Final Draft Report

Filter-Clogging Algae Mitigation Evaluation

Prepared for

Prepared by

March 2012

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CH2MHILL®

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Executive Summary

This report presents the study undertaken by the City of Bellingham (City) to evaluate alternatives to mitigate the adverse impacts of seasonal algae in Lake Whatcom to the City's Whatcom Falls Water Treatment Plant (WTP). This study was undertaken in the second half of 2011 and completed in early 2012.

ES.1 Background and Purpose

In late July and August of 2009 the filters at the City's WTP began clogging much earlier in filter runs than typical, requiring more frequent filter backwashing. The result was greatly reduced WTP capacity – to the point the City implemented mandatory water restrictions, for the first time, to reduce customer demand to match the reduced WTP capacity.

Filter clogging was attributed to algae in Lake Whatcom – the City's source water. Although the reasons for the intense algae bloom of the summer of 2009 is the subject of varied speculation, historical and on-going algae monitoring shows that summertime algae blooms in Lake Whatcom have been increasing over the past decade.

In 1998, Lake Whatcom water quality failed to meet the Washington State dissolved oxygen standard and was placed on Washington's list of polluted waters (Section 303d of the Clean Water Act). As a result of the listing, Ecology initiated a Total Maximum Daily Load (TMDL) study to restore lake water quality. The TMDL study showed that human actions were causing increased phosphorous loading and therefore reduced dissolved oxygen. Meeting the TMDL requirements for phosphorous and dissolved oxygen is expected to take many years to complete, and compliance with the TMDL requirements is the cornerstone of the long-term response to improving lake quality.

Despite on-going coordinated efforts, via the Lake Whatcom Management Program, by the City, Whatcom County, and Lake Whatcom Water and Sewer District to reverse this trend, summertime algae blooms are expected to continue increasing in intensity over the near-term future. Recognizing that it is unacceptable to be in a position wherein it risks falling short of meeting summertime customer water demand, the City initiated this study to evaluate alternative solutions and select a path forward for subsequent implementation.

ES.2 Alternatives Evaluated

The alternatives evaluated for mitigating clogging of the filters at the City's WTP were grouped into three main categories: treatment, intake, and lake management. These alternatives are presented in Table ES-1. In addition to these pro-active alternatives, the "No Action" alternative was included in the Triple Bottom Line Plus evaluation phase as a means of establishing a lowest-cost baseline for comparison.

Each of the treatment alternatives considered for this study are commonly used in the municipal water treatment industry and are commonly-considered alternatives for algae removal. Each would be implemented somewhere at the existing WTP site. They are not, however, equal with respect to removal performance, advantages, disadvantages, and cost.

Three intake alternatives were identified for consideration and evaluation. Each of the intake

TABLE ES-1
Summary of Alternatives Evaluated

Treatment	Intake	Lake Management
Dissolved Air Flotation	Secondary Intake via In-Water Pipeline	Lake Management
Ballasted Sedimentation	Secondary Intake via Over-Land Pipeline	
Plate and Tube Settling	New Dual-Intake System	
Upflow Clarification		
Conventional Sedimentation		
Micro-Screening		
Ozonation		
Additional Filters		

Notes:
Other potential solutions were acknowledged and considered but not evaluated in detail because their feasibility was believed to be questionable based on prior experience and/or a lack of prior application or success. These other potential solutions include: hypolimnetic oxygenation, floating shade balls, lake aeration.

alternatives includes withdrawing water from Lake Whatcom at a location different from the existing intake location that has a substantially lower concentration of algae. Each of the intake alternatives includes the capability to withdraw water at more than one depth.

The Lake Management alternative is essentially the Lake Whatcom Management Program. The Lake Whatcom Management Program is the management forum for improving lake quality and via which compliance with the TMDL requirements for dissolved oxygen and phosphorous is being pursued. Lake management will be implemented regardless of the results of this evaluation. Meeting the TMDL requirements is the cornerstone of the long-term strategy to improve water quality, including reducing algae concentrations.

ES.3 Evaluation of Alternatives

Evaluation of the alternatives to mitigate the adverse impacts of filter-clogging algae at the City's water treatment WTP was implemented in three distinct phases. These three phases include:

- **Screening of Alternatives:** This first phase, "screening of alternatives," was implemented to eliminate from further consideration and evaluation alternatives that were deemed "not selectable" based on one or more screening criteria. This approach enabled more subsequent focus and effort in developing and evaluating those alternatives that were deemed to have greater promise for selection and implementation. Three treatment alternatives, one intake alternative, and the lake management alternative were eliminated from further consideration during screening because they did not meet all of the screening criteria.

- **Evaluation of Alternatives:** This second phase of the evaluation process reflects a more-detailed evaluation of the remaining alternatives. This evaluation phase resulted in identification of the best alternative within categories as well as a best overall alternative based primarily on technical criteria. During this evaluation phase Dissolved Air Flotation (DAF) was determined to be the best treatment alternative and "Secondary Intake via In-Water Pipeline" (Intake Alternative 1) was determined to be the best intake alternative. DAF was determined to be the best overall alternative based on technical performance criteria.

- **Triple Bottom Line Plus Evaluation:** This third phase of the evaluation process reflects evaluation based on a "Triple Bottom Line Plus" (TBL+) approach for the best alternatives per category (as determined in the second phase of evaluation). Additionally, the "No Action" alternative was evaluated as a baseline comparison. This approach enabled scrutiny with respect to financial, social, environmental, and technical objectives. The alternatives evaluated using the TBL+ approach included: DAF, Intake Alternative 1, Additional Filters, and No Action.

The results of the TBL+ evaluation are presented in Figure ES-1 at the end of the Executive Summary. The evaluation criteria are presented in Section 7 of the main body of the report. The TBL+ evaluation results, as well as the results of the more-technically-based second phase of the evaluation process, showed DAF to be the superior alternative for mitigating the filter-clogging algae condition at the City's WTP.

In recognition of the fact that DAF technology is ideally suited to address the filter-clogging algae issue at the Lake Whatcom Water Treatment Plant, DAF was pilot testing during the summer of 2011 to confirm its performance. The pilot testing showed that DAF was very effective at removing algae from the Lake Whatcom supply. Not only was it effective at removing algae, but it was also shown to be effective at removing total organic carbon (TOC), reducing (by up to 25 percent) the formation potential for total trihalomethanes (TTHMs) – a key disinfection byproduct, and most-importantly it was shown to greatly extend filter runs. Extended filter runs results in increased total filter production during algae bloom conditions, which was the primary limitation during the 2009 Lake Whatcom algae bloom.

ES.4 DAF Implementation

In recognition of DAF's ranking as the best alternative for filter-clogging algae mitigation at the City's WTP, a discussion of DAF implementation was developed. Key elements of the implementation discussion relate to project schedule and options for reducing initial capital cost – should the City decide to pursue implementation of a DAF system. An example project schedule that reflects compliance with key Washington State Department of Health requirements and milestones is presented in Figure ES-2 at the end of this Executive Summary. The example schedule conveys the overall timeframe for DAF implementation.

A summary of the initial capital cost (construction and non-construction) for three DAF facility capacities, ranging from 30 mgd to 16 mgd is presented in Table ES-2. A three-train DAF system offers maximum redundancy and capacity to meet significant growth in long-term future customer water demand. The 2-train DAF options are geared toward matching initial capacity with recent trends in peak customer water demand and minimizing initial capital cost. Regardless of the initial capacity and the number of parallel treatment trains, a new DAF facility would be designed to be easily expanded if customer water demand changes.

TABLE ES-2
Summary of Initial Capital Cost for DAF Implementation Options

3-Train 30-mgd system	2-Train 20-mgd system	2-Train 16-mgd system
$ 14,500,000	$ 11,000,000	$ 10,400,000

ES.5 Recommendation

Annual seasonal Lake Whatcom algae blooms present an on-going seasonal risk to the City with respect to meeting the supply needs of its customers. As a result, the City should pursue the design and construction of a new DAF facility in a phased approach based on an initial two-train DAF facility with easy expansion for a potential future third train. The overall timeframe for this first phase of implementation, as well as key milestones, would be similar to that presented in Figure ES-2. A key ancillary benefit of DAF implementation based on the pilot testing completed in the late summer of 2011 is that DAF can be expected to lead to a reduction of the City's TTHMs by 25 percent.

The phased approach will eliminate the potential for constructing more DAF capacity than is necessary to ensure a continuous, reliable, high-quality drinking water supply – even during intense algae blooms in Lake Whatcom. The phased DAF-implementation approach complements the City's on-going commitment to lake management, water quality improvement, and TMDL compliance via the Lake Whatcom Management Program. Over the long-term future, as phosphorous-reducing lake management measures demonstrate success at improving water quality and reducing algae blooms, the need for further expansion of the initial phase of DAF implementation could potentially be avoided entirely.

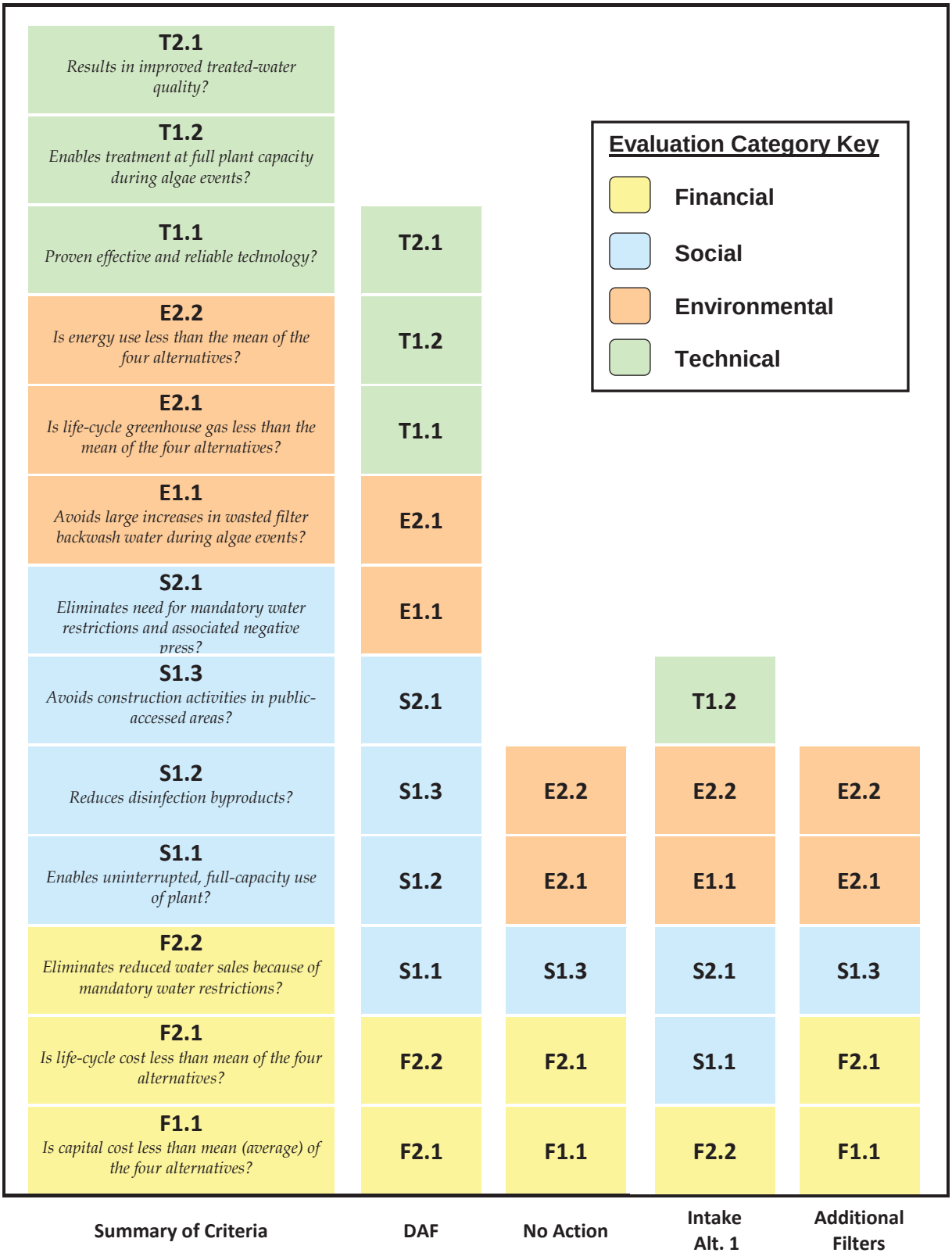

FIGURE ES-1
TBL+ Evaluation Results

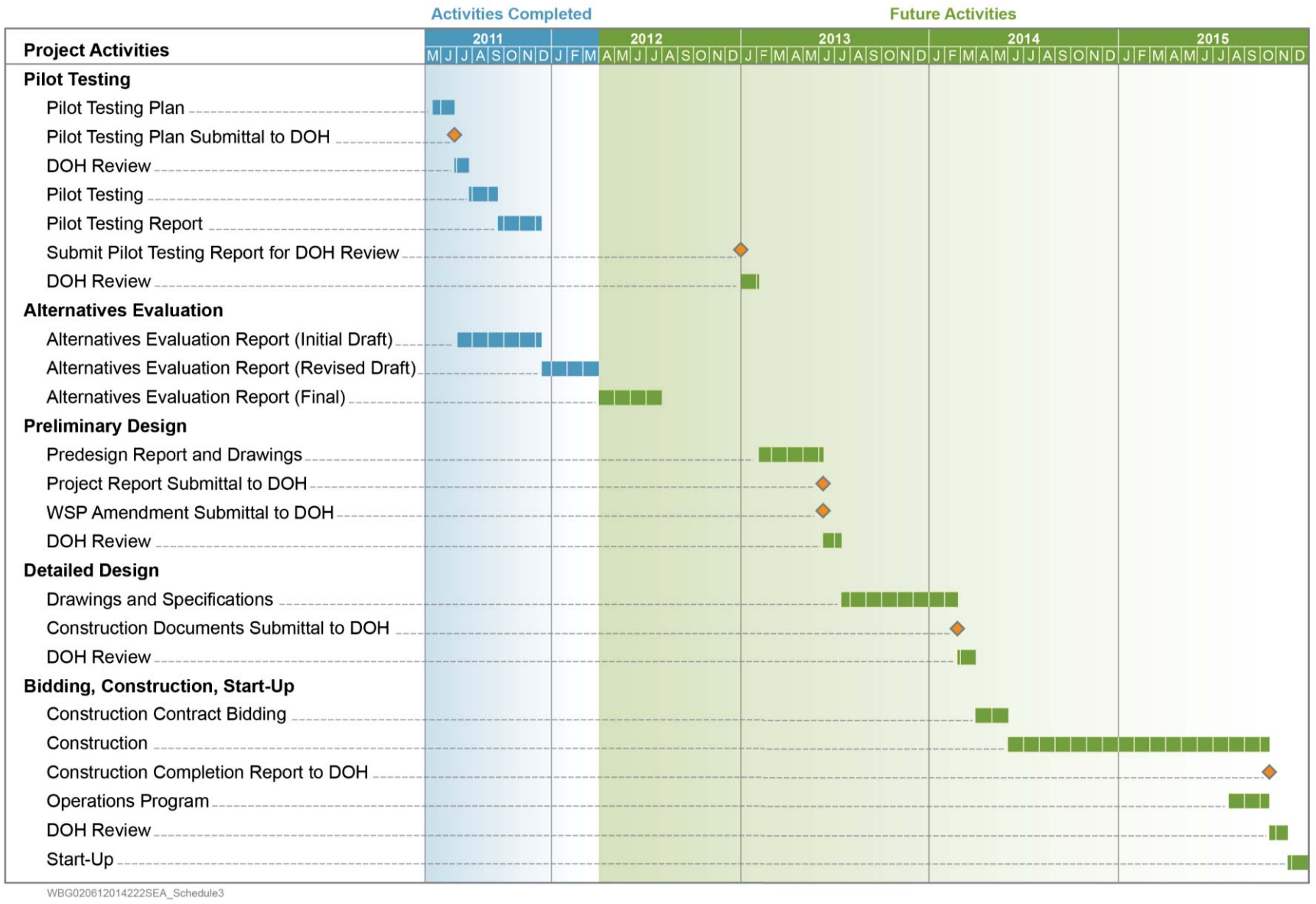

FIGURE ES-2
Example DAF Project Schedule

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1. Introduction

This report presents the study undertaken by the City of Bellingham (City) to evaluate alternatives to mitigate the adverse impacts of seasonal algae in Lake Whatcom to the City's Whatcom Falls Water Treatment Plant (WTP). This study was undertaken in the second half of 2011 and completed in early 2012.

1.1. Background

In late July and August of 2009 the filters at the City's WTP began clogging much earlier in filter runs than typical. Filter runs became substantially shorter than normal, requiring more frequent filter backwashing. The result of shorter filter runs and increased filter backwashing was greatly reduced WTP capacity – to the point the City implemented mandatory water restrictions, for the first time, to reduce customer demand. It should be noted that voluntary water restrictions are implemented each summer as a means of encouraging conservation during this time of typically-high customer water demand. The water restrictions were successful in reducing customer demand to match WTP capacity. Toward the end of August and into September, filter runs gradually began to return to normal and customer demand dropped, as it customarily does at that time of the year.

Filter clogging was attributed to algae in Lake Whatcom. Monitoring revealed higher than typical counts of most algae species. Although the reasons for the intense algae bloom of the summer of 2009 is the subject of varied speculation, historical and on-going algae monitoring shows that summertime algae blooms in Lake Whatcom have been increasing over the past decade. It is speculated that despite efforts to reverse this trend, summertime algae blooms in Lake Whatcom will continue to increase in intensity and duration over the near-term future. Increased Lake Whatcom algae could again result in summertime algae blooms that prevent the WTP from treating sufficient supply to meet customer demand.

1.2. Purpose

The City recognizes that it is unacceptable to be in a position wherein it annually risks falling short of meeting existing and future summertime customer water demand. As a result, it initiated the study to evaluate alternative solutions and select a path forward for subsequent implementation. Specifically, the purpose of this study is to:

- Document existing Lake Whatcom water quality conditions in the context of historical conditions and potential future conditions
- Identify, describe, and evaluate treatment, intake, and lake management alternatives to mitigate clogging by algae of the filters at the City's WTP
- Select an alternative for potential implementation that most efficiently and cost-effectively benefits the City and its customers

2. Existing Conditions

The City has carefully monitored its Lake Whatcom supply for decades. Historical and ongoing monitoring reflects a gradual decline in water quality conditions, including the increased algae growth that has adversely impacted the City's Whatcom Falls Water Treatment WTP. While efforts are underway to reverse this decline via on-going watershed management activities, the time it will take to achieve measurable improvement is uncertain. Therefore, evaluation of alternatives to mitigate the adverse effects of algae must be undertaken with an understanding of current and past observed water quality conditions as well as the recognition that current conditions and declining water quality trends may continue for several or even many years. A summary of the events, conditions, and activities that have led to the need for this study are presented in the following sections.

2.1. 2009 Summer Algae Impacts

Lake Whatcom remains a highly reliable, high quality supply. However, steadily declining water quality and increasing algae in Lake Whatcom over the years has concerned the City. The summertime algae bloom of 2009 provided the specific impetus for the City to initiate this study to find the best way to mitigate the adverse impacts of increased algae.

As stated above, in late July and August of 2009 the filters at the City's WTP began clogging early in filter runs, resulting in substantially reduced WTP capacity. Monitoring revealed elevated algae counts in Lake Whatcom and that a slime produced by blue-green algae was responsible for the filter clogging. Most of the algae were tiny rod-shaped and spherical Cyanobacteria that have been collectively referred to as *Aphanocapsa* and *Aphanothece* – or more commonly "blue-green" algae. These do not appear to produce algal toxins. They are, however, extremely slimy because the individual cells are embedded in a thick, sticky mucilage.

Historical algae monitoring has shown that algae production in Lake Whatcom has been steadily increasing for the past decade. However, it is speculated that factors contributing to elevated algae in Lake Whatcom during the summer of 2009 included: (1) a very large January rain storm event in the Lake Whatcom watershed, (2) extended, record-setting hot summer weather, and (3) discontinued diversion of Nooksack River water because of blockage of the Nooksack River intake resulting from the January 2009 rain storm event. While similar elevated algae counts and lowered filtration capacity was observed in the summers of 2010 and 2011, it was not as severe as in 2009. There was not a problem meeting customer demand in the summers of 2010 and 2011. It should be noted that weather, storm, and Nooksack River diversion conditions were all different in 2010 and 2011 than in 2009.

A key indicator of WTP capacity is the measure of Unit Filter Run Volume (UFRV). UFRV is the measure of how much water is passed through a filter before that filter becomes clogged to the point that it has to be cleaned by backwashing. UFRVs at the WTP typically range from 7,000 to 10,000 gal/sf during late winter and spring to 2,000 to 3,000 gal/sf and sometimes lower during mid-to late summer. During early August of 2009, UFRVs dropped to below 900 gal/sf on several days in a row. At that point, filter run times were down to an average of 3.5 hours and a new filter was being be placed into backwash mode every 30

minutes. These short filter runs and increased backwashing frequency lowered WTP capacity below customer water demand and at that time the voluntary water restriction program was marketed more heavily than normal to encourage reduction in customer water demand. After voluntary water restrictions were deemed insufficient, mandatory water restrictions were implemented. Within two days, mandatory water restrictions sufficiently reduced demand to below WTP capacity.

The WTP UFRVs over the past few years is presented in Figure 2-1 below.

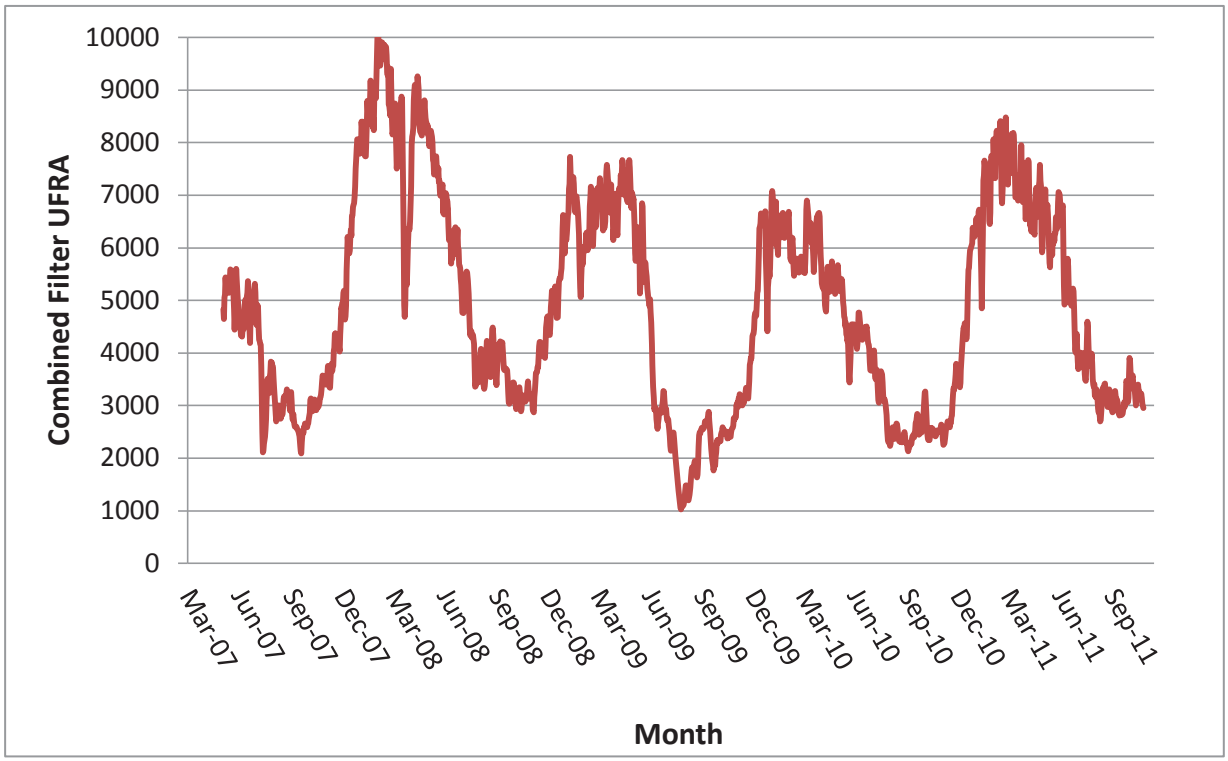


FIGURE 2-1
Trend of Plant Unit Filter Run Volumes

2.2. Lake Whatcom Water Quality

Current and recent Lake Whatcom water quality is documented annually by Western Washington University (WWU) in collaboration with the City. Annual reports of this documentation dating back to the 1990s can be found at www.wwu.edu/iws. Each annual report comprises the historical data of the previous year's report. So, the latest report comprises the entire historical water quality record that is available.

The City and WWU have collaborated on Lake Whatcom water quality monitoring since the early 1960s. In 1981, the City and WWU began regular data collection on: temperature, pH, dissolved oxygen, conductivity, turbidity, nutrients (nitrogen and phosphorus), and other representative parameters. The primary objective this monitoring effort is to provide a record of Lake Whatcom's water quality over time and identify water quality trends. Water

quality data have been collected at several sites in Lake Whatcom. The latest available water quality data for Lake Whatcom at the City's WTP intake is presented in Table 2-1. Table 2-1 is a duplication of Table 3 of the "Lake Whatcom Monitoring Project 2009/2010 Final Report."

TABLE 2-1
Summary of Lake Whatcom Water Quality at City Intake (Water Quality Data Year Oct. 2009 – Sept. 2010)
(This table is excerpted from Table 3 of "Lake Whatcom Monitoring Project 2009/2010 Final Report")

Parameter	Minimum	Median	Mean[1]	Maximum	No. of Samples
Alkalinity (mg/L $CaCO_3$)	18.0	19.0	19.2	20.8	30
Conductivity (μS/cm)	56.8	58.1	58.2	60.3	110
Dissolved oxygen (mg/L)	9.2	10.9	10.7	12.3	110
pH	7.2	7.8	7.7	8.3	110
Temperature (^{o}C)	6.8	12.7	13.2	21.7	110
Turbidity (NTU)	<2	<2	<2	<2	30
Nitrogen – ammonium (μg-N/L)	<10	<10	<10	13.3	30
Nitrogen – nitrate/nitrite (μg-N/L)	100.9	254.3	231.4	355.8	30
Nitrogen – total (μg-N/L)	239.4	388.8	369.9	480.9	30
Phosphorus – soluble (μg-P/L	<5	<5	<5	11.0	30
Phosphorus – total (μg-P/L)	<5	<5	<5	13.5	30
Chlorophyll (μg/L)	2.0	3.4	3.5	5.9	30
Secchi depth (m)	4.3	5.4	5.5	7.0	10
Coliforms – fecal (cfu/100 mL)[2]	<1	1	1	1	10

1 Uncensored arithmetic means except coliforms (geometric mean);
2 Censored values replaced with closest integer (i.e., <1 $\Rightarrow$ 1).

In addition to the data presented in Table 2-1, data for total organic carbon (TOC), metals, and algae are also presented in the annual Lake Whatcom water quality reports. Current TOC in Lake Whatcom at the existing WTP intake ranged as follows (per data collected by WWU):

- February 9, 2010: 1.4 mg/L at the surface and 4.6 mg/L at a depth of 10 meters
- August 5, 2010: 2.6 mg/L at the surface and 8.0 mg/L at a depth of 10 meters

It should be noted that raw water TOC measured at the WTP has been shown to be very consistent over the years, as presented in Table 2-2.

The TOC data presented in Table 2-2 averages 2.1 mg/L with only three annual maximums exceeding 3.0 mg/L.

The only metals that were measured at the WTP intake site above detection limits were iron and zinc – at very low levels. No year-to-year trend has been indentified in Lake Whatcom for metals.

TABLE 2-2
WTP Historical Raw Water TOC (mg/L)

Year	Annual Average	Annual Maximum
2000	1.8	2.4
2001	2.2	3.9
2002	2.1	3.1
2003	2.3	4.1
2004	2.1	2.3
2005	2.2	2.7
2006	2.2	2.5
2007	2.1	2.6
2008	2.1	2.4
2009	2.2	2.4
2010	2.1	2.7
2011	2.1	2.3

Algae counts for various algae types are presented in the annual Lake Whatcom water quality reports. These counts show the relative breakdown of algae types for a given sample. Recent algae counts show blue-green algae to be the dominant type in terms of counts. These blue-green algae are known to be primary filter-clogging algae. Algal counts are difficult to measure accurately and consistently from sample to sample and are time consuming. Therefore they are not done on a daily basis to observe changes in the source water. Chlorophyll is an indirect measure of algal biomass and is an effective parameter for assessing changes in biological productivity of a lake on a daily basis. Chlorophyll does not exhibit a consistent relationship with algal counts.

While some of the parameters presented above have remained relatively steady over the years, long-term Lake Whatcom monitoring reveals a few trends that are reflective of conditions that favor or are reflective of increased algae growth. These trends include:

- **Dissolved Oxygen (DO):** DO has been trending lower in the lower parts of Lake Whatcom – in the hypolimnion. Basins 1 and 2 already exhibit severely anoxic conditions. In 2010, Basin 3 was shown to have some lower DO values at depth. The decline in DO is increased algae in the upper parts of the Lake – the epilimnion. These algae then die and fall into the hypolimnion and are consumed by bacteria that also consume DO.

- **pH:** Variation in pH values between daytime highs and night time lows have increased. pH increases with photosynthesis. Increased algae increases photosynthesis.

- **Nitrogen:** Dissolved nitrogen has trended lower in the epilimnion. Lower concentrations of dissolved nitrogen reflect increased algae growth.

- **Chlorophyll:** Chlorophyll has trended substantially upward (approximately doubling since 1994), reflecting increased algal biomass.

- **Phosphorous:** Overall Phosphorous concentration has trended upward in Lake Whatcom. However, total phosphorous is difficult to accurately track at any given time because it transitions between soluble and insoluble forms via consumption by algae as well as other processes. Phosphorous is the limiting nutrient for algae production in Lake Whatcom, and as it relates to algae growth is the most concerning of all the water quality parameters collected.

- **TOC:** Despite some variation reported by WWU at the existing WTP intake at 10 meters depth in Basin 2, TOC measured in the raw water at the WTP has remained relatively consistent since 2000.

- **Algae:** Algae has trended upward. As stated above, Chlorophyll is an indirect measure of algal biomass, which reflects the total mass of all combined algae types – of which there are many. Essentially all of the individual algae types monitored show increases over time to current. Blue-green algae of the type that are believed primarily responsible for filter clogging at the WTP have shown substantial increases since initial monitoring in the early 1990s. Similar to Chlorophyll, blue-green algae have roughly doubled in concentration since 1994.

Most of the conditions, activities, and factors in the Lake Whatcom watershed that have contributed to a downward trend in water quality parameters remain in place. Efforts to reverse this trend are well underway, but are expected to take years to have measurable beneficial impact.

2.3. Disinfection By-Products

The City's WTP consistently meets all state and federal regulations currently in place for potable drinking water – even during periods of reduced production due to algal blooms. However, one area of specific interest over the past 15 years within the municipal drinking water industry is disinfection by-products (DBPs), which result from the chlorination of drinking water. DBPs occur when water that contains organic material (measured as TOC) reacts with chlorine, which is added for disinfection.

The primary DBPs regulated by the state and federal government are Total Trihalomethanes (TTHMs) and Haloacetic Acids (HAA5s). Figure 2-2 shows the trend of these DBPs in the City's water system. As shown, the City is well below the standards of 0.080 mg/L for TTHMs and of 0.060 mg/L for HAA5. Therefore, meeting the current regulations is not of concern. Even changes in monitoring and reporting requirements in 2012 related to compliance at each specific sampling site, as opposed to averaging over all sampling sites, are not anticipated to present a concern with respect to compliance.

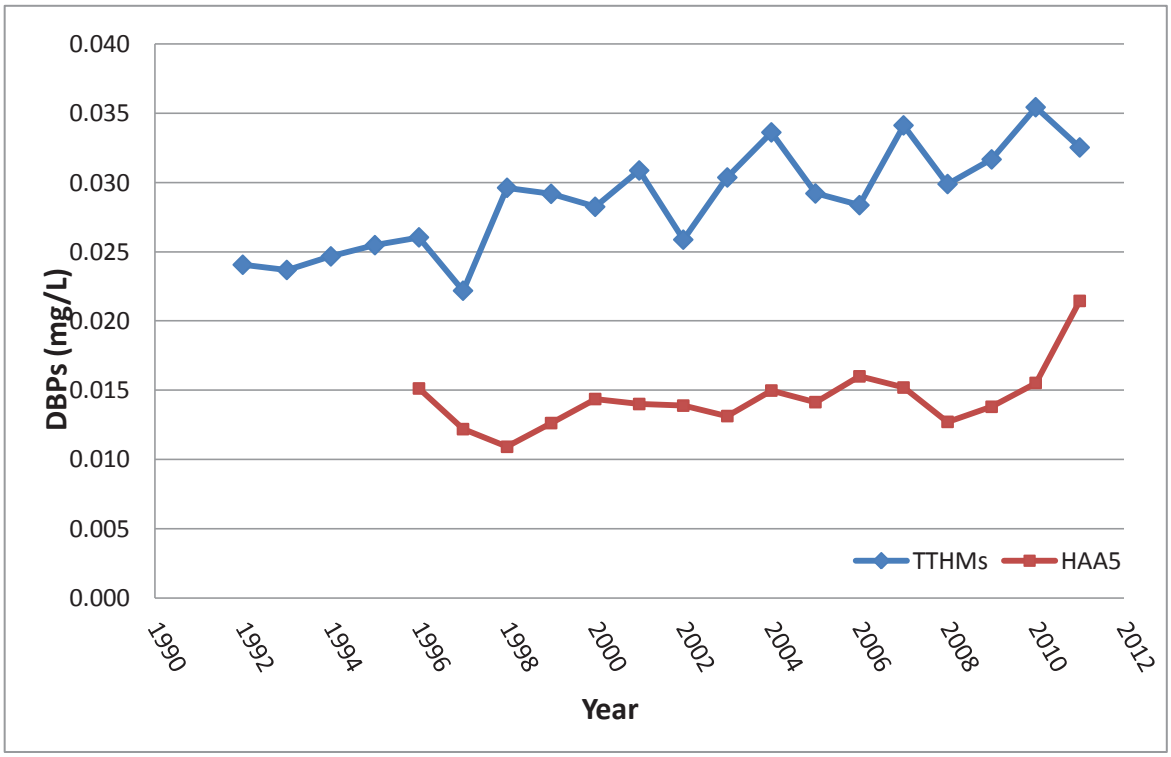


FIGURE 2-2
Historical DBPs in the City's Water System

However, it is important to note that the long-term trend of TTHM formation is increasing. The trend of HAA5 has been flat since monitoring began in 2001. In the future, the regulations may tighten and reduce the permitted levels of TTHMs and HAA5s. While the City's DBP levels may remain below regulatory requirements for some time to come, increases in DBPs reflect a level of source and finished water quality degradation.

2.4. Historical Lake Whatcom Management and TMDL Study

Lake Whatcom management surfaced as a major focus in the 1980s. Since then, several key management efforts, studies, and programs have been undertaken to address concerns about lake water quality. These include:

- In 1981, the City, Whatcom County, and Water District No. 10 (now Lake Whatcom Water and Sewer District) discussed jointly sharing local match contribution for a state grant to conduct a Lake Whatcom Restoration Study. Those early discussions led to the first Lake Whatcom Watershed Management Plan (LWWMP), which was released in draft form in late 1986 and revised in 1987. The LWWMP identified management actions to address key watershed issues. However, the LWWMP concluded that even though phosphorus was the limiting nutrient in Lake Whatcom, the lake would experience no significant change in water quality – even under the most intensive land use scenario evaluated.

- In 1986, the City completed a study of Lake Whatcom's continued use as a water source. The study concluded that water quality at the time was very good and would continue to meet water quality standards into the future.

- In the late 1980s, at Whatcom County's request, Washington Department of Natural Resources negotiated a land exchange that brought 7,500 acres in the Lake Whatcom watershed into public ownership.

- In 1992, the City, Whatcom County, and Lake Whatcom Water and Sewer District adopted the "Lake Whatcom Management Policies" by joint resolution (Whatcom County No. 92-73; City of Bellingham No. 92-68; Lake Whatcom Water and Sewer District No. 560). The policies included several goals and focus areas that were later consolidated into program areas, such as land preservation, stormwater, recreation, etc.

- The current Lake Whatcom Management Program was established in 1998 by Interlocal Agreement between the City, Whatcom County, and Water District No. 10. The goal of the program was to jointly manage and implement programs affecting the Lake Whatcom watershed and that continues to be the primary program goal today.

- In 2000, the Inter-jurisdictional Coordinating Team (ICT) was created to help coordinate the activities and programs from the Lake Whatcom Management Program. The ICT continues to meet regularly, and is comprised of staff from the City, Whatcom County, and Lake Whatcom Water and Sewer District (LWWSD). The ICT continually evaluates program effectiveness and reviews the progress of tasks identified for the five-year Lake Whatcom Management Program.

Despite coordinated historical lake management efforts, Lake Whatcom water quality continues to deteriorate. Phosphorous entering the lake from residential development, forest practices, other human-caused sources, and natural sources has been identified as the key factor leading to this deterioration. Increased phosphorus entering the lake has resulted in widespread seasonal algal blooms and dissolved oxygen deficits.

In 1998, Lake Whatcom water quality failed to meet the Washington State dissolved oxygen standard and was placed on Washington's list of polluted waters (Section 303d of the Clean Water Act). Section 303d states that in lakes, human actions may not decrease the one-day minimum oxygen concentration more than 0.2 mg/L below estimated natural conditions. As a result of the listing of Lake Whatcom per Section 303d, Ecology initiated a Total Maximum Daily Load (TMDL) study to determine what needed to be done to restore lake water quality. The TMDL study was completed by Ecology in November 2008. The TMDL study showed that human actions were causing an exceedance of this dissolved oxygen standard.

Although there are no specific numerical standards or criteria for phosphorous, phosphorous was listed for Lake Whatcom per Section 303d. The TMDL addressed total phosphorus as the primary cause of reduced dissolved oxygen. Previous study had shown that phosphorous is the limiting nutrient for algae growth in Lake Whatcom. Increased algae growth is the cause of reduced dissolved oxygen.

In response to these listings, a Total Maximum Daily Load (TMDL) study was initiated by Washington State Department of Ecology (Ecology) to determine the amount of phosphorous reduction needed to return Lake Whatcom to acceptable water quality standards. The TMDL study was completed in 2008.

The TMDL study computed that approximately 86 percent of developed acreage in the watershed would need to be returned to "natural" conditions to achieve the phosphorous reduction goal. The amount of phosphorous reduction computed to meet the TMDL goal is approximately 1,100 kilograms (2,400 pounds)/year.

Compliance with the TMDL is being pursued by the City, Whatcom County, and LWWSD through the Lake Whatcom Management Program (LWMP). The LWMP's 2010-2014 Work Plan was submitted to Ecology in 2010 to satisfy the requirements of the Summary Implementation Strategy, which is also the first step in the development of a Detailed Implementation Plan (DIP). The DIP details how TMDL compliance will be achieved. Specifically, it will identify phosphorus reduction activities, the implementation schedule for those activities, the cost of implementation (annual and total), and the period of time to achieve TMDL compliance.

In 2012 Ecology is planning to include TMDL compliance as part of the new NPDES stormwater program requirements. Completion of the DIP and ongoing assessment of implementation actions and monitoring will be NPDES permit requirements. The Lake Whatcom stakeholders acknowledge that meeting the TMDL requirements for phosphorous and dissolved oxygen is expected to take many years to complete. Meeting the TMDL requirements is therefore the cornerstone of the long-term response to improving lake quality, including reducing algae concentrations. Consequently, relying solely on lake management to achieve reduced algae growth and associated algae impacts at the City's WTP would not be an effective short-term mitigation strategy.

2.5. Historical Water Demand

Any alternative approach to mitigating the adverse algae impacts must be implemented in consideration of the City's current and projected water usage. Assessing projected water usage is facilitated by reviewing historical water demand.

Like many municipalities in western Washington State, the City's water system demand has held steady or declined in the last 10 to 20 years. The primary reason observed throughout western Washington State for this decline is the reduction in per-capita water usage. That reduction is related to a variety of water conservation efforts, including in-home water – reduction devices and reduced outdoor watering. The greatest single contributor has been shown to be reduced summertime outdoor watering. It is this reduced summertime outdoor watering that has the biggest reduction impact in peak day water use (maximum day demand [MDD]). In addition, reduced economic activity for businesses and industries that have traditionally used large amounts of water has also contributed to this decline. A key stimulus in the reduction of per-capita water demand is the overall heightened awareness of the need to conserve water. That awareness has been brought about, in part, through years of education as well as several drought periods during the 1990s and early 2000s.

What has been observed to be an even greater stimulus in reducing per-capita and overall water use is increases in the pricing of water, particularly as it relates to the consumption portion of pricing structures. Increasing-block rate structures that result in higher usage charges, as greater volumes of water are consumed, have greatly curbed summertime water use. Most of the City's residential customers do not have service meters. Therefore, the impact of such rate structures is not applicable to the City of Bellingham. However, service meters are planned to be installed on these un-metered customers by 2017. Rate-impact stimulus on per-capita water use reduction could become evident at that time.

The City's historical municipal water demand and associated service connections are presented in Figure 2-3. The data in Figure 2-3 reflects demand for treated water from the Whatcom Falls Water Treatment WTP and does not include past non-potable water demand from the old Georgia-Pacific paper mill or from the Puget Sound Energy co-generation facility. Figure 2-3 presents the steady, somewhat declining, average day demand (ADD) in contrast to the steady increase in service connections. More noticeable is the greater decline in maximum day demand (MDD) since 1990.

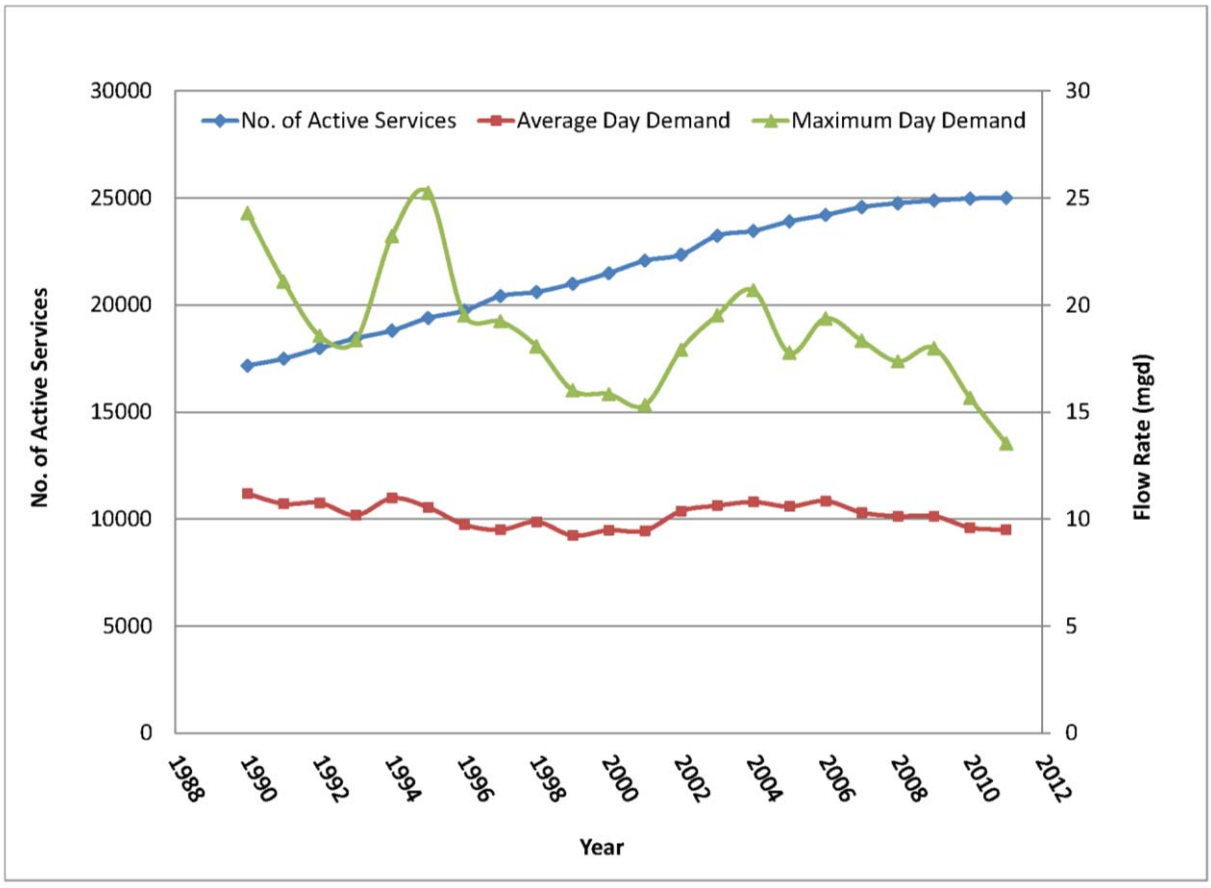


FIGURE 2-3
Historical City of Bellingham Municipal Water Demand

Supply, treatment, and pumping systems are typically designed to meet the anticipated MDD. In the case of supply and treatment, water that is used as a byproduct of treatment,

must also be included. For example, approximately 4 percent of the water entering the City's WTP goes to backwashing filters and subsequent ripening prior bringing the filters on line for production. The hydraulic capacity of the alternatives considered herein for mitigating the adverse algae conditions must be sized to meet the current and projected MDD of the City's municipal water system. In addition, sizing must also address other anticipated water uses, such as selling treated water to other municipalities in Whatcom County on a wholesale basis or possibly non-potable (non-treated) water to potential future industries within the City.

2.6. Algal Impacts on Northwest Water Utilities

In the past five years, much like the City of Bellingham, several Northwest water utilities have experienced significant impacts resulting from algal blooms in their source water. A summary of these utilities, including the City of Bellingham, and the algal impacts is presented in Table 2-3. Further discussion of these algae issues, with the exception of the City of Bellingham's, is presented in subsections below.

TABLE 2-3
Summary of Algal Impacts on Selected Northwest Water Utilities

Treatment Plant	Capacity (mgd)	Existing Treatment	Algal Impacts	Mitigation Considered
Bellingham, WA – Lake Whatcom WTP	25	In-Line Filtration	Reduced production from filter clogging algae	Evaluating several alternatives
Everett, WA – Lake Chaplain WTP	120	Direct Filtration	Reduced production from filter clogging zooplankton	Evaluating DAF and lake management
Seattle, WA – Cedar WTP	180	Ozone – Ultraviolet Disinfection	Reduced production, clogging of screens, valves, meters, and monitoring equipment	Evaluating several alternatives
Joint Water Commission WTP (Hillsboro, OR)	75	Conventional Filtration	Taste and odor events, screen clogging, and detection of algal toxins in source water	Selected ozone and biological filtration
Salem, WA – Geren Island WTP	100	Slow Sand Filtration	Reduced production from filter clogging algae	Evaluating alternatives including DAF
Medford, OR – Duff WTP (Medford Water Commission)	45	Conventional Filtration with Ozone	Detection of algal toxins in up-stream sources	Monitoring and ozone disinfection

2.6.1. Everett – Lake Chaplain WTP

The City of Everett treats water from Chaplain Reservoir by direct filtration with alum as a primary coagulant. Chlorine and soda ash are added after filtration for disinfection and corrosion control. Since 2004, the City has experienced five episodes of shortened filter run times as a result of holopedium, a zooplankton that feeds on freshwater algae.

The episodes reduced average filter run times from approximately 30 hours to a range of 6 to 20 hours. Of the five episodes since 2004, two were strongly correlated to holopedium, two were weakly correlated to holopedium, and one may be correlated to daphnia.

Contributing factors were identified in some cases. For example, in 2004 and 2007 high winter turbidity was experienced in the reservoir which is believed to have added additional nutrients. In 2005, unusually warm winter temperatures may have contributed to the increase in holopedium concentration. The City is currently evaluating mitigation measures, including:

- Raw water screening
- Dissolved air flotation
- Relocating the intake to deeper water
- Adding surface wash to filters
- Introducing "zooplanktivorous" fish
- Lighting areas of the lake to attract holopedium
- Adding calcium to the reservoir

2.6.2. Seattle – Cedar WTP

The City of Seattle's treats its unfiltered Cedar supply using ozone, UV disinfection, chlorination, fluoride, and lime. Water for the Cedar WTP is typically withdrawn from the Lake Youngs reservoir. A temporary, backup alternative is bypassing of Lake Youngs, whereby water is withdrawn directly from the Cedar River. This backup approach can only be implemented when Cedar River turbidity is low.

In 2008, 2010, and 2011 large blooms of cyclotella that formed dense filaments were experienced in Lake Youngs. The 2008 bloom was the first incident of identified cyclotella in the City's Lake Young's reservoir. The blooms resulted in clogging of treatment equipment (analyzer screens, ozone cooling system, chemical feed pump strainers, fish screens, flow meters) as well as downstream distribution system clogging of PRV pilots, meter screens, distribution analyzers, and even customer washing machines.

Contributing factors were identified as nutrient inputs to the lake from storm events via the Cedar River and upstream fluoride addition on incoming water to the lake, which contributes phosphorous as a byproduct to the fluoride.

The City is currently evaluating mitigation measures that include:

- Temporarily bypassing Lake Youngs during blooms
- Physical changes to prevent equipment and instrument clogging
- Nutrient reduction strategies (relocating fluoride addition, reduced input to Lake Youngs during storms, hypolimnetic withdrawal)
- Installing continuous monitoring stations

- Improved management of invasive species

2.6.3. Joint Water Commission (Hillsboro, OR)

The Joint Water Commission (JWC) is a water supply commission jointly owned and operated by the Oregon cities of Hillsboro, Beaverton, and Forest Grove, as well as the Tualatin Valley Water District. JWC's water treatment plant treats water from the Tualatin River using a conventional water treatment process (flocculation, clarification, and filtration), with alum as a primary coagulant and pre-chlorination in the sedimentation basins to control algae.

In 2008 and 2010, JWC experienced significant taste and odor events at the water treatment plant related to upstream algae. The taste and odor events were treated with the addition of powdered activated carbon. While helpful, this approach did not completely eliminate the problem. During the 2008 taste and odor event, the algal toxin, microsysten-LR, was detected in the Tualatin River, which raised additional concerns about algal toxin impacts.

JWC is currently pilot testing ozone and biological filtration for taste and odor control, as well as algal toxin destruction.

2.6.4. Salem, OR – Geren Island WTP

The City of Salem, OR treats water from the Santiam River using slow sand filtration. The City has four 5-acre slow sand filter cells and two unlined cells that are used as a pretreatment roughing filter. Cleaning each cell is a labor-intensive and time-consuming process that typically takes twelve days to scrape the top layer of filtered material (schmutzdecke), add sand, and ripen the filter.

In 2009 and 2011, the City experience algal blooms that reduced filter runs to as short as three days, which is very poor for slow sand filtration facilities. The City of Salem developed an accelerated cleaning procedure to put the sand filters back on line within seven days of scraping. Still, there were supply shortfalls during the algae bloom. In 2009, the City first experienced even more extensive filter clogging and since that time it has put a monitoring program in place for algae and has begun evaluating mitigation measures. These measures include limiting light to the active slow sand cells as well as pretreatment using dissolved air flotation.

2.6.5. Medford, OR – Duff WTP (Medford Water Commission)

The Medford Water Commission (MWC) treats water from the Rogue River using ozone and conventional filtration. In 2009, 2010, and 2011 health advisories were listed on Lost Creek Lake, which is located approximately 30 miles upstream from Medford, OR on the Rogue River. The health advisories were issued because of high levels of cyanobacteria levels in the lake. Testing for algal toxins in the lake confirmed the presence of microcystis-LR and anatoxin-a. There are currently no state or federal guidelines or maximum contaminant levels for these compounds, and MWC is proceeding pro-actively and cautiously – in close coordination with Oregon State regulators.

In 2002, MWC installed ozone at its water treatment plant for taste and odor control and operates its ozone system at low dosages to meet these needs. However, ozone can also be used to effectively destroy algal toxins at similar low dosages. MWC is pro-actively monitoring for algal toxins at its plant and upstream in Lost Creek Lake and continuing close coordination with Oregon State regulators.

3. Description of Alternatives

The alternatives considered for mitigating clogging of the filters at the City's WTP are grouped into three main categories, treatment, intake, and lake management. Descriptions of these alternatives, in their respective categories, are presented in the following subsections.

3.1. Treatment Alternatives

The City has long reaped the benefits of having such a high quality water supply as Lake Whatcom. When the existing WTP was constructed in 1968, the Lake Whatcom supply only required the addition of a single coagulant chemical to enable effective filtration – followed by disinfection. This type of water treatment plant is referred to as "in-line filtration" because it does not include either a flocculation process or a clarification process prior to the filters. In-line filtration is a low-cost filtration approach that is only suitable to low-turbidity water supplies like Lake Whatcom. Effective operations of in-line filtration, such as is the case at the City's WTP, results in lower overall treatment costs as opposed to other treatment systems that include flocculation and clarification. Because high-quality water supplies like Lake Whatcom are not typical, the City is one of only a few communities that are supplied from in-line treatment plants.

While Lake Whatcom continues to be a high-quality, low-turbidity supply, the increasing presence and concentration of seasonal algae could potentially drive the City to implement treatment prior to filtration. Treatment prior to filtration is typically referred to as "pretreatment." Pretreatment is common throughout the municipal treatment industry and oftentimes is comprised of a clarification process.

Several treatment alternatives were proposed for evaluation. Each treatment alternative would be sited somewhere at the existing WTP site. Each treatment alternative, except the "Additional filters" alternative, are "pretreatment" alternatives in that they would be incorporated upstream of the existing filters to remove algae and particulate material prior to filtration.

For the purpose of this study, the pre-treatment alternatives are assumed to have a capacity of 30 mgd which roughly matches the capacity of the Whatcom Falls Water Treatment Plant with all six filters operating. It should also be noted that each of the pre-treatment alternatives could be designed to be expandable in the future. Should the City decide to move forward with design and implementation of one of the treatment alternatives, sizing criteria would be based on the latest projections of customer demand at that time.

Each of the treatment alternatives considered for this study are commonly used in the municipal water treatment industry and are commonly-considered alternatives for algae removal. They are not, however, equal with respect to their removal performance, advantages, disadvantages, and cost. The treatment alternatives considered for this study include:

- Dissolved Air Flotation (DAF)
- Ballasted Sedimentation (Actiflo®)
- Plate and Tube Settling

- Upflow Clarification (Superpulsator®)
- Conventional Sedimentation
- Micro-Screening
- Ozonation
- Additional Filters

3.1.1. Dissolved Air Flotation (DAF)

DAF was first used as a pretreatment for conventional granular media in South Africa and Scandinavia in the 1960s and became more widely used worldwide in the 1980s and 1990s. DAF has become relatively common in the U.S. because it provides a cost-effective alternative to conventional sedimentation, including where removal of algae is necessary. There are over 30 municipal installations in North America with capacities greater than 5 mgd in operation, the largest of which is a 200 mgd plant in New Jersey.

In the DAF process, the solids are separated out by floating the floc to the water surface, as opposed to settling the floc to the bottom of the basin. After the flocculation process, DAF introduces air bubbles at the bottom of a contactor to float the floc. The air bubbles are produced by reducing pressurized recycle water stream saturated with air to ambient pressure.

The "float" is scraped or floated from the top of the reactor, and the clarified water is removed via underflow channels at the bottom of the reactor. A schematic of a typical DAF unit is provided in Figure 3-1.

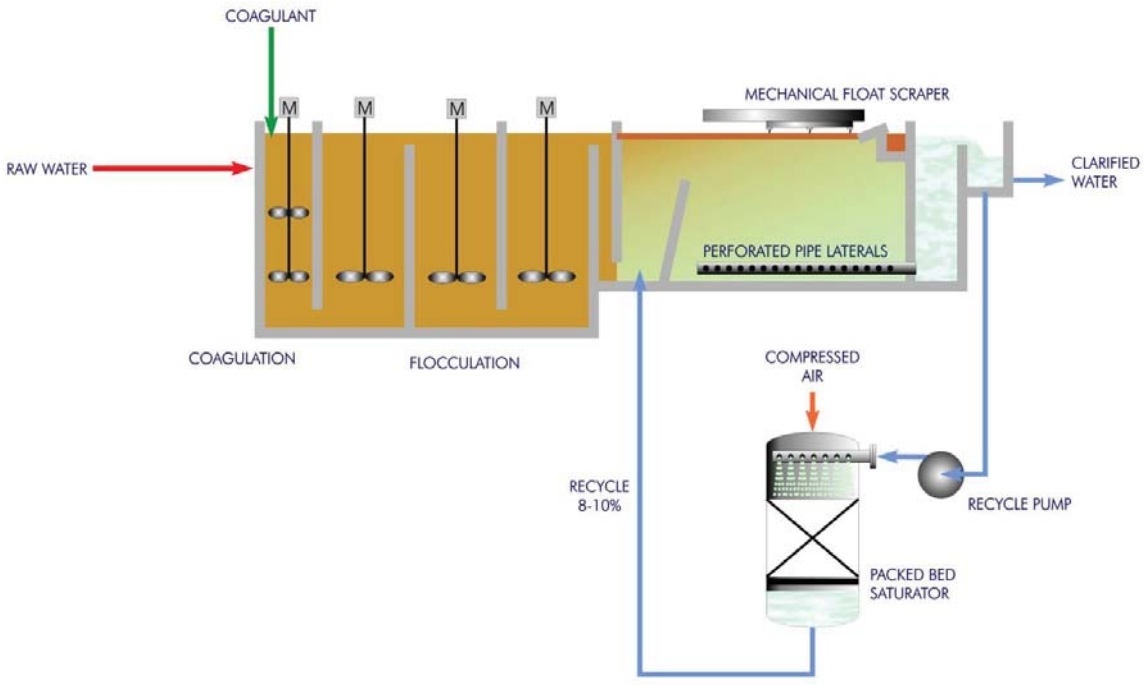


FIGURE 3-1
DAF Schematic

Advancements in DAF technology have enabled increasing loading rates from a high of 8 gpm/sf for "standard" systems to "high-rate" DAF systems that can be run up to 16 gpm/sf or higher. Three manufacturers provide high-rate DAF systems in North America for municipal water treatment above 5 mgd. These manufacturers and their associated DAF models are:

- Infilco Degremont: AquaDAF®
- ITT Leopold: ClariDAF®
- Roberts Filter/Enpure: EnfloDAF®

The first high rate DAF systems were introduced in North America about 10 years ago, and now most of the DAF systems being installed are "high-rate" systems. Each manufacturer has developed modifications to the traditional DAF system to allow for higher surface loading rates. These include:

- AquaDAF®: false floor with orifice plates and float basin with width larger than length
- EnfloDAF®: deeper float basin and patented dispersion nozzles
- ClariDAF®: modification to orifices in collection laterals

A summary of the advantages and disadvantages of DAF for mitigating the adverse impacts of filter-clogging algae is presented in Table 3-1.

TABLE 3-1
Advantages and Disadvantages of DAF

Advantages	Disadvantages
DAF is very effective at removing algae because of algae's low density and propensity to float.	DAF is less compatible with the addition of powdered activated carbon (PAC) than other clarification processes because PAC tends to settle and DAF is a flotation process. This disadvantage relates to DAF's potential future utility to mitigate taste and odor, algal toxins, and other contaminants that could potentially impact the City's supply.
Flocculation for DAF typically requires only 5 to 10 minutes of detention time, which is less than for conventional settling and plate settlers.	DAF requires more energy than conventional sedimentation and plate settling
The DAF process is less likely to lyse (rupture) algal cells than Actiflo, ozonation, and micro-screening; thus, reducing the potential to release algal toxins and produce taste and odor.	DAF includes more mechanical equipment than conventional sedimentation and plate settling.
DAF can produce more concentrated sludge using mechanical removal than other clarification processes.	
DAF typically operates at surface loading rates ranging from 8 to 16 gpm/sf. This high loading rate enables a smaller footprint than other clarification processes, except Actiflo.	
This process does not impart much additional headloss to the existing system. Therefore, pumping of the process flow stream is not required.	

3.1.2. Ballasted Sedimentation (Actiflo®)

Actiflo® is a proprietary process of high-rate clarification that uses microsand-enhanced flocculation and plate settling to produce a clarified effluent. Actiflo consists of a rapid mix chamber where a coagulant is added, followed by an injection chamber where microsand and a polymer are added (high-energy mixing environment), and then a maturation chamber (lower-energy mixing to build floc and attach to sand). Typical detention time for these three steps is about 6 minutes. Following these chambers, water enters the settling tank where the microsand-floc settles quickly. The process water is further clarified by flowing upward through settling tubes and into effluent channels. Total Actiflo retention time is between 10 and 15 minutes. The microsand sludge at the bottom of the settling tank is pumped to a hydrocyclone, where it is separated from the sludge by centrifugal force. The sand is then returned to the head of the process for reintroduction in the injection chamber. The separated sludge is removed at concentrations of 0.1 to 0.2.

A schematic of the Actiflo process is presented in Figure 3-2.

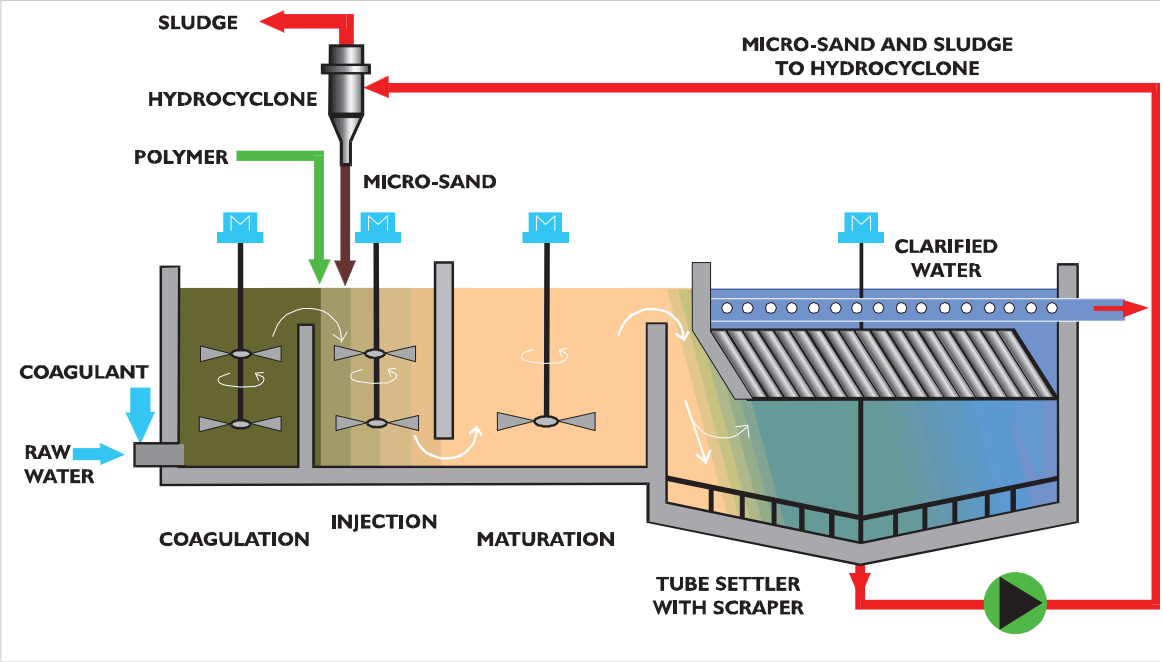


FIGURE 3-2
Actiflo® Process Schematic

A summary of the advantages and disadvantages of ballasted sedimentation for mitigating the adverse impacts of filter-clogging algae is presented in Table 3-2.

TABLE 3-2
Advantages and Disadvantages of Actiflo

Advantages	Disadvantages
Actiflo operates at very high loading rates (15 to 25 gpm/sf) – higher than other clarification processes, which reduces facility footprint and associated cost.	Actiflo requires continual replenishment of sand because of sand losses from the sludge separation process. Lose sand would end up in the sanitary sewer system. Although the amount of sand would be minor to negligible, it could contribute to collection system pump wear.

TABLE 3-2
Advantages and Disadvantages of Actiflo

Advantages	Disadvantages
Although not currently an issue with the Lake Whatcom supply, Actiflo easily adjusts to changes in raw water quality, including large swings in raw water turbidity.	Like DAF, Actiflo requires more energy than other clarification processes.
Actiflo is compatible with the addition of powdered activated carbon (PAC), like other clarification processes that involve gravity settling. This advantage relates to the potential future need to mitigate taste and odor, algal toxins, and other contaminants that could potentially impact the City's supply.	Similar to DAF, Actiflo includes more mechanical equipment than other clarification processes.
Actiflo has been shown to be as effective as, or better than, other clarification processes except DAF at removing algae (DAF has been shown to be the most effective at removing algae). The key to this performance is the microsand.	Although Actiflo has been shown to be moderately effective at removing algae, it's high energy mixing combined with micro-sand addition have greater potential for lysing algal cells and releasing toxins or taste and odor compounds.
This process does not impart much additional headloss to the existing system. Therefore, pumping of the process flow stream is not required.	Actiflo can require high polymer dosages to be effective and these high polymer dosages can have a negative effect on downstream filtration processes.

