City of Franklin, Tennessee

Integrated Water Resources Plan

Appendix A:

Modeling and Modeling Inputs

- City of Franklin IWRP, Phase II: Water Resources Demand Projections
- Phase I Results: Technical Memorandum

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Technical Memorandum

To: *City of Franklin IWRP Steering Committee*

From: *CDM*

Date: *December 8, 2010*

Subject: *City of Franklin IWRP, Phase II*
Water Resources Demand Projections

1.0 Introduction

The City of Franklin's unique setting–characterized by its commitment to preserve the City's history and heritage and its location in one of the fastest growing regions of the country–make Franklin a popular place to live and conduct business. From 1997 through 2010, the population of the City has nearly doubled and a continued trend of population growth is expected into the foreseeable future. This population growth has resulted from a combination of immigration, natural growth (births minus deaths), and annexation. Specifically over the past four decades, the City has annexed almost 24,000 acres. The rate at which Franklin continues to grow will depend on several factors—including how the nation and region rebounds from the current economic slowdown along with the availability and capacity to extend municipal services to undeveloped areas.

The City of Franklin Planning department has estimated that the population of Franklin would be at or near 78,000 within the City limits and the Urban Growth Boundary (UGB) by 2020. In its efforts to plan for future growth, the City has developed a land use plan to manage growth within the city and UGB. New construction continues in areas throughout the city and within Williamson County near the city borders. Commercial development continues in the northeastern portion of the city near the Cool Springs and McEwen Drive interchanges. The next major growth generator for Franklin is the land around the Goose Creek/I-65 Interchange at the southern boundary of the city. New residential development is planned and being constructed on the east, west and south sides of the city. With continued growth comes increasing pressure on City services and infrastructure. All areas of the City's infrastructure have experienced growth pressure, including roads and streets, water supply and treatment, wastewater treatment and disposal, and other City services. These ever-increasing demands

have led the City administration and staff to reevaluate their water resources from a long-term, holistic perspective that encompasses water supply and treatment, stormwater management, wastewater collection and treatment, and reclaimed water distribution. The process being used to accomplish this goal is the ongoing Integrated Water Resources Plan (IWRP).

The central points of the IWRP are reliably meeting the City's water needs and protecting the City's water resources, including the Harpeth River. The ecology and value of the river as a recreational resource is not only required component of this plan, but is desired as part of the quality of life in Franklin. At the same time, the City needs a reliable long-term source and infrastructure for drinking water and irrigation and sustainable solutions and infrastructure for reclaiming wastewater and discharging treated effluent when reclaimed water use is not feasible. In order to meet these challenges, the City looked to develop a long-range plan that establishes governing science and engineering principles, evaluates and debates alternatives, and builds broad consensus around a comprehensive set of sustainable, affordable actions that will provide for effective management of the City's water resources.

In order to develop the IWRP to address a 30-year planning period, estimates of future water resources demand are necessary to develop in-depth analyses of the costs and benefits of the project options identified in Phase I. Because the City of Franklin Planning Department has documented past growth and future population projections, the IWRP demand projections will be based upon existing population projection data and historical per capita demand data. If other previous demand information was available for given water resource systems, that information will be used in Phase II evaluations. This memorandum summarizes the population and water resource demand projections from previous studies; where previous demand projections have not been developed, this memorandum describes how demand projections have been developed for the purposes of Phase II of the IWRP.

2.0 Population Projections

The City of Franklin estimates population in their annual Development Report; the most recent version of that report was published in 2009. Future projections for the City of Franklin's population were based on a constant 2.9-percent growth rate from 2010 through 2030, in that report. For the purposes of developing 30-year projections for the IWRP, this growth rate has been extrapolated through 2040. Alternative population projections were gathered from several sources to compare to the City's Development Report projections. **Table 2-1** shows the City's population projections along with alternative high and low population projections for the City of Franklin at five year increments for the 30-year IWRP planning period of 2010 through 2040.

Table 2-1
Population Projections for City of Franklin

Year	Low Projection (Various Sources)	COF Projection (2009 Development Report)	High Projection (TACIR, 2010)
2010	57,471[a]	62,675[d]	62,742[e]
2015	65,878[a]	72,305[d]	79,028[e]
2020	78,000[b]	83,416[d]	90,970[e]
2025	83,174[a]	96,233[d]	103,980[e]
2030	90,506[c]	111,020[d]	118,731[e]
2035	94,094[c]	128,079[d]	135,353[e]
2040	97,681[c]	147,759[d]	154,303[e]

[a]*Population Projections for the State of Tennessee*, a joint publication of the Tennessee Advisory Commission on Intergovernmental Relations (TACIR) and The University of Tennessee Center for Business and Economic Research (CBER), 2003

[b]City of Franklin 2006 Annexation Study

[c]Extension of the observed trend from 1990 population data through the 2025 projection based on historical population data from the City of Franklin (Fiscal Year 2005-2006 City of Franklin Operating and Capital Budget)

[d]City of Franklin 2009 Development Report

[e]*Population Projections for the State of Tennessee*, TACIR and CBER, 2010

As shown in **Figure 2-1**, the projections diverge significantly after 2020 and result in very different population estimates for the end of the planning period in 2040. The primary cause of the divergence is the assumed growth rate: the City's projection uses a consistent 2.9-percent increase in population for the next 30 years, whereas the low projection assumes that the growth rate will level off after 2020. The growth rate after 2020, in the low projection, decreases from around 3 – 4-percent per year to 2-percent per year in 2030 and to less than 1-percent per year in 2040. The high projection, from the recently released TACIR data, assumes an initial growth rate of 5-percent from 2010 to 2015, which decreases to approximately 3-percent from 2020 through 2040.

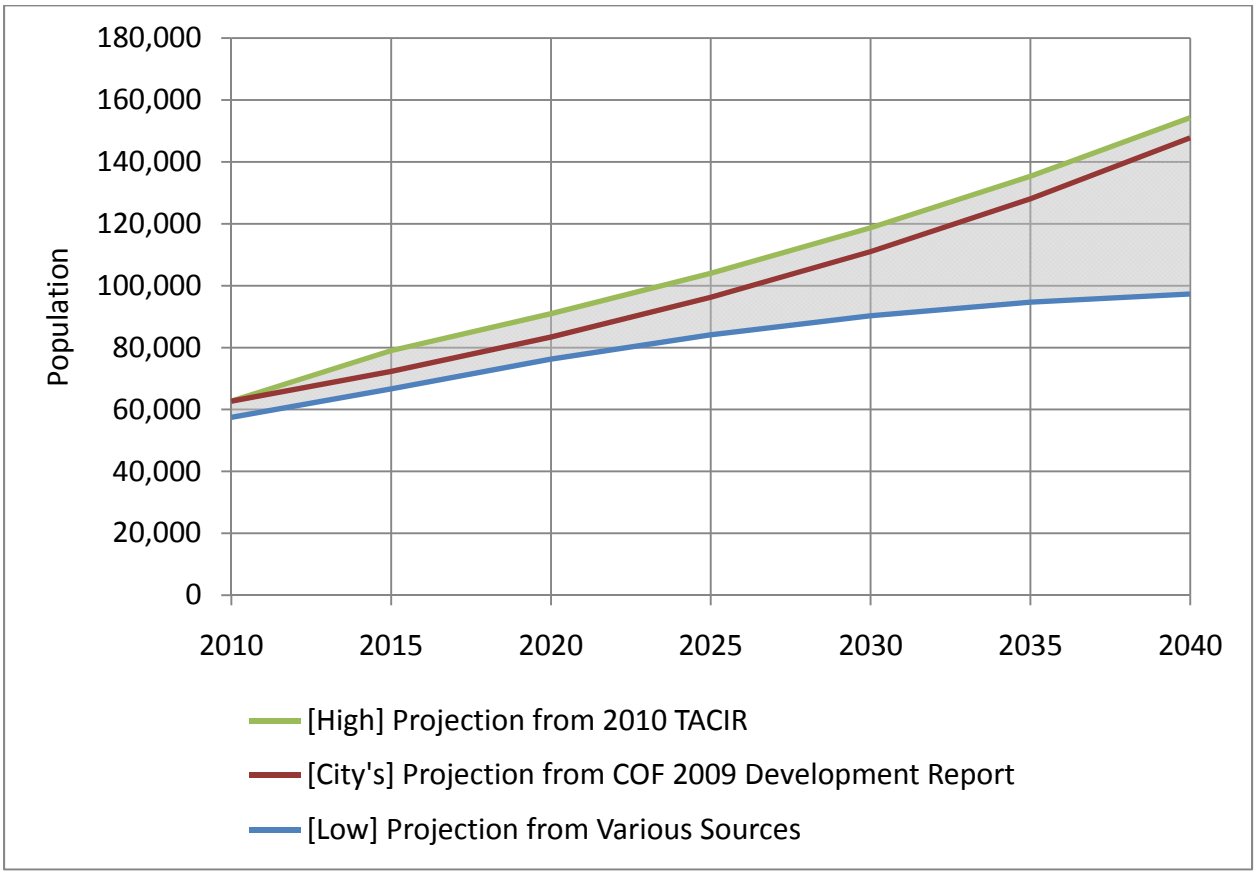

Figure 2-1
Population Projections for the City of Franklin

3.0 Water Demands

In June 2008, Jackson Thornton Utilities conducted an independent evaluation of a previous feasibility study on upgrading the Franklin Water Treatment Plant capacity. The original study included total annual water demand projections through 2038, which were reviewed in the evaluation report. The demand projections are based on the actual 2005 monthly water use values, and included a 0.18 million gallons per day (mgd) per year growth rate from 2005 to 2020, and a 0.09 mgd per year growth rate beyond 2020. This demand scenario generally follows the low population projection trend previously described in Section 2. A growth rate of 0.18 mgd is approximately 3-percent of the average daily demand in 2005; a growth rate of 0.09 mgd is approximately 1-percent of the average projected demand in 2020.

Alternative water demand projections were developed by adjusting the feasibility study projections proportionally to the City's population projection and the TACIR population projection. **Table 3-1** and **Figure 3-1** show the high and low water demand projections for the planning period, 2010 to 2040.

Table 3-1
Water Demand Projections for City of Franklin

Year	Low Demand Projection in million gallons per day	COF Demand Projection in million gallons per day	High Demand Projection in million gallons per day
2010	6.6	7.2	7.2
2015	7.5	8.2	8.9
2020	8.4	9.2	10.1
2025	8.9	10.2	11.0
2030	9.3	11.5	12.3
2035	9.8	13.3	14.0
2040	10.2	15.6	16.3

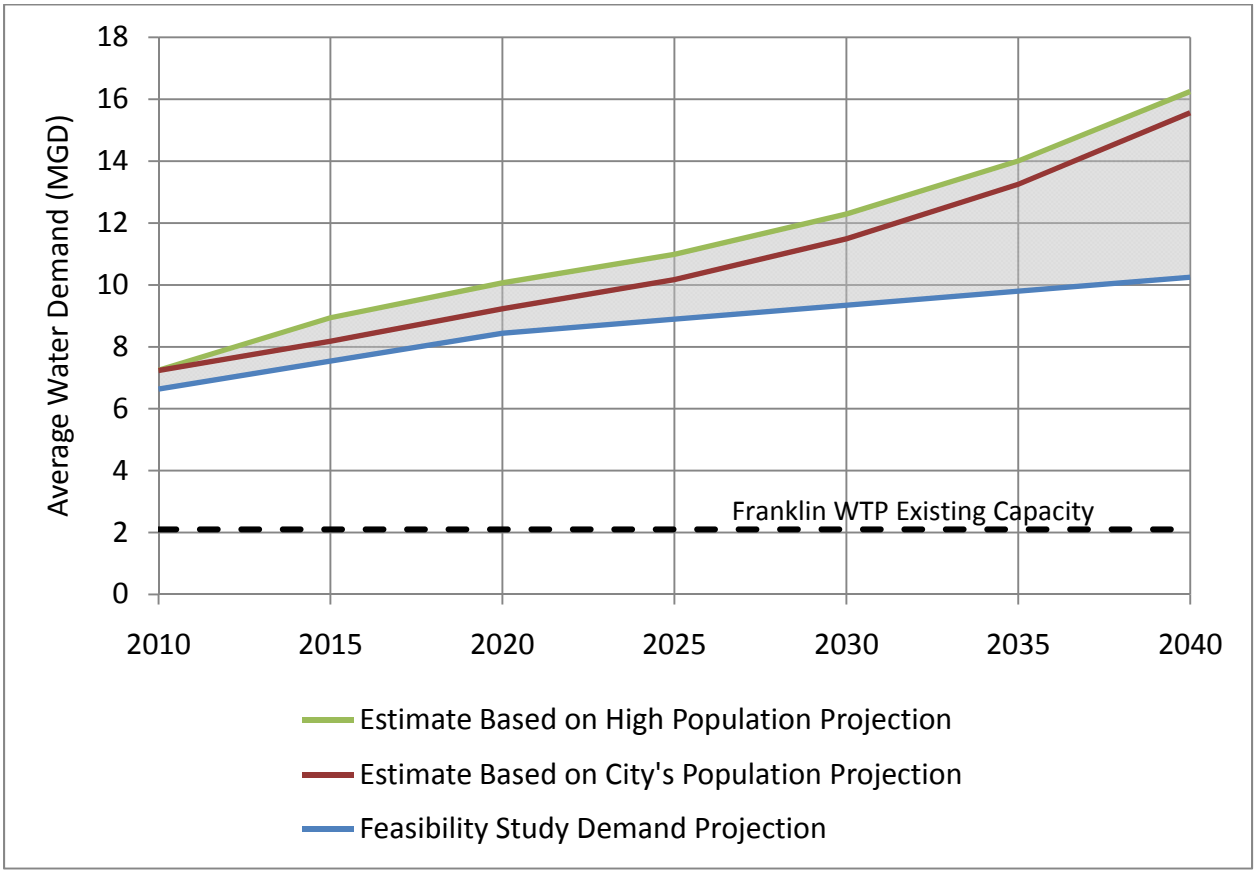

Figure 3-1
Water Demand Projections for the City of Franklin

Seasonal variability in water use will cause daily demand to fluctuate, often resulting in greater demands in dry summer months when residents and businesses use significant volumes of water for landscape irrigation. Seasonal variations were developed during Phase I of the IWRP process and are explained in detail for the various water use sectors in Section 4 of the Phase I Report. **Figure 3-2** shows the projections for average water demand with shaded bands

