C_{20}$ | 9.08 | mg/L |
| $P_b/P_a$ | 0.93 | N/A |
| $C_{s,T,H}$ | 8.46 | mg/L |
| $P_{atm,H}$ | 9.64 | m |
| Diffuser Height From Bottom | 0.610 | m |
| Tank Height | 5.82 | m |
| Assumed Oxygen Transfer Efficiency | 0.19 | N/A |
| DO In Aeration Basin | 2 | mg/L |
| $C_{s,T,H}$ | 10.34 | mg/L |
| α | 0.5 | N/A |
| β | 0.95 | N/A |
| F | 0.9 | N/A |
| SOTR | 228.7 | kg/hour |
| Assumed Efficiency | 0.35 | N/A |
| Air Density | 1.204 | $kg/m^3$ |
| Air Flowrate | 2419.6 | $m^3$/hour |
| Air Flowrate | 40.3 | $m^3$/minute |
| Air Flowrate | 1424 | $ft^3$/minute |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| Required Blower Capacity Assuming 2 Safety Factor | 2848.7 | $ft^3$/minute |
| **Alkalinity Requirements** | | |
| Alkalinity Consumed | 299.88 | mg/L as CaCO3 |
| Alkalinity Residual for pH 6.8-7 | 80 | mg/L as CaCO3 |
| Alkalinity Provided in CABI | 271 | mg/L as CaCO3 |
| Alkalinity Required w/o Denitrification | 108.88 | mg/L as CaCO3 |
| Alkalinity Required w/Denitrification | 2.77 | mg/L as CaCO3 |
| **Internal Recycle Pump Sizing** | | |
| Internal Recycle Pump Sizing | 4.4 | MGD |
| Internal Recycle Pump Sizing | 3056 | gpm |
| Total Dynamic Head | 25 | ft |
| Pump Efficiency | 0.8 | N/A |
| Brake Horsepower | 25 | Hp |
| **Return Activated Sludge Pump Sizing** | | |
| Return Activated Sludge Pump Sizing | 8.8 | MGD |
| Return Activated Sludge Pump Sizing | 6111 | gpm |
| Total Dynamic Head | 18 | ft |
| Pump Efficiency | 0.8 | N/A |
| Brake Horsepower | 35 | Hp |
| **Horizontal Mixer Sizing** | | |
| Energy Requirement per Mixer | 7.0 | W/$m^3$ |
| Number of Mixers | 3 | N/A |
| **Blower Sizing** | | |
| Required Blower Capacity | 2848.7 | $ft^3$/min |
| Static Head | 8.280 | psi |
| Diffuser Headloss | 0.70 | psi |
| Piping Headloss | 0.15 | psi |
| Inlet Valve and Filter Headloss | 0.30 | psi |
| System Head | 9.430 | psig |
| Atmospheric Pressure (2000 ft) | 13.779 | psia |
| Discharge Pressure | 23.209 | psia |
| Efficiency (blower & motor) | 0.8 | N/A |
| Brake Horsepower | 128 | Hp |
| **Energy Requirement** | | |
| Energy Required from Internal Recycle Pump | 18.65 | kW |
| | 447.6 | kWh/day |
| Energy Required from Return Activated Sludge Pump | 26.11 | kW |
| | 626.64 | kWh/day |
| Energy Required from Mixers | 170.096 | kWh/day |

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| Energy Required from Blower | 95.488 | kW |
| | 2291.712 | kWh/day |
| **Total Energy Consumption** | 3536.048 | kWh/day |

**Table A 12 – Anaerobic Tank Design at 8.8 MGD for CAS WRP**

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| **Flow Parameters** | | |
| Maximum Month | 2.2 | MGD |
| Maximum Month | 3.4034 | cfs |
| Flow Type | Complete Mix Assumed | |
| 4 Trains | | |
| **Tank Sizing** | | |
| Length | 24 | ft |
| Width | 26 | ft |
| Depth | 19.1 | ft |
| Volume | 11918.4 | $ft^3$ |
| **Influent Parameters** | | |
| Assume 50% BOD Removal in Fine Screens | 0.5 | |
| Assume 80% TSS Removal in Fine Screens | 0.8 | |
| VFA | 43 | mg/L |
| TSS | 61.8 | mg/L |
| BOD | 125 | mg/L |
| $BOD_L$ | 187.5 | mg/L |
| Acetate to COD Conversion | 1.048 | g COD/g Acetate |
| **Microbiological Parameters – PAOs** | | |
| K | 1 | mg VFA/L |
| Y | 0.3 | g VSS/g VFA |
| $\hat{q}$ | 3.17 | g VFA/g VSS-day |
| $\hat{\mu}$ | 0.95 | g VSS/g VSS-day |
| b | 0.04 | g VSS/g VSS-day |
| $f_d$ $\hat{q}$ | 0.8 | |
| **Anaerobic Zone Design** | | |
| SRT | 10 | days |
| HRT | 0.041 | days |
| HRT | 58 | minutes |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| $\mathbf{S_{min}}$ | 0.044 | mg VFA/L |
| S | -1.0400 | mg VFA/L |
| $X_{a\ \text{Generated In An Zone}}$ | 9.2049 | mg VSS/L |
| $X_{i\ \text{Generated In An Zone}}$ | 0.0104 | mg VSS/L |
| VFAs Remaining | 0.044 | mg VFA/L |
| VFAs Remaining to $BOD_L$ | 0.046 | mg $BOD_L$/L |
| VFAs Removed | 42.956 | mg VFA/L |
| VFAs Removed to $BOD_L$ | 45.02 | mg $BOD_L$/L |
| $BOD_L$ to Anoxic Zone | 142 | mg $BOD_L$/L |

**Table A 13 – Anoxic Tank Design at 8.8 MGD for CAS WRP**

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| **Flow Parameters** | | |
| Maximum Month | 2.2 | MGD |
| Maximum Month | 3.4034 | cfs |
| Flow Regime | Complete Mix Assumed | |
| 4 Trains | | |
| **Tank Sizing** | | |
| Length | 24 | ft |
| Width | 26 | ft |
| Depth | 19.1 | ft |
| Volume | 11918.4 | $ft^3$ |
| **Influent Parameters** | | |
| TSS | 102 | mg/L |
| BOD | 95 | mg/L |
| $BOD_L$ | 142 | mg/L |
| **Microbiological Parameters – Denitrifiers (Pseudomonas)** | | |
| K | 12.6 | mg $BOD_L$/L |
| Y | 0.26 | g VSS/g $BOD_L$ |
| $\hat{q}$ | 12.00 | g $BOD_L$/g VSS-day |
| $\hat{\mu}$ | 3.12 | g VSS/g VSS-day |
| b | 0.05 | g VSS/g VSS-day |
| $f_d$ | 0.8 | |
| **Anoxic Zone Design** | | |
| $S^o$ | 36 | mg $NO_3^-$-N/L |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| SRT | 10 | days |
| HRT | 0.041 | days |
| HRT | 58 | minutes |
| IR Actual | 2 | N/A |
| Estimated $N_e$ For IR | 4.95 | mg $NO_3^-$-N/L |
| IR Equation | 5.17 | N/A |
| $X_b$ | 1053.5 | mg/L |
| Flow Rate to Anoxic Tank | 24984 | $m^3$/day |
| $NO_3$ Estimate for $NO_X$ Feed | 5.80 | mg $NO_3^-$-N/L |
| $NO_X$ Feed | 144906 | g $NO_3^-$-N/day |
| $V_{nox}$ | 337.49 | $m^3$ |
| $F/M_b$ | 2.22 | g/g-day |
| Assumed rbCOD/COD | 0.30 | N/A |
| SDNR | 0.34 | g $NO_3^-$-N/g VSS-day |
| $SDNR_{adj}$ | 0.30 | g $NO_3^-$-N/g VSS-day |
| $NO_r$ | 108375 | g $NO_3^-$-N/day |
| SDNR (MLSS) | 0.12 | g/g-day |
| $BOD_L$ Consumed | 110.0 | mg/L |
| $X_a$ | 783.9 | mg VSS/L |
| $X_i$ | 78.4 | mg VSS/L |
| $X_v$ | 862.3 | mg VSS/L |
| $\Delta\bar{X}_V/\Delta t$ | 47.2 | kg VSS/day |
| Oxygen Credit | 708 | kg/d |
| Oxygen Credit | 29.5 | kg/hour |
| Alkalinity Produced | 106.1 | mg/L as CaCO3 |
| Phosphorous Removed | 0.11 | mg P/L |

**Table A 14 – Aerobic Tank Design at 8.8 MGD for CAS WRP**

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| **Flow Parameters** | | |
| Maximum Month | 2.2 | MGD |
| Maximum Month | 3.4034 | cfs |

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| Maximum Month | 8328 | $m^3$/day |
| Flow Regime | Plug Flow Assumed | |
| 4 Trains | | |
| **Tank Sizing** | | |
| Length | 120 | ft |
| Width | 26 | ft |
| Depth | 19.1 | ft |
| Volume | 59592 | $ft^3$ |
| **Influent Parameters** | | |
| TSS | 102 | mg/L |
| Assumed Volatile Portion of TSS | 0.81 | NA |
| VSS | 82.62 | mg/L |
| BOD | 95 | mg/L |
| $BOD_L$ | 142 | mg/L |
| TKN | 42 | mg/L |
| P | 4.68 | mg/L |
| **BOD Microbiological Parameters – Heterotrophic Microorganisms** | | |
| K | 10 | mg BODL/L |
| Y | 0.45 | mg VSS/mg $BOD_L$ |
| $\hat{q}$ | 20.00 | mg BODL/mg VSS |
| $\hat{\mu}$ | 9 | mg VSS/mg VSS-day |
| b | 0.15 | mg VSS/mg VSS-day |
| $f_d$ | 0.8 | |
| $[\theta_x^{min}]_{lim}$ | 0.113 | day |
| **Nitrification Microbiological Parameters – Nitrosomonas (Ammonia Donor)** | | |
| K | 1 | mg $NH_4^+$-N/L |
| Y | 0.33 | g VSS/g $NH_4^+$-N |
| $\hat{q}$ | 2.30 | g $NH_4^+$-N/g VSS-day |
| $\hat{\mu}$ | 0.76 | g VSS/g VSS-day |
| b | 0.11 | g VSS/g VSS-day |
| $f_d$ | 0.8 | |
| $[\theta_x^{min}]_{lim}$ | 1.541 | day |
| **Nitrification Microbiological Parameters – Nitrobacter (Nitrite Donor)** | | |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| K | 1.3 | mg $NO_2^-$-N/L |
| Y | 0.083 | g VSS/g $NO_2^-$-N |
| $\hat{q}$ | 9.80 | g $NO_2^-$-N/g VSS-day |
| $\hat{\mu}$ | 0.81 | g VSS /g VSS-day |
| b | 0.11 | g VSS/g VSS-day |
| $f_d$ | 0.8 | |
| $[\theta_x^{min}]_{lim}$ | 1.422 | day |
| **Effluent Criteria** | | |
| BOD | 30 | mg BOD/L |
| TSS | 30 | mg TSS/L |
| Ammonia | 0.4 | mg $NH_4^+$-N/L |
| Total P | 0.2 | mg P/L |
| Total N | 10 | mg N/L |
| **Operational Assumptions** | | |
| SRT | 10 | day |
| R[1] | 1 | N/A |
| θ | 0.20 | day |
| $\theta_r$ | 0.10 | day |
| $\theta_{totalsystem}$ | 0.24 | day |
| **BOD** | | |
| Left side of Equation 5.54, 5.55, 5.57 | 0.100 | $day^{-1}$ |
| S | 1.00E-200 | mg $BOD_L$/L |
| e for Equation 5.55 | 929.6 | |
| Right Side of Equation 5.55 | -0.01 | $day^{-1}$ |
| Right Side of Equation 5.57 | 0.12 | $day^{-1}$ |
| $S_{min}$ | 0.17 | mg $BOD_L$/L |
| $S^i$ | 71 | mg $BOD_L$/L |
| $\bar{X}_a$ | 1053.5 | mg VSS/L |
| Right Side of Equation 5.54 | 0.15 | $day^{-1}$ |
| $\theta_r$ left side of Equation 5.53 | 0.10 | day |
| Right side of Equation 5.53 | 0.22 | day |
| $\bar{X}_i$ | 263.36 | mg VSS/L |
| $\bar{X}_v$ | 1316.9 | mg VSS/L |
| $\Delta\bar{X}_v/\Delta t$ | 277.3 | kg VSS/day |