3.1.3. Plate Settling

Inclined parallel plates or tubes are an enhancement of the traditional conventional sedimentation process that enables a substantial reduction in facility footprint from what conventional sedimentation requires. Loading rates for inclined plate settling can typically range from 2 to 4 gpm/sf based on facility footprint as opposed to 0.5 gpm/sf for conventional sedimentation. Both plates and tubes are used in the municipal water treatment industry. Plates tend to be more efficient, while tubes tend to be less expensive. For the purpose of this study, and because of the greater removal efficiency, this alternative is assumed to be comprised of inclined plates.

Inclined plate settling is accomplished in an open basin where water flow is conveyed in either of the following ways though the plates: (1) from top to bottom downward between the plates (co-current), (2) from bottom to top upward between the plates (counter-current), or horizontally from one side of the plates to the other (cross-current). Most new plate settling processes use a combination of cross- and counter-current flow by introducing the process water near the bottom of one side of the plates and withdrawing it at the top of the other side of the plates. A schematic diagram of a counter-current inclined plate settling process is presented in Figure 3-3.

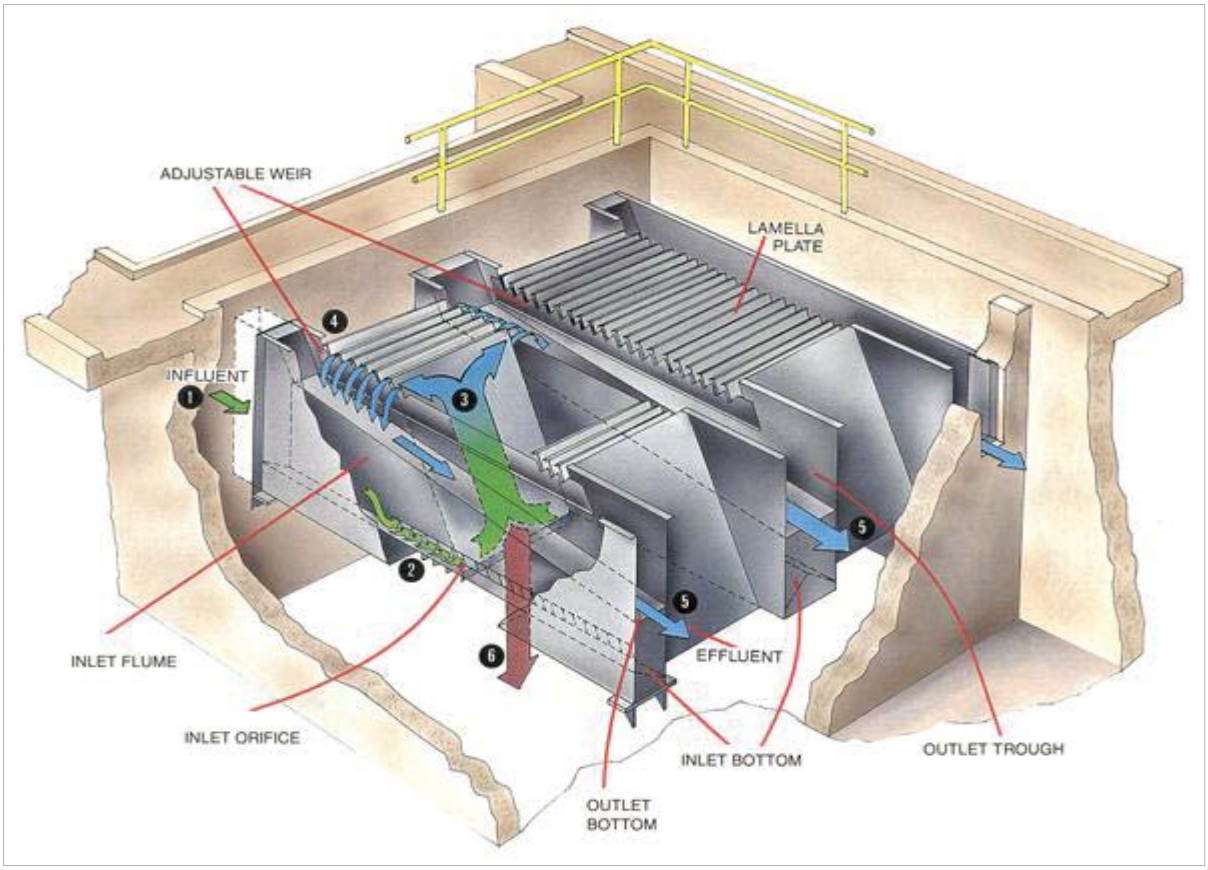


FIGURE 3-3
Counter-Current Plate Sedimentation

The material costs for the plates or tubes can vary depending on the materials required for the installation. Solids loading on surfaces and removal of solids can be a problem in some configurations. Similar to conventional sedimentation, 30 minutes or more of detention time in the flocculation process is necessary. Plate and tube settlers have been in use for many years in water treatment and are a widely accepted technology for settling of flocculated solids.

A summary of the advantages and disadvantages of plate settling for mitigating the adverse impacts of filter-clogging algae is presented in Table 3-3.

TABLE 3-3
Advantages and Disadvantages of Plate Settling

Advantages	Disadvantages
Less mechanical equipment and complexity than DAF, Actiflo, and SuperPulsator.	Moderate to poor effectiveness at removing algae when compared to DAF. Increased coagulant and polymer chemical are required to optimize algal flocc formation and settling because of low density of algae. Increased chemical usage will result in increased waste sludge for disposal.
Similar to conventional sedimentation, flocculation for plate settling requires 30 minutes or more of detention time.	Larger facility footprint than DAF, Actiflo, and SuperPulsator because it's loading rate is substantially less than these two other clarification processes and requires more flocculation time.

TABLE 3-3
Advantages and Disadvantages of Plate Settling

Advantages	Disadvantages
Like conventional sedimentation and DAF, plate settling is less likely to lyse (rupture) algal cells than Actiflo, ozonation, and micro-screening; thus, reducing the potential to release algal toxins and produce taste and odor.	
This process does not impart much additional headloss to the existing system. Therefore, pumping of the process flow stream is not required.	
Plate settling is compatible with the addition of powdered activated carbon (PAC), like other clarification processes that involve gravity settling. This advantage relates to the potential future need to mitigate taste and odor, algal toxins, and other contaminants that could potentially impact the City's supply.	

3.1.4. Upflow Clarification

Upflow clarification combines flocculation and sedimentation into a single unit process. It is preceded by rapid mixing where a coagulant chemical is added. Eliminating the separate flocculation process reduces facility footprint. Upflow clarification maintains a large, set volume of flocculated solids within the unit, which further enhances flocculation by forcing inter-particle collision and agglomeration. The flocculated solids form what is referred to as a "solids blanket." Cohesion of the blanket is achieved through the use of coagulant and polymer addition – additional to the rapid mixing process.

Upflow clarifiers typically operate at a relatively high loading rate (2 to 4 gpm/sf) with respect to conventional sedimentation, similar to plate settling. The facility footprint of upflow clarification is less than that for plate settling because of the elimination of the flocculation process.

Degremont Technologies, a subsidiary of Suez Environment, manufacturers a popular version of the upflow clarification process for municipal water treatment, referred to as a "Superpulsator®" clarifier. The Superpulsator® clarifier uses a vacuum pump and vacuum chamber to produce a "pulsing" effect within the solids blanket, which serves as the flocculation zone. Pulsing expands the blanket to increase the rate of inter-particle collisions. Inclined plates are included and are situated within the solids blanket. The inclined plates aid horizontal distribution of upward flow and enhance separation of the upward-flowing clarified water from the solids blanket that is held stationary. Clarified water flows upward from the sludge blanket and collects in effluent troughs.

A schematic diagram of an upflow clarifier is provided in Figure 3-4.

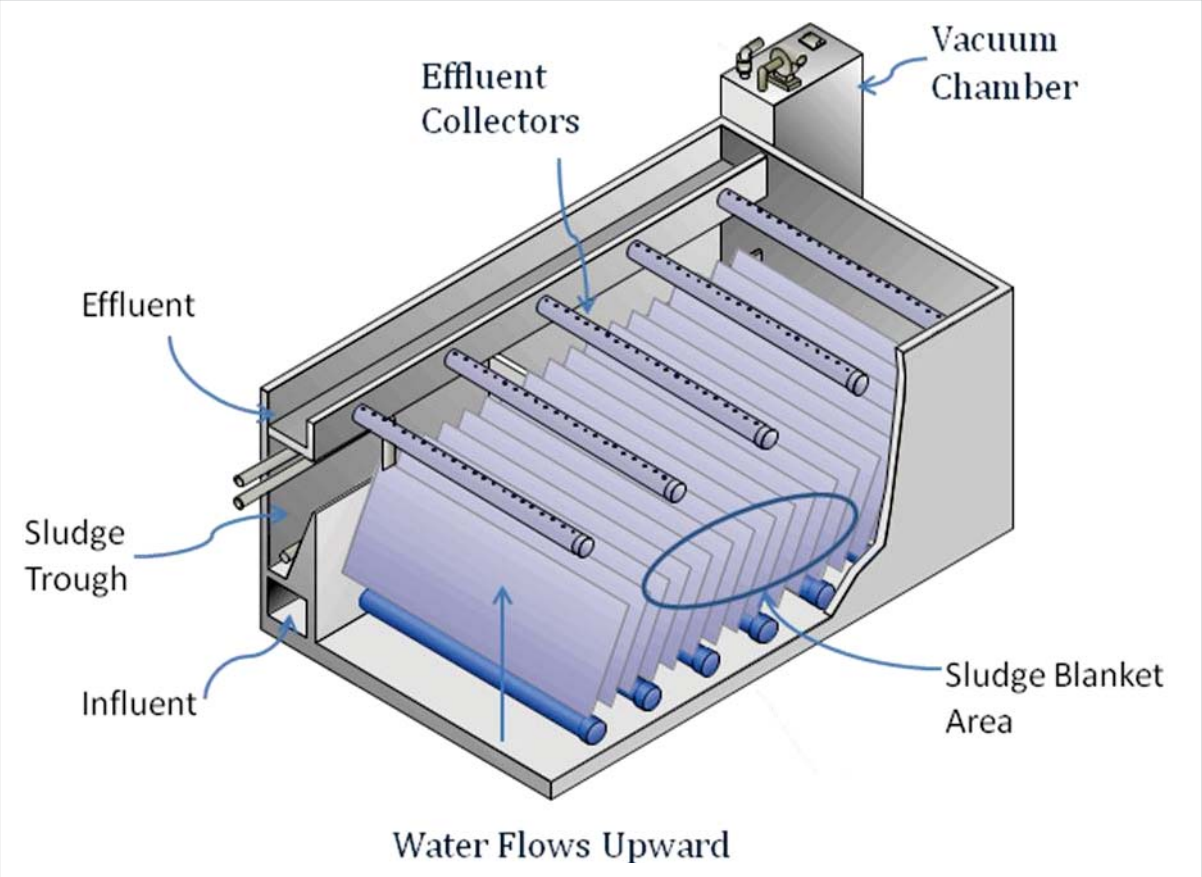


FIGURE 3-4
Upflow Clarifier Schematic

The solids blanket is maintained at a set height within the unit by use of a central, submerged solids overflow weir. As solids accumulate in the blanket, they continually overflow the submerged weir into a hopper that is evacuated at a set interval, thus removing solids from the process. Typical solids concentrations range from 0.5 to 2 percent in the concentrated sludge, depending on the solids residence time.

A summary of the advantages and disadvantages of upflow clarification for mitigating the adverse impacts of filter-clogging algae is presented in Table 3-4.

TABLE 3-4
Advantages and Disadvantages of Upflow Clarification

Advantages	Disadvantages
Slightly less mechanically equipment and complexity than DAF and Actiflo. No submerged moving parts.	Moderate effectiveness at removing algae when compared to DAF. Increased coagulant and polymer chemical are required to optimize algal flocc formation and settling because of low density of algae. Increased chemical usage will result in increased waste sludge for disposal.
No separate flocculation step. Flocculation occurs within the upflow clarification basin.	Larger facility footprint than DAF and Actiflo because it's loading rate is substantially less than these two other clarification processes.

TABLE 3-4
Advantages and Disadvantages of Upflow Clarification

Advantages	Disadvantages
Similar to conventional sedimentation, plate settling, and DAF, upflow clarification is less likely to lyse (rupture) algal cells than Actiflo, ozonation, and micro-screening; thus, reducing the potential to release algal toxins and produce taste and odor.	Although rapid changes in raw water quality are not typical in Lake Whatcom, upflow clarification does not respond well to such changing conditions. Operational challenges result with rapid changes in raw water quality.
This process does not impart much additional headloss to the existing system. Therefore, pumping of the process flow stream is not required.	Upflow clarification may require a period of one to two days of operation to establish the solids blanket for consistent effluent quality.
Upflow clarification is advantageous with the addition of powdered activated carbon (PAC) because of the lengthy detention time of the solids within the unit, which maximizes the time that PAC is in contact with the flowstream. This advantage relates to the potential future need to mitigate taste and odor, algal toxins, and other contaminants that could potentially impact the City's supply.	Polymer addition is typically necessary to maintain cohesion of the solids blanket.

3.1.5. Conventional Sedimentation

Conventional sedimentation has a long history of effective performance throughout the municipal water treatment industry in this country as well as world-wide. Because of its low loading rate (typically 0.5 gpm/sf) it occupies a large facility footprint as compared to other clarification processes. Detention times in conventional sedimentation basins is typically 3 to 4 hours to ensure effective settling, depending on how challenging the raw water material is to settle. High rate clarification processes such as DAF, Actiflo, plate and tube settling, and upflow clarification are all process that have been developed as higher-efficiency alternatives to conventional sedimentation.

Conventional sedimentation is preceded by two processes: (1) rapid mixing to effect particle coagulation using a coagulant chemical and polymer and (2) flocculation to develop flocc that settles effectively. Rapid mixing is a high-energy process with a detention time typically two minutes or less to connect particles and the coagulant chemicals. Flocculation is typically effected in two or three low-energy mixing stages of progressively-reduced mixing energy to produce large flocc that will settle effectively. Flocculation times of 30 minutes or more are typically required for effective flocc development.

A summary of the advantages and disadvantages of conventional sedimentation for mitigating the adverse impacts of filter-clogging algae is presented in Table 3-5.

TABLE 3-5
Advantages and Disadvantages of Conventional Sedimentation

Advantages	Disadvantages
Less mechanically equipment and complexity than high-rate clarification processes.	Moderate to poor effectiveness at removing algae when compared to DAF. Increased coagulant and polymer chemical are required to optimize algal flocc formation and settling because of low density of algae. Increased chemical usage will result in increased waste sludge for disposal.
Conventional sedimentation is compatible with the addition of powdered activated carbon (PAC), like other clarification processes that involve gravity settling. This advantage relates to the potential future need to mitigate taste and odor, algal toxins, and other contaminants that could potentially impact the City's supply.	Because of its low loading rate (0.5 gpm/sf), conventional sedimentation has a larger facility footprint than any other clarification process. Large footprint results in siting challenges as well as high capital cost.
Like plate settling and DAF, conventional sedimentation is less likely to lyse (rupture) algal cells than Actiflo, ozonation, and micro-screening; thus, reducing the potential to release algal toxins and produce taste and odor.	Flocculation for conventional sedimentation requires 30 minutes or more of detention time.
Although not an issue with the Lake Whatcom supply, conventional sedimentation can accommodate large quantities of settled sludge from high-turbidity waters or waters requiring large quantities of coagulant and polymer chemicals to promote effective settling.	Because of the large facility footprint, automated sludge collection is typically extensive and expensive.
This process does not impart much additional headloss to the existing system. Therefore, pumping of the process flow stream is not required.	

3.1.6. Micro-Screening

Micro-screening refers to the use of a stainless steel screen for straining or filtering particulate material. Several micro-screening manufacturers exists. These products are most commonly used in the municipal drinking water industry as a preliminary process to membrane filtration or reverse osmosis. The process works by trapping particulate material on the screen and building up a filter cake on the screen. The filter cake screens much smaller material than the openings in the screen. Build up of the filter cake results in corresponding buildup of headloss across the screen. Headloss buildup to 7psi (16 feet) is typical before cleaning of the screen is initiated. Screen cleaning is an automated process that involves suction pressure on the upstream side of the screen to dislodge and remove the filter cake. In some systems brushes are also used. A schematic diagram of a micro-screen filter manufactured by Amiad Filtration Systems is presented in Figure 3-5.

Micro-screening is typically considered for the purpose of algae removal because of the small facility size, its associated low initial capital cost, and because it will retain algae if the screen mesh size is small enough. However, micro-screening requires more available head than other technologies. There is insufficient head available between the City's existing WTP and the screen house to operate micro-screens without pumping. Additionally, micro-screens typically clog and become very difficult and problematic to clean when they are sized with openings small enough to filter algae. Although incidental removal of algae with micro screens is possible, as long as algae concentrations are low, there is no track record in this country of micro-screening being implemented primarily for the removal of algae at a municipal water treatment facility.

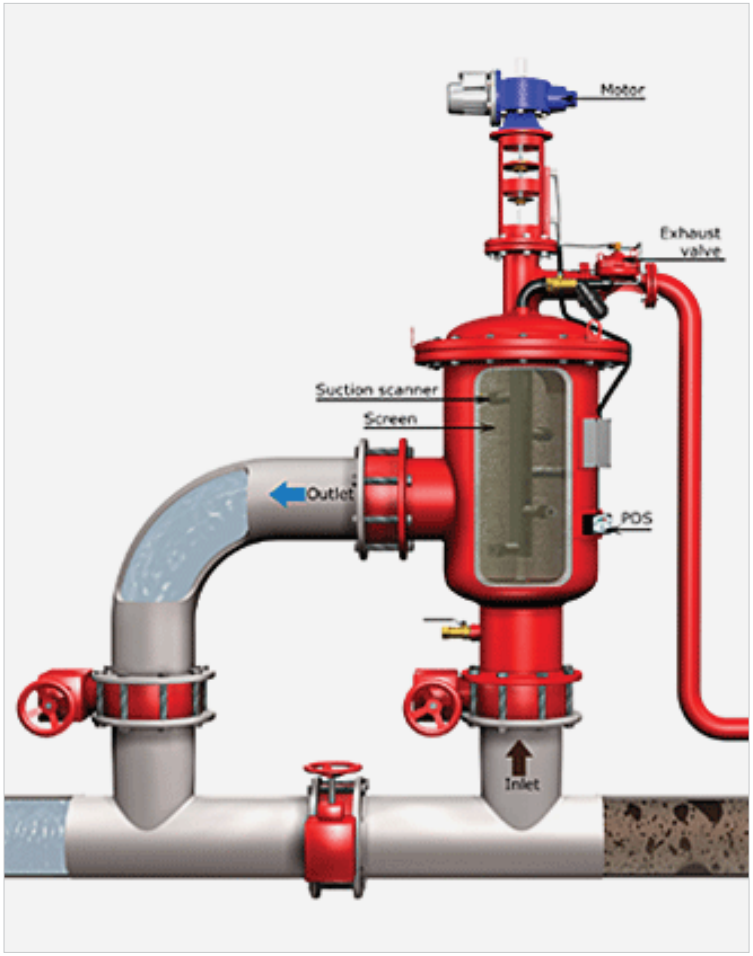


FIGURE 3-5
Micro-Screen Filter (Amiad Filtration Systems)

A summary of the advantages and disadvantages of micro-screening for mitigating the adverse impacts of filter-clogging algae is presented in Table 3-6.

TABLE 3-6
Advantages and Disadvantages of Micro-Screening

Advantages	Disadvantages
Micro-screening has a small facility footprint and associated low initial capital cost.	If micro-screens are used for algae removal, they will clog and be difficult clean.
Coagulant or polymer addition is not necessary.	Head available for micro-screening must typically be 7 psi or greater. This head is not available at the City's WTP.
	There is no track record of successful use of micro-screening primarily for removing algae from the flow stream at a municipal water treatment facility in this country.

3.1.7. Ozonation

Ozone is one of the most powerful disinfectants and oxidants available for use in the municipal water treatment industry. It is generated on site by passing dry air or oxygen between two electrodes, which converts some of the oxygen to ozone. Ozone is typically imparted to the process flowstream through micro bubbles in a concrete contact basin or in a pipeline that provides contact between the water and the ozone bubbles. Ozonation has used been successfully in this country for many years and in Europe since the early 1900s. In addition to being used to meet disinfection requirements, ozonation is a common and successful means of neutralizing taste and odor compounds, many of which are byproducts of algae respiration.

Because of its association with algae via effective taste and odor neutralization of algae-based compounds, and history of providing enhancement to filtration in many water treatment plants, it warrants consideration for neutralizing the filter-clogging impacts of algae. However, ozonation does not have a track record of success reducing the filter-clogging effects of algae. Ozonation kills algae, lysing algal cell structure, but does not remove it from the flowstream. An ozonation system that would lyse algal cell structure would need to be designed to also neutralize the release of potential toxins and taste and odor compounds.

A summary of the advantages and disadvantages of ozonation for mitigating the adverse impacts of filter-clogging algae is presented in Table 3-7.

TABLE 3-7
Advantages and Disadvantages of Ozonation

Advantages	Disadvantages
Pumping of the process flow stream is not required.	It is unclear whether or not ozonation would reduce the filter-clogging effects of algae. This impact would need to be pilot tested. There is no track record of using ozonation alone ahead of filtration to reduce algae filter clogging.
System can be designed to neutralize algal toxins and taste and odor compounds.	High ozonation doses could produce disinfection byproducts such as bromates, aldehydes, and ketones.
No liquid waste stream associated with this process.	Ozone converts some of the total organic carbon (TOC) to assailable organic carbon (AOC), which would need to be removed prior to the distribution system to prevent biological re-growth.

3.1.8. Additional Filters

This alternative involves the addition of two filters to the existing six filters at the City's WTP. The two filters would be situated in line, adjacent to the two filters furthest from the WTP control room to form parallel rows of four filters each. Two additional filters would expand filter area and therefore increase the capacity of the WTP, which could be used to mitigate the capacity-reducing effects of the filter-clogging algae. Two additional filters, in the absence of filter-clogging algae, at a rate of 5 gpm/sf, would result in an additional

8 mgd of WTP capacity. The resulting WTP capacity would be approximately 32 mgd with one filter out of service. During these normal-operating times, filter run times (the time a filter produces water prior to being taken out of service for backwashing) are generally long – typically 30 hours or more.

When filter run length is reduced, there is a minimal impact on WTP capacity as long as the filter run length is not reduced to less than 5 hours or so. When filter run lengths are reduced by filter-clogging algae to 5 hours and less, WTP capacity is greatly reduced. This WTP capacity reduction occurs because of the following three key reasons:

- Increased percentage of WTP capacity must be dedicated to filter backwashing
- The filter-to-waste process that precedes re-starting the filter after backwashing
- The associated non-filtration time before and in-between these processes

During the summertime 2009 algae bloom, filter run times dropped to as low as 3.5 hours. As discussed previously, this filter run time reduction resulted in a corresponding reduction in WTP capacity. As a consequence, the City implemented voluntary and mandatory water use restrictions to enable the reduced-capacity of the WTP to meet the restricted customer demand. If the City had two additional filters during the 2009 algae bloom, it may have been able to meet customer demand if no further reduction in filter run times had occurred. Further reduction in filter run times to less than two hours would likely have made any addition in filtration capacity – whether two new filters, four new filters, or more – ineffective at meeting customer demand. More discussion of this is presented in Section 5.1.

A summary of the advantages and disadvantages of additional filters for mitigating the adverse impacts of filter-clogging algae is presented in Table 3-8.

TABLE 3-8
Advantages and Disadvantages of Additional Filters

Advantages	Disadvantages
Minimal additional operation and system complexity.	The primary disadvantage of additional filters to relates to the uncertainty of the intensity of future algae blooms and the associated intensity of impact they would have on filter clogging. More intense algae blooms could render the addition of filters relatively to completely ineffective at increasing WTP capacity.
Space for two additional filters is readily available on site and the site disruption and complexity to add two addition filters is minimal.	Additional filters will not have any ancillary treatment benefits that other pre-filtration clarification processes will have such as the reduction in taste and odor compounds, reduction in disinfection byproduct pre-cursors, reduction in emerging contaminants.
Pumping of the process flow stream is not required.	Because of greatly-increased filter backwashing during filter-clogging algae blooms, a high percentage of WTP water will be used for filter backwashing, and therefore wasted.
Although additional treatment WTP capacity is not needed at this time, additional filters would provide additional treatment WTP capacity that could be put to beneficial use in the longer-term future. This additional capacity would be reduced, potentially greatly reduced, or even negated during algae events – depending on the intensity of the algae event.	Increased filter backwashing will increase flows to the City's wastewater treatment plant, which will increase pumping and other treatment costs there.

3.2. Intake Alternatives

Three intake alternatives were identified for consideration and evaluation. Each of the intake alternatives includes withdrawing water from Lake Whatcom at a location different from the existing intake location that has a substantially lower concentration of algae. Each of the intake alternatives includes the capability to withdraw water at more than one depth in the lake. Two of the alternatives involve maintaining continued use of the existing intake as a measure of redundancy, operational flexibility, and preserving peak hydraulic capacity. The third alternative involves replacement and abandonment of the existing intake. The intake alternatives are listed as follows:

- Secondary Intake via In-Water Pipeline (Intake Alternative 1)
- Secondary Intake via Over-Land Pipeline (Intake Alternative 2)
- New Dual-Intake System (Intake Alternative 3)

A summary of the intake alternatives is presented in the Sections 3.2.4 through 3.2.6 below.

Each of these alternatives involve extending the new, secondary, or replacement intake from the existing Gate House on the shoreline of Lake Whatcom to the same model-predicted location and depth where algae concentration is substantially lower than the location of the existing intake. The CE-QUAL-W2 model developed and used for the Lake Whatcom TMDL study was used to identify the "algae-favorable" location upon which these alternatives are based. A discussion of this modeling effort and the results is presented in Section 3.2.1.

The primary difference between the first two intake alternatives identified above is the routing of the intake pipeline. Intake Alternative 1 (Secondary Intake via In-Water Pipeline) involves installing the pipeline within the lake (laid on the bottom and weighted down and/or anchored on a pile-bent structure). Intake Alternative 2 (Secondary Intake via Over-Land Pipeline) would be installed in Lake Whatcom Boulevard and equipped with a pump station at the location where the intake pipeline extends from on-shore into the lake. Intake Alternative 3 (New Dual-Intake System) is similar to Intake Alternative 1 (same in-lake pipeline alignment); however, it includes abandoning the existing intake and replacing it with a new intake in Basin 2 at the same 30-foot depth as the existing intake. A map showing the intake pipeline alignments of these three alternatives is presented in Figure 3-6.

3.2.1. Modeling the Location of a New Intake

For the purposes of this study, the location of the new intake was identified using the Corps of Engineer's CE-QUAL-W2 model that was previously developed and calibrated for Lake Whatcom as part of the Lake Whatcom TMDL Study. The model was calibrated in 2003 from 2002 and 2003 data and was acknowledged to be a reasonably representative model simulation of algae conditions for the purposes of this study. It is understood that re-modeling with updated information may be warranted, depending on the results of this study and whether the City elects to pursue implementation of a new intake.

The CE-QUAL-W2 modeling is summarized in the report presented in Appendix A. The modeling addressed base case (2003), current (2011), and future build-out conditions. Current and future build-out conditions were projected from the base case condition. In addition to the existing intake location in Basin 2, three other locations in Basin 2 were

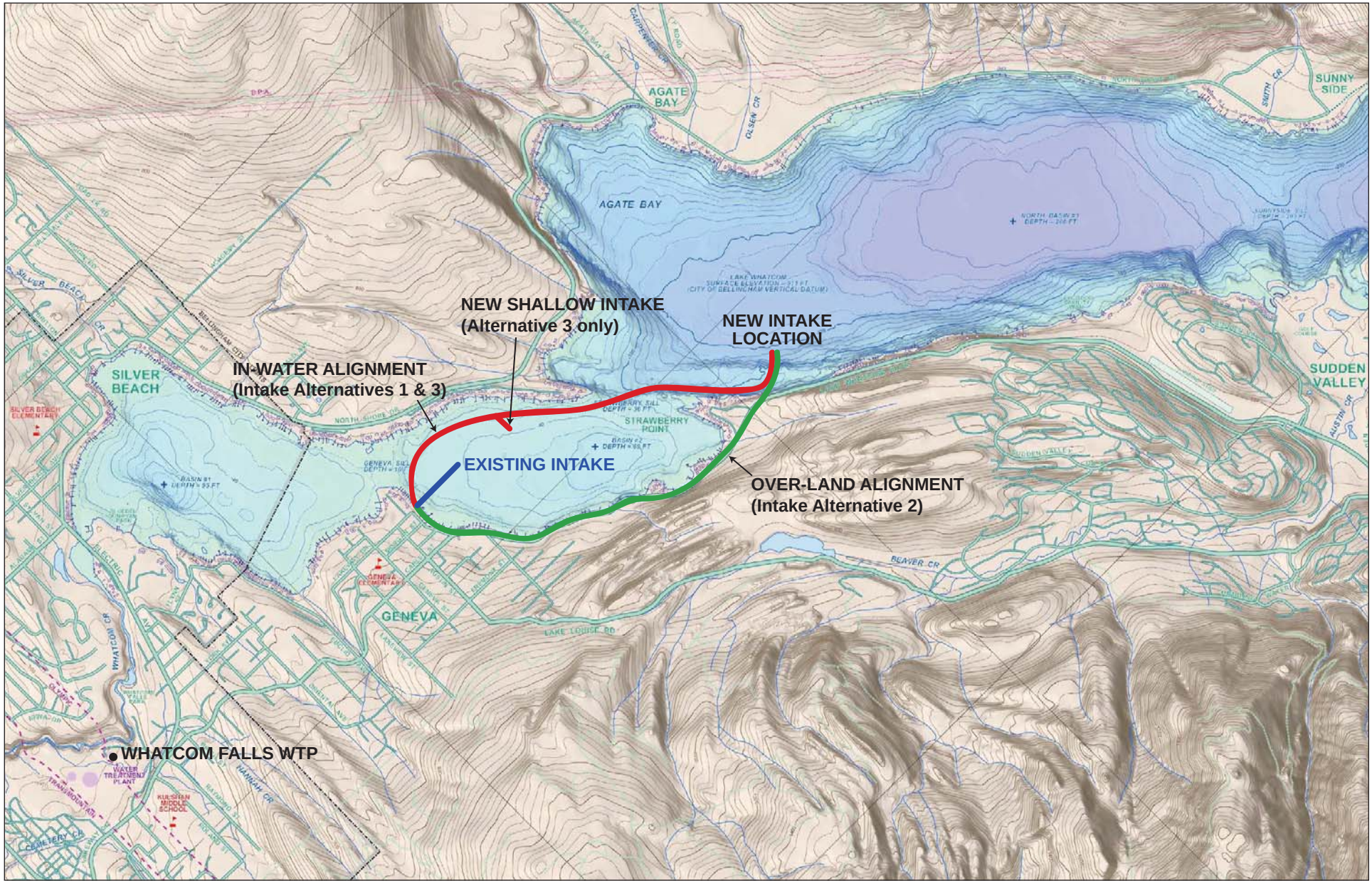


FIGURE 3-6
ALTERNATIVE INTAKE PIPELINE ALIGNMENTS

The CE-QUAL-W2 modeling is summarized in the report presented in Appendix A. The modeling addressed base case (2003), current (2011), and future build-out conditions. Current and future build-out conditions were projected from the base case condition. In addition to the existing intake location in Basin 2, three other locations in Basin 2 were simulated, and four locations in Basin 3 were simulated. Shallow, medium, and deep locations in the water column were simulated.

The following is a summary of key results of the modeling:

- Algae concentrations were estimated to be lowest at depths below 30 meters in Basin 3. Varying the location within Basin 3 at depths below 30 meters had negligible impact on estimated algae concentrations.

- Lower algae concentrations could be attained within Basin 2 by moving the intake to the deepest part of Basin two at approximately 20 meters. The model predicts blue-green algae concentrations at approximately 38 percent less than at the existing intake location. This was the only location within Basin 2 where significantly lowered algae concentrations were predicted. However, it should be noted that the model predicted substantially-reduced dissolved oxygen concentration at this location in the hypolimnion of Basin 2, which is in alignment with years of actual data collected in the hypolimnion of Basins 1 and 2. Even if there were interest in withdrawing water from this location in Basin 2 because of the moderately-lower algae concentration modeled, doing so would not be advisable because of the low dissolved oxygen (anoxic) conditions, as described in Section 3.2.2.

- Algae concentrations at shallow depths less than 10 meters in Basin 3 were predicted to be 20 to 30 percent less than the algae concentrations at shallow depths in Basin 2.

- Model-predicted algae concentrations only varied by up to 5 percent between existing land use and full build-out land use scenarios.

The modeling enabled establishment of a location within Basin 3 for a new intake that could work as a replacement to the existing intake or as a supplemental intake to the existing intake. An optimized intake location with respect to algae is one at depth within the hypolimnion of Basin 3. The model predicts minimal change in algae concentration at depth within the hypolimnion from the south end to the north of Basin 3. Therefore, there is no reason to extend a new intake pipeline from the existing gate house any further than necessary into Basin 3. For the purpose of this study it was assumed that the new intake location would be at a depth of approximately 120 feet near the northern end of Basin 3, as shown in Figure 3-6.

3.2.2. Avoiding Low Dissolved Oxygen

As stated above in Section 3.2.1, the deepest part of Basin 2, which extends to a depth of approximately 60 feet, is an area of lower algae concentration that might be of interest for locating a new, supplemental intake. While a reduction in blue-green algae concentration of 38 percent is likely not enough to warrant situating a new intake at this location, it does

raise an issue related to the impact at the WTP of withdrawing water from Lake Whatcom with low

or zero dissolved oxygen. Water near the bottom of lakes sometimes becomes anoxic as organic debris and algae die, sink, decompose and consume oxygen. Waters with low or zero dissolve oxygen are challenging to treat, whether those levels are permanently low or become low, seasonally, because of summertime temperature stratification, as is the case in Lake Whatcom. The reasons why these waters are challenging to treat and should be avoided are as follows:

- Seasonal low dissolved oxygen conditions produce dramatic and unpredictable changes in raw water quality during the fall "turn over" period when stratified water becomes de-stratified. During this time, chemical dosages would need to be changed to keep up with the changing raw water quality and the precise nature of those chemical changes would not be easy to predict. It may be necessary to take the WTP out of production to trouble-shoot and test new chemical dosage combinations. There would be similar but less dramatic changes during late spring and summer as dissolved oxygen concentration is gradually reduced to zero or close to zero.

- Anoxic waters are themselves difficult to treat simply because of the low dissolved oxygen. These waters tend to change certain already-oxidized metals in the lake sediments, such as iron and manganese, into dissolved constituents that are conveyed to the treatment process where they are subsequently oxidized and can be conveyed into the distribution system where they create aesthetic, taste, and odor problems.

- Sulfur, which is a component of living tissue and most organic material, is released when it decays and forms hydrogen sulfide in anoxic conditions. Hydrogen sulfide smells like rotten eggs and thus makes water objectionable. Hydrogen sulfide has been measured since 1999 in the hypolimnion of Basins 1 and 2 – reflective of the severely anoxic conditions at these locations.

- Nitrogen, which is also leached from organic material in both well-oxygenated and anoxic conditions. In anoxic conditions it is transformed into ammonia. Ammonia has a high chlorine demand and thus can interfere with chlorine disinfection. Additional chlorine becomes necessary to overcome this increased demand. Additionally, when ammonia combines with chlorine it forms various types of "chloramines" – some of which produce odors.

3.2.3. Hydraulic Capacity Considerations

In identifying each of these alternatives, it is important that hydraulic capacity of the existing intake system be considered. Historical withdrawals through the intake were substantially greater when the Georgia-Pacific Mill was in operation than they are today. Current municipal peak summertime demand is approximately 20 mgd. The other on-going, intermittent water use via the existing intake is the occasional use at Puget Sound Energy's small power plant on the City's waterfront that is used as a peaking supply. When in use, which is intermittent, this power plant consumes about 0.8 mgd of untreated water from the City's Lake Whatcom supply.

Although withdrawals to meet current demand through the existing intake are much less than historical withdrawals, the City intends to maintain hydraulic capacity of the intake system for future growth within its service area, potential expansion of municipal water supply service to areas currently served by other supplies, for potential future power generation projects, and for potential future industrial uses.

The capacity of the existing intake system is limited by the hydraulic capacity of the tunnel between the gate house and the screen house, which is approximately 108 mgd. The City's instantaneous water right from Lake Whatcom is 82 mgd. While minimizing the hydraulic capacity of a new intake system would reduce its cost, doing so could be disadvantageous as it relates to the future and potential future uses described above. The intake alternatives were developed with these two key flow parameters mind. The hydraulic capacities for each of the alternatives are presented as part of the descriptions of the intake alternatives in Sections 3.2.5 through 3.2.7.

3.2.4. Fish Guard Requirement

Per Revised Code of Washington (RCW) 77.57.010 fish guard devices are required on all intakes and diversions from lakes and rivers. Fish guard devices are oftentimes screens but can be other devices and structures, such as velocity barriers, if those other devices are demonstrated to be applicable and effective. Washington Department of Fish and Wildlife (WDFW) is the agency with jurisdiction over fish guards in Washington State.

A similar federal requirement (Section 7 of Endangered Species Act [ESA]) for fish guard devices involving the National Marine Fisheries Service (NMFS) and/or the United States Fish and Wildlife Service (USFWS) applies if there are threatened, endangered or anadromous fish species. The fish guard device design criteria for fish screens are the same for the federal and state criteria. However, because there are no threatened, endangered, or anadromous fish species in Lake Whatcom, the ESA Section 7 requirement does not apply.

As a result, although the City's existing Lake Whatcom intake does not have a fish screen, it is assumed for the purposes of this study that a fish screen will be required for a new Lake Whatcom intake – should the City select to pursue implementation of a new intake. Evaluation of fish guard devices, including a fish screen or other devices that may be less costly to clean and maintain, would be undertaken as part a preliminary design process.

As part of development of design criteria for a fish guard device, the City may elect to evaluate whether a fish guard device is necessary and whether such a device could be avoided. If a new intake were to be designed to be deep and away from the shoreline it could potentially be demonstrated that a fish guard devices is not necessary because no fish commonly reside at this location. RCW 77.55.231 allows for the potential to implement less-rigorous fish guard devices. It would be necessary to negotiate with WDFW and present a solid case based on a biological evaluation. While it is not common, there are instances where fish screens have been avoided on lakes and rivers.

3.2.5. Secondary Intake via In-Water Pipeline (Intake Alternative 1)

This alternative includes implementing a new, secondary intake that would function as a supplemental or alternate intake to the existing intake. It would be operated when algae conditions warrant. The new intake pipeline would extend from the existing Gate House along the bottom of Lake Whatcom to a location in Basin 3, as shown in Figure 3-6.

The new intake would be equipped with new fish-screened intake openings at one or two depths within the hypolimnion of Basin 3. In combination with the existing intake, which is at a depth of 30 feet, the combined intake system would have multi-level intake capability.

For the purpose of this evaluation, the hydraulic capacity of the new, supplementary intake would be 40 mgd. This hydraulic capacity was established in recognition of the following factors:

- The existing intake would be available to meet demands that exceed 40 mgd. However, the existing system would not have the same algae-favorable water quality as the new, supplemental intake.
- The 40 mgd capacity exceeds current projections of future peak day water demands for the City's municipal water system. It should be noted that water from the new, supplementary intake, with is low algae concentration, could be blended with water from the existing intake to meet demands that exceed 40 mgd during periods of high algae at the City's existing intake.
- Establishing the capacity of the new, supplemental intake system at 40 mgd results in a less costly new supplemental intake than if implementing it at the full Lake Whatcom water right of 82 mgd. This implementation strategy incorporates continued reliance on the existing intake for potential future high flows.
- The new intake system would retain its existing 108 mgd hydraulic capacity via the existing intake and would have a hydraulic capacity of 40 mgd through the new, supplemental intake.

Any future demands ensure adequate capacity to meet current and projected municipal, which comfortably exceeds current and projected municipal demand. The existing intake would remain in service and retain its current gravity capacity of 108 mgd.

A summary of the advantages and disadvantages of Intake Alternative 1 (Secondary Intake via In-Water Pipeline) for mitigating the adverse impacts of filter-clogging algae is presented in Table 3-9.

TABLE 3-9
Advantages and Disadvantages of Intake Alternative 1 (Secondary Intake via In-Lake Pipeline)

Advantages	Disadvantages
Minimal on-land disruption of Lake Whatcom Boulevard, which is a primary access route to	Extensive examination, study, and evaluation of subsurface geotechnical conditions and bathymetry

TABLE 3-9
Advantages and Disadvantages of Intake Alternative 1 (Secondary Intake via In-Lake Pipeline)

Advantages	Disadvantages
residents and businesses.	needed to develop effective design of intake pipeline installation.
New supplemental and combined (existing and new supplemental) intake system conveys lake withdrawal entirely by gravity.	Similar to each of the intake alternatives, extensive environmental permitting will be required, as well as extended time to complete the permitting process.
New supplemental portion of intake system has a reduced diameter in comparison with Intake Alternative 3, which reduces the cost of this improvement.	
With respect to maximizing use of existing infrastructure, this alternative makes continued use of the City's existing intake, which remains functional after nearly 70 years of service.	
New supplemental portion of intake system would provide complete intake redundancy "upstream" of the existing Gate House, which would improve intake reliability.	

3.2.6. Secondary Intake via Over-Land Pipeline (Intake Alternative 2)

Like Intake Alternative 1, this alternative includes implementing a new, secondary intake that would function as a supplemental, secondary, or alternate intake to the existing intake. It would be operated when algae conditions warrant. The new intake pipeline would extend from the existing Gate House overland in Lake Whatcom Boulevard along the south side of the lake to a location in Basin 3, as shown in Figure 3-6.

Like Intake Alternative 1, the new intake would be equipped with new fish-screened intake openings at one or two depths within the hypolimnion of Basin 3. In combination with the existing intake, which is at a depth of 30 feet, the combined intake system would have multi-level intake capability.

Although this alternative minimizes the length of new in-lake pipeline installation and its associated installation challenges and uncertainties, a new over-land intake pipeline system would require a pump station to convey water through the over-land pipeline in Lake Whatcom Boulevard, above lake level, back into the existing tunnel at the Gate House. For the purpose of this evaluation, and for the reasons described above for Intake Alternative 1, the hydraulic capacity of the new, supplementary intake would be 40 mgd.

A summary of the advantages and disadvantages of Intake Alternative 2 (Secondary Intake via Over-Land Pipeline) for mitigating the adverse impacts of filter-clogging algae is presented in Table 3-10.

TABLE 3-10
Advantages and Disadvantages of Intake Alternative 2 (Secondary Intake via Over-Land Pipeline)

Advantages	Disadvantages
A lesser amount (than the other intake alternatives) of evaluation of uncertain subsurface geotechnical and bathymetric conditions within Lake Whatcom.	Extensive disruption to the public and to traffic along Lake Whatcom Boulevard during construction.
New supplemental portion of intake system would provide complete intake redundancy "upstream" of the existing Gate House, which would improve intake reliability.	Pumping of the flow through the new supplemental intake will be necessary.
	Property rights acquisition (purchase or easement) will be necessary for the intake pump station and will likely also be necessary for several segments of the intake pipeline.

3.2.7. New Dual-Intake System (Intake Alternative 3)

This alternative is essentially the same as Intake Alternative 1(Secondary Intake via In-Water Pipeline) except that the existing intake is removed from service and replaced with a new intake at a 30-foot depth in Basin 2 (at a different location than the existing intake). The intake pipeline alignment would be in the lake and would extend to the same location in Basin 3 as Intake Alternatives 1 and 2, as shown in Figure 3-6. The rationale behind this alternative is implementing a new intake and abandoning the existing intake, which will eventually need repair and replacement.

The new intake would be equipped with two fish-screened intake locations. One of the screened intake locations would be relatively shallow (in the epilimnion) along the alignment of the new intake pipeline location in Basin 2. The other would be at greater depth in the hypolimnion at the same depth and location as the other two alternatives in Basin 3. The hydraulic capacity of this intake alternative would be 108 mgd for the nearest and shallow intake opening in Basin 2 (matching the gravity conveyance capacity of the City's existing intake) and 40 mgd for the intake openings in Basin 3 (the same as Intake Alternatives 1 and 2).

A summary of the advantages and disadvantages of Intake Alternative 3 (New Dual-Intake System) for mitigating the adverse impacts of filter-clogging algae is presented in Table 3-11.

TABLE 3-11
Advantages and Disadvantages of Intake Alternative 3 (New Dual-Intake System)

Advantages	Disadvantages
Minimal on-land disruption of Lake Whatcom Boulevard, which is a primary access route to residents and businesses.	Extensive examination, study, and evaluation of subsurface geotechnical conditions and bathymetry needed to develop effective design of intake pipeline installation.
The new intake system conveys lake withdrawal entirely by gravity.	Similar to each of the intake alternatives, extensive environmental permitting will be required, as well as extended time to complete the permitting process.
New intake system is entirely new and does not rely on the existing intake that is constructed of wood, is 70 years old, and presumably has a limited remaining useful life.	This new intake system will have two fish-screened intake openings as opposed to only one for Intake Alternatives 1 and 2. More fish screens results in more annual operations and maintenance costs.

TABLE 3-11
Advantages and Disadvantages of Intake Alternative 3 (New Dual-Intake System)

Advantages	Disadvantages
	With respect to maximizing use of existing infrastructure, this alternative does NOT make continued use of the City's existing intake, which remains functional after nearly 70 years of service.

3.3. Lake Management Alternative

This alternative was identified and included for consideration in recognition of the fact that the City and Whatcom County have already implemented the Lake Whatcom Management Program (LWMP) for the purpose of improving Lake Whatcom water quality, as is described in Section 2.4. While the City's and County's efforts with respect to the LWMP predate the Lake Whatcom TMDL Study described in Section 2.4, it is the management forum via which compliance with the TMDL requirements for dissolved oxygen and phosphorous is being pursued. Although this lake management alternative, based upon compliance with the TMDL requirements, is considered as part of this evaluation as a stand-alone strategy for mitigating the algae issues at the City's WTP, it will be implemented regardless of the results of this evaluation. As stated in Section 2.4, meeting the TMDL requirements is the cornerstone of the long-term strategy to improve water quality in Lake Whatcom, including reducing algae concentrations.

The City and Whatcom County are both entirely committed to continuing vigorous pursuit of implementation of activities and opportunities to improve Lake Whatcom water quality, which includes reduction in seasonal algae production. Compliance with TMDL standards is a requirement and key primary objective of the LWMP. The City and County are continuing their efforts through the LWMP regardless of the results of this evaluation and regardless of whether the City ultimately pursues implementation of other alternatives to mitigate the adverse impacts of filter-clogging algae at the City's WTP. Consequently, lake management will be, at a minimum, an important complementary element of the overall long-term strategy to address filter-clogging algae and maintain Lake Whatcom as a high quality drinking water supply.

Therefore, for the purposes of this evaluation, the Lake Whatcom Management Program is essentially the lake management alternative considered herein for mitigating the filter-clogging algae conditions that have been observed in recent years. As stated in Section 2.4, via the LWMP, the City of Bellingham, Whatcom County, and Lake Whatcom Water and Sewer District will be completing a Detailed Implementation Plan (DIP) to comply with the TMDL requirements. The DIP will identify phosphorous-reduction measures, annual program budgets for implementing those measures, estimated target time-frames for implementation of measures, and an overall estimate of the duration needed to meet the TMDL standards for dissolved oxygen and phosphorous.

Although these elements of the LWMP remain to be developed, it is understood by the stakeholders involved with the TMDL that the duration to meet the TMDL standards will be many years if not decades. Presumably, when the TMDL standards for dissolved oxygen and phosphorous are met, algae conditions in Lake Whatcom will not present the same challenges to filtration at the City's WTP that they did in the summer of 2009. The

uncertainty with respect to the duration needed to meet the TMDL standards represents the primary disadvantage of this lake management alternative for implementation to mitigate the filter-clogging algae at the City's WTP.

One of the key elements of the City's preliminary development of its long-term lake management strategy, as well as development of the DIP, is an initial comparison of several selected phosphorus-reducing and phosphorus-removal strategies with respect to their cost and their impact on reducing phosphorous entering Lake Whatcom. This comparison is presented in tech memo format in Appendix B. The results of this initial work present a relative comparison of phosphorous–reduction measures on a cost per unit of phosphorous removed. This work not only forms one of the initial steps toward development of the DIP and the long-term lake management strategy to be implemented by the LWMP, but is intended to inform policy decisions by the City of Bellingham now and in the short-term future. Additionally, aggressive long-term pursuit of the TMDL requirements could enable a more-cost-effective initial implementation of a stand-alone algae mitigation strategy for the City's WTP, as is discussed Section 8.It should be noted that a lake management approach that has been implemented at other lake locations was also considered as part of an overall lake management approach for this application. This other lake management approach is referred to as "hypolimnetic oxygenation" which is the process of oxygenating the hypolimnion of the lake to keep it from becoming anoxic during summertime temperature stratification. Hypolimnetic oxygenation is typically accomplished by generating oxygen on shore and piping it into the lake, along the lake bottom, through pipelines with diffusers to disseminate the oxygen. The cost and complexity of this approach increases substantially with lake size.

Because anoxic conditions are known to leach phosphorous from the lake bottom sediments and settled organic material (including decaying algae), phosphorous from this source can be a substantial contributor to the overall phosphorous concentration in the lake. During the fall "lake turnover," when the lake becomes de-stratified, the phosphorous is liberated to the epilimnion where it becomes an available nutrient source for algae in spring and summer. Effective hypolimnetic oxygenation keeps this phosphorous contribution at bay, and therefore helps reduce algae growth.

Hypolimnetic oxygenation was not developed further beyond this initial consideration because the amount of the total Lake Whatcom phosphorous budget for phosphorous leached from lake bottom sediments was identified as being negligible as part of the lake modeling effort described in Section 3.2.1. The relative contribution of phosphorous from this source is presented in Figure 3 on page 6 of Appendix A.

Other lake management approaches such as covering the lake with black polypropylene balls to shade the lake from sunlight (to reduce algae growth) and aerating the lake with surface mixers or fountains were identified, but not considered. In the case of each of these approaches, both would be prohibitively expensive, ineffective, and would likely result in a multitude of other problems.

4. Screening of Alternatives

Evaluation of the alternatives to mitigate the adverse impacts of filter-clogging algae at the City's water treatment WTP was implemented in three distinct phases. These three phases include:

- **Screening of Alternatives:** This first phase, "screening of alternatives," was implemented to eliminate from further consideration and evaluation alternatives that were deemed "not selectable" based on one or more screening criteria. The results of this screening are presented in this section of the report. This approach enabled more subsequent focus and effort in developing and evaluating those alternatives that were deemed to have greater promise for selection and implementation.

- **Evaluation of Alternatives:** This second phase of the evaluation process is presented in Section 6 of this report and reflects a more-detailed evaluation of the remaining alternatives. This evaluation phase results in identification of the best alternative within each of the three main alternative categories (as applicable for alternatives evaluated beyond the screening phase) as well as a best overall alternative based on detailed evaluation criteria and ranking based mostly, but not entirely, on technical performance.

- **Triple Bottom Line Plus Evaluation:** This third phase of the evaluation process is presented in Section 7 of this report and reflects evaluation based on a "Triple Bottom Line Plus" approach. In this evaluation phase, the best alternative for each of the three main alternative categories (as applicable for alternatives evaluated beyond the screening phase) are evaluated along with a "No Action" alternative and any other selected alternatives that may appear attractive despite not ranking highest with its main alternative category. This Triple Bottom Line Plus evaluation approach enabled focusing this City-accepted method on the alternatives warranting the greatest scrutiny with respect to financial, social, environmental, and technical objectives.

The process for screening of alternatives is presented in the following three subsections that address the screening criteria that were developed, the screening matrix itself, and a discussion of the screening results.

4.1. Screening Criteria

Criteria used for screening the alternatives were developed in recognition that there are a few "deal breakers" related to mitigating the summertime algae condition at the WTP. Alternatives that do not respond positively to these "deal breakers" were deemed to not warrant additional evaluation. The criteria developed for screening that represent these "deal breaker" issues include the following:

- **History of successful performance for algae removal?** Alternatives that do not have some history or documented track record of having been successfully and reliably implemented for the purpose of removing algae were deemed to not warrant further consideration and evaluation. Alternatives that do not have such a history may possibly

have some level of success at removing algae and alleviating the City's filter-clogging algae condition. However, the intention of this criterion is to avoid capital expenditure on alternatives that have an uncertain level of performance, potentially leading to substantial sunk cost.

- **Can flow stream be conveyed by gravity?** A new pump station to convey the entire flow stream will be expensive to construct and operate, add substantial complexity to the City's supply and treatment system, and reduce supply reliability. As a result, because there are other viable treatment and intake alternatives that do not require the addition of a pump station, any treatment or intake alternatives that do require a pump station were deemed to not warrant further consideration and evaluation.

- **Can alternative reasonably be accommodated on WTP site?** This screening criterion applies to the treatment alternatives, only. Some of the treatment alternatives can be accommodated on the City's existing WTP site within area that is already cleared of forest or with relatively minimal additional clearing, excavation, and utility relocation. Inter-department transfer of property from the adjacent City of Bellingham Whatcom Falls Park would be necessary for some alternatives. Because there are viable treatment alternatives with a relatively small facility footprint that can be accommodated on site, treatment alternatives that require large amounts of forest clearing, excavation, or private property acquisition were deemed to not warrant additional consideration and evaluation.

- **Addresses problem upon implementation?** Any alternative selected for implementation must effectively address the problem, functionally and reliably removing algae from the City's supply flow stream prior to the existing filters at the WTP. Alternatives that are known take many years and/or are known to have an uncertain period of time to implement and achieve success were deemed to no warrant additional consideration and evaluation.

The screening criteria were developed so that an alternative for which a "yes" answer is appropriate, warrant further evaluation. Conversely, those alternatives for which a "no" answer is appropriate for any one of the screening criteria, were dropped from further evaluation.

4.2. Screening Matrix

Screening of the alternatives was completed as a group in a workshop setting by the project team comprised of City of Bellingham and CH2M HILL staff. Assigning "yes," "no," or "n/a" was discussed among the group for each alternative and each screening criteria until a consensus was reached. The resulting screening matrix is presented in Exhibit 4-1.

Exhibit 4-1
Screening Matrix

Alternative	Screening Criteria (1)			
	History of successful performance for algae removal?	Can flowstream be conveyed by gravity?	Can alternative reasonably be accommodated on WTP site?	Addresses problem upon implementation?
Treatment Alternatives				
Clarification				
Dissolved Air Flotation (DAF)	yes	yes	yes	yes
Ballasted Sedimentation (Actiflo)	yes	yes	yes	yes
Plate and Tube Settling	yes	yes	yes	yes
Upflow Clarification (Superpulsator)	yes	yes	yes	yes
Conventional Sedimentation	yes	yes	no	yes
Micro-Screening	no	no	yes	yes
Ozonation	no	yes	yes	yes
Additional Filters	yes	yes	yes	yes
Intake Alternatives				
Secondary Intake via In-Water Pipeline	n/a	yes	n/a	yes
Secondary Intake via Overland Pipeline	n/a	no	n/a	yes
Replace Existing Intake	n/a	yes	n/a	yes
Lake Management Alternative	yes	n/a	n/a	no

Notes:
(1) Alternatives given a "no" to any of the screening criteria were dropped from further evaluation.

4.3. Results of Screening

As stated above, the purpose of the screening process was to eliminate alternatives from further consideration and evaluation that were deemed "not selectable." In achieving that purpose, the screening process resulted in eliminating the following alternatives from further consideration and evaluation:

- **Conventional Sedimentation:** This alternative was deemed to be unreasonably large to be accommodated at the WTP site without extensive environmental impacts. Given its large size, it was deemed unnecessary to further evaluate this alternative given that there are other viable and more-effective treatment alternatives with a much smaller facility footprint.
- **Micro-Screening:** This alternative was deemed to not warrant further consideration for two reasons: (1) there is no track record of its successful and effective use as a

stand-alone process for algae removal in a municipal water treatment plant, and (2) micro-screening cannot be implemented without pumping in this application.

- **Ozonation:** This alternative was deemed to not warrant further consideration because there is no track record of its successful implementation for the expressed purpose of reducing or eliminating algae-filter-clogging problems.
- **Secondary Intake via Overland Pipeline:** This alternative was deemed to not warrant further consideration because it requires a pump station for the intake flow that would be conveyed from the new secondary intake.
- **Lake Management Alternative:** The City and County are committed to on-going and future efforts to improving water quality in Lake Whatcom. These efforts are being pursued as part of the Lake Whatcom Management Program with a key goal of meeting the TMDL requirements for phosphorous and oxygen. However, this alternative was deemed to not warrant further consideration for the immediate and near-term purpose of mitigating the adverse impacts of filter-clogging algae. The reason for discontinuing consideration of this alternative is that its implementation and the observation of beneficial results will take many years, if not decades. The actual duration cannot be accurately predicted. An alternative without a definite, predictable timeframe was deemed unacceptable for further consideration.

The alternatives that were not screened (eliminated) from further evaluation were further developed and evaluated in greater detail, as presented in Sections 5 and 6, respectively. These alternatives include:

Treatment Alternatives

- Dissolved Air Flotation (DAF)
- Ballasted Sedimentation (Actiflo)
- Plate Settling
- Upflow Clarification (Superpulsator)
- Additional Filters

Intake Alternatives

- Secondary Intake via In-Water Pipeline (Intake Alternative 1)
- Replacement of Existing Intake (Intake Alternative 3)

5. Development of Alternatives

Screening of the initial list of alternatives reduced the number of alternatives remaining to be evaluated to five treatment alternatives and two intake alternatives. Further development of these alternatives is presented below in Sections 5.1 and 5.2. A key part of the development of these alternatives and their estimated costs is presented in Section 5.3.

5.1. Treatment Alternatives

The five treatment alternatives remaining after the screening process include the following:

- Dissolved Air Flotation (DAF)
- Ballasted Sedimentation (Actiflo)
- Plate Settling
- Upflow Clarification (Superpulsator)
- Additional Filters

These treatment alternatives can be divided into two groups – pretreatment or high rate clarification (each of these pretreatment processes are high-rate clarification) and filtration. The high rate clarification processes have varying treatment effectiveness with respect to algae and have varying hydraulic loading rates, as discussed in Section 3. These hydraulic loading rates have a greater impact on facility area requirements than all other design criteria. In fact, the area requirements for some of these high rate clarification processes would make it challenging to situate them at the WTP site.

Figure 5-1 shows the approximate layout area requirements of each of the high rate clarification process alternatives. Included in Figure 5-1 for reference is conventional clarification, which was dropped from further consideration as part of the alternatives screening process. It is clear from Figure 5-1 why it would be exceedingly challenging and invasive to accommodate conventional clarification at the WTP site. Conversely, the area requirements presented in Figure 5-1 clearly indicate that DAF and Ballasted Sedimentation (Actiflo) have the most siting flexibility. As stated above, the reason for these reduced areas is because of their high loading rates compared to the other high rate clarification processes.

While the general siting location presented in Figure 5-1, is possible, substantial excavation would be necessary because of an existing hill. Other siting options could be substantially less costly to implement. Because of their smaller area requirements, DAF and Actiflo offer much greater siting flexibility and could be accommodated on other parts of the site where less excavation is necessary and where connections to existing yard piping is less costly. Two siting options that can accommodate DAF and Actiflo but cannot accommodate the other high rate clarification processes are presented in Figure 5-2.

The Additional Filters alternative does not provide many of the same ancillary treatment benefits provided by the pretreatment alternatives, such as TOC reduction, reduction in disinfection byproduct formation potential, and the extension of filter runs. However, adding two filters to the existing WTP would be less costly than a pretreatment system with

a capacity of 30 mgd and it would substantially increase WTP capacity when algae concentrations are negligible, low, or moderate. The key concern regarding the Additional Filters alternative, as discussed in Section 3.1.8, is the potential limited benefit on WTP capacity if greater concentrations of algae in Lake Whatcom were to reduce filter run lengths to two hours or less. Whether or not Lake Whatcom algae concentrations will continue to increase to a point where such filter run length reductions occur is not known.

Given the possibility that algae blooms of greater intensity than what occurred in the summer of 2009 could occur in the future, net production capacity (excluding filter-backwashing and filter-to-waste volumes) of the WTP was plotted for varying filter run times. The capacity of the existing six filters at varying filter loading rates (up to the 6 gpm/sf allowed by Washington State Department of Health) was plotted to show how existing WTP capacity varies with changing filter run time. On the same graph, the capacity of an expanded WTP (two additional filters for a total of eight filters) was also plotted at the same filter loading rates.

These plots, presented in Figure 5-3, demonstrate how net WTP capacity varies with filter run time. When filter run times are long, which is the case when raw water algae concentration is low, two additional filters add substantially to the capacity of the WTP – up to 8 mgd. As filter run times are reduced to less than 3 hours, the increase associated with two additional filters is to approximately 5 mgd and the overall capacity of the WTP is greatly reduced. As filter run times drops to between 1 and 2 hours, the benefit to WTP capacity of two additional filters is minimal and insufficient to assist in meeting customer water demand.

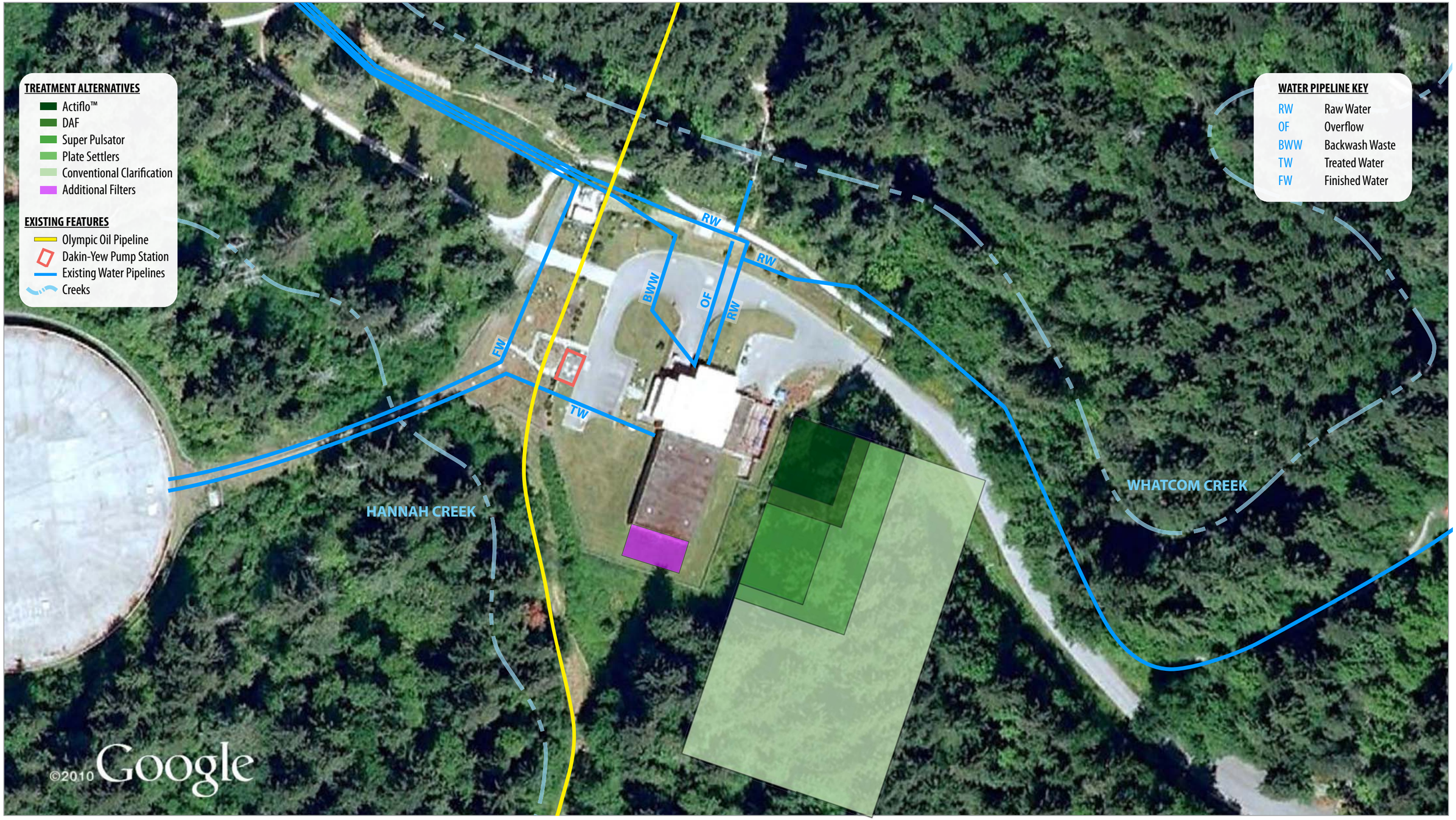
The uncertainty of the extent to which future algae events in Lake Whatcom reduce filter run time at the WTP is the key disadvantage of the Additional Filters alternative and why it could prove to be ineffective when the additional capacity is needed the most.