representing the potential high and low seasonal demands. For comparative purposes, the values shown are normalized to daily demands (mgd) however the bands represent the average demand for the highest and lowest month in the year (not the greatest expected demand in a single day). The graph shows the estimated water demands based on the low population projection and based on the City's population projection.

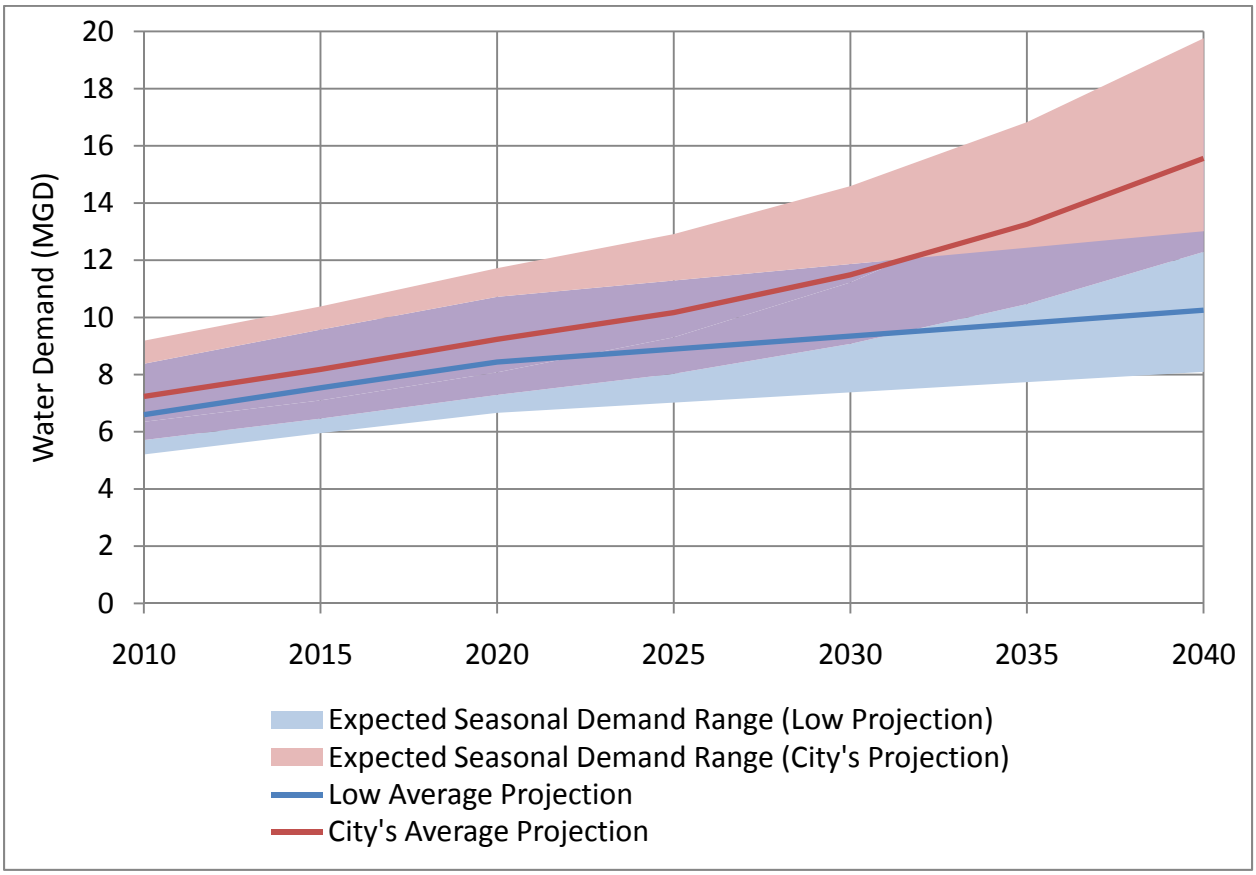

Figure 3-2
Water Demand Projections with Seasonal High and Low Estimates

4.0 Wastewater Demands

Wastewater demand projections were developed using the most recent ten years of wastewater influent data with population to estimate wastewater generated per person per day. While the sanitary sewer service area may not have covered all of the City of Franklin's population over the past 10 years, it was assumed for this estimate, that the growth rate of sewer customers is proportional to the growth rate of the City's population. An average of 150 gallons of wastewater generated per capita per day (gpcd) from 1999 through 2009 was calculated based on the reported average monthly wastewater influent. Calculations were performed using population as shown in Table 2-1 resulting in a minimum of 120 and maximum of 190 gpcd; 150 gpcd was applied to the high and low population projections, previously discussed in Section 2 to develop average daily wastewater demand projections shown in **Table 4-1** and **Figure 4-1**.

Table 4-1
Average Daily Wastewater Demand Projection

Year	Low Projection in million gallons per day	COF Projection in million gallons per day	High Projection in million gallons per day
2010	8.6	9.4	9.4
2015	10.0	10.8	11.9
2020	11.4	12.5	13.6
2025	12.6	14.4	15.6
2030	13.5	16.7	17.8
2035	14.2	19.2	20.3
2040	14.6	22.2	23.1

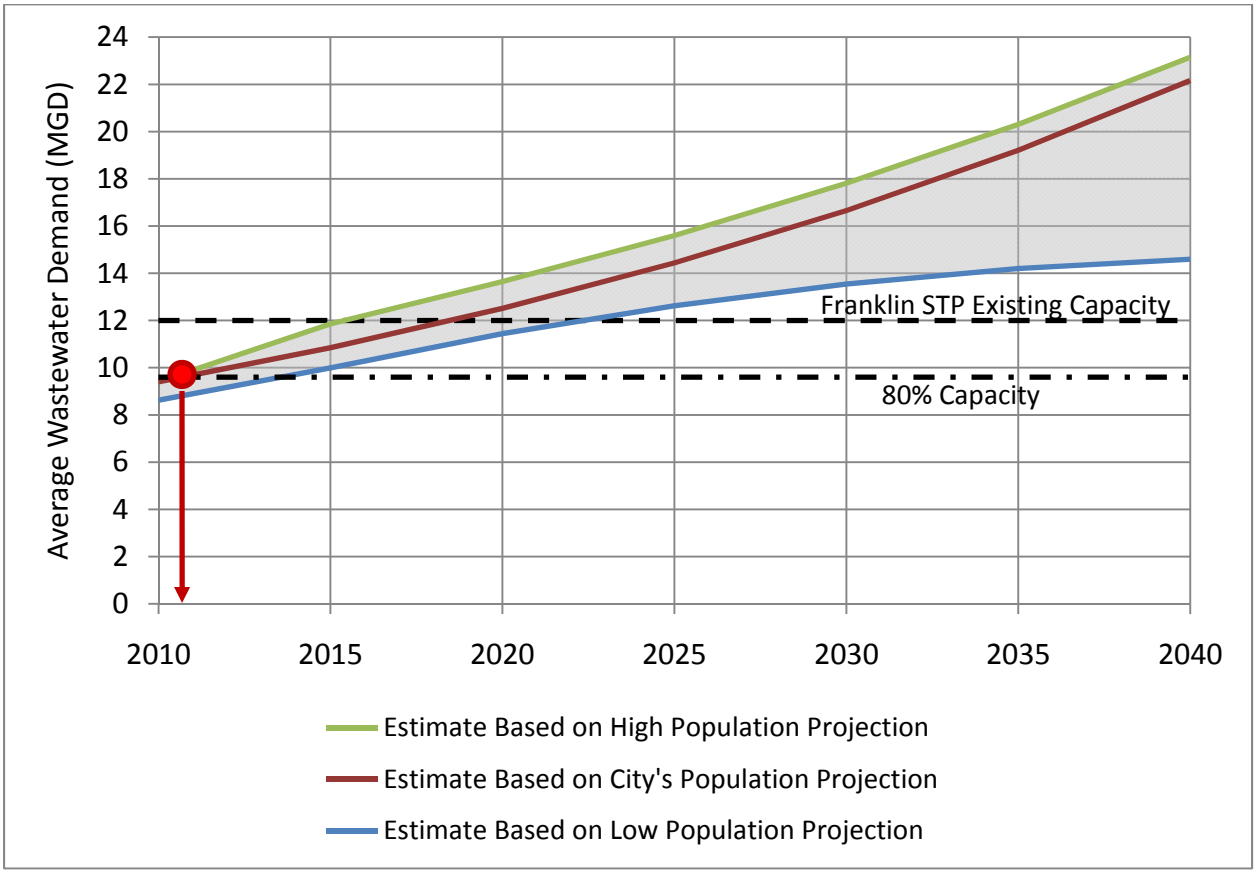

Figure 4-1
Wastewater Demand Projections for the City of Franklin

Wastewater demand projections are greater than water demand projections because each utility has different service areas and because of inflow and infiltration (I/I) to the wastewater collection system. This has been reported in historical data; the reported total water demand in 2005 was 2,282 million gallons while the total influent to the wastewater treatment plant was

reported to be 2,371 million gallons (based on records of average monthly flow). As part of the integrated modeling performed for Phase I of the IWRP, the I/I to Franklin's wastewater collection system was estimated to be as high as 4.4 mgd (Section 4 of the Phase I Report). Additionally, Franklin's calculated wastewater flow of 150 gpcd is greater than domestic wastewater flow specified for facility design in standard references such as the Ten States' Standards (100 gpcd) or in the WEF MOP 8 (70 - 100 gpcd).

Seasonal variations in water use, rainfall, and groundwater table elevation will cause wastewater demand to fluctuate throughout the year. Seasonal variations were developed during Phase I of the IWRP and were based on available wastewater influent data, water use data, and seasonal rainfall totals (Section 4 of the Phase I Report). **Figure 4-2** shows projections for average wastewater demand, with shaded bands representing potential high and low seasonal demands. For planning purposes, values are normalized to daily demands (mgd); however, the bands represent the average demand for the highest and lowest month in the year and not the greatest demand in a single day. Wastewater demand estimates shown in Figure 4-2 are based on the low population projections and the City's population projections.