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| Ammonia Removed by BOD Bacteria | 4.1 | mg $NH_4^+$-N/L |
| Phosphorous removed by BOD Bacteria | 0.67 | mg P/L |
| **Nitrosomonas** | | |
| $S^0$ | 37.9 | mg $NH_4^+$-N/L |
| Left side of Equation 5.54, 5.55, 5.57 | 0.100 | $day^{-1}$ |
| S | 1.00E-16 | mg $NH_4^+$-N/L |
| e for Equation 5.55 | 79.6 | |
| Right Side of Equation 5.55 | 0.13 | $day^{-1}$ |
| Right Side of Equation 5.57 | 0.23 | $day^{-1}$ |
| $S_{min}$ | 0.17 | mg $NH_4^+$-N/L |
| $S^i$ | 19 | mg $NH_4^+$-N/L |
| $\bar{X}_a$ | 243.7 | mg VSS/L |
| Right Side of Equation 5.54 | 0.14 | $day^{-1}$ |
| $\theta_r$ left side of Equation 5.53 | 0.10 | day |
| Right side of Equation 5.53 | 0.10 | day |
| $\bar{X}_i$ | 44.7 | mg VSS/L |
| $\bar{X}_v$ | 288.3 | mg VSS/L |
| $\Delta\bar{X}_v/\Delta t$ | 60.19 | kg VSS/day |
| Nitrite Removed by Ammonia Bacteria | 0.9 | mg $NO_2^-$-N/L |
| Phosphorous removed by Ammonia Bacteria | 0.14 | mg P/L |
| **Nitrobacter** | | |
| $S^0$ | 37.0 | mg $NO_2^-$-N/L |
| Left side of Equation 5.54, 5.55, 5.57 | 0.100 | $day^{-1}$ |
| S | 1.00E-10 | mg $NO_2^-$-N/L |
| e for Equation 5.55 | 51.9 | |
| Right Side of Equation 5.55 | 0.18 | $day^{-1}$ |
| Right Side of Equation 5.57 | 0.31 | $day^{-1}$ |
| $S_{min}$ | 0.20 | mg $BOD_L$/L |
| $S^i$ | 19 | mg $NO_2^-$-N/L |
| $\bar{X}_a$ | 59.8 | mg VSS/L |
| Right Side of Equation 5.54 | 0.14 | $day^{-1}$ |
| $\theta_r$ (left side of Equation 5.53) | 0.10 | day |
| Right side of Equation 5.53 | 0.09 | day |
| $\bar{X}_i$ | 11.0 | mg VSS/L |

| Parameter | Value | Unit |
|---|---|---|
| $\bar{X}_v$ | 70.7 | mg VSS/L |
| $\Delta\bar{X}_v/\Delta t$ | 14.77 | kg VSS/day |
| Nitrate Removed by Nitrite Bacteria | 0.2 | mg $NO_3^-$-N/L |
| Phosphorous removed by Nitrite Bacteria | 0.04 | mg P/L |
| **PAOs** | | |
| $\bar{X}_a$ – aerobic zone due to recycle | 379 | mg VSS/L |
| $\bar{X}_i$ – aerobic zone due to recycle | 30.3 | mg VSS/L |
| $\bar{X}_v$ | 408.9 | mg VSS/L |
| $\Delta\bar{X}_v/\Delta t$ | 82.8 | kg VSS/day |
| Phosphorous removed by PAO Bacteria | 2.98 | mg P/L |
| Nitrate Removed by PAO Bacteria | 1.2 | mg $NO_3^-$-N/L |
| **Effluent** | | |
| Phosphorous Effluent Estimated (Rittmann, et al., 2001) | 0.74 | mg P/L |
| Total Influent COD | 250.00 | mg COD/L |
| $f_{us}$ | 0.05 | mg/mg COD |
| $f_{up}$ | 0.13 | mg/mg COD |
| $f_{cv}$ | 1.48 | mg COD/mg VSS |
| $b_{hT}$ | 0.24 | g VSS/g VSS-day |
| SRT | 10.00 | days |
| $Y_h$ | 0.45 | mg VSS/mg COD |
| $\gamma$ | -0.21 | mg P/mg VSS |
| $P_f$ | 2.70 | |
| $S_{bsa}$ | 43.00 | mg/L |
| $f_{xa}$ | 0.15 | |
| $f_p$ | 0.015 | mg P/mg VSS |
| $f$ | 0.2 | mg/mg VSS |
| Part 1 of Equation 15.15 (WEF, 2010a) | -2.2E-02 | |
| Part 2 of Equation 15.15 | 1.3E-03 | |
| Phosphorous Removal Estimated by Equation 15.15 | -5.10 | mg P/L |
| Effluent P by Equation 15.15 | 0 | mg P/L |
| P Removal by Figure 15.45 (WEF, 2010a) | 0.017 | mg P/mg COD |
| P Removal by Figure 15.45 | 4.250 | mg P/L |
| Effluent P by Figure 15.45 | 0 | mg P/L |
| Reactive Nitrate In Influent for P Reactor | 1.208 | mg/L |
| rbCOD/nitrate ratio | 6.6 | g rbCOD/g Nitrate |
| rbCOD (VFA) | 43 | mg/L |

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| rbCOD Equivalent | 7.98 | mg/L |
| rbCOD available for P Removal | 35.03 | mg/L |
| rbCOD/P Ratio | 10 | g rbCOD/g P |
| Biological P Removal | 3.50 | mg/L |
| P Removal by other Bacteria | 0.96 | mg/L |
| Total P Removal | 4.46 | mg/L |
| P in Effluent | 0.22 | mg P/L |
| Inert VSS pass through | 19.38 | mg VSS/L |
| Inert VSS Recycled pass through | 797 | mg VSS/L |
| $\Delta\bar{X}_i/\Delta t$ pass through | 161.4 | kg VSSi/day |
| MLSS Total | 3744.1 | mg TSS/L |
| **Oxygen Requirements** | | |
| Input $O_2$ Requirements | 3014 | kg OD/day |
| Soluble Output $O_2$ Equivalents | 9.79 | kg OD/day |
| Solid Output $O_2$ Equivalents | 926.7 | kg OD/day |
| Oxygen Requirements | 2077.71 | kg OD/day |
| Oxygen Requirements | 86.57 | kg OD/hour |
| **Oxygen Requirements w/Oxygen Credit** | | |
| Input $O_2$ Requirements | 2306 | kg OD/day |
| Soluble Output $O_2$ Equivalents | 9.79 | kg OD/day |
| Solid Output $O_2$ Equivalents | 926.7 | kg OD/day |
| Oxygen Requirements | 1369.80 | kg OD/day |
| Oxygen Requirements | 57.08 | kg OD/hour |
| **Fine Bubble Diffuser Design** | | |
| $C_{20}$ | 9.08 | mg/L |
| $P_b/P_a$ | 0.93 | N/A |
| $C_{s,T,H}$ | 8.46 | mg/L |
| $P_{atm,H}$ | 9.64 | m |
| Diffuser Height From Bottom | 0.610 | m |
| Tank Height | 5.82 | m |
| Assumed Oxygen Transfer Efficiency | 0.19 | N/A |
| DO In Aeration Basin | 2 | mg/L |
| $C_{s,T,H}$ | 10.34 | mg/L |
| $\alpha$ | 0.7 | N/A |
| $\beta$ | 0.95 | N/A |
| F | 0.9 | N/A |
| SOTR | 163.3 | kg/hour |
| Assumed Efficiency | 0.35 | N/A |

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| Air Density | 1.204 | $kg/m^3$ |
| Air Flowrate | 1728.3 | $m^3$/hour |
| Air Flowrate | 28.8 | $m^3$/minute |
| Air Flowrate | 1017 | $ft^3$/minute |
| Required Blower Capacity Assuming 2 Safety Factor | 2034.8 | $ft^3$/minute |
| **Alkalinity Requirements** | | |
| Alkalinity Consumed | 299.88 | mg/L as CaCO3 |
| Alkalinity Residual for pH 6.8-7 | 80 | mg/L as CaCO3 |
| Alkalinity Provided in CABI | 271 | mg/L as CaCO3 |
| Alkalinity Required w/o Denitrification | 108.88 | mg/L as CaCO3 |
| Alkalinity Required w/Denitrification | 2.77 | mg/L as CaCO3 |
| **Internal Recycle Pump Sizing** | | |
| Internal Recycle Pump Sizing | 4.4 | MGD |
| Internal Recycle Pump Sizing | 3056 | gpm |
| Total Dynamic Head | 25 | ft |
| Pump Efficiency | 0.8 | N/A |
| Brake Horsepower | 25 | Hp |
| **Return Activated Sludge Pump Sizing** | | |
| Return Activated Sludge Pump Sizing | 2.75 | MGD |
| Return Activated Sludge Pump Sizing | 1910 | gpm |
| Total Dynamic Head | 30 | ft |
| Pump Efficiency | 0.8 | N/A |
| Brake Horsepower | 19 | Hp |
| **Horizontal Mixer Sizing** | | |
| Energy Requirement per Mixer | 7.0 | $W/m^3$ |
| Number of Mixers | 3 | N/A |
| **Blower Sizing** | | |
| Required Blower Capacity | 2034.8 | $ft^3$/min |
| Static Head | 8.280 | psi |
| Diffuser Headloss | 0.70 | psi |
| Piping Headloss | 0.15 | psi |
| Inlet Valve and Filter Headloss | 0.30 | psi |
| System Head | 9.430 | psig |
| Atmospheric Pressure (2000 ft) | 13.779 | psia |
| Discharge Pressure | 23.209 | psia |
| Efficiency (blower & motor) | 0.8 | N/A |
| Brake Horsepower | 91 | Hp |
| **Energy Requirement** | | |
| Energy Required from Internal Recycle Pump | 18.65 | kW |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| | 447.6 | kWh/day |
| Energy Required from Return Activated Sludge Pump | 14.17 | kW |
| | 340.18 | kWh/day |
| Energy Required from Mixers | 170.096 | kWh/day |
| Energy Required from Blower | 67.886 | kW |
| | 1629.264 | kWh/day |
| **Total Energy Consumption** | 2587.136 | kWh/day |

*A-5 Membranes*

Parameters used in the design of the membrane portion of the MBR system can be found in Table A 15.