5.2. Intake Alternatives

The two intake alternatives deemed to warrant further consideration and evaluation, Intake Alternative 1 (Secondary Intake via In-Water Pipeline) and Intake Alternative 3 (New Dual-Intake System), both involve installation within Lake Whatcom. These alternatives avoid the extensive cost and disruption associated with installation in Lake Whatcom Boulevard as well as a new pump station and its associated cost and complexity. As presented in Figure 3-6, the alignment of the intake pipelines for both Intake Alternatives 1 and 3 are the same.

However, the intake pipeline diameter for each would be different. The diameter of the entire new supplemental intake pipeline for Intake Alternative 1 would be 60 inches to enable conveyance of 40 mgd by gravity from Basin 3. The diameter of the new intake pipeline for Intake Alternative 3 would be 78 inches to the new screened intake in Basin 2 to enable up to 108 mgd of gravity conveyance capacity (to match the existing intake pipeline) and 60 inches between the Basin 2 intake and the new intake in Basin 3 to enable 40-mgd gravity conveyance capacity.

Intake Alternative 3 includes a new shallow intake in Basin 2 with fish screens to replace the existing intake that would be abandoned. This new shallow intake is not necessary for Intake Alternative 1 because Intake Alternative makes continued use of the existing intake.



0 75 150 225 300

Approximate scale in feet

North

FIGURE 5-1
AREA REQUIREMENTS FOR
TREATMENT ALTERNATIVES

CH2MHILL

0 75 150 225 300

Approximate scale in feet

North

FIGURE 5-2
ALTERNATIVE SITE LAYOUTS FOR DAF AND ACTIFLO

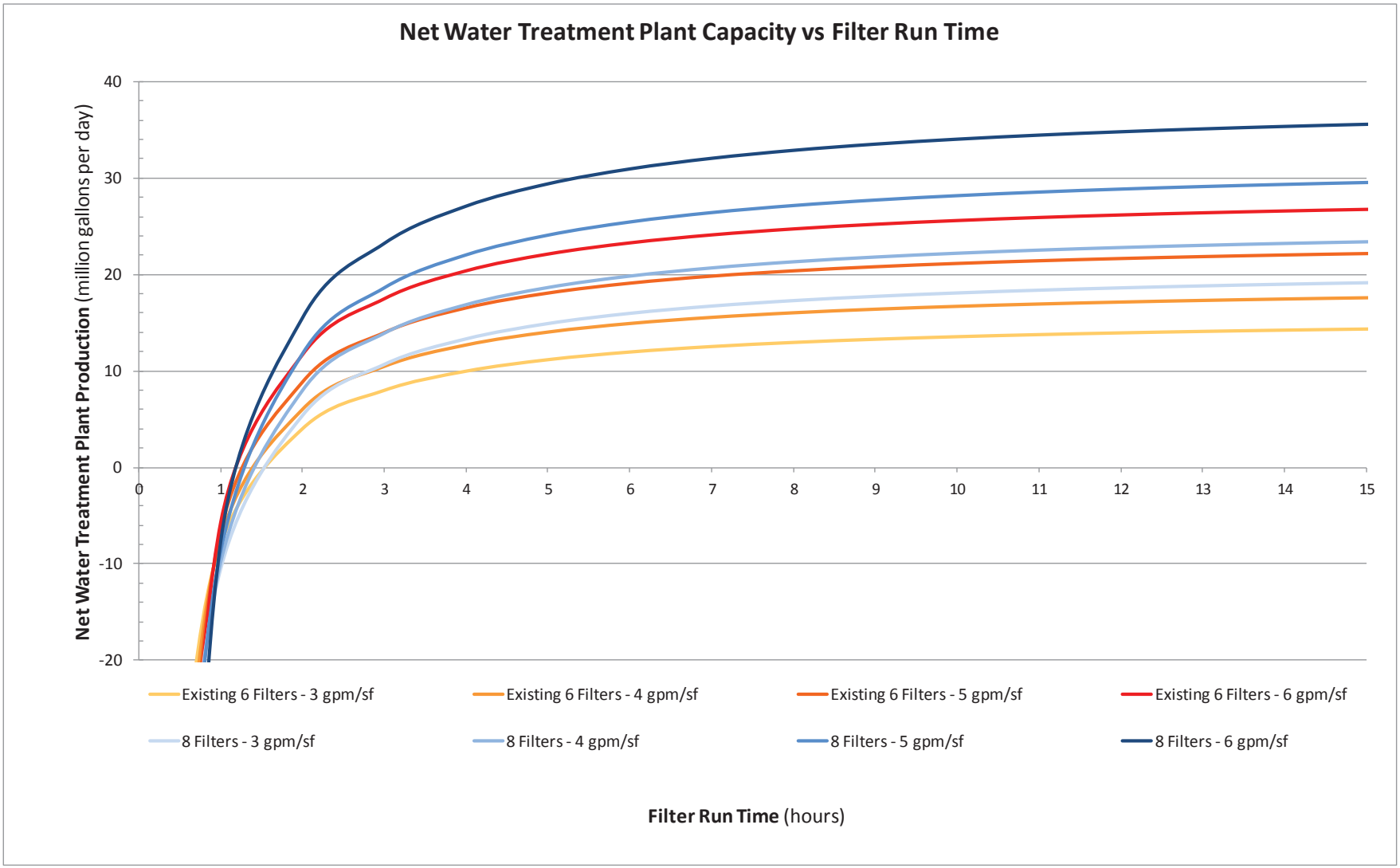


FIGURE 5-3
Net Water Treatment Plant Capacity vs. Filter Run Time

Both intake alternatives would include a new fish-screened intake in Basin 3 that extends to a depth of approximately 120 feet to ensure it is well within the hypolimnion and below the typical range of depths where Lake Whatcom algae reside. Schematic detail of the configuration of the extension of the intake pipeline in Basin 3, the associated fish screens, and the on-shore housing of electric control equipment is presented in Figure 5-4. It should also be noted that Intake Alternative 3 has three times the number of fish screens as Intake Alternative 1 and two on-shore electrical equipment housings instead of one. The reason for these additional fish screens and extra on-shore electrical equipment housing is that Intake Alternative 3 includes the new, shallow-depth intake in Basin 2, at twice the capacity as the new intake in Basin 3, to replace the existing intake system.

These additional fish screens add substantially to system complexity as well as capital and operations and maintenance costs. The detail presented in Figure 5-4 would be similar for the new intake in Basin 2 associated with Intake Alternative 3, except that the new electrical equipment housing would be on the north shore of Lake Whatcom.

5.3. Estimated Costs

Estimated initial capital, annual operations and maintenance, and 20-year life-cycle costs were developed for each of the alternatives. The estimates were developed to the "concept level" or "Class 5" level of accuracy as defined by the Association for the Advancement of Cost Engineering International (AACEI). This level of cost estimating is considered accurate to +30 to -20 percent.

The estimated costs were prepared for guidance in evaluation of alternatives and selection of a preferred alternative for implementation based on information available at the time of the estimate. The final cost of the project will depend upon the actual labor and material costs, competitive market conditions, implementation schedule, and other variable factors. As a result, the final project costs will vary from the estimates presented herein. Because of this, project feasibility and funding needs must be carefully reviewed prior to making specific financial decisions.

5.3.1. Initial Capital

Initial capital costs were developed for each of the alternatives. A summary of these costs is presented in Table 5-1.

Markups applied in developing the construction portion of the initial capital cost estimate are listed below.

- General Conditions: 5%
- Contractor Overhead: 10%
- Profit: 6%
- Mobilization/Bond/Insurance: 10%
- Contingency: 30%
- Escalation Rate to Midpoint of Construction: 12.8%
- Whatcom County, WA Sales Tax: 8.7%

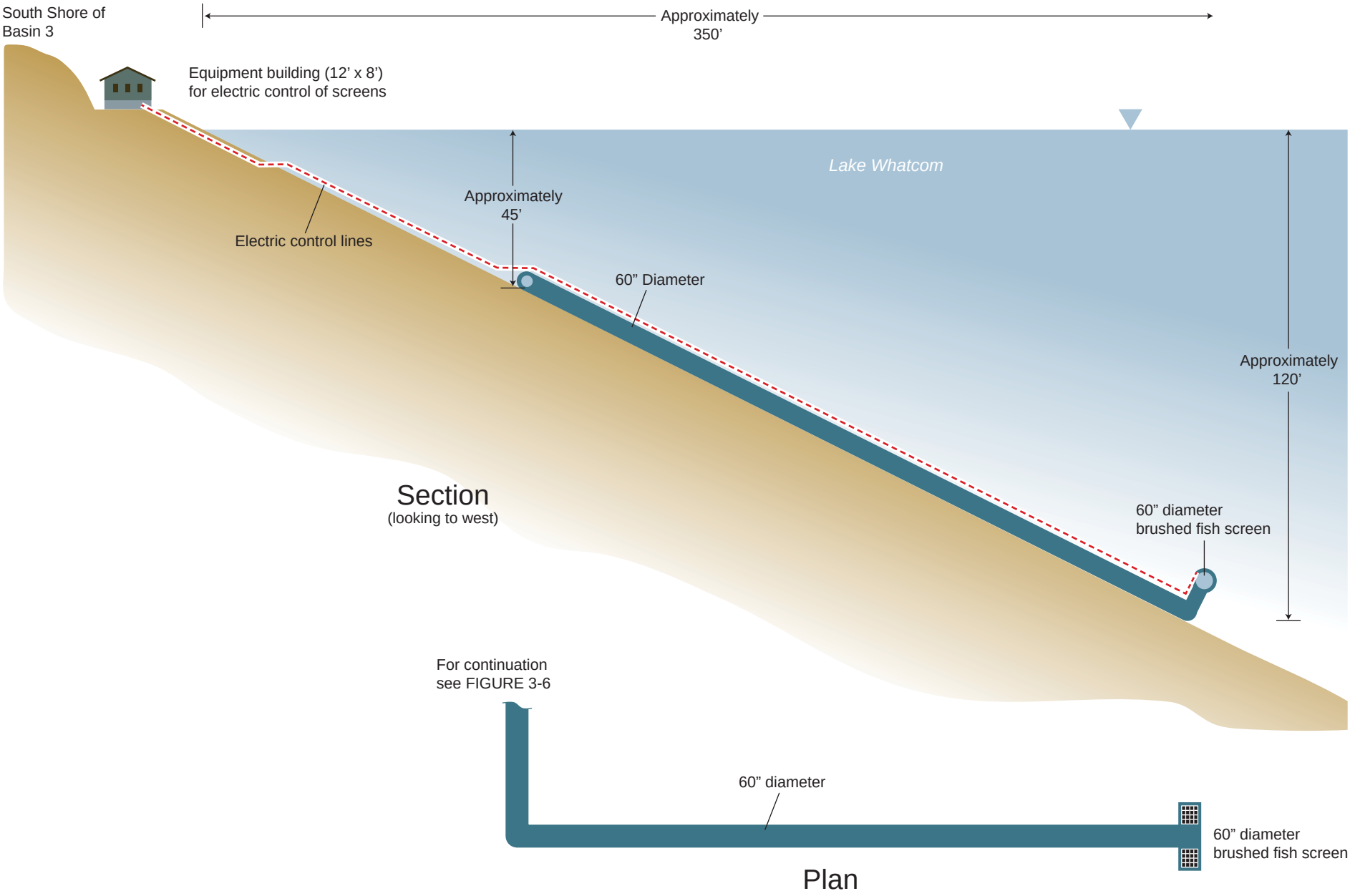


FIGURE 5-4
BASIN 3 INTAKE DETAIL FOR
INTAKE ALTERNATIVES 1 AND 3

TABLE 5-1
Summary of Estimated Initial Project Costs

Process Improvement	Plate Settling	DAF	SuperPulsator	Actiflo	Additional Filters	Intake Alternative 1	Intake Alternative 3
Construction Costs:							
Construction Cost Subtotal	$ 9,342,000	$ 5,756,000	$ 6,653,000	$ 5,143,000	$ 2,338,000	$ 13,463,000	$ 14,679,000
Subtotal with Contractor OH (10%)	$ 10,276,000	$ 6,332,000	$ 7,318,000	$ 5,657,000	$ 2,572,000	$ 14,809,000	$ 16,146,000
Subtotal with Contractor Profit (6%)	$ 10,893,000	$ 6,711,000	$ 7,757,000	$ 5,997,000	$ 2,726,000	$ 15,698,000	$ 17,115,000
Subtotal with Contractor Mob, Bonds Ins (10%)	$ 11,982,000	$ 7,383,000	$ 8,533,000	$ 6,596,000	$ 2,999,000	$ 17,268,000	$ 18,827,000
Subtotal with Contingency (30%)	$ 15,577,000	$ 9,597,000	$ 11,093,000	$ 8,575,000	$ 3,898,000	$ 22,448,000	$ 24,475,000
Escalation to Yr 2014 (12.8%)	$ 17,570,000	$ 10,826,000	$ 12,513,000	$ 9,673,000	$ 4,397,000	$ 25,321,000	$ 27,607,000
Construction w/ Sales Tax (8.7%)	**$ 19,099,000**	**$ 11,768,000**	**$ 13,602,000**	**$ 10,515,000**	**$ 4,780,000**	**$ 27,524,000**	**$ 30,009,000**
Non-Construction Costs:							
Pilot Testing	NA	$130,000	$ 200,000	$ 200,000	NA	NA	NA
Property for Screen Control House	NA	NA	NA	NA	NA	$ 500,000	$ 500,000
Geotechnical/Bathymetry[1]	$ 70,000	$ 70,000	$ 70,000	$ 70,000	$ 70,000	$ 400,000	$ 400,000
Modeling/WQ Monitoring	NA	NA	NA	NA	NA	$100,000	$100,000
Permitting	$ 200,000	$ 200,000	$ 200,000	$ 200,000	$ 200,000	$ 400,000	$ 400,000
Engineering and Construction Management [2]	$ 3,438,000	$ 2,118,000	$ 2,448,000	$ 1,893,000	$ 860,000	$ 2,752,000	$ 3,001,000
Startup[3]	$ 382,000	$ 235,400	$ 272,000	$ 210,300	$ 95,600	$ 275,200	$ 300,100
Total	**$ 23,189,000**	**$ 14,521,000**	**$ 16,792,000**	**$ 13,087,000**	**$ 6,006,000**	**$ 31,952,000**	**$ 34,710,000**

Notes:

1. Treatment and intake alternatives both require geotechnical evaluation; however, only the intake alternatives require detailed bathymetric survey of Lake Whatcom.

2. Eighteen percent of construction was used for treatment alternatives. Ten percent was used for intake alternatives because design is anticipated to be less complicated and more costly.

3. Startup percentage for treatment alternatives estimated at two percent of construction cost with sales tax; one percent for intake alternatives.

Note that "Escalation Rate to Midpoint of Construction" is assumed to reflect a midpoint of construction timeframe of approximately July 2014. It is understood that the actual timeframe for implementation of either a treatment alternative or an intake alternative are uncertain. It is also understood that it is expected to take approximately one additional year to complete environmental permitting for an intake alternative as opposed to a treatment alternative; however, no distinction in the escalation rate was made with respect to this difference.

No estimate of land acquisition (none anticipated), legal, and project administration/management by the City are included in the estimated initial capital costs.

The estimated non-construction costs were prepared as follows:

- **Pilot Testing:** Pilot testing was deemed necessary to implement DAF, Actiflo, and Superpulsator clarification, but not plate settling. DAF pilot testing has already been completed. No pilot testing is necessary for additional filtration, and pilot testing is not applicable to the intake alternatives.
- **Property for Screen Control House:** The estimated cost for property rights acquisition for the Screen Control House is an allowance that is subject to substantial potential variation – depending on whether the property is purchased, an easement is required, and whether the property is directly adjacent to Lake Whatcom or not.
- **Geotechnical/Bathymetry:** Substantial geotechnical evaluation will be necessary for the intake alternatives to assess conditions of the lake bottom along the intake pipeline alignment. It will be necessary to drill boreholes at multiple locations along the alignment from a floating barge in the lake. Geotechnical evaluation of the WTP site will be more limited and focused. Bathymetric survey applies only to the intake alternatives.
- **Modeling/WQ Monitoring:** Modeling was completed as part of this project to identify a suitable location and depth for a new intake. However, given the cost of a new intake, additional modeling and focused lake water quality monitoring were deemed warranted to provide further confirmation of the initial modeling.
- **Permitting:** Permitting requirements for a new supplemental intake will be more extensive for a new supplemental intake than for additional treatment. Permits/approvals/lease agreements and associated work products for a new intake system are anticipated to include: US Army Corps of Engineers Section 10 and 404, Biological Assessment, Hydraulic Project Approval, Ecology 401, DNR Lease, Shoreline Development, Critical Area Review, Environmental Impact Statement, and Building. Similarly, new treatment improvements are anticipated to require the following permits/approvals: Shoreline development (depending on siting of the treatment unit), SEPA, and Building.
- **Engineering and Construction Management:** The estimated cost for engineering and construction management was 18 percent for the treatment improvements and 15 percent for the intake improvements. These percentages were based on the construction cost subtotal. The reason for the difference is the reduced complexity of the design for the intake alternatives.
- **Startup:** An allowance for start-up, testing, and trouble-shooting of the new treatment system and equipment, as well as the new intake system, including the new fish screen, was included at a rate of 2 percent of the construction cost subtotal.

5.3.2. Annual Operations and Maintenance

Annual operations and maintenance costs were developed for each of the alternatives and are presented in Table 5-2. It should be noted that no additional estimated labor costs were included for either the treatment improvements or the intake improvements. In both cases, the amount of additional labor by City staff is expected to be minimal, and it is expected that additional routine operations and maintenance requirements can be covered by existing City staff.

For the intake alternatives, the two greatest contributors to the annual operations and maintenance costs are related to the fish screens. Annual fish screen inspections are anticipated to be necessary to confirm the condition and functionality of the fish screens. These inspections can be accomplished by divers or potentially by a remote-controlled, underwater camera. No repair work is associated with these inspection dives.

In addition to the annual fish screen inspections, "10-yr Fish Screen Maintenance" was included as a cost allowance to account for the fact that the fish screens will need to be removed and rehabilitated/repaired or removed and replaced. The fish screens are motorized mechanical devices that will eventually experience some level of failure. Extraction and subsequent re-installation of the fish screens will be costly and require barge and crane equipment. These once-in-10-year costs were divided evenly by 10 to develop an annual estimate of the cost.

Annual operations and maintenance cost for the treatment alternatives was divided into two line items – one based on continuous year-round operation and the other based on 3-month operation. The approach here is based on the acknowledgement that the City may elect to only operate the new treatment systems for the summertime months when they are anticipated to be necessary to address increased algae in Lake Whatcom. That stated, because the new pretreatment systems are expected to substantially improve treatment performance of the existing WTP, it is understood the City may elect operate the new treatment systems on a year-round basis.

5.3.3. Net Present Value

Estimates of net present value over a 20-year period were computed for each of the alternatives to enable comparison of the combined initial capital and annual operations and maintenance costs. Annual operations and maintenance costs presented in Table 5-3 were used in the net present value estimates. An annual interest rate and inflation rate of 5 percent and 3 percent, respectively, were used in the computation. Estimates of net present value were developed for the treatment alternatives based on continuous year-round operation and on 3-month operation.

TABLE 5-2
Summary of Estimated Annual O&M Costs

Process Improvement	Plate Settling	DAF	SuperPulsator	Actiflo	Additional Filters	Intake Alternative 1	Intake Alternative 3
Chemical	$ 25,400	NA	$ 42,400	$ 45,600	NA	NA	NA
Electrical	$ 17,800	$ 30,300	$ 42,800	$ 30,100	$ 5,500	$ 890	$ 3,550
Residuals	$ 1,800	NA	$ 2,100	$ 3,900	NA	NA	NA
Equipment Repair, Replacement, Misc (30%)[1]	$ 13,500	$ 9,100	$ 26,200	$ 23,800	$ 1,600	NA	NA
Fish Screen Building Maintenance	NA	NA	NA	NA	NA	$ 4,230	$ 4,230
Annual Fish Screen Inspection	NA	NA	NA	NA	NA	$ 9,210	$ 27,640
10 yr Fish Screen Maintenance [2]	NA	NA	NA	NA	NA	$ 200,600	$ 477,000
Total (year-round operation)	**$ 59,000**	**$ 39,000**	**$ 113,000**	**$ 103,000**	**$ 7,000**	**$ 34,400**	**$ 83,100**
Total (3-month operation) [3]	**$ 14,750**	**$ 9,750**	**$ 28,250**	**$ 25,750**	**$ 1,750**	*NA*	*NA*

Notes:
1. Equipment repair, replacement, and associated miscellaneous costs for treatment alternatives based on 30% of chemical, electrical, and residuals handling costs.
2. Every 10 years each fish screen will need to be extracted for repair and rehabilitation and replaced. These once-per-10yr costs were divided by 10 to annualize them.
3. Three-month O&M costs based on the assumption that new treatment facilities would only be needed during late summer algae-bloom period.

TABLE 5-3
Summary of Estimated Net Present Value Costs

Process Improvement	Plate Settling	DAF	SuperPulsator	Actiflo	Additional Filters	Intake Alternative 1	Intake Alternative 3
Initial Capital	$ 23,189,000	$ 14,521,000	$ 16,792,000	$ 13,087,000	$ 6,006,000	$ 31,952,000	$ 34,710,000
O&M (year-round operation)	$ 59,000	$ 39,000	$ 113,000	$ 103,000	$ 7,000	$ 34,400	$ 83,100
O&M (3-month operation)	$ 14,750	$ 9,750	$ 28,250	$ 25,750	$ 1,750	NA	NA
NPV Cost (year-round operation)	**$ 24,159,000**	**$ 15,163,000**	**$ 18,650,000**	**$ 14,781,000**	**$ 6,121,000**	**$ 32,490,000**	**$ 36,011,000**
NPV Cost (3-month operation)	**$ 23,432,000**	**$ 14,682,000**	**$ 17,257,000**	**$ 13,512,000**	**$ 6,036,000**	**NA**	**NA**

6. Evaluation of Alternatives

The alternatives that passed the screening phase of evaluation presented in Section 4 were further evaluated, as presented in this section. This further evaluation was completed in two steps. First, the treatment alternatives and the intake alternatives were evaluated separately based on non-cost criteria. Note that the lake management alternative did not pass the alternatives screening phase. Then, once the best treatment alternative and intake alternative were identified based on non-cost evaluation criteria, those two alternatives were evaluated with respect to each other – also based on non-cost criteria. The cost information presented in Section 5.3 was then incorporated into this phase of the evaluation process. The evaluation criteria for this phase of the evaluation process, completed evaluation matrices, and the results of this phase of the evaluation process are presented in the subsections below.

Results of this phase of evaluation were used to select alternatives warranting further scrutiny and evaluation based on the Triple Bottom Line Plus approach. The Triple Bottom Line Plus phase of evaluation is presented in Section 7.

6.1. Evaluation Criteria

A brief summary of the evaluation criteria for the treatment alternatives is presented as follows:

- **Algae removal effectiveness:** This evaluation criterion relates to the effectiveness that treatment processes have demonstrated within the municipal drinking water industry at removing algae.

- **Minimizes algal toxin release:** Although algal toxins are not believed to be an issue with algae that have historically been present in Lake Whatcom, it could potentially become an issue in the future as the algae community in Lake Whatcom changes and as regulatory requirements tighten.

- **Maximizes flexibility to treat emerging contaminants:** Emerging contaminants are constituents that are not currently regulated, but that could be regulated in the future. These contaminants could include micro-biological constituents, pharmaceutical products, fire-retardant products, etc. These constituents are not currently believed to be in Lake Whatcom at substantial or even measurable levels. And, while they are not expected to increase in concentration in the future, it is not out of the realm of possibility that they could increase. Treatment processes that can accommodate adsorption via the addition of powdered activated carbon are generally believed to be more effective at removing these emerging contaminants.

- **T&O effectiveness:** While algae-induced taste and odor has not been a challenging issue for the City's existing WTP, minor to moderate taste and odor observed during the summer time algae season has been observed. Treatment effectiveness related to

minimizing taste and odor impacts could become very important in the future if and when the character and composition of the algae in Lake Whatcom change.

- **Minimizes system complexity/ease of operation:** Treatment systems that are complex, having many mechanical parts and complicated controls, are more prone to equipment failure and are generally not favored. Conversely, treatment systems that are easy to operate with changing raw water quality and equipment failure are generally favored. These systems tend to result in more consistent and reliable performance.

- **Maximizes "sustainability":** Each of the treatment systems considered for this project use energy, produce residual solids that must be handled and disposed, and are comprised to varying degrees of materials. This criterion was included to provide a cursory assessment of the relative sustainability of each treatment alternative.

- **Minimizes "footprint"/siting flexibility:** This criterion enables differentiation between treatment processes that have a larger facility layout. Primary options for siting a new pretreatment facility is the east side of the existing WTP, the north side (influent side), and the west side of the WTP. Treatment processes with a larger facility footprint cannot be as easily on any of these sides of the existing facility, may require purchase of substantial additional property, and possibly environmental mitigation of adjacent forested and wet areas.

- **Minimizes disinfection byproducts:** Clarification processes are generally effective to some degree at removing TOC. TOC combines with chlorine disinfectant to produce disinfectant byproducts. Removing TOC typically results in reduced production of disinfection byproducts. Clarification, when combined with filtration, results in greater combined TOC removal than filtration alone.

A brief summary of the evaluation criteria for the intake alternatives is presented as follows:

- **Minimizes construction disruption:** This criterion relates primarily to construction of the intake pipeline between the existing Gate House and the new intake location. With limited roadway access and egress to the south side of Lake Whatcom, disruption of traffic flow for several weeks would present substantial challenge to the local community.

- **Minimizes permitting challenges:** A new intake will require several environmental permits. Environmental permits will be challenging to obtain regardless of the intake alternative selected.

- **Preserves existing hydraulic capacity:** Although the amount of typical flow through the existing intake system is substantially less than hydraulic capacity, preserving intake capacity for growth and other potential future uses is important and warrants consideration. Alternatives that supplement the continued use of the existing intake preserve existing hydraulic capacity.

- **System complexity / ease of operation:** A new intake with a screen to prevent fish from entering the system will be more complex than the City's existing intake, which does not have a fish screen. Therefore, the alternative with fewer fish screens will be less complex and be easier to operate.

The evaluation criteria for comparing the best of the treatment and the best of the intake alternatives are listed below and are mostly the same or similar to the evaluation criteria summarized above. These criteria were modified, where necessary and applicable, to be relevant to both treatment and intake alternatives. More discussion of the evaluation of each of these "best" alternatives with respect to these evaluation criteria is presented in Section 6.2.3.

- Minimizes construction disruption to the community
- Minimizes permitting challenges
- Long-term certainty of continued effectiveness
- Maximizes sustainability
- Minimizes WTP disruption
- Minimizes system complexity / ease of operation

6.2. Evaluation Matrices

Evaluating the alternatives was completed as a group in a workshop setting by the project team comprised of City of Bellingham and CH2M HILL staff. Evaluation matrices presented as Exhibits 6-1, 6-2, and 6-3 were completed by the group based on the evaluation criteria described in Section 6.1. Exhibit 6-1 was completed for the treatment alternatives based on the selected criteria relevant to these types of alternatives. Similarly, Exhibit 6-2 was completed based on evaluation criteria relevant to the intake alternatives. The alternatives presented in Exhibits 6-1 and 6-2 with the highest score – the alternatives deemed the best within their category – were evaluated and ranked, as presented in Exhibit 6-3.

To complete the evaluation matrices, the group assigned a relative weight or importance (from 1 to 5) to each of the evaluation criteria. Then, each alternative was ranked (also from 1 to 5) with respect to each evaluation criteria. Total scores for each alternative were computed by multiplying the weight for each evaluation criterion by the assigned ranking. Each of those multiplication products were summed to produce a total score for each alternative.

6.3. Results of Evaluation

Evaluation results are presented separately in the subsections below for the treatment alternatives, intake alternatives, and the best alternatives from each of those two categories.

Exhibit 6-1

Evaluation Matrix for Treatment Alternatives

Alternative	Algae removal effectiveness	Minimizes algal toxin release	Maximizes flexibility to treat emerging contaminants	T&O effectiveness	Minimizes system complexity / ease of operation	Maximizes "Sustainability"	Maximizes siting flexibility	Minimizes disinfection byproducts	Total Score
	Evaluation Criteria								
	Criteria Weighting (1)								
	5	4	3	4	4	4	3	3	
Dissolved Air Flotation (DAF)	5	5	3	5	3	3	4	4	**122**
Ballasted Sedimentation (Actiflo)	4	2	4	4	2	2	5	4	**99**
Plate Settling	3	3	4	4	4	4	1	3	**99**
Upflow Clarification (Superpulsator)	3	3	4	4	2	3	3	4	**96**
Additional Filters	3	3	1	1	4	4	4	1	**81**

Notes:

(1) Criteria weighting reflects relative importance (5 = most important; 1 = least important).

(2) Relative scoring of each alternative with respect to each criterion:
5 = excellent; 4 = very good; 3 = satisfactory; 2 = questionable; 1 = unacceptable

Exhibit 6-2

Evaluation Matrix for Intake Alternatives

Alternative	Minimizes Construction Disruption	Minimizes Permitting Challenges	Preserves Existing Hydraulic Capacity	System Complexity / Ease of Operation	Total Score
	Evaluation Criteria				
	Criteria Weighting (1)				
	3	2	5	4	
Secondary Intake via In-Water Pipeline	3	3	3	4	**46**
Replace Existing Intake	3	3	1	3	**32**

Notes:

(1) Criteria weighting reflects relative importance (5 = most important; 1 = least important).

(2) Relative scoring of each alternative with respect to each criterion:

5 = excellent; 4 = very good; 3 = satisfactory; 2 = questionable; 1 = unacceptable

Exhibit 6-3
Evaluation of Best Intake and Best Treatment Alternatives

	Evaluation Criteria						
	Minimizes construction disruption to the community	Minimizes permitting challenges	Long-term certainty of continued effectiveness	Maximizes sustainability	Minimizes WTP disruption	Minimizes system complexity / ease of operation	Total Score
	Criteria Weighting (1)						
Alternative	**3**	**2**	**5**	**2**	**2**	**4**	
Intake Alternative	2	2	2	4	4	2	**44**
Treatment Alternative (DAF)	3	4	5	3	2	3	**64**

Notes:
(1) Criteria weighting reflects estimated relative importance of each criterion (5 = most important; 1 = least important).
(2) Relative scoring of each alternative with respect to each criterion:
5 = excellent; 4 = very good; 3 = satisfactory; 2 = questionable; 1 = unacceptable

6.3.1. Treatment Alternatives

As presented in Exhibit 6-1, DAF is the treatment alternative that received the highest ranking with respect to the other alternatives within the treatment category, and was thereby deemed to be the "best" of the treatment alternatives to mitigate the algae condition at the City's WTP. As shown by the ranking, DAF was deemed to be superior to the other alternatives by a wide margin.

Ranking of the treatment alternatives produced the following key results:

- DAF was far superior to the other alternatives. The main reason for this was the superior algae removal effectiveness of DAF, which was deemed to be the most important evaluation criteria. Secondarily, DAF was ranked markedly higher than the other pretreatment alternatives with respect to minimizing algal toxin release as well as treating for taste and odor impacts. While these two factors have not yet substantially evidenced themselves in the Lake Whatcom supply, it is uncommon for water supplies with algae issues to not have also have algal toxin or taste and odor issues to some degree.

- The pretreatment alternatives other than DAF (Ballasted Sedimentation, Plate Settling, and Upflow Clarification) each had similar scores that were substantially lower than the ranking for DAF and substantially greater than the ranking for the Additional Filters alternative. The primary reason for this is their reduced performance with respect to DAF, as stated above. However, it should also be noted that these pretreatment

alternatives each offer substantial benefit with respect to algae removal and the other evaluation criteria when combined with the effective filtration process the City currently employs.

- The Additional Filters alternative received an overall ranking that was substantially below all of the other treatment alternatives. Its ranking was lower primarily because it does not offer most of the same primary and ancillary benefits offered by the pretreatment improvements. Simply adding more filters does not result in the ancillary benefits achieved by pretreatment process that substantially improve water quality prior to the filtration process, which results in improved filter run times as well as capacity. The greatest concern with implementing this alternative is that if future algae blooms in Lake Whatcom result in filter run times that are markedly lower than what was observed during the algae bloom of 2009, this alternative may offer no additional benefit.

Evaluating the treatment alternatives based on non-criteria resulted in a clearly-preferred technological approach. Comparing those results with the estimated costs presented in Section 5.3 enables further confirmation of the treatment approach deemed best-suited for this application.

The results of the evaluation ranking presented in Exhibit 6-1 and the estimated Net Present Values presented in Table 5-3 are presented graphically in Figure 6-1. What is clear from Figure 6-1 is that the lowest-ranking alternative, Additional Filters, had the lowest estimated cost and the highest-ranking alternative, DAF, had an estimated cost similar to Actiflo, which was the second lowest cost and the lowest of the high-rate clarification processes. Given that the lowest-ranking alternative is the least-well-suited for this application from a treatment technology standpoint, it would be challenging to make a case for its selection. Therefore, the highest-ranking alternative from a treatment technology standpoint appears even more attractive given that its cost is nearly the same as lowest of the other treatment alternatives.

6.3.2. Intake Alternatives

As presented in Exhibit 6-2, Intake Alternative 1 was ranked higher than Intake Alternative 3. The primary reasons for this result are: (1) Intake Alternative 1 makes use of the existing intake pipeline, which has capacity that exceeds the hydraulic capacity of the existing tunnel, and (2) Intake Alternative 1 has fewer new fish screens than Intake Alternative 3 and is therefore less complex and challenging to operate.

Adding the estimated costs presented in Section 5.3 provides further confirmation of Intake Alternative 1 as the intake approach deemed best-suited for this application. The results of the evaluation ranking presented in Exhibit 6-2 and the estimated Net Present Values presented in Table 5-3 are presented graphically in Figure 6-2. What is clear from Figure 6-2 is that Intake Alternative 1 is the best approach based on non-cost evaluation criteria and is also the lower-cost intake alternative.

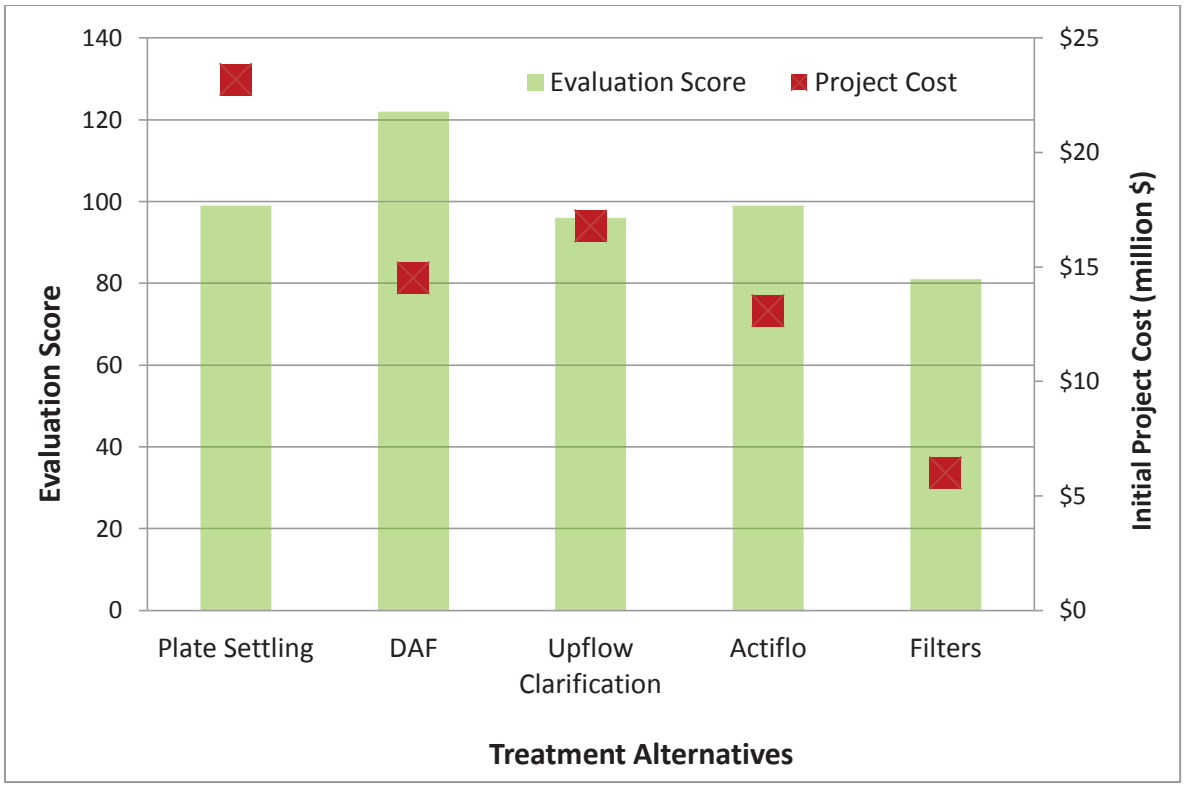


FIGURE 6-1
Treatment Alternative Evaluation Results and Estimated NPV Costs

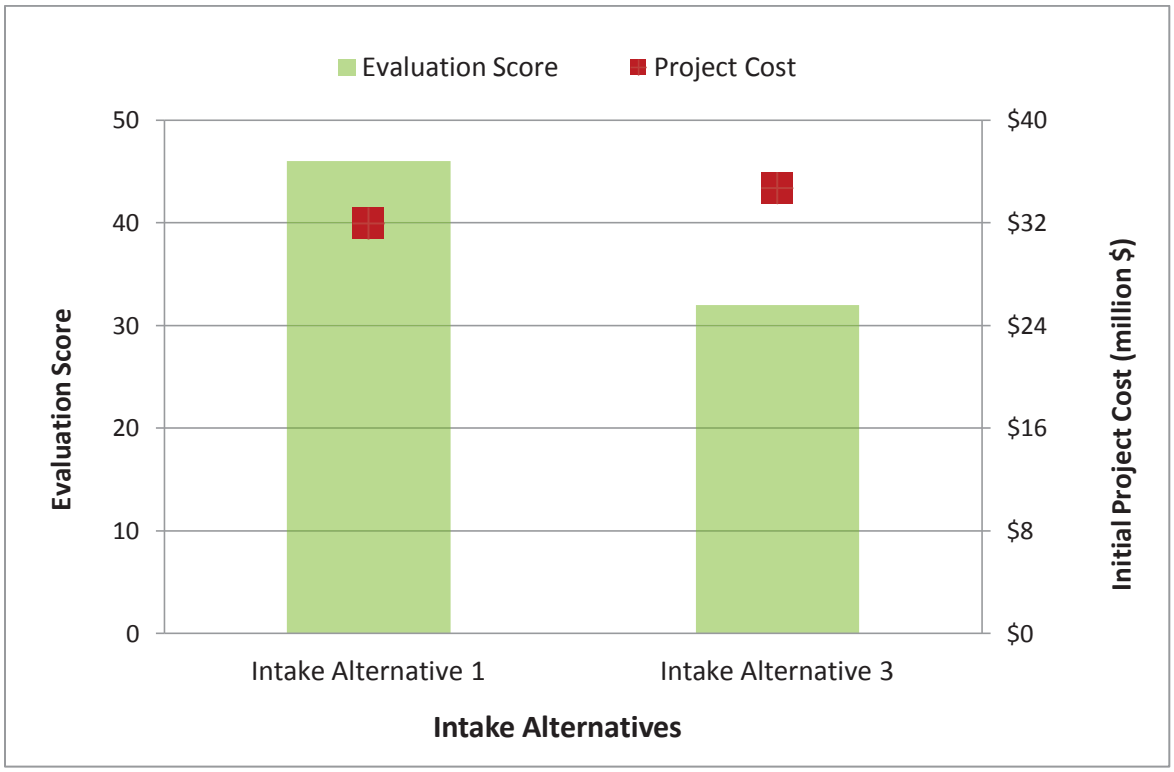


FIGURE 6-2
Intake Alternative Evaluation Results and Estimated NPV Costs

6.3.3. Comparison of Best Intake and Treatment Alternatives

As presented in Exhibit 6-3, DAF was ranked higher than Intake Alternative 1 with respect to the non-cost evaluation criteria developed specifically for this direct comparison. Specifically, the difference between these two alternatives with respect to each evaluation criteria is as follows:

- **Minimizes construction disruption to the community:** While there will be construction vehicle to and from the WTP site for a new DAF treatment unit, there is anticipated to be less disruption to the community because of construction at the WTP site than in and on the shoreline of Lake Whatcom. The WTP is mostly out of site from the residential and commercial public.

- **Minimizes permitting challenges:** Because of the extensive in-water work in Lake Whatcom, the Intake Alternative 1 will require more permitting and the associated time and expense associated with permitting than the DAF treatment alternative.

- **Long-term certainty of continued effectiveness:** The DAF treatment process has an extensive history of effectively and reliably removing algae and is known to be the best available treatment technology available for algae removal. DAF is acknowledged to be effective at removing algae of varying speciation and concentration; therefore, its certainty of continued effectiveness is high. Intake Alternative 1 is based on locating a new, supplemental intake deep within the hypolimnion of Basin 3 – below the level where most algae, in particular blue-greens, are known to reside. Additionally, historical and ongoing monitoring of dissolved oxygen levels in Basin 3, at depth, show that they are mostly relatively high when compared to the anoxic conditions in the hypolimnion Basins 1 and 2. What is uncertain is how long those dissolved oxygen levels will remain high enough to avoid problem associated with anoxia. Therefore, the uncertainty as it relates to Intake Alternative 1 does not relate primarily to the future presence of algae at depth, but instead to the potential for anoxic conditions that could result in the type of treatment challenges described in Section 3.2.2.

- **Maximizes sustainability:** DAF and Intake Alternative 1 were ranked similarly with respect to sustainability, but it was believed that Intake Alternative 1 could be viewed slightly more sustainable because it is associated with less electrical power consumption, chemical consumption, and production of solids that require disposal.

- **Minimizes water treatment WTP disruption:** While construction of a new DAF treatment process would be undertaken without extended disruption to the existing WTP, it would present extensive coordination challenges to operations staff and result in periodic WTP shut downs. Intake Alternative 1 could be implemented with minimal impact to WTP operations, with only one or two brief shut downs when the connection to the existing Gate House is made.

- **Minimizes system complexity / ease of operation:** The two alternatives were relatively similar with respect to this criterion; however, Intake Alternative 1 was deemed to be less attractive with respect to this criterion. The primary reason for this is the mechanical fish

screen deep in Lake Whatcom will require annual inspection and periodic retrieval and repair to maintain. These activities, while periodic will require extensive coordination to address equipment that is not readily accessible.

The results of the evaluation ranking presented in Exhibit 6-3 and the estimated Net Present Values presented in Table 5-3 are presented graphically in Figure 6-3. While DAF was ranked substantially higher with respect to non-cost criteria, as presented in Exhibit 6-3, its estimated 20-year Net Present Value (based on year-round operation) was much less than the same for Intake Alternative 1. DAF is ranked higher than Intake Alternative 1 based on non-cost evaluation criteria and is much less costly to implement.

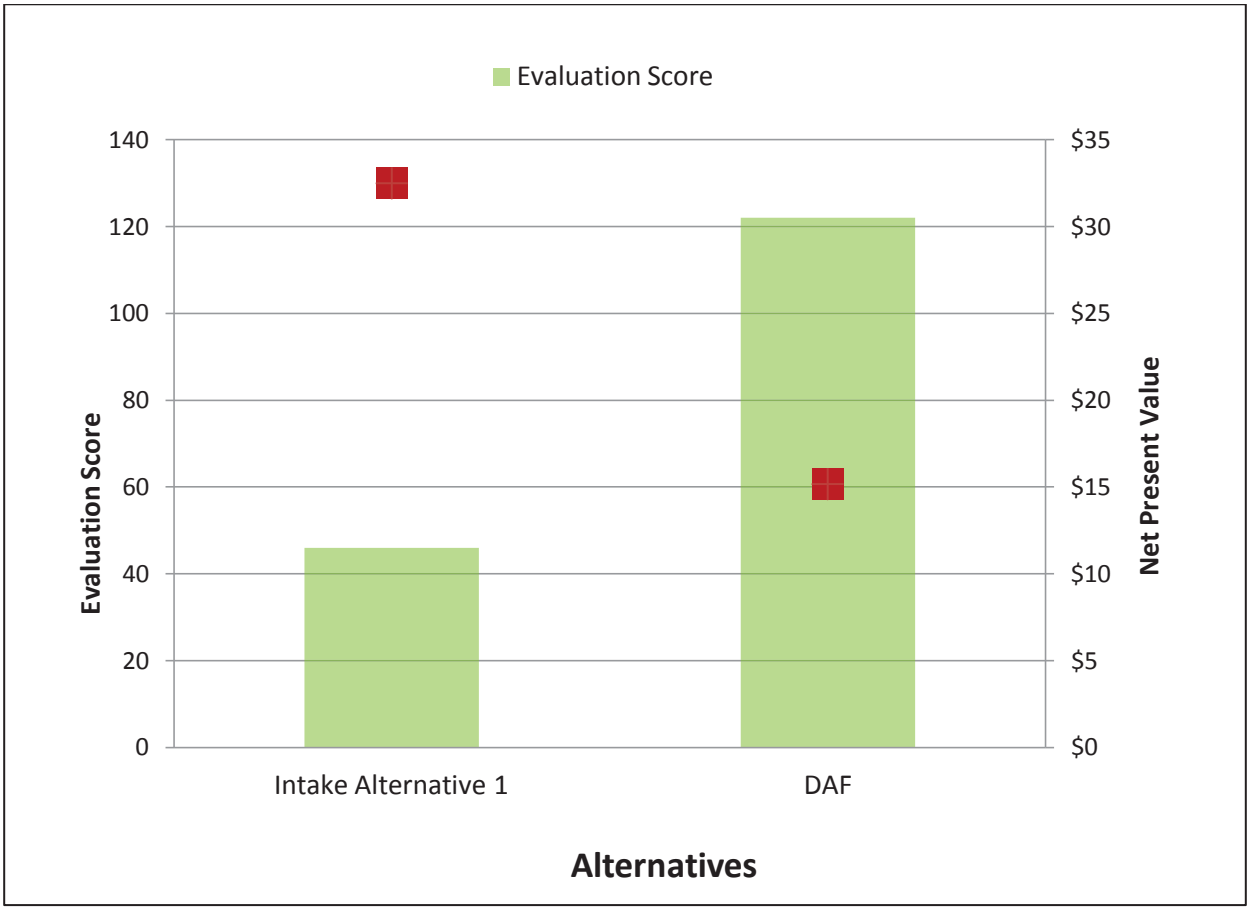


FIGURE 6-3
Best Treatment and Intake Alternatives Evaluation Results and Estimated NPV Costs

6.3.4. Summary of DAF Pilot Testing

DAF was ranked as the best approach for mitigating the adverse impacts of filter-clogging algae at the City's WTP. This result is consistent with DAF's recognized standing in the municipal water treatment industry as the best, most effective, and most reliable available technology for removing algae prior to filtration. In recognition of this standing, as well as the need to pro-actively develop an effective mitigation approach to the filter-clogging algae, pilot testing of DAF was undertaken during the summer of 2011 when algae

concentrations in Lake Whatcom were expected to be at their highest for the season. While the DAF pilot testing was implemented prior to completion of the formal process to evaluate alternatives to mitigate the filter-clogging at the City's WTP, the results of the DAF pilot testing process were not incorporated into the evaluation process. A copy of the DAF pilot testing report is presented in Appendix C.

The pilot testing showed that DAF was very effective at removing algae from the Lake Whatcom supply. Not only was it effective at removing algae, but it was also shown to be effective at removing total organic carbon (TOC), reducing the formation potential for total trihalomethanes (TTHMs) – a key disinfection byproduct, and most-importantly it was shown to greatly extend filter runs. Extended filter runs results in increased total filter production during algae bloom conditions, which was the primary limitation during the 2009 Lake Whatcom algae bloom.

The DAF process tested during the 2011 testing was shown to be effective at a wide range of DAF hydraulic loading from 10 up to 20 gpm/sf. Hydraulic loading rate is a key design criterion with respect to facility cost. The higher the loading rate, the lower the plan layout of the facility, and the lower the facility cost. The demonstrated success of the 20 gpm/sf DAF loading rates during pilot testing enables consideration of a phased implementation approach. A phased implementation approach could result in an initial capital cost substantially less than that presented in Table 5-1. This phased implementation approach is presented in Section 8.

7. Triple Bottom Line Plus

The Triple Bottom Line Plus (TBL +) evaluation approach has a demonstrated track record of enabling consideration of alternatives based on criteria that fall into four key categories – financial, social, environmental, and technical. The City is familiar with the TBL + evaluation approach and has experience using it to evaluate alternative infrastructure improvements. The TBL + approach was used for this project, as presented in this section, for evaluating alternatives that:

- Passed the alternatives screening process presented in Section 4, thus eliminating alternatives deemed to not be feasible, effective, or otherwise warrant further consideration
- Were determined to be the best within their alternative category (i.e. treatment, intake, and lake management), as presented in Section 5
- Were relatively low cost and potentially attractive even if they were not determined to be the best within a particular alternativecategory

In addition, the alternatives selected for evaluation using the TBL + approach were also compared against doing nothing to address this filter-clogging algae issue – the "No Action."

7.1. Applicability of TBL+ to Evaluation Approach

The TBL + evaluation approach is effective at addressing different types or categories of alternatives because they tend to vary with respect to each of the four key evaluation categories (financial, social, environmental, and technical). Conversely, while TBL + can be used to evaluate similar alternatives, such as the treatment alternatives considered for this project, it is more effective to evaluate such alternatives based on key technical and financial criteria – as was done for this project. For this project, this TBL + evaluation approach was reserved, as stated above, for evaluating the best technical alternatives, a selected low-cost promising alternative, and the "No Action" alternative.

This TBL + evaluation phase builds on the alternatives screening phase presented in Section 4 by focusing only on alternatives that were deemed feasible and warranting of further evaluation and scrutiny – alternatives that don't have any "fatal flaw" characteristics associated with them. As stated, this phase of the evaluation process was not employed to distinguish between similar types of alternatives that are best evaluated based on specific technical evaluation criteria.

The alternatives evaluated using the TBL+ approach presented herein and the rationale for their inclusion in this phase of the evaluation process is presented below:

- **Dissolved Air Flotation:** DAF was determined to be the best of the treatment alternatives, as presented in Section 6.
- **No Action:** This alternative enables direct comparison of the best alternative, regardless of its alternative category, with deferring action.

- **Intake Alternative 1:** This alternative was determined to be the best of the intake alternatives, as presented in Section 6.
- **Additional Filters:** This alternative is the lowest-cost treatment alternative and could be effective if algae events do not reduce filter run times to less than two hours.

7.2. TBL+ Evaluation Method and Criteria

Each of the four evaluation categories (e.g. financial, social, environmental, and technical) were weighted equally in terms of importance and each of the categories were divided into two specific objectives. One, two, or three key evaluation criteria were identified for each objective to enable assessment of whether the objectives were met or not. The evaluation criteria were developed to allow for either a "yes" or "no" response. "Yes" responses indicate that the criterion has been met. A summary of the evaluation objectives and criteria within each evaluation category are presented in Table 7-1. Each objective and criterion are designated with an identification number to aid correlation between Table 7-1 and Figure 7-1, presented in the following section.

TABLE 7-1
TBL + Evaluation Objectives and Criteria

Evaluation Category	Objective	Criteria
Financial	**F1:** Minimize capital cost	**F1.1:** Is capital cost less than mean (average) of the four alternatives?
	F2: Minimize life-cycle cost	**F2.1:** Is life-cycle cost less than mean of the four alternatives?
		F2.2: Eliminates reduced water sales because of mandatory water restrictions?
Social	**S1:** Protect public health and safety	**S1.1:** Enables uninterrupted, full-capacity use of plant?
		S1.2: Reduces disinfection byproducts?
		S1.3: Avoids construction activities in public-accessed areas?
	S2: Preserve community reputation, status, and economic vitality	**S2.1:** Eliminates need for mandatory water restrictions and associated negative press?
Environmental	**E1:** Minimize local impact	**E1.1:** Avoids large increases in wasted filter backwash water during algae events?
	E2: Minimize global impact	**E2.1:** Is life-cycle greenhouse gas less than the mean of the four alternatives?
		E2.2: Is energy use less than the mean of the four alternatives?
Technical	**T1:** Maximize treatment reliability	**T1.1:** Proven effective and reliable technology?
		T1.2: Enables treatment at full plant capacity during algae events?
	T2: Maximize treatment performance	**T2.1:** Results in improved treated-water quality?

7.3. City of Bellingham Values

The TBL+ evaluation objectives and criteria presented above are in alignment with the City's goals and objectives, which are summarized in the "Legacies and Strategic Commitments" document presented in Appendix D. They are also in alignment with the City's Public Works Department's mission statement, which is comprised of:

"Enhance Bellingham's quality of life through the construction and operation of a safe, effective physical environment; to protect public health & safety and the natural environment; and to provide our neighborhoods, our businesses and our visitors with the efficient, quality services necessary to meet the demands of our growing, diverse community."

The mission statement addresses each of the three categories of the TBL evaluation approach. Combining the technical evaluation criteria forms the basis of a complete, comprehensive evaluation framework. The City has a long and established commitment to social equity and environmental protection in addition to balancing financial impacts with technical performance. This TBL+ phase of the evaluation process reflects even-handed consideration of these four evaluation categories.

7.4. Results of TBL+ Evaluation

The results of the TBL+ evaluation process is presented graphically in the bar chart presented in Figure 7-1. The bar chart presents the relative ranking of each of the four alternatives. Based on the evaluation criteria developed, DAF was ranked highest of the four alternatives. This result is in alignment with the evaluation presented in Section 6, which ranks DAF as the best alternative.

7.5. Discussion of Results of TBL+ Evaluation

A discussion of the rationale for the evaluation results for each of the evaluation criteria and each of the alternatives is presented in the following subsections. The subsection headers below identified as each of the evaluation criteria.

7.5.1. Is capital cost less than the mean (average) of the four alternatives? (F1.1)

The estimated capital cost for the alternatives is listed below – along with the mean of the estimated capital cost for each alternative. These costs are the same as those presented in Section 5. Clearly, the capital cost for the No Action alternative is $0. The alternatives are listed in order of least cost to greatest cost – reflecting that the No Action and Additional Filters alternatives met this evaluation criterion.

- No Action: $0
- Additional Filters: $6,006,000
- **Mean: $13,119,750**
- DAF: $14,521,000
- Intake Alternative 1: $31,952,000

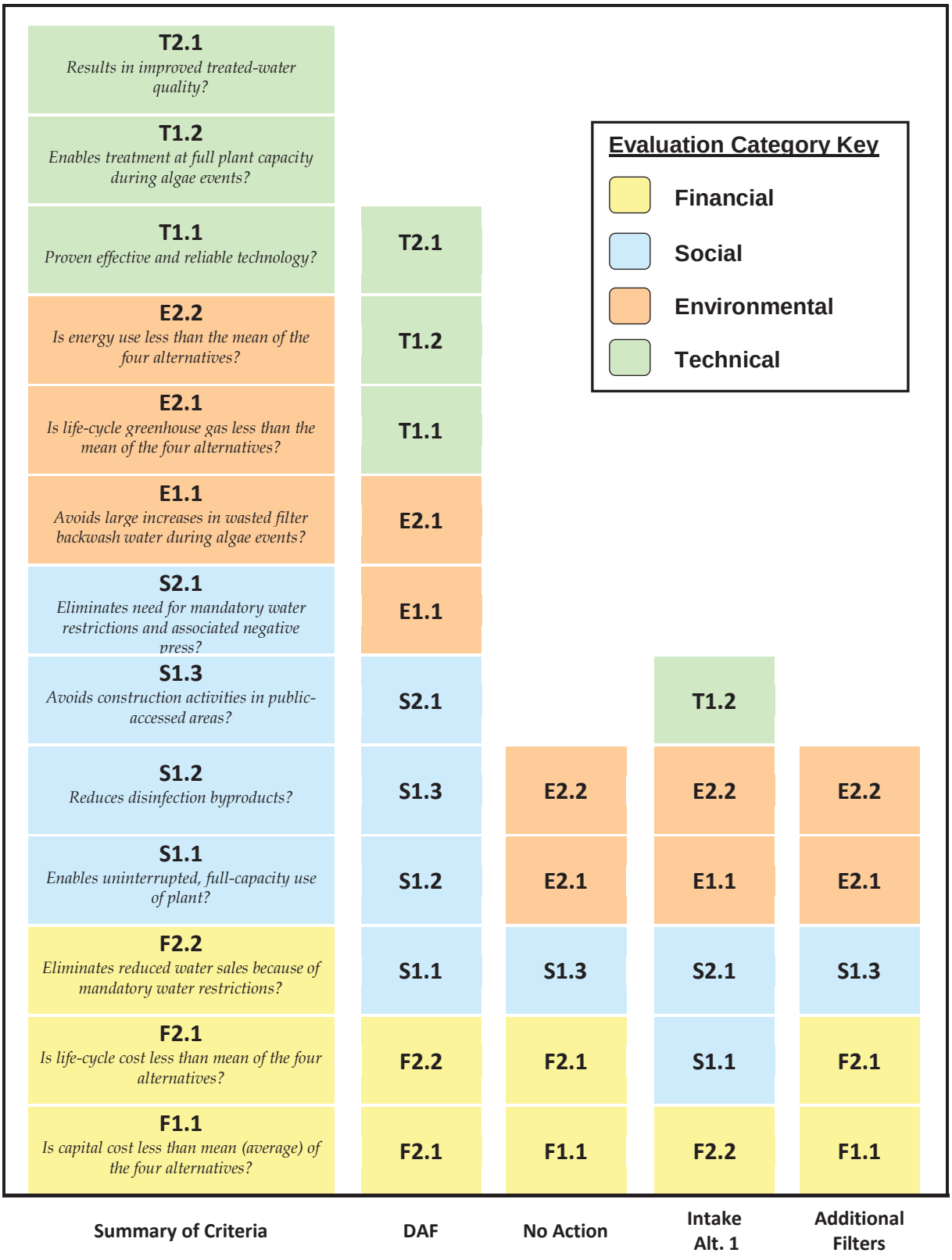


FIGURE 7-1
TBL+ Evaluation Results

7.5.2. Is life-cycle cost less than mean of the four alternatives? (F2.1)

The results for this criterion were the same as for F1.1 presented above. The estimated life-cycle cost for the alternatives is listed below – along with the mean of the estimated capital cost for each alternative. These costs are the same as those presented in Section 5 for the DAF, Additional Filters, and Intake Alternative 1 alternatives.

The life-cycle cost for the No Action alternative is based on a 20-year period of lost revenue due to the impact of mandatory water restrictions. An annual interest rate and inflation rate of 5 percent and 3 percent, respectively, were used in the computation. The following assumptions were made in developing this estimated life-cycle cost:

- The entire City is metered. This assumption is justified because the entire City will be metered by 2017, as required by statute. This assumption enables use of the City's volume rate to compute the lost revenue.
- The City's 2012 Inside-City volume rate of $1.53 per 1 CCF (100 cubic feet) was used to compute the annual lost revenue.
- It was assumed that mandatory water restrictions would occur every other year and when they occur they dampen total City demand by 10 mgd for 15 days, 5 mgd for 20 additional days, and 3 mgd for 25 additional days. These are speculative assumption is based on 15 days of mandatory water restrictions (which severely reduces customer demand by 10 mgd) and a residual follow-on impact on customer demand once the mandatory water restrictions are removed. The total reduction in customer demand would be 350 million gallons or approximately 470,000 CCF. This equates to a total of $720,000 of lost revenue over the course of the year when the mandatory restrictions occur.

It should be noted that no similar estimate of lost revenue from water sales was developed for the Additional Filters alternative in recognition of the difficulty in assessing the intensity of any future algae blooms and whether or not they would result in the need for mandatory water restrictions. Therefore, the estimated life-cycle cost of the Additional Filters alternative should be considered to be "at least" the value presented in Table 5-3 – given that it could be greater if future potential lost revenue were accounted.

The alternatives are listed in order of least cost to greatest cost – reflecting that the No Action and Additional Filters alternatives met this evaluation criterion.

- No Action: $6,000,000
- Additional Filters: $6,036,000
- DAF: $14,682,000
- **Mean: $14,802,000**
- Intake Alternative 1: $32,490,000

7.5.3. Eliminates reduced water sales because of mandatory water restrictions? (F2.2)

DAF removes algae from the raw water prior to filtration. Therefore, the filters operate efficiently and there is no need for mandatory water restrictions. Intake Alternative 1 avoids live algae by withdrawing water from deep in Basin 3. Therefore, there would be no mandatory water restrictions related to this alternative. Conversely, the No Action alternative is anticipated to result in mandatory water restrictions and associated lost revenue from water sales. Although it was quantified, as discussed above, it should be expected that the Additional Filters alternative would, to a lesser degree than the No Action alternative, result in mandatory water restrictions and associated lost revenue from water sales.

7.5.4. Enables uninterrupted, full-capacity use of plant? (S1.1)

DAF and Intake Alternative 1 both enable full-capacity use of the City's WTP because they either remove or avoid filter-clogging algae prior to the filtration process. Conversely, the No Action and Additional Filters alternatives do not include changing the algae concentration of the water treated by the filters. As such, these two alternatives are defined by their reduced-filter-capacity characteristics.

7.5.5. Reduces disinfection byproducts? (S1.2)

Disinfection byproducts such as total trihalomethanes and haloacetic acids have been linked to cancer development and are regulated at the federal and state level because of their health impacts. The concentration of these byproducts in the City's distribution system is well below the regulatory limit. However, reducing disinfection byproducts – regardless of the absolute concentration – is generally considered to be beneficial from a public health standpoint.

As presented in Section 3, DAF reduces disinfection byproducts (primarily total trihalomethanes) because it removes the organic precursors that combine with chlorine to form the disinfection byproducts. The No Action and Additional Filter alternatives do nothing to change the treatment process at the WTP; therefore, there is no reduction in disinfection byproducts. Intake Alternative 1 withdraws water from a different part of the lake than current; however, the total organic carbon (TOC) concentration at lower lake depths has been shown to typically be greater than at shallow depths. Therefore, disinfection byproducts would, at best, not be reduced with Intake Alternative 1, and could, in fact, increase.

7.5.6. Avoids construction activities in public-accessed areas? (S1.3)

DAF and the Additional Filters alternatives both involve construction at the City's WTP site, which is in a portion of Whatcom Falls Park that is restricted from public access. The No Action alternative does not involve construction activities. Only Intake Alternative 1 would include construction within areas that are access by the public. Public health and safety is

more easily achieved when interaction between the public and construction activities can be avoided.

7.5.7. Eliminates need for mandatory water restrictions and associated negative press? (S2.1)

Negative press related to water restrictions could adversely impact the City's reputation and reduce its ability to attract new business and preserve economic vitality. For the reasons described above for evaluation criteria F2.2 and S1.1, DAF and Intake Alternative 1 avoid mandatory water restrictions and associated negative press while the No Action and Additional Filters alternatives do not.

7.5.8. Avoids large increases in wasted filter backwash water during algae events? (E1.1)

DAF and Intake Alternative 1 mitigate the filter-clogging algae that enter the City's WTP via the existing intake by removing the algae and by avoiding live algae, respectively. Therefore, there are no large increases in wasted filter backwash water during algae events. Conversely, both the No Action alternative and Additional Filters alternatives rely on the increased frequency of filter backwashing during algae events to keep the filters in service, which results in large increases in filter backwash water wasted to the sanitary sewer system.

7.5.9. Is life-cycle greenhouse gas less than the mean of the four alternatives? (E2.1)

A 20-year life-cycle greenhouse gas total was computed, in terms of CO_2 production, for each of the alternatives based on their initial construction combined with 20 years of annual CO_2 production. The greenhouse gas estimates are for the incremental additional quantity associated with each alternative beyond what is already produced annually as part of normal WTP operations. The largest component of annual production was electric power consumption, which is what the annual CO_2 production was based upon. The initial construction total was based on all key elements of construction, including transport of materials and labor as well as disposal of waste excavation.

- No Action: 240 tons of CO_2 (0 construction; 12 annual)
- Additional Filters: 480 tons of CO_2 (240 construction; 12 annual)
- DAF: 1,420 tons of CO_2 (380 construction; 50 annual)
- **Mean: 2,120 tons of CO_2**
- Intake Alternative 1: 6,350 tons of CO_2 (6,250 construction, 5 annual)

It is interesting to note that the life-cycle estimates for each of the four alternatives varied substantially with respect to their total, but also with respect to the component elements comprising the total (initial construction and annual operations). The No Action alternative produces the least amount of greenhouse gas because it involves no new construction and because the associated additional filter backwash pumping is for only a portion of the year.