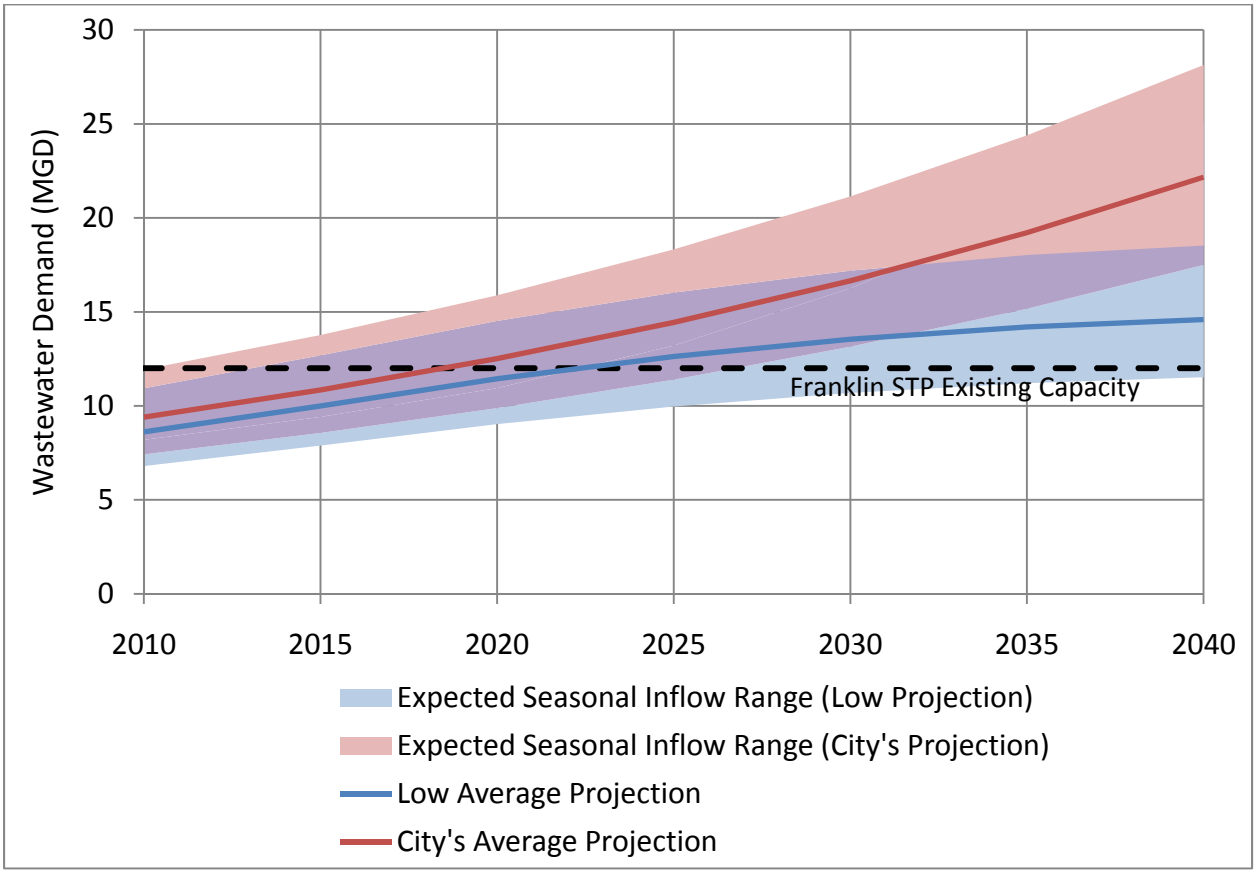

Figure 4-2
Wastewater Demand Projections with Seasonal High and Low Estimates

5.0 Reclaimed Water Demands

Demand projections for reclaimed water were calculated during Phase I of the IWRP based on values in the 2009 Reclaimed Water System Master Plan. This document lists various potential users and potential demands, which were aggregated into water use sectors (residential, commercial, and recreational); the demands were projected over the planning period. The growth of reclaimed water demand is specifically tied to the timeframe during which infrastructure improvements are implemented to supply those customers, and is not based on past reclaimed water or non-essential use patterns. The City predicts that there is an untapped demand for reclaimed water and extrapolating historical reclaimed or non-essential water use into the future would likely under predict the actual demand once the infrastructure and the resource itself are fully available. **Table 5-1** and **Figure 5-1** show the projected average reclaimed water demands by projection year and water use sector.

Table 5-1
Average Daily Reclaimed Water Demand Projection

Year	Reclaimed Water Demand in million gallons per day			
	Residential	Commercial	Recreational	Total
2010	0.5	0.7	1.9	3.1
2015	0.5	0.7	1.9	3.1
2020	0.8	0.9	2.0	3.7
2025	1.2	1.2	2.1	4.4
2030	1.9	2.0	2.1	6.0
2035	2.7	2.7	2.1	7.5
2040	3.5	3.5	2.1	9.1

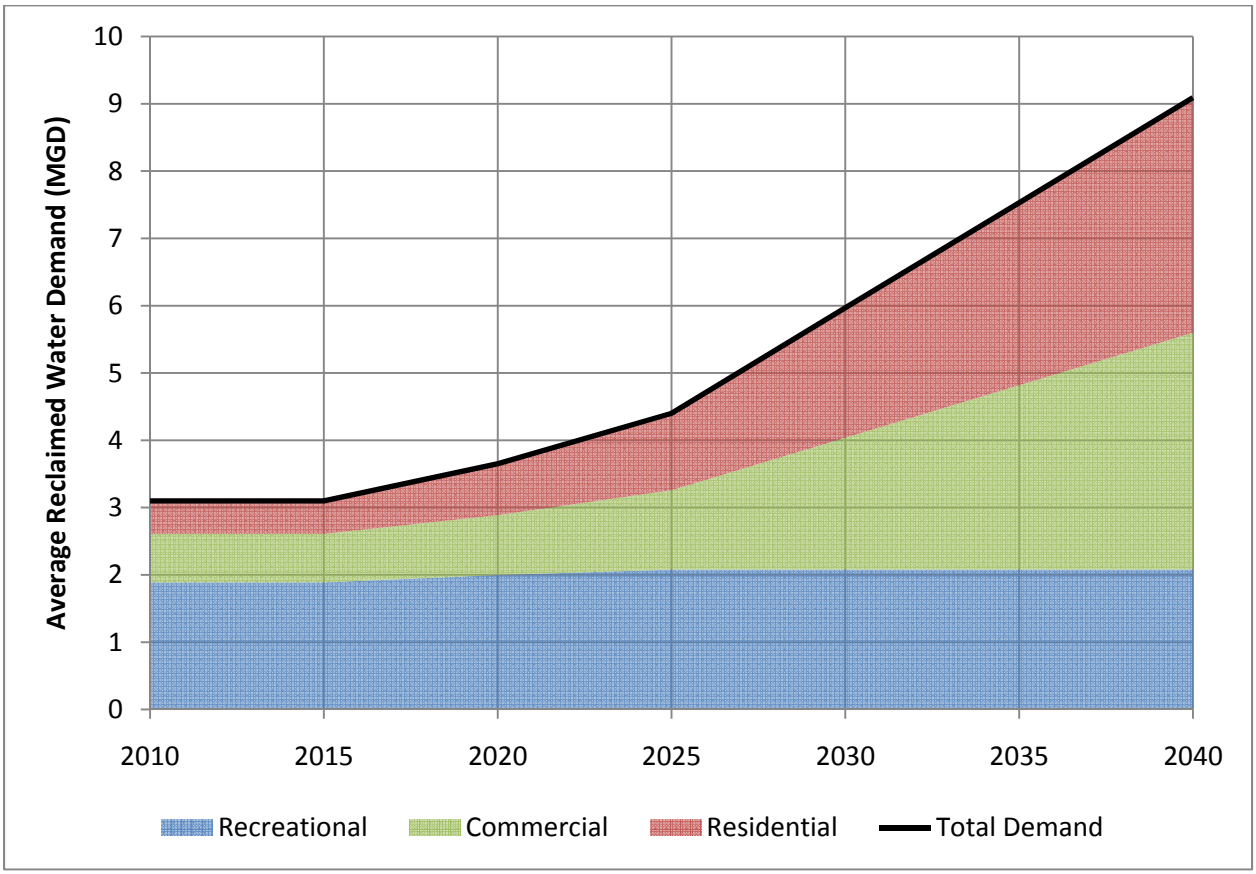

Figure 5-1
Reclaimed Water Demand Projections for the City of Franklin

There are large seasonal variations in reclaimed water demand, considering that most reclaimed water use is for irrigation in the dry summer months. The monthly variations were developed using records of wastewater effluent sent to reuse and discussions with IWRP Stakeholders and the City of Franklin staff. **Figure 5-2** shows the monthly variation in projected reclaim demands. Residential and commercial demands share the same seasonal pattern, while recreational demands for reclaimed water (including golf courses) tend to vary more drastically over the year. The graph shows the estimated monthly demands for reclaimed water compared to the projected annual average demand over the entire year. The projected monthly demand for any month in the planning period can be calculated by multiplying the percentage on the y-axis of Figure 5-2 with the annual average demand shown in Table 5-1 and Figure 5-2. Note that reclaimed water storage would be required to meet the peak seasonal demand.

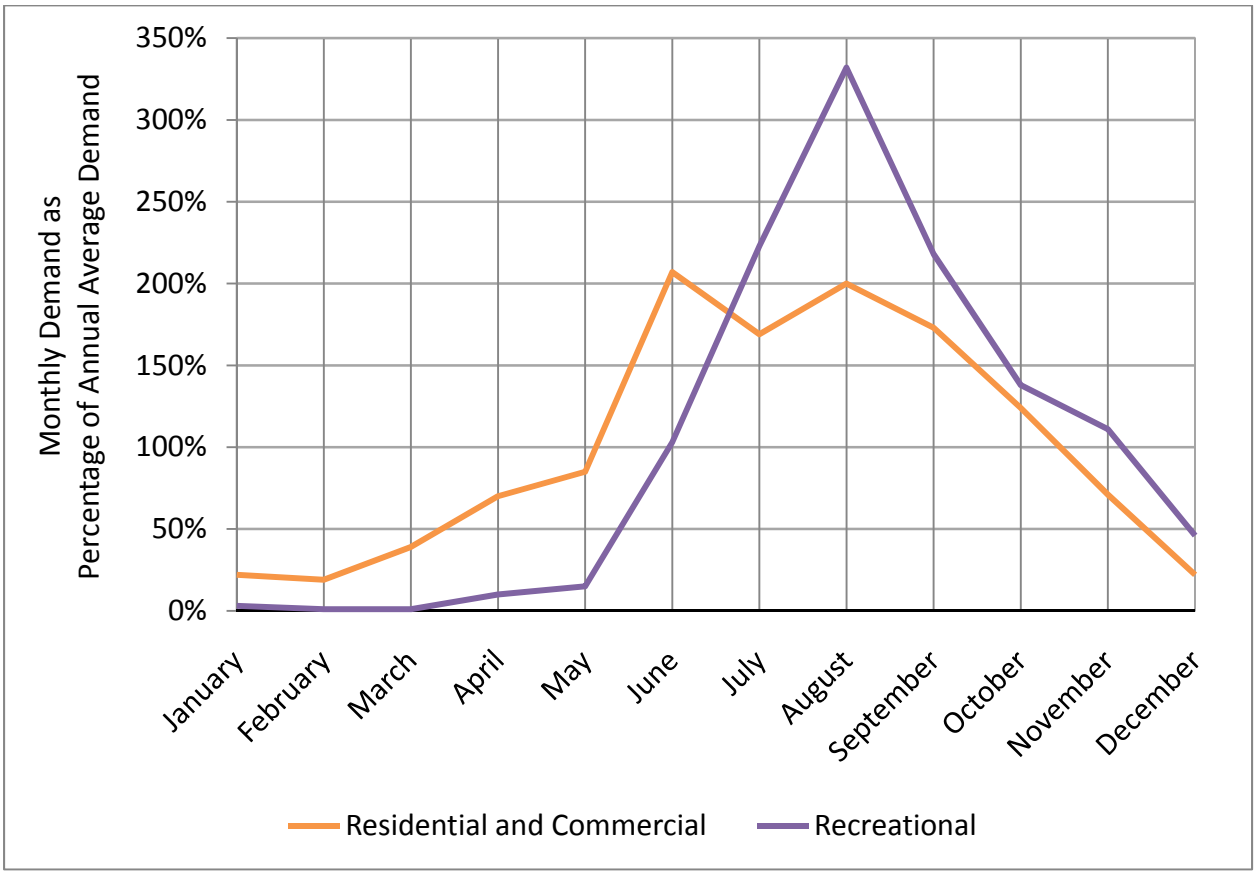

Figure 5-2
Reclaimed Water Demand Projections for the City of Franklin

6.0 Summary

The projected City of Franklin population, water demands, wastewater demands, and reclaimed water demands are summarized in **Table 6-1**. High, medium, and low demand projections for population, water demands, and wastewater demands are based on multiple data references for population growth projections. The low projection values were developed assuming a conservative growth rate in the City of approximately 3- to 4-percent population growth per year from 2010 to 2020, then approximately 2-percent per year from 2020 to 2030, and approximately 1-percent per year from 2030 to 2040. The City's (medium) projection values assume a constant growth rate of approximately 3-percent per year during the planning period (2010 to 2040). The high projection values assume a 5-percent per year growth rate until 2020, which decreases to about 3-percent per year for 2020 to 2040. For each year, the top line shows the low projected value, the middle line shows the value based on the City's 2009 population projections, and the bottom line shows the high projected value.