**Table A 15 – Membrane Filtration Design Parameters**

| Parameter | Value | Unit | Reference |
|---|---|---|---|
| **RAS recycle ratio** | 400-500 | % | (WEF, 2012) |
| **$MLSS_{membrane\ tank}$** | 8,000-12,000 | mg/L | (WEF, 2012) |
| **Design peak-day flux** | 13.5 | $gal/ft^2/day$ | (Menniti, et al., 2011) |
| **Design average-day flux** | 12.9 | $gal/ft^2/day$ | (Menniti, et al., 2011) |
| **Spare membrane area ratio** | 10 | % | (WEF, 2012) |
| **Membrane area per small subunit** | 32 | $m^2$ | (WEF, 2012) |
| **Number of small subunits per large membrane subunit** | 48 | – | (WEF, 2012) |
| **Volume required for each large subunit** | 20 | $m^3$ | (WEF, 2012) |
| **Air scour rate at average-day flowrate** | 10/30 | seconds on/ seconds off | (WEF, 2012) |
| **Air scour rate at peak-day flowrate** | 10/10 | seconds on/ seconds off | (WEF, 2012) |
| **Online factor including relaxation interval and maintenance cleaning** | 95 | % | (WEF, 2012) |
| **Air scour flowrate at average-day per small subunit** | 10 | $Nm^3/hr$ | (WEF, 2012) |
| **Air scour flowrate at peak-day per small subunit** | 20 | $Nm^3/hr$ | (WEF, 2012) |

To determine the MLSS concentration inside the membrane tank the following equation was used

$MLSS_{membrane\ tank} = (R+1) / R \times MLSS_{bioreactor}$ (WEF, 2012)

where $MLSS_{membrane\ tank}$ = TSS inside the membrane tank; $R$ = RAS recycle ratio; and $MLSS_{bioreactor}$ = TSS inside the bioreactor tank. To determine the required membrane area the following equation was used

$$J = \frac{Q}{A}$$ (WEF, 2012)

where $J$ = design net flux; $Q$ = influent flowrate; and $A$ = membrane area. Typical membrane parameters including membrane area per small subunit, number of small subunits per large subunit, and volume required per large subunits were taken from (WEF, 2012). The air scour cycle rates during average and peak-day flowrates were 10 sec on/30 sec off and 10/10, respectively (WEF, 2012). An online factor of 95% percent was also used to include relaxation intervals and maintenance cleaning (WEF, 2012). Energy consumption for the membranes is driven by air scour blowers, permeate pumps, backpulse pumps, and WAS pumps. The consumption of energy was calculated using the blower and pump equations. The combined and pump efficiencies used for both the blower and WAS pumps, respectively, were 80% (Metcalf & Eddy, Inc, 2003; Davis, 2010), and the pump efficiencies used for permeate and backpulse pumps were 70% (Goulds Pumps, 2012). Table A 16 shows the design for membrane filtration for the 8.8 MGD MBR WRP facility.

**Table A 16 – Membrane Filtration Design at 8.8 MGD**

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| **Membrane and Tank Design Parameters** | | |
| Peak-day flow | 1.2 | MGD/train |
| Number of trains | 8 | |

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| $MLSS_{bioreactor}$ | 6340 | mg TSS/L |
| RAS Recycle Ratio | 400 | % |
| Design net flux | 13.5 | gal/ft$^2$/day |
| Spare membrane area ratio | 10 | % |
| Membrane area per small subunit | 32 | m$^2$ |
| Number of small subunits per large membrane subunit | 48 | |
| Volume required for each large subunit | 20 | m$^3$ |
| Air scour rate at peak-day flowrate | 10/10 | sec on/sec off |
| Online factor | 95 | % |
| Air scour flowrate at peak-day per small subunit | 20 | Nm$^3$/hr |
| **Membrane and Tank Calculations** | | |
| $MLSS_{membrane\ tank}$ | 7925.09 | mg TSS/L |
| Required membrane area | 88889 | ft$^2$ |
| Number of small subunits | 288 | |
| Number of large subunits | 6 | |
| Actual spare membrane area | 11.60 | % |
| Membrane tank volume | 4237.76 | ft$^3$ |
| Blower flowrate | 5760 | Nm$^3$/hr |
| **Blower Sizing** | | |
| Required blower capacity | 3390 | ft$^3$/min |
| Static head | 8.280 | psi |
| Diffuser headloss | 0.70 | psi |
| Piping headloss | 0.15 | psi |
| Inlet valve and filter headloss | 0.30 | psi |
| System head | 9.43 | psig |
| Atmospheric pressure (2000 ft) | 13.779 | psia |
| Discharge pressure | 23.209 | psi |
| Efficiency (blower & motor) | 0.8 | |
| Brake horsepower | 152 | Hp |
| **Permeate Pump Sizing** | | |
| Permeate pump sizing | 1.145 | MGD |
| Permeate pump sizing | 795 | gpm |
| Total dynamic head | 50 | ft |
| Pump efficiency | 0.7 | |
| Brake horsepower | 15 | Hp |
| **Backpulse Pump Sizing** | | |
| Permeate pump sizing | 2.4 | MGD |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| Permeate pump sizing | 1667 | gpm |
| Total dynamic head | 30 | ft |
| Pump efficiency | 0.7 | |
| Brake horsepower | 19 | Hp |
| **Waste Activated Sludge Pump Sizing** | | |
| Waste Activated Sludge Pump Sizing | 0.051 | MGD |
| Waste Activated Sludge Pump Sizing | 36 | gpm |
| Total Dynamic Head | 15 | ft |
| Pump Efficiency | 0.8 | |
| Brake Horsepower | 0.25 | Hp |
| **Energy Requirement** | | |
| Energy required from blower | 113.392 | kW |
| | 646.3344 | kWh/day |
| Energy required from permeate pump | 11.19 | kW |
| | 255.132 | kWh/day |
| Energy required from backpulse pump | 14.174 | kW |
| | 17.0088 | kWh/day |
| Energy Required from WAS Pump | 0.1865 | kW |
| | 4.476 | kWh/day |
| **Total Energy Consumption** | 922.9512 | kWh/day |

### *A-6 Secondary Clarifier*

Parameters used in the design of the secondary clarifier can be found in Table A 17.

**Table A 17 – Secondary Clarifier Design Parameters**

| Parameter | Value | Unit | Reference |
|---|---|---|---|
| **Average overflow rate** | 400-700 | gpd/ft$^2$ | (WEF, 2005; Metcalf & Eddy, Inc, 2003; WEF, 2010a) |
| **Peak overflow rate** | 1,000-1,600 | gpd/ft$^2$ | (WEF, 2005; Metcalf & Eddy, Inc, 2003; WEF, 2010a) |
| **Average solids loading rate** | 20-30 | lb/day·ft$^2$ | (WEF, 2005; Metcalf & Eddy, Inc, 2003; WEF, 2010a) |
| **Peak solid loading rate** | 40-50 | lb/day·ft$^2$ | (WEF, 2005; Metcalf & Eddy, Inc, 2003; WEF, 2010a) |
| **Average weir loading** | <15,000 | gpd/ft | (WEF, 2005) |
| **Peak weir loading** | <30,000 | gpd/ft | (WEF, 2010a) |

To determine the sizing of the clarifier, the area is solved for using both overflow rate and solids loading rate. The overflow rate equation is as followed

$$SOR = \frac{Q + Q_R}{A}$$ (Metcalf & Eddy, Inc, 2003; WEF, 2005; WEF, 2010a)

where $SOR$ = surface over flow rate; $Q$ = influent flowrate; $Q_R$ = RAS flowrate; and $A$ = clarifier cross-sectional area. The solids loading rate equation is as followed

$$SLR = \frac{(Q + Q_R)X}{A}$$ (Metcalf & Eddy, Inc, 2003; WEF, 2005; WEF, 2010a)

where $SLR$ = solids loading rate (solids flux); $Q$ = influent flowrate; $Q_R$ = RAS flowrate; $X$ = MLSS concentration; and $A$ = clarifier cross-sectional area. The two equations are used to solve for both average and peak-flow conditions. The highest value of the four governed the design. Weir loading was checked for during both average and peak flows to ensure the loadings were under legal limits (WEF, 2005; WEF, 2010a). Energy consumption for the secondary clarifier is driven by the size of the motor that provides the torque for the rake arm and the WAS pump. The required power to move the rake arm was calculated using (WEF, 2005):

$$P = T\omega$$

where P = power required by the motor, W; T = required torque, J, $T = Wr^2$ where $W$ = rake arm loading, N/m and $r$ = radius of rake arm, m; and $\omega$ = angular velocity, rad/s. A rake arm loading value of 95 N/m was used and fell within the recommended range for secondary sludge (WEF, 2005). The energy requirement for the WAS pump was determined using equation 2. A pump efficiency of 80% was used (Goulds Pumps, 2012). Table A 18 shows the design for secondary clarifier for the 8.8 MGD CAS WRP facility.

**Table A 18 – Secondary Clarifier Design at 8.8 MGD**

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| **Clarifier Design Parameters** | | |
| Peak day flow | 2.4 | MGD/train |
| Number of trains | 4 | |
| $MLSS_{bioreactor}$ | 3744 | mg TSS/L |
| Average overflow rate | 400-700 | gpd/$ft^2$ |
| Peak overflow rate | 1000-1600 | gpd/$ft^3$ |
| Average solids overflow rate | 20-30 | lb/day·$ft^2$ |
| Peak solids overflow rate | 40-50 | lb/day·$ft^2$ |
| Weir overflow rate | < 15000 | gpd/ft |
| **Clarifier Over Flow Calculations** | | |
| Peak | | |
| Cross-sectional area (from $OF_{flux}$) | 2999 | $ft^2$ |
| Cross-sectional area (from OF) | 3000 | $ft^2$ |
| Average | | |
| Cross-sectional area (from $OF_{flux}$) | 4296 | $ft^2$ |
| Cross-sectional area (from OF) | 5500 | $ft^2$ |
| Sizing Requirements | | |
| Controlling coss-sectional area | 5500 | $ft^2$ |
| Diameter | 84 | ft |
| Manufacturer diameter | 85 | ft |
| Depth | 12 | ft |
| Freeboard | 2 | ft |
| **Solids Loading Calculations** | | |
| Solids loading rate - peak flow | 0.85 | lb/$ft^2$/hr |
| Solids loading rate - average flow | 0.78 | lb/$ft^2$/hr |
| **Weir Design Calculations** | | |
| Center to center spacing | 8 | in |
| Individual weir length | 6 | in |
| Spacing between v-notch | 2 | in |
| Height of v-notch | 3 | in |
| Max wetted perimeter | 8.49 | in |
| Number of v-notches | 401 | |
| Weir length | 283 | ft |
| Weir loading - peak | 8474 | gal/day/ft |
| Weir loading - average | 7767 | gal/day/ft |

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| **Rake Arm Calculations** | | |
| Torque K-value | 95 | N/m |
| Required torque | 15940 | J |
| Alarm torque | 19128 | J |
| Shut-off torque | 22316 | J |
| Failure torque | 31880 | J |
| Typical Peripheral Velocity | 5.5 | m/min |
| Typical Peripheral Velocity | 18 | ft/min |
| Typical Peripheral Velocity | 0.30 | ft/sec |
| Angular Velocity | 0.0071 | rad/sec |
| Time For one Revolution | 14.7979 | min |
| Required Motor Size | 0.1128 | kW |
| Required Motor Size | 0.15 | Hp |
| Motor Size Used | 0.25 | Hp |
| **Waste Activated Sludge Pump Sizing** | | |
| Waste Activated Sludge Pump Sizing | 0.103 | MGD |
| Waste Activated Sludge Pump Sizing | 71 | gpm |
| Total Dynamic Head | 15 | ft |
| Pump Efficiency | 0.8 | |
| Brake Horsepower | 0.5 | Hp |
| **Energy Requirement** | | |
| Energy required from rake arm | 0.1865 | kW |
| | 4.476 | kWh/day |
| Energy required from WAS pump | 0.373 | kW |
| | 8.952 | kWh/day |
| **Total Energy Consumption** | 13.428 | kWh/day |

*A-7 Dual Media Filters*

Parameters used in the design of the dual media filters can be found in Table A 19.