The Additional Filters alternative is essentially the same as the No Action alternative, except for the construction of the two new filters. DAF operations, because of seasonal higher electric power consumption, produces more greenhouse gas than the No Action and Additional Filters alternatives, but still less than the mean of the four alternatives. Intake Alternative 1 produces by far the most greenhouse gas because of the large amount associated with the steel pipeline and steel piles. Steel production and fabrication is a very high producer of greenhouse gas compared to other activities such as concrete construction and excavation.

7.5.10. Is energy use less than the mean of the four alternatives? (E2.2)

Annual energy (electrical) use for each of the alternatives, except the No Action alternative, is presented in Table 5-2. At first, it may appear that the No Action alternative uses no energy because it does not involve additional mechanical equipment that requires electrical power. However, for the purpose of this evaluation criterion the increased energy associated with increased filter backwashing during an algae event was estimated.

This electrical power was based on estimating the extra filter backwashing that would be necessary as opposed to if there were little or no algae to contribute to filter clogging. The electrical power estimate was based on use of filter backwash pumps (two 75-hp motors), the air-scour blower (one 75-hp motor), and an estimate of roughly 10 times the amount of filter backwashing at the peak of an algae bloom as when no algae bloom exists, which is consistent with observed conditions from the summer 2009 algae bloom. The estimated increased power usage on the peak day was estimated to be 1,300 kilowatt hours. For the purpose of this evaluation it was assumed that there would be roughly 20 days of peak-algae-bloom impact every year.

It is understood that an actual algae bloom would likely extend for a longer period of time, but not at the same intensity; therefore, the assumption presented here represents a reasonable "ballpark" estimate the additional seasonal electrical power impact from additional filter backwashing. Also for the purpose of this evaluation criterion, this additional electrical power usage was included with the Additional Filters alternative because with respect to electrical power consumption, these two alternatives are nearly the same, with the exception that the Additional Filters alternative produces more net treated water under most conditions because it includes eight filters instead of the existing six. The power use associated each of four alternatives is presented as follows:

- Intake Alternative 1: 10,000 kilowatt hours
- No Action: 26,000 kilowatt hours
- Additional Filters: 26,000 kilowatt hours
- **Mean: 36,000 kilowatt hours**
- DAF: 84,000 kilowatt hours (3-month operation)

Intake Alternative 1 has relatively little electrical power consumption, and it is related entirely to the continually-operated rotating cleaning brushes on the fish screen. The No

Action and Additional Filters alternatives have a moderate amount of additional electrical power consumption related to the additional filter backwashing that occurs seasonally. DAF is more mechanically complex because of its flocculation mixers and diffused-air system and requires more electrical power than the other three alternatives over the anticipated 3 months of operation.

7.5.11. Proven effective and reliable technology? (T1.1)

DAF has demonstrated its effectiveness and reliability at removing algae under a variety of conditions for many years. DAF is ideally suited to remove algae and is ideally suited to remove algae from the consistent, low-turbidity Lake Whatcom supply. Conversely, the No Action and Additional Filters alternatives do not have a history of demonstrated success at removing algae. Withdrawing water from a part of the lake where live algae doesn't currently exist would be beneficial from the standpoint of reducing filter clogging. However, other water quality problems should be expected, as presented in Section 3.2.2, that makes pursuing intake relocation unattractive.

7.5.12. Enables treatment at full plant capacity during algae events? (T1.2)

A treatment system that cannot operate efficiently at its maximum capacity at a time when that maximum capacity is needed, is ineffective at meeting the peak customer demands for which it was designed. For the reasons described above for evaluation criteria F2.2 and S1.1, DAF and Intake Alternative 1 avoid filter-clogging algae impacts and enable operation of the City's WTP at its full capacity – even during algae events. Conversely, the No Action and Additional Filter alternatives do not.

7.5.13. Results in improved treated-water quality? (T2.1)

The No Action and Additional Filters alternatives do not include any modification of the existing treatment process and therefore have no impact on treated-water quality. Intake Alternative 1, as described in Section 3.2.2 and Section 7.5.5, will likely result in reduced treated-water quality. Conversely, DAF will result in improved treated-water quality because it will reduce disinfection byproducts (primarily total trihalomethanes) and TOC. DAF pilot testing completed during the late summer of 2011on the Lake Whatcom supply showed that DAF could reduce total trihalomethanes production by 25 percent. DAF will also reduce algal toxins that may enter the system and will help reduce taste and odor resulting from algae and algae respiration byproducts. DAF also will enable improved treatment of contaminants that may be regulated in the near- or long-term future, such as microbiological, pharmaceutical, and fire-retardant contaminants. It is understood that these contaminants are likely not present at measurable levels in Lake Whatcom presently, and that ongoing efforts to improve Lake Whatcom quality should keep Lake Whatcom clear of these contaminants.

8. DAF Implementation

In recognition of the results of Sections 6 and 7 – DAF's ranking as the best alternative for filter-clogging algae mitigation – a discussion of DAF implementation is presented in this section. The implementation discussion presented herein covers regulatory agency requirements, an example project schedule, and options for reducing project cost. In doing so, it describes the approximate overall process and timeframe for DAF implementation. The purpose for presenting this DAF implementation discussion is to aid the decision-making process with respect to pursuit of a filter-clogging algae mitigation approach. The implementation discussion here is not intended to establish at this time the details of an implementation approach, sequence, and schedule.

8.1. Regulatory Agency Requirements

The purpose of this section is to summarize the approval requirements of the regulatory agency with jurisdiction over municipal water treatment, which is Washington Department of Health (DOH). Other permits and approvals will also be necessary, including but not limited to, State Environmental Policy Act (SEPA) approval, a building permit, a clearing and grading permit, and possibly critical areas and shorelines permits. However, compliance with DOH approval at key milestones will define the order of implementation activities. The key DOH approval activities and a brief associated discussion are summarized as follows:

- **Pilot Testing Plan (Protocol):** Pilot testing is required for all treatment system improvements, with very limited exceptions. Pilot Testing Plans are required per Section 12. 3.3 of DOH's Water System Design Manual and WAC 246-290-676(3)(b). The Pilot Testing Plan for the DAF pilot testing that was completed during the late summer of 2011 was already reviewed and approved by DOH in June of 2011.

- **Pilot Testing Report:** A Pilot Testing Report summarizing the activities and results of a pilot test is required per Section 12.3.4 of the Water System Design Manual and WAC 246-290-676(3)(e). Upon completion in September 2011DAF pilot testing, a Pilot Testing Report was completed; however, it has not been submitted to DOH for approval because the City has not decided to pursue DAF implementation. The Pilot Testing Report is available to be submitted to DOH should the City decides to pursue implementation of a new DAF treatment process.

- **Water System Plan Amendment:** An amendment to the City's Water System Plan is required per WAC 246-290-110 because the DAF project is not included in the City's existing Water System Plan. The reason this project is not included in the current Water System Plan is that the Water System Plan was completed prior to the filter-clogging algae bloom of 2009. The Water System Plan Amendment could be completed using information already developed and included in this report.

- **Project Report:** A Project Report is required for review and approval by DOH per WAC 246-290-110. It would detail key elements of a preliminary design, including: design

criteria, alternatives analysis, estimated costs, proposed methods for startup, testing, operations, and other relevant project planning information. Much of the information presented in this report could be used to supplement the required elements of a Project Report.

- **Construction Documents:** Construction documents that detail the design of a new treatment process or facility are required to be reviewed and approved by DOH per WAC 246-290-120. These are the documents that would be used to bid the construction contract a new DAF treatment process. Construction documents would be completed as part of the design phase of the project, which would follow completion of the Project Report and its review and approval by DOH.

- **Construction Completion Report:** A Construction Completion Report (DOH Form 331-121) must be completed and submitted to DOH for approval per WAC 246-290-120(5) prior to sending DAF-treated water to customers. The Construction Completion Report certifies the project was construction in conformance with the previously-DOH-approved Construction Documents.

- **Operations Program:** An amendment to the City's existing Operations Program for the WTP must be developed per WAC 246-290-645(5). It is required to be submitted for DOH review and approval as an addendum to the Water System Plan. Addenda to the Water System Plan are required in conformance with WAC 246-290-100. The Operations Program would be developed, reviewed, and approved by DOH during construction, prior to start-up and testing of the new DAF treatment process.

8.2. Example Project Schedule

DAF implementation requires obtaining the DOH approvals presented above. Those approvals are typically sought and obtained sequentially using work products developed at various key stages of project completion. A possible project schedule that includes key activities and milestones that define the critical path and overall duration is presented in Figure 8-1.

For the purpose of this presentation, the project schedule presented in Figure 8-1 assumes the City begins implementation at the beginning of 2013 – given that the decision to pursue implementation could occur sometime over the course of 2012. The primary purpose of the example project schedule, in addition to identifying key, critical-path activities, is to quantify the approximate duration to placing a new DAF system into service. The project schedule presented in Figure 8-1 would have the same activities and same activity durations regardless of when it might be initiated.

The activity durations presented in Figure 8-1 are approximate and based on implementation at a steady but not overly-aggressive pace and mostly in sequential order of key, critical-path activities (minimal parallel completion of such future activities). Activity durations could potentially be shortened and parallel completion of some activities could be pursued if implementation became necessary on a shorter timescale.

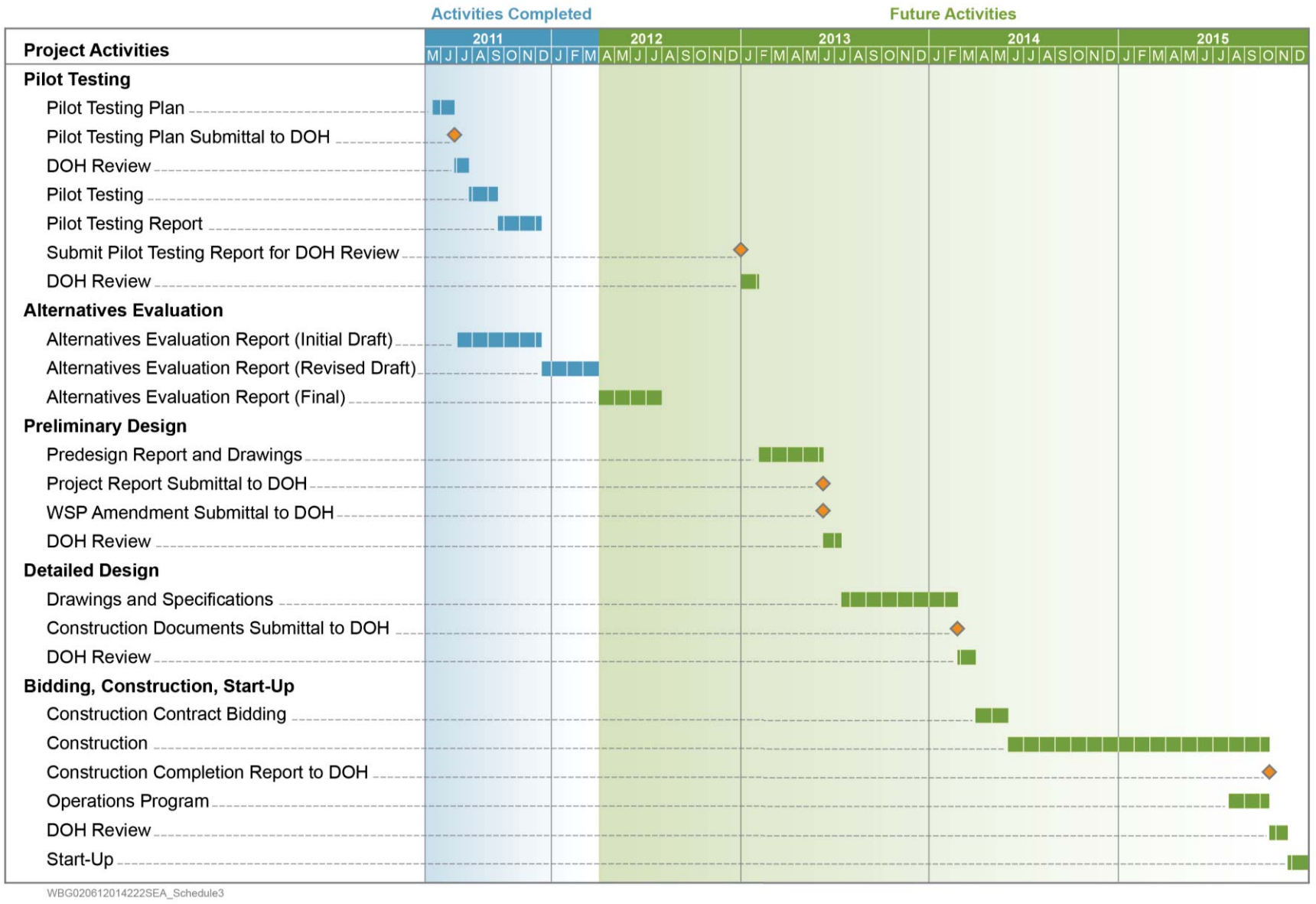


FIGURE 8-1
Example DAF Project Schedule

8.3. Cost-Reduction Options

The project costs presented in Section 5 for DAF and the other pretreatment alternatives were based on a capacity of 30 mgd, as explained in Section 3.1. Establishing a set capacity facilitated an equitable comparison of pretreatment alternatives based on cost. However, as presented in Section 2.5, it appears that 30 mgd of DAF capacity may not be necessary now or in the near-term future to meet summertime peak water demand. Also, a new DAF facility is not necessary for regulatory compliance with treatment and water quality standards. Its purpose is to enable the existing filters to perform more effectively to meet peak customer water demand.

As a result, given that falling short with respect to regulatory compliance is not a primary concern, it appears possible and prudent to examine options and rationale for reducing project cost by reducing the initial capacity and associated cost of the facility. The cost-reduction options relate to reducing the number of treatment trains, optimizing the initial DAF facility capacity in alignment with current and near-term future customer water demand, and evaluating the impact of DAF hydraulic loading rates on facility cost.

When a treatment process is necessary to ensure regulatory compliance, flexibility, reliability, and redundancy are of paramount importance. In this case, however, the DAF treatment system would not be needed for regulatory compliance, but instead to aid meeting customer water demand. Consequently, balancing initial capital cost savings with reduced flexibility, reliability, and redundancy warrants consideration.

8.3.1. Two vs. Three DAF Treatment Trains

Pretreatment processes are not subject to the same rigorous regulatory standard applied to filters, which requires facility capacity be based on one unit being out of service. This requirement applies to filters in recognition that they are regularly out of service for backwashing. Current design of clarification processes like DAF enable continuous operation – even as solids are being removed from the flowstream. Therefore, it is possible and somewhat common, depending on the goals and objectives of the specific installation, to have only two parallel clarification treatment trains. Doing so provides a reasonable level of system redundancy in the event that one train is not operational.

In such cases, one of the two clarification treatment trains may be operated at a higher hydraulic loading rate than the design criterion loading rate – resulting in an expected reduction in treatment performance. Reduced clarification performance over a short period of time may be acceptable given that the intent of pretreatment is to improve the quality of the water entering the filtration stage.

The initial capital cost estimate for DAF presented in Table 5-1 is based on three parallel DAF treatment trains with a capacity of 10 mgd each based on a hydraulic loading rate of 16 gpm/sf. This three-train cost-estimate-development approach was based on a more-robust approach with respect to reliability, operational flexibility, and redundancy. Also note that hydraulic loading rate is a key planning, cost-development, and design criterion for pretreatment processes. The results of the DAF pilot testing support the use of 16 gpm/sf as a relatively high loading rate, which enables keeping the capital cost of the DAF facility as

low as practical and the facility footprint as small as possible. More discussion of the DAF pilot testing and the use of hydraulic loading rate is presented Section 6.3.4, Appendix C, and below in Section 8.3.3.

All treatment processes and systems, including pretreatment, are designed with some level of redundancy to ensure reliability and operational flexibility. This is accomplished with parallel treatment trains and process that have a combined capacity equal to that necessary to meet the anticipated maximum day customer demand. Three parallel trains for pretreatment is a common approach, as stated above, because it offers a high degree of operational flexibility, reliability, and redundancy.

For example, with three parallel 10-mgd trains and one out of service, there would still be 20 mgd of capacity to help meet maximum day water demand. Based on recent historical data, 20 mgd appears to be greater than the maximum day water demand for the City of Bellingham. Conversely, two parallel 15-mgd trains would have the same overall capacity as three parallel 10-mgd trains, but would only leave 15 mgd of available capacity if one of the two 15-mgd trains were out of service. A single 15-mgd DAF treatment train may not be adequate to enable meeting customer water needs during maximum day water demand.

As a result, it is clear that three parallel 10-mgd trains provide somewhat greater reliability, operational flexibility, and redundancy than two parallel 15-mgd trains. However, three treatment trains are not required to meet treatment goals or standards, and are not absolutely necessary. Additionally, if the needed capacity of the overall system to meet anticipated customer water demand is substantially less than 30 mgd, the initial need for three parallel treatment trains may be less important. Because of the additional equipment and mechanized systems associated with an additional treatment train, three trains cost more than two trains for facilities with the same combined capacity.

8.3.2. Optimizing Initial Installed Capacity

While three 10-mgd DAF treatment trains provide greater reliability, operational flexibility, and redundancy than two 15-mgd DAF treatment trains, this advantage may not be put to beneficial use if 30 mgd of treatment capacity is not necessary. As presented in Section 2.5, the City's recent historical water demand, in particular the maximum day water demand, has declined in recent years. Given this fact, it would be more cost effective and technically sound to install a new DAF facility with an initial capacity that better reflects anticipated water demand. The new DAF facility would also need to have maximum flexibility to be expanded in the future when additional DAF treatment capacity becomes necessary.

As described above, and as presented in Section 2.5, it is not necessary to initially install 30 mgd of DAF treatment capacity to meet maximum day customer water demand. Installing an initial capacity of 20 mgd would provide adequate DAF treatment capacity to meet current and expected near-term future maximum day water demand. A DAF facility with an initial 20-mgd capacity would be comprised of two parallel 10-mgd treatment trains based on a hydraulic loading rate of 16 gpm/sf. A parallel, third 10-mgd DAF treatment train could be added in the future, as necessary, to meet peak summertime demand. The timing for the need for this third train is uncertain, but if demand trends continue, it may not be necessary for well beyond 20 to 30 years from now.

It would be necessary to operate both of the parallel 10-mgd treatment trains to meet the City's current and near-term future anticipated maximum day water demand. Reliance on both trains to meet demand without having one out of service incorporates less redundancy and flexibility than a three-train system, but is a common and acceptable treatment approach. In the event that one treatment train is out of service, the other can be operated at a higher hydraulic loading rate than the 16 gpm/sf design criteria and a portion of the raw water flow can be bypassed around the DAF process directly to the filters, matching the existing in-line filtration mode. This approach, even though less than optimal, would greatly reduce algae concentration in the raw water flow stream and extend filter run times enough to help them meet peak summertime customer water demand.

8.3.3. DAF Hydraulic Loading Rate

As presented in Appendix C, DAF pilot performance was consistently excellent at hydraulic loading rates up to 16 gpm/sf. While 16 gpm/sf is a relatively high rate for DAF system operation, and as a design criterion, it is in keeping with industry trends toward maximizing hydraulic loading rate in an effort to optimize cost efficiency. All other project elements and aspects equal, facility cost decreases with increased loading rate criteria, which applies to any clarification process.

Also as presented in Appendix C, the DAF pilot system also performed well at 20 gpm/sf. This hydraulic loading rate is at the upper limit of where high performance would be expected. Because this hydraulic loading rate is at the upper end of the loading rates tested on the Lake Whatcom supply, and because there are no known DAF systems designed with capacities based on a loading rate so high, 16 gpm/sf was the loading rate used for estimated the capital costs presented in Section 5. This 16 gpm/sf loading rate would be a reasonable hydraulic loading rate upon which to base a cost effective DAF system design for the City's needs, should the City pursue implementation.

It should be noted that two parallel 10-mgd treatment trains based on a hydraulic loading rate of 16 gpm/sf have a capacity of 25 mgd based on a hydraulic loading rate of 20 gpm/sf. While the DAF pilot testing on Lake Whatcom water from the late summer period of 2011 showed impressive results at 20 gpm/sf, it would not be prudent to rely on consistent performance at this rate in the absence of additional pilot data to provide confirmation. However, because capacity beyond 20 mgd is not necessary at this time, there would be ample opportunity over the years to test a new DAF system at this higher rate under actual conditions to assess when there might actually be a need for a third DAF treatment train.

In keeping with how loading rate impacts DAF facility capacity and initial capital cost, an optional approach to initially implementing a two-train 20-mgd DAF facility would be to base the design on 20 gpm/sf instead of the more-conservative 16 gpm/sf. While this impact would not have much of a cost-reduction impact as reducing the number of treatment trains from three to two, it would save capital cost and it warrants consideration. Two more-aggressively designed, 10-mgd DAF treatment trains based on 20 gpm/sf – resulting in a 20-mgd DAF facility capacity – would have a capacity of 16 mgd if operated at 16 gpm/sf.

A new DAF facility based on this loading rate criteria could potentially be able to meet peak summertime customer demand during algae bloom conditions for many years to come –

especially considering that some raw water flow could be bypassed around the DAF process to meet the total supply need. Continued monitoring of summertime demand conditions through 2012 and into the coming years would aid selection of a hydraulic loading rate criterion along with balancing the potential for reduced algae-removal performance on overall WTP capacity.

8.3.4. Summary of Initial Capital Costs

Initial capital costs were developed and presented in Table 5-2 for a 30-mgd, three-train DAF facility based on 16 gpm/sf. Initial capital costs for the two cost-reducing options described above in Section 8.3.3 were developed to compare against the DAF cost presented in Table 5-2. These three estimated initial capital costs are presented in Table 8-1.

TABLE 8-1
Summary of Initial Capital Cost for DAF Implementation Options

	DAF Implementation Options		
Cost Elements	**3-Train 30 mgd @ 16 gpm/sf**	**2-Train 20 mgd @ 16 gpm/sf** (25 mgd @ 20 gpm/sf)	**2-Train 16 mgd @ 16 gpm/sf** (20 mgd @ 20 gpm/sf)
Construction Costs:			
Construction Cost Subtotal	$ 5,756,000	$ 4,310,000	$ 4,070,000
Subtot. with Contr. OH (10%)	$ 6,332,000	$ 4,741,000	$ 4,477,000
Subtot. with Contr. Profit (6%)	$ 6,711,000	$ 5,025,000	$ 4,746,000
Subtot. w/ Mob, Bonds, Ins (10%)	$ 7,383,000	$ 5,528,000	$ 5,221,000
Subtotal with Contingency (30%)	$ 9,597,000	$ 7,186,000	$ 6,787,000
Escalation to Yr 2014 (12.8%)	$ 10,826,000	$ 8,106,000	$ 7,656,000
Constr. w/ Sales Tax (8.7%)	**$ 11,768,000**	**$ 8,811,000**	**$ 8,322,000**
Non-Construction Costs:			
Pilot Testing	$130,000	$130,000	$130,000
Geotechnical	$ 70,000	$ 70,000	$ 70,000
Permitting	$ 200,000	$ 200,000	$ 200,000
Eng. & Constr. Man. (18%)	$ 2,118,000	$ 1,586,000	$ 1,498,000
Startup (2%)	$ 235,400	$ 176,000	$ 166,000
Total	**$ 14,521,000**	**$ 10,973,000**	**$ 10,386,000**

What is clear is that the greatest savings in initial capital cost is achieved by reducing the number of treatment trains from three to two. Additional cost-reduction is achieved by basing the capacity of the facility on a higher, less-conservative hydraulic loading rate.

9. Summary of Key Conclusions and Recommendations

This report summarizes the identification, description, and evaluation of treatment, intake, and lake management alternatives to mitigate the adverse impacts of the seasonal filter-clogging algae at the City's WTP. This work resulted in an alternative that was deemed best-suited to mitigate the filter-clogging algae. Key conclusions and recommendations are presented in the following subsections.

9.1. Conclusions

- DAF is the best available treatment technology for mitigating the filter-clogging algae at the City's WTP. DAF is also the best, most-reliable overall technical approach for mitigating the filter-clogging algae at the City's WTP.
- DAF is acknowledged in the municipal water treatment industry as the best, most effective, and most reliable available technology for removing algae.
- DAF pilot testing showed that DAF can effectively treat Lake Whatcom algae at a relatively high rate.
- DAF will help improve the City's water quality by reducing the disinfection byproduct known as total trihalomethanes (TTHMs), total organic carbon (TOC), and other algae byproducts such as algal toxins and taste and odor compounds. A reduction in TTHMS of 25 percent can be anticipated. While these individual water quality parameters do not currently present regulatory compliance problems for the City, more intense algae blooms should be expected to present greater associated challenges.
- An intake solution to the filter-clogging algae condition at the City's WTP is more than double the cost of DAF and comes with uncertainty with respect to the quality of water it would withdraw.
- Additional filters could be effective at increasing plant capacity during an algae bloom as long as the intensity of the bloom does not reduce filter run times at the WTP to approximately 2 hours or less. The intensity of future algae blooms is unknown.
- The City is fully committed to reducing phosphorous entering Lake Whatcom to preserve and improve lake water quality and to meet the TMDL for Lake Whatcom. However, because of the long time duration to achieve the TMDL goals, lake management is not a viable, stand-alone alternative for mitigating the adverse algae clogging conditions at the City's WTP in the near-term future. Lake management will, over the long-term future, be part of a combined solution to minimize algae impacts at the WTP.
- As presented in this report, DAF was determined to be the best alternative using a technical evaluation approach as well as a TBL+ evaluation approach – even when considering the "Do Nothing" alternative.

9.2. Recommendations

Algae blooms occur annually in Lake Whatcom during the late summer and early fall timeframe. To date, the blooms have only resulted in mandatory water restrictions once. However, these blooms are expected to get progressively more intense over time, despite the fact that conditions in 2010 and 2011 were more favorable to reduced algae bloom intensity. Such future blooms present a risk to the City with respect to meeting the supply needs of its customers.

As a result, the City should pursue the design and construction of a new DAF facility in a phased approach, as discussed in Section 8, DAF Implementation. The phased approach should be based on an initial two-train DAF facility with easy expansion for a future third train, which would likely not be needed for many years into the future. The phased implementation of DAF will minimize the initial capital cost of a DAF facility. The phased approach will also eliminate the potential for constructing more DAF capacity than is necessary to ensure a continuous, reliable, high-quality drinking water supply – even during challenging times when there are intense algae blooms in Lake Whatcom. Based on the pilot testing completed in the late summer of 2011, DAF can be expected to lead to the reduction of the City's TTHMs by 25 percent.

This phased DAF-implementation approach complements the City's on-going commitment to lake management, water quality improvement, and TMDL compliance via the Lake Whatcom Management Program. Over the long-term future, as phosphorous-reducing lake management measures demonstrate success at improving water quality and reducing algae blooms, the need for further expansion of the initial phase of DAF implementation could be avoided entirely.

Appendix A
Lake Whatcom CE-QUAL-W2 Modeling Of Possible Water Intake Locations for the City of Bellingham

Lake Whatcom CE-QUAL-W2 Modeling Of Possible Water Intake Locations for the City of Bellingham

Prepared for:

City of Bellingham

Prepared by:

Chris Berger and Scott Wells

November 2011

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Executive Summary

The City of Bellingham's existing water supply system has experienced problems with water containing high algae concentrations flowing into the intake located in Lake Whatcom. An existing CE-QUAL-W2 water quality model of Lake Whatcom (Berger and Wells, 2005; Berger and Wells, 2007) was used to help identify potential new locations for the intake within the lake where concentrations of algae and particulate organic matter might be reduced. The CE-QUAL-W2 model was originally developed as part of a Total Maximum Daily Load Study performed by the Washington State Department of Ecology (Pickett and Hood, 2008). The model simulates a wide range of water quality and hydrodynamic parameters including blue-green algae, total algae, dissolved oxygen, nutrients, organic matter, temperature, water level and water velocity.

A total of 31 scenarios were simulated with the intake located at different locations within Lake Whatcom and at varying depths. Also simulated were 3 different land use conditions including a base case (2002/2003), existing (2011), and full build out of the watershed. The Lake Whatcom HSPF watershed model (The Cadmus Group, Inc. and CDM, 2007a and 2007b) was used to develop tributary inflows for the land use conditions. Because of the long residence time of Lake Whatcom (5-10 years), the scenarios were simulated over an extended period of time (11 years for the existing and base case land uses, 6 years for the full build out land use) so that model predictions were dependent on the land use conditions rather than the initial conditions of the model.

Model predictions indicated that order of magnitude reductions of algae concentrations in the intake only occurred when the intake was moved to deep locations (>30 meters) within basin #3. Algae grew closer to the water surface where more light for photosynthesis was available, so deeper intake locations had much lower algae concentrations. When the intake was placed at locations within the much shallower basin #2, the reduction of algae concentrations were smaller. At the deepest point in basin #2 (approximately 20 meters deep), blue-green algae concentrations in the intake for the July-October period were 39% less than at the existing location. At this location water with low dissolved oxygen concentrations water was withdrawn by the intake. With the intake placed at a relatively shallow depth (10 meters) at various locations in basin #3, the model predicted only a 20%-30% reduction in algae concentrations relative to the existing location.

Figure 1 shows maximum algae, maximum particulate organic matter, and minimum dissolved oxygen concentrations with the intake located near the bottom at various locations moving southeast from basin #2 into basin #3. At the north end of basin #3 (model segment 29), peak blue-green algae were reduced 80% and peak total algae concentrations were 90% less relative to concentrations at the existing intake locations in basin #2. Particulate organic matter (POM) concentrations were only slightly less than those in basin #2. POM consists mostly of dead algae cells and settles out of the water column toward the bottom.

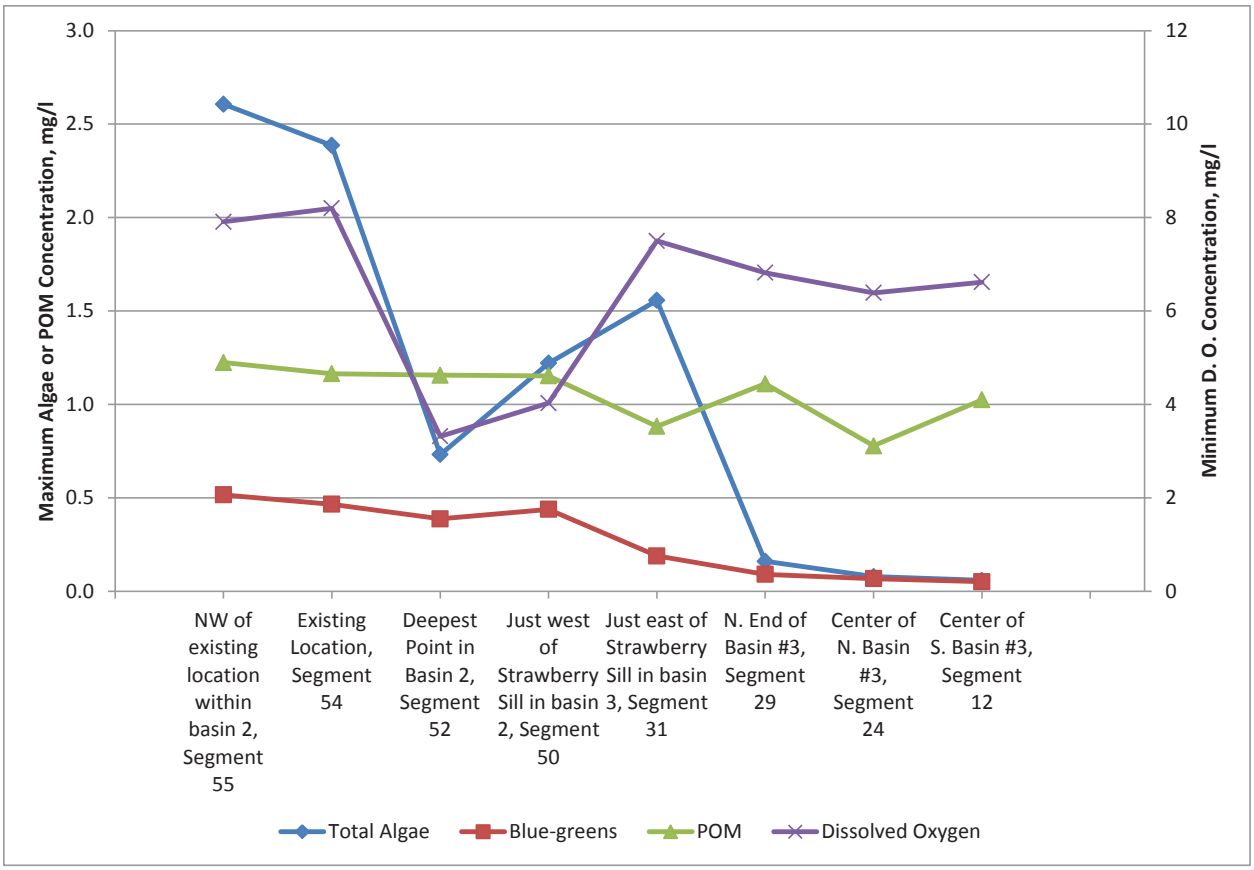

Figure 1. Maximum total algae, blue-green algae and particulate organic matter (POM) concentrations with intake located near bottom and at different locations within lake, moving along the lake axis from north end of basin 2 to the south. Minimum dissolved oxygen concentrations were also shown. The watershed loading for these scenarios was for existing conditions.

Introduction

The purpose of this modeling effort is to identify one or more locations in Lake Whatcom (Figure 2) that are predicted to be favorable for a new water supply intake for the City of Bellingham with respect to algae and particulate organic matter (POM). Because of increased algae growth in Lake Whatcom, the water supply intake can be susceptible to inflows of algae and POM. The intake can convey algae and POM into the treatment plant as a result of their deposition in the water column. This can have a deleterious effect on the water treatment plant process.

The model study was made to compare intake amounts of algae and POM based on estimated current water quality conditions and projected future water quality conditions. Intake locations that were deemed favorable with respect to algae and POM were those that were modeled to have less algae and POM than the City's existing intake location. As such, favorable intake locations were favorable only with respect to other locations within Lake Whatcom, which is the primary objective of this work – identifying "favorable" locations within Lake Whatcom for a new intake. The results of this modeling are based on a calibrated hydrodynamic and water quality model of Lake Whatcom (Berger and Wells, 2005; Berger and Wells, 2007). This modeling effort then is based on field data from 2002 and 2003 and is considered an estimate of "favorable" locations based on this prior work.

Background

Lake Whatcom is a large natural lake which was first listed on the 1998 Washington State 303(d) list of water bodies that do not meet the criterion for dissolved oxygen. Located next Bellingham, it is approximately 10 miles long, has a surface area of approximately 5000 acres, and a maximum depth of over 100 meters. Residence time is approximately 5-10 years. Lake Whatcom is within a relatively small watershed, and the lake's surface area is large in comparison to the size of its watershed. Eutrophication processes in the lake have been facilitated by the availability of nutrients, leading to concerns about land development within the watershed.

A water quality and hydrodynamic model of Lake Whatcom, Washington was developed as part of a Total Maximum Daily Load Study (Berger and Wells, 2005) using the model CE-QUAL-W2 Version 3.2. In further work on the TMDL, the model was upgraded to Version 3.5 and recalibrated based on updated information (Berger and Wells, 2007; Pickett and Hood, 2008).

The Corps of Engineer's model CE-QUAL-W2 is two-dimensional (longitudinal and vertical, x-z) consisting of directly coupled hydrodynamic and water quality transport models. This model has been under development for many years and is a public-domain code maintained by the Corps of Engineers, Waterways Experiments Station (WES), located in Vicksburg, Mississippi. Version 3.5 has undergone rigorous testing and has been successfully applied to many river basin systems (Cole and Wells, 2005). Further information about CE-QUAL-W2 Version 3 is shown at http://www.ce.pdx.edu/w2. The current release version of the model is Version 3.7.

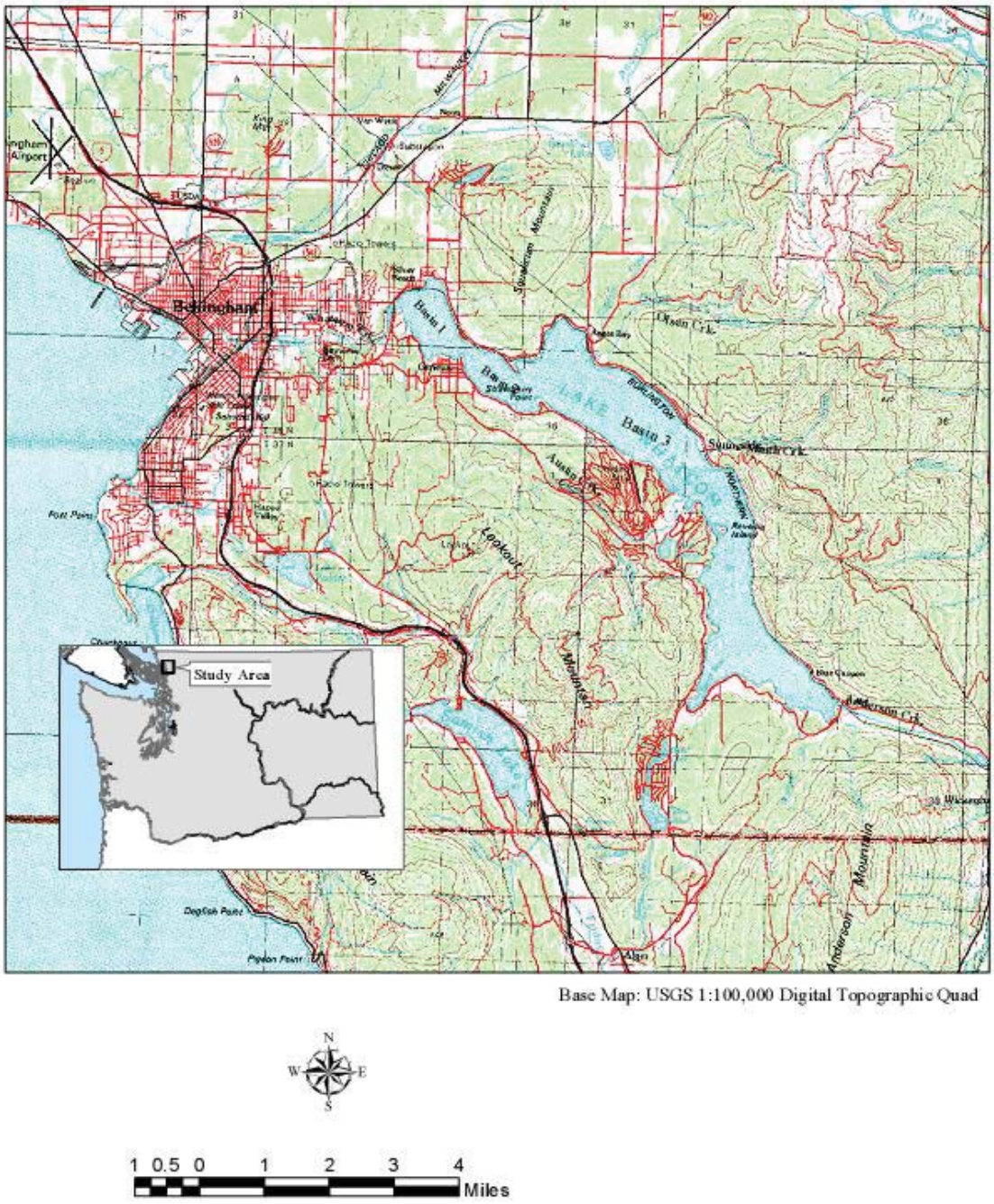

Figure 2. Lake Whatcom and vicinity (Pitz, 2005).

Primary physical processes simulated were surface heat transfer, short-wave and long-wave radiation and penetration, convective mixing, wind and flow induced mixing, inflow density stratification as impacted by temperature and dissolved and suspended solids. Major chemical constituents and biological processes simulated include: atmospheric exchange on DO, photosynthesis, respiration, organic matter decomposition, nitrification, and chemical oxidation of reduced substances; uptake, excretion, and regeneration of phosphorus and nitrogen and nitrification-denitrification under aerobic

and anaerobic conditions; carbon cycling and alkalinity-pH-CO2 interactions; trophic relationships for 3 phytoplankton species; and the accumulation and decomposition of detritus and organic sediment. For this application phosphorus and nitrogen organic matter compartments were added specially to represent the phosphorus (P) and nitrogen (N) mass contained in dissolved and particulate organic matter and the sediments. Thus the stoichiometry of the organic matter is variable and the user is able to set dynamic values of N and P in all tributaries associated with organic matter and to track these quantities within the domain of the CE-QUAL-W2. There were also sufficient data to model 3 phytoplankton species: diatoms, greens, and blue-greens.

The pie chart in Figure 3 shows the relative magnitude of phosphorus sources in Lake Whatcom. Inflows from tributaries and groundwater inflows account for approximately three-quarters of the phosphorus loading. Organic sediment contributions to phosphorus were simulated using two methods. The first method uses a constant, or zero order, release and demand and was labeled “anoxic sediment release” in Figure 3. The 0 order process uses a specified sediment oxygen demand and anoxic release rates for phosphorus, ammonium and inorganic carbon that were temperature dependent. Nutrient releases do not occur when dissolved oxygen concentrations were above a minimum value (0.1 mg/l for the Lake Whatcom model). Anoxic sediment release accounts for only a small fraction of the total phosphorus load to the lake because the actual volume of water containing less than 0.1 mg/l dissolved oxygen was very small relative to the total volume of the lake.

The second method uses a sediment compartment to accumulate organic sediments and allow their decay. The organic sediments consist primarily of particulate organic matter (detritus) settled to the bottom, and their decay accounts for approximately one quarter of the phosphorus load (labeled “oxic sediment release” in pie chart). Nutrient releases and oxygen demand were thus dependent upon sediment accumulation – a 1st ¬order process. However, there was no release of phosphorus or other diagenesis products when overlying water was anoxic because this sediment compartment only simulates the oxic (oxygenated) decay of organic matter.

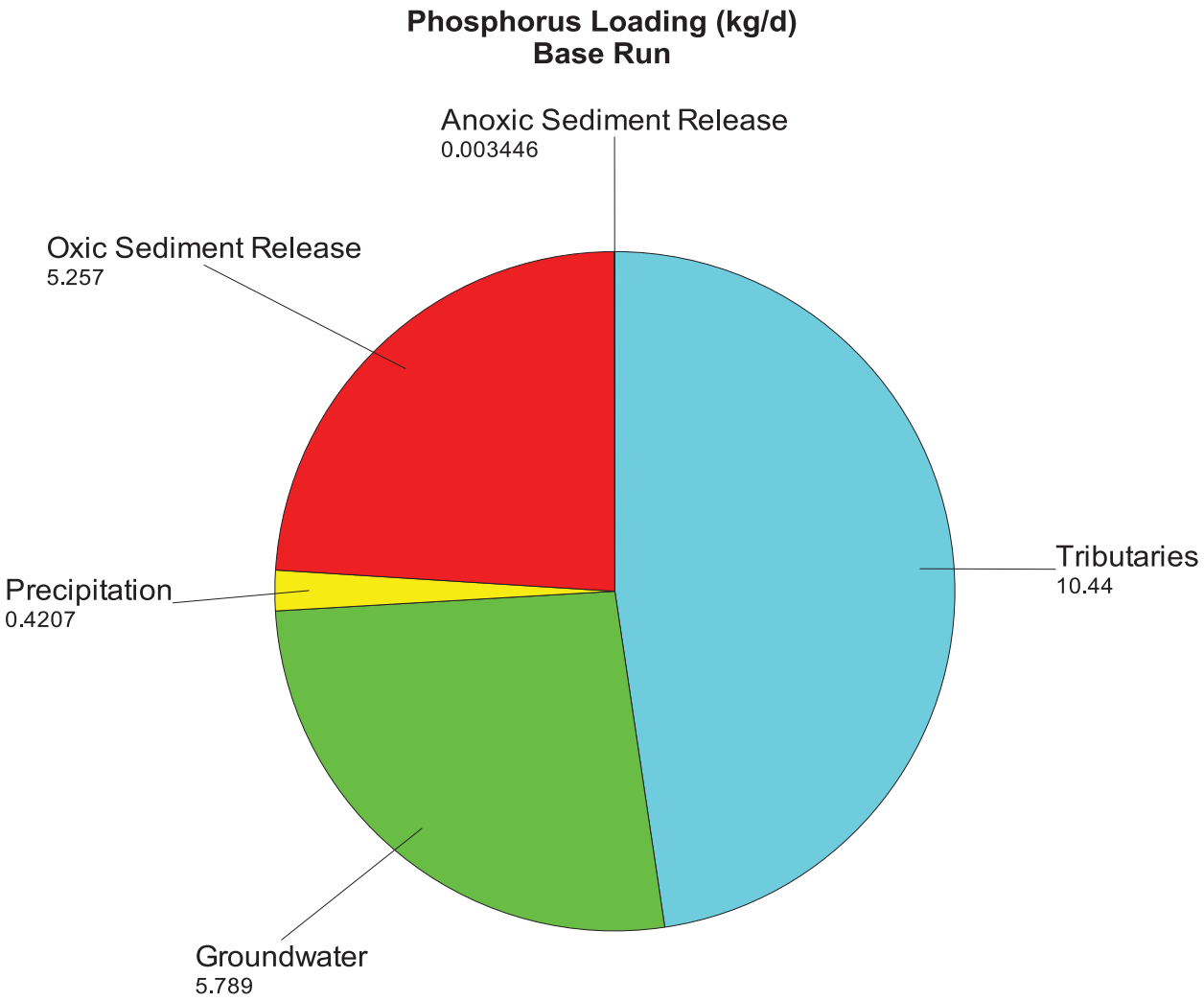

Figure 3. Pie chart showing sources of phosphorus loads to Lake Whatcom.

The Lake Whatcom bathymetry and basins are shown in Figure 4. Basin 3, much larger and deeper than the other two basins, contains 96% of the lake's volume. A plan view of the CE-QUAL-W2 grid layout is shown in Figure 5. The model was divided into five branches and two water bodies. Branches 1 through 3 simulated basin 3 and branches 4 and 5 represented basins 1 and 2. The length of the model segments ranged from 16 meters to 821 meters. Model layers have a thickness of 1 or 3 meters. Three meter layer thicknesses were used only in the deeper sections of Basin 3. Model vertical layers and longitudinal segments are shown in Figure 6.

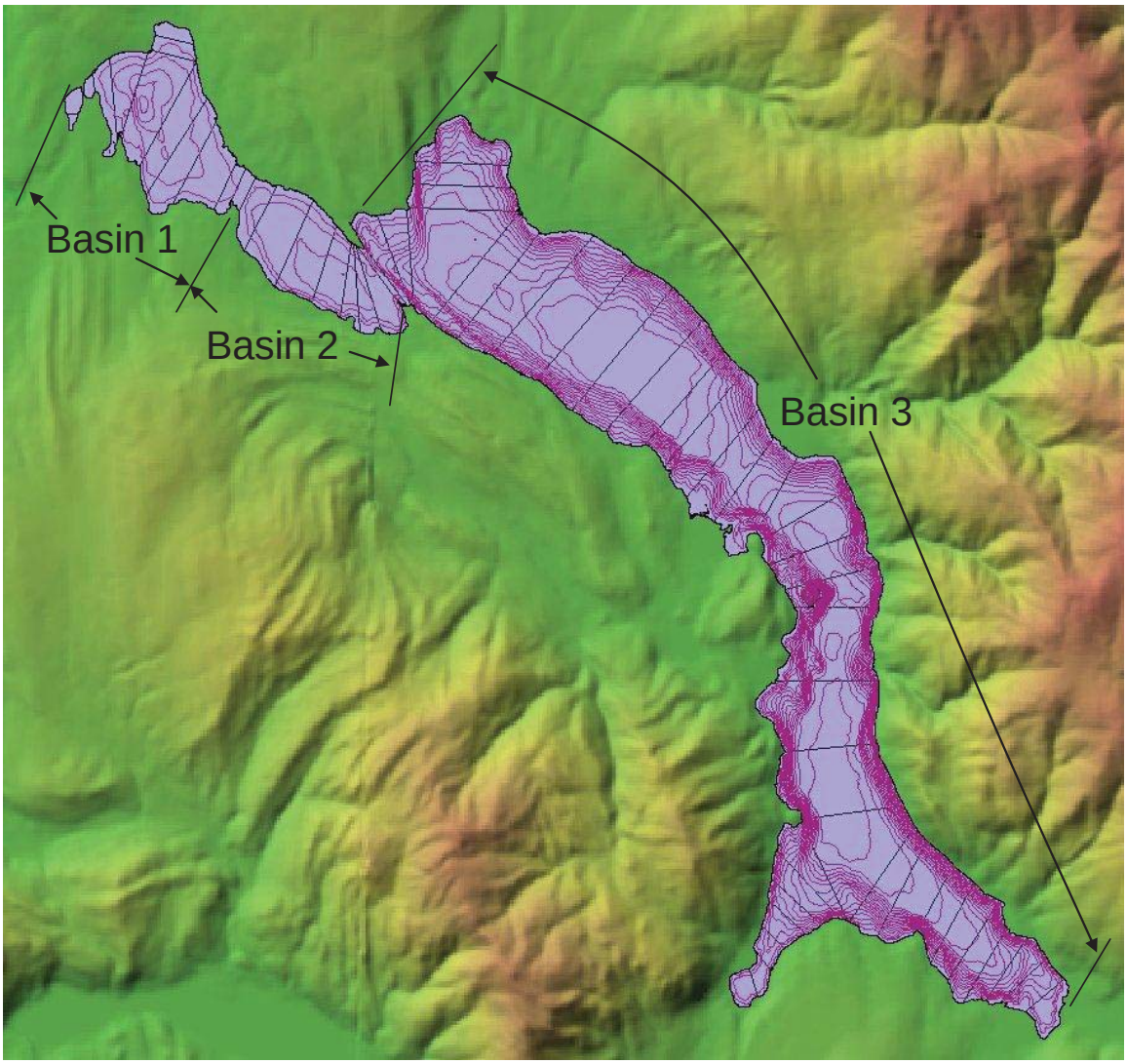

Figure 4. Lake bathymetry and topography around Lake Whatcom. The lake's basins were also shown.

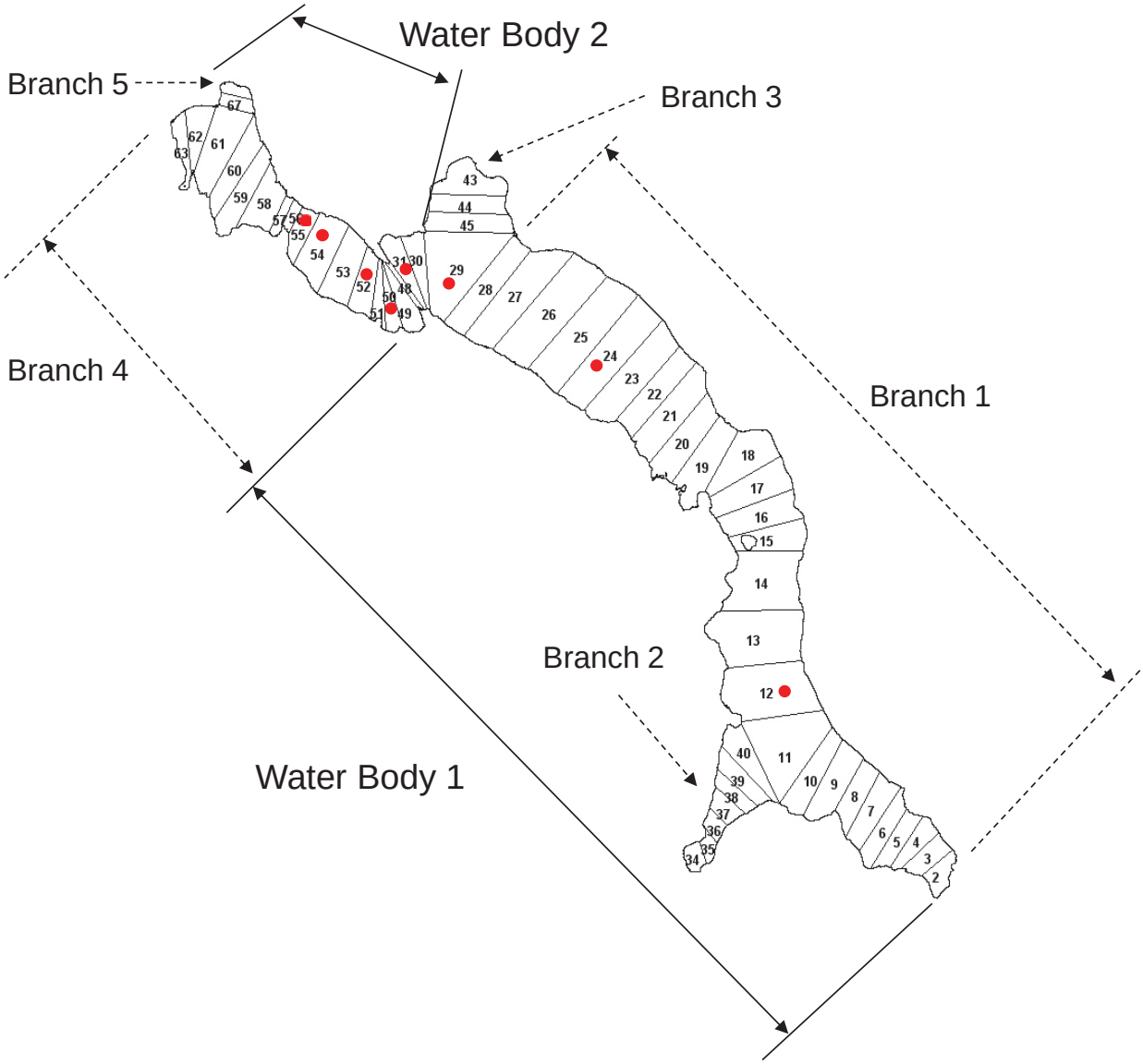

Figure 5. Plan view of the Lake Whatcom model grid showing model segments, branches, and water bodies. Segments where potential intake locations were investigated were marked with the "●" symbol.

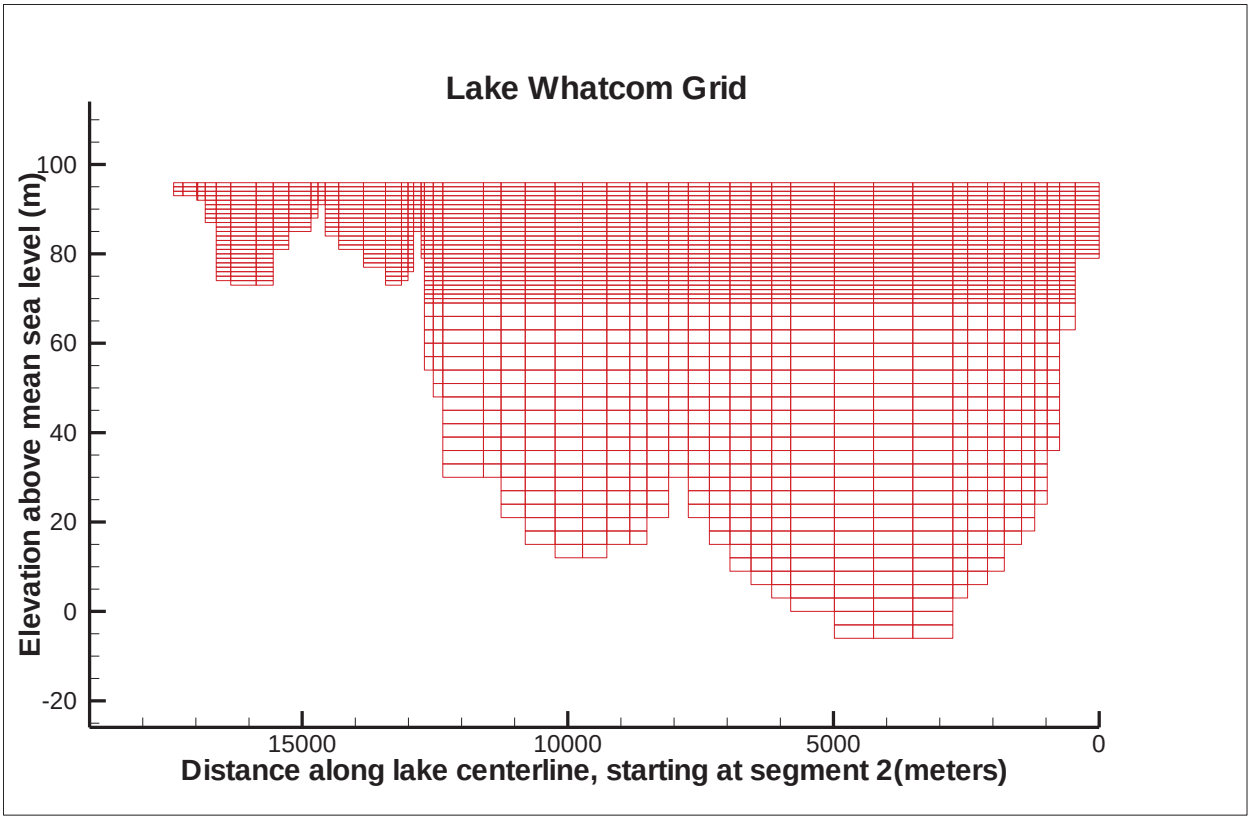

Figure 6. Longitudinal segments and vertical layer elevations of Lake Whatcom model. Only layers below the full pool elevation are shown.

Scenarios

The location of the City of Bellingham's water intake was moved to various locations and depths within Lake Whatcom. Model segments where the intake was placed are identified in Figure 5. In addition, scenarios were simulated with different land use and tributary nutrient loadings. These included a base case (2002/2003), existing land use (2011), and full build-out land use simulations with tributary loads generated by the Lake Whatcom HSPF watershed model (The Cadmus Group, Inc. and CDM, 2007a and 2007b). Base case inputs were the same as those used for the calibration years 2002 and 2003 of the CE-QUAL-W2 model developed for the Lake Whatcom TMDL Study. The water surface elevations for the simulations were generally between 95 m to 96 m MSL. Scenarios were simulated for up to 11 years so that the model results would not be overly influenced by the initial conditions. In doing so, for each of the three different land use conditions (base case, existing, and future build-out), the model predicts future algae and other water quality parameters. This was accomplished by using a looping tool which permitted multiple runs with the same boundary conditions where the initial condition of a run was equal to the final conditions of the previous simulation. This ensured that model predictions were more dependent upon tributary inflows rather than the initial conditions. The base case and existing land use scenarios were simulated then for a total of 11 years (or 10 loops, with the first loop being 2 years and all others being 1 year). However, the full build-out scenario was only simulated through 5 loops (6 years) because of nitrogen limitation (insufficient nitrogen available for increases in algae growth) in the

input files for this scenario. Nitrogen limitation is rare in freshwater systems and it is believed that the nitrogen limitation for the build-out condition results from a deficiency in the input data. A total of 31 scenarios were simulated, and these were listed in Table 1.

To search for favorable intake locations in Lake Whatcom, the existing condition scenario was simulated at multiple points and depths within the lake. If a location seemed promising or where simulating the base case or full build out might be informative, these scenarios were also simulated. As shown in the next section, algae and POM concentrations of the full build out scenario were generally within 10% of that of the existing conditions scenario, so it was unlikely that promising intake locations were overlooked by only simulating the existing conditions scenario.

Table 1. Scenario simulations.

Scenario #	Intake Segment #	Intake Elev. (m)	Approx. Depth (m)	Watershed Loading Scenario*	Comments
1	54	85	10	Base	Existing Location of Intake
2	54	85	10	Existing	Existing Location of Intake
3	54	85	10	FBO	Existing Location of Intake
4	52	85	10	Existing	SE of existing location within basin 2
5	55	85	10	Existing	NW of existing location within basin 2
6	50	78	17	Existing	Just west of Strawberry Sill in basin 2
7	31	60	35	Existing	Just east of Strawberry Sill in basin 3, close to bottom
8	31	60	35	FBO	Just east of Strawberry Sill in basin 3, close to bottom
9	31	60	35	Base	Just east of Strawberry Sill in basin 3, close to bottom
10	31	85	10	Existing	Just east of Strawberry Sill in basin 3, 10 m depth
11	24	85	10	Existing	Center of N. Basin #3, 10 m depth
12	24	50	45	Existing	Center of N. Basin #3, 45 m depth
13	24	50	45	FBO	Center of N. Basin #3, 45 m depth
14	24	18	77	Existing	Center of N. Basin #3, 77 m depth
15	24	18	77	FBO	Center of N. Basin #3, 77 m depth
16	24	18	77	Base	Center of N. Basin #3, 77 m depth
17	12	85	10	Existing	Center of S. Basin #3, 10 m depth
18	12	50	45	Existing	Center of S. Basin #3, 45 m depth
19	12	50	45	FBO	Center of S. Basin #3, 45 m depth
20	12	0	95	Existing	Center of S. Basin #3, 95 m depth
21	12	0	95	FBO	Center of S. Basin #3, 95 m depth
22	29	35	60	Existing	North end of Basin #3, 60 m depth
23	29	35	60	FBO	North end of Basin #3, 60 m depth
24	29	35	60	Base	North end of Basin #3, 60 m depth
25	29	85	10	Existing	North end of Basin #3, 10 m depth
26	29	75	20	Existing	North end of Basin #3, 20 m depth
27	29	65	30	Existing	North end of Basin #3, 30 m depth
28	29	55	40	Existing	North end of Basin #3, 40 m depth

Scenario #	Intake Segment #	Intake Elev. (m)	Approx. Depth (m)	Watershed Loading Scenario*	Comments
29	29	45	50	Existing	North end of Basin #3, 50 m depth
30	52	75	20	Existing	Deepest location in Basin #2
31	52	75	20	FBO	Deepest location in Basin #2

*Base=Base Conditions, FBO=Full Build Out Land Use, Existing Land Uses

Results

Intake concentrations of algae and particulate organic matter (POM) concentrations were compared for the different scenarios. Average annual concentrations for the intake were listed in Table 2. Average concentrations for the July-October period were shown in Table 3. Table 4 shows the results in Table 3 but ranked based on total algae and POM from least to the greatest, and Table 5 shows the results of Table 3 sorted according to model segment location. Table 6 lists the maximum intake concentrations. Generally, the deeper the intake, the lower the algae concentrations withdrawn by the intake. Figure 7 shows predicted total algae concentrations with the intake located at various depths in segment 29 (north end of basin #3). Watershed loadings for these scenarios corresponded to existing land use. Concentrations decrease by an order of magnitude between an intake depth of 10 m to near the bottom at a depth of 60 m. Algae require light to grow and concentrations were greater near the surface where more light was available. On the other hand particulate organic matter concentrations in the intake remained approximately the same with increasing depth (Figure 8). Particulate organic matter in Lake Whatcom consisted primarily of dead algae during the summer months, which would settle out of the photic zone to the bottom.

When the intake was placed at different locations within basin #2 (scenarios #1 through #6) algae concentrations remained roughly the same at most locations. However at the deepest point in basin #2 (scenarios 30 and 31), the average July-October total algae concentration decreased 47% relative to the existing location for the full build out scenario. Concentrations of blue green algae, which are thought to be the primary cause of the clogging in the intake, decreased 39%. Particulate organic matter increased by 27% relative to the existing location and conditions. Maximum blue-green concentration decreased by 10% and maximum total algae concentration decreased by 67%. Minimum dissolved oxygen concentration, which was 8.11 mg/l at the existing location for the full build out scenario (scenario 3), dropped to 3.05 mg/l with the intake at the deepest point of basin. With the intake at the existing location, the model predicted zero dissolved oxygen concentrations in the hypolimnion of basin #2 during the summer. This prediction was consistent with measured dissolved oxygen data (Matthews et al., 2011). When the intake was moved to the deepest point in basin #2, a vertical current was created where water drawn into the intake was replaced by oxygenated water flowing from above. This vertical current prevented the minimum dissolved oxygen concentrations from dropping to zero for scenarios 30 and 31. Order of magnitude decreases in concentrations in the intake only occurred when the intake was moved across the sill separating basin #2 and #3 (Strawberry Sill) and into deeper water.

With the intake at a depth of 10 m total algae concentrations during the July-October dropped approximately 20% when moving the intake along the axis of the lake from basin #2 into basin #3 (Figure 9). Particulate organic matter concentrations decreased 35% and blue green algae concentrations dropped 28%. Peak concentrations showed a similar pattern (Figure 10), with total algae concentration in the intake being reduced 24% with the intake (at a 10 m depth) being moved from the existing location in basin #2 to the south of basin #3.

Much larger decreases of algae in the intake occurred when it was placed near the bottom in the deep areas of basin #3. Figure 11 shows total algae, blue-green algae, and POM concentrations in the intake as it was shifted from the relatively shallow basin #2 southeast along the lake axis through basin #3. In segment 31, just east of the sill separating basin #2 and #3, total algae and blue-green algae concentrations for the July-October period had decreased 95% relative to basin #2 concentrations.. Peak concentrations of total algae, blue-green algae, and POM with the intake near bottom were plotted in Figure 12. At the bottom of segment 29, near the north end basin #3, maximum total algae concentrations were 90% less than concentrations in basin #2 and peak blue-green concentrations were 80% less.