Table 6-1
Summary of Average Daily Water Resource Demand Projections

Year	Source	Population	Average Water Resource Demand in million gallons per day		
			Water	Wastewater	Reclaimed Water
2010	a	57,471	6.6	8.6	3.1
	b	62,675	7.2	9.4	
	c	62,742	7.2	9.4	
2015	a	65,878	7.5	10.0	3.1
	b	72,305	8.2	10.8	
	c	79,028	8.9	11.9	
2020	a	78,000	8.4	11.4	3.7
	b	83,416	9.2	12.5	
	c	90,970	10.1	13.6	
2025	a	83,174	8.9	12.6	4.4
	b	96,233	10.2	14.4	
	c	103,980	11.0	15.6	
2030	a	90,506	9.3	13.5	6.0
	b	111,020	11.5	16.7	
	c	118,731	12.3	17.8	
2035	a	94,094	9.8	14.2	7.5
	b	128,079	13.3	19.2	
	c	135,353	14.0	20.3	
2040	a	97,681	10.2	14.6	9.1
	b	147,759	15.6	22.2	
	c	154,303	16.3	23.1	

[a]Low projection (various sources, see Table 2-1)

[b]City's projection (2009 City of Franklin Development report)

[c]High projection (2010 TACIR Report)

Data presented in Table 6-1 for population and water resources demands have been developed for a 30-year planning period; this information will be used during in-depth analyses of the costs and benefits of the project options identified in Phase I. The data have been identified or developed from previous studies as described above, such that efforts for developing cost/benefit information is based on and coordinated with the City of Franklin Planning documents, as well as other infrastructure studies conducted for the City.

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City of Franklin, Tennessee

Integrated Water Resources Plan

Phase I Results

Technical Memorandum

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Executive Summary

The City of Franklin has completed Phase I of an Integrated Water Resource Plan (IWRP) that incorporates potable water, wastewater, reclaimed water, and stormwater into a long-term plan that identifies infrastructure improvements and policy recommendations to meet the City's needs and customer requirements.

Historically, as in most communities, planning for these separate utilities in Franklin has been conducted independently. However, based on results of IWRP studies throughout the United States, the City realized that there are three fundamental advantages of integrating the plans for water resource management:

- comprehensive understanding of the impacts of decisions over all aspects of water management;
- cost savings to the City and ratepayers; and
- common means—in this case, the Harpeth River—to measure progress and, ultimately, success.

The Harpeth River and its watershed are affected by almost every water management decision in Franklin. It currently provides drinking water, receives wastewater, and conveys stormwater away from the City; and its flows and water quality are affected by the amount of water recycled for other beneficial uses within the City. As such, it was identified early in the IWRP process as the principal means of evaluating the benefits and/or impacts of water management alternatives, though it is not the only means.

Before the effectiveness of a plan can be measured, there must be goals or standards against which it can be assessed, based on common interests of the community and other interested parties. A series of workshops was held with regional stakeholders to identify project objectives, alternatives, and performance measures for success of Franklin's IWRP. Participants were associated with a diverse group of interested organizations, including:

- City of Franklin
- Regional water providers
- Harpeth River Watershed Association
- Tennessee Department of Environment and Conservation
- Williamson County
- United States Geological Survey
- Tennessee Wildlife Resources Agency

- Tennessee Department of Transportation
- Vanderbilt University

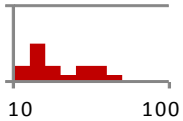
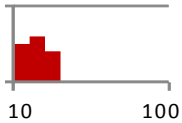
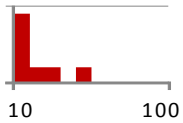
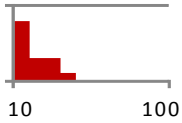
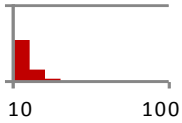
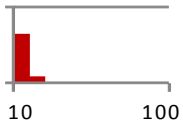
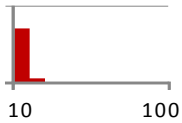
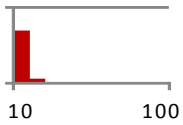
Over the course of methodical, facilitated workshops, the stakeholders developed consensus recommendations for IWRP objectives, performance measures for each objective (ways of measuring success), and projects, policies, and management tools that would be grouped into alternatives for comparison. Stakeholders were able to work toward an integrated plan that is most broadly beneficial and acceptable to Franklin and the surrounding region. Each piece of the plan will be linked to at least one of the nine objectives identified by the stakeholders. These objectives were ranked, by the stakeholders, in order of importance using weights, in which each stakeholder was asked to distribute 100 points between the 9 objectives. A summary of the results of this weighting is provided in **Table ES-1**.

The stakeholders identified ways in which performance against these objectives could be measured and formulated comprehensive sets of projects (draft alternatives) aimed at addressing the most heavily weighted (most important) objectives. A planning-level computer model was developed to examine the relationships between the utilities (for example, how recycled wastewater affects flow and pollutant loads into the stream, and potentially offsets a portion of the potable water demand).

Through a combination of computer simulation scenarios and evaluation of the draft alternatives in consultation with the steering committee for this IWRP (a group overseeing the process for Franklin and interpreting stakeholder feedback for the Board of Mayor and Aldermen), a group of four new alternatives—each on its own a comprehensive alternative for an integrated plan—was selected to advance to the next phase of the study process for more detailed evaluation of specific options. The four alternatives, shown in **Figure ES-1** and compared with the do nothing alternative, were formulated around specific individual objectives that had a high weighting, but ultimately evolved into more comprehensive plans formulated at addressing broader objectives.

The bar charts illustrate how each alternative scored with respect to each of the nine objectives, using the performance measures developed by the stakeholders. The results in Figure ES-1 have been normalized to common scales and weighted according to the values shown in Table ES-1. The graph is not intended to serve as a recommendation of one plan over another, but rather, to illustrate tradeoffs between alternatives and demonstrate that each alternative plan has merits that warrant more detailed evaluation in Phase II of the IWRP. For example, although the revised reliability alternative has the lowest composite score, it ranks highest in reliability, which was ranked the most important objective by the stakeholders. However, it is also the most expensive alternative. This example illustrates how the results can be used to understand tradeoffs in alternative costs and benefits among the four alternatives recommended for further study. The specific composition of each alternative can be found in **Table ES-2**.

Table ES-1
Franklin IWRP Objectives and Weights

Objectives		Weights			
Name	Description	Min	Max	Average	Histogram
Reliability	Meet current and future demands for water and wastewater reliably	0	70	31.1	
Efficiency	Maximize efficiency of water use and value of water resources	5	25	15.5	
Water Quality & Ecological Health	Improve water quality and ecological health of Harpeth River and watershed	0	50	13.5	
Service at a Reasonable Cost	Provide excellent level of water/wastewater utility services at reasonable cost	0	40	13.2	
Safety & Security	Provide safety and security of water resources systems	0	25	8.3	
Regional Acceptance	Achieve regional acceptance	0	15	5.7	
Sustainable Biosolids Management	Achieve sustainable biosolids management	0	15	4.7	
Improved River Access	Provide improved access and aesthetics of Harpeth River	0	15	4.5	
Carbon Footprint	Minimize carbon footprint of water resources operations	0	10	3.5	

Graphs represent the number of respondents at each level of importance (weight), and are intended to illustrate whether the group's values were generally unified or dispersed.

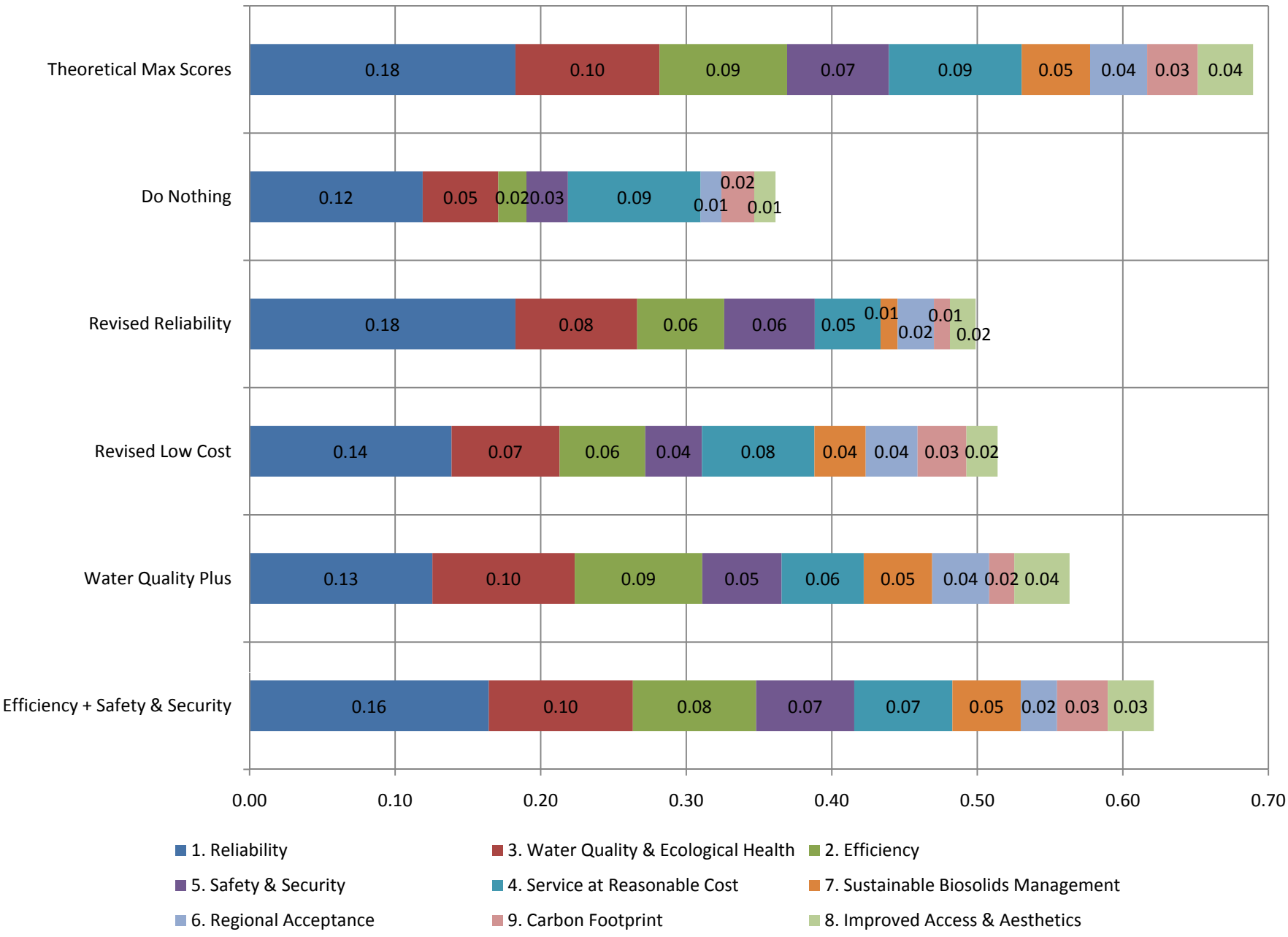


Figure ES-1
Stakeholder Recommended Alternatives for Analysis in Phase II of the IWRP

Category	Options	Efficiency + Safety & Security	Water Quality Plus	Revised Low Cost	Revised Reliability
Stormwater Options	Residential rain barrels	X	X		X
	Commercial stormwater reuse	X	X		X
	Recreational stormwater reuse	X	X		X
	Rain gardens	X	X		
	Pervious pavement	X	X		
	Constructed wetlands	X	X		
	Conveyance upgrades	X	X		
	Increased storage	X	X		
Water Treatment Plant	Upgrade existing 2.1 mgd WTP and purchase remaining water from HVUD			X	
	Expand existing WTP to 4.0 mgd, upgrade WTP intake structure and purchase remaining water from HVUD	X			
	Repair water reservoir (ongoing)	X		X	
	Shut down existing WTP and purchase all water from HVUD		X		
	Construct raw water transmission line from the Cumberland River and upgrade water treatment plant to supply all City demand				X
Distribution System	Address water loss	X	X	X	
	Install advanced metering	X	X	X	X
	Remove outdated tanks	X	X		
	System management practices	X	X	X	X
Conservation Options	Indoor and outdoor conservation (public education, etc)	X	X	X	X
	Conservation ordinances	X	X	X	X
	Low flow incentives	X	X	X	X
	Rate block structure, etc	X	X	X	X
Wastewater Treatment Plant	Upgrade and rerate existing WWTP		X	X	X
	Construct new WWTP at Goose Creek	X			X
	Collect and treat wastewater from adjacent communities or other small systems (e.g., Lynwood, Cartwright Creek)	X	X		
	Treat discharged effluent to higher standard during summer months		X		
Collection System	Address inflow and infiltration	X	X	X	
	Hook up septic users to sewer	X	X		X
	System management practices	X	X		
Ecological Restoration Options	Removal of low head dam at the water treatment plant intake	X	X	X	
	Address old dump site (from downtown to Liberty Creek) and convert to Harpeth River access area				
	Use of Robinson Lake to provide enhanced base flow in the Harpeth River during dry periods	X	X		
	Cattle exclusion	X	X		
	Widespread stream and bank restoration	X	X		
Reclaimed Water Options	Complete the 12" Long Lane line and retrofit the existing 500,000 gallon Long Lane water reservoir for reclaimed water service	X	X		
	Complete the distribution loop around the city by constructing the 12" Columbia Avenue/Southeast Parkway reclaimed line and construct a 500,000 gallon storage tank in the vicinity of Winstead Hill	X	X		X
	Convert the Franklin Green/Horton Lane sanitary force main for reclaimed water distribution	X	X	X	
	Increase City-wide reuse by increasing customer base	X	X	X	
	Install additional pumps to increase the station capacity to approximately 12 million gallons per day	X	X		X
	Establish additional reclaimed water storage facilities/ convert existing water storage tanks to reclaimed storage tanks	X	X		X
	Identify and establish dedicated reclaimed water sites	X	X	X	
	System management practices	X	X	X	
Biosolids Options	Upgrade solids handling facilities to produce Class A solids				X
	Upgrade solids handling facilities to drying/ERS (ash disposal)				X
	Upgrade solids handling facilities to produce higher TS content sludge				
	Solids disposal at BFI (108 miles/trip)				
	Solids trucked to Metro Nashville for disposal/processing			X	
	Class A biosolids to Franklin's composting facility		X		
	Land application (Switch grass production)	X			
	Upgrade biosolids facilities for biogas to energy	X	X	X	