**Table A 19 – Dual Media Filter Design Parameters**

| Parameter | Value | Unit | Reference |
|---|---|---|---|
| **Media Type** | Anthracite and Sand | - | (Metcalf & Eddy, Inc, 2003) |
| **Anthracite Depth** | 360-900 | mm | (Metcalf & Eddy, Inc, 2003) |
| **Anthracite Effective Size** | 0.8-2.0 | mm | (Metcalf & Eddy, Inc, 2003) |
| **Anthracite Uniformity Coefficient** | 1.3-1.6 | - | (Metcalf & Eddy, Inc, 2003) |
| **Sand Depth** | 180-360 | mm | (Metcalf & Eddy, Inc, 2003) |
| **Sand Effective Size** | 0.4-0.8 | mm | (Metcalf & Eddy, Inc, 2003) |
| **Sand Uniformity Coefficient** | 1.2-1.6 | - | (Metcalf & Eddy, Inc, 2003) |
| **Dual Media Filtration Rate** | 0.2 | $m^3/m^2 \cdot min$ | (GLUMRB, 2004) |
| **Dual Media Backwash Rate Needed to Fluidize Bed** | 0.8-1.2 | $m^3/m^2 \cdot min$ | (Metcalf & Eddy, Inc, 2003) |
| **Dual Media Backwash Rate w/Air Scour** | 0.38 | $m^3/m^2 \cdot min$ | (Metcalf & Eddy, Inc, 2003; WEF, 2010a) |
| **Dual Media Backwash Air Flow Rate** | 1.07 | $m^3/m^2 \cdot min$ | (Metcalf & Eddy, Inc, 2003; WEF, 2010a) |

The number and size of the filters were determined using (WEF, 2010a) and the filtration rate (GLUMRB, 2004). Filter sizes were rounded to the nearest increment of 25 square feet to allow for ease of construction. The filters were designed with one filter out of service for backwashing cycles. The cleanwater headlosses were determined to be 0.81 and 1.45 feet for average and peak filtration rates, respectively, using the Rose equation:

$$h_L = \frac{1.067}{\phi} \frac{L v_s^2}{\alpha^4 g} \sum C_d \frac{P}{d_g^2}$$ (Metcalf & Eddy, Inc, 2003)

where $h_L$ = headloss; $\phi$ = particle shape factor; $L$ = depth of filter bed; $v_s$ = superficial filtration velocity; $\alpha$ = porosity; $g$ = gravitational acceleration; $C_d$ = coefficient of drag; $P$ = fraction of particles within adjacent sieve sizes; and $d_g$ = geometric mean diameter between sieve sizes. Backwash cycles were design to be 36 hours, determined using solids holding capacity for clogged headloss determination, Figure 11-10 (Metcalf & Eddy, Inc, 2003; WEF, 2010a). Energy consumption for the dual media filters is driven by the backwash blower and backwash pump. A combined blower and motor efficiency of 80% was used for the backwash blower (Metcalf & Eddy, Inc, 2003) and a pump efficiency of 78% was used for the backwash pump (Goulds Pumps, 2012). Table A 20 shows the design for dual media filters for the 8.8 MGD CAS WRP facility.

**Table A 20 – Dual Media Filter Design at 8.8 MGD**

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| **Flow Parameters** | | |
| Average Flow | 8.8 | MGD |
| Average Flow | 13.62 | $ft^3/sec$ |
| Peak Flow | 9.6 | MGD |
| Peak Flow | 14.85 | $ft^3/sec$ |
| **Design Parameters** | | |
| Minimum number of filters using equation 11-17 of WEF 2010 | 3.72 | |
| Minimum number of filters with one filter out of service | 3.00 | |
| Filtration Rate (10 States Standard) at peak flow | 5 | $gpm/ft^2$ |
| Area per Filter Calculated | 444 | $ft^2$ |
| Number of Filters Assumed | 4 | |
| Number of Filters Assumed with one Filter out of service | 3 | |
| Area per Filter Recalculated | 444 | $ft^2$ |
| Area per Filter Assumed From Above | 500 | $ft^2$ |
| Filter Saftey Factor | 1.125 | |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| Anthracite Depth | 0.56 | m |
| Sand Depth | 0.36 | m |
| Calculated Filtration Rate at Average Flow | 4.07 | $gpm/ft^2$ |
| Calculated Filtration Rate at Average Flow | 10 | m/h |
| Calculated Filtration Rate at Peak Flow | 12 | m/h |
| Cleanwater Headloss at Average Flow | 0.81 | ft |
| Cleanwater Headloss at Peak Flow | 1.45 | ft |
| **Backwash Cycle w/Water** | | |
| Average Backwash Rate | 1.05 | m/min |
| Average Backwash Rate | 25.8 | $gpm/ft^2$ |
| Maximum Backwash Rate | 1.2 | m/min |
| Maximum Backwash Rate | 29.5 | $gpm/ft^2$ |
| Backwash Pump Sizing | 14725 | gpm |
| Backwash Cycle - WEF Operations | 8 | minutes |
| **Backwash Cycle w/Water & Air** | | |
| Air Flow rate | 3.5 | $ft^3/ft^2 \cdot min$ |
| Required Blower Sizing | 1750 | $ft^3/min$ |
| Backwash Rate With Air Scour | 0.38 | m/min |
| Backwash Rate With Air Scour | 0.0064 | m/s |
| Backwash Rate With Air Scour | 9.4 | $gpm/ft^2$ |
| D60 Fluidization Flow - Amirtharajah | 1.05 | m/min |
| D60 Fluidization Flow | 0.0175 | m/s |
| Left Side of Equation for Amirtharajah | 41.9 | |
| Backwash Cycle for Air | 4.0 | min |
| Backwash Cycle for Water | 8.0 | min |
| **Filter Recovery** | | |
| Filtration Rate ($v_F$) | 4.07 | $gpm/ft^2$ |
| Duration of Filter Run ($t_F$) | 28 | hr |
| Duration of Filter Run ($t_F$) | 1680 | min |
| Unit Filter Run Volume (UFRV) | 6844 | $gal/ft^2$ |
| Backwash Rate ($v_{BW}$) | 25.8 | $gpm/ft^2$ |
| Duration of Backwash Cycle ($t_{BW}$) | 8 | min |
| Unit Backwash Volume (UBWV) | 206 | $gal/ft^2$ |
| Duration of Filter-To-Waste Cycle ($t_{BW}$) | 15 | min |
| Unit Filter-To-Waste Cycle (UFWV) | 61 | $gal/ft^2$ |
| Recovery Rate ($r$) = (UFRV-UBWV-UFWV)/(UFRV) - MWH | 96 | % |
| **Backwash Blower Sizing** | | |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| Required Blower Capacity | 1750.0 | $ft^3$/min |
| Static Head | 1.309 | psi |
| Diffuser Headloss | 0.70 | psi |
| Piping Headloss | 0.15 | psi |
| Inlet Valve and Filter Headloss | 5.00 | psi |
| System Head | 7.159 | psig |
| Atmospheric Pressure (2000 ft) | 13.779 | psia |
| Discharge Pressure | 20.938 | psia |
| Efficiency (blower & motor) | 0.8 | |
| Brake Horsepower | 62 | Hp |
| **Backwash Pump Sizing** | | |
| Backwash Pump Sizing | 4688 | gpm |
| Total Dynamic Head | 50 | ft |
| Pump Efficiency | 0.78 | |
| Brake Horsepower | 76 | Hp |
| **Energy Requirement** | | |
| Energy Required from Backwash Blower | 46.252 | kW |
| | 8.22257778 | kWh/day |
| Energy Required from Backwash Pump | 56.696 | kW |
| | 20.1585778 | kWh/day |
| **Total Energy Consumption** | 28.3811556 | kWh/day |

*A-8 UV Disinfection*

Parameters used in the design of the UV disinfection system can be found in Table A 21.

**Table A 21 – UV Disinfection Design Parameters**

| Parameter | Value | | Unit | Reference(s) |
|---|---|---|---|---|
| | *Low Pressure* | *Medium Pressure* | | |
| **Lamp length** | 1.50 | 0.25 | m | |
| **Lamp and sleeve diameter** | 23 | 76 | mm | |
| **Lamp and sleeve area** | 4.15E-4 | 4.54E-3 | $m^2$ | |
| **Lamp spacing (O.C.)** | 102 | 127 | mm | |
| **UV input/output range** | 60-100 | 30-100 | % | (Trojan Technologies, 2007; Trojan Technologies, 2008) |
| **Maximum UV input** | 260 | 3200 | W | (Trojan Technologies, 2007; Trojan Technologies, 2008; Metcalf & Eddy, Inc, 2003) |
| **Maximum UV output** | 85 | 384 | W | |
| **Minimum UV dosage required – Membrane Effluent** | 80 | | $mW{\cdot}s/cm^2$ | (U.S. EPA, 2004a; Metcalf & Eddy, Inc, 2007; WEF, 2010a; NWRI, 2012) |
| **Minimum UV dosage required – Filter Effluent** | 100 | | $mW{\cdot}s/cm^2$ | (NWRI, 2012) |

When designing the UV disinfection system with low-pressure UV lamps, a point-source-summation method was used to determine the water quality factor and effluent coliform number using suspended solids concentrations and UV dosage, respectively (U.S. EPA, 1986; WEF, 2010a). The low-pressure high intensity lamps have a maximum input power of 260 W with an efficiency of 33% (Metcalf & Eddy, Inc, 2003; Trojan Technologies, 2008). The variable output (dimming) capabilities of this lamp are from 60 to 100% (Trojan Technologies, 2008). When designing the UV disinfection system with medium-pressure UV lamps, a point-source-summation was also done. Point-source-summation is a model used for estimating the UV intensity. The following equation is used for this model

$$I = \frac{S}{4\pi R^2}$$ (U.S. EPA, 1986)

where $I$ = intensity at distance $R$; $S$ = power available from UV source; and $R$ = distance of point-source. From here the UV dose can be determined by

$$D = I_{AVG}\theta$$ (WEF, 2010a)

where $D$ = Average UV dose; $I_{AVG}$ = array-averaged intensity from point-source-summation; and $\theta$ = average HRT within UV light. To determine the effluent coliform number after exposure, a variation of the Chick-Watson first-order model was developed. The following equation is this variation

$$\ln\left(\frac{N}{N_O}\right) = -kI_{AVG}t \Rightarrow N = N_O e^{-k/t}$$ (Metcalf & Eddy, Inc, 2003; U.S. EPA, 1986; Qasim, 1999; Lin, 2007; WEF, 2010a)

where $N$ = total number of surviving disperse coliform bacteria; $N_o$ = total number of disperse coliform bacteria prior to UV light; $k$ = inactivation rate coefficient; $I$ = average intensity of UV light; and $t$ = exposure time. The medium-pressure high intensity lamps have a maximum input power of 3,200 W with an efficiency of 12% (Metcalf & Eddy, Inc, 2003; Trojan Technologies, 2007). The variable output capabilities of this lamp are from 30 to 100% (Trojan Technologies, 2007). To determine the headloss through the UV channel the following equation is used

$$h_L = 1.8\frac{v^2}{2g}$$ (Metcalf & Eddy, Inc, 2003; Qasim, 1999)

where $h_L$ = headloss; v = approach velocity; and g = gravitational acceleration. Table A 22 and Table A 23 shows the design for UV disinfection of membrane effluent with low-pressure high intensity lamps and medium-pressure high intensity lamps for the 8.8 MGD WRP facilities, respectively. Table A 24 and Table A 25 shows the design for UV disinfection of filter effluent with low-pressure high intensity lamps and medium-pressure high intensity lamps for the 8.8 MGD WRP facilities, respectively.