The use of base case, existing land use, full build-out land use watershed loadings did not result in large differences in intake algae and other water quality concentrations. For instance, at the existing location of the outlet (scenarios 1-3) the peak blue-green concentration for base case was 0.454 mg/l for the base case, 0.466 mg/l for existing land use, and 0.473 mg/l for full build-out land use. At the bottom of segment 29 near the north end of basin #3, peak blue-green concentrations were 0.087 mg/l for the base case, 0.091 for existing land use, and 0.092 mg/l for full build out land use (scenarios 22-24).

Predictions of Algae Growth in Basin 3

Data collected by Western Washington University as part of annual Lake Whatcom water quality monitoring reveals a doubling of chlorophyll a in the shallow depths of Basin 3 between 2002 and 2010. The model was run in an attempt to demonstrate duplication of these results. To accomplish this, a base case simulation was run with the intake placed at a shallow depth of approximately 3 meters in segment 29. Model segment 29 was located in the north end of basin 3 (Figure 5). The model was run through 8 loops (9 years) so that water quality differences between year 9 and year 1 could be evaluated. This 8 year period can then be used to simulate the difference between years 2010 and 2002. This approach assumes no change to land use and watershed conditions in the base case model input files. Table 7 and Table 8 show that algae concentrations approximately doubled over the 8 year period which is similar to actual chlorophyll observations documented in the annual Lake Whatcom reports. Because chlorophyll a is an indirect measure of algae concentration, their trends are typically similar.

Table 2. Average concentrations of algae, particulate organic matter (POM) and dissolved oxygen in intake for the entire year.

Scenario #	Intake Segment #	Approx. Depth (m)	Water-shed Loading Scenario*	Diatoms (mg/l)	Chloro-phyta (mg/l)	Blue-greens (mg/l)	Total Algae (mg/l)	POM (mg/l)	D. O. (mg/l)
1	54	10	Base	0.323	0.007	0.134	0.463	0.404	10.50
2	54	10	Existing	0.335	0.008	0.139	0.482	0.420	10.52
3	54	10	FBO	0.356	0.011	0.140	0.507	0.442	10.57
4	52	10	Existing	0.295	0.007	0.130	0.433	0.405	10.32
5	55	10	Existing	0.381	0.008	0.149	0.538	0.421	10.76
6	50	17	Existing	0.171	0.006	0.104	0.281	0.446	9.30
7	31	35	Existing	0.037	0.004	0.039	0.080	0.355	9.57
8	31	35	FBO	0.042	0.006	0.040	0.087	0.376	9.58
9	31	35	Base	0.035	0.004	0.038	0.076	0.344	9.61
10	31	10	Existing	0.298	0.007	0.131	0.436	0.346	10.53
11	24	10	Existing	0.255	0.007	0.108	0.370	0.323	10.52
12	24	45	Existing	0.008	0.004	0.028	0.040	0.357	9.21
13	24	45	FBO	0.010	0.005	0.028	0.043	0.375	9.21
14	24	77	Existing	0.003	0.004	0.023	0.029	0.338	8.56
15	24	77	FBO	0.003	0.005	0.023	0.032	0.354	8.54
16	24	77	Base	0.002	0.003	0.022	0.028	0.329	8.62
17	12	10	Existing	0.259	0.007	0.107	0.374	0.341	10.46
18	12	45	Existing	0.006	0.004	0.024	0.033	0.370	9.15
19	12	45	FBO	0.008	0.005	0.023	0.036	0.387	9.16
20	12	95	Existing	0.002	0.003	0.019	0.024	0.350	8.56
21	12	95	FBO	0.002	0.005	0.019	0.026	0.365	8.56
22	29	60	Existing	0.005	0.004	0.026	0.035	0.368	8.95
23	29	60	FBO	0.007	0.006	0.026	0.038	0.386	8.95
24	29	60	Base	0.005	0.004	0.025	0.033	0.357	9.01
25	29	10	Existing	0.286	0.007	0.125	0.418	0.333	10.51
26	29	20	Existing	0.128	0.005	0.064	0.197	0.359	10.06
27	29	30	Existing	0.049	0.004	0.044	0.097	0.358	9.66
28	29	40	Existing	0.019	0.004	0.033	0.056	0.359	9.38
29	29	50	Existing	0.007	0.004	0.028	0.040	0.361	9.12
30	52	20	Existing	0.148	0.005	0.099	0.252	0.451	9.03
31	52	20	FBO	0.157	0.007	0.103	0.268	0.474	9.01

*Base=Base Conditions, FBO=Full Build Out Land Use, Existing Land Uses

Table 3. Average concentrations of algae, particulate organic matter (POM) and dissolved oxygen in intake for the July-October period.

Scenario #	Intake Segment #	Approx. Depth (m)	Water-shed Loading Scenario*	Diatoms (mg/l)	Chloro-phyta (mg/l)	Blue-greens (mg/l)	Total Algae (mg/l)	POM (mg/l)	D. O. (mg/l)
1	54	10	Base	0.249	0.007	0.230	0.486	0.441	9.47
2	54	10	Existing	0.258	0.008	0.238	0.504	0.461	9.50
3	54	10	FBO	0.271	0.011	0.242	0.525	0.487	9.51
4	52	10	Existing	0.240	0.007	0.220	0.467	0.444	9.29
5	55	10	Existing	0.263	0.009	0.255	0.527	0.450	9.80
6	50	17	Existing	0.141	0.005	0.150	0.296	0.545	7.27
7	31	35	Existing	0.008	0.002	0.013	0.023	0.435	8.88
8	31	35	FBO	0.009	0.003	0.013	0.025	0.480	8.80
9	31	35	Base	0.008	0.002	0.013	0.023	0.435	8.88
10	31	10	Existing	0.217	0.007	0.229	0.453	0.354	10.07
11	24	10	Existing	0.218	0.007	0.183	0.408	0.302	10.44
12	24	45	Existing	0.003	0.003	0.013	0.019	0.460	8.72
13	24	45	FBO	0.004	0.004	0.013	0.021	0.487	8.70
14	24	77	Existing	0.001	0.003	0.012	0.016	0.455	7.86
15	24	77	FBO	0.001	0.005	0.012	0.018	0.485	7.80
16	24	77	Base	0.001	0.003	0.012	0.015	0.437	7.94
17	12	10	Existing	0.238	0.008	0.195	0.442	0.299	10.37
18	12	45	Existing	0.003	0.003	0.013	0.019	0.447	8.86
19	12	45	FBO	0.004	0.004	0.012	0.020	0.473	8.85
20	12	95	Existing	0.000	0.003	0.012	0.015	0.428	8.21
21	12	95	FBO	0.000	0.004	0.012	0.016	0.456	8.18
22	29	60	Existing	0.002	0.003	0.012	0.018	0.489	8.37
23	29	60	FBO	0.002	0.005	0.012	0.019	0.519	8.33
24	29	60	Base	0.002	0.003	0.012	0.017	0.469	8.44
25	29	10	Existing	0.216	0.007	0.217	0.440	0.337	10.18
26	29	20	Existing	0.053	0.002	0.052	0.107	0.413	9.37
27	29	30	Existing	0.011	0.002	0.016	0.029	0.448	8.91
28	29	40	Existing	0.005	0.003	0.013	0.021	0.469	8.75
29	29	50	Existing	0.003	0.003	0.013	0.018	0.484	8.52
30	52	20	Existing	0.116	0.004	0.137	0.257	0.561	6.80
31	52	20	FBO	0.117	0.006	0.145	0.268	0.589	6.61

*Base=Base Conditions, FBO=Full Build Out Land Use, Existing Land Uses

Table 4. Reordered Table 3 (average concentrations of algae, particulate organic matter (POM) and dissolved oxygen in intake for the July-October period) ranked based on total algae and POM from least to greatest.

Scenario #	Intake Segment #	Approx. Depth (m)	Watershed Loading Scenario*	Diatoms (mg/l)	Chloro-phyta (mg/l)	Blue-greens (mg/l)	Total Algae (mg/l)	POM (mg/l)	D. O. (mg/l)	Total algae and POM, mg/l
20	12	95	Existing	0.000	0.003	0.012	0.015	0.428	8.21	0.443
16	24	77	Base	0.001	0.003	0.012	0.015	0.437	7.94	0.452
7	31	35	Existing	0.008	0.002	0.013	0.023	0.435	8.88	0.458
9	31	35	Base	0.008	0.002	0.013	0.023	0.435	8.88	0.458
18	12	45	Existing	0.003	0.003	0.013	0.019	0.447	8.86	0.466
14	24	77	Existing	0.001	0.003	0.012	0.016	0.455	7.86	0.471
21	12	95	FBO	0.000	0.004	0.012	0.016	0.456	8.18	0.472
27	29	30	Existing	0.011	0.002	0.016	0.029	0.448	8.91	0.477
12	24	45	Existing	0.003	0.003	0.013	0.019	0.460	8.72	0.479
24	29	60	Base	0.002	0.003	0.012	0.017	0.469	8.44	0.486
28	29	40	Existing	0.005	0.003	0.013	0.021	0.469	8.75	0.490
19	12	45	FBO	0.004	0.004	0.012	0.020	0.473	8.85	0.493
29	29	50	Existing	0.003	0.003	0.013	0.018	0.484	8.52	0.502
15	24	77	FBO	0.001	0.005	0.012	0.018	0.485	7.80	0.503
8	31	35	FBO	0.009	0.003	0.013	0.025	0.480	8.80	0.505
22	29	60	Existing	0.002	0.003	0.012	0.018	0.489	8.37	0.507
13	24	45	FBO	0.004	0.004	0.013	0.021	0.487	8.70	0.508
26	29	20	Existing	0.053	0.002	0.052	0.107	0.413	9.37	0.520
23	29	60	FBO	0.002	0.005	0.012	0.019	0.519	8.33	0.538
11	24	10	Existing	0.218	0.007	0.183	0.408	0.302	10.44	0.710
17	12	10	Existing	0.238	0.008	0.195	0.442	0.299	10.37	0.741
25	29	10	Existing	0.216	0.007	0.217	0.440	0.337	10.18	0.777
10	31	10	Existing	0.217	0.007	0.229	0.453	0.354	10.07	0.807
30	52	20	Existing	0.116	0.004	0.137	0.257	0.561	6.80	0.818
6	50	17	Existing	0.141	0.005	0.150	0.296	0.545	7.27	0.841
31	52	20	FBO	0.117	0.006	0.145	0.268	0.589	6.61	0.857
4	52	10	Existing	0.240	0.007	0.220	0.467	0.444	9.29	0.911
1	54	10	Base	0.249	0.007	0.230	0.486	0.441	9.47	0.927
2	54	10	Existing	0.258	0.008	0.238	0.504	0.461	9.50	0.965
5	55	10	Existing	0.263	0.009	0.255	0.527	0.450	9.80	0.977
3	54	10	FBO	0.271	0.011	0.242	0.525	0.487	9.51	1.012

*Base=Base Conditions, FBO=Full Build Out Land Use, Existing Land Uses

Table 5. Reordered Table 3 (average concentrations of algae, particulate organic matter (POM) and dissolved oxygen in intake for the July-October period) based on model segment number.

Scenario #	Intake Segment #	Approx. Depth (m)	Watershed Loading Scenario*	Diatoms (mg/l)	Chloro-phyta (mg/l)	Blue-greens (mg/l)	Total Algae (mg/l)	POM (mg/l)	D. O. (mg/l)	Total algae and POM, mg/l
17	12	10	Existing	0.238	0.008	0.195	0.442	0.299	10.37	0.741
18	12	45	Existing	0.003	0.003	0.013	0.019	0.447	8.86	0.466
19	12	45	FBO	0.004	0.004	0.012	0.020	0.473	8.85	0.493
20	12	95	Existing	0.000	0.003	0.012	0.015	0.428	8.21	0.443
21	12	95	FBO	0.000	0.004	0.012	0.016	0.456	8.18	0.472
11	24	10	Existing	0.218	0.007	0.183	0.408	0.302	10.44	0.710
12	24	45	Existing	0.003	0.003	0.013	0.019	0.460	8.72	0.479
13	24	45	FBO	0.004	0.004	0.013	0.021	0.487	8.70	0.508
16	24	77	Base	0.001	0.003	0.012	0.015	0.437	7.94	0.452
14	24	77	Existing	0.001	0.003	0.012	0.016	0.455	7.86	0.471
15	24	77	FBO	0.001	0.005	0.012	0.018	0.485	7.80	0.503
25	29	10	Existing	0.216	0.007	0.217	0.440	0.337	10.18	0.777
26	29	20	Existing	0.053	0.002	0.052	0.107	0.413	9.37	0.520
27	29	30	Existing	0.011	0.002	0.016	0.029	0.448	8.91	0.477
28	29	40	Existing	0.005	0.003	0.013	0.021	0.469	8.75	0.490
29	29	50	Existing	0.003	0.003	0.013	0.018	0.484	8.52	0.502
24	29	60	Base	0.002	0.003	0.012	0.017	0.469	8.44	0.486
22	29	60	Existing	0.002	0.003	0.012	0.018	0.489	8.37	0.507
23	29	60	FBO	0.002	0.005	0.012	0.019	0.519	8.33	0.538
10	31	10	Existing	0.217	0.007	0.229	0.453	0.354	10.07	0.807
7	31	35	Existing	0.008	0.002	0.013	0.023	0.435	8.88	0.458
9	31	35	Base	0.008	0.002	0.013	0.023	0.435	8.88	0.458
8	31	35	FBO	0.009	0.003	0.013	0.025	0.480	8.80	0.505
6	50	17	Existing	0.141	0.005	0.150	0.296	0.545	7.27	0.841
4	52	10	Existing	0.240	0.007	0.220	0.467	0.444	9.29	0.911
30	52	20	Existing	0.116	0.004	0.137	0.257	0.561	6.80	0.818
31	52	20	FBO	0.117	0.006	0.145	0.268	0.589	6.61	0.857
1	54	10	Base	0.249	0.007	0.230	0.486	0.441	9.47	0.927
2	54	10	Existing	0.258	0.008	0.238	0.504	0.461	9.50	0.965

*Base=Base Conditions, FBO=Full Build Out Land Use, Existing Land Uses

Table 6. Maximum concentrations of algae and particulate organic matter (POM) and the minimum concentration of dissolved oxygen in the intake.

Scenario #	Intake Segment #	Approx. Depth (m)	Watershed Loading Scenario*	Dia-tom Max. (mg/l)	Chloro-phyta Max. (mg/l)	Blue-green Max. (mg/l)	Total Algae Max. (mg/l)	POM Max. (mg/l)	D. O. Min. (mg/l)
1	54	10	Base	2.180	0.017	0.454	2.279	1.120	8.22
2	54	10	Existing	2.280	0.019	0.466	2.386	1.165	8.20
3	54	10	FBO	2.440	0.024	0.473	2.546	1.234	8.11
4	52	10	Existing	1.980	0.017	0.419	2.079	1.103	8.35
5	55	10	Existing	2.500	0.019	0.517	2.607	1.224	7.91
6	50	17	Existing	1.140	0.012	0.439	1.221	1.153	4.03
7	31	35	Existing	1.470	0.016	0.190	1.557	0.883	7.50
8	31	35	FBO	1.550	0.021	0.196	1.640	0.941	7.43
9	31	35	Base	1.400	0.014	0.183	1.483	0.847	7.61
10	31	10	Existing	2.190	0.020	0.455	2.298	1.052	9.22
11	24	10	Existing	1.780	0.020	0.372	1.883	0.999	9.17
12	24	45	Existing	0.099	0.006	0.104	0.138	0.816	7.40
13	24	45	FBO	0.107	0.009	0.106	0.150	0.839	7.34
14	24	77	Existing	0.021	0.005	0.068	0.079	0.779	6.39
15	24	77	FBO	0.028	0.007	0.067	0.081	0.794	6.27
16	24	77	Base	0.020	0.005	0.065	0.074	0.771	6.52
17	12	10	Existing	1.870	0.021	0.403	1.972	0.959	8.67
18	12	45	Existing	0.123	0.007	0.056	0.164	0.979	7.50
19	12	45	FBO	0.144	0.009	0.056	0.187	1.002	7.44
20	12	95	Existing	0.005	0.005	0.052	0.059	1.025	6.62
21	12	95	FBO	0.007	0.007	0.051	0.061	1.049	6.55
22	29	60	Existing	0.103	0.009	0.091	0.161	1.111	6.82
23	29	60	FBO	0.125	0.014	0.092	0.184	1.137	6.72
24	29	60	Base	0.089	0.007	0.087	0.143	1.097	6.93
25	29	10	Existing	2.070	0.020	0.430	2.178	0.977	9.30
26	29	20	Existing	1.870	0.020	0.340	1.974	0.948	8.39
27	29	30	Existing	1.730	0.018	0.191	1.828	0.897	7.87
28	29	40	Existing	0.454	0.009	0.138	0.516	0.843	7.57
29	29	50	Existing	0.115	0.008	0.127	0.176	0.942	7.16
30	52	20	Existing	0.668	0.011	0.388	0.732	1.157	3.32
31	52	20	FBO	0.719	0.017	0.418	0.787	1.216	3.05

*Base=Base Conditions, FBO=Full Build Out Land Use, Existing Land Uses

Table 7 . July-October averages of base case model predictions with intake located at North End of Basin 3 at a depth of 3 meters (model segment 29).

Description	Diatoms (mg/l)	Chloro-phyta (mg/l)	Blue-greens (mg/l)	Total Algae (mg/l)	POM (mg/l)	D. O. (mg/l)
Year 1 (2002)	0.06	0.01	0.12	0.19	0.13	9.34
Year 9 (2010)	0.13	0.01	0.25	0.38	0.23	9.57

Table 8. Maximums of base case model predictions with intake located at North End of Basin 3 at a depth of 3 meters (model segment 29).

Description	Diatoms (mg/l)	Chloro-phyta (mg/l)	Blue-greens (mg/l)	Total Algae (mg/l)	POM (mg/l)	D. O. (mg/l)
Year 1 (2002)	1.00	0.03	0.20	1.05	0.70	8.84
Year 9 (2010)	2.03	0.02	0.41	2.13	0.81	9.03

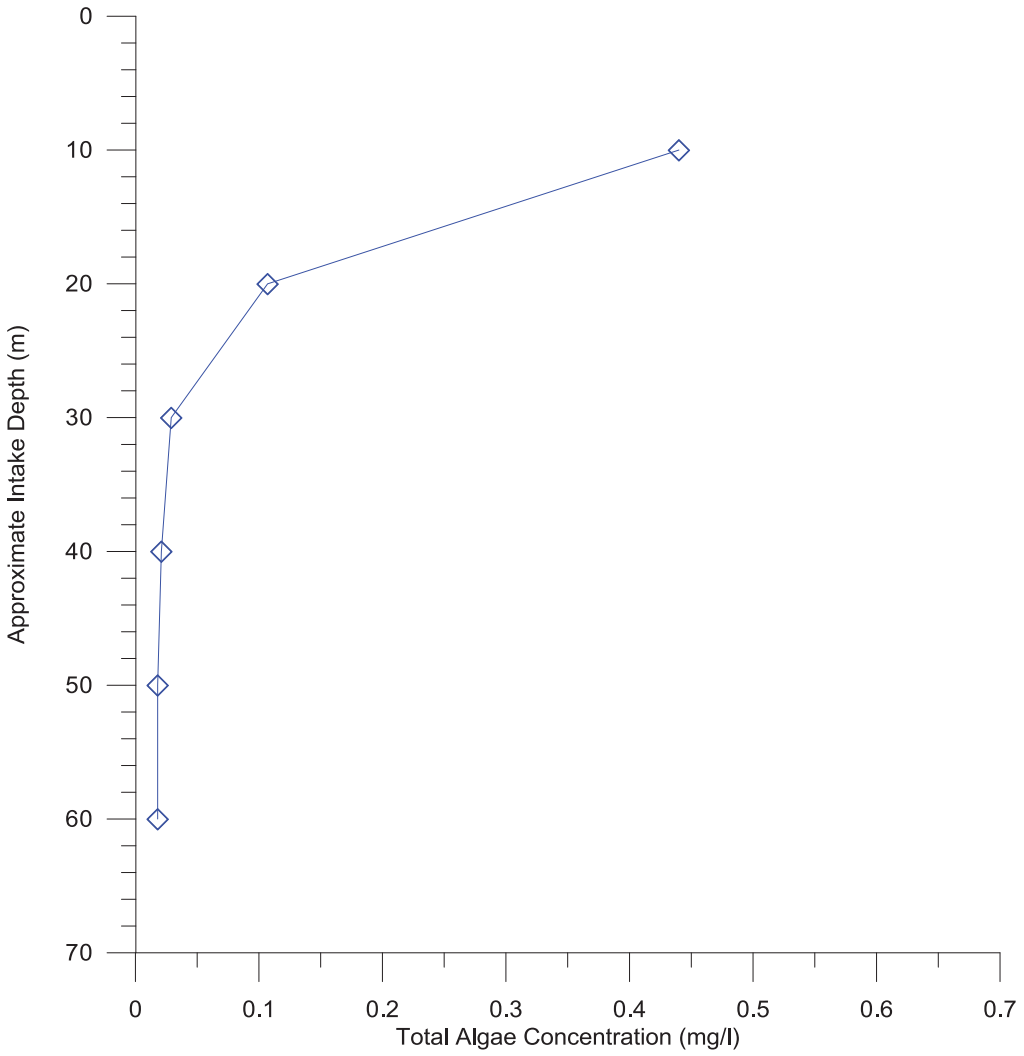

Figure 7. Total algae concentrations in intake at different depths at segment 29 (North end of basin #3). The concentrations were July-October averages (Table 3). Plotted scenarios include scenario 22 and scenarios 25 through 29 (Existing land uses).

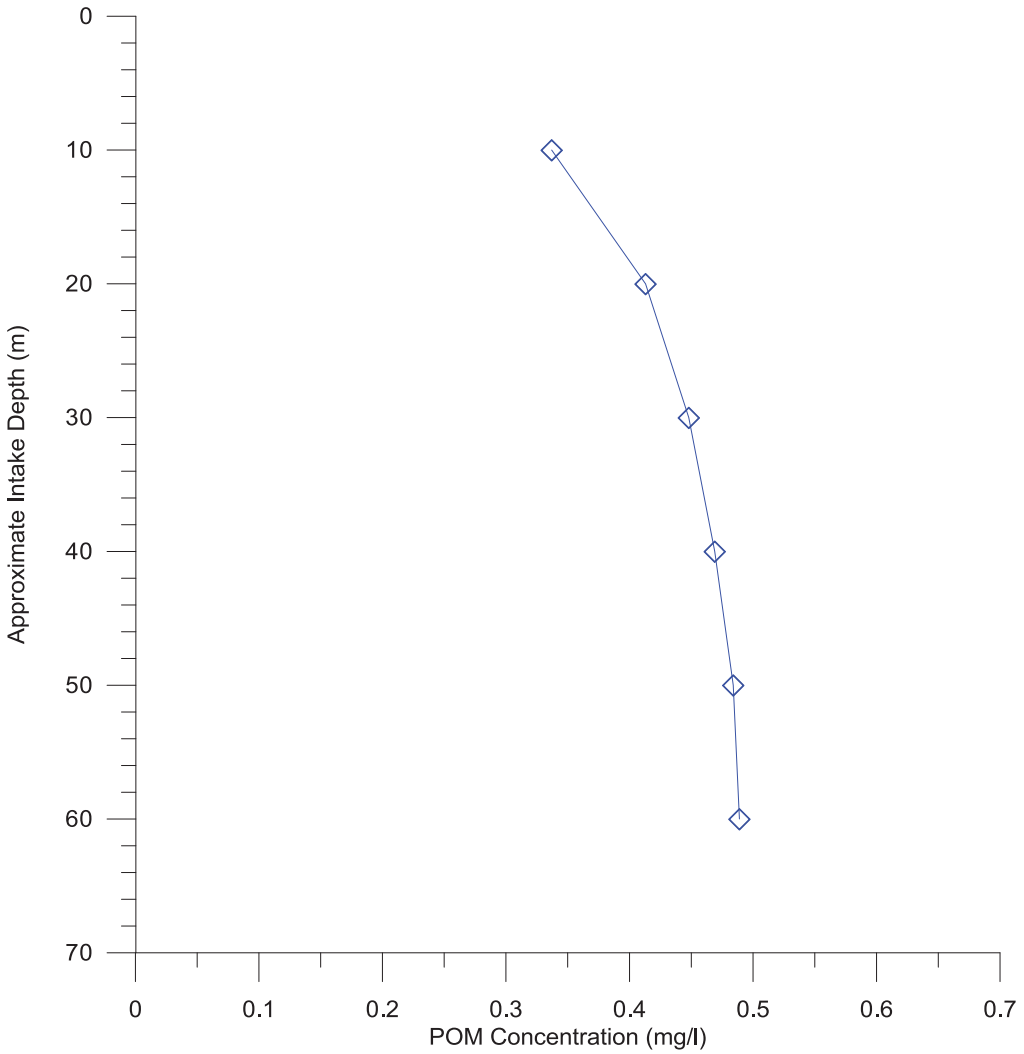

Figure 8. Particulate organic matter concentrations in intake at different depths at segment 29 (North end of basin #3). The concentrations were July-October averages (Table 3). Plotted scenarios include scenario 22 and scenarios 25 through 29 (Existing land uses).

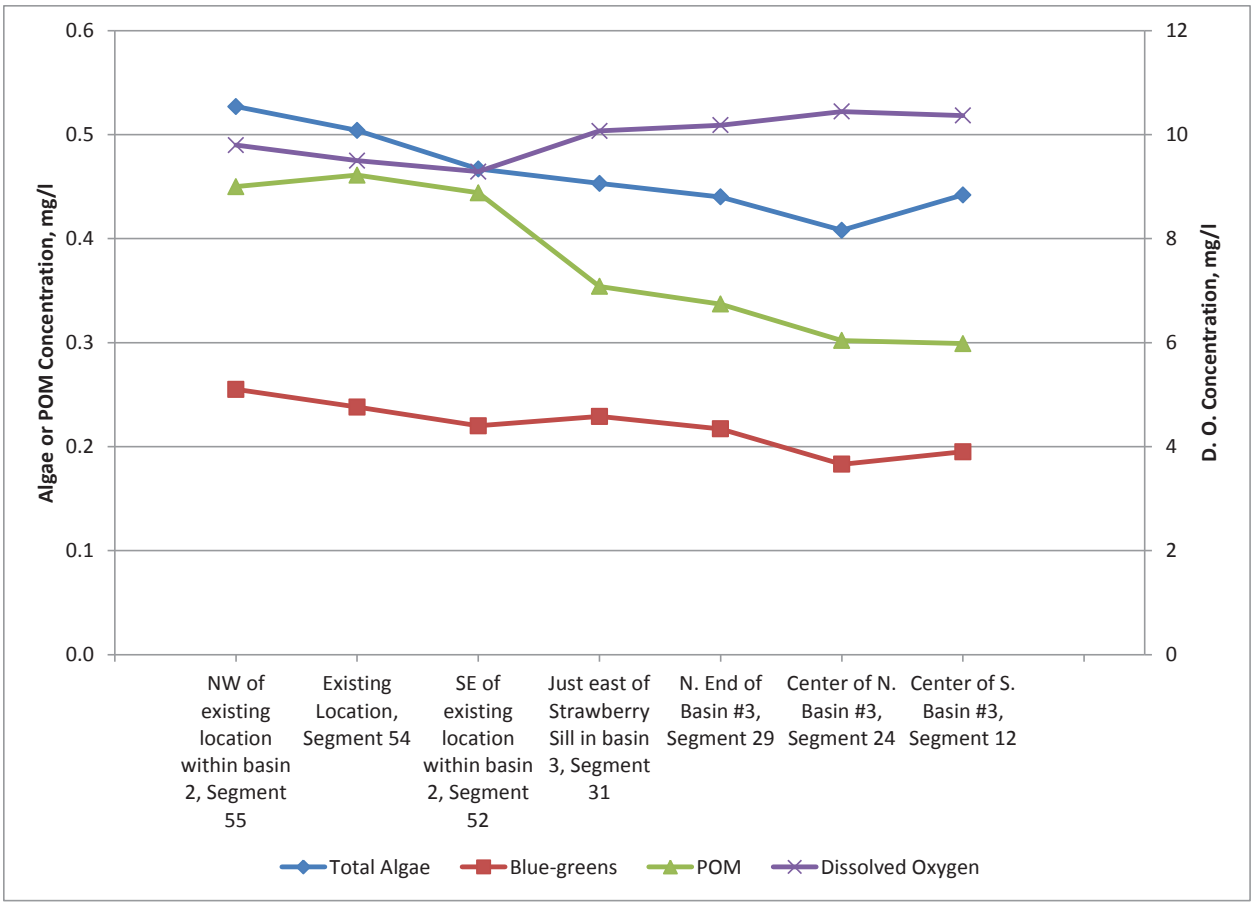

Figure 9. Total algae, blue-green algae, particulate organic matter (POM) and dissolved oxygen concentrations for July-October period with intake at an approximate depth of 10 m and at different locations within lake, moving along lake axis from north end of basin 2 to the south of basin 3. The watershed loading for these scenarios was for existing conditions.

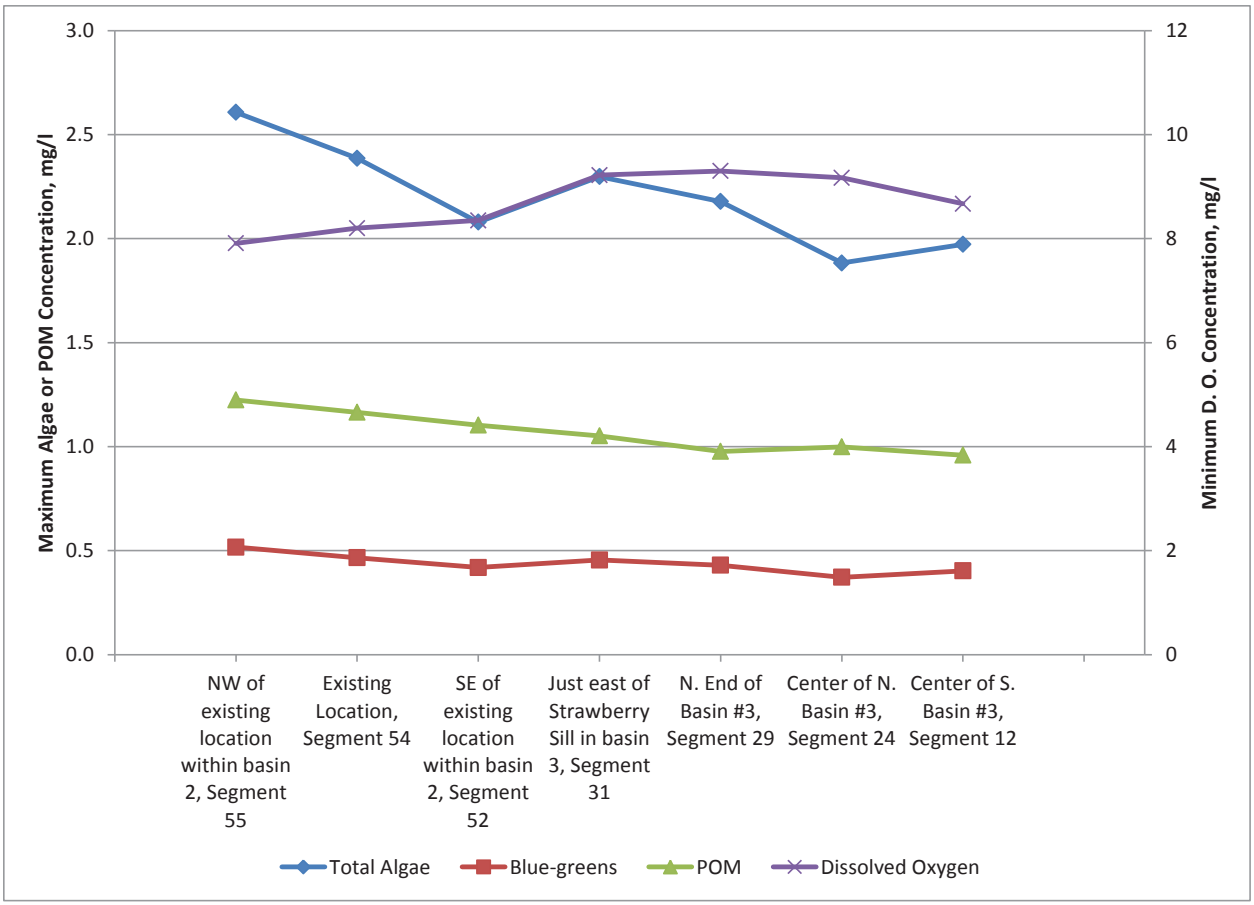

Figure 10. Maximum total algae, blue-green algae and particulate organic matter (POM) concentrations with intake at an approximate depth of 10 m and at different locations within lake, moving along lake axis from north end of basin 2 to the south. Minimum dissolved oxygen concentrations were also shown. Minimum dissolved oxygen concentrations were also shown. The watershed loading for these scenarios was for existing conditions.

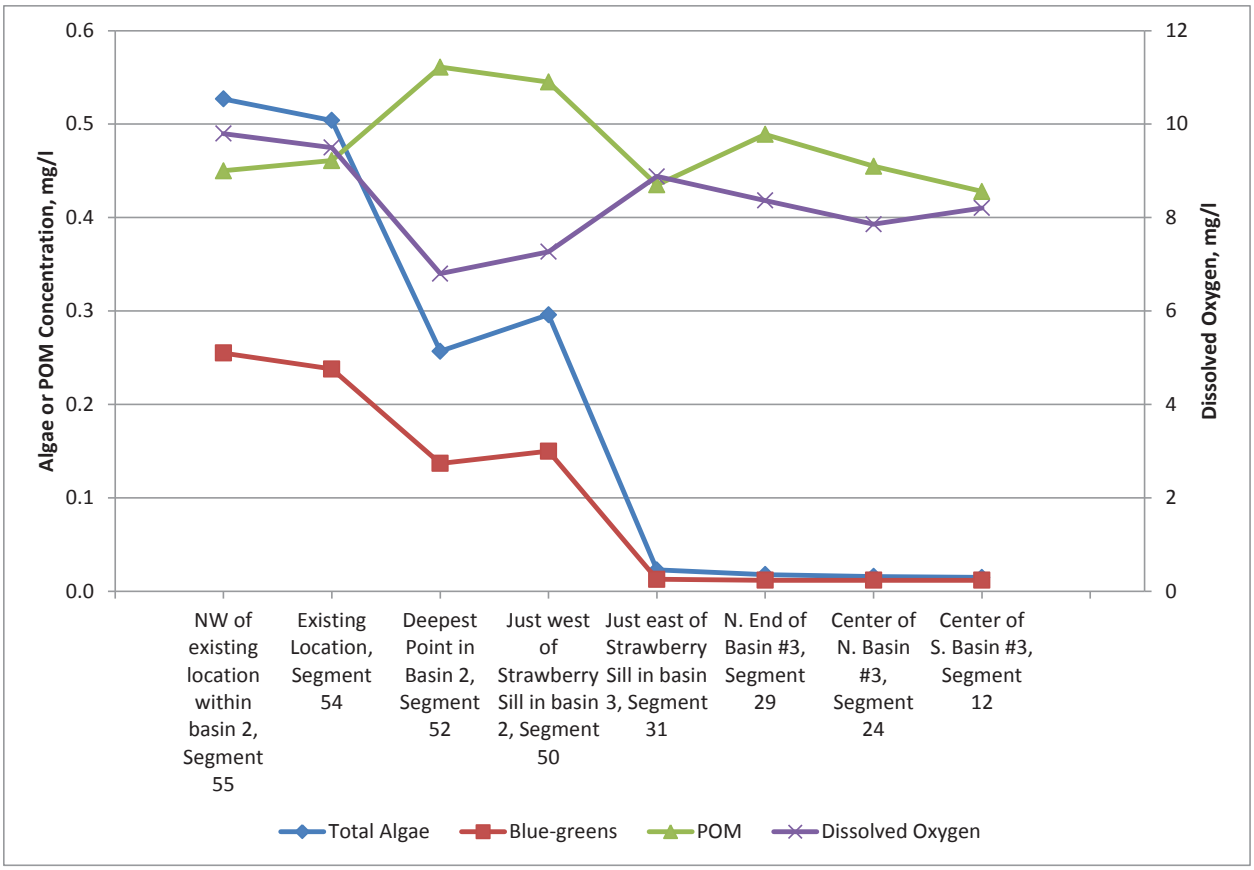

Figure 11. Total algae, blue-green algae, particulate organic matter (POM), and dissolved oxygen concentrations for July-October period with intake near bottom and at different locations within lake, moving along lake axis from north end of basin 2 to the south. The watershed loading for these scenarios was for existing conditions.

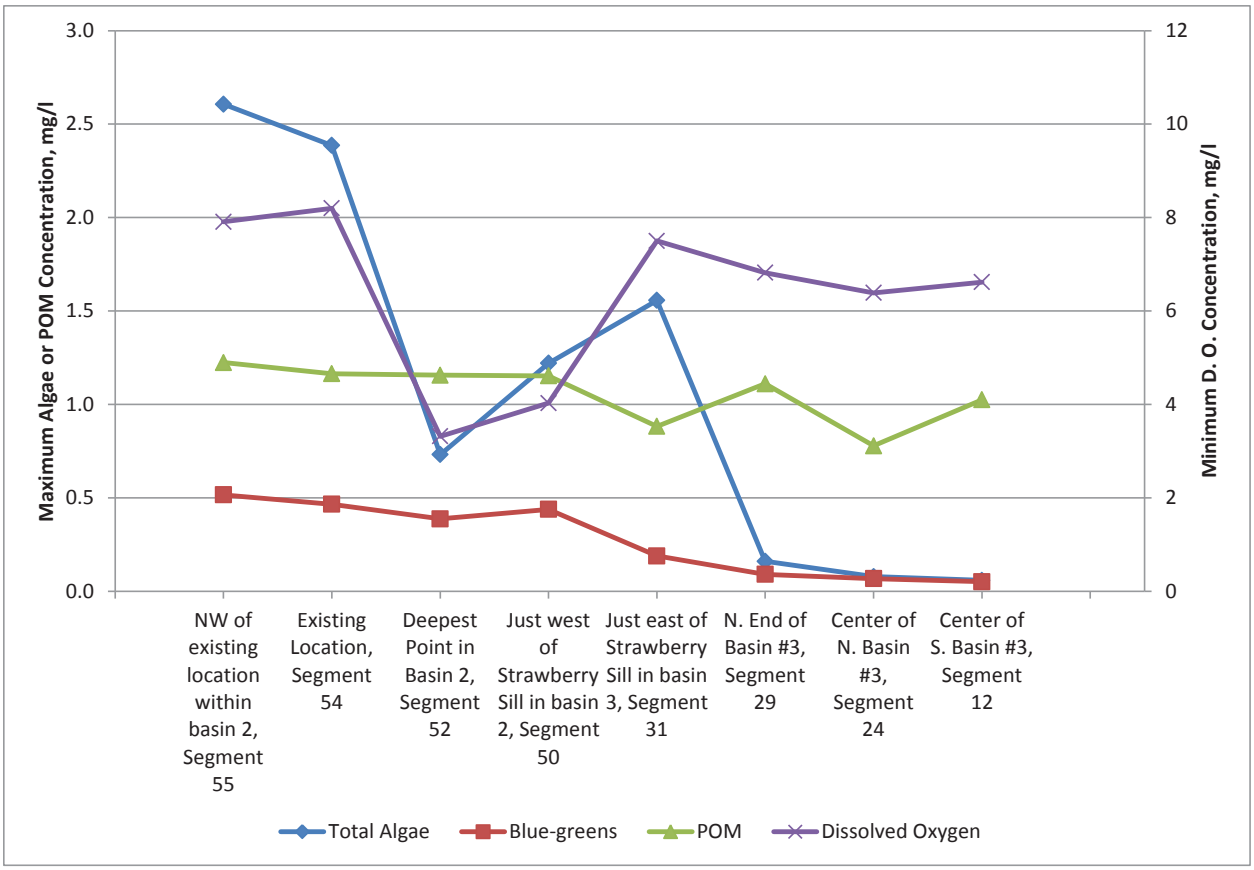

Figure 12. Maximum total algae, blue-green algae and particulate organic matter (POM) concentrations with intake located near bottom and at different locations within lake, moving along the lake axis from north end of basin 2 to the south. Minimum dissolved oxygen concentrations were also shown. The watershed loading for these scenarios was for existing conditions.

Summary and Conclusions

Using the CE-QUAL-W2 Lake Whatcom water quality model the City of Bellingham's water intake was moved to various locations within the lake with the goal of withdrawing water minimizing algae and particulate organic matter concentrations. Multiple scenarios were simulated with different intake locations and depths using watershed loadings corresponding to 2002-2003 land use (base case), existing land use, and full build-out land use. Tributary inflows were created by the Lake Whatcom HSPF model. Model simulations were performed for a period of 11 years in order for the model results not to be influenced by initial conditions. From the model predictions the following conclusions can be made:

- Algae concentrations dropped significantly when the intake was moved into the deep waters of basin 3. The intake had to be at least 30 m deep before an order of magnitude decrease in algae concentrations occurred.

- Within basin #2, the largest drop in algae concentration occurred when the intake was placed at the basin's deepest point. The July-October average blue-green algae concentrations were 39% less than at the existing location for the full build out scenario, but particulate organic matter concentrations increased 27%. Average total algae concentrations for the July-October period

decreased by 47%. Maximum blue-green concentration decreased by 10% and maximum total algae concentration decreased by 67%. Minimum dissolved oxygen concentration in the intake dropped to 3.05 mg/l, indicating possible anaerobic conditions near the bottom of the basin. Large decreases of intake algae concentrations did not occur at other intake locations within basin #2.

- When the intake was placed at a relatively shallow depth of 10 m in basin #3, algae concentrations were only reduced 20%-30% relative to concentrations for basin #2. To have a large reduction in algae concentrations, the intake had to be placed in much deeper waters.

- Although algae concentrations were greatly reduced when the intake was placed deep in basin #3, particulate organic matter (POM) concentrations were similar to POM concentrations near the surface.

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Appendix B
Benefit and Cost of Phosphorus-Reducing Activities in the Lake Whatcom Watershed

Benefit and Cost of Phosphorus-Reducing Activities in the Lake Whatcom Watershed

PREPARED FOR: Clare Fogelsong, City of Bellingham
Martin Kjelstad, PE, City of Bellingham
REVIEWED BY: Phil Martinez, PE, CH2M HILL
PREPARED BY: Amy Carlson, PE, CH2MHILL
DATE: December 15, 2011

Background and Purpose

Management of the Lake Whatcom drinking water reservoir is challenged by excessive nutrient loading that causes increased algal blooms that in turn results in reduced treatment capacity, increased treatment costs, and increased disinfection by-products at the City's Whatcom Falls Water Treatment Plant. In addition to the basic response needed to protect water supply delivery, a response is also required to the listing of Lake Whatcom as an impaired water body under the tenants of the Clean Water Act, the Total Maximum Daily Load (TMDL) listing for Total Phosphorus.

Lake Whatcom Management Program (LWMP) members, the City of Bellingham, Whatcom County, and Lake Whatcom Water and Sewer District, have researched, selected, and implemented several actions over the past 20 years to improve lake water quality. These actions have been described in work plans produced every five years that guide a multi-faceted response to many pollution issues. Although previous work plans have included many actions to reduce phosphorus loading, the current five year work plan (2010-2014 Work Plan) significantly increases the emphasis on phosphorus reduction actions. The reason for this emphasis is that phosphorous reduction was also the basis for the Summary Implementation Strategy (SIS), the first step in the TMDL response process. Even though the SIS is not yet completed, LWMP staff are beginning development of the next component of the TMDL response, the Detailed Implementation Plan (DIP) which requires identification of specific actions, costs of those actions, committed funding sources, and an implementation timeline for reducing the pollutant load (phosphorus) and thereby removing the lake from TMDL impaired status (delist).

The purpose of this study is to provide an initial comparison of several selected phosphorus-reducing and phosphorus-removal strategies that may inform policy decisions and the development of the DIP. To this end, the project team estimated phosphorus reduction and associated costs on a per-unit basis of specific in-watershed activities selected by City staff from the Lake Whatcom Management Program 2010-2014 Work Plan. The results of this work form an initial step in guiding further evaluation of activities and development of the DIP to comply with TMDL requirements.

Assumptions, Limitations, and Context

This work can inform the prioritization of watershed activities based on phosphorus reduction benefit. It can also guide and prioritize further evaluation of specific activities to assess their feasibility and effectiveness throughout the watershed for inclusion in the forthcoming DIP. It is understood that other factors beyond the computed phosphorous-reduction and associated unit costs presented herein also contribute to prioritization. These other factors may including: other studies, regulatory requirements, public expectations, political will, and others.

The following are assumptions and limitations upon which the estimates presented herein were based, including:

- The information upon which the estimates were based is from existing sources, where available and applicable, as well as information provided by the City. No new site-specific data was collected as part of this study. Additionally, anecdotal evidence of the phosphorous-reduction effectiveness is not incorporated into this work.

- The information provided in this summary memorandum is for planning-level purposes only to aid the City of Bellingham and Whatcom County in decision-making and/or prioritization and should not be used for design or construction.
- For this study, it was assumed each individual activity was independent of the others. Therefore, the phosphorus reduction estimates are for that activity only and don't include phosphorus reduction benefits of other activities.
- 'Benefit' only includes phosphorus reduction benefit, not a broader public benefit. Activities that may not be as cost-effective at reducing phosphorous may warrant consideration based on other factors. These other factors may include aesthetics, temperature reduction, public expectation, or other (non-phosphorus) water quality improvements.
- Costs are those that would primarily impact public entities (the City, County, and/or Water/Sewer District), not the cost to the private sector (such as cost to developers).
- Costs do not include the cost of land acquisition that might be required for implementation of activities such as bio-filtration swales or rain gardens.
- Costs do not include lost tax revenue to the County and City associated with changing the zoning ("down-zoning") of properties in the watershed to preclude development was not included in this work. In addition, the lost value to the property owner of down-zoning was not included in this work.
- Costs to property owners related to ordinance changes that govern development or changes related to forest practices were not included. In addition, costs associated with attempting to negotiate and implement such changes have not been included.
- Costs do not include annual operating and maintenance costs.
- The activities considered in this work were identified by members of the Lake Whatcom Management Program and do not represent all phosphorous-reducing activities that could be undertaken. Addressing the impact of Asian Clams was not included in this work because this issue came to light after this work was substantially complete.
- The extent to which each of the phosphorous reduction activities can be implemented in the watershed was not covered in this work. This, additional work task will be a key element of developing an effective DIP.

Watershed Activities

A working group consisting of staff from the City of Bellingham and Whatcom County prepared an initial draft of in-watershed activities based upon the Lake Whatcom Reservoir Management Program 2010-2014. CH2MHILL, the City, and the County collaboratively refined the list of activities, as presented in the summary list below:

1. Reducing development potential / developable land
2. Restoration of natural functions on acquisition properties
3. Bio-filtration: vegetated swales
4. Bioretention: rain gardens
5. Bio-filtration: street trees
6. Lawn replacement & landscaping: retrofit to provide bioretention
7. Infiltration: dry wells
8. Infiltration: trenches
9. Infiltration: pervious pavement
10. Infiltration: basin
11. Rainwater reuse
12. Onsite dispersion
13. Media filters
14. Sizing culverts to eliminate erosion

15. Street sweeping
16. Controlling erosion through streambank stabilization or restoring stream buffer vegetation
17. Regulations: Phosphorus fertilizer ban
18. Education: Watershed signs
19. Education: Mass mailings
20. Education: Online information
21. Education: Newspaper ads
22. Education: Video presentations
23. Education: Community events (public meetings)
24. Education: Onsite training/workshops
25. Education: Resident contact
26. Education: Project consultation
27. Incentives
28. Transition from Ecology Water Quality Assurances of Forest Practices to pre-development conditions
29. Design standards for new and retrofitted roads
30. Reconfigure roadside ditches
31. Reconfigure streets
32. Vehicle trips - reduce and redirect
33. Recreational facility design and use (Improving existing facilities)
34. Watershed-wide enforcement
35. Animal waste: wildlife (goose)
36. Septic system transition to sewer connection

Summary of Results

Exhibit 1 presents a graphical summary of the cost-benefit in terms of dollars per pound of phosphorus removed for each of the activities.

Exhibit 2 presents a tabular summary of the cost-benefit in terms of dollars per pound of phosphorus removed for each of the activities.

Exhibit 3 contains the detailed information which is the basis for phosphorus reduction estimates and costs contained in this memorandum. Exhibit 3 includes all of the 36 activities except for the education, incentives, and enforcement activities, which were separated out to allow for an off-line comparison. This was done to allow the City and County to compare education activities amongst themselves separate from the other activities.

Exhibit 4 contains a summary of information available in the literature about effectiveness of different education methods and also incentives and enforcement. This summary provides the City of Bellingham and Whatcom County with a planning tool to assist in prioritizing which type of education methods to implement.

Exhibits 1-4 are attached to this memorandum.

Benefit in Terms of Phosphorus Reduction

For this cost-benefit study, benefit is defined solely as phosphorus reduction. While each activity may have other benefits such as aiding in regulatory compliance or to addressing a public safety issue, the benefit described in this study is only phosphorus reduction. These activities may not lead to a measurable phosphorus reduction but may be a good idea for those other reasons. In the case of some of the activities, quantifying a phosphorus reduction was not possible. This was because information was not found in the literature.

Cost of Activities

These costs are estimates of capital costs. In some cases, where they were readily available, annual maintenance costs are also provided within Exhibit 3. Note that costs shown in Exhibit 3 only reflect public cost (that is, cost to the public agency) and not other costs such as cost to developers.

Attachments:

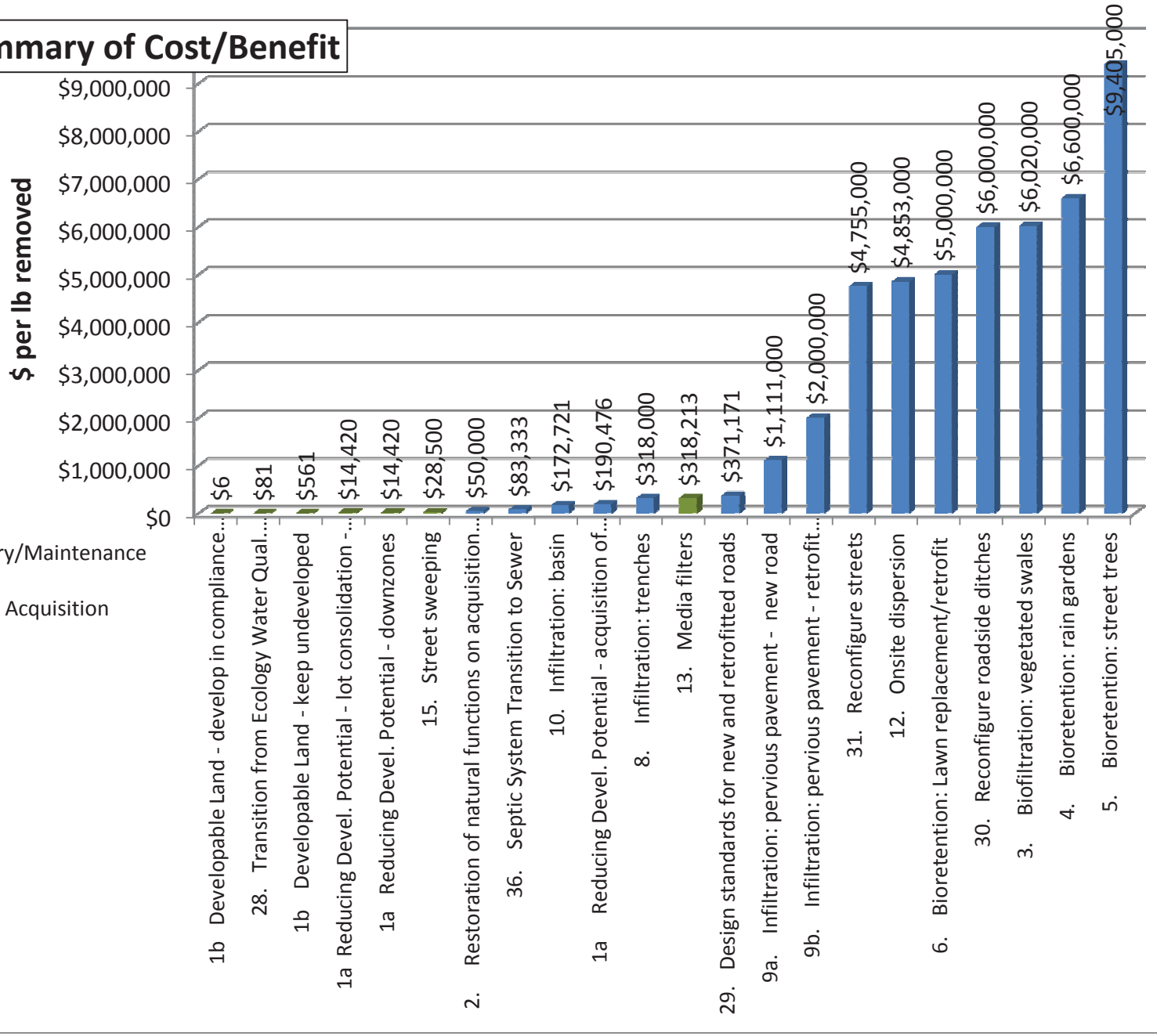
Exhibit 1 –Summary of Cost-Benefit

Exhibit 2 - Tabular Summary of Cost-Benefit

Exhibit 3 – Details of Cost-Benefit Analysis for Watershed Activities (except for education/incentives/enforcement)

Exhibit 4 - Education/Incentives/Enforcement Activities

Exhibit 1 - Summary of Cost/Benefit



GREEN =
Programmatic/Regulatory/Maintenance
BLUE =
Capital Investment/Land Acquisition

Exhibit 2: Tabular Summary of Cost-Benefit Evaluation

Activity	Benefit		Cost		Cost/Benefit ($ / lb)
	Phosphorus Reduction	Units	Initial Capital	Units	
1. Reducing development potential / developable land					
Reducing Devel. Potential - acquisition of open space	1.05	lb/acre/year	$200,000	per acre	$190,476
Reducing Devel. Potential - lot consolidation - residential	minimal	lb/acre/year	$20,000	total	$14,420
Reducing Devel. Potential - downzones	minimal	lb/acre/year	$20,000	total	$14,420
Developable Land - develop in compliance with City ordinance [1]	2498	lb/yr for the watershed	$15,000	total	$6
Developable Land - keep undeveloped [1]	2775	lb/yr for the watershed	$1,402,000	total	$561
2. Restoration of natural functions on acquisition properties	1.0	lb/acre/year	$50,000	per acre	$50,000
3. Biofiltration: vegetated swales	20	%	$8	per sf of treated area	$6,020,000
4. Bioretention: rain gardens	50	%	$22	per sf of treated area	$6,600,000
5. Bioretention: street trees	40	%	$25	per sf of treated area	$9,405,000
6. Bioretention: Lawn replacement / retrofit	75	%	$25	per sf of treated area	$5,000,000
7. Infiltration: dry wells [2]	-	-	-	-	-
8. Infiltration: trenches	70	%	$1.48	per sf of treated area	$318,000
9a. Infiltration: pervious pavement - new road	60	%	$4.43	per sf of treated area	$1,111,000
9b. Infiltration: pervious pavement - retrofit existing	60	%	$8	per sf of treated area	$2,000,000
10. Infiltration: basin	100	%	$50,000	per acre of treated area	$172,721
11. Rainwater reuse [3]	minimal	lb/year	-	-	-
12. Onsite dispersion	40	%	$12.90	per sf of treated area	$4,853,000
13. Media filters [4]	52	%	$1.10	per sf of treated area	$318,213
14. Sizing ditches/culverts to eliminate erosion [3]	minimal	lb/year	-	-	-
15. Street sweeping [5]	60	%	$0.11	per sf swept	$28,500
16. Controlling erosion through streambank stabilization or restoring stream buffer vegetation [8]	moderate	lb/year	-	-	-
17. Regulations: Phosphorus fertilizer ban [6]	0.5	lb/acre/year	-	-	-
18. Education: Watershed signs [7]	-	-	-	-	-
19. Education: Mass mailings [7]	-	-	-	-	-
20. Education: Online information [7]	-	-	-	-	-
21. Education: Newspaper ads [7]	-	-	-	-	-
22. Education: Video presentations [7]	-	-	-	-	-
23. Education: Community events (public meetings) [7]	-	-	-	-	-
24. Education: Onsite training/workshops [7]	-	-	-	-	-
25. Education: Resident contact [7]	-	-	-	-	-
26. Education: Project consultation [7]	-	-	-	-	-
27. Incentives [7]	-	-	-	-	-
28. Transition from Ecology Water Qual. Assurances of Forest Practices to pre-development conditions	0.1	lb/acre/year	$200,000	total	$80.65
29. Design standards for new and retrofitted roads	60	%	$1.48	per sf of treated area	$371,171
30. Reconfigure roadside ditches	20	%	$8.00	per sf of treated area	$6,000,000
31. Reconfigure streets	30	%	$10.00	per sf of treated area	$4,755,000
32. Vehicle trips - reduce and redirect [3]	minimal	lb/year	-	-	-
33. Recreational facility design and use (improving existing facilities) [3]	minimal	lb/year	-	-	-
34. Watershed-wide enforcement [7]	-	-	-	-	-
35. Animal waste: wildlife (goose) [9]	0.3	lb/goose/year	-	-	-
36. Septic System Transition to Sewer	0.6	lb/septic system/yr	$50,000	per septic system	$83,333

Notes:

[1] As compared to developing conventionally

[2] At the direction of the City did not assess cost-benefit of this activity because activity is not feasible throughout most of the watershed (due to soil and groundwater conditions)

[3] Did not characterize cost or cost-benefit after determination of minimal direct phosphorus reduction benefit

[4] Evaluated several types of media filters; Phosphorus Reductions and Costs shown in this table are on conservative end of range

[5] $/lane mile swept to be determined

[6] no cost-benefit characterized because cost to implement ordinance already expended; assuming no cost for enforcement; enforcement/education/incentives covered on Table 2

[7] Activities 18 through 37 (education/incentives/enforcement) summarized on Table 2 of this deliverable and not summarized here

[8] stream buffer provides indirect phosphorus reduction benefit (by reducing velocities and promoting infiltration); stream channel stabilization could have significant phosphorus reduction benefit

[9] At the direction of the City did not assess cost-benefit of this activity because implementation of this activity is not acceptable

Exhibit 3. Lake Whatcom Phosphorus-Reduction: Cost-Benefit Analysis Detail

Activity Number	Activity	Units of Phosphorus (TP) Removal	Phosphorus Loading Removed	Cost	Cost/Benefit	Notes
1a	**Reducing development potential**					
1a(i)	Acquisition of existing open space	lb/acre/year	1.05	$200,000 per acre for land acquisition**	In the first year, $200,000/acre cost with a 1.05 lb/acre benefit = $190,476/lb	Acquisition does not include any land cover changes or site management. Phosphorus removal estimate is equal to the difference between developed land loading (blended pervious and impervious) and undeveloped land loading, per the Lake Whatcom TMDL study (Ecology 2008a). Developed pervious (82% of developed area): 1.24 lb/acre/year; Developed impervious (18% of developed area); The land use acreages indicated that 18 percent of the developed area is impervious. Therefore, the blended loading calculates to 1.20 lb/acre/year: 0.99 lb/acre/year (CDM 2008; Ecology 2008) Deciduous Forest: 0.14 lb/acre/year; Evergreen Forest: 0.16 lb/acre/year; Mixed Forest: 0.14 lb/acre/year; An overall total phosphorus loading of 0.15 lb/acre/year was assigned. 1.20 (Developed land loading) – 0.15 (Undeveloped land loading) = 1.05 lb/acre/year (developed land loading and undeveloped land loading from CDM 2008; Ecology 2008) Cost equals the sum of the current price of open space parcels and cost to process acquisition **Note that this study does not consider costs to the private sector (i.e. developers) only to the public sector (i.e. administrative costs to administer development). This is especially important to note for these items 1a and 1b.
1a(ii)	Lot consolidation - residential	lb/acre/year	*Minimal benefit*	$20,000 total, watershed wide**	'minimal benefit' estimated as 0.01 lb/acre/year; Assuming can implement on only 5% of developable land (5% of 2,774 acres or 138.7 acres); $20k / (138.7 * 0.01) = $14,420/lb for the first year	Phosphorus removal estimate would equal the difference between multi-family and single family residential phosphorus loading, which is not likely a significant difference. Furthermore, lot consolidation would occur as opportunities present themselves, not uniformly across the watershed. The actual phosphorus loading reduction is therefore expected to be relatively small. Cost to process lot consolidation – 2 FTEs for 1 month, assuming $100,000 per FTE for 1 year plus additional administration work
1a(iii)	Downzones	lb/acre/year	*Minimal benefit*	$20,000 total, watershed wide**	'minimal benefit' estimated as 0.01 lb/acre/year; Assuming can implement on only 5% of developable land (5% of 2,774 acres or 138.7 acres); $20k / (138.7 * 0.01) = $14,420/lb for the first year	Phosphorus loading reduction would result from reduced lot density (i.e., residential to public/recreational, commercial to public/recreational, commercial to residential). There are too few industrial zoned parcels in the watershed to include in this evaluation. Minton (2011) does not report a significant difference in phosphorus loading between single-family residential, multi-family residential, and commercial/industrial land. As with lot consolidation, downzoning would occur as opportunities present themselves, not uniformly across the watershed. The actual phosphorus loading reduction is therefore expected to be relatively small. Cost to process downzone – 2 FTEs for 1 month, assuming $100,000 per FTE for 1 year plus additional administration work

Exhibit 3. Lake Whatcom Phosphorus-Reduction: Cost-Benefit Analysis Detail

Activity Number	Activity	Units of Phosphorus (TP) Removal	Phosphorus Loading Removed	Cost	Cost/Benefit	Notes
1b	**Developable land**					Assumes 2,774 acres of developable land in the watershed. This represents the difference between developed land under the "full buildout" and the "base scenarios" in the Lake Whatcom TMDL Study (Ecology 2008a). Phosphorus loading rate assumptions used: • 0.20 lb/acre/year from undeveloped land (see item 1a, reference: CDM 2008; Ecology 2008) • 0.3 lb/acre/year (City-estimated) from land developed in compliance with City's LID standards • 1.20 lb/acre/year from conventionally developed land (reference: CDM 2008; Ecology 2008)
1b(i)	Benefit of developing all conventionally	lb/year (watershed-wide)	0 *(as baseline for other options under item 1b)*	$0 (*as baseline for other options under item 1b)*	--	
1b(ii)	Benefit of developing all land in compliance with City ordinance *as compared to* developing all of conventionally	lb/year (watershed-wide)	2,498	$15,000 **	for the first year: $15,000 / 2,498lbs/yr = $6/lb	Assumes that the "baseline" condition is conventional development of all 2,774 acres of developable land in the watershed. (1.20 lb/acre/year developed land loading)(2,774 acres of developable land) = 3,330 lb/year (0.3 lb/acre/year developed land loading)(2,774 acres of developable land) = 832 lb/year 3,330 – 832 = 2,498 lb/year (Cost equal to the permitting and processing cost for developing a 1) SFR residential and 2) commercial in compliance with City LID standards, administrative costs only estimated at 1 FTE for 2 months, assuming $100k per FTE per year.)
1b(iii)	Benefit of keeping developable land undeveloped *as compared to* developing all of it conventionally	lb/year (watershed-wide)	2,775	$15,000 + $500/acre (estimated same as above PLUS $500/acre) **	for the first year: ($15,000 +$500/acre * 2,774 acres) / 2,498lbs/yr = $561/lb	(1.20 lb/acre/year developed land loading)(2,774 acres of developable land) = 3,330 lb/year (0.2 lb/acre/year undeveloped land loading)(2,774 acres of developable land) = 555 lb/year 3,330 – 555 = 2,775 lb/year (Cost equal to the building department revenue that would have been generated from a 1) SFR residential and 2) commercial) PLUS the administrative costs as determined above)
2	**Restoration of natural functions on acquisition properties**	lb/acre/year	1.0	$50,000 / acre **	In the first year, $50,000/acre cost with a 1.0 lb/acre benefit = $50,000/lb	Assumes restoration is to forested conditions. This would take several years to achieve. The phosphorus loading under restored conditions would be expected to be lower than that for land developed in compliance with City LID standards (see item #1a). The phosphorus loading benefit would be the difference between developed and restored land loading rates: 1.20 – 0.20 lb/acre/year = 1.0 lb/acre/year TP removed by restoring land to forested conditions. See also notes for item #1. Continued access and limited use of restored lands would probably occur to a higher degree than in the case of natural land preservation. Cost equal to the total cost for planning, design, construction and planting, and maintenance until the site is self-sufficient, estimated at $50,000 per acre. **Note that this study does not consider costs to the private sector (i.e. developers) only to the public sector (i.e. administrative costs to administer development). This is especially important to note for these items 1a and 1b.