Table ES-2
Recommended Alternatives

Section 1
Introduction

The City of Franklin's unique community, characterized by its commitment to preserving the City's history and heritage and coupled with its location in one of the fastest growing regions of the country, makes it a popular place to reside and conduct business. From 1997 through 2008, the population of the City nearly doubled. With continued growth comes increasing pressure on City services and infrastructure. All areas of the City's infrastructure have experienced growth pressure, including roads and streets, water supply and treatment, wastewater treatment and disposal, and other services. These ever-increasing demands have led the City administration and staff to reevaluate their water resources from a long-term, holistic perspective that encompasses water supply and treatment, stormwater management, wastewater collection and treatment, and reclaimed water distribution. The process being used to accomplish this goal is an Integrated Water Resources Plan (IWRP) which is a facilitated process that engages stakeholders from the inception of the project throughout the entire planning process.

1.1 Project Background

The City of Franklin is a growing city relying on a small river for their water needs. As the City continues to grow, the stresses placed on that water resource continue to compound; and when weather extremes occur the stresses are intensified. Protecting the Harpeth River's ecology and value as a recreational resource is not only required, from a regulatory standpoint, but is desired as part of the quality of life in Franklin. At the same time, the City needs a reliable long-term source and infrastructure for drinking water and irrigation and sustainable solutions and infrastructure for reclaiming wastewater and discharging treated effluent when reclaimed water use is not feasible. With these challenges, the City looked to develop a long-range plan that establishes governing science and engineering principles, evaluates and debates alternatives, and builds broad consensus around a comprehensive set of sustainable, affordable actions that will provide for effective management of the City's water resources.

1.2 IWRP Project Approach and Scope

The purpose and scope of this project includes two major phases of work during which a comprehensive, implementable, and fundable IWRP will be developed with the City of Franklin. This approach has focused on stakeholder-derived objectives as the central measure of success, allowing a progressive screening of alternatives in a way that is technically robust and broadly acceptable to the City, the regulatory community, advocacy groups, and citizens.

The IWRP is developed in two phases:

- Phase I – Completed in 2010, the purpose of Phase I was to convene a stakeholder advisory group and steering committee and formulate a list of objectives that the IWRP will address. A preliminary evaluation of potential water, wastewater,

stormwater, and reclaimed water projects to be included in the final IWRP was performed using an integrated systems simulation model and performance measures derived by the stakeholder groups. Alternative groupings of project options were compared using decision support methodology described herein. The outcome of Phase I is a greater understanding of Franklin's water resources systems, consensus amongst stakeholders on the objectives of the IWRP, and a refined list of project options to be studied further in Phase II.

- Phase II – The purpose of Phase II will be to perform more in-depth analyses of the costs and benefits of the project options identified in Phase I. A dynamic watershed simulation model, detailed engineering studies, and conceptual designs will provide refined estimates of the performance of project options over the 30-year planning period. The integrated systems model will be updated with the new information learned in the Phase II studies and will assist in the development of the final IWRP. Stakeholder involvement will continue throughout Phase II, and the final IWRP is intended to be a recommendation from the stakeholders to the City of Franklin for implementation.

The Franklin IWRP will consider the many aspects of the City's water resources and their interactions including the following:

- Harpeth River – flooding, low flow frequency, water quality and ecological health.
- Water Supply – Source, treatment, distribution, and conservation.
- Wastewater Treatment – The collection system, treatment plant facilities, and discharge permitting.
- Reclaimed Water Use – Availability, distribution, and demand for this resource.
- Stormwater – Conveyance, best management practices, impacts on the Harpeth River, and reuse.

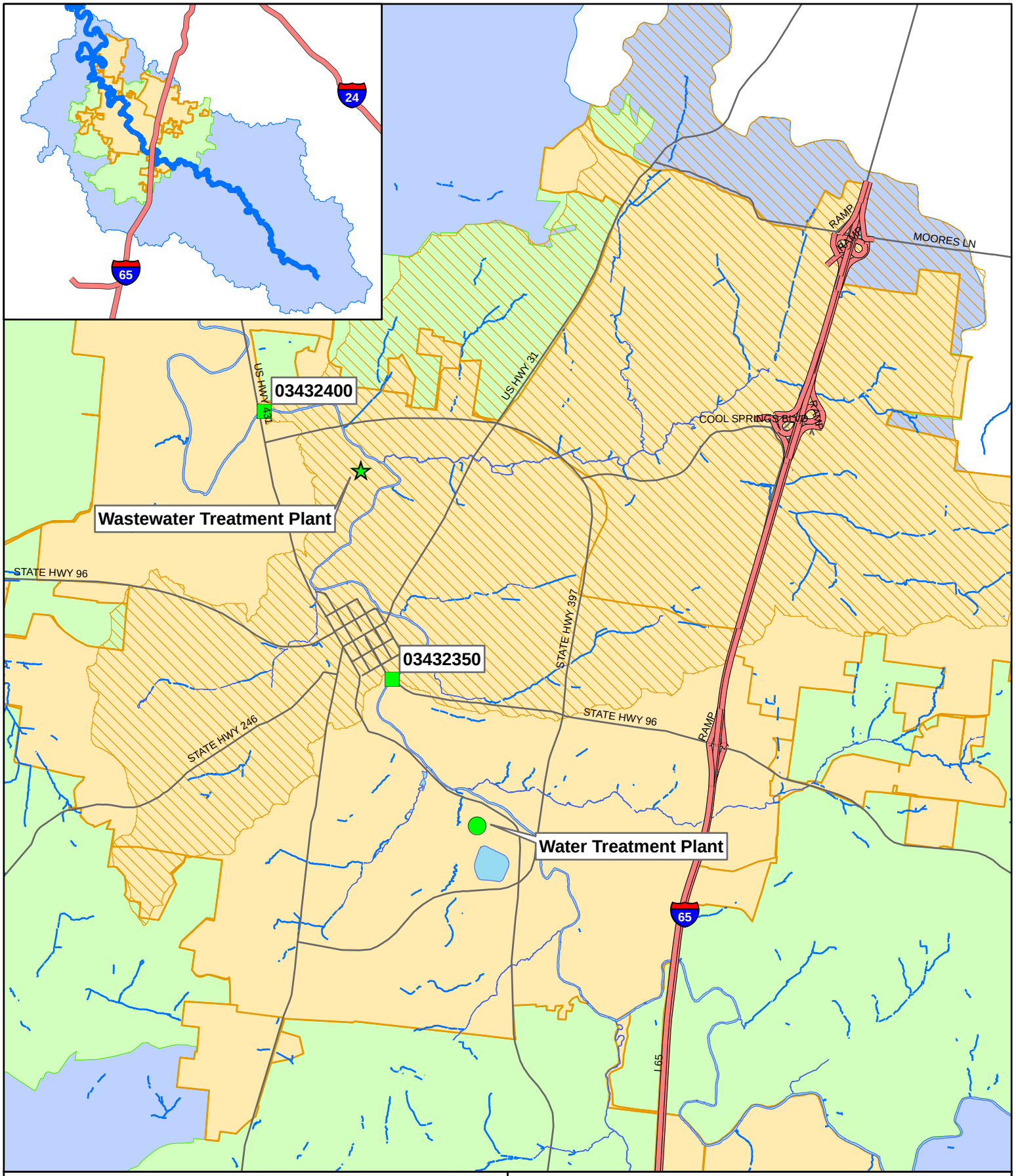
1.3 IWRP Project Area and Planning Period

In order to define the geographic boundaries of the project, as well as allow for opportunities to incorporate options that address regionalization goals, a steering committee reached a consensus on a 3-circle concept that includes the following distinctions for the project planning area:

1. The City of Franklin's existing service area needs that must be addressed.
2. The City of Franklin's urban growth boundary that will be considered in the planning.
3. The regional/watershed area that will be considered, providing opportunities for mutual cooperation among stakeholders.

Defining these geographic boundaries will add some necessary focus to the existing service area, without being exclusionary of a larger relevant geographical scope, and will foster discussion among stakeholders regarding regional issues. In addition to this project planning area, the steering committee defined the Harpeth River as the central point of project integration. This designation provided a guideline for measuring the success of each project alternative evaluated. The project area is shown in **Figure 1-1**.

In addition to defining the geographic constraints of the project, the steering committee discussed a specific period for the project plan. There was strong consensus that a planning period of 25—30 years would be most appropriate for the IWRP, indicating 20 years is too short for project implementation and 50 years has too much uncertainty for developing alternatives.



USGS Streamgages
Water Plant
Wastewater Plant
Interstate
Streets
Harpeth River and Tributaries
Hydrologic Reach
Franklin Service Areas
Urban Growth Boundary
Harpeth River Watershed

Figure 1-1 Franklin IWRP Project Area

0 2,500 5,000 10,000 Feet

N

Section 2
Integrated Planning Process

The ultimate goal of the IWRP process is to develop a plan that garners broad consensus and support from the people and organizations that will be affected in some way by the decisions. Together with technical and economic feasibility, support from public citizens, advocacy groups, regulatory agencies, participating utilities, and scientific agencies is the third pillar of a successful IWRP. Phase I of this IWRP process focused primarily on consensus building—specifically on agreements between participating stakeholders regarding the types of plans and decisions which would be most broadly beneficial to the most people. Phase II will carry this basic tenet forward, while also focusing more detailed attention on technical and economic feasibility of alternative ideas that have already been identified as preferable. The way that consensus was achieved in Phase I was by discussing and agreeing upon strategic goals, rather than on preferences of individuals or their organizations for specific projects or decisions. Plans that are focused on consensus goals are much more likely to receive broad support than plans formulated on individual preferences for specific projects. The rest of this section outlines this goal-oriented process of consensus building.

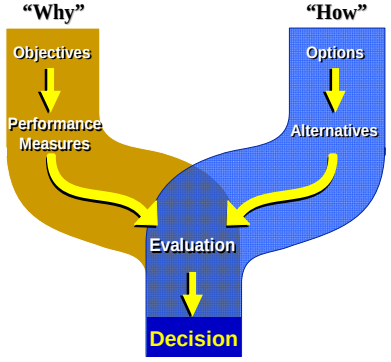
2.1 Identification of Stakeholders and Decision Makers

The first step in the IWRP process was to identify the people who would in some way be affected by (or have authority over) the integrated plan. These stakeholders represented public citizens of Franklin, Franklin's utilities, other regional utilities, watershed advocators, scientific advisors, and regulatory officials. The stakeholders are discussed in more detail in Section 3.1. The four categories of stakeholders and their parts in the IWRP process are described below.

1. **Decision Makers –** Board of Mayor and Aldermen
2. **Steering Committee –** City Staff, Scientific Advisor, BOMA Member
   - Participated with stakeholder advisory group in interactive workshops.
   - Made recommendations to BOMA.
   - Worked with facilitators to direct the process.
3. **Stakeholder Advisory Group –** Citizen representatives, watershed organizations, utility directors, state regulatory representatives, technical experts
   - Participated in collaborative workshops (discussed below).
   - Made recommendation to the steering committee.
4. **Public Citizens**
   - Received reports on project progress.

- Offered opportunity to share ideas and information with stakeholder advisory group and steering committee.

2.2 Separating Reasons from Methods (Why vs. How)



Blending the two tracks of water resource planning enables us to move from technical needs to "interest-based" solutions.