**Table A 22 – UV Disinfection Design of Membrane Effluent with Low-Pressure High Intensity Lamps at 8.8 MGD**

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| **Flow Parameters** | | |
| Average Flow | 8.8 | MGD |
| Average Flow | 13.62 | $ft^3$/sec |
| Average Flow | 23133 | L/min |
| Peak Flow | 9.6 | MGD |
| Peak Flow | 14.85 | $ft^3$/sec |
| Peak Flow | 25236 | L/min |
| **Lamp Parameters** | | |
| Lamp Length | 1.50 | m |
| Lamp Length | 4.922 | ft |
| Lamp and Sleeve Diameter | 23 | mm |
| Lamp and Sleeve Diameter | 0.906 | in |
| Lamp and Sleeve Area | 4.15E-04 | $m^2$ |
| Lamp and Sleeve Area | 4.47E-03 | $ft^2$ |
| Lamp Spacing (O.C.) | 102 | mm |
| Lamp Spacing (O.C.) | 4.02 | in |
| Lamps Per Module | 8 | |
| Modules Per Bank | 14 | |
| Banks Per Channel | 2 | |
| Standby Banks Per Channel | 1 | |
| Lamps Per Channel Not Including Standby | 224 | |
| Lamps Per Channel Including Standby | 336 | |
| UV Input/Output Range | 60-100 | % |
| Maximum UV Input | 260 | W |
| Minimum UV Input | 156.00 | W |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| Maximum UV Output | 85 | W |
| Minimum UV Output | 51.00 | W |
| Minimum UV Dosage According To Ten States Standards | 80.00 | $mW{\cdot}s/cm^2$ |
| **Initial Coliform Bacteria** | | |
| Initial Coliform Bacteria Count | 1E+03 | MPN/100 mL |
| Effluent Coliform Bacteria Count Requirement | 2.2 | MPN/100 mL |
| Effluent Coliform Bacterial Count Goal | 2 | MPN/100 mL |
| **Reactor Design (EPA Method) Check Assuming Maximum Output** | | |
| Required Width of Channel | 4.7 | ft |
| Required Depth of Channel | 2.7 | ft |
| Freeboard | 2 | ft |
| Required Area of Channel | 12.5 | $ft^2$ |
| Cross Sectional Area of Channel | 12.0 | $ft^2$ |
| Volume of Liquid Per Lamp ($V_v$) | 15.0 | L |
| UV Density | 5.7 | W/L |
| Assumed Transmittance | 80 | % |
| au/cm from Equation 12-72 (Metcalf & Eddy, Inc, 2003) | 0.095 | au/cm |
| Right Side of Equation 12-72 | 80 | % |
| Absorbance coefficient ($\alpha$) | 0.22 | 1/cm |
| Nominal Average Intensity ($I_{avg}$) From Figure 7-28 of EPA 1986 | 23 | $mW/cm^2$ |
| Adjusted Average Intensity ($I_{avg}$) | 12.88 | $mW/cm^2$ |
| Collins-Selleck b | 4 | $mJ/cm^2$ |
| Collins-Selleck $\Lambda_{CS}$ | 26 | |
| Average Flow Contact Time Per Bank | 4.35 | sec |
| Average Flow Contact Time Per Channel | 8.71 | sec |
| Peak Flow Contact Time Per Bank | 3.99 | sec |
| Peak Flow Contact Time Per Channel | 7.98 | sec |
| Dosage at Average Flow | 112.1 | $mW{\cdot}s/cm^2$ |
| Dosage at Peak Flow | 102.8 | $mW{\cdot}s/cm^2$ |
| $\Lambda_{CS}$[ln(It)-ln(b)] at Average Flow | -37.6 | |
| $\Lambda_{CS}$[ln(It)-ln(b)] at Peak Flow | -36.7 | |
| Effluent Coliform Bacteria at Average Flow Collins-Selleck | 2E-35 | MPN/100 mL |
| Effluent Coliform Bacteria at Peak Flow Collins-Selleck | 1.20E-13 | MPN/100 mL |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| E-Coli First Order Inactivation Constant From WEF 1998 | 0.72 | $cm^2/mW \cdot s$ |
| $kI_{avg}t$ at average flow | -80.7 | |
| $kI_{avg}t$ at peak flow | -74.0 | |
| Effluent Coliform Bacteria at Average Flow WEF 1998 | 9E-33 | MPN/100 mL |
| Effluent Coliform Bacteria at Peak Flow WEF 1998 | 7.27E-30 | MPN/100 mL |
| Emerick Darby Water Quality Factor (*f*) From Figure 19.36 of WEF 2010 Assuming 5 mg/L TSS | 2.00E+04 | |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Average Flow | 2 | MPN/100 mL |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Peak Flow | 2 | MPN/100 mL |
| **Headloss** | | |
| Velocity at Average | 0.34 | m/s |
| Velocity at Average | 34.47 | cm/s |
| Headloss at Average (Metcalf & Eddy, Inc, 2003) | 0.03 | m |
| Headloss at Average | 0.11 | ft |
| Velocity at Peak | 0.38 | m/s |
| Velocity at Peak | 37.60 | cm/s |
| Headloss at Peak (Metcalf & Eddy, Inc, 2003) | 0.04 | m |
| Headloss at Peak | 0.13 | ft |
| **Average Flow Actual Dosage** | | |
| UV Input Required | 156 | W |
| UV Output Required | 51.0 | W |
| Percent Illuminated | 60.0 | % |
| UV Density | 3.4 | W/L |
| Nominal Average Intensity ($I_{avg}$) From Figure 7-28 of EPA 1986 | 17 | $mW/cm^2$ |
| Adjusted Average Intensity ($I_{avg}$) | 9.52 | $mW/cm^2$ |
| Dosage at Average Flow | 82.9 | $mW \cdot s/cm^2$ |
| $\Lambda_{CS}$[ln(It)-ln(b)] at Average Flow | -34.2 | |
| Effluent Coliform Bacteria at Average Flow Collins-Selleck | 6E-32 | MPN/100 mL |
| $kI_{avg}t$ at average flow | -59.67 | |
| Effluent Coliform Bacteria at Average Flow WEF 1998 | 1E-23 | MPN/100 mL |

| Parameter | Value | Unit |
|---|---|---|
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Average Flow | 2 | MPN/100 mL |
| Energy Per Channel At Average Flow | 34.94 | kW |
| Energy Per Channel Per Day at Average Flow | 839 | kWh/day |
| **Peak Flow Dosage** | | |
| UV Input Required | 168 | W |
| UV Output Required | 54.9 | W |
| Percent Illuminated | 64.6 | % |
| UV Density | 3.7 | W/L |
| Nominal Average Intensity ($I_{avg}$) From Figure 7-28 of EPA 1986 | 18.5 | $mW/cm^2$ |
| Adjusted Average Intensity ($I_{avg}$) | 10.36 | $mW/cm^2$ |
| Dosage at Peak Flow | 82.7 | $mW\cdot s/cm^2$ |
| $\Lambda_{CS}[\ln(It)-\ln(b)]$ at Peak Flow | -34.2 | |
| Effluent Coliform Bacteria at Peak Flow Collins-Selleck | 1.41E-12 | MPN/100 mL |
| $kI_{avg}t$ at average flow | -59.52 | |
| Effluent Coliform Bacteria at Average Flow WEF 1998 | 1E-23 | MPN/100 mL |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Average Flow | 2 | MPN/100 mL |
| Energy Per Channel At Peak Flow | 37.63 | kW |
| Energy Per Channel Per Day at Peak Flow | 903 | kWh/day |