Exhibit 3. Lake Whatcom Phosphorus-Reduction: Cost-Benefit Analysis Detail

Activity Number	Activity	Units of Phosphorus (TP) Removal	Phosphorus Loading Removed	Cost	Cost/Benefit	Notes
3	**Biofiltration: vegetated swales**[1]	%	20	Construction: $8/sf of area treated	Assuming 20% removal of phosphorus at $8 per square foot of treated area = $6,020,000/lb (*Example calculation, which applies to all other cost-benefit calculations in this study: 8,974 lb/year total Phosphorus loading watershed-wide, with 31,000 acres in the watershed equates to an average of 0.29 lb/year per acre; 20% removal yields a 0.06 lb of phosphorus removed per acre per year. At $8/sf of treated area, and 43,560 sf in an acre, the cost to treat one acre is $348,480. $348,480 to remove 0.06 lb of phosphorus equates to $6,020,000 per lb of Phosphorus removed. Note that the results of this calculation are independent of the scale on which it is based. i.e. yields the same answer regardless of 1 acre, 10 acres, or 100 acres, or watershed-wide implementation*)	Assumes no infiltration. CWP (2008) reports 10-20% flow reduction plus an additional 20-40% reduction of TP in the surface flow. Geosyntec (2008) reports 36% reduction. The "Dayton" swale in Seattle in the NPDES BMP database reported no TP removal (inflow concentration of 0.18 mg/l, outflow concentration of 0.19 mg/l). Other literature reports only limited TP removal for biofiltration swales.
		%	77-100			"Geneva Swales" testing results (provided by Whatcom County via email, 10/3/2011).
		lb/swale/year	0.02 – 6.71 Avg: 0.86			2010 City data – range of phosphorus removal of all 11 facilities listed as swales: 0.02 – 6.71 lbs (Bellingham 2011). 2009 City data – range of phosphorus removal: 0.07 – 2.65 lbs Cost - Silver Beach Creek Outreach Program cited a cost for the Lahti Drive bio-infiltration swale (design, survey, and permitting): $24,356.04; OR Total cost (including design, survey, permitting, construction, and annual maintenance) of at least 5 of the 10 swales included in the Detailed Phosphorus analysis.
4	**Bioretention: rain gardens**[1]	%	50	Construction: $18.90 - $32.80/sf of treated area	Assuming 50% removal of phosphorus at $22 per square foot of treated area = $6,600,000/lb	CWP (2008) reports 80% flow reduction (40% if there are underdrains) plus an additional 25-50% reduction of TP in overflow. Tetra Tech (2008) reports overall 70 % reduction. Given limited infiltration capability of the soils in the watershed, assume a 50% TP reduction credit.
		mg/L TP	+0.04			NPDES Urban BMP Performance Tool - University of Connecticut 2005; higher outflow concentration; no volume data available.
		lb/rain garden/year	0.04-5.90			2009/2010 City data: Bloedel Raingarden 1 – 5.51 acres treated, 100% treatment efficiency; BloedelRaingarden 2 – 0.34 acres treated, 10% treatment efficiency Net phosphorus removal requires that rain gardens are sized correctly. The City has indicated that monitoring found one of its rain gardens exports phosphorus. Total cost (including design, survey, permitting, construction, and annual maintenance) of the Bloedel rain gardens.
		lb/year (watershed-wide)	423			An estimated 30% of the developed basin can accommodate rain gardens; 50% TP removal efficiency assumed; 2,352 acres of developed land in the watershed (Ecology 2008a); loading rate from developed land assumed to be 1.20 lb/acre/year (see item #1). Cost: Estimate number of rain gardens needed to treat the full 30% of the developed basin that can accommodate it. Multiply the number of rain gardens by the cost estimated from the above Bloedel rain gardens (the result will likely be a cost range).

Exhibit 3. Lake Whatcom Phosphorus-Reduction: Cost-Benefit Analysis Detail

Activity Number	Activity	Units of Phosphorus (TP) Removal	Phosphorus Loading Removed	Cost	Cost/Benefit	Notes
5	**Bioretention: street trees**[1]	%	15	No quote provided by Filterra Systems (as surrogate, use $18.90 - $32.80/sf of treated area)	Assuming 40% removal of phosphorus at $25 per square foot of treated area = $9,405,000/lb	Center for Watershed Protection (CWP 2008) reports a 15% flow reduction; but gives no further credit for reduction of TP in the remaining surface flow. Washington Department of Ecology (Ecology) TAPE Program (Ecology 2011) made a preliminary determination that Filterra has poor phosphorus reduction and does not meet phosphorus treatment criteria of 50% removal.
		lb/tree/year	0.0014			Assumes 0.36 mg/L TP in runoff (Minton 2011); 845 gal/tree stormwater runoff reduction (McPherson et al. 1999); Filterra's whitepaper reports 56.5% TP removal efficiency in a Bellingham study. Cost: Quotes for Filterra system
		%	74			Charles River Watershed Association (CRWA) 2009 reports 74% TP removal from street trees.
6	**Lawn replacement & landscaping: retrofit to provide bioretention**[1]	%	75	As a surrogate, use item 4 above: Construction: $18.90 - $32.80/sf of treated area	Assuming 75% removal of phosphorus at $25 per square foot of treated area = $5,000,000/lb	Lawns often sit atop poorly draining subsoils, resulting in limited infiltration of rainfall and extensive runoff. The subsoils can be amended with compost, greatly increasing infiltration capacity. The amended soils can then be replanted either with native plants or reseeded in grass. CWP (2008) reports a 75% flow reduction attributable to amended soils. No treatment for TP is credited for surface runoff. Therefore assign a 75% TP reduction credit for the replaced lawn area. Assumes fertilizer restriction is enforced.
7	**Infiltration: dry wells**[1]	n/a	n/a	--	--	Phophorus removal is likely similar to rain gardens or infiltration trenches. New dry wells are not likely an option in the Lake Whatcom watershed, because Ecology requires them to be located in cobble areas, which are not naturally present in the watershed. Therefore, this item was not evaluated further.
8	**Infiltration: trenches**[1]	%	(-)100 - 65	$1.48 per sf of treated impervious surface.	Assuming 70% removal of phosphorus at $1.48 per square foot of treated area = $318,000/lb	NPRPD 2007 – Infiltration

Exhibit 3. Lake Whatcom Phosphorus-Reduction: Cost-Benefit Analysis Detail

Activity Number	Activity	Units of Phosphorus (TP) Removal	Phosphorus Loading Removed	Cost	Cost/Benefit	Notes
9a	**9a - Infiltration: pervious pavement (new road)** [1]	%	60	$4.43 per sf of treated impervious surface.	Assuming 60% removal of phosphorus at $4.43 per square foot of treated area = $1,111,000/lb	CWP (2008) reports 45% flow reduction (no underdrains) plus an additional 25% reduction of TP in the underflow. Tetra Tech (2008) reports overall 67% reduction. Assign a 60% TP reduction credit (for the pervious pavement area, only). This estimate addresses the TP removal from the retrofitted road surface only. There would be a lower % removal of TP on a watershed-wide basis. Cost for new road (from SPU); Retrofitting existing road would be more expensive, as it would include the breakup and removal of the existing road surface and road bed. The difference between the $8/sf for the retrofit and the $4.43/sf for the new is to remove the existing road and roadbed to make way for the new pervious pavement.
9b	**9b - Infiltration: pervious pavement (retrofit existing road)** [1]	%	60	$8 per sf of treated impervious surface.	Assuming 60% removal of phosphorus at $8 per square foot of treated area = $2,000,000/lb	
10	**Infiltration: basin**[1]	%	100	$6/sf of infiltration basin ($100,000 for an infiltration basin treating approx. 2 acres)	Assuming 100% removal of phosphorus at $50,000 per acre of treated area = $172,721/lb	Assumes 100% infiltration to groundwater, with no discharge to surface runoff. This item was formerly listed as "Sand filter" – see also item #13, Media Filters Cost: based upon recent experience on M Street project for City of Auburn (infiltration pond as stormwater management)
11	**Rainwater reuse**	lb/L water	*Minimal benefit*	--	--	Roof runoff presumably contributes a minimal fraction of the total phosphorus load, as rooftop runoff does not contain significant phosphorus concentrations. However, it likely contributes to phosphorus loading reductions when combined with other residential BMPs such as bioretention (i.e., lawn replacement) and if used widely throughout the watershed. Also note that phosphorus concentrations in rooftop runoff can vary seasonally, with a presumed high loading from leaf litter in the fall season. However, since no documentation is available, did not include this within the context of this study. CWP (2008) reports 40% flow reduction. A Bellingham study of the Rain Barrel Program concluded that 24% of captured roof runoff was infiltrated and the remainder dispersed into landscaped areas.
12	**Onsite dispersion**	%	40	$12.90/SF	Assuming 40% removal of phosphorus at $12.90 per square foot of treated area = $4,853,000/lb	Evaluated as a stand-alone activity. CWP (2008) reports 50-75% flow reduction but gives no further credit for reduction of TP in the remaining surface flow. Given limited infiltration capability of the soils in the watershed, assign a 40% TP reduction credit.

Exhibit 3. Lake Whatcom Phosphorus-Reduction: Cost-Benefit Analysis Detail

Activity Number	Activity	Units of Phosphorus (TP) Removal	Phosphorus Loading Removed	Cost	Cost/Benefit	Notes
13	**Media filters**	%	50-90	Aquip: Construction cost= $0.81 to 1.8/sf of treated impervious surface Annual Maintenance cost = $0.29 to $0.80 per sf of treated surface. BaySaver: Construction Cost = 0.34 to 0.91 per sf of treated impervious, Maintenance cost = $0.06 to $0.17 per sf of treated impervious surface area.	8974 lb/year P loading watershed-wide, assuming 70% removal of phosphorus; Watershed-wide implementation reduces phosphorus by 6,281 lbs and costs $1,620,000,000 at $1.20/sf, for a cost/benefit for the first year only of approximately $257,957/lb	The TAPE website (Ecology 2011) identifies the following six proprietary devices certified by Ecology as effective for stormwater phosphorus treatment, reducing total phosphorus by at least 50%. The percentage removals shown for Pilot and Conditional Uses are preliminary. Aquip Enhance Stormwater Filtration System: 60-90% TP reduction (Conditional Use) Americast Filterra System: less than 50% TP removal (Conditional Use) FloGard Perk Filter: 62% TP removal (General Use) Request quote from BaySaver Technologies BayFilter: 55% TP removal (Conditional Use) Aquashield AquaFilter: No TP removal % identified (Pilot Use) WSDOT Media Filter Drain: 86% TP removal (General Use) Cost: from Manufacturers
	Sand Filters	%	50	Construction cost= $1.00 sf of treated impervious surface	8974 lb/year P loading watershed-wide, assuming 50% removal of phosphorus; Watershed-wide implementation reduces phosphorus by 4,487 lbs and costs $1,350,000,000 at $1.00/sf, for a cost/benefit for the first year only of approximately $300,949/lb	Sand filters: CWP (2008) credits a 59% reduction of TP. Geosyntec (2008) reports 30%. WA Dept. of Ecology bestows a 50% phosphorus removal credit for this BMP (i.e., a minimum 50% P removal). Assign a 50% TP reduction credit. Assumes all treated water is discharged to surface runoff.
		lb/filter/year	4.77 – 34.84 Avg: 14.91			2009/2010 City data – “Sandfilter” and “Sandfilter/Infiltration” (Bellingham 2011). Does not include the Electric Ave Sandfilter.
	StormFilter® with PhosphoSorb ™	lb/acre/year	0.80	Construction Cost: $0.52 to 1.31 per sf of treated impervious. Maintenance Cost: $0.06 to $0.25 per year per sf of treated impervious surface area.	8974 lb/year P loading watershed-wide, assuming 52% removal of phosphorus; Watershed-wide implementation reduces phosphorus by 4,666 lbs and costs $1,350,000,000 at $1.10/sf, for a cost/benefit for the first year only of approximately $318,312/lb	Contech StormFilter® with PhosphoSorb ™ (www.contech-cpi.com) reports: Influent: 0.02 – 0.49 mg/L; Effluent: 0.025-0.083 mg/L (these values reflect manufacturer’s field testing at Cable Street, Whatcom County). Phosphorus removal rate of 67% is reported for influent TP concentrations greater than 0.1 mg/l (CONTECH 2010). Loading rate of 1.20 lb/acre treated/year assumed (see item #1). Cost: Request quotes from manufacturers
		%	21-83 Avg: 52			PhosphoSorb Media – testing at Cable Street StormFilter (total phosphorus).

Exhibit 3. Lake Whatcom Phosphorus-Reduction: Cost-Benefit Analysis Detail

Activity Number	Activity	Units of Phosphorus (TP) Removal	Phosphorus Loading Removed	Cost	Cost/Benefit	Notes
	StormFilter® with ZPG media	%	-24-79 Avg: 28	Construction Cost: $0.52 to 1.31 per sf of treated impervious. Maintenance Cost: $0.06 to $0.25 per year per sf of treated impervious surface area.	8974 lb/year P loading watershed-wide, assuming 28% removal of phosphorus; Watershed-wide implementation reduces phosphorus by 2,812lbs and costs $1,350,000,000 at $1.10/sf, for a cost/benefit for the first year only of approximately $591,151/lb	ZPG Media – testing at Cable Street StormFilter (total phosphorus). Cost: Request quotes from manufacturers
		lb/filter/year	0.02 – 3.16 Avg: 1.04			2009/2010 City data: "Filter" (11 total) (Bellingham 2011).
		lb/filter/year	0.02 – 1.47 Avg: 0.49			2009/2010 City data: "Stormfilter" (subset of "Filter" – 4 total) (Bellingham 2011). Electric Ave (two values reported), Poplar (WQF 132), Poplar (WQF 133)
		lb/filter/year	0.00 – 4.69 Avg: 1.11			2009/2010 City data: "Enhanced Filter" (4 total) (Bellingham 2011). E. Beachview, Silvern Lane, Alabama Vault, Lakeside Vault.
	StormCeptor with Imbrium	lb/acre/year	0.21	Construction Cost: $1.50 per sf of treated impervious. Maintenance Cost: $0.25 per year sf of treated impervious surface area.	For the first year, 0.21 lb/acre removed with $1.50/sf ($65,340/acre) is $311,142/lb	Enhanced settling technology: Imbrium – Stormcepter: 20% removal efficiency and 1.07 lb/acre treated/year pre-treatment loading assumed. The Imbrium testing reports: "The Stormceptor can remove approximately 20-30% of the Total Phosphorus from influent stormwater (Madison, Wisconsin study; Como Park, Minnesota study)." Cost: Request quotes from manufacturers
14	**Sizing ditches and culverts to eliminate erosion**	lb/culvert/year	*Minimal*	--	--	Standard circular culverts replaced with flat bottom culverts to eliminate exit scour (erosion at the culvert outlet). Very little literature exists quantifying the TP reduction attributable to stream channel stabilization. Long-term studies performed at Lake Tahoe indicate that shoreline disturbance contributes 4% of the lake's phosphorus load while stream channel erosion of stream channels draining to the lake contribute 2% of the lake's phosphorus load (California Regional Water Quality Control Board 2010). Thus non-urban erosion at Lake Tahoe contributes only a minor portion of the phosphorus load. It should be pointed out that Lake Tahoe has a considerably different soils and geology regime than Lake Whatcom, making a direct comparison tentative. Therefore, it is estimated that a minimal to moderate phosphorus loading benefit from this activity could be expected only if this is applied consistently over the entire watershed.
15	**Street sweeping**	%	60 (street) 10 (basin)	$6,000 per lane mile (per year)	$6000 per lane mile = $0.11 per sf;	Shoemaker (2000) states that 40-75% of street-related TP can be collected by mechanical and vacuum-assisted sweepers, respectively. However, on a watershed basis, USGS (2002) reports a TP reduction of 5-14% while Northern Virginia Planning District Commission reports a 9-11% reduction. The City of Seattle conducted extensive street sweeper testing (Seattle Public Utilities and Herrera Consultants 2009) but did not report phosphorus data. The City of Bellingham conducted a single analysis of phosphorus content of sweepings but the data was insufficient to allow calculation of removal rate. Assign the following TP reduction credit: 60% for the street area; 10% on a basin-wide basis. Cost: *City provided $6,000 per lane mile per year*

Exhibit 3. Lake Whatcom Phosphorus-Reduction: Cost-Benefit Analysis Detail

Activity Number	Activity	Units of Phosphorus (TP) Removal	Phosphorus Loading Removed	Cost	Cost/Benefit	Notes
16	**Controlling erosion through 1) streambank improvements and 2) restoring stream buffer vegetation**		*Moderate benefit*	--	--	The benefit to phosphorus load reductions from stream buffer protection is attributed to erosion control; however, stream buffer vegetation also contributes phosphorus by export of organic material. Note: Phosphorus loading associated with streambank erosion depends on the TSS/TP correlation (soil composition) and local hydrology regime. Restoring and preserving the stream buffer's vegetation would involve enforcement of the City's designated stream buffer widths. As with item #14 (culvert sizing to control erosion), the maximum phosphorus loading benefit from this activity could be expected only if this is applied consistently over the entire watershed. Available for farming land, but limited data available for urban areas. See notes for item #14. Excerpt from literature: "Although buffer zone vegetation reduces erosion, it is not considered effective for the removal of phosphorus over the long term because phosphorus retained by plants in the spring and summer is released with plant senescence in the fall. Therefore, lakeside residents have been asked to circumvent this natural recycling by collecting beach debris and cutting, harvesting, and removing excess buffer zone vegetation two to three times per year as suggested by Dillaha et al. _1986_. Measurements indicate that typical shoreline debris material has a water content of about 75% and contains about 0.25% phosphorus by dry weight. Therefore, a total phosphorus loading reduction of about 70 kg/year could be attained if each lakeside property owner removed 225 kg of vegetative litter and beach debris _wet weight_ from their property per year." (Canale, RP; Redder, T; Swiecki; W, Whelan, G. Phosphorus Budget and Remediation Plan for Big Platte Lake, Michigan. *Journal of Water Resources Planning and Management.* 2010, 136 (5), 576-586). Cost: Cost would be equal to the total cost of planning, design, construction and planting, and maintenance until the site is self-sufficient, based on research results (reference in the region).
17	**Regulations: Phosphorus fertilizer ban**	lb/acre/year	0.5	$0	--	A number of articles review general lawn phosphorus contribution but none of them identify numeric contributions. Vadas (2008) documents very high phosphorus losses if heavy rainfall occurs within a few days of fertilization. Lehman (2009) documents a 28% reduction in TP concentrations in a river following a ban on fertilizers. Gross (1990) and Erickson et al. (2005) measured measured phosphorus losses due to leaching caused by infiltrating rainfall from grassed areas. Both reported minimal phosphorus losses due to surface runoff because the sandy soils used in these studies had very high rates of infiltration and produced very little surface runoff. Easton (2004) also reported that the majority of phosphorus loss observed from lawn plots was due to leaching. However, his study also reported substantial surface runoff from the lawn plots and an associated phosphorus loss of 0.5-1.0 kg/ha/yr (0.5-0.9 lbs/acre/year). Unfertilized lawns can also contribute soil-bound phosphorus to surface runoff. Given the uncertainty of the contribution of soil-associated phosphorus in the Lake Whatcom Watershed and the very limited data on lawn-generated phosphorus loadings, a unit-load reduction due to the fertilizer ban cannot be reliably estimated at this time. A conservative estimate of 0.5 lbs TP/acre/yr of lawn area may be justified. Cost: assumes no cost for annual enforcement.

Exhibit 3. Lake Whatcom Phosphorus-Reduction: Cost-Benefit Analysis Detail

Activity Number	Activity	Units of Phosphorus (TP) Removal	Phosphorus Loading Removed	Cost	Cost/Benefit	Notes
18	**Education: Watershed signs**	-	-	-	-	See Exhibit 4: Education, Incentives and Enforcement Matrix.
19	**Education: Mass mailings**	-	-	-	-	
20	**Education: Online information**	-	-	-	-	
21	**Education: Newspaper ads**	-	-	-	-	
22	**Education: Video presentations**	-	-	-	-	
23	**Education: Community events (public meetings)**	-	-	-	-	
24	**Education: Onsite training/workshops**	-	-	-	-	
25	**Education: Resident contact**	-	-	-	-	
26	**Education: Project consultation**	-	-	-	-	
27	**Incentives**	-	-	-	-	
28	**Transition from Department of Ecology Water Quality Assurances of Forest Practices to 'Pristine', pre-development conditions**	lb/acre/year	0.10	$200,000 (Administrative only, no capital)	For the first year, 2,480 lbs reduction for $200,000 is $80.65/lb	Phosphorus reduction is equal to the difference between loading from pristine forested land and forested land that meets forest practice conditions. Forest loading is 0.15 lbs/acre/year (see item #1). 'Pristine' forest loading assumed to be 0.05 lbs/acre/year, so the benefit is 0.15-0.05 = 0.10 lbs/acre/yr. At base (2003) conditions, 88% of 31,183.9 acre watershed is in wetland or forest cover, so the total (watershed-wide) net benefit is 2,480 lbs. (note: TP removal stated as lb/acre/year applies only to acres under forest/wetland land uses). Assume 2 FTEs for 1 year, with 1 FTE per year at $100,000
29	**Design standards for new and retrofitted roads**	%	60	$1.48 per sf of treated impervious surface.	8974 lb/year P loading watershed-wide, assuming 60% removal of phosphorus; Watershed-wide implementation reduces phosphorus by 5,384 lbs and costs $1,350,000,000 at $1.48/sf, for a cost/benefit for the first year only of approximately $371,171/lb	Narrower roads constructed under new design standards would produce less runoff than conventional road widths, but it is not clear that reduced runoff would result in reduced TP loads. Use of pervious pavement for new and retrofitted roads could reduce TP by that shown in item #9: 60% TP reduction credit (pervious pavement area, only). Pervious-paved parking lanes might achieve a similar reduction if travel lane runoff was directed across the parking lanes.
30	**Reconfigure roadside ditches**	%	20	$8 /sf of swale	8974 lb/year P loading watershed-wide, assuming 20% removal of phosphorus; Watershed-wide implementation reduces phosphorus by 1,795 lbs and costs $10,802,880,000 at $8/sf, for a cost/benefit for the first year only of approximately $6,000,000/lb	Most roadside ditches could likely be reconstructed into biofiltration swales. This could reduce TP by that shown in item #3 (Biofiltration Swales): 20% TP reduction credit. Cost: see item #3 (Biofiltration Swales). Cost assumed similar

Exhibit 3. Lake Whatcom Phosphorus-Reduction: Cost-Benefit Analysis Detail

Activity Number	Activity	Units of Phosphorus (TP) Removal	Phosphorus Loading Removed	Cost	Cost/Benefit	Notes
31	**Reconfigure streets**	%	20-40	$10/sf of street	8974 lb/year P loading watershed-wide, assuming 30% removal of phosphorus; Watershed-wide implementation reduces phosphorus by 2,692 lbs and costs $12,801,000,000 at $9.48/sf, for a cost/benefit for the first year only of approximately $4,755,000/lb	This approach would result in narrow paved streets, on the order of 22-26 feet in width. The remainder of the street ROW would be largely devoted to bioretention for street slopes less than 3%. For streets with slopes ranging from 3-10%, biofiltration swales with check dams and other measures would be installed to enhance infiltration. The City of Seattle has more than 10 years of experience with this approach through its nationally-known SEA Streets Project and subsequent street edge projects (Seattle Public Utilities 2011). The enhanced biofiltration swales on the moderately-sloped streets could be expected to readily achieve the phosphorus reduction identified under Reduction Measure #3: Biofiltration Swales – 20%. The bioretention areas installed along the gently sloped streets would likely experience relatively higher runoffs than the typical bioretention facilities discussed under Reduction Measure #4: Bioretention. Therefore the assigned TP reduction credit is reduced from 50% to 40% for street bioretention. Prior to installing bioretention areas in a public ROW, it is important that the infiltration rates and water table conditions of the local subsoils be documented. Otherwise seasonally high water table levels could result in undesirable ponding or local seepage onto adjacent properties. Cost: equal to the sum of the costs of items #3 and #29
32	**Vehicle trips - reduce and redirect**	n/a	Minimal direct benefit	--	--	Vehicle trip reduction via increased transit and bike. Vehicles by themselves are not expected to contribute a substantial amount of phosphorus loading to runoff. Therefore, this item will be characterized in the main text rather than quantified.
33	**Recreational facility design and use (Improving existing facilities)**	lb/acre/year	Minimal direct benefit	--	--	Assumes there would be a net phosphorus decrease from reduction of shoreline erosion and elimination of fertilizer. However, no numeric estimate is made of phosphorus reduction or avoidance for this reduction measure. Of the four recreational facilities with Lake Whatcom shoreline, Bloedel-Donovan Park has the most developed land area and supports the most intensive shoreline recreation. The remaining three parks have limited road and trail access to the lake shore but no shoreline development, are mostly forested, and could be expected to contribute minimal TP loading. See Item #33 – Figure 1.
34	**Watershed-wide enforcement**	-	-	-	-	See Exhibit 4: Education, Incentives and Enforcement Matrix.
35	**Animal waste: wildlife (goose)**	lb/goose/year	0.25	--	--	Unit estimates of TP production (lb TP/goose/year): Sherer et al. (1995): 1.2 Manny (1975): 0.35 Kear (1963): 1.4 The middle value of 1.2 lb TP/goose/year is assigned. For resident goose, the majority of the food eaten by a goose is likely to come directly from the watershed. Therefore the net new phosphorus produced by a goose is assumed to be 20%. Thus the TP contribution of is assumed to be 0.2 x 1.2 = 0.25 lb TP/goose/year. No estimate of the Lake Whatcom goose population has been found. Cost: an order-of-magnitude estimate may be available by researching other jurisdictions with goose control programs (or restoration sites and treatment wetland facilities with such a program). Cost estimate would be a general, annual programmatic cost.

Exhibit 3. Lake Whatcom Phosphorus-Reduction: Cost-Benefit Analysis Detail

Activity Number	Activity	Units of Phosphorus (TP) Removal	Phosphorus Loading Removed	Cost	Cost/Benefit	Notes
36	**Septic system transition to sewer connection**	lb/septic system/year	0.6	$50,000 per septic system ($50,000 x 92 septic systems = $4,600,000)	0.6 lb/septic system x 92 septic systems = 55 lb total at $4,600,000 is $83,333/lb (for the first year)	Table 12 of the Soil Conservation Service report for Whatcom County indicated that virtually all of the shoreline soils had severe limitations for septic systems. The subset of soils used for this analysis was limited to those soils that were identified as having a shallow (three feet or less) depth to seasonal bedrock, hardpan or water table, or as being subject to seasonal flooding. These are the conditions that would promote phosphorus migration. According to a visual inspection of the known septic systems mapped in the Lake Whatcom 2008 Stormwater Comprehensive Plan (CH2M HILL 2008), there are approximately 92 septic systems within 150 feet of the lake shoreline located in soils susceptible to phosphorus leaching. These were assumed to be capable of leaching TP to Lake Whatcom. Data from EPA (2002) indicate that for the typical septic systems 0.8 lb TP/person/year reaches the leach field. Assuming three persons per house, leach field phosphorus was calculated to be 3 x 0.8 = 2.4 lbs/house/year. Using a midpoint value of leaching phosphorus from Dudley and May (2007) of 25%, it is assumed that one-quarter of leach field phosphorus reaches the lake. 0.25 x 2.4 yields an estimate of 0.6 lb/year of phosphorus loading to the lake for each of the 92 identified houses. The total estimated phosphorus loading benefit from converting these homes to sewered is therefore equal to 92 septic systems x 0.6 lb/septic system/year. This loading estimate is somewhat speculative but it provides an order of magnitude of the potential lake phosphorus loading attributable to septic systems. [a] Cost (lb/septic/year): research with other jurisdictions the total administrative and capital cost to transition one SFR from septic to sewer. Cost estimate not to include annual sewer operations. [b] Cost: equal to the cost estimate in ([a]) multiplied by 92 septic systems.
		lb/year (watershed-wide)	55.2			

Notes:

[1]definitions of the terms biofiltration, bioretention, and infiltration, in the context of this study:

Infiltration: water percolates into the ground and does not re-enter the surface water flows; mechanism of phosphorus reduction is stormwater volume reduction (activities in this study: dry wells, infiltration trenches, pervious pavement, and infiltration basin (activities 7, 8, 9, and 10, respectively)

Biofiltration: Mechanism of phosphorus reduction is plant uptake of pollutants, assumes no infiltration (activities in this study: vegetated swales (activity 3))

Bioretention: Mechanisms of phosphorus reduction are both plant uptake of pollutants and some infiltration (activities in this study: rain gardens (activity 4), street trees (activity 5), and lawn replacement (activity 6))

Exhibit 4. Education, Incentives and Enforcement Matrix

		Reference																Average % [1]	Relative Estimated Effectiveness (by Category)				
	Activity	a	a	a	b	c	d	e	f	g	h	h	i	j	k	l			Pet Waste	Fertilizer	Car Washing	SW Mgmt on Private Property	Septic System Maintenance
Category	Performance Measure	% effectiveness	% would change behavior	% followed advice of	% in favor	% in favor	% noticed	% noticed	% noticed	% influenced by	% supportive of	% motivated by	% interested	% interested	% made changes to yard care habits	% very or somewhat							
EDUCATION	Watershed interpretive signs					2	4					1					2		Very Low	Very Low	Very Low	Very Low	Very Low
	Mass mailings	55		50	10	31			4	55	49	12	23	46			34		Moderate	Moderate	Moderate	Moderate	Moderate
	Online information (website)	45			13		3						8				17		Low	Low	Low	Low	Low
	Newspaper ads	56				15	33		38	46	66	36	12				38		Moderate	Moderate	Moderate	Moderate	Moderate
	Video presentations (or TV ads)	56.9	19			19	71	66	28	43		27	15		52.4		40		Moderate	Moderate	Moderate	Moderate	Moderate
	Community events (public meetings)					2	0				75	4		26			21		Low	Low	Low	Low	Low
	Resident contact (home visit)										49			18			34		Moderate	Moderate	Moderate	Moderate	Moderate
	Onsite training/workshops	39								49					23.5	35	37		n/a	Moderate	n/a	Moderate	Moderate
	Technical assistance	45									67						56		n/a	High	n/a	High	High
INCENTIVES	Convenient disposal		17														17		Low	n/a	n/a	n/a	n/a
	Store coupons										82			54			68		n/a	High	n/a	High	n/a
	Yard waste pickup													48			48		n/a	Moderate	n/a	Moderate	n/a
	Rain barrel													44			44		n/a	Moderate	n/a	Moderate	n/a
	Food waste pickup													37			37		n/a	Moderate	n/a	Moderate	n/a
	Compost bin													28			28		n/a	Low	n/a	Low	n/a
ENFORCEMENT	Watershed-wide		7								58						33		Moderate	Moderate (*Note: No literature found on effectiveness of enforcement actions on fertilizer use. 'Moderate' effectiveness based on surrogate described below.*)	Moderate	Moderate	Moderate
																			Notes for this Category: ***0.3 lb TP/dog/year;*** *A dog averages 1/3 pound of waste per day with a phosphorus content of 1% (Carrasco 2003). A dog therefore produces up to 1.2 pounds of phosphorus per year. Conservatively assume that all waste remains on the ground and that up to 25% of the phosphorus reaches the lake via surface runoff. Thus, 1.2 lb/dog/year x 25% = 0.3 lb/dog/year TP reduction benefit with pet waste elimination. An estimate of the number of dogs in the watershed could yield an estimate for the whole watershed.*	Notes for this Category: ***0.5 lb/acre/year*** *- See Table 1 'phosphorus fertilizer ban' - estimate of loading from fertilizer use. A moderate estimated effectiveness is assumed based on available literature reviewed to-date.*	Notes for this Category: *No information found in the literature on P-loading from car washes. Phosphorus may be phased out of detergents and car wash soaps. Effective enforcement is not likely to be feasible; incentives such as car wash coupons have been found somewhat popular, and are likely to be more effective than education alone because cost is a main reason cited by survey participants (Silver Beach Creek - reference j) as the reason for washing vehicles at home instead of a commercial car wash.*	Notes for this Category: ***0.62 lb/acre/year;*** *Assumes a loading rate from conventionally-developed SFRs of 1.24 lb/acre/year, and 50% TP removal from retrofits (see Table 1, Bioretention). Thus, the TP reduction benefit is equal to 0.62 lb/acre/year. Could assume that 30% of the developed area would potentially be retro-fitted to compliance with City LID standards.*	Notes for this Category: *To calculate the TP reduction benefit, need a loading rate from failing septics - not found in the literature to-date. The loading rate from properly maintained septics within 150 feet of the lake shoreline is estimated to be* ***0.6 lb/septic system/year*** *(see Table 1, Septic System Transition to Sewer).*

[1] **These results are intended to be used for discussion only due to the widely varying nature and purpose of the supporting studies/references.**

Reference

a	Center for Watershed Protection. 1999. A Survey of Residential Nutrient Behavior in the Chesapeake Bay. Ellicott City, Maryland. July 1999.
b	Cascadia Consulting Group, Inc. 2009. Evaluation of Seattle Public Utilities' Public Involvement and Education Programs. Prepared for Seattle Public Utilities. Seattle, Washington. February 2009.
c	Fairbank, Maslin, Maullin & Associates. 2001. San Mateo Countywide Stormwater Pollution Prevention Program: 2001 Residential Survey. June 2001.
d	Pelegrin Research. 2004. Storm Water Public Education Program: Resident Population Telephone Survey – 2004 Evaluation and Next Steps. Prepared for [California] State Water Resources Control Board and Rogers and Associates. Los Angeles, California. October 2004.
e	Maine Department of Environmental Protection. 2005. Assessment of Maine's Stormwater Phase II & NPS Outreach Campaign: 2003/2004. February 2005.
f	Evans/McDonough Company, Inc. 2003. Evaluation of Watershed Watch Campaign Effectiveness: 2003 Public Opinion Survey and Focus Groups. Prepared for Santa Clara Valley Urban Runoff Pollution Prevention Program. Oakland, California. November 2003.
g	Rockwood Research, University of Minnesota Extension Service. 1999. Evaluation of Spring 1999 Stormwater Media Outreach: Think Clean Water Campaign. Twin Cities, Minnesota.
h	Riley Research Associates. 2002. Do Clean Rivers Begin at Home? Exploring the Obstacles and Motivations of Homeowner Behavior: A Survey of Public Habits in Oregon's Tualatin River Watershed. Prepared for Oregon Clean Water Services and Tualatin Basin Public Awareness Committee.
i	Gilmore Research Group. 2005. Surface Water Study [telephone survey of Bellevue, Redmond, and Shoreline residents].
j	Silver Beach Creek Outreach Program. Final Report. Accessed at: http://www.cob.org/documents/pw/lw/psp-final-report.pdf
k	Cascadia Consulting Group, Inc. and Applied Research Northwest. 2008. Yard Care and Water Quality Study: A Telephone Survey of Lake Whatcom and Samish Watershed Residents. Prepared for Whatcom County Public Works. June 2008.
l	Applied Research Northwest and Cascadia Consulting Group, Inc. 2008. Final Report: City of Bellingham Water Conservation Survey. Prepared for the City of Bellingham, Washington. January 2008.

Appendix C
Whatcom Falls Water Treatment Plant Dissolved Air Flotation Pilot Testing Report

Final Report

Whatcom Falls Water Treatment Plant Dissolved Air Flotation Pilot Testing

City Project Number: EW-180

Prepared for

City of Bellingham, WA

January 2012

CH2MHILL®

21 Bellwether Way, Unit 111
Bellingham, WA 98225

Final Report

Whatcom Falls Water Treatment Plant Dissolved Air Flotation Pilot Testing

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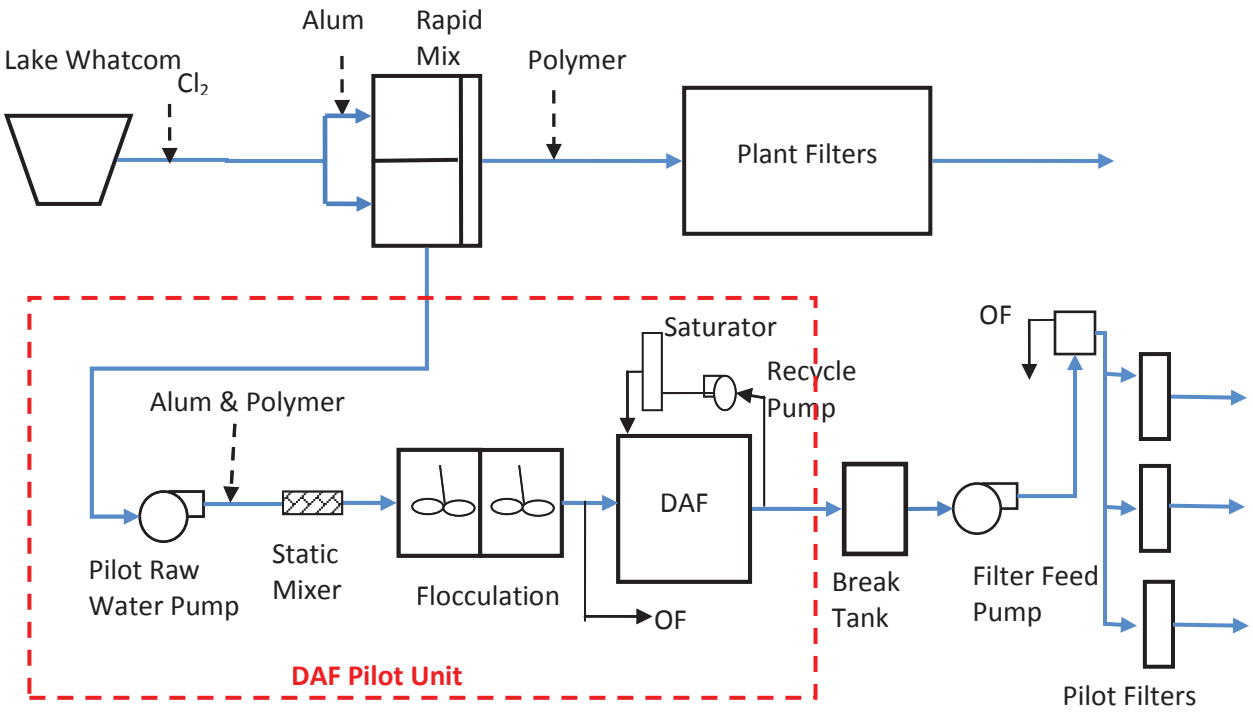
Executive Summary

The City of Bellingham (City) operates the Whatcom Falls Water Treatment Plant (WTP). This in-line filtration plant has a peak capacity of 29 million gallon per day (MGD) when all six filters are on line and a firm capacity of 24 MGD when one filter is off line for backwashing or maintenance. The source water is Lake Whatcom, a large natural lake that in recent years has seen increases in algal counts that have affected the performance of the WTP during late summer. A 6-week pilot testing of dissolved air flotation (DAF) was conducted at the WTP in the late summer of 2011. The goal of the testing program was to evaluate the performance of DAF with respect to how it improves overall WTP performance and capacity during summertime algae conditions.

Figure ES-1 shows a schematic of the DAF pilot system (including pumping, mixing, and flocculation) within the dashed-red border, how it was connected to the City's WTP, and its position upstream of the City's pilot filters. The City's WTP pilot filters were used as a key performance measure of the beneficial impact of the DAF process.

FIGURE ES-1
Schematic of DAF Pilot Unit



DAF loading rates were varied from 10 gallon per minute per square foot (gpm/sf) to 20 gpm/sf over the course of the testing to evaluate the performance of DAF at ever-increasing loading rates with the understanding that successful performance at higher loading rates can lead to reduced DAF construction cost. Table ES-1 summarizes the DAF runs completed during pilot testing.

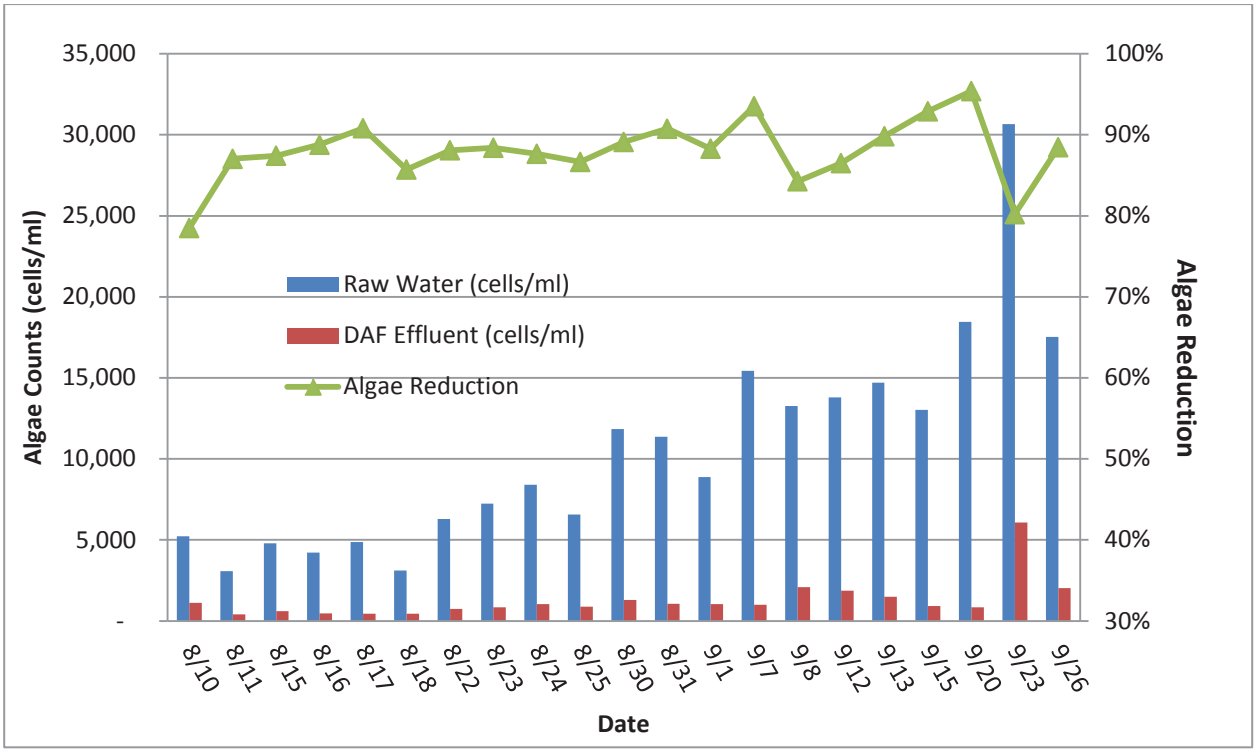
The DAF pilot test met all of the key goals and objectives established in the testing plan, which was developed prior to the pilot test, and which was approved by Washington State Department of Health (DOH). The pilot test demonstrated that for the Lake Whatcom supply, DAF effectively removed algae, increased filter production, reduced total organic carbon (TOC) and color, and reduced the formation potential for total trihalomethanes (TTHMs). While DAF's demonstrated beneficial impact to the treatment process was widespread among the parameters listed above, the two most critical and defining parameters are algae count reduction and improved filter performance.

TABLE ES-1
DAF Pilot Test Runs and Conditions

Week	Objective	Floc Time (min)	Alum Dose (mg/L)	Polymer Dose (mg/L)	DAF Loading (gpm/sf)	Pilot Filters Loading (gpm/sf)
1	10 gpm/sf DAF loading rate. No polymer	5	10	-	10	5, 6 & 7
2	10 gpm/sf DAF loading rate. No polymer	5	10	-	10	5, 6 & 7
3	10 gpm/sf DAF loading rate. With polymer	5	10	0.1-0.35	10	5, 6 & 7
4	14 gpm/sf DAF loading rate. With polymer	5	11	0.3	14	5, 6 & 7
5	16 gpm/sf DAF loading rate. With polymer	4.4	11	0.3	16	5, 6 & 7
6	20 gpm/sf DAF loading rate. With polymer	8	10	0.4	20	5, 6 & 7
7	Repeat Optimal Testing Run (16 gpm/sf DAF loading rate)	4.4	10	0.2-0.3	16	5, 6 & 7

First, the algae-count reduction achieved by the DAF process is presented in Figure ES-1 (same figure as Figure 4-2). These algae reduction results were encouraging given that, while the algae population in Lake Whatcom is effective at clogging the City's WTP filters, its total algae counts are relatively low when compared to other algae-laden waters. Reducing the amount of algae in the raw water is important because it enables improved treatment performance of filtration, disinfection, and other processes downstream.

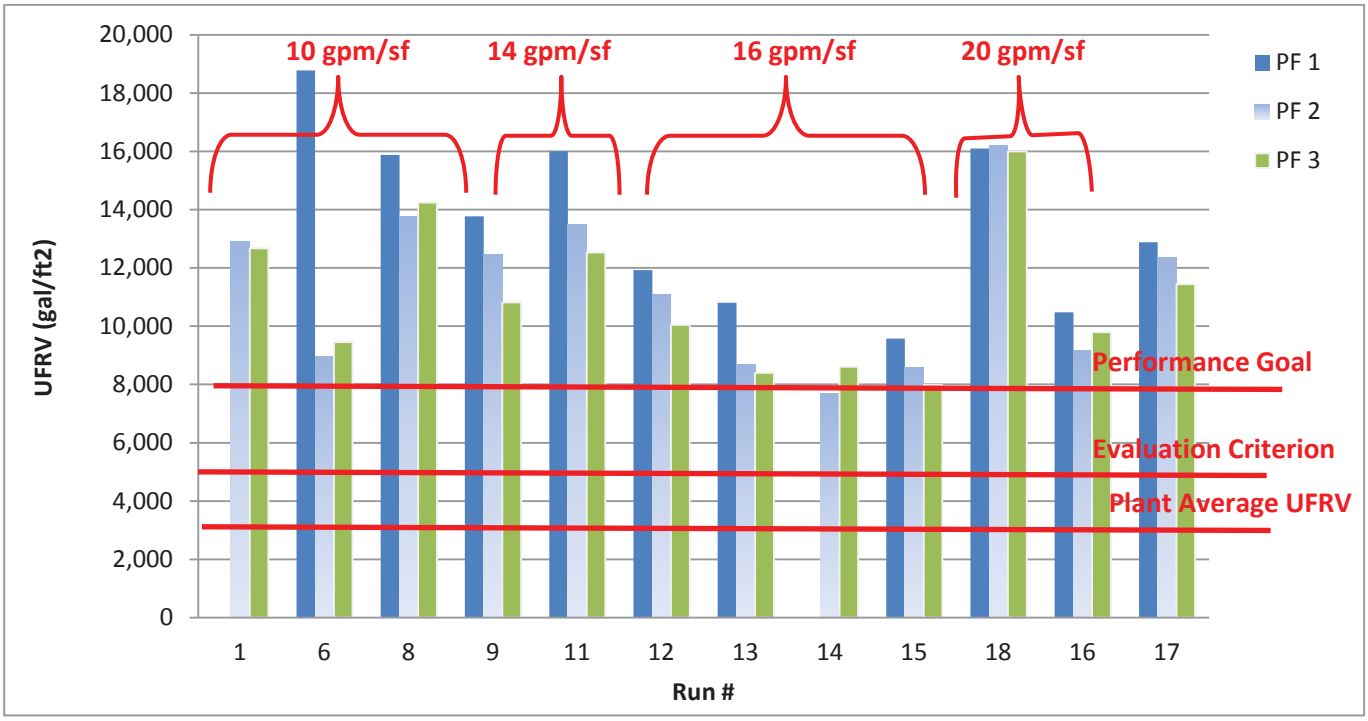
FIGURE ES-1
Algae Counts and Reduction



Second, even more important than the algae reduction results are the pilot filtration performance results. It is important to keep in mind that the specific reason for evaluating DAF pretreatment is the dramatically reduced performance of the City's WTP filters during summertime algae blooms. The most common measure of filter performance used under these circumstances is unit filter run volume (UFRV), which is a measure of the volume of

water passed through a square foot of filter area before the filter needs to be backwashed. Lake Whatcom blue-green algae, even at relatively low counts, severely reduces UFRV at the City's WTP filters to roughly 3,000 gallons per minute per square foot (gpm/ft^2) – down from roughly 7,000 to 9,000 gpm/ft^2 at other "peak-UFRV times" of the year. As presented in Figure ES-2, the pilot test results showed that DAF increased UFRV in the pilot filters to above 8,000 gpm/ft^2 (the performance goal for pilot test), which is more than double what the City's full-scale WTP filters were able to do during the same DAF pilot test period. This UFRV result is the single-most defining parameter demonstrating the success of the DAF process at removing algae from the Lake Whatcom supply and improving performance of the City's WTP.

FIGURE ES-2
Pilot Filter UFRV in Each Pilot Test Run



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B Process Instrumentation Diagrams of Pilot Flocculation and DAF System by Roberts
C Weekly Pilot Test Raw Data and Lab Analysis Results (available electronically by request)

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SECTION 1

Introduction

1.1 Purpose and Scope

The City of Bellingham (City) operates the Whatcom Falls Water Treatment Plant (WTP). This in-line filtration plant has a peak capacity of 29 million gallon per day (MGD) when all six filters are on line and a firm capacity of 24 MGD when one filter is off line for backwashing or maintenance. The source water is Lake Whatcom, a large natural lake that in recent years has seen increases in algal counts that have affected the performance of the WTP during late summer. A 6-week pilot testing of dissolved air flotation (DAF) followed by filtration was conducted at the WTP in summer of 2011. The goal of the testing program was to evaluate the performance of the DAF, specifically in terms of algae reduction and improvement in filtration productivity.

The original testing protocol for the pilot testing is described in the memorandum "Whatcom Falls Water Treatment Plant: Pilot Testing Plan for Dissolved Air Flotation" (CH2M HILL, June 2011; Attachment A). The testing program was designed to:

1. Establish ability of DAF to effectively remove algae prior to filtration
2. Establish coagulant and polymer dosage rates required with DAF
3. Determine impacts of DAF pretreatment on filtration performance
4. Monitor other water quality parameters in filtered water

1.2 Organization of Document

This report summarizes the flocculation/DAF and filtration pilot testing conducted for the Whatcom Falls WTP. The remainder of this report is divided into the following sections:

- **Section 2, Testing Methodology**, includes methodology, descriptions, and layouts of the pilot unit processes, equipment used, and sampling analysis/frequency.
- **Section 3, Testing Runs and Conditions**, describes all the testing runs completed during the testing period.
- **Section 4, Testing Results**, evaluates the key items of interest based on pilot test results as compared to goals or regulatory limits.
- **Section 5, Conclusions**, summarizes the pilot testing results and describes how they relate to the proposed processes and design criteria.

SECTION 2

Background

2.1 Raw Water Supply

The raw water source of the Whatcom Falls WTP is Lake Whatcom. Raw water is taken from the lake and pre-treated through travelling screens at the Screen House. A small dose of chlorine (0.6 mg/L) is added to raw water at the Screen House from which water is conveyed via a 66-inch pipe to the WTP. The plant consists of two rapid mix basins and six dual-media filters. Normally only one rapid mix basin is on line. The plant uses Alum as a coagulant along with polymer, both of which can be added at multiple locations prior to and at the rapid mix basins.

Historical raw water characteristics of the Whatcom Falls WTP are presented in Table 2-1. The data show that this water has low turbidity, hardness, alkalinity, color, and metals. Total organic carbon (TOC) concentrations are low and consist primarily of dissolved organics (DOC).

TABLE 2-1
Whatcom Falls WTP Historical Raw Water Quality (2007-2010)

Parameter	Units	Minimum	Average	Maximum
Temperature	Celsius	6	12	18
Turbidity	NTU	0.41	0.74	2
Alkalinity	mg/L as CaCO3	19.5	20.7	22.5
Hardness	mg/L as CaCO3	17.3	21.2	23
pH	S.U.	7.2	7.3	7.4
Conductivity	umohs/cm	57	60.6	75
Apparent Color	PtCo	13	14	15
TOC	mg/L	1.8	2.2	2.6
DOC	mg/L	1.8	2.1	2.3
UV254	1/cm	0.046	0.056	0.103
Iron	mg/L	<0.01	-	0.08
Manganese	mg/L	<0.001	-	0.012
Aluminum	mg/L	<0.010	0.06	0.098
Chloride	mg/L	<2	2.2	3
Sodium	mg/L	2	4.4	5
Sulfate	mg/L	3.6	7.4	10
Chlorophyll	µg/L	2	3.5	5.9
Algae [a]	#/ml	-	-	100,000

[a] Estimated algae counts based on historical algae counts

Raw water used at the pilot test was drawn from the rapid mix basin no. 1 via the basin drain line. This rapid mix basin was filled initially with raw water without any chemical addition. The inlet valve to the rapid mix basin was then adjusted to obtain a fill rate close to the drain rate so that the basin water surface was maintained relatively constant. The basin water surface level was also monitored and interlocked with the plant control system throughout the pilot testing to avoid the rapid mix basin overflow. Typically, the level in rapid mix basin no. 1 was a few inches

higher than the level in rapid mix basin no. 2 and the downstream filter influent flume to prevent backflow of plant treated water into the pilot plant raw water.

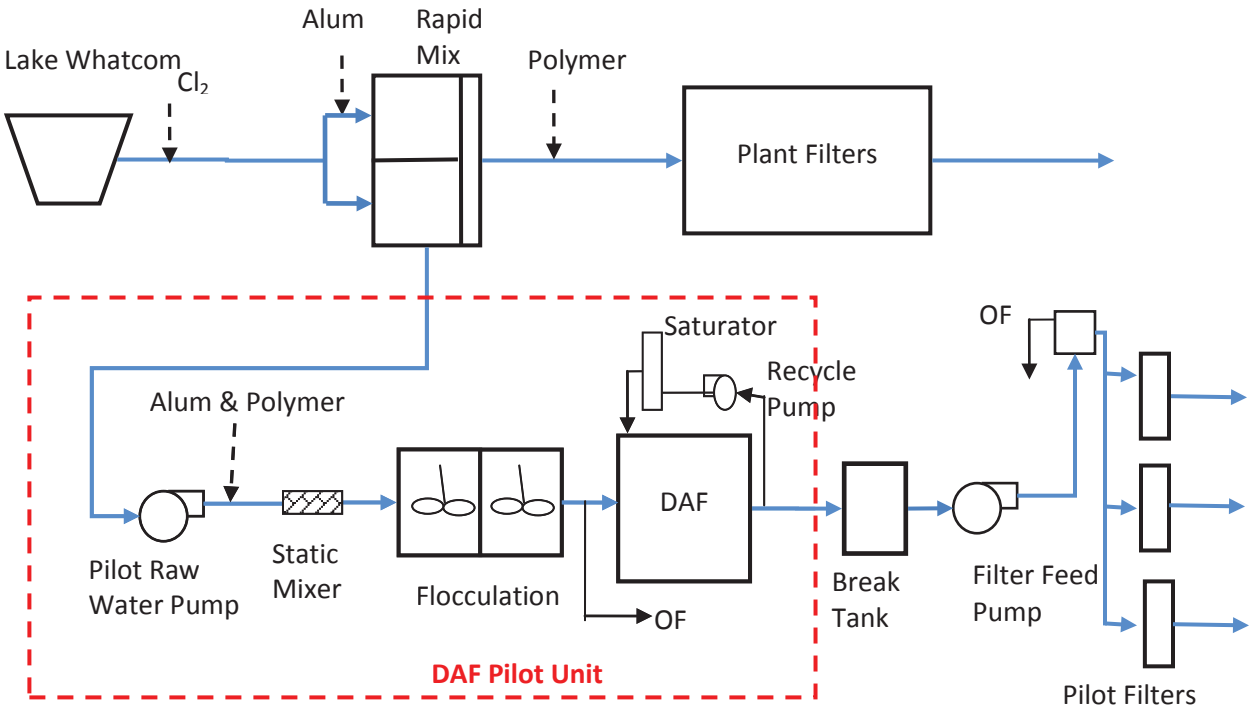
2.2 Pilot Unit Processes

The pilot testing setup includes the following key components:

- Roberts Flocculation/DAF pilot trailer with influent pumping and chemical feed systems
- The City's existing pilot filters and filter influent pumping

The schematic of the process units (Figure 2-1) shows the key elements of the WTP and the DAF pilot system. Figure 2-1 also shows where the DAF pilot system connected to the WTP.

FIGURE 2-1
Schematic of DAF Pilot Unit



2.2.1 Flocculation/ Dissolved Air Flotation Pilot Trailer

The flocculation and DAF pilot units were provided in a trailer by Roberts Water Technologies and Enpure (Roberts). The trailer also houses an influent pump, flow control valves, chemical feed system, on-line water quality analyzers, and system control panel. Figure 2-2 shows the inside and outside of the trailer.

FIGURE 2-2 (a)
Inside of Pilot Trailer

FIGURE 2-2 (b)
Outside of Pilot Trailer

2.2.1.1 Raw Water Pumping

A constant speed pump, a flow meter, and flow control valve assembly were used to deliver the desired raw water flow from the plant rapid mix basin to the pilot flocculation tanks. Three metering pumps and chemical day tanks could deliver three different chemicals to the raw water line, followed by an in-line static mixer. Raw water and DAF effluent pH, turbidity, and UV254 were continuously monitored at 1-minute intervals using the on-line analyzers.

2.2.1.2 Flocculation Unit

Two 2-stage flocculation basins are provided at the top of the trailer. Each stage has a dimension of 24 inches long by 36 inches wide by 42 inches deep. Each stage is equipped with a vertical mixer. The speed of each individual mixer can be controlled to up to 342 rpm via the pilot system control panel. During the testing, one or both flocculation basins have been put in service depending on the flocculation time requirement. Attachment B includes the process and instrumentation diagram (P&ID) of the flocculation system.

2.2.1.3 Dissolved Air Flotation Unit

Flocculated water gravity flows to the DAF unit through a baffled entrance. A flow meter and flow control valve assembly controls the desired flow rate to DAF. Excess flow overflows via a drainpipe to the City sewer system. The DAF unit consists of an air saturator and compressor, recycle pumping, and a DAF tank with sludge removal and effluent collection capability. The DAF tank has a flotation surface area of approximately 4 square feet. The recycle pump uses between 6 and 12 percent of the DAF effluent and pumps it to a pressurized vessel called saturator, where the water is supersaturated with air. The saturator receives compressed air. It is pressurized at 60 to 80 psi to saturate air into the water solution. This air-saturated water is then recycled back to the head of the tank. During the pilot testing, the recycle rate was optimized and held between 10 and 12 percent by switching tank nozzles in the contact zone when raw water flow changes were made.

The air in the water comes partially out of solution and floats upwards, carrying flocculated particles with it. The particles rise all along the length of the tank and accumulated at the top, where they are removed by periodic hydraulic de-sludge. The underflow from the DAF tank is the treated effluent that is then sent to filtration.

The system is set up to run automatically. All operating parameters were available for display and control via the control panel inside the trailer. Figure 2-3 shows additional pictures of the DAF components. Attachment B includes the P&ID of DAF by Roberts.

FIGURE 2-3 (a)
DAF Effluent End

FIGURE 2-3 (b)
Floc Float on Top of DAF

2.2.2 Pilot Filters

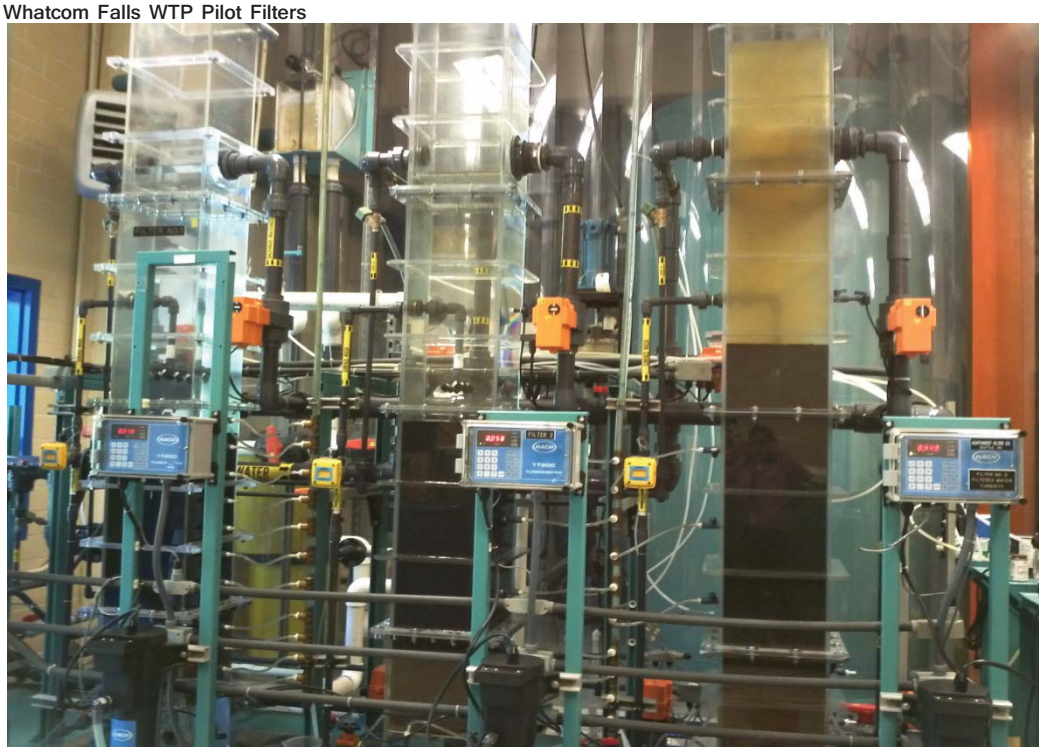
The WTP has three pilot filters, which were used in the pilot testing. Each pilot filter has dimensions of 12 square inches (1 square foot) by 12 feet high. All three are loaded with filter media matching the existing filters: 31 inches anthracite over 11 inches sand. The pilot filters are operated as constant head, effluent flow control filters.

The filter pilot plant is operated by a programmable logical controller (PLC), which can operate the filters in one of three modes: 1) run, backwash, and stop, 2) run, stop, and wait, 3) run, backwash, and continue with next cycle. During the pilot test, the filter system was set up to run in mode 1), that is, when either turbidity or headloss breakthrough is reached the filter will stop running, automatically backwash and standby until it is commanded to start running again. The filter breakthrough criteria include reaching 8.2 feet of headloss, reaching 0.07 NTU of turbidity for 120 minutes, or reaching particles of 100 count/mL. The third breakthrough criterion, 100 particles/mL, was not included in the filter automatic backwash and stop command. Therefore, filters would run until one of the other pre-set parameters was met, or an operator manually stopped them.

On-line analyzers on each filter provide continuous monitoring of turbidity, particle counts and head loss. The data was retrieved at 5-minute intervals for the pilot test. During the pilot testing, each filter was operated at 5, 6, and 7 gallon per minute per square foot (gpm/sf).

The clarified water from DAF was pumped to an elevated overflow box that, in turn, fed by gravity to the pilot filters. The WTP owns the filter influent pump. To avoid the situation that the filter influent pump runs dry when the DAF system is off-line or during a de-sludge event, a break tank was installed at the filter influent pump suction to provide buffering. Figure 2-4 shows the pilot filters.

FIGURE 2-4
Whatcom Falls WTP Pilot Filters



2.3 Sampling and Analysis

2.3.1 Sampling Methodology

Sampling of each process consisted of both grab and continuous sampling methods, depending upon the unit process and sample analysis to be performed. Table 2-2 lists the types of samples that were taken, frequency of samples, and the unit processes sampled.

Continuous samples were taken from the process using equipment recommended flow rates and nonreactive tubing and materials to obtain the most representative samples.

TABLE 2-2
Whatcom Falls WTP Pilot Test Sampling Schedule

Parameter	DAF Feed	DAF Effluent	Pilot Filter 1 Effluent	Pilot Filter 2 Effluent	Pilot Filter 3 Effluent
Turbidity	c	c	c	c	c
Particle Counts			c	c	c
Apparent Color	2	2	2	2	2
Alkalinity	1	1	1		
pH	2	2	2	2	2
Total Iron	1 per week	1 per week	1 per week	1 per week	1 per week
Dissolved Iron	1 per week	1 per week	1 per week	1 per week	1 per week
Total Manganese	1 per week	1 per week	1 per week	1 per week	1 per week
Dissolved Manganese	1 per week	1 per week	1 per week	1 per week	1 per week
Total Aluminum	2	2	2	2	2

TABLE 2-2
Whatcom Falls WTP Pilot Test Sampling Schedule

Parameter	DAF Feed	DAF Effluent	Pilot Filter 1 Effluent	Pilot Filter 2 Effluent	Pilot Filter 3 Effluent
Dissolved Aluminum	2	2	2	2	2
UV254	2	2	2	2	2
Temperature	2	2	2	2	2
Phycocyanin/Chlorophyll a using handheld instrument	2	2	2		
Algae/Chlorophyll a by offsite lab	1	1			
TOC/DOC by offsite lab	1	1			

Notes: The numbers refer to the number of times a day that samples are collected except noted otherwise. “C” refers to continuous sampling by the on-line analyzers.

2.3.2 Sampling Location

Grab samples were taken twice (typically 9 AM and 2 PM) daily from each unit process for lab analysis. The sampling points were selected to obtain the representative samples. The raw water and clarified water (DAF effluent) samples were taken from the drain line of the raw water turbidity meter and clarified water turbidity meter inside of flocculation/DAF trailer. The filter effluent samples were taken from the corresponding filter effluent turbidity meters located in the pilot filter room.