The most fundamental aspect of building consensus is distinguishing between the reasons for making certain decisions and the means of implementing them. This can be thought of as separating the Why from the How. In other words, beginning to solve problems by discussing how to solve them can often lead to disagreements, for no other reason than the problems themselves have not been agreed upon. By beginning with a common understanding and statement of the problems to be addressed, the participants in an IWRP process can agree on why any given decision must be made. Only then can they engage in cooperative discussions on how to address the problems. The IWRP process was divided into two very distinct tracks to help stakeholders first identify goals and objectives for this plan (the Why) and then to identify the projects or management measures that could best address these common goals (the How). In short, big-picture goals must be agreeable before the problems at their source can be discussed on common ground.

2.3 Integrated Planning Framework

To help facilitate the distinction between Why and How, a commonly employed IWRP framework was used with the City of Franklin and its stakeholders. At its core are four key terms that were used throughout the planning process to focus attention, first, at a common statement of goals and, then, on consensus-building around the best way to achieve the goals. The terms defined here will be used throughout this report:

Objectives (Why)	Major goals of the IWRP, in broad and understandable terms
Performance Measures (Why)	Specific metrics that indicate whether or not (or how well) an objective is being achieved
Options (How)	Individual projects or policies that will be assembled into comprehensive alternatives
Alternatives (How)	Packages of individual projects and policies designed to meet objectives

During the planning process, options were grouped into alternatives only after the objectives were agreed upon. With agreeable objectives, the options could be compared on common ground, because the performance measures clearly indicated how well each alternative satisfied the group's collective objectives. Without this common basis of measurement, decisions can be largely based on personal preference. Even when based on scientific, economic, and social rationale, consensus can be extremely difficult to achieve, if the objectives are not stated explicitly and cannot be measured in agreeable ways.

2.4 Collaborative Workshops and Meetings

Stakeholders are engaged from the beginning and throughout the entire planning process, which helps define the objectives of the plan, identify potential solutions, collaborate on the formulation of analysis tools, and develop recommendations for BOMA. Steering committee meetings were held monthly during Phase I to discuss and facilitate project progress. Stakeholder advisory group workshops were held at key points throughout the development of the IWRP integrated model. A summary of the stakeholder meetings and their objectives is provided in **Table 2-1**. The minutes for all meetings have been provided for the public on the City's IWRP website at http://www.franklin-gov.com/index.aspx?page=623.

Stakeholder Workshops

The four definitions above (objectives, performance measures, options, and alternatives) became the primary framework for a series of collaborative workshops with the IWRP stakeholder advisory group and steering committee. Four workshops were held to facilitate consensus on each of the major components of the IWRP process and then to formulate recommendations on preferred alternatives that warrant more detailed technical and economic analysis in Phase II. The workshops reflect both the Why vs. How concept and the four basic definitions put into practice.

Public Forums

The project team coordinated two public forums to provide information to the general public regarding the project objectives and alternatives arising from the selection process. The focus of these meetings was to educate the community about the purpose and scope of the project and provide the general public an opportunity to give feedback to the consulting team and the stakeholder advisory group. The presentations and meeting minutes from both public forums have been provided for the public on the City's IWRP website, provided above.

Table 2-1
Franklin IWRP Key Meetings

Meeting	Date	Goals
Introductory Stakeholder Meeting	December 17, 2009	1. Outline the approach and timeline for Phase I. 2. Define the roles of the stakeholders. 3. Explain the first need for information from stakeholders.
Workshop 1: Objectives	January 20, 2010	1. Identify project objectives and discuss objective weighting. 2. Identify performance measures. 3. Identify constraints that bound the scope of the project plan.
Public Forum 1	February 22, 2010	1. Explain IWRP and why it is needed 2. Scope and process 3. Present draft objectives from Workshop #1 4. Solicit feedback
Workshop 2: Performance Measures	March 24, 2010	1. Review objectives and present results of objective weighting 2. Discuss performance measures. 3. Discuss specific project options for meeting objectives. 4. Introduce concept of grouping options into themed alternatives. 5. Conduct brief demonstration of integrated system model.
Workshop 3: Alternatives	June 2, 2010	1. Review of alternatives formulation process. 2. Review list of specific project options. 3. Conduct initial grouping of options into themed alternatives.
Public Forum 2	July 12, 2010	1. Review IWRP process and objectives 2. Discuss development of options and performance measures 3. Explain alternatives formulation 4. Present integrated system model
Workshop 4: Comparing and Modifying Alternatives	August 18, 2010	1. Explain the analysis process and score card methodology. 2. Review the results and scores of the alternatives. 3. Select and refine alternatives for Phase II.

Technical Workshops

In addition to the workshops and public forums, the project team hosted two technical meetings for interested parties to review the formulation and functionality of the integrated model used for analyzing alternatives. During these meetings, technical specialists were available to provide detailed information regarding the model assumptions, construction, and integrations of model relationships and the overall process of running the model.

2.5 Integrated System Modeling and Analysis

The stakeholders were provided with technical information on the performance of the alternatives with the help of an integrated system simulation model (discussed in detail in Section 4). This computer tool simulated the connectivity between Franklin's

water supply, wastewater, stormwater, and reclaimed water both directly and through their interactions with the region's common natural water resource, the Harpeth River. Fundamentally, the tool was developed to support the following two functions:

- **Understanding of System Dynamics –** An important part of the consensus-building process is developing a common understanding of the dynamics (cause-and-effect relationships), tradeoffs, and vulnerabilities of interconnected water resource systems. Before examining specific alternatives, the model was used in two technical forums to help illustrate the potential effects and tradeoffs of certain key decisions that might someday be made. For example, the group explored the effects of increased water supply withdrawals on the Harpeth River over its entire flow regime and, also, the effects of increased reclaimed water usage on water quality in the river. Basically, the model was used in these venues to help answer important what if questions about certain prospective options in a technical, objective way.

- **Providing Performance Measures for Alternatives –** The model was also used to provide most of the quantitative performance measures for the alternatives (life-cycle costs were calculated externally). Examples of model output included frequency of low river flows, pollutant loads into the river, energy consumption, amount of wastewater and stormwater reclaimed for beneficial uses, etc. These values were used directly to help the stakeholders compare alternatives.

2.6 Identification of Preferred Plans

Once the alternatives were formulated and each had been analyzed with the integrated systems simulation model, a composite score for each alternative was developed. Part of this process included interactive work with the steering committee to assign qualitative scores to alternatives where numerical scores would be inappropriate or infeasible (generally, a scale such as poor/good/better/best). The score for each performance measure was then multiplied by the weight of the affiliated objective (modified in some cases to emphasize performance measures that best distinguished alternatives over those that showed little distinction). These weighted values were then added together into a composite score for each alternative. Composite scores could be easily compared to identify the most preferred alternatives, tradeoffs between alternatives, and the component parts of each alternative that seemed to contribute most broadly to its effectiveness. This information was used by the stakeholders to regroup certain options and form hybrid alternatives aimed at as many objectives as possible.

At the conclusion of Phase I, the process yielded four preferred hybrid alternatives which, by consensus of the stakeholders, should be carried forward into Phase II for more detailed technical and economic analysis.

Section 3
Franklin IWRP Framework

The development of the Franklin IWRP has followed a stakeholder-driven process for defining the objectives, objective weights, and performance measures used to rank and compare alternatives. During the workshops discussed in Section 2 the stakeholder advisory group defined nine objectives, assigned weights to the objectives, and developed a set of one to five performance measures by which to gauge the achievement of each objective. Project options (or options) were derived from existing plans, reports, and stakeholder input and represent a consolidated list of all projects that could potentially be included in the plan. During a workshop, stakeholders worked in groups to formulate alternatives by grouping together options aiming to meet the five most heavily weighted objectives. The following sections describe the process of developing the framework of the IWRP process – from brainstorming objectives to grouping options into alternatives.

3.1 Stakeholders

Stakeholder participation in the IWRP process is critical to the success of the project. Stakeholder concerns play a key role in the IWRP development process, particularly in regards to public agencies. The involvement of stakeholders during the decision-making process of the IWRP allows for open, fair, and accepted solutions to issues.

The City of Franklin is governed by an elected Board of Mayor and Aldermen (BOMA) made up of nine people—the mayor and eight aldermen. BOMA selected the IWRP process as the most effective and beneficial method of developing a resource plan for the City of Franklin's water systems. BOMA is ultimately the deciding body for plan selection, which is the objective of Phase II of the IWRP. The decision process is illustrated in **Figure 3-1** and includes three levels of stakeholder input: steering committee, stakeholder advisory group, and public forums.

The steering committee consists of a select group of knowledgeable stakeholders who provide guidance on scope and process and help distill overall stakeholder recommendations to BOMA. The stakeholder advisory group is a larger group of individuals who represent various parties with interest in Franklin's water resources decisions (e.g. state regulators, watershed advocates, citizens, and water utilities). Public forums are the venue by which the general public can provide input to the planning process and learn about the suggested solutions.

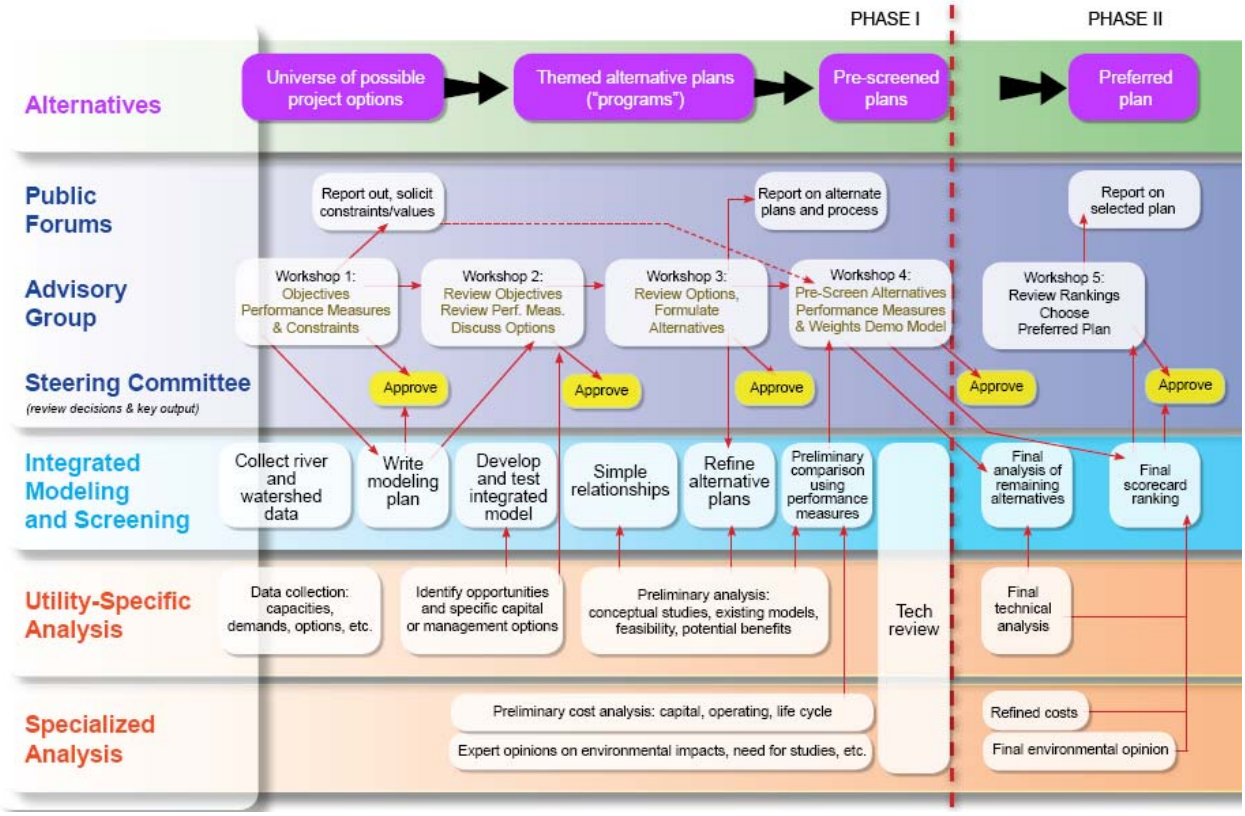


Figure 3-1
Franklin IWRP Stakeholder Involvement Workflow

The levels of stakeholder input within the IWRP allow BOMA and the City of Franklin to feel confident that the alternative sets developed so far and, ultimately, the approved plan have undergone rigorous evaluation by all parties with vested interest. The final plan will be developed during and through a series of steering committee meetings, stakeholder advisory group workshops and public forums.

3.1.1 Steering Committee

The steering committee was established to provide guidance on scope and process, answer questions and offer suggestions to stakeholders, and help distill information and recommendations to BOMA. The steering committee members have a high degree of knowledge of the water resources systems and/or the City of Franklin. **Table 3-1** lists the members of the steering committee and their affiliations.