**Table A 23 – UV Disinfection Design of Membrane Effluent with Medium-Pressure High Intensity Lamps at 8.8 MGD**

| Parameter | Value | Unit |
|---|---|---|
| **Flow Parameters** | | |
| Average Flow | 8.8 | MGD |
| Average Flow | 13.62 | $ft^3/sec$ |
| Average Flow | 23133 | L/min |
| Peak Flow | 9.6 | MGD |
| Peak Flow | 14.85 | $ft^3/sec$ |
| Peak Flow | 25236 | L/min |
| **Lamp Parameters** | | |
| Lamp Length | 0.25 | m |
| Lamp Length | 0.820 | ft |
| Lamp and Sleeve Diameter | 76 | mm |
| Lamp and Sleeve Diameter | 2.992 | in |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| Lamp and Sleeve Area | 4.54E-03 | $m^2$ |
| Lamp and Sleeve Area | 4.88E-02 | $ft^2$ |
| Lamp Spacing (O.C.) | 127 | mm |
| Lamp Spacing (O.C.) | 5.00 | in |
| Lamps Per Module | 24 | |
| Modules Per Bank | 4 | |
| Banks Per Channel | 1 | |
| Standby Banks Per Channel | 1 | |
| Lamps Per Channel Not Including Standby | 96 | |
| Lamps Per Channel Including Standby | 192 | |
| UV Input/Output Range | 30-100 | % |
| Maximum UV Input | 3200 | W |
| Minimum UV Input | 960.00 | W |
| Maximum UV Output | 384 | W |
| Minimum UV Output | 115.20 | W |
| Minimum UV Dosage According To Ten States Standards | 80 | $mW·s/cm^2$ |
| **Initial Coliform Bacteria** | | |
| Initial Coliform Bacteria Count | 1E+03 | MPN/100 mL |
| Effluent Coliform Bacteria Count Requirement | 2.2 | MPN/100 mL |
| Effluent Coliform Bacterial Count Goal | 2 | MPN/100 mL |
| **Reactor Design (EPA Method) Check Assuming Maximum Output** | | |
| Required Width of Channel | 10.0 | ft |
| Required Depth of Channel | 1.7 | ft |
| Freeboard | 2 | ft |
| Required Area of Channel | 16.7 | $ft^2$ |
| Cross Sectional Area of Channel | 12.0 | $ft^2$ |
| Volume of Liquid Per Lamp ($V_v$) | 2.9 | L |
| UV Density | 132.5 | W/L |
| Assumed Transmittance | 80 | % |
| au/cm from Equation 12-72 (Metcalf & Eddy, Inc, 2003) | 0.095 | au/cm |
| Right Side of Equation 12-72 | 80 | % |
| Absorbance coefficient ($\alpha$) | 0.22 | 1/cm |
| Nominal Average Intensity (Iavg) From PSS Method | 282.3 | $mW/cm^2$ |
| Adjusted Average Intensity ($I_{avg}$) | 158.1 | $mW/cm^2$ |
| Collins-Selleck b | 4 | $mJ/cm^2$ |
| Collins-Selleck $\Lambda_{CS}$ | 26 | |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| Average Flow Contact Time Per Bank | 0.72 | sec |
| Average Flow Contact Time Per Channel | 0.72 | sec |
| Peak Flow Contact Time Per Bank | 0.66 | sec |
| Peak Flow Contact Time Per Channel | 0.66 | sec |
| Dosage at Average Flow | 114.1 | $mW\cdot s/cm^2$ |
| Dosage at Peak Flow | 104.6 | $mW\cdot s/cm^2$ |
| $\Lambda_{CS}$[ln(It)-ln(b)] at Average Flow | -37.8 | |
| $\Lambda_{CS}$[ln(It)-ln(b)] at Peak Flow | -36.9 | |
| Effluent Coliform Bacteria at Average Flow Collins-Selleck | 1E-35 | MPN/100 mL |
| Effluent Coliform Bacteria at Peak Flow Collins-Selleck | 9.90E-14 | MPN/100 mL |
| E-Coli First Order Inactivation Constant From WEF 1998 | 0.72 | $cm^2/mW\cdot s$ |
| $kI_{avg}t$ at average flow | -82.1 | |
| $kI_{avg}t$ at peak flow | -75.3 | |
| Effluent Coliform Bacteria at Average Flow WEF 1998 | 2E-33 | MPN/100 mL |
| Effluent Coliform Bacteria at Peak Flow WEF 1998 | 2.00E-30 | MPN/100 mL |
| Emerick Darby Water Quality Factor (*f*) From Figure 19.36 of WEF 2010 Assuming 5 mg/L TSS | 4.50E+04 | |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Average Flow | 2 | MPN/100 mL |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Peak Flow | 2 | MPN/100 mL |
| **Headloss** | | |
| Velocity at Average | 0.35 | m/s |
| Velocity at Average | 34.65 | cm/s |
| Headloss at Average (Metcalf & Eddy, Inc, 2003) | 0.02 | m |
| Headloss at Average | 0.07 | ft |
| Velocity at Peak | 0.38 | m/s |
| Velocity at Peak | 37.80 | cm/s |
| Headloss at Peak (Metcalf & Eddy, Inc, 2003) | 0.03 | m |
| Headloss at Peak | 0.09 | ft |
| **Average Flow Actual Dosage** | | |
| UV Input Required | 2250 | W |
| UV Output Required | 270.0 | W |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| Percent Illuminated | 70.3 | % |
| UV Density | 93.2 | W/L |
| Nominal Average Intensity (Iavg) From PSS Method | 198.4 | $mW/cm^2$ |
| Adjusted Average Intensity ($I_{avg}$) | 111.104 | $mW/cm^2$ |
| Dosage at Average Flow | 80.2 | $mW{\cdot}s/cm^2$ |
| $\Lambda_{CS}$[ln(It)-ln(b)] at Average Flow | -33.9 | |
| Effluent Coliform Bacteria at Average Flow Collins-Selleck | 1E-31 | MPN/100 mL |
| $kI_{avg}t$ at average flow | -57.73 | |
| Effluent Coliform Bacteria at Average Flow WEF 1998 | 9E-23 | MPN/100 mL |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Average Flow | 2 | MPN/100 mL |
| Energy Per Channel At Average Flow | 216.00 | kW |
| Energy Per Channel Per Day at Average Flow | 5184 | kWh/day |
| **Peak Flow Dosage** | | |
| UV Input Required | 2500 | W |
| UV Output Required | 300.0 | W |
| Percent Illuminated | 78.1 | % |
| UV Density | 103.5 | W/L |
| Nominal Average Intensity (Iavg) From PSS Method | 220.5 | $mW/cm^2$ |
| Adjusted Average Intensity ($I_{avg}$) | 123.48 | $mW/cm^2$ |
| Dosage at Peak Flow | 81.7 | $mW{\cdot}s/cm^2$ |
| $\Lambda_{CS}$[ln(It)-ln(b)] at Peak Flow | -34.1 | |
| Effluent Coliform Bacteria at Peak Flow Collins-Selleck | 1.61E-12 | MPN/100 mL |
| $kI_{avg}t$ at average flow | -58.81 | |
| Effluent Coliform Bacteria at Average Flow WEF 1998 | 3E-23 | MPN/100 mL |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Average Flow | 2 | MPN/100 mL |
| Energy Per Channel At Peak Flow | 240.00 | kW |
| Energy Per Channel Per Day at Peak Flow | 5760 | kWh/day |

**Table A 24 – UV Disinfection Design of Filter Effluent with Low-Pressure High Intensity Lamps at 8.8 MGD**

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| **Flow Parameters** | | |
| Average Flow | 8.8 | MGD |
| Average Flow | 13.62 | $ft^3$/sec |
| Average Flow | 23133 | L/min |
| Peak Flow | 9.6 | MGD |
| Peak Flow | 14.85 | $ft^3$/sec |
| Peak Flow | 25236 | L/min |
| **Lamp Parameters** | | |
| Lamp Length | 1.50 | m |
| Lamp Length | 4.922 | ft |
| Lamp and Sleeve Diameter | 23 | mm |
| Lamp and Sleeve Diameter | 0.906 | in |
| Lamp and Sleeve Area | 4.15E-04 | $m^2$ |
| Lamp and Sleeve Area | 4.47E-03 | $ft^2$ |
| Lamp Spacing (O.C.) | 102 | mm |
| Lamp Spacing (O.C.) | 4.02 | in |
| Lamps Per Module | 8 | |
| Modules Per Bank | 20 | |
| Banks Per Channel | 2 | |
| Standby Banks Per Channel | 1 | |
| Lamps Per Channel Not Including Standby | 320 | |
| Lamps Per Channel Including Standby | 480 | |
| UV Input/Output Range | 60-100 | % |
| Maximum UV Input | 260 | W |
| Minimum UV Input | 156.00 | W |
| Maximum UV Output | 85 | W |
| Minimum UV Output | 51.00 | W |
| Minimum UV Dosage According To Ten States Standards | 100.00 | $mW \cdot s/cm^2$ |
| **Initial Coliform Bacteria** | | |
| Initial Coliform Bacteria Count | 1E+06 | MPN/100 mL |
| Effluent Coliform Bacteria Count Requirement | 2.2 | MPN/100 mL |
| Effluent Coliform Bacterial Count Goal | 2 | MPN/100 mL |
| **Reactor Design (EPA Method) Check Assuming Maximum Output** | | |
| Required Width of Channel | 6.7 | ft |
| Required Depth of Channel | 2.7 | ft |
| Freeboard | 2 | ft |
| Required Area of Channel | 17.9 | $ft^2$ |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| Cross Sectional Area of Channel | 17.2 | $ft^2$ |
| Volume of Liquid Per Lamp ($V_v$) | 15.0 | L |
| UV Density | 5.7 | W/L |
| Assumed Transmittance | 75 | % |
| au/cm from Equation 12-72 (Metcalf & Eddy, Inc, 2003) | 0.125 | au/cm |
| Right Side of Equation 12-72 | 75 | % |
| Absorbance coefficient ($\alpha$) | 0.29 | 1/cm |
| Nominal Average Intensity ($I_{avg}$) From Figure 7-28 of EPA 1986 | 18.5 | $mW/cm^2$ |
| Adjusted Average Intensity ($I_{avg}$) | 10.36 | $mW/cm^2$ |
| Collins-Selleck b | 4 | $mJ/cm^2$ |
| Collins-Selleck $\Lambda_{CS}$ | 26 | |
| Average Flow Contact Time Per Bank | 6.22 | sec |
| Average Flow Contact Time Per Channel | 12.44 | sec |
| Peak Flow Contact Time Per Bank | 5.70 | sec |
| Peak Flow Contact Time Per Channel | 11.40 | sec |
| Dosage at Average Flow | 128.1 | $mW \cdot s/cm^2$ |
| Dosage at Peak Flow | 118.1 | $mW \cdot s/cm^2$ |
| $\Lambda_{CS}$[ln(It)-ln(b)] at Average Flow | -39.2 | |
| $\Lambda_{CS}$[ln(It)-ln(b)] at Peak Flow | -38.2 | |
| Effluent Coliform Bacteria at Average Flow Collins-Selleck | 6E-34 | MPN/100 mL |
| Effluent Coliform Bacteria at Peak Flow Collins-Selleck | 2.51E-11 | MPN/100 mL |
| E-Coli First Order Inactivation Constant From WEF 1998 | 0.72 | $cm^2/mW \cdot s$ |
| $kI_{avg}t$ at average flow | -92.8 | |
| $kI_{avg}t$ at peak flow | -85.0 | |
| Effluent Coliform Bacteria at Average Flow WEF 1998 | 5E-35 | MPN/100 mL |
| Effluent Coliform Bacteria at Peak Flow WEF 1998 | 1.18E-31 | MPN/100 mL |
| Emerick Darby Water Quality Factor (*f*) From Figure 19.36 of WEF 2010 Assuming 5 mg/L TSS | 2.50E+04 | |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Average Flow | 2 | MPN/100 mL |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Peak Flow | 2 | MPN/100 mL |
| **Headloss** | | |
| Velocity at Average | 0.24 | m/s |
| Velocity at Average | 24.17 | cm/s |
| Headloss at Average (Metcalf & Eddy, Inc, 2003) | 0.02 | m |
| Headloss at Average | 0.05 | ft |
| Velocity at Peak | 0.26 | m/s |
| Velocity at Peak | 26.32 | cm/s |
| Headloss at Peak (Metcalf & Eddy, Inc, 2003) | 0.02 | m |
| Headloss at Peak | 0.06 | ft |
| **Average Flow Actual Dosage** | | |
| UV Input Required | 160 | W |
| UV Output Required | 52.3 | W |
| Percent Illuminated | 61.5 | % |
| UV Density | 3.5 | W/L |
| Nominal Average Intensity ($I_{avg}$) From Figure 7-28 of EPA 1986 | 15 | $mW/cm^2$ |
| Adjusted Average Intensity ($I_{avg}$) | 8.4 | $mW/cm^2$ |
| Dosage at Average Flow | 104.5 | $mW \cdot s/cm^2$ |
| $\Lambda_{CS}$[ln(It)-ln(b)] at Average Flow | -36.8 | |
| Effluent Coliform Bacteria at Average Flow Collins-Selleck | 1E-31 | MPN/100 mL |
| $kI_{avg}t$ at average flow | -75.21 | |
| Effluent Coliform Bacteria at Average Flow WEF 1998 | 2E-27 | MPN/100 mL |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Average Flow | 2 | MPN/100 mL |
| Energy Per Channel At Average Flow | 51.20 | kW |
| Energy Per Channel Per Day at Average Flow | 1229 | kWh/day |
| **Peak Flow Dosage** | | |
| UV Input Required | 190 | W |
| UV Output Required | 62.1 | W |
| Percent Illuminated | 73.1 | % |
| UV Density | 4.1 | W/L |
| Nominal Average Intensity ($I_{avg}$) From Figure 7-28 of EPA 1986 | 16.5 | $mW/cm^2$ |
| Adjusted Average Intensity ($I_{avg}$) | 9.24 | $mW/cm^2$ |
| Dosage at Peak Flow | 105.3 | $mW \cdot s/cm^2$ |

| Parameter | Value | Unit |
|---|---|---|
| $\Lambda_{CS}$[ln(It)-ln(b)] at Peak Flow | -36.9 | |
| Effluent Coliform Bacteria at Peak Flow Collins-Selleck | 9.12E-11 | MPN/100 mL |
| $kI_{avg}t$ at average flow | -75.84 | |
| Effluent Coliform Bacteria at Average Flow WEF 1998 | 1E-27 | MPN/100 mL |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Average Flow | 2 | MPN/100 mL |
| Energy Per Channel At Peak Flow | 60.80 | kW |
| Energy Per Channel Per Day at Peak Flow | 1459 | kWh/day |