2.3.3 Sampling Analysis

2.3.3.1 Onsite Analysis

Particle and turbidity of various flow streams were continuously monitored. Raw water and DAF effluent turbidity samples were analyzed using Hach 1720E online turbidimeters installed in Roberts’ flocculation/DAF pilot trailer. Turbidity and particle counts of three pilot filters’ effluent were continuously analyzed using Hach 1720C online turbidimeters and Chemtrac PC2400 particle counters provided by the City.

Throughout the pilot testing, the dedicated staff of the City conducted all the onsite sampling and lab analysis. Temperature and pH were measured using pH meter with automatic temperature compensation. Onsite Phycocyanin/Chlorophyll a were measured using a handheld fluorometer AquaFluor manufactured by Turner Designs. All other parameters were analyzed onsite with manual titration or using the Hach DR-5000 UV-spec at the appropriate wavelength following the Hach methods. The analysis methods are listed in Table 2-3.

TABLE 2-3
Onsite Analyses and Methods

Parameter	Analysis Methods	Description
Apparent Color	Hach Method 8025	Platinum-Cobalt Standard Method (15 to 500 CU)
Alkalinity	SM 2320B	Potentiometric Titration
Total or Dissolved Iron	Hach Method 8008	FerroVer Method (0.02 – 3.00 mg/L)
Total or Dissolved Manganese	Hach Method 8149	1-(2-Pyridylazo)-2-Naphthol PAN Method (0.006 to 0.700 mg/L)
Total or Dissolved Aluminum	Hach Method 8012	Aluminon Method (0.008 – 0.800 mg/L)
UV254	Hach Method 10054	Ultraviolet Absorption at 254 nm wavelength

2.3.3.2 Offsite Analysis

The remaining parameters, including TOC, DOC, algal counts, and chlorophyll a concentrations, were analyzed at offsite labs. TOC/DOC samples were collected and sent to Edge Analytical Laboratories for analysis. They were measured using SM 5310B method with the method detection limit of 0.12 mg/L for DOC and 0.065 for TOC. Algae and chlorophyll samples were collected and sent to the laboratory at the Institute for Watershed Studies, which is a research and academic support facility that is affiliated with Huxley College of the Environment, Western Washington University (WWU), for analysis by Dr. Robin Matthews, PhD.

During the pilot testing one pilot filter sample was collected for simulated distribution total trihalomethanes (TTHMs) and haloacetic acids (HAA5) analysis. This analysis was conducted in CH2M HILL's Applied Sciences Laboratory.

SECTION 3

Testing Runs and Conditions

3.1 Pilot Test Performance Goals

Before beginning pilot testing, quantitative minimum evaluation criteria and goals for the quality of the treatment processes were determined. Table 3-1 presents these parameters and values. The evaluation criteria denote the level of achievement expected as a result of the pilot testing, whereas the goal reflects the desired limits beyond what is necessary. Meeting or exceeding the evaluation criteria is a measure of the success of the pilot study, while meeting or exceeding the goals is an extra benefit.

TABLE 3-1
Performance Goals for Whatcom Falls WTP Pilot Study

Sample Point	Parameter	Evaluation Criteria	Goal
Clarified water	Total algae removal	--	>95% removal
Clarified water	Turbidity (steady-state)	<1.0 NTU	0.5 NTU
Filter Effluent	Turbidity (steady-state)	<0.07 NTU	<0.05 NTU
Filter Effluent	Turbidity spike (ripening)	0.2 NTU	<0.1 NTU
Filter Effluent	Particle counts (steady-state)	< 100 p/ml >2 μm	< 20 p/ml > 2 μm
Filter Effluent	Particle removal (steady-state)	2-log	2.5 log
Filter Effluent	Ripening time	30 minutes	< 15 min. to <0.1 NTU
Filter Production	Unit filter run volume	>5,000 gal/ft2	>8,000 gal/ft2

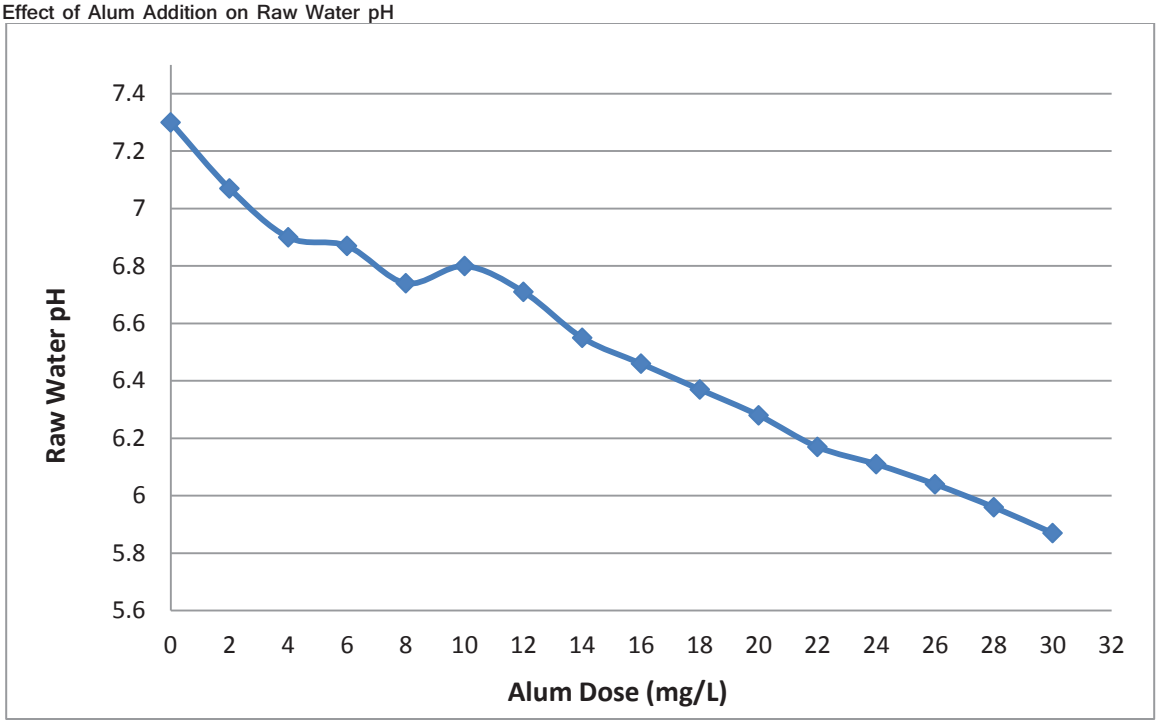
3.2 Bench-Scale Tests

A series of bench-scale tests were performed to help determine the optimal chemical doses used in the pilot testing. The determined chemical doses were adjusted minimally throughout the pilot testing while the DAF loading rates changed.

3.2.1 Titration – Effects of Alum Addition on Water pH

The first bench-scale test was performed to determine the effects of various doses of alum addition on raw water pH. Since alum is acid solution and the historical data show that raw water alkalinity is approximately 20 mg/L as $CaCO_3$, caution is needed so that raw water pH will not drop too much by alum addition. Titration was done by adding an incremental volume of pre-prepared alum solution into 1,000 mL of raw water and measuring pH at each step. Figure 3-1 shows the trend between alum dose and raw water pH obtained from the titration test. At the alum dose range of 6 to 16 mg/L, raw water pH was maintained above 6.4.

FIGURE 3-1
Effect of Alum Addition on Raw Water pH



3.2.2 Jar Test – Alum/Polymer Doses

The bench-scale tests were conducted using the jar testing equipment provided by Roberts, which consists of the air compressor, a saturator, and a series of jars to simulate the flocculation tanks and DAF (Figure 3-2). Chemical injection and mixing could be supplied as needed and the air-saturated water at varied rates could be injected to simulate different recycle rates.

FIGURE 3-2
Flocculation/DAF Bench-Scale Test Equipment by Roberts

Various doses of alum and polymer that are currently used at the plant were tested. Turbidity and UV254 of the clarified water from each run were measured. Two sets of tests were completed. The first one tested alum addition only. The second one tested alum addition with polymer (Sumaclear P20, Summit Chemical Company). Table 3-2 shows the run conditions, observed floc appearance, and the water quality data during the first set of tests. When 6 to 16 mg/L of alum was added, raw water turbidity was reduced from 0.77 NTU to approximately 0.2 NTU. Raw water UV254 (as an indicator of TOC) was reduced by approximately 50 percent. It appears that alum doses between

8 to 10 mg/L during the jar test obtained the optimal clarified water quality in terms of the combination of both parameters. This is consistent with what the WTP is currently dosing at the rapid mixing upstream of the plant filters.

TABLE 3-2
Bench-scale Flocculation and DAF Testing Results – Alum Only

Jar Test Number	Alum Dose (mg/L)	Floc Appearance	Turbidity (NTU)	UV254 (1/cm)
1	6	Small, scattered	0.249	0.026
2	8	Pin	0.200	0.028
3	10	Pin	0.219	0.019
4	12	Pin	0.250	0.018
5	14	Pin	0.223	0.019
6	16	Getting larger	0.214	0.018

Notes:
Raw water turbidity: 0.77 NTU, UV254: 0.048 1/cm
Rapid mix: 1 minute retention time at 250 rpm
1st stage flocculation: 5 minutes at 80 rpm, 2nd stage flocculation: 5 minutes at 40 rpm
DAF float time: 5 minutes, recycle rate: 10 to 12 percent

Table 3-3 represents the testing conditions and results of the second jar testing where both alum and polymer were dosed. It appeared that increasing polymer dose from 0.1 to 0.3 mg/L consistently reduced the UV254. There was little benefit to add over 0.3 mg/L polymer in terms of turbidity reduction. However, lower doses of polymer ranging from 0.1 to 0.2 mg/L may help reduce turbidity by forming smaller and easily floatable floc. Based on the bench-scale test results, the following chemical addition was selected as a starting point during the pilot testing

- Alum dose = 10 mg/L
- Polymer = 0.1 to 0.2 mg/L (when used)

TABLE 3-3
Bench-scale Flocculation and DAF Testing Results – Alum and Polymer

Jar Test Number	Alum Dose (mg/L)	Polymer Dose (mg/L)	Floc Appearance	Turbidity (NTU)	UV254 (1/cm)
1	8	0.1	Smaller pin	0.187	0.020
2	8	0.2	Pin	0.185	0.019
3	8	0.3	Medium pin	0.246	0.017
4	10	0.1	Pin	0.216	0.018
5	10	0.2	Pin	0.202	0.016
6	10	0.3	Medium pin	0.232	0.016

Notes:

Raw water turbidity: 0.77 NTU, UV254: 0.048 1/cm

Rapid mix: 30-second retention time at 250 rpm; adding polymer at end of rapid mix after coagulation

1st stage flocculation: 5 minutes at 60 rpm, 2nd stage flocculation: 5 minutes at 40 rpm

DAF float time: 5 minutes, recycle rate: 10 percent

3.3 Pilot Testing Runs

The original schedule of the pilot testing was four to five weeks. One set of conditions would be tested each week, with the last week reserved for repeating the optimal run determined based on the previous tests. The pilot testing actually lasted for seven weeks, due to some equipment and operations and maintenance (O&M) issues with the pilot system. Table 3-4 summarizes all the runs conducted during the testing. Some of them did not reach the pre-set termination criteria, that is, terminal headloss, turbidity, or particle counts. They are included in the table and evaluation because they still provide valuable information to determine the pilot unit performance and the filter headloss accumulation. All the raw data from testing are available in the electronic format by request (Attachment C).

The following paragraphs provide brief descriptions of the test goals, as well as what actually occurred in each week.

Week 1: Troubleshooting was conducted on a couple of issues after system startup, including raw water supply modification, DAF recycle pump power trip issue, filter feed pump air binding. A first run with 10 gpm/sf of DAF loading and 10 mg/L alum only was completed.

Week 2: Pilot Runs 2 through 5 were conducted. Pilot filter 1(PF 1) was offline during Runs 2 and 3 due to a backwash problem. Runs 2, 3, and 4 were terminated before filter terminal headloss was achieved due to operational issues or maintenance on the pilot system (DAF and/or filters). Run 5 was a full run at 10 gpm/sf of DAF loading without polymer addition.

Week 3: Pilot Runs 6 to 9 were conducted with 10 gpm/sf of DAF loading and alum plus polymer addition. Occasional DAF recycle pump and pilot filter valves issues occurred. Run 6 and Run 8 were fully completed runs.

Week 4: Pilot Runs 10 to 12 were conducted. DAF loading was increased to 14 gpm/sf while the chemical dosages were kept the same. The raw water flow to the flocculation was maintained at approximately 63 gpm as well. Run 10 was terminated before terminal headloss, turbidity, or particle breakthrough was achieved due a continuing power issue with the DAF recycle pump. Runs 11 and 12 were complete runs.

Week 5: Pilot Runs 13 to 15 were conducted. Raw water flow was increased to approximately 72 gpm and DAF feed was increased to 64 gpm (equivalent to 16 gpm/sf loading). Run 14 terminated earlier for PF 1 due to a plant-wide power failure.

Week 6: Pilot Runs 16 through 18 were completed. Runs 16 and 17 tested system performance at 20 gpm/sf of DAF loading rate and the same chemical doses. The clarified water flow rate was accomplished by modifying the overflow piping between flocculation basins and DAF so that no overflow occurred when raw water flow was increased to approximately 80 gpm. Run 18 was a duplicate run of the optimal test condition – 16 gpm/sf of DAF loading.

Week 7: This was a week outside of the original testing plan. The WTP staff conducted additional experimental runs to determine the effectiveness of flocculation without DAF during an algae bloom condition. These were tested in Runs 19 and 20 where raw water was still fed to the flocculation and DAF with typical chemical addition, but the DAF recycle pump and air compressor were turned off. The purpose of Runs 21 and 22 was to provide a baseline comparison between the pilot filters and plant filters. Plant flocculated water was fed to the pilot filters and plant filters. Both sets of filters were operated at the same loading rates (5 and 6 gpm/sf). The effluent headloss, turbidity, particles, and filter run times were compared.

TABLE 3-4
Pilot Test Runs and Conditions

Week	Run	Duration	Raw Water Flow (gpm)	Floc Time (min)	Alum Dose (mg/L)[a]	Polymer Dose (mg/L)[a]	DAF Influent Flow (gpm)	DAF Loading (gpm/sf)	DAF Recycle Rate[a]	Pilot Filter 1 Loading (gpm/sf)	Pilot Filter 2 Loading (gpm/sf)	Pilot Filter 3 Loading (gpm/sf)	Note
1	1	15:30 8/10 - 14:45 8/12	63	5	10	-	40	10	12%	5	6	7	DAF @ 10 gpm/sf, alum
2	2	17:00 8/12 - 12:00 8/13	63	5	10	-	40	10	12%	-	5	6	Repeat 10 gpm/sf run, alum
	3	14:45 8/14 - 8:00 8/15	63	5	10	-	40	10	12%	-	5	6	Repeat 10 gpm/sf run, alum
	4	15:05 8/15 - 12:25 8/16	63	5	10	-	40	10	12%	5	6	7	Repeat 10 gpm/sf run, alum
	5	14:50 8/16 - 11:40 8/19	63	5	10	-	40	10	12%	5	6	7	Repeat 10 gpm/sf run, alum
3	6	14:10 8/19 - 10:05 8/22	63	5	10	0.1-0.2	40	10	12%	5	6	7	DAF @ 10 gpm/sf, alum & polymer
	7	16:10 8/22 - 6:35 8/23	63	5	10	0.1-0.2	40	10	12%	5	6	7	Repeat 10 gpm/sf, alum & polymer
	8	12:05 8/23 - 21:10 8/25	63	5	10	0.35	40	10	12%	5	6	7	Repeat 10 gpm/sf, alum & polymer
	9	8:10 8/26 - 6:05 8/28	63	5	11	0.3	40	10	12%	5	6	7	Repeat 10 gpm/sf, alum & polymer
4	10	15:10 8/29 - 4:40 8/30	63	5	11	0.3	56	14	12%	5	6	7	DAF @ 14 gpm/sf, both chemicals
	11	13:45 8/30 - 16:25 9/1	63	5	11	0.3	56	14	12%	5	6	7	Repeat 14 gpm/sf run
	12	9:00 9/2 - 9:00 9/4	63	5	11	0.3	56	14	12%	5	6	7	Repeat 14 gpm/sf run
5	13	9:45 9/4 - 8:10 9/6	72	4.4	11	0.3	64	16	11-12%	5	6	7	DAF @ 16 gpm/sf
	14	14:50 9/6 - 13:10 9/7	72	4.4	11	0.3	64	16	11-12%	5	6	7	Repeat 16 gpm/sf run
	15	14:05 9/7 - 8:10 9/9	72	4.4	11	0.3	64	16	11-12%	5	6	7	Repeat 16 gpm/sf run
6	16	16:50 9/9 - 3:45 9/11	78	8	10	0.4	78	20	12%	5	6	7	DAF @ 20 gpm/sf
	17	8:50 9/12 - 3:45 9/14	78	8	10	0.4	78	20	12%	5	6	7	Repeat 20 gpm/sf run
	18	8:00 9/15 - 15:50 9/17	72	4.4	10	0.3	64	16		5	6	7	Optimal run @ 16 gpm/sf
7	19	17:10 9/17 - 5:40 9/18	72	4.4	10	0.3	64	16		5	6	7	No DAF recycle
	20	11:40 9/19 - 22:50 9/19	63	5	10	0.2	40	10		5	6	7	No DAF recycle
	21[b]	9:50 9/20 - 2:25 9/21	-	-	-	-	-	-		5	6	7	Post-pilot Filter Comparison
	22[b]	11:00 9/21 - 3:20 9/22	-	-	-	-	-	-		5	6	7	Post-pilot Filter Comparison

Notes:

[a] The listed parameters were average values or range measured. The actual values in the pilot testing varied around the values shown due to the accuracy of the instrument and equipment in the real operation.

[b] Pilot filters and plant filters were tested at the same loading rates with plant flocculated raw water. DAF was not in operation.

SECTION 4

Testing Results

4.1 Flocculation Performance

Two 2-stage flocculation basins were installed upstream of the DAF. After chemical injection, raw water passes through an in-line mixer and then enters one or two of the flocculation basins. Two important operating parameters, flocculation time and mixing intensity, were evaluated during the pilot testing. Table 4-1 summarizes these parameters at varied influent flow rates.

TABLE 4-1

Flocculation Operating Conditions during Pilot Testing

Floc Influent Flow (gpm)	Number of Duty Floc Train	Floc Time (min)	Stage 1 G (1/s)	Stage 2 G (1/s)	Stage 1 GT	Stage 2 GT
63	1	5	135	68	20,000	10,000
72	1	4.4	135	68	18,000	8,900
78	2	8	135	68	33,000	16,000

During the test runs with DAF loading rates of 10 through 16 gpm/sf, only one flocculation train was in service. The total detention time (flocculation time) with one flocculation train was between 4.4 and 5.0 minutes. When two flocculation basins were used at DAF loading rate of 20 gpm/sf (influent flow increased to 78 gpm), the flocculation time was increased to about 8 minutes. This was done to ensure adequate flocculation time was available at this peak loading. With a single floc train in service, the detention time would have been 4 minutes or less.

Typical detention times for conventional coagulation/gravity settling processes are 30 to 45 minutes and three stages. For DAF, most WTPs have 10 minutes of flocculation time with two stages at maximum flow. During this pilot, it was demonstrated that short flocculation times during moderate water temperatures (15 to 20 degrees Celsius) did not affect the DAF and filter performance based on the discussion below. Visual observations of the floc formed with 5 minutes flocculation time and the mixer speeds above show a pin-floc that is good for flotation.

Mixing intensity within the basins is typically represented by velocity gradient (G) or the product of G and detention time (GT). The typical G values for two-stage tapered floc basins range from 70 to 120 1/s (stage 1) and 40 to 60 1/s (stage 2) for DAF processes. The GT values presented in Table 4-1 resulted in G values in these ranges.

4.2 Dissolved Air Flotation Performance

Roberts had a dedicated operator for their pilot trailer throughout the testing. The operator was responsible for controlling raw water to the flocculation tanks, setting up the appropriate chemical addition, controlling appropriate flow to the DAF, optimizing the flocculation and DAF operation and accurate operation of the equipment and instrument within the trailer. The major O&M activities involved include:

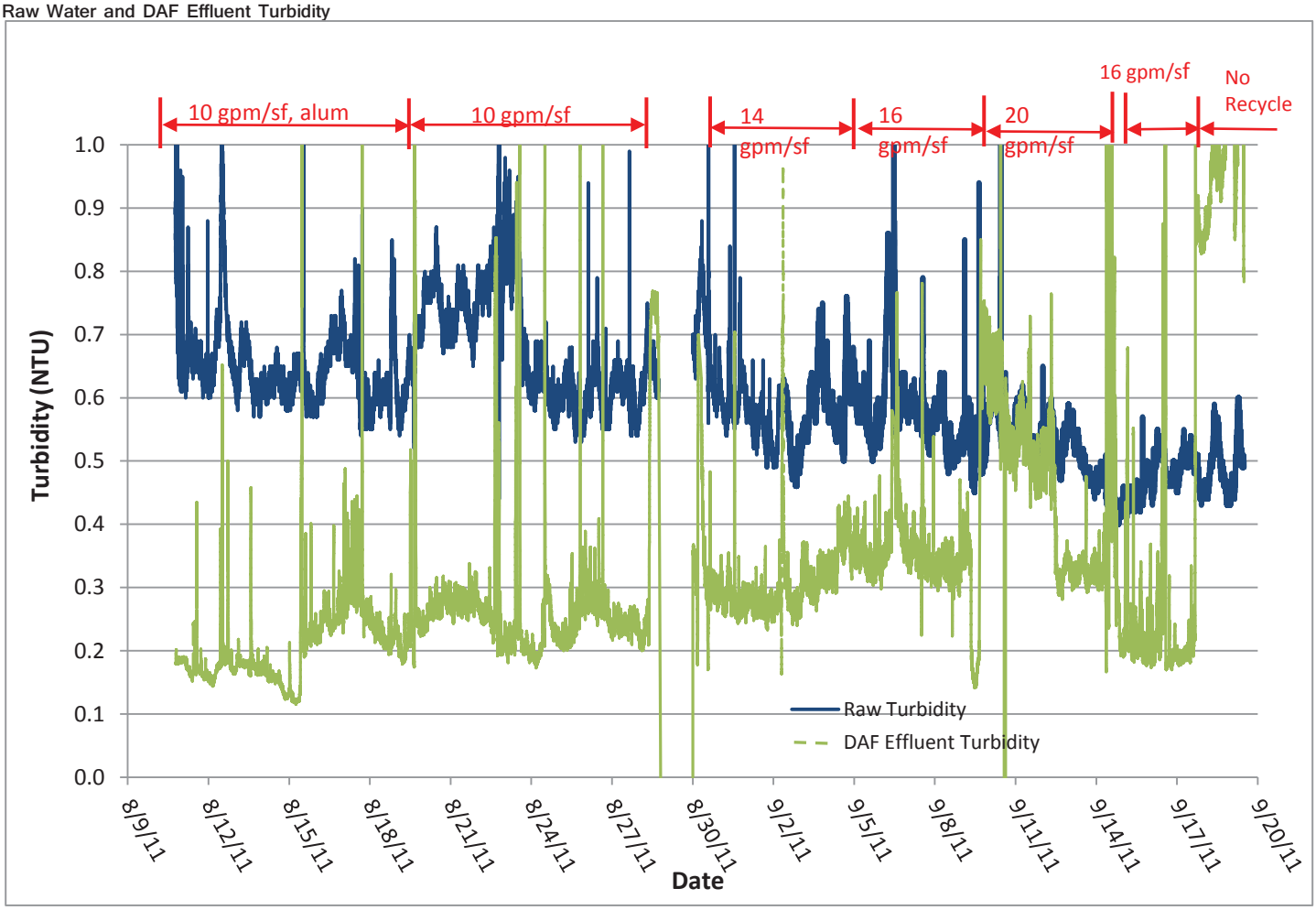
- Set up and calibrate chemical metering pumps each time before raw water flow rate changed.
- Change out the injection nozzles at the bottom of the DAF tank to obtain the desired DAF recycle rate when DAF loading rates changed.
- Modify the overflow piping between flocculation and DAF before 20 gpm/sf DAF loading was tested.

Overall, the flocculation and DAF had a fairly consistent and satisfactory performance throughout the testing.

4.2.1 Turbidity

Figure 4-1 shows the turbidity of raw water and DAF effluent during the entire pilot testing. The observed DAF effluent turbidity spikes occurred when the DAF system was shut down for maintenance.

FIGURE 4-1
Raw Water and DAF Effluent Turbidity



The trends indicate that raw water turbidity during the first 10 days ranged between 0.6 and 0.7 NTU, then increased to between 0.7 and 1.0 NTU from August 19 to 23, 2011. After that date, it reverted to the 0.6 to 0.7 NTU range. During the later stage of the testing, raw water turbidity gradually dropped to below 0.6 NTU and as low as 0.4 NTU at the end of the testing.

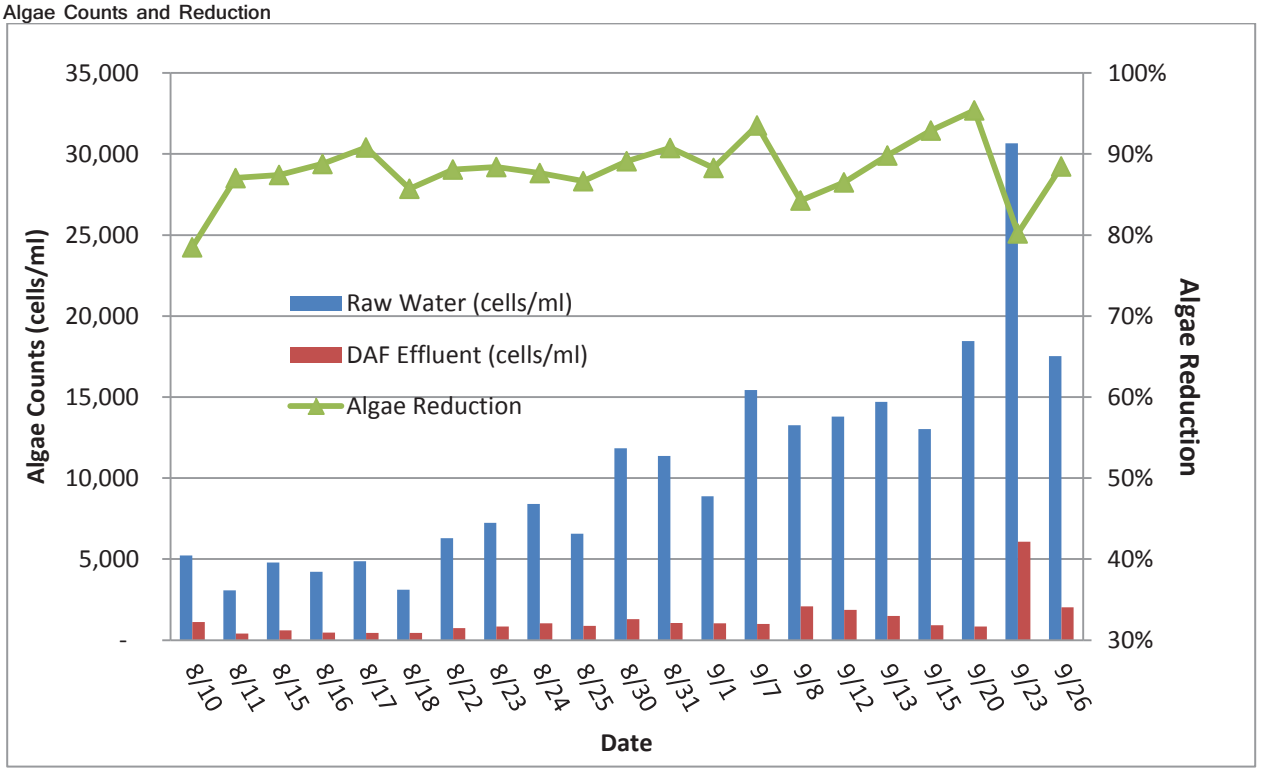
When DAF was loaded at 10 gpm/sf, the DAF effluent turbidity was initially below 0.2 NTU for about a week then increased to between 0.2 and 0.3 NTU range. It appeared that the DAF effluent turbidity levels were independent of raw water turbidity and polymer addition. However, the DAF effluent turbidity did slightly increase when DAF loading rate increased. For example, when DAF loading rate was 14 gpm/sf, the average DAF effluent turbidity was about 0.3 NTU. When DAF loading rate increased to 16 gpm/sf, the DAF effluent turbidity increased to between 0.3 and 0.4 NTU. In addition, when DAF loading further increased to 20 gpm/sf, the DAF effluent turbidity was almost at the same level as raw water during the first 20 gpm/sf run and then reduced to 0.3 and 0.35 NTU range in the second 20 gpm/sf run. During the optimal run at 16 gpm/sf DAF loading, the DAF effluent turbidity was back to the level obtained at the beginning of the testing, which is approximately 0.2 NTU. When DAF recycle pump was turned off, the DAF effluent turbidity jumped higher than raw water turbidity, which is over 1 NTU most of the time. This high effluent turbidity was likely caused by the floc that suspended in the effluent when the DAF did not work properly.

Except during the first 20 gpm/sf run where the DAF effluent turbidity was close to raw water turbidity, DAF consistently produced effluent with turbidity below 0.5 NTU, which was the DAF turbidity performance goal.

4.2.2 Algae

Daily algae counts in raw water and DAF effluent during the pilot testing are from the grab samples collected by WTP staff and counted by WWU. Note the algae counting approach for the pilot test samples was different from that used for the routine raw water algae monitoring. The algae counting during the pilot testing included all algae in whole water samples. It involved settled raw water counts with full taxonomic identification and estimation of cell density in cyanobacteria colonies. While the routine algae monitoring, which provided the basis for raw water historical algae counts, utilizes 20 μm plankton net tows to collect only large cells. In addition, the counting was to the level of cells or colonies identified to division. Therefore, the historical algae data are not comparable to the algae data obtained during the pilot testing. Figure 4-2 shows the daily algae counts during the pilot testing. It shows that raw water algae counts increased as the testing proceeded. The highest value (30,665 cells/mL) occurred at the end of the test. Compared to raw water algae counts, DAF effluent appeared to have a relatively stable algae count. The algae reduction by DAF ranged from 78 to 95 percent, with an average of 88 percent, which is below the algae reduction goal of 95 percent. However, the removal of algae by the DAF pilot system is still significant, and the reduction in algae to the filtration process has a significant impact on filtration performance, as detailed later in this report.

FIGURE 4-2
Algae Counts and Reduction



During the testing, the handheld fluorometer was used to obtain the daily readings of chlorophyll a and Phycocyanin of raw water and DAF effluent. The instrument gave relative values that are supposed to be proportional to the measured fluorescence compared to an adjustable secondary standard. The intent of using the handheld fluorometer is to provide an easier way to quantify the algae relatively by tracking the relative chlorophyll a or phycocyanin values in the water. Figure 4-3 shows the comparison between chlorophyll a counted by WWU and relative chlorophyll a measured in the plant lab using the handheld fluorometer. Note that chlorophyll a counted by WWU has a unit of μg/L, while the relative chlorophyll a does not have a unit. Despite of some issues with the handheld instrument repeatability, two sets of measurements have shown a very consistent trend. Table 4-2 shows the removal efficiencies of algae, chlorophyll a, and phycocyanin obtained using different methods. They also showed general agreement among each method.

FIGURE 4-3
Raw Water Chlorophyll a Measurement Comparison

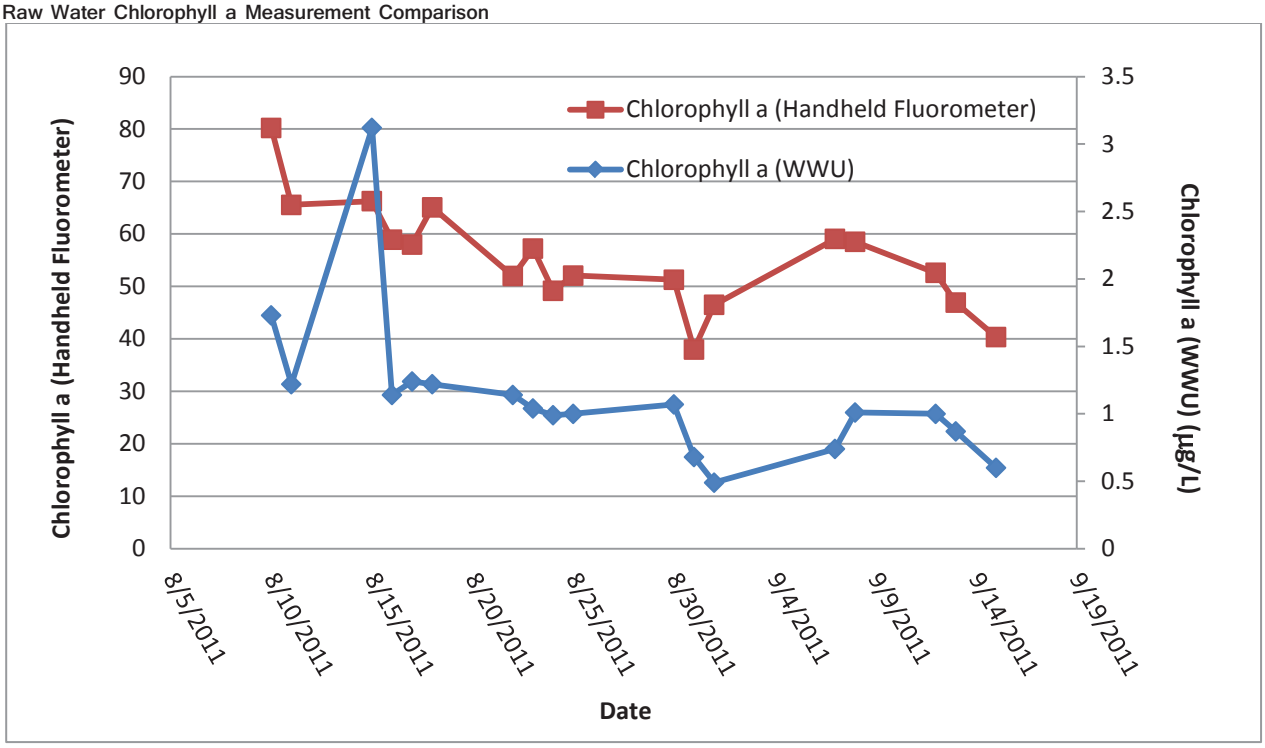


TABLE 4-2

Removal Efficiency Comparison of Algae Counts by Different Methods

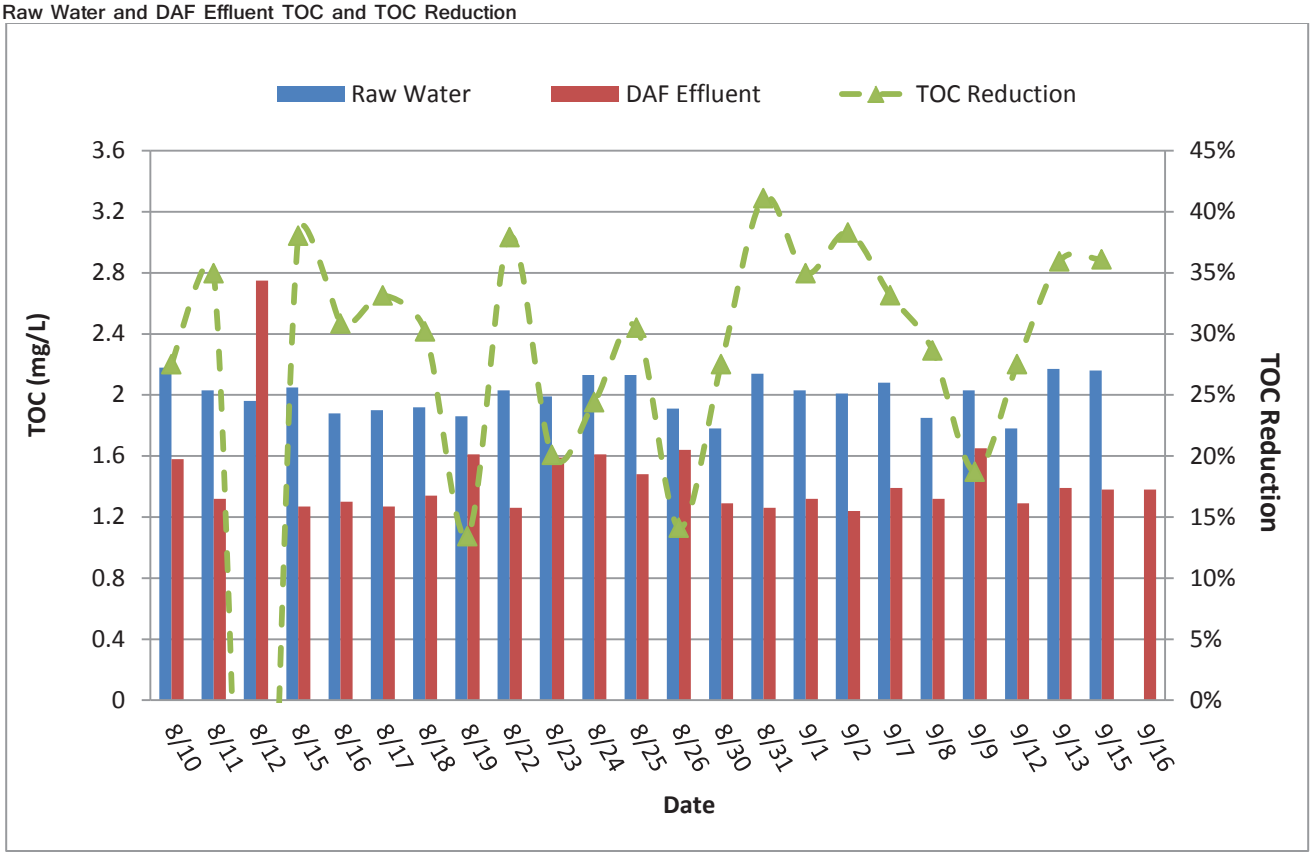
	Algae Removal (WWU)	Chlorophyll Removal (WWU)	Chlorophyll Removal (handheld fluorometer)	Phycocyanin Removal (handheld fluorometer)
Average	88%	86%	72%	48%
Max.	95%	97%	84%	98%
Min.	78%	40%	15%	14%

4.2.3 TOC/DOC

TOC is a primary measurement of organic content in water supplies and a measurement of disinfection by-product precursors, which are organic compounds that can combine with chlorine disinfectant to form disinfection by-products. The City currently meets the regulatory standards for disinfection by-products and is not expected have difficulty meeting these standards anytime soon. As a result, TOC/DOC removal was not a primary objective for this pilot testing and the pilot testing was not optimized for TOC/DOC removal. However, during the pilot test influent and effluent TOC/DOC concentrations were measured. The reason for monitoring TOC/DOC during the pilot testing is that the addition of a clarification process such as DAF to the existing in-line filtration system could potentially subject the City to TOC removal requirements under the Stage 1 Disinfection by-product rule (DBPR).

Grab samples were also collected daily for TOC/DOC measurement. Raw water TOC and DOC data from the first 13 days indicated that over 90 percent of TOC was DOC. Therefore, only TOC was measured afterwards. Figure 4-4 shows the raw water and DAF effluent TOC data and TOC reduction by DAF. The reduction ranged from 14 to 40 percent. No correlation between TOC reduction and DAF loading rates was observed.

FIGURE 4-4
Raw Water and DAF Effluent TOC and TOC Reduction



Enhanced coagulation is a requirement under the Stage 1 DBPR (U.S. Environmental Protection Agency [USEPA], 1998). For the rule, USEPA developed a matrix (Table 4-3) to determine the amount of TOC reduction required in a clarification process. The matrix is based on the amount of raw water TOC present and on the alkalinity of the source water.

TABLE 4-3
Required Removal of TOC by Enhanced Coagulation

	Source Water Alkalinity (mg/L as CaCO3)		
Source Water TOC (mg/L)	**0-60**	**>60-120**	**>120**
> 2.0 - 4.0	35%	25%	15%
> 4.0 - 8.0	45%	35%	25%
> 8.0	50%	40%	30%

Source: USEPA, 1998

The average raw water alkalinity of the Whatcom Falls WTP was 20.7 mg/L as $CaCO_3$ and the average raw water TOC was higher than 2.0 mg/L. According to Table 4-3, the required removal of TOC by enhanced coagulation would be 35 percent. One of the alternative compliance criteria set forth by USEPA in the Stage 1 D/DBPR is that if the finished water (post-filtration) TOC is less than 2.0 mg/L, the plant would be exempt from meeting the 35 percent removal requirement for TOC removal in enhanced coagulation. Based on the historical plant filter effluent TOC data, the post-filtration water from Whatcom Falls WTP would easily be below 2.0 mg/L; therefore, the WTP would qualify for the exemption. The additional removal of TOC in the clarification process may reduce the formation of disinfection by-products (DBPs). To confirm this potential reduction, one Simulated Distribution System (SDS) test was conducted during the pilot testing. Results of the SDS testing are presented in Section 4.3.

4.2.4 Other Water Characteristics

Other water characteristics that could help evaluate DAF performance include pH, apparent color, metal (iron, manganese, and aluminum) and UV254. Table 4-4 summarizes the data analyzed by the WTP lab during the pilot testing.

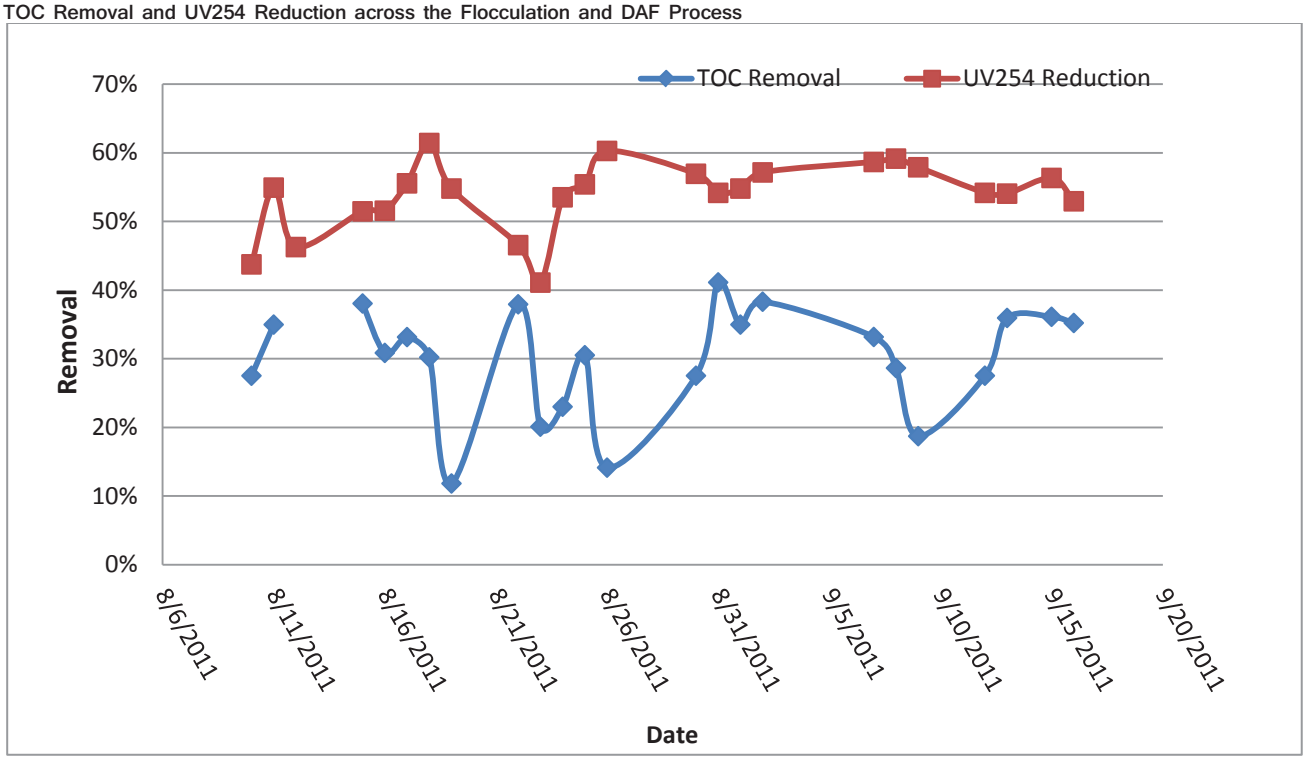
TABLE 4-4
Other Characteristics of Raw Water and DAF Effluent

	Raw Water				DAF Effluent			
	Average	Minimum	Maximum	Number of Data Points	Average	Minimum	Maximum	Number of Data Points
pH (S.U.)	7.40	7.07	7.61	46	6.85	6.54	7.55	46
Apparent Color (C.U.)	11	1	19	45	7	0	20	44
Alkalinity (mg/L as $CaCO_3$)	19.6	0.0	23.1	26	14.0	13.4	20.9	24
Total Iron (mg/L)	0.014	0.000	0.040	5	0.008	0.000	0.020	5
Dissolved Iron (mg/L)	0.006	0.000	0.020	5	0.004	0.000	0.010	5
Total Manganese (mg/L)	0.004	0.000	0.010	5	0.003	0.000	0.009	5
Dissolved Manganese (mg/L)	0.002	0.000	0.006	5	0.002	0.000	0.005	5
Total Aluminum (mg/L)	0.030	0.001	0.192	44	0.111	0.077	0.653	44
Dissolved Aluminum (mg/L)	0.009	0.000	0.037	43	0.020	0.001	0.042	43
UV254 (1/cm)	0.035	0.028	0.039	46	0.016	0.011	0.027	46

Based on the table, alum addition during the testing lowered pH down to 6.5. This was consistent with titration study performed before the pilot testing. Apparent color of DAF effluent varied in a much wider range compared to raw water. This was likely caused by alum floc that occasionally captured in the DAF effluent sample. Alkalinity was reduced from approximately 20 mg/L as $CaCO_3$ to 14 mg/L as $CaCO_3$ from the addition of alum as coagulant. Dissolved and total iron and manganese were virtually non-detects on the raw and DAF effluent samples. Total and dissolved aluminum were measured to ensure that complete coagulation was occurring and that overdosing was not happening. Significant concentrations of dissolved aluminum would be an indicator of this. The low level of aluminum (<0.2 mg/L total aluminum and < 0.04 mg/L dissolved aluminum) detected in raw water was likely from leakage coming from the common effluent channel for both plant rapid mix basins. Water in the effluent channel was flocculated water from the duty rapid mix basin. Some of the flocculated water may have leaked back to the rapid mix basin no. 1, which was used as pilot test raw water wet well. DAF effluent had slightly higher total and dissolved aluminum levels than raw water, due to the addition of alum. The resulting aluminum level in the DAF effluent was still too low to cause any concern.

UV254 was measured as an on-line surrogate to assess the efficiency of coagulation in reducing TOC. UV254 is typically used in the water industry as a surrogate for TOC because in many waters a direct relationship can be developed for TOC to UV254. Raw water UV254 during pilot testing was relatively stable (in the range of 0.028 to 0.039 1/cm). There was between 41 and 61 percent of UV254 reduction across the flocculation and DAF process. Figure 4-5 shows TOC removal and UV254 reduction across the flocculation and DAF process. No discernable relationship was observed between TOC and UV254 removal, likely due to the low organics levels in raw water.

FIGURE 4-5
TOC Removal and UV254 Reduction across the Flocculation and DAF Process

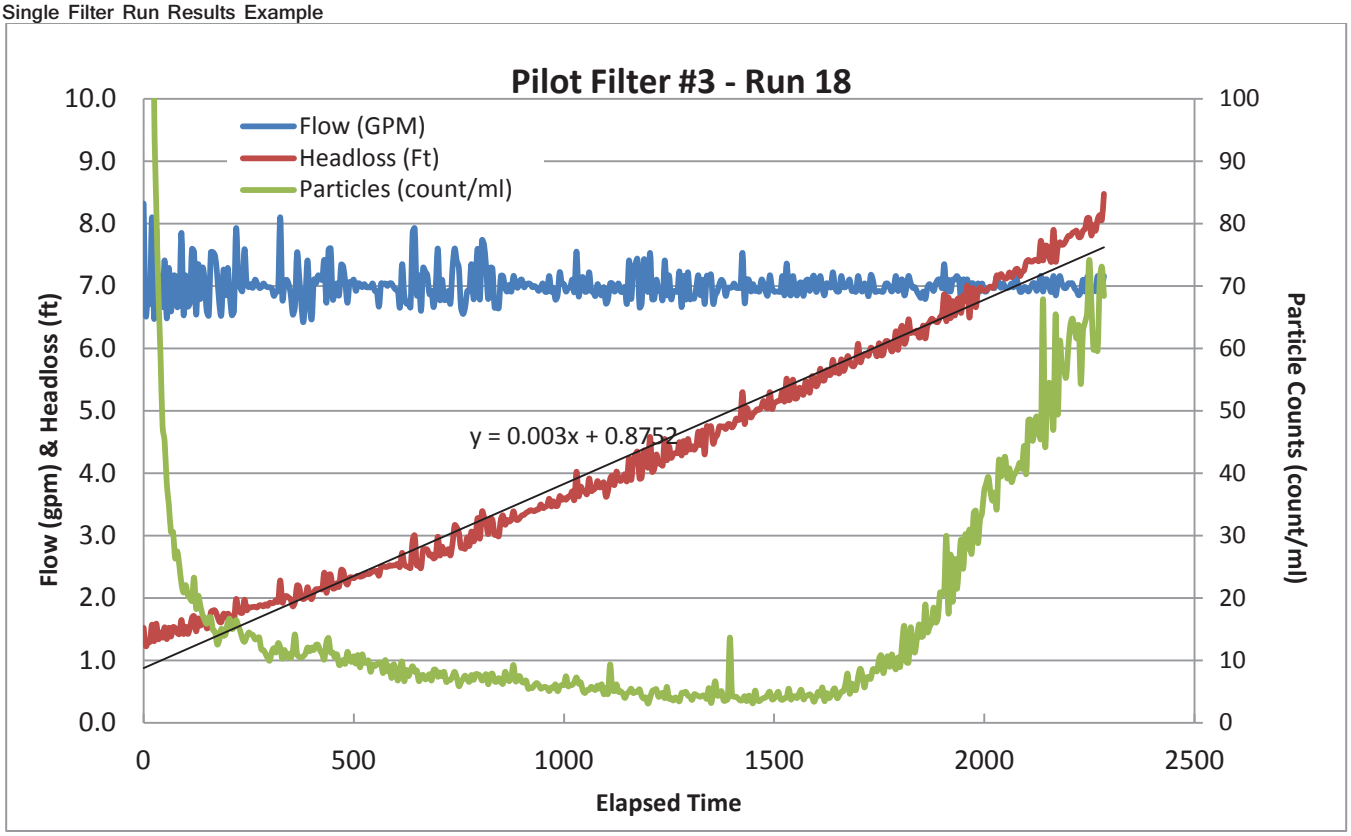


UV254 is also important to establish specific ultraviolet absorbance (SUVA), which can be an indicator of coagulant performance for organics removal. Typically, SUVA values less than 2 L/mg-m indicates that coagulation is optimized for organics removal and very little additional organics can be removed by continued optimization of coagulation. The SUVA is calculated by dividing the ultraviolet absorbance of the sample (in 1/cm) by the DOC of the sample (in mg/L) and then multiplying by 100 cm/m. The DAF effluent SUVA during testing ranged from 1.0 to 1.9 L/mg-m, with an average of 1.3 L/mg-m. Therefore, there would be little gained for organics removal by adding additional coagulant.

4.3 Filter Performance

The WTP staff was responsible for pilot filter operation. Filter effluent turbidity and particle counts, as well as the headloss across the filter medium were monitored and recorded. Figure 4-6 provides an example of how each filter performance was trended and evaluated. Filter run time, ripening time, and effluent quality were evaluated based on the recorded data.

FIGURE 4-6
Single Filter Run Results Example

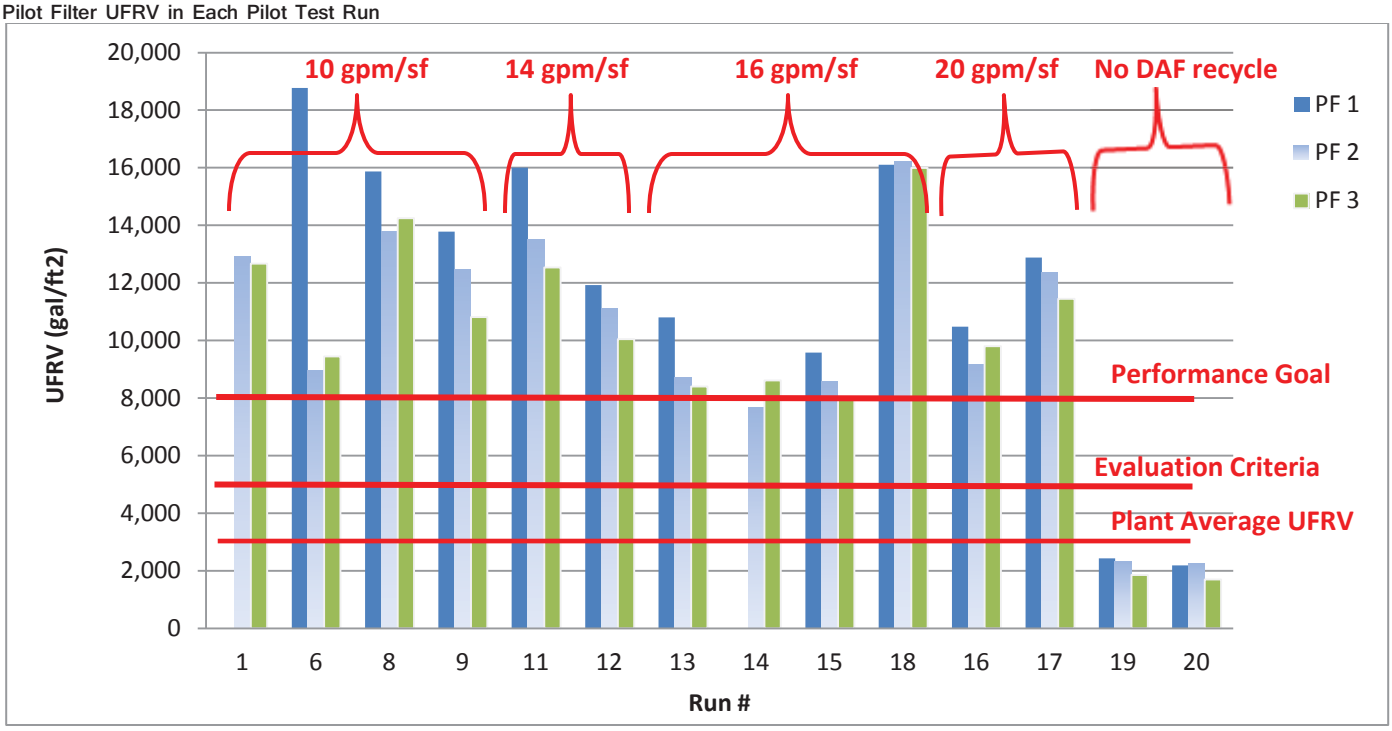


4.3.1 Unit Filter Run Volume

Unit filter run volume (UFRV) is the measure of the production capability of a filter. It is the amount of water treated between backwash events in gallons per square foot (gal/ft^2). This parameter is utilized to compare productivity/performance of filters running at different loading rates. A criterion value of 5,000 gal/ft^2 (considered a minimum to assess DAF success) was used for this testing, with a performance goal of 8,000 gal/ft^2 identified for this pilot test. The performance goal of 8,000 gal/ft^2 was established because it reflects an average of the highest UFRV values the City observes during a typical year – representing relatively favorable treatment conditions.

Figure 4-7 shows the UFRV of the three pilot filters in pilot runs that were completed to breakthrough. On almost every run, the pilot filter UFRV was based on particle breakthrough greater than 100 particles/mL. They are grouped based on the DAF loading rates, which are shown at the top of the bar chart. The loading rates of three pilot filters were 5, 6, and 7 gpm/sf for PF 1, PF 2, and PF 3, respectively. There was an exception in Run 1 where PF 2 was loading at 5 gpm/sf and pilot filter 3 at 6 gpm/sf. Excluding the last two runs where the DAF recycle was turned off, all runs achieved UFRVs above 5,000 gpm/sf, which is the pilot evaluation criterion. All runs except pilot filter 2 in Run 14 achieved the UFRV above 8,000 gpm/sf, which is the pilot performance goal. The filter 2 UFRV in Run 14 was approximately 7,680 gpm/sf – essentially at the 8,000 gpm/sf goal). For comparison purposes, the average of the actual City WTP UFRV during the DAF pilot test period is shown as 3,000 gpm/ft^2 (UFRV ranged from 2,600 to 3,600 gal/ft^2). The plant filters are normally operated at about 3.5 gpm/sf loading. The pilot filters with the DAF pretreatment process had UFRVs 5,000 to 13,000 gal/ft^2 greater than the plant filters during the pilot test period. Note that the pilot filters were operated at a higher loading rate than the plant filters.

FIGURE 4-7
Pilot Filter UFRV in Each Pilot Test Run



No obvious correlation between the DAF loading rate or DAF effluent turbidity and the filter UFRV has been identified. For example, Runs 14 and 15 had relatively low UFRVs based on Figure 4-7. However, the DAF effluent turbidity during these two runs (from 14:50 September 6 to 8:10 September 9) from Figure 4-1 did not show obvious difference from turbidity in other runs. It appeared that other factors, such as particle count and size distribution in DAF effluent, might have impact that is more significant on filter run time.

When DAF recycle was turned off in Runs 19 and 20, the UFRV dropped dramatically to about 2,000 gal/ft^2. All filters were terminated for backwash due to the effluent turbidity breakthrough (0.07 NTU). Clearly, operating the DAF flocculation system and not operating the DAF recycle system to simulate complete DAF operation was not observed to be effective at improving filter performance.

4.3.2 Filter Effluent Turbidity

The filter effluent turbidities during steady-state operation for all pilot filters were low, below the turbidity breakthrough criterion. All the pilot filters had particle or headloss breakthrough earlier than turbidity breakthrough (0.07 NTU criterion). When a filter run was deemed completed (due to particle or headloss breakthrough), the filter effluent turbidity was 0.03 to 0.04 NTU.

In Runs 19 and 20, when DAF recycle was turned off, turbidity became the controlling factor to terminate the filter runs before the backwash. Again, this demonstrated the importance of the complete DAF process to ensure long filter life.

4.3.3 Filter Ripening Time

The filter ripening time is the amount of time before the filter is ready to produce water that meets a specified turbidity goal. The water produced by the filter before it is “ripened” is the amount of filter effluent that would go to waste. The ripening turbidity goal for this pilot testing was 0.1 NTU in less than 15 minutes.

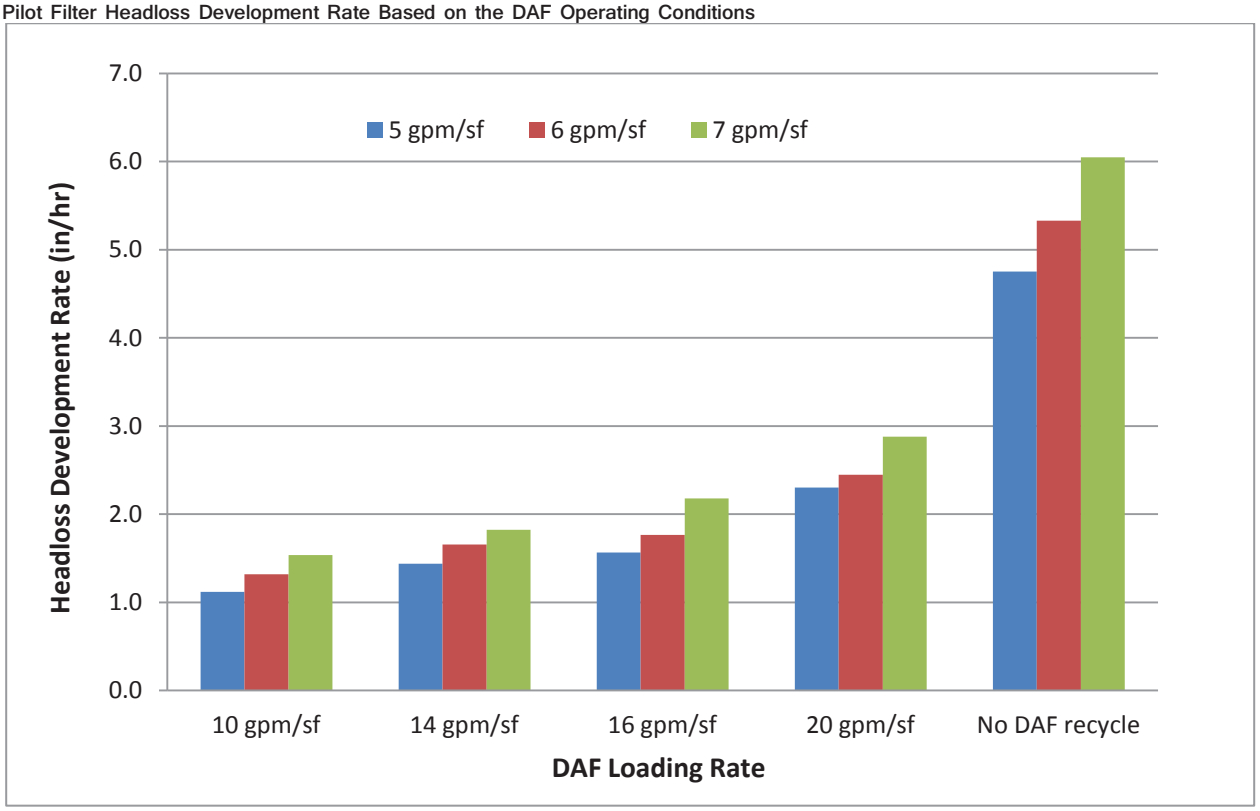
The pilot filter effluent turbidity during testing was recorded in 5-minute intervals; therefore, the filter ripening time was determined to the nearest 5-minute interval. It was found that it took between 0 and 15 minutes for the filter effluent to drop below 0.1 NTU. Most of the runs were able to obtain less than 0.1 NTU during the first 5 minutes. Comparatively, the full-scale plant filters had ripening times of 5 minutes or less From this limited data, and the DAF-reduction of particles in the filter influent, ripening times could increase slightly in the future but are not expected to

exceed 10 minutes. The difference between the current ripening time of 5 minutes versus a 10 minute ripening time, at a filter loading rate of 3.5 gpm/sf, results in a small reduction in the volume of water each individual filter produces of approximately 0.6 %.

4.3.4 Filter Headloss

During testing, filter headloss was monitored and the headloss development rate was determined on each filter on every run to provide a sense of how fast the headloss was built up at varied DAF and filter loading rates. Figure 4-8 shows the average rate for each filter during runs with the same DAF loading rates. It indicates that the rate of filter headloss development increased with increases in filter loading rate or DAF loading rates. The last set of columns on the figure show that when DAF recycle was turned off, the coagulated water fouled the pilot filters with a rate much faster.

FIGURE 4-8
Pilot Filter Headloss Development Rate Based on the DAF Operating Conditions



4.3.5 Particles Removal

Particles in the size larger than 2 μm were measured continuously. They are used as a surrogate for determining the likelihood of microbial breakthrough. Although particle counts and particle reduction are not regulated, many utilities have begun using particle counters to augment their turbidimeters to monitor filter effluent. Figures 4-9 and 4-10 show the average filter effluent particles counts and the log reduction of particles through the entire pilot process (including flocculation, DAF, and filters) in steady state for each run. These figures demonstrate that during the entire pilot testing, the particle counts in filter effluent averaged below 20 count/mL. In addition, the pilot system achieved greater than 2-log particle reduction during steady-state operation.

Particle counts also have value in observing the operation of filters as they approach and run through breakthrough. As mentioned previously, in most of the filter runs 100 counts/mL particle threshold value was exceeded before the headloss or turbidity threshold values were reached. Determining the filter run time and productivity based on this particle threshold provided a greater level of protection of the integrity of the process and finished water quality.

In every run, the particle breakthrough always occurred first for the filter with the highest loading (7 gpm/sf), then for the filter with the medium loading (6 gpm/sf). The filter with 5 gpm/sf loading had the latest particle

breakthrough. This can be demonstrated by one example shown in Figure 4-11. The full-scale plant historically terminates filter runs based on headloss of 8.2 feet. At this termination point, the particle counts are typically well below 100 particles/ml.

The pilot filter particle breakthrough occurring well before terminal headloss demonstrates there may be potential for additional gains in filter productivity beyond which was demonstrated in this pilot test, with additional pre-treatment optimization to enhance particle retention on the filters. The other conclusion is that the pumping of the DAF effluent to the filters may have altered the size distribution of particles in the filter influent, thereby having an effect on the particle retention in the filters.