Table 3-1
Franklin IWRP Steering Committee

Name	Affiliation
Eric Gardner	City of Franklin, Engineering Director
Mark Hilty	City of Franklin, Water Management Director
Dr. Eugene LeBoeuf	Vanderbilt University, Franklin citizen
Dr. Ken Moore	BOMA
David Parker	City of Franklin, City Engineer
Eric Stuckey	City of Franklin, City Administrator

3.1.2 Stakeholder Advisory Group

The stakeholder advisory group includes representatives of organizations with a vested interest in the projects planned for the City of Franklin. The types of organizations included are watershed nonprofits, regulatory agencies, private or public utilities, city officials, and public representatives. The input from these types of organizations is important, because it is only with their support that the IWRP is accepted and implemented.

The responsibilities of the stakeholder advisory group are to participate in planning workshops to give input and to make informed decisions and recommendations to the steering committee. Each stakeholder advisory group member brings input from the stakeholder group that he or she represents, including the public. **Table 3-2** lists the members of the stakeholder advisory group.

3.1.3 Public Citizens

The citizens of the City of Franklin are the largest and most important group affected by the decisions made during the IWRP. The public is invited to participate in the IWRP process by attending public forums, contacting a member of the stakeholder advisory group, or directly contacting the steering committee and consultants.

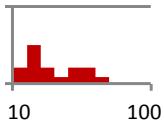
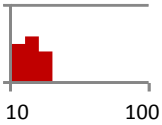
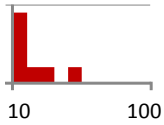
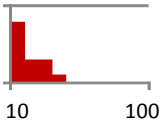
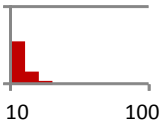
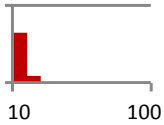
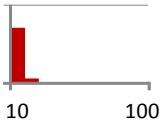
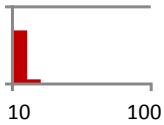
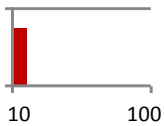
3.2 Defining and Weighting Objectives

The IWRP project objectives were developed by the stakeholder advisory group during Workshop 1. During this workshop, stakeholders identified nine overarching objectives for the IWRP, and agreed that they represented the collective interests of all participating groups. These objectives are described in **Table 3-3**. Each stakeholder was asked to assign a relative importance (weight) to each objective by distributing 100 total points among the nine objectives. This was accomplished outside of the workshops so that individual stakeholders could consult with their constituents on the weights that would be most representative of their interests. For each objective, the average of all stakeholder weights was used to represent the relative importance of that objective to the group as a whole. In this way, all nine objectives were ranked from most to least important, and all were applied commensurately in the subsequent comparison of alternatives. Table 3-3 also lists the minimum and maximum values given to each objective by stakeholders and the distribution of responses.

Table 3-2
Franklin IWRP Stakeholder Advisory Group

Name	Affiliation
Dorie Bolze	Harpeth River Watershed Association
Dan Crunk	Community Representative
Kristi Earwood	Attorney for Williamson County
Scott Gain	USGS
Tim Ham	Mallory Valley
Doug Hausken	Cumberland River Compact
Dr. Deedee Kathman	TDOT, Environmental Division
Lee Keck	TDEC
Dan Klatt	Community Representative
Greg Langeliers	Thompson Station
Roger Lindsey	Franklin Planning Commission
John McClurkan	TN Department of Agriculture, Water Resources
Tom Puckett	HB&TS
Howard Smithson	Milcroften
Rob Todd	TWRA
Dr. Sherry Wang	TDEC
Bobby Worthington	HVUD

Table 3-3
City of Franklin IWRP Objectives and Weights

Objectives		Weights			
Name	Description	Min	Max	Average	Histogram
Reliability	Meet current and future demands for water and wastewater reliably	0	70	31.1	
Efficiency	Maximize efficiency of water use and value of water resources	5	25	15.5	
Water Quality & Ecological Health	Improve water quality and ecological health of Harpeth River and watershed	0	50	13.5	
Service at a Reasonable Cost	Provide excellent level of water/wastewater utility services at reasonable cost	0	40	13.2	
Safety & Security	Provide safety and security of water resources systems	0	25	8.3	
Regional Acceptance	Achieve regional acceptance	0	15	5.7	
Sustainable Biosolids Management	Achieve sustainable biosolids management	0	15	4.7	
Improved River Access	Provide improved access and aesthetics of Harpeth River	0	15	4.5	
Carbon Footprint	Minimize carbon footprint of water resources operations	0	10	3.5	

Histograms (graphs in the last column) represent the number of respondents at each weight and are intended to illustrate whether the group's values were generally unified or dispersed. In all the graphs, the horizontal axis is the weight of the objective (in 10 point intervals from 0-10 to 90-100), and the vertical axis is the number of respondents at each weight.

Table 3-4
Franklin IWRP Performance Measures

Objectives	Weight	Performance Measures	Units
Reliability	31.1	Percent of time all demands met	% time (all days)
		Average magnitude of deficits (all uses)	volume, MG
		Volume of WW capacity surplus or shortfall	MGD
		Supply redundancy	% volume
Efficiency	15.5	Volume of stormwater put to beneficial use	MGD (all days)
		Percent of total reuse demand satisfied	% volume
		Percent of demand reduction	% volume
		Reduction in inflow and infiltration	qualitative
		Percent reduction in unaccounted for water	% volume
Water Quality & Ecological Restoration	13.5	Frequency of low flow < September median	% time (all days)
		Average summer BOD load (lb/day)	LB/day (summer only)
		Average summer nitrogen load (lb/day)	LB/day (summer only)
		Ecological indicators	qualitative
		Negative impacts of stormwater reduced	qualitative
Service at a Reasonable Cost	13.2	Life-cycle cost of projects and policies	dollars
		Combined change in water and sewer rates	qualitative
		Meet secondary drinking water standards	qualitative
Safety & Security	8.3	Percent of total wastewater on septic	% volume
		Change in 100-year flood elevation	qualitative
		Vulnerability of infrastructure & facilities	qualitative
		Emerging water quality concerns	qualitative
Achieve Regional Acceptance	5.7	Extent of regional focus	qualitative
		Likelihood of public acceptance	qualitative
Sustainable Biosolids Management	4.7	Biosolids handled sustainably	qualitative
Improved Access & Aesthetics	4.5	Percent of stream flow that is WWTP effluent	% volume (Sept. only)
		Extent of bank stabilization	qualitative
		Erosion potential	qualitative
		Public accessibility	qualitative
Carbon Footprint	3.5	Average energy requirements (kWh/day)	average kWh/day

3.3 Defining Performance Measures

Performance measures are quantitative and qualitative ways in which progress toward each of the nine objectives could be measured. Performance measures for the Franklin IWRP were developed during Workshop 2 and refined throughout Phase I by the stakeholder advisory group and steering committee. Each performance measure supports one of the nine objectives as shown in Table 3-4 and gives a means

for evaluating the effectiveness of an alternative in meeting an objective. Some of these metrics were quantitative, and could rely on numbers derived from engineering or cost analysis. Others were qualitative, and relied on the expert judgment and consensus of the steering committee on the relative ability of alternatives to satisfy the objectives.

3.4 Options and Alternatives

A list of potential project options for each system, including water, wastewater, stormwater, reclaimed water, and the Harpeth River, were provided to the stakeholders. These options were compiled from existing studies and plans and did not exclude any feasible project option from consideration. Stakeholders were asked to review these project options and provide additional ideas or other feedback.

During Workshop 3, stakeholders designed alternatives around the five most heavily weighted objectives. The process consisted of selecting options determined to best meet each of the objectives individually, and each group of options resulted in an alternative named for that objective. Table 3-3 lists the alternatives and associated options described below.

- **Water Quality and Ecological Health–** The focus of this alternative is improving the water quality and ecological health of the Harpeth River and includes project options specifically aimed at that goal. This alternative is hereafter referred to as Water Quality.

- **Cost –** This alternative is designed to provide a level of services for water resources at a reasonable cost and includes project options that seem to meet the needs of the City at the lowest estimated cost.

- **Efficiency –** The options selected for this alternative are intended to maximize the efficiency of water use and emphasize the value of water resources.

- **Reliability –** This alternative includes options designed to reliably meet current and future demands for water and wastewater handling.

- **Safety and Security –** Options included in this alternative were selected with the intent to provide safety and security of water resources systems.

During Workshop 4, stakeholders reviewed the comparative study of the five original alternatives; that is, the composite value of performance measures, weighted by the importance of the objective linked to each performance measure (see Section 3.6 for a detailed explanation of this process). This was accomplished with the use of an integrated system simulation model (discussed in Section 3.5 and Section 4), which provided additional information on the broad impacts of various options. The group then identified tradeoffs between the themed alternatives, as well as options that appeared to perform well by addressing more than one objective. The alternatives were regrouped into hybrid alternatives, which effectively mixed the best of the best to yield balanced alternatives aimed not at one objective, but at as many as possible.

The workshop concluded with a consensus that four remaining hybrid alternatives be carried forward to Phase II for more detailed technical and economic analysis.

3.5 Integrated System Model

An integrated system model was developed to simulate the stakeholder-defined alternatives with project options in each of the water utilities. This model provides output to stakeholders in the context of their own stated objectives. The software used for analysis for the IWRP is STELLA. More information on STELLA, as well as a trial version of the software, may be found at the following website:

http://www.iseesystems.com/softwares/Education/StellaSoftware.aspx .

STELLA is a dynamic and graphical tool used to simulate interactions between and within subsystems that are part of a larger, interconnected system. It is frequently used in environmental engineering venues to better understand the implications of decisions across a broad array of social and environmental sectors. The City's water resources systems—the Harpeth River, water supply, wastewater, reclaimed water, and stormwater—were modeled with the software STELLA. The model is a representation of the interconnections between the major water resources systems and includes planning-level calculations of flow, pollutant loads, energy requirements, and operational costs. A complete explanation of the system model development, demand projections, and assumptions is provided in Section 4.

3.6 Scorecard Analysis

The performance measures listed in Table 3-4 were scored for each alternative by using either direct model output for quantifiable performance measures or qualitative scores developed by the steering committee. The software program, Criterion Decision Plus (CDP), was used to perform the scorecard analysis, which involves standardizing the raw performance measure scores, applying the objective weights as determined by the stakeholder advisory group, and ranking the alternatives based on the aggregate scores across all objectives. CDP is a visual model with multiple ways of displaying results. CDP was selected as the decision modeling tool because of its sophistication, ease of understanding and use, and its ability to conduct sensitivity analyses on all of the various values input to the model, such as criteria weights, performance, and satisfaction levels. More information on CDP can we found at the website www.infoharvest.com/ihroot/index.asp.

Goals, objectives, performance measures, and weights are input into CDP. In order to rank alternatives (groups of options), raw portfolio scores for each performance measure are also input to CDP. Each score is standardized on a linear scale from 0 to 1, with the best possible score translating to 1 and the worst possible score translating to 0. In this way, the various units in which the performance measures are quantified are eliminated, and it is possible to compare all scores. **Figure 3-2** shows an example of how the cost of an alternative is translated into a unit-less score.

A composite score for each objective was determined based on the sum of scores of its performance measures, and this score was multiplied by the weight of that objective as determined by the stakeholder involvement process (see Table 3-3). These values were then summed for comparison across all alternatives.