**Table A 25 – UV Disinfection Design of Filter Effluent with Medium-Pressure High Intensity Lamps at 8.8 MGD**

| Parameter | Value | Unit |
|---|---|---|
| **Flow Parameters** | | |
| Average Flow | 8.8 | MGD |
| Average Flow | 13.62 | $ft^3$/sec |
| Average Flow | 23133 | L/min |
| Peak Flow | 9.6 | MGD |
| Peak Flow | 14.85 | $ft^3$/sec |
| Peak Flow | 25236 | L/min |
| **Lamp Parameters** | | |
| Lamp Length | 0.25 | m |
| Lamp Length | 0.820 | ft |
| Lamp and Sleeve Diameter | 76 | mm |
| Lamp and Sleeve Diameter | 2.992 | in |
| Lamp and Sleeve Area | 4.54E-03 | $m^2$ |
| Lamp and Sleeve Area | 4.88E-02 | $ft^2$ |
| Lamp Spacing (O.C.) | 127 | mm |
| Lamp Spacing (O.C.) | 5.00 | in |
| Lamps Per Module | 24 | |
| Modules Per Bank | 5 | |
| Banks Per Channel | 1 | |
| Standby Banks Per Channel | 1 | |
| Lamps Per Channel Not Including Standby | 120 | |
| Lamps Per Channel Including Standby | 240 | |
| UV Input/Output Range | 30-100 | % |
| Maximum UV Input | 3200 | W |
| Minimum UV Input | 960.00 | W |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| Maximum UV Output | 384 | W |
| Minimum UV Output | 115.20 | W |
| Minimum UV Dosage According To Ten States Standards | 100 | $mW{\cdot}s/cm^2$ |
| **Initial Coliform Bacteria** | | |
| Initial Coliform Bacteria Count | 1E+06 | MPN/100 mL |
| Effluent Coliform Bacteria Count Requirement | 2.2 | MPN/100 mL |
| Effluent Coliform Bacterial Count Goal | 2 | MPN/100 mL |
| **Reactor Design (EPA Method) Check Assuming Maximum Output** | | |
| Required Width of Channel | 10.0 | ft |
| Required Depth of Channel | 2.1 | ft |
| Freeboard | 2 | ft |
| Required Area of Channel | 20.8 | $ft^2$ |
| Cross Sectional Area of Channel | 15.0 | $ft^2$ |
| Volume of Liquid Per Lamp ($V_v$) | 2.9 | L |
| UV Density | 132.5 | W/L |
| Assumed Transmittance | 75 | % |
| au/cm from Equation 12-72 (Metcalf & Eddy, Inc, 2003) | 0.125 | au/cm |
| Right Side of Equation 12-72 | 75 | % |
| Absorbance coefficient ($\alpha$) | 0.29 | 1/cm |
| Nominal Average Intensity (Iavg) From PSS Method | 217.4 | $mW/cm^2$ |
| Adjusted Average Intensity ($I_{avg}$) | 121.7 | $mW/cm^2$ |
| Collins-Selleck b | 4 | $mJ/cm^2$ |
| Collins-Selleck $\Lambda_{CS}$ | 26 | |
| Average Flow Contact Time Per Bank | 0.90 | sec |
| Average Flow Contact Time Per Channel | 0.90 | sec |
| Peak Flow Contact Time Per Bank | 0.83 | sec |
| Peak Flow Contact Time Per Channel | 0.83 | sec |
| Dosage at Average Flow | 109.8 | $mW{\cdot}s/cm^2$ |
| Dosage at Peak Flow | 100.7 | $mW{\cdot}s/cm^2$ |
| $\Lambda_{CS}[\ln(It)-\ln(b)]$ at Average Flow | -37.4 | |
| $\Lambda_{CS}[\ln(It)-\ln(b)]$ at Peak Flow | -36.4 | |
| Effluent Coliform Bacteria at Average Flow Collins-Selleck | 4E-32 | MPN/100 mL |
| Effluent Coliform Bacteria at Peak Flow Collins-Selleck | 1.52E-10 | MPN/100 mL |

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| E-Coli First Order Inactivation Constant From WEF 1998 | 0.72 | $cm^2/mW \cdot s$ |
| $kI_{avg}t$ at average flow | -79.1 | |
| $kI_{avg}t$ at peak flow | -72.5 | |
| Effluent Coliform Bacteria at Average Flow WEF 1998 | 5E-29 | MPN/100 mL |
| Effluent Coliform Bacteria at Peak Flow WEF 1998 | 3.33E-26 | MPN/100 mL |
| Emerick Darby Water Quality Factor (*f*) From Figure 19.36 of WEF 2010 Assuming 5 mg/L TSS | 4.50E+04 | |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Average Flow | 2 | MPN/100 mL |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Peak Flow | 2 | MPN/100 mL |
| **Headloss** | | |
| Velocity at Average | 0.28 | m/s |
| Velocity at Average | 27.72 | cm/s |
| Headloss at Average (Metcalf & Eddy, Inc, 2003) | 0.01 | m |
| Headloss at Average | 0.05 | ft |
| Velocity at Peak | 0.30 | m/s |
| Velocity at Peak | 30.24 | cm/s |
| Headloss at Peak (Metcalf & Eddy, Inc, 2003) | 0.02 | m |
| Headloss at Peak | 0.06 | ft |
| **Average Flow Actual Dosage** | | |
| UV Input Required | 2950 | W |
| UV Output Required | 354.0 | W |
| Percent Illuminated | 92.2 | % |
| UV Density | 122.1 | W/L |
| Nominal Average Intensity (Iavg) From PSS Method | 200.4 | $mW/cm^2$ |
| Adjusted Average Intensity ($I_{avg}$) | 112.224 | $mW/cm^2$ |
| Dosage at Average Flow | 101.2 | $mW \cdot s/cm^2$ |
| $\Lambda_{CS}$[ln(It)-ln(b)] at Average Flow | -36.5 | |
| Effluent Coliform Bacteria at Average Flow Collins-Selleck | 3E-31 | MPN/100 mL |
| $kI_{avg}t$ at average flow | -72.89 | |
| Effluent Coliform Bacteria at Average Flow WEF 1998 | 2E-26 | MPN/100 mL |

| Parameter | Value | Unit |
|---|---|---|
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Average Flow | 2 | MPN/100 mL |
| Energy Per Channel At Average Flow | 354.00 | kW |
| Energy Per Channel Per Day at Average Flow | 8496 | kWh/day |
| **Peak Flow Dosage** | | |
| UV Input Required | 3200 | W |
| UV Output Required | 384.0 | W |
| Percent Illuminated | 100 | % |
| UV Density | 132.5 | W/L |
| Nominal Average Intensity (Iavg) From PSS Method | 217.4 | $mW/cm^2$ |
| Adjusted Average Intensity ($I_{avg}$) | 121.744 | $mW/cm^2$ |
| Dosage at Peak Flow | 100.7 | $mW{\cdot}s/cm^2$ |
| $\Lambda_{CS}$[ln(It)-ln(b)] at Peak Flow | -36.4 | |
| Effluent Coliform Bacteria at Peak Flow Collins-Selleck | 1.52E-10 | MPN/100 mL |
| $kI_{avg}t$ at average flow | -72.48 | |
| Effluent Coliform Bacteria at Average Flow WEF 1998 | 3E-26 | MPN/100 mL |
| Effluent Total Coliform From Figure 19.37 of WEF 2010 At Average Flow | 2 | MPN/100 mL |
| Energy Per Channel At Peak Flow | 384.00 | kW |
| Energy Per Channel Per Day at Peak Flow | 9216 | kWh/day |

### *A-9 Chlorination*

The alternative disinfection process used to contrast UV disinfection was chlorination. Chloramination can also be used with the addition of ammonia ahead of the chlorine contact basin. Parameters used in the design of the chlorination contact basin are seen in Table A 26.

**Table A 26 – Chlorination Design Parameters**

| Parameter | Value | Unit | Reference |
|---|---|---|---|
| **Minimum chlorine contact time** | 450 | mg-min/L | (Hirani, et al., 2010; WEF, 2010a) |
| **Effluent total coliform concentration** | 2.2 | MPN/ 100mL | (Hirani, et al., 2010; U.S. EPA, 2004a; Metcalf & Eddy, Inc, 2007; WEF, 2010a) |
| **Membrane effluent total coliform concentration** | 807±1314 | MPN/ 100mL | (DeCarolis Jr, et al., 2007) |
| **Chlorine residual required** | 3 | mg/L | |
| **Detention time at peak flow** | 30 | min | |
| **Dispersion number at peak flow** | 0.0150 | – | (Metcalf & Eddy, Inc, 2003) |

To determine the required chlorine dosage to disinfect the membrane effluent the following equation was used

$$\frac{N}{N_o} = \left[\frac{C_R t}{b}\right]^{-n}$$ (Metcalf & Eddy, Inc, 2003)

where $N$ = total number of surviving disperse coliform bacteria; $N_o$ = total number of disperse coliform bacteria prior to chlorine dose; $C_R$ = chlorine residual remaining at the end of time $t$; $t$ = contact time; $n$ = slope of inactivation curve; and $b$ = value of x-intercept when $N/N_o$ = 1 or log $N/N_o$ = 0. The membrane effluent total coliform bacterium has a typical range of 10 to 1000 MPN/100mL (Metcalf & Eddy, Inc, 2003; DeCarolis Jr, et al., 2007). Chlorine residual required is assumed to be 3 mg/L leaving the facility. Once a design scheme for the layout of the chlorine contact basin was chosen, the basin was then sized. To ensure proper dispersion, the dispersion number for the chosen design is calculated using

$$D = 1.01\nu\left(N_R\right)^{0.875}$$ (Metcalf & Eddy, Inc, 2003)

where $D$ = coefficient of dispersion; $\nu$ = kinematic viscosity; and $N_R$ = Reynolds number, $4\mathrm{v}R/\nu$ (Sturm, 2010): where $R$ = hydraulic radius and v = velocity in open channel. If the dispersion number calculated is more than the desired peak dispersion number of 0.0150 (Metcalf & Eddy, Inc, 2003) then an alternative design will have to be done to achieve a lower value. Energy consumption for chlorination is driven by the size of the diaphragm pump used to inject chlorine before the contact basin. This energy requirement can be calculated using the pump equation. A pump efficiency of 70% was used (Goulds Pumps, 2012). Table A 27 and Table A 28 shows the design for chlorination for the 8.8 MGD MBR and CAS WRP facilities, respectively.