FIGURE 4-9
Pilot Filter Effluent Particle Counts in Each Run

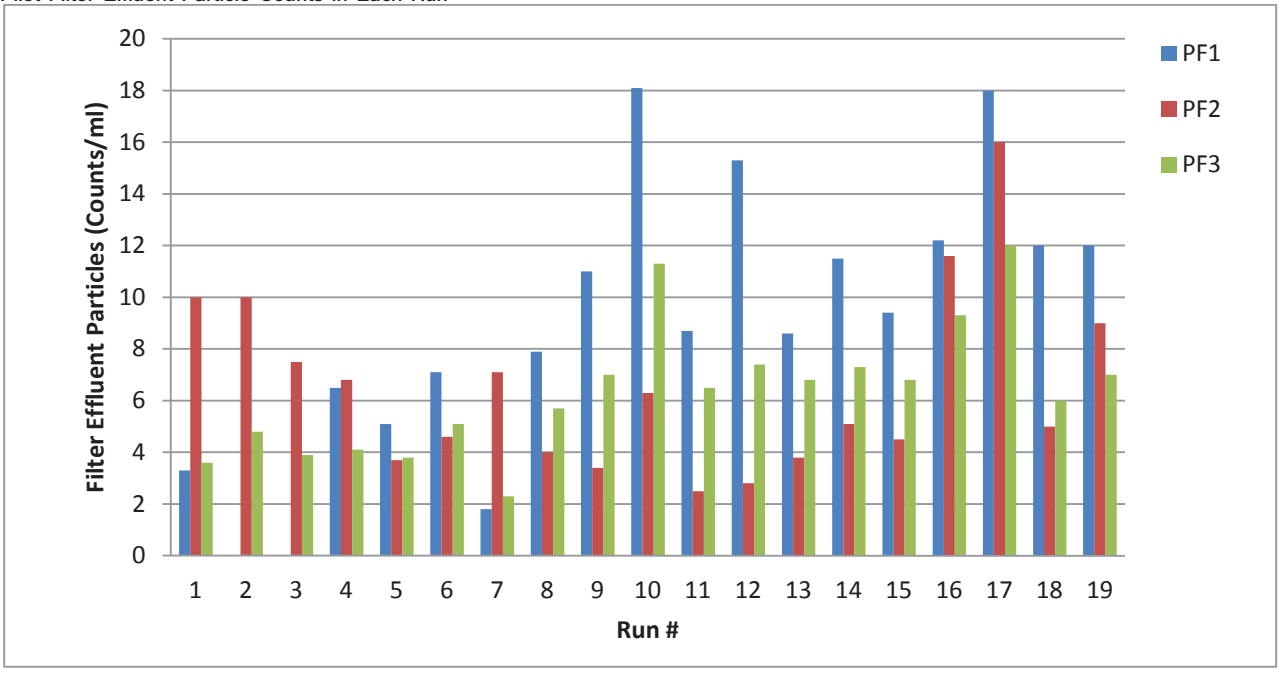


FIGURE 4-10
Particles Log Removal across the Entire Pilot Processes

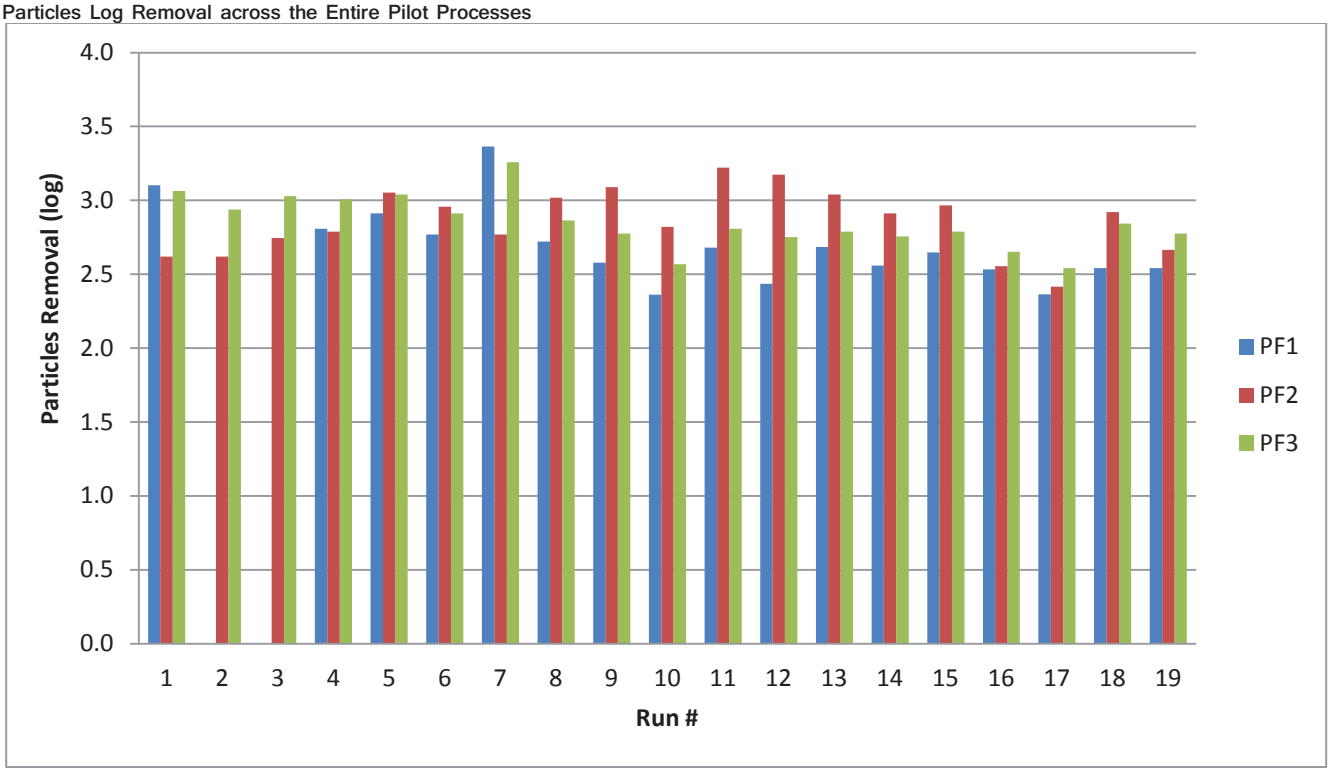
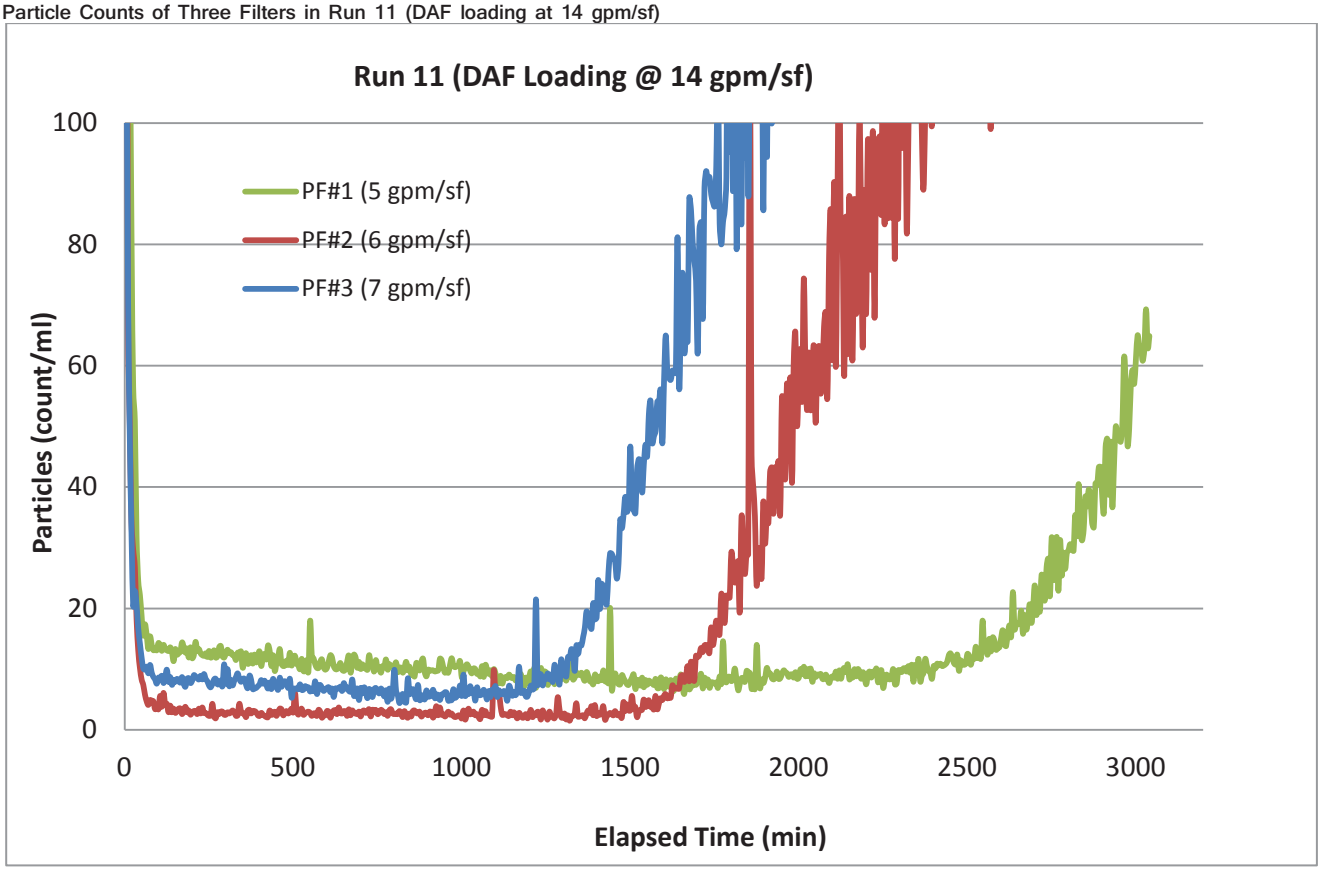


FIGURE 4-11
Particle Counts of Three Filters in Run 11 (DAF loading at 14 gpm/sf)



4.4 Simulated Distribution System Disinfection By-Products

TOC is a precursor compound. When it is combined with free chlorine used for disinfection, it can result in the formation of DBPs known as TTHMs and HAA5s. These compounds are regulated by the Washington Department of Health under the Stage 1 and Stage 2 disinfection by-product rule at concentrations of 80 parts per billion (ppb) and 60 ppb, respectively. The City has never exceeded these concentrations under the regulation's compliance method.

When TOC removal is enhanced, DBPs are typically reduced by two methods:

1. Less organic material in the treated water reduces the initial chlorine demand/decay, thereby making it possible in some cases to reduce the initial chlorine dose for disinfection and for distribution system residual.
2. Less organic material in the water results in less reactions to form DBPs

To determine the effects of the DAF system in removal of organics and the resulting DBPs, a sample of the filter effluent was taken on September 13, concurring with Run 17 of the pilot test, and the City's quarterly sampling for DBPs. The pilot filter effluent was shipped to CH2M HILL's Applied Sciences Laboratory to conduct SDS analysis. In this testing, a sample is dosed with free chlorine, and is held for prescribed times that correspond to water ages in the distribution system. At each time, the water is quenched of chlorine to stop the formation of DBPs and sent to the laboratory for determination of the DBP concentration.

For the SDS testing for the City, an initial dose of 1.5 mg/L of chlorine was chosen at 19 degrees Celsius and a pH of 7.4, with 1 day, 3 day, and 7 day holding times. The 1.5 mg/L dose of chlorine was chosen to ensure a minimum of 0.2 mg/L chlorine residual would remain after 7 days. While 7 days is excessive for the City's distribution system, it gives us a "worst-case" scenario to review.

Figures 4-12 and 4-13 show the results from the City's sampling and the laboratory SDS sampling. Some conclusions can be drawn from the data collected.

- Filter effluent TOC from the pilot filters was 1.09 mg/L against a TOC concentration on average of 1.3 mg/L from the plant filters. Therefore, the pilot filter effluent has less organic material to react.
- The SDS testing chlorine dose of 1.5 mg/L was significantly higher than the plant dose on the day of testing of 1.0 mg/L. Therefore, the testing is conservative in that more chlorine (approx 0.5 mg/L) may have been added than was required for a 3- to 4-day detention time.
- With the two variables above in consideration, a decrease was observed in TTHM formation of 25 percent at 1-day detention time between the current distribution system and the SDS sample. The 3-day SDS TTHM formations were at or below the 1-day detention time system samples.
- For HAA5s, the SDS samples were higher by 15 to 35 percent than the distribution samples. This can primarily be attributed to the biodegradation of HAA5s in the City's distribution system. This is evident in the small reduction in HAA5s from the Marietta (4-day) sample as compared to the 1- and 3-day system samples. In the SDS testing, this biological reduction could not be simulated.

FIGURE 4-12
Comparison of TTHM Results from the City's Sampling and the Laboratory SDS Sampling

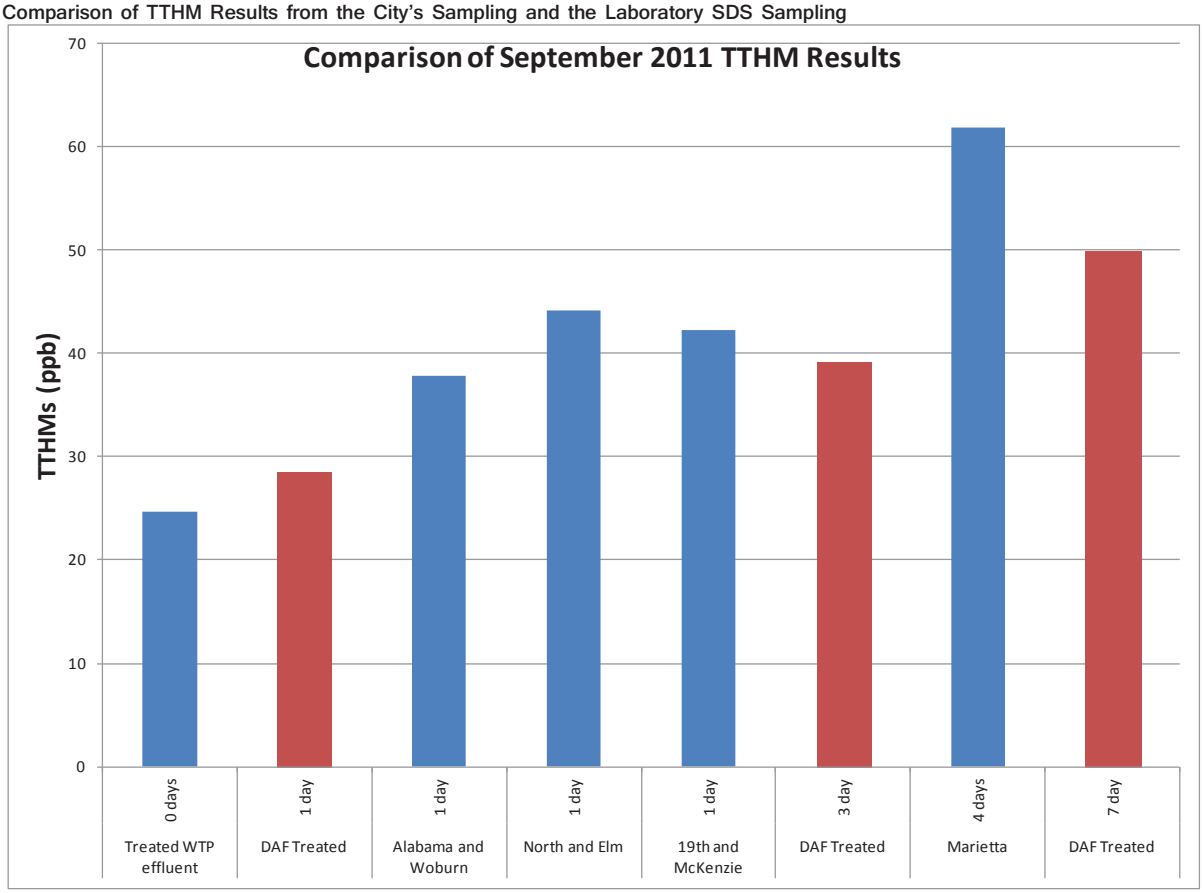
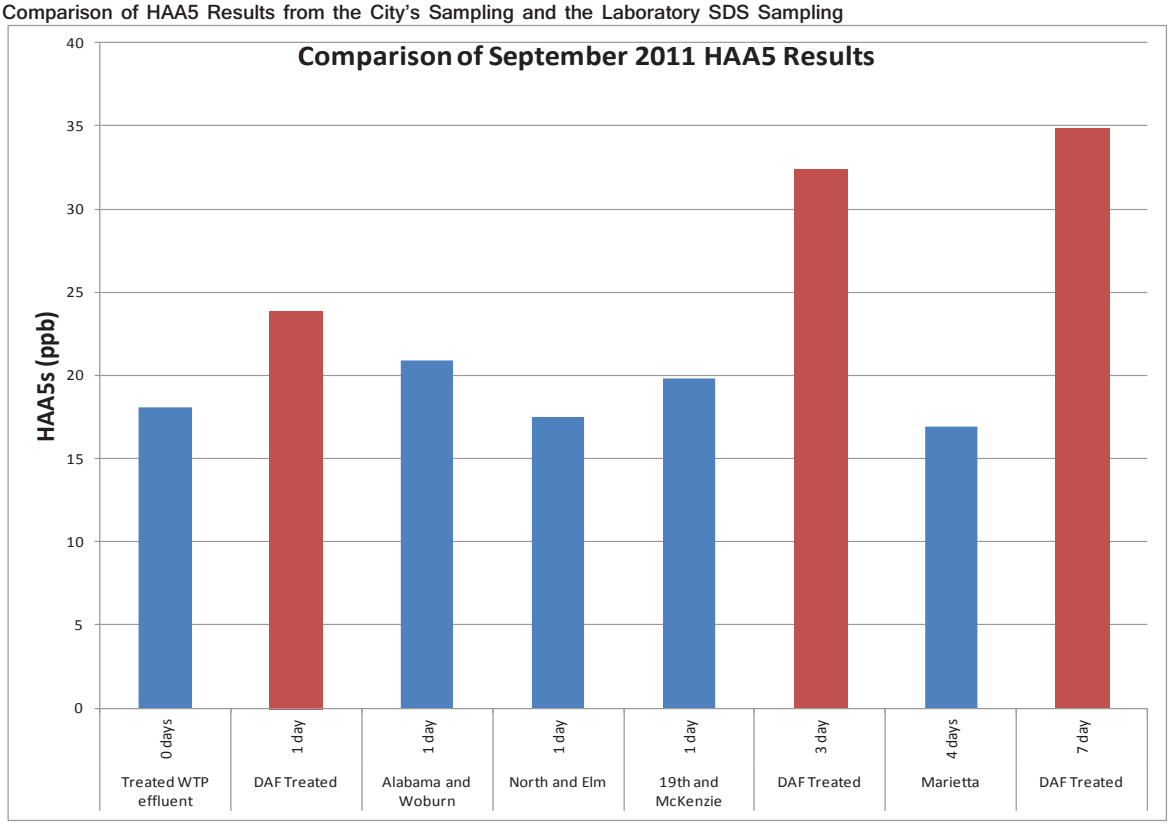


FIGURE 4-13
Comparison of HAA5 Results from the City's Sampling and the Laboratory SDS Sampling



4.5 Post-Pilot Filter Comparison

Two additional filter runs were conducted after the DAF and filter pilot testing. The purpose was to compare the performance of full-scale plant filters and pilot filters at the same loading rates. Three pilot filters were loaded at 5, 6, and 7 gpm/sf using the plant coagulated raw water – the same water feeding to the plant filters during normal operation. Two of plant filters were started at the same time and loaded at 5 and 6 gpm/sf. Particles, turbidity, and headloss were monitored.

The results of these two post-pilot tests were consistent. They showed the following:

- All pilot filter runs terminated on particle counts (over 100 count/mL), while both plant filter runs terminated on headloss (exceeded 8.2 feet of headloss). Based on these termination criteria pilot filters had 16 to 19 percent higher UFRV compared to plant filters, as shown in Figure 4-14. In another words, on average the pilot filters had 300 to 700 gal/ft^2 higher UFRV than the plant filters. The difference was within the reasonable range considering the scaling factor from the pilot filters to the plant filters. The other variable was that flocculated raw water gravity flows to the plant filters while it was pumped to the pilot filter. The shearing force imposed by the centrifugal filter feed pump may change the characteristics of the particles in the filter feed.
- The headloss buildup rates of the plant filters were higher than the rates of the pilot filters. As shown in Figure 4-15, in one of post-pilot runs, plant filter at 6 gpm/sf reached the headloss cutoff criterion (8.2 feet) after about 450 minutes of operation. It was followed by the plant filter operated at 5 gpm/sf, which had run time of approximately 660 minutes. Three pilot filters had slower headloss development.
- During the steady-state operation, the plant filter effluent particle counts were consistently lower than pilot filter effluent particle counts. Based on Figure 4-16, the plant filter effluent particle counts were always maintained below 12 count/mL, while the pilot filter effluent turbidity varied between 10 and 20 count/mL.
- Filter effluent turbidity levels were similar for pilot filters and full-scale plant filters. They ranged between 0.03 and 0.04 NTU throughout the testing.

FIGURE 4–14
Average UFRV of Plant Filters and Pilot Filters during Post-Pilot Filter Comparison Test

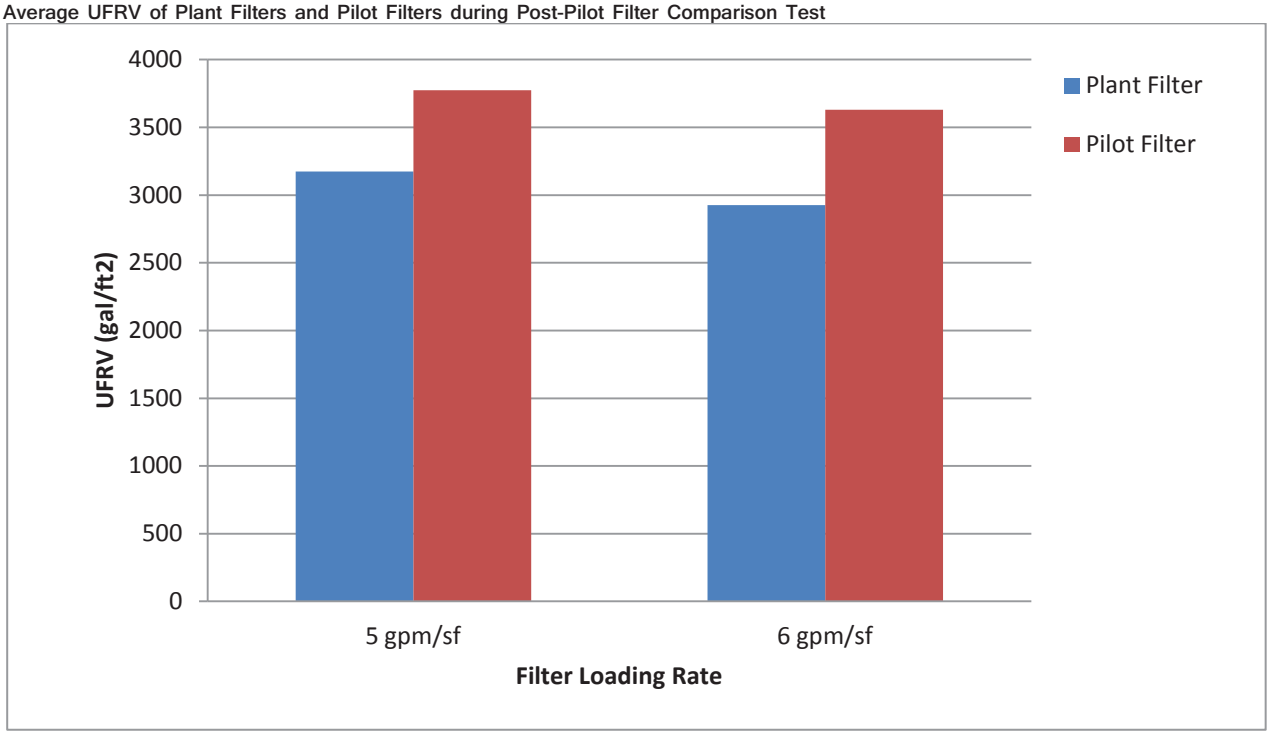


FIGURE 4-15
Filter Effluent Headloss during Post-Pilot Filter Comparison Test

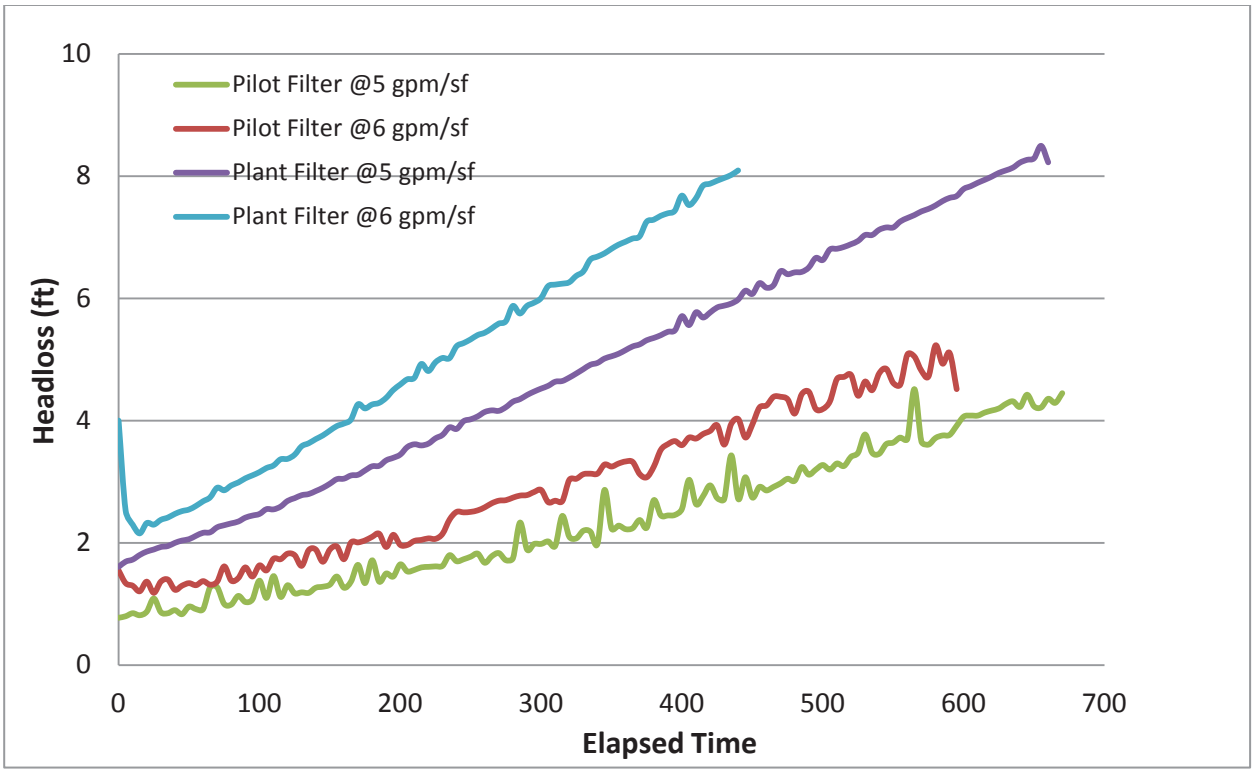
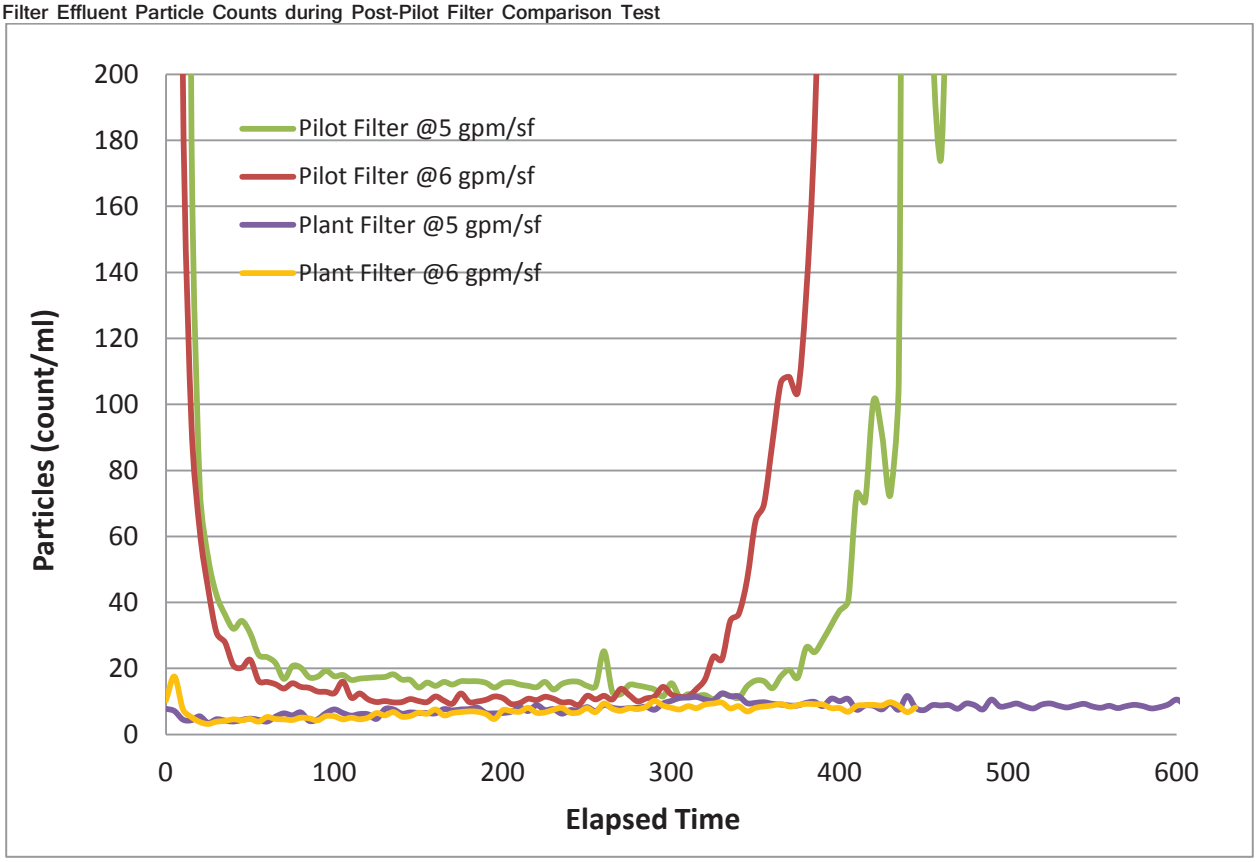


FIGURE 4-16
Filter Effluent Particle Counts during Post-Pilot Filter Comparison Test



SECTION 5

Conclusions

The pilot testing demonstrated that for the Lake Whatcom supply, DAF effectively removed algae, increased filter production, reduced TOC and color, and reduced the formation potential for TTHMs. In addition, specific results and conclusions are summarized below:

- The primary purpose of the pilot testing was to evaluate the performance of DAF on algae removal and filter production capacity. The testing was conducted during August and September to capture the most likely algal bloom period. The test demonstrated that DAF had exceptional performance at a wide range of loading rates. Besides algae removal, DAF also improved other water quality parameters, such as TOC, DOC, color and turbidity.
- Flocculation at 5 minutes detention time was adequate for DAF performance.
- There was no clear correlation between DAF loading rate and the DAF performance, or between DAF loading rate and pilot filter performance. Tests with DAF loading rate up to 20 gpm/sf were able to achieve UFRV close to or higher than 8,000 gal/ft^2.
- Pilot filters with DAF clarified water had superior performance during the testing than did the full-scale plant filters without DAF pretreatment – as measured by UFRV. The pilot filters had significantly higher UFRV (over 8,000 gal/ft^2) than did the plant filters, which had average UFRV values of approximately 3,000 gal/ft^2.
- The simulated distribution system test indicated that filtration with DAF pretreatment reduces the TTHM formation potential by over 25 percent.
- Table 5-1 summarizes the pilot test performance goals previously presented in the report. Results from the testing were added to the last column in the table to compare against the criteria and goals.
  - The 95% algae removal goal was established during planning phase based on observed performance of DAF in other high-algae water treatment applications. Meeting or not meeting this goal does not define success for the DAF process. Total algae removal through the DAF process ranged from 78 to 95 percent. This was slightly lower than the performance goal (>95 percent). However, the pilot testing was conducted with raw water algae at relatively low levels (< 12,000 cells/mL), which drove the percent removal lower than would have been anticipated for higher levels. Nevertheless, DAF-clarified water algae levels were consistently very low, which led to the superior UFRV performance summarized above. Improved UFRVs more directly indicate the success of the testing than algae reduction.
  - Clarified water turbidity varied between 0.2 NTU and 0.4 NTU in most runs (well below the performance goal), except in the first 20 gpm/sf run, where the clarified water turbidity was close to raw water. For this particular run accumulated floc was discovered to have clogged the turbidimeter and connecting tubing.
  - Pilot filter effluent turbidity during steady state was between 0.03 and 0.04 NTU, below the 0.05 NTU performance goal.
  - Although pilot filter effluent had higher particle count compared to the full-scale plant filters, pilot filter effluent particle counts during steady state were still lower than the performance goal of 20 count/mL. The particle reduction was between 2.4 and 3.4 logs, mostly at or above the 2.5 log reduction goal.
  - Pilot filter ripening time (the time needed to achieve a filter effluent turbidity of 0.1 NTU) was always less than the 15-minute performance goal. In most of the runs, the ripening time was less than 5 minutes.
  - Pilot filter UFRV was between 7,680 and 18,800 gal/ft^2. This well exceeded the evaluation criterion and approached or exceeded the performance goal of 8,000 gal/ft^2. This represents significant improvement of filter production in summer time.

TABLE 5-1
Performance Goals Compared to Actual Results for Whatcom Falls WTP Pilot Study

Sample Point	Parameter	Evaluation Criteria	Goal	Results
Clarified water	Total Algae Removal	--	>95% removal	78% - 95% [a]
Clarified water	Turbidity (steady-state)	<1.0 NTU	0.5 NTU	< 0.4 NTU [b]
Filter Effluent	Turbidity (steady-state)	<0.07 NTU	<0.05 NTU	0.03- 0.04 NTU
Filter Effluent	Turbidity spike (ripening)	0.2 NTU	<0.1 NTU	< 0.2 NTU
Filter Effluent	Particle counts (steady-state)	< 100 p/ml >2 μm	< 20 p/ml > 2 μm	< 20 p/ml
Filter Effluent	Particle removal (steady-state)	2-log	2.5 log	2.4 – 3.4 log
Filter Effluent	Ripening time	30 minutes	< 15 min. to <0.1 NTU	< 15 min
Filter Production	Unit Filter Run Volume	>5,000 gal/ft^2	>8,000 gal/ft^2	7,680 – 18,800 gal/ft^2

[a] The 95% algae removal goal was established during planning phase based on observed performance of DAF in other high-algae water treatment applications. Meeting or not meeting this goal does not define success for the DAF process. The pilot testing at Whatcom Falls WTP was conducted with raw water algae at relatively low levels (< 12,000 cells/mL), which drove the percent removal lower than would have been anticipated for higher levels. Nevertheless, DAF-clarified water algae levels were consistently very low, which led to the superior UFRV performance summarized in this table. Improved UFRVs more directly indicate the success of the testing than algae reduction.

[b] Except during Run 16 (the 20 gpm/sf DAF loading run referenced above) where turbidity was measured at 0.5 to 0.6 NTU. This was likely incorrect data due to the contamination of the turbidimeter.

Attachment A
Whatcom Falls Water Treatment Plant: Pilot Testing Plan for Dissolved Air Flotation

Whatcom Falls Water Treatment Plant:
Pilot Testing Plan for Dissolved Air Flotation

Draft for DOH Review

Prepared for

City of Bellingham, WA

June 2011

CH2MHILL

This Pilot Testing Plan was prepared under the direct guidance of a Professional Engineer certified in Washington State.

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3 Pilot Testing Plan for Lake Whatcom WTP
4 Sampling Program for Pilot Testing 1
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1 DAF Schematic
2 Pilot Testing System Schematic
3 Percent of Water Lost to Backwash vs. UFRV

Introduction

The City of Bellingham operates the Whatcom Falls Water Treatment Plant (WTP), a 24 million gallon per day (MGD) in-line filtration plant (with one filter out of service). The source water is Lake Whatcom, a large natural lake that in recent years has seen increases in algal counts that have affected the performance of the WTP during late summer. Most recently, in 2009, algal counts reduced the filter production to unacceptably low levels, resulting in mandatory water restrictions. Table 1 lists historic raw water quality for Lake Whatcom.

TABLE 1
Whatcom Falls Water Treatment Plant Raw Water Quality

Parameter	Units	Min	Average	Max
Temperature	Celsius	6	12	18
Turbidity	NTU	0.41	0.74	2
Alkalinity	mg/L as CaCO3	19.5	20.7	22.5
Hardness	mg/L as CaCO3	17.3	21.2	23
pH	S.U.	7.2	7.3	7.4
Conductivity	umohs/cm	57	60.6	75
Apparent Color	PtCo	13	14	15
TOC	mg/L	1.8	2.2	2.6
DOC	mg/L	1.8	2.1	2.3
UV254	1/cm	0.046	0.056	0.103
Iron	mg/L	<0.01	-	0.08
Manganese	mg/L	<0.001	-	0.012
Aluminum	mg/L	<0.010	0.06	0.098
Chloride	mg/L	<2	2.2	3
Sodium	mg/L	2	4.4	5
Sulfate	mg/L	3.6	7.4	10
Chlorophyll	ug/L	2	3.5	5.9
Algae	#/ml	0	-	100,000

The City is now in the process of evaluating several alternatives, including new treatment prior to filtration at the WTP to mitigate these impacts to the WTP. One of the potential treatment alternatives is dissolved air flotation (DAF). Because of the demonstrated successful performance of DAF throughout the municipal water treatment industry, the City is planning to pilot test DAF during the anticipated period of increased algae during the late summer of 2011. This pilot testing is planned for implementation in parallel with the overall evaluation by the City of treatment alternatives as well as non-treatment alternatives

for algae mitigation. The City will incorporate the results of the DAF pilot testing into the overall evaluation of alternatives to mitigate the impacts of algae in Lake Whatcom to the filters at the existing WTP.

This proposed testing plan was developed to guide DAF pilot testing as well as to solicit Washington State Department of Health (DOH) review, comment, and approval, which is required per WAC 246-290.

Background on DAF Technology

DAF was first used as a pretreatment for conventional granular media in South Africa and Scandinavia in the 1960s and became more widely used worldwide in the 1980s and 1990s. DAF is becoming more common in the U.S. because it provides a cost-effective alternative to conventional sedimentation when the contaminant material to be removed is more-easily floated than settled, as is the case with algae.

In DAF, the solids are separated out by floating the floc to the water surface, as opposed to settling to the bottom of the basin. The process introduces air bubbles at the bottom of the contactor to float the floc. The air bubbles are produced by reducing to ambient pressure a pressurized recycle water stream saturated with air. The "float" is scraped mechanically or removed hydraulically from the top of the reactor, and the clarified water is removed from a location well beneath the surface. A schematic of a typical DAF unit is provided in Exhibit 1. Note that the unit that will be tested at Whatcom Falls Water Treatment Plant does not include lateral draw-off piping for the clarified water.

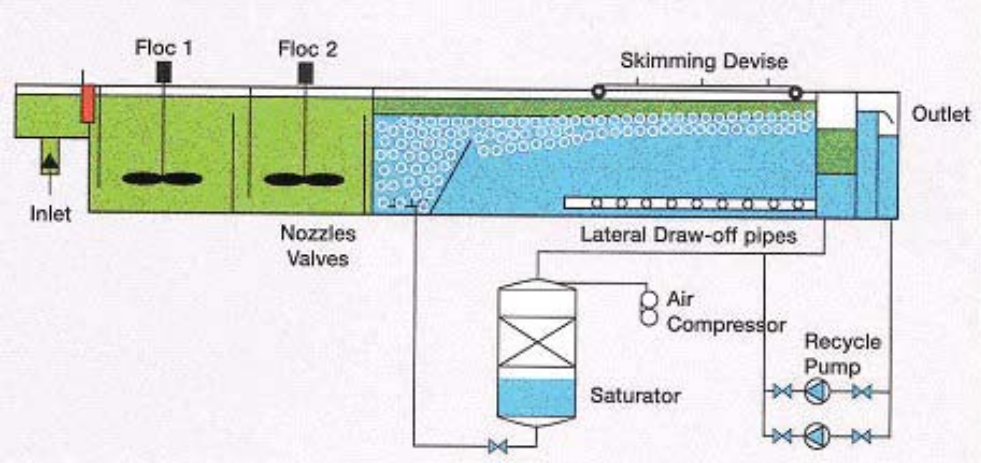


EXHIBIT 1
DAF Schematic

DAF is less costly than conventional flocculation-sedimentation for two reasons: the flocculation section is less than half the size of a conventional process. Detention times required for both flocculation and clarification are less than in conventional treatment. This results in a much smaller reactor than is possible for a conventional process. DAF also produces a more concentrated sludge than conventional treatment, although the sludge may contain entrapped air and need to be de-aerated. DAF requires much more energy input than conventional treatment and considerably more mechanical equipment to run the system.

High-rate DAF (above 8 gpm/sf surface loading rate) has been used in drinking water treatment in the last 10 to 15 years to increase the surface loading rate and decrease the footprint of the system. These High-Rate DAF systems have been designed up to 16 gpm/sf loading rate and run up to 20 gpm/sf in some situations. The largest High-Rate DAF facility in North America is located in Oradell NJ at 200 MGD.

Pilot testing DAF is necessary to help define the key unit process design parameters, predict effluent water quality (under various conditions), and simulate the effects of DAF pretreatment on filtration. The key parameters that need to be obtained in pilot testing include:

- Coagulant (dose and type)
- Coagulant polymer (dose and type; if it is needed)
- Flocculation time
- DAF surface overflow rate
- DAF sludge production and concentration

Pilot Testing Timing and Duration

The DAF pilot testing is planned for mid-August through mid-September of 2011 to coincide with the historical peak in blue-green algae growth. Blue-green algae have been identified as the dominant algae species that most-impacted filter performance at the WTP. The precise timing for the pilot testing may be shifted by a week or two based on the actual observed and measured growth in algae biomass over the summer.

The duration of the DAF pilot testing is anticipated to be approximately four weeks to achieve the pilot testing goals cited later in this testing plan, including demonstrating effective DAF performance under actual high-algae Lake Whatcom conditions during peak water demand periods. Since the primary objective of the use of DAF is for algae removal, testing during other seasons would not provide data necessary for the design basis of the pretreatment system.

Anticipated Full-Scale DAF Operations

While DAF is primarily a best available technology for algae removal, it can offer other benefits to water quality and WTP performance such as reduced total organic carbon (TOC) and longer filter run times even during periods when algae is not a significant impact to the existing treatment process (in-line filtration).

During the testing, we will gather data to determine what effects the DAF pretreatment will have on other water quality and performance parameters. Pending the review of this data and the City's long-term goals, a decision will be made as to whether full-scale DAF pretreatment would be for periodic, seasonal use only or as a full-time, year-round pretreatment step.

Pilot Test Goals

Pilot testing goals are important to establish at the beginning of testing to ensure the initial test plan, and adjustments during testing, are maintained with the end in mind. For the pilot testing on Lake Whatcom, there are four main goals:

1) Establish ability of DAF to effectively remove algae prior to filtration
2) Establish coagulant and polymer dosage rates required with DAF
3) Determine impacts of DAF pretreatment on filtration performance
4) Monitor other water quality parameters in filtered water

The evaluation criteria listed in Table 2 are the minimum values that need to be achieved in order for a pilot test run to be considered successful.

TABLE 2
Performance Goals for Whatcom Falls WTP DAF Pilot Study

Sample Point	Parameter	Evaluation Criteria	Goal
Clarified water	Total Algae Removal	--	>95% removal
Clarified water	Turbidity (steady-state)	<1.0 NTU	0.5 NTU
Filter Effluent	Turbidity (steady-state)	<0.07 NTU	<0.05 NTU
Filter Effluent	Turbidity spike (ripening)	0.2 NTU	<0.1 NTU
Filter Effluent	Particle counts (steady-state)	< 100 p/ml >2um	< 20 p/ml > 2 μm
Filter Effluent	Particle removal (steady state)	2-log	2.5 log
Filter Effluent	Ripening time	30 minutes	< 15 min. to <0.1 NTU
Filter Production	Unit Filter Run Volume	>5,000 gal/ft2	>8,000 gal/ft2

Piloting Setup

The pilot testing setup will include the following key components:

1) Pumping from WTP raw water pipeline
2) Roberts/Enpure Floc/DAF pilot with chemical feed systems
3) City's existing pilot filters

A schematic of the pilot testing setup is shown in Exhibit 2. Between the DAF effluent and pilot filters, there may be a need to pump to get the water to the flow split box on the pilot filters. This is being evaluated now. If a pump is needed, it will be a non-shearing type pump that will maintain the characteristics of any floc carried out of the DAF system to maintain filterability and replicate what we would expect in a gravity arrangement at the full-scale system.

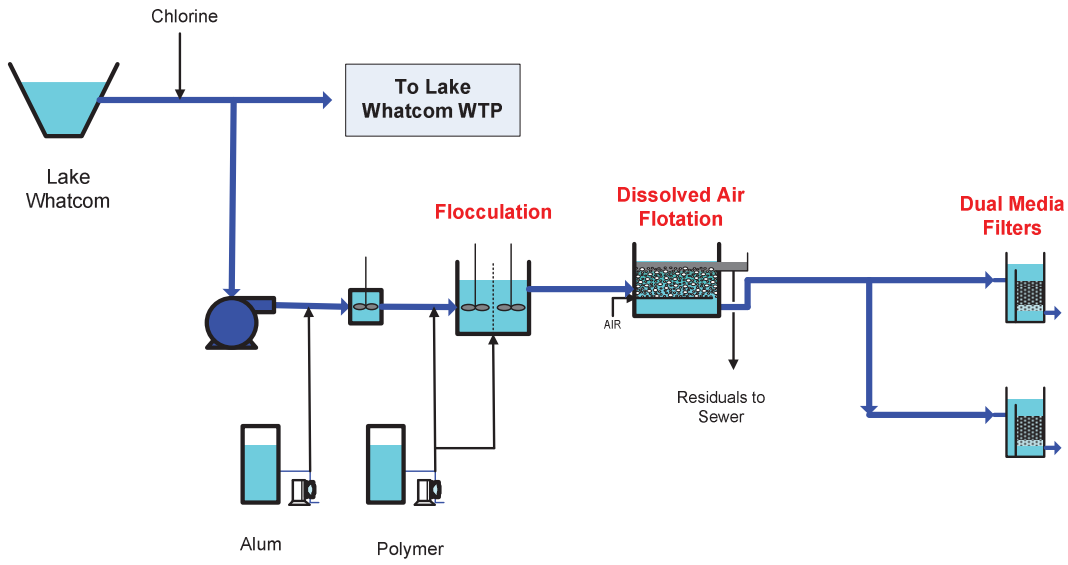


EXHIBIT 2
Pilot Testing System Schematic

Testing Methodology

The pilot testing will focus on the following areas:

- Flocculation Time
- DAF performance
- Filter performance
- Determination of filter run volume

Table 3 presents a summary of the pilot test runs that are expected to be conducted.

TABLE 3
Pilot Testing Plan for Lake Whatcom WTP

Week	Objective	DAF Loading Rate	Filter Loading Rate
1	Coagulant/Polymer Dosage	10 gpm/sf	5, 6, and 7 gpm/sf
2	Flocculation Times	10 gpm/sf	5, 6, and 7 gpm/sf
3	DAF at various loading rates	12, 16, 20 gpm/sf	5, 6, and 7 gpm/sf
4	Conduct optimal testing runs	Optimal	5, 6, and 7 gpm/sf

Coagulant and Polymer Dosage and Flocculation Time

Alum dosages will be tested at the bench scale during the startup week at doses between 5 and 20 mg/L to determine the best starting dose for pilot testing. A jar testing unit will be provided on-site by Roberts/Enpure for this purpose. Additionally, polymer may be tested to determine if it is necessary to enable the DAF system to meet the cited pilot testing

performance goals. The best starting dosages for Alum and polymer will be used in the first week to establish pilot scale performance and make adjustments as necessary.

During the second week of testing, the impact of flocculation time on the DAF performance will be assessed. Flocculation time will start at 10 minutes, and then be reduced to 7.5 minutes and then 5 minutes.

DAF Performance

During weeks 1 and 2 we will run DAF at 10 gpm/sf to demonstrate baseline performance of the unit at a historically-typical loading rate. During weeks 3 and 4, the DAF loading rate will be increased from 12 to 20 gpm/sf. The optimal coagulant/polymer dose will be used, along with the best performing flocculation time. The DAF recycle rate will be tested starting at 8 percent and will be increased to 10 and 12 percent at times to observe impacts on DAF effluent turbidity and particles. In week 4, the best performing DAF loading rate will be tested for a minimum of two consecutive runs to confirm performance.

Filter Performance

The existing pilot filters at the WTP will be utilized for this pilot study. The pilot plant consists of three individual filter columns with dimensions of 12" square (1 square foot) by 12' high (see photo).

The filter pilot plant is operated by a PLC which can operate the filters in one of three modes: 1) run, backwash, and stop, 2) run, backwash, and filter, 3) run continuously past breakthrough. On-line monitors on each filter can provide turbidity, and head loss data at 5-minute intervals. Particle counts can also be taken as necessary.

Two filter columns will be loaded with filter media matching the existing filters: 31 inches anthracite over 11 inches sand. The filters will be operated at different loading rates, between 5 and 7 gpm/sf.

Unit Filter Run Volume

The amount of water treated by each filter will be estimated by projecting the head loss development to 8 feet (current WTP terminal head loss parameter) or based on the actual turbidity breakthrough (0.07 NTU). The amount of water treated per unit area of filter between backwash events in gallons per square foot (gal/ft^2) is termed the unit filter run volume (UFRV). This filtration parameter will be used to evaluate filter performance within the context of DAF performance at various loading rates and to compare filters at different loading rate. For the purposes of this pilot testing, the UFRV should be greater than 5,000 gal/ft^2 to be classified as a successful filter run. Exhibit 3 shows that the relationship between the UFRV and the percent of produced water lost to backwash, based on typical municipal water treatment filtration performance, is not linear. As the UFRV increases to values greater than 8,000 gal/sf, the produced water lost to backwashing is minimized. Therefore, UFRV is an effective parameter for evaluating overall filtration performance.

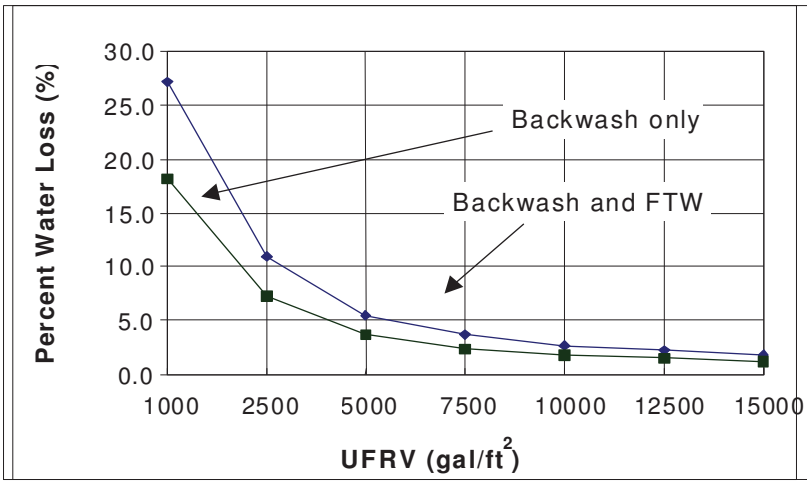


EXHIBIT 3
Percent of Water Lost to Backwash vs. UFRV

Data Collection and Analysis

Data to Be Collected

Table 4 provides a summary and frequency of the data that will be collected during the pilot testing. Table 5 provides a summary of the types of analyses to be performed for each sample. The City's existing HACH DR5000 in the WTP laboratory will be utilized for the on-site wet chemistry testing.

TABLE 4
Sampling Program for Pilot Testing 1

Parameter	Raw	DAF Effluent	Filtered Effluent
Turbidity	c	c	c
Color	2	2	2
pH	c	c	3
Alkalinity	1	1	1
Temperature	c	3	3
Particle Count	c (from plant)	c	c
Iron and Manganese (total and dissolved)[3]	1	1	1
Aluminum (total and dissolved)	2	2	2
Algal Counts	1 or 2	1 or 2	1 or 2
TOC	1 (each run)	1 (each run)	1 (each filter)
DOC	1 (each run)	1 (each run)	1 (each filter)
UV_{254}	c	c	3

[1] Numbers refer to the daily frequency samples are collected. "c" refers to continuous sampling.

[2] A run consists of a 24 to 36-hour run time. Approximately 4 runs will be completed each week.

[3] Iron and Manganese will be collected once or twice per week since very low concentration is anticipated.

TABLE 5
Analysis Type For Parameters

Parameter	Continuous	Laboratory (Off-Site)	On-Site Lab (Hach or EPA Method #)
Turbidity	X		
Color			X (8025)
pH	X		X (8156)
Alkalinity			X (SM2320B)
Temperature	X		
Particle Count	X		
Iron and Manganese		X	X (8008,8034)
Aluminum		X	X (8012)
TOC/DOC		X	
UV_{254}	X		X (10054)

Data Review

Pilot plant data will be summarized and reviewed on a weekly basis to ensure that the testing is on the right track, to prepare for the next week's planned pilot testing, to assess the need for modifications of the testing plan, and to assess whether the duration of the testing needs to be extended.

Pilot Plant Operation and Schedule

The DAF pilot plant will be set up starting on Monday August 1st. Testing is expected to start by Monday August 15th through Friday September 9th. The startup schedule for testing will be modified if weekly algal counts performed by the City show that the algal bloom is beginning earlier than expected.

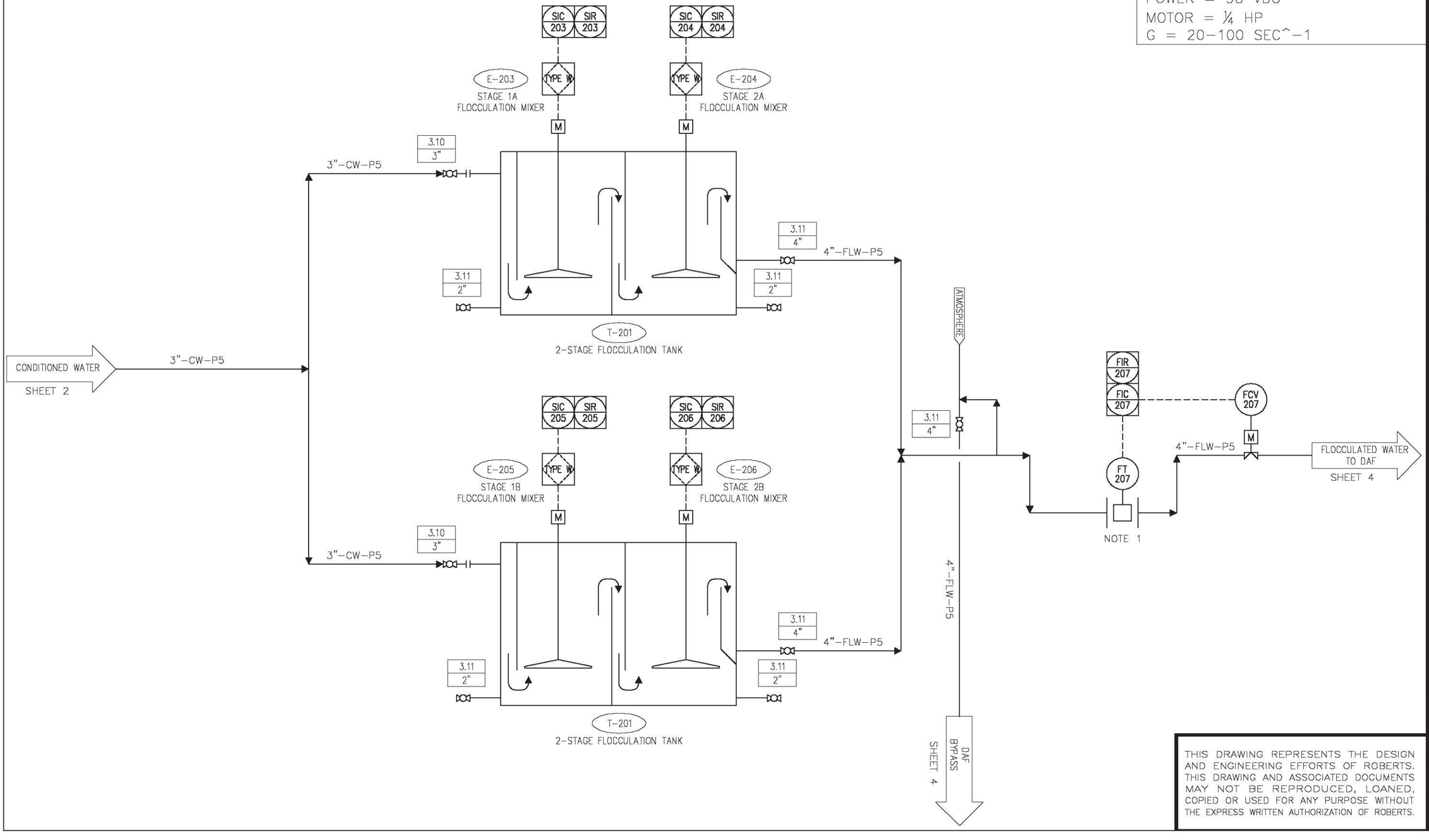
The pilot plant will operate 5 days a week, 24 hours per day for a period of 4 weeks. The pilot facilities will be staffed a period of approximately 8 hours per day. The automatic data logging equipment on each pilot trailer will allow for unstaffed operation at night.

Attachment B
Process Instrumentation Diagrams of Pilot Flocculation and DAF System by Roberts

NOTES:

1. ALLOW FIVE (5) STRAIGHT PIPE DIAMETERS UPSTREAM AND TWO (2) STRAIGHT PIPE DIAMETERS DOWNSTREAM OF MAGMETER. ALSO, ENSURE FULL PIPE FLOW.

T-201/T-202
2-STAGE FLOCCULATION TANKS
VOLUME = 393 GAL/TANK
HRT/STAGE = 2.5 MINS (@40 GPM)
MAT'L = HDPE
E-203/E-204/E-205/E-206
FLOCCULATION MIXERS
SPEED RANGE = 0-300 RPM
POWER = 90 VDC
MOTOR = ¼ HP
G = 20-100 SEC^-1



NO.	REVISION	DATE	BY

DRAWN MRB | DATE 5/12/11 | CHECKED GMC | DATE 6/6/11

ROBERTS WATER TECHNOLOGIES, INC.
DARBY PENNSYLVANIA 19023

PROCESS PIPING AND INSTRUMENTATION DIAGRAM SHEET #3
DAF PILOT PLANT

SCALE NTS

CONTRACT 51153-T

SECTION A

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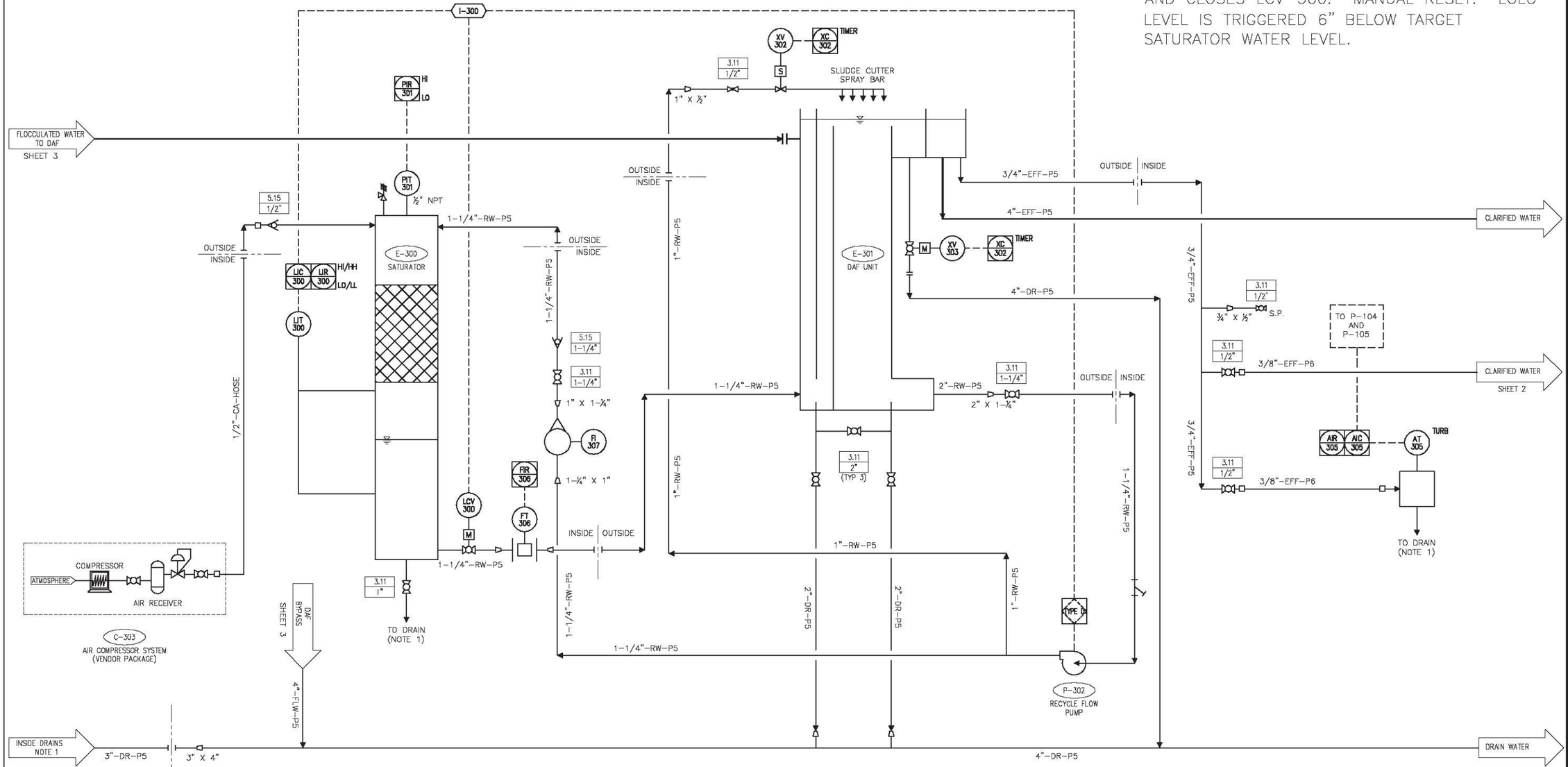
NOTES:

1. ALL DRAINS TO 3" COLLECTION MANIFOLD INSIDE CONTAINER OR 4" MANIFOLD OUTSIDE CONTAINER.
2. ALLOW FIVE (5) STRAIGHT PIPE DIAMETERS UPSTREAM AND TWO (2) STRAIGHT PIPE DIAMETERS DOWNSTREAM OF MAGMETER. ALSO, ENSURE FULL PIPE FLOW.

I-300: – HIHI LEVEL IN SATURATOR STOPS P-302 AND CLOSES LCV-300. MANUAL RESET. HIHI LEVEL IS TRIGGERED 6" ABOVE TARGET SATURATOR WATER LEVEL.
– LOLO LEVEL IN SATURATOR STOPS P-302 AND CLOSES LCV-300. MANUAL RESET. LOLO LEVEL IS TRIGGERED 6" BELOW TARGET SATURATOR WATER LEVEL.



E-300	E-301	P-302	C-303
SATURATOR	DAF UNIT	RECYCLE PUMP	AIR COMPRESSOR SYSTEM
OPERATING PRESS. = 65 PSIG PACKING MAT'L = JEAGER TRIPAK TANK MAT'L = 304 SS	SURFACE AREA = 4 SF DE-SLUDGE = HYDRAULIC MAT'L = 304 SS NOZZLES = 3	Q = 22 GPM TDH = 260' TYPE = CENTRIFUGAL POWER = 230/460/3/60 MOTOR = 1.5 HP	AIR FLOW = 2.6 SCFM @ 90 PSI MAX. PRESS. = 150 PSI POWER = 120 VAC MOTOR = 0.8 HP

NO.	REVISION	DATE	BY

DRAWN MRB
DATE 5/12/11
CHECKED GMC
DATE 5/27/11

ROBERTS WATER TECHNOLOGIES, INC.
DARBY PENNSYLVANIA 19023

PROCESS PIPING AND INSTRUMENTATION DIAGRAM SHEET #4
DAF PILOT PLANT

SCALE NTS
CONTRACT 51153-T
SECTION A
DWG NO. 10583-E4

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