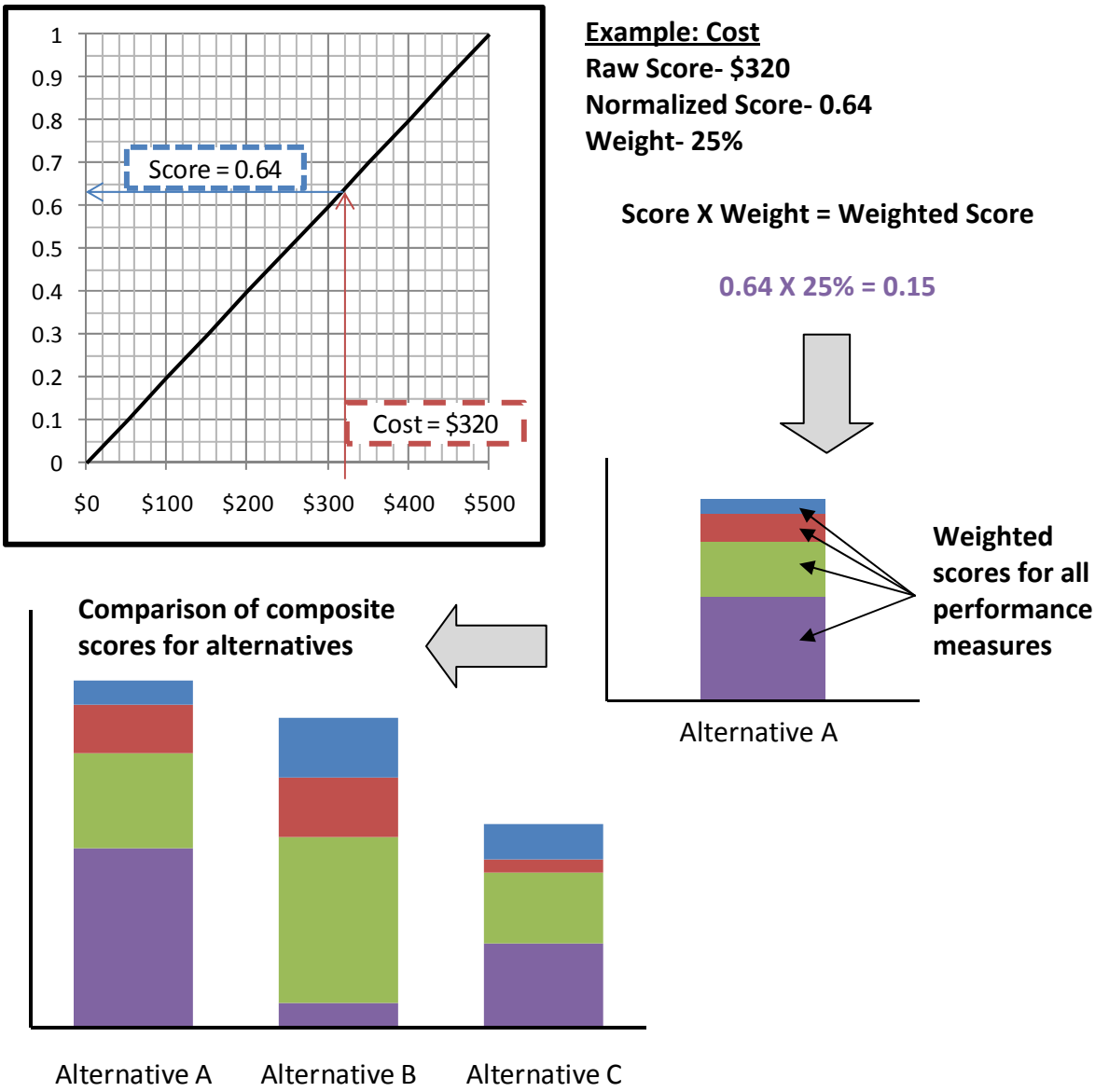


Figure 3-2
Example of Normalizing and Weighting Performance Measure Scores

Section 4
Model Documentation

4.1 Overview

Unlike many engineering models, the integrated model is not a high resolution parametric model aimed at reproducing hydraulic or biochemical processes in a given system. Rather, it serves as a platform in which to integrate general response patterns and interdependencies of various subsystems in a way that will indicate preferability of one alternative over another. This is accomplished by developing mathematical empirical relationships within and between the water, wastewater, stormwater, and reuse subsystems. It was developed specifically to help stakeholders and decision makers understand the interconnectivity between the resources and utilities in Franklin.

The simulation model measures a variety of system responses by simulating different plans and their far-reaching impacts on flows, demands, pollutant loads, costs, and usability of the water resources in the study area.

The goals of the model are to provide the following functions in support of stakeholder decisions:

Provide Technical Information

- Performance measures that are quantitative.
- Impacts of decisions aimed at one utility on all others.
- Sufficient detail to distinguish the broad benefits and impacts of alternatives across the resources and utilities that are under evaluation (e.g., water, wastewater, stormwater, water reuse, Harpeth River).

Screening and Plan Formulation

- What projects appear to work well together?
- Are there certain pairings of project that counteract each other?
- What projects offer little or no benefit?

Alternative Comparison

- How well does any given alternative (groups of projects and policies) satisfy the collective interests of the stakeholders?
- What are the alternatives that will most effectively address the broad interests of the Stakeholder Advisory Group?

4.1.1 Model Approach

Franklin's water resources system is a network of natural and manmade systems that exist to satisfy numerous demands on water (e.g., irrigation, industrial use, human consumption, habitat, and recreation). Water moves between segments in various

mechanisms, including completely natural pathways, altered natural pathways, and manmade pathways. The simulation model of Franklin's water resources system is a representation of the system's segments and their interconnectivity. The model will simulate the movement of water and, in some cases, pollutant loads through the system. The following sections will describe how the Franklin system is represented in the model and how the model simulates different configurations and alternatives.

Figure 4-1 is a schematic representation of the Franklin water resources system model. The colored boxes represent the model segments described below. The colored arrows that link the segments represent the flow of water throughout the system. Each colored arrow has an indicator for representation of flow or flow and load. The gray boxes and black arrows indicate data input and calculations involved in determining how the system operates. There are four different types of calculations or values used in the integrated system model, described below and indicated on the schematic using the corresponding number:

1. Data – information input directly into the model from historical records or known values (e.g., plant capacity, rainfall records).

2. Residual Calculations – values resulting from mass balance calculations (e.g., wastewater effluent flowing to the river is the total effluent created less the effluent needed for reuse as irrigation water).

3. Scientific Calculations – calculations using engineering equations or theoretical values (e.g., Manning's equation for open-channel flow).

4. Relational Calculations – values resulting from dependencies on other variables (e.g., phosphorus loading to the river depends on volume of wastewater effluent flowing to the river).

The simulation model operates on a daily time scale in order to examine the effects of system operations on low flows in the river. While a monthly time scale would be most appropriate for the resolution of this model, monthly averaging tends to hide the occurrence of low-flow periods that are important to recognize and consider. A single major storm event can cause an otherwise dry month to appear normal when flows are averaged over that time period. Furthermore, because the flow data are directly available from USGS measurements at two stream gauges in Franklin, there is no reason to consolidate or average the information, as is sometimes done to diminish uncertainty in hydrologic estimates. **Table 4-1** below shows the frequency with which the daily, weekly, and monthly average flow values were at or below the indicated threshold. Monthly average values were applied to each day in the averaging period, so that each time series contains the number of days from January 1, 1975 through December 31, 2007.

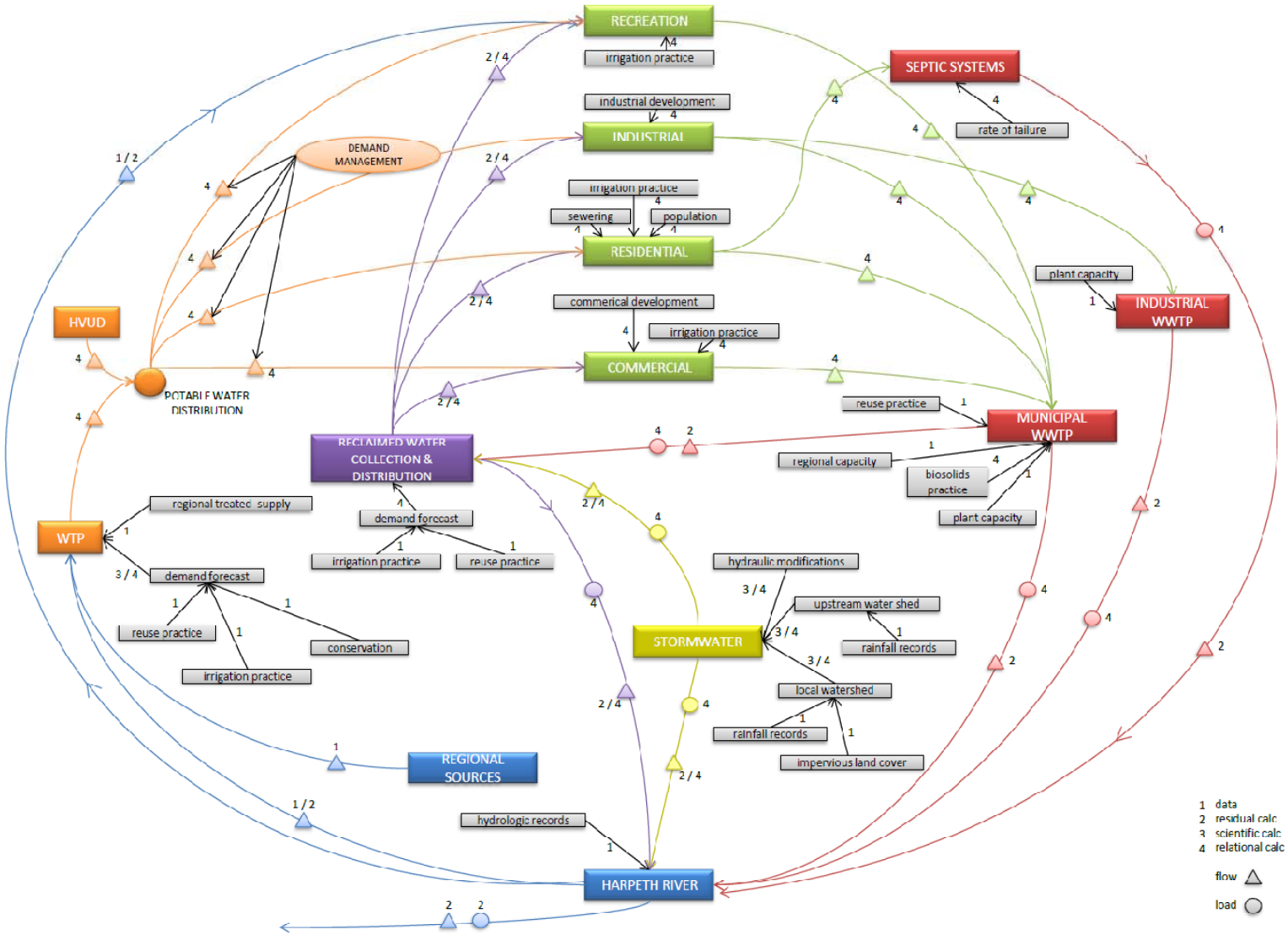


Figure 4-1
Franklin Integrated System Model Schematic

Table 4-1
Flow Frequency Statistic Based on Daily and Monthly Time Scales

	Streamflow (cfs)	Frequency Evidenced in Daily Time Series	Frequency Evidenced in Monthly Time Series
September Median[1]	5.85	14.6%	7.91%
95th Percentile Daily Low Flow[1]	1.70	5.07%	1.96%
99th Percentile Daily Low Flow[1]	0.81	1.06%	0.25%
7Q10[2]	0.50	0.31%	0.00%
Low Flow Threshold[3]	10.0	20.3%	10.6%

1. USGS Gauge data, 1974-2008
2. USGS, 1995. Flow Duration and Low Flows of Tennessee Streams through 1992.
3. 2007 Aquatic Resource Alteration Permit

4.1.2 Model Sectors

The model is segmented into sectors that represent the categorization of Franklin's water resources: the Harpeth River, water supply, reclaimed water, stormwater, and wastewater. The sectors are interconnected such that decisions or policies aimed at managing water within one sector will affect the rest as appropriate. For example, increasing the reclaimed water distribution infrastructure would decrease the demand on potable water for irrigation and decrease the volume of effluent discharged to the river. The model sectors and their connections are explained in detail in the following sections.

4.1.3 Software

The model was developed with STELLA software (Systems Thinking Experimental Learning Laboratory with Animation). STELLA is a dynamic and graphical tool used to simulate interactions between and within subsystems that are part of a larger interconnected system. It is frequently used in environmental engineering venues to better understand the implications of decisions across a broad array of social and environmental sectors.

STELLA is a graphical system simulation package that allows users to model physical flow systems with operational- or planning-level resolution. The software allows users to develop on-screen control interfaces that facilitate rapid adjustments of system variables for alternatives and sensitivity analyses. When dozens of alternatives are feasible (be they alternate water sources, use and reuse guidelines, operational triggers, etc.), STELLA can rapidly help planners and decision makers screen information, identify key drivers, and understand the causal relationships throughout the big picture of a complex system.

Fundamentally, STELLA helps screen options and alternatives, providing numeric scores for performance measures identified as quantitative. In this

context, STELLA does not make decisions, but it can be used to generate information and promote more informed and balanced decisions via rapid comparison of the performance of alternatives using physical, environmental, and economic metrics. Its ability to include multi-sectoral interests in an analytical framework is what distinguishes it from more traditional hydraulic or hydrologic models, which evaluate systems in a purely physical setting. The tradeoff is in resolution. STELLA models do not simulate finely discretized river basins, channels, or pipes but include key system elements and their interdependencies in a lower-resolution network framework in which physical, environmental, and economic response patterns can be effectively examined.

4.1.4 Model Validation

The integrated model is not a parameterized model: that is, it does not rely on calibrated coefficients to reproduce natural or physical processes. Rather, the relationships in the model are based largely on empirical data (stormwater loads, for example) and straightforward combinations of mathematical terms (such as the linear addition or subtraction of flows and