**Table A 27 – Chlorination of Membrane Effluent Design at 8.8 MGD**

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| **Chlorination Dosage Design Parameters** | | |
| Minimum chlorine contact time | 450 | mg-min/L |
| Effluent total coliform concentration | 2.2 | MPN/100mL |
| Membrane effluent total coliform concentration | 807±1314 | MPN/100mL |
| Chlorine residual required in effluent | 3 | mg/L |
| **Chlorination Dosage Calculations at Peak Flow** | | |
| Chlorine contact time | 30 | min |
| Chlorine demand due to decay during contact time | 2.5 | mg/L |
| Chlorine residual remaining | 1.55 | mg/L |
| Chlorine dosage | 7.05 | mg/L |
| Chlorine consumption | 256.2 | kg/day |
| **Chlorination Contact Basin Design Parameters** | | |
| Detention time at peak flow | 30 | min |
| Dispersion number at peak flow | 0.0150 | |
| Number of parallel channels including redundancy | 2 | |
| Width | 8 | ft |
| Depth | 8 | ft |
| **Chlorination Contact Basin Design Calculation** | | |
| Length | 208.85 | ft |
| Velocity at peak flow | 0.116 | ft/sec |
| Reynolds number | 19178 | |
| Coefficient of dispersion | 6.832E-02 | $ft^2/sec$ |
| Dispersion number - check | 0.0028 | |

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| Average Flow Check | | |
| Velocity at average flow | 0.106 | ft/sec |
| Reynolds number | 17580 | |
| Coefficient of dispersion | 6.331E-02 | $ft^2$/sec |
| Dispersion number - check | 0.003 | |
| **Chlorination Dosage Calculations at Average** | | |
| Chlorine contact time | 33 | min |
| Chlorine demand due to decay during contact time | 2.5 | mg/L |
| Chlorine residual remaining | 1.42 | mg/L |
| Chlorine dosage | 6.92 | mg/L |
| **Pump Sizing** | | |
| Sodium hypochlorite | 12.5 | % by weight |
| Required amount of sodium hypochlorite | 2049.95 | kg/day |
| Required volume of sodium hypochlorite | 434.55 | gal/day |
| | 0.302 | gal/min |
| Total dynamic head | 25 | ft |
| Pump efficiency (diaphragm/peristaltic) | 0.7 | |
| Brake horsepower required | 0.0027 | Hp |
| Brake horsepower used | 0.25 | Hp |
| **Energy Required** | | |
| Energy required from chlorinator pump | 0.1865 | kW |
| | 4.476 | kWh/day |
| **Total Energy Consumption** | 4.476 | kWh/day |

**Table A 28 – Chlorination of Filter Effluent Design at 8.8 MGD**

| *Parameter* | *Value* | *Unit* |
|---|---|---|
| **Chlorination Dosage Design Parameters** | | |
| Minimum chlorine contact time | 450 | mg-min/L |
| Effluent total coliform concentration | 2.2 | MPN/100mL |
| Membrane effluent total coliform concentration | $10^4$-$10^6$ | MPN/100mL |
| Chlorine residual required in effluent | 3 | mg/L |
| **Chlorination Dosage Calculations at Peak Flow** | | |
| Chlorine contact time | 30 | min |
| Chlorine demand due to decay during contact time | 2.5 | mg/L |
| Chlorine residual remaining | 13.98 | mg/L |
| Chlorine dosage | 19.48 | mg/L |
| Chlorine consumption | 707.9 | kg/day |

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| **Chlorination Contact Basin Design Parameters** | | |
| Detention time at peak flow | 30 | min |
| Dispersion number at peak flow | 0.0150 | |
| Number of parallel channels including redundancy | 2 | |
| Width | 8 | ft |
| Depth | 8 | ft |
| **Chlorination Contact Basin Design Calculation** | | |
| Length | 208.85 | ft |
| Velocity at peak flow | 0.116 | ft/sec |
| Reynolds number | 19178 | |
| Coefficient of dispersion | 6.832E-02 | $ft^2$/sec |
| Dispersion number - check | 0.0028 | |
| Average Flow Check | | |
| Velocity at average flow | 0.106 | ft/sec |
| Reynolds number | 17580 | |
| Coefficient of dispersion | 6.331E-02 | $ft^2$/sec |
| Dispersion number - check | 0.003 | |
| **Chlorination Dosage Calculations at Average** | | |
| Chlorine contact time | 33 | min |
| Chlorine demand due to decay during contact time | 2.5 | mg/L |
| Chlorine residual remaining | 12.81 | mg/L |
| Chlorine dosage | 18.31 | mg/L |
| **Pump Sizing** | | |
| Sodium hypochlorite | 12.5 | % by weight |
| Required amount of sodium hypochlorite | 5663.17 | kg/day |
| Required volume of sodium hypochlorite | 1200.50 | gal/day |
| | 0.834 | gal/min |
| Total dynamic head | 25 | ft |
| Pump efficiency (diaphragm/peristaltic) | 0.7 | |
| Brake horsepower required | 0.0075 | Hp |
| Brake horsepower used | 0.25 | Hp |
| **Energy Required** | | |
| Energy required from chlorinator pump | 0.1865 | kW |
| | 4.476 | kWh/day |
| **Total Energy Consumption** | 4.476 | kWh/day |

*A-10 Anaerobic Digester*

Parameters used in the design of the single-stage high-rate mesophilic anaerobic digester can be found in Table A 29. The HRT, equivalent to the SRT, was used in the determination of the volume required for the digester (Metcalf & Eddy, Inc, 2003). The amount of methane-forming volatile solids synthesized per day was determined using the complete-mix high-rate digester equation, followed by the calculation of the volume of methane gas using kinetic equations (Metcalf & Eddy, Inc, 2003; Davis, 2010). These were done taking into account the volume of methane gas at the operating temperature of 35ºC. An egg-shaped digester was used in the design to provide a higher mixing efficiency, improved homogeneous biomass, and most importantly, a smaller real estate area in the WRP (Metcalf & Eddy, Inc, 2003; WEF, 2010b).

The anaerobic digestion process produces methane gas that can be used for energy generation; however, digestion itself consumes energy. Energy consumption for the anaerobic digester is driven by the mixers providing a homogeneous biomass mixture and by the heat-exchanger providing heating for the sludge and heat losses through the digester walls. Mixer energy requirements were determined based on the volume of the digester, using an average energy consumption of 6.5 W/m$^3$ (WEF, 2010b). The energy requirement to heat the sludge was determined using (Metcalf & Eddy, Inc, 2003; Davis, 2010; WEF, 2010b):

$$q = M_s C_s \left(T - T_i\right)$$

where $q$ = heat required, J/day; $M_s$ = mass flow of sludge, kg/day; $C_s$ = specific heat of sludge, J/kg·ºC; $T$ = digestion temperature, ºC; and $T_i$ = influent sludge temperature, ºC. For purposes of this research, 4200 J/kg·ºC was used for the specific heat of sludge

(Metcalf & Eddy, Inc, 2003). The energy required to compensate for the loss of heat through the walls of the digester were determined as (Metcalf & Eddy, Inc, 2003; Davis, 2010; WEF, 2010b):

$$q = UA\Delta T$$

where q = heat loss, J/sec; U = overall coefficient of heat transfer, J/m$^2$·sec·ºC; A = cross-sectional area perpendicular to heat flow, m$^2$; and ΔT = change in temperature between digestion and surface in question. Coefficients of heat transfer used in the research are 0.68, 0.85, and 0.91 W/m$^2$·ºC for the walls, floor, and roof, respectively (Metcalf & Eddy, Inc, 2003; Davis, 2010; WEF, 2010b). Energy production from the combustion of digester gas was determined using:

$$E = HVe$$

where $E$ = energy generated, kJ/day; $H$ = heat of combustion, kJ/m$^3$; $V$ = volume of gas produced per day, m$^3$/day; and $e$ = electrical efficiency. In this research, 37,000 kJ/m$^3$ was used for the heat of combustion of methane (WEF, 2010b). An electrical efficiency of 33% was used based off the efficiency for an internal combustion engine (ICE) (WEF, 2010b). Table A 29 shows the design for the anaerobic digester at 8.8 MGD.

**Table A 29 – Anaerobic Digester Design Parameters and Design at 8.8 MGD**

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| **Anaerobic Digester Design Parameters** | | |
| Dry volatile solids | 0.15 | kg/m$^3$ |
| Biodegradable COD removed | 0.14 | kg/m$^3$ |
| Waste utilization efficiency | 70 | % |
| Bacterial yield | 0.08 | kg VSS/ kg bCOD |
| Bacterial decay coefficient | 0.03 | d$^{-1}$ |
| Percentage of methane in digester gas | 65 | % |
| Solids retention time | 15 | day |

| ***Parameter*** | ***Value*** | ***Unit*** |
|---|---|---|
| Sludge specific gravity | 1.02 | |
| Sludge moisture content | 95 | % |
| Temperature | 35 | °C |
| **Anaerobic Digester Calculations** | | |
| Sludge volume | 97.98 | $m^3$/day |
| bCOD loading | 4663.6 | kg/day |
| Digester volume required | 1469.63 | $m^3$ |
| Volumetric loading | 3.17 | kg/$m^3$·day |
| Total volatile solids produced | 180.1 | kg/day |
| Volume of methane produced | 1050.1 | $m^3$/day |
| Total volume of gas produced | 1615.5 | $m^3$/day |
| **Heating Requirements Design Parameters** | | |
| Heat-transfer coefficient - dry earth embanked for entire depth | 0.68 | W/$m^2$·°C |
| Heat-transfer coefficient - floor of digester in groundwater | 0.85 | W/$m^2$·°C |
| Heat-transfer coefficient - roof exposed to air | 0.91 | W/$m^2$·°C |
| Temperature - air | 25 | °C |
| Temperature - earth next to wall | 15 | °C |
| Temperature - incoming sludge | 18.3 | °C |
| Temperature - earth below floor | 12 | °C |
| Temperature - sludge contents in digester | 35 | °C |
| Specific heat of sludge | 4200 | J/kg·°C |
| **Heating Requirements Calculations** | | |
| Digester diameter | 18.0 | m |
| Digester side depth | 6.0 | m |
| Digester mid depth | 9.0 | m |
| Digester volume provided | 1781.28 | $m^3$ |
| Safety factor | 1.21 | |
| Wall area | 339.3 | $m^2$ |
| Floor area | 268.2 | $m^2$ |
| Roof area | 254.5 | $m^2$ |
| Digester capacity | 84789 | kg/day |
| Heat requirement for sludge | 5.95E+09 | J/day |
| Heat loss by conduction - walls | 3.99E+08 | J/day |
| Heat loss by conduction - floor | 4.53E+08 | J/day |
| Heat loss by conduction - roof | 2.00E+08 | J/day |
| Heat loss - total | 1.05E+09 | J/day |

| ***Parameter*** | ***Value*** | ***Unit*** |
| --- | --- | --- |
| Heat-exchanger capacity | 7.00E+09 | J/day |
| **Energy Requirement** | | |
| Energy required from mixers | 6.5 | $W/m^3$ |
| | 277.88 | kWh/day |
| Energy required from heat-exchanger | 7.00E+09 | J/day |
| | 1944.15 | kWh/day |
| **Total Energy Consumption** | 2222.03 | kWh/day |
| **Energy Generation** | | |
| Energy content of methane gas | 37000 | $kJ/m^3$ |
| | 10.28 | $kWh/m^3$ |
| Electrical efficiency | 33 | % |
| **Energy Generation from Digester** | 3561.44 | kWh/day |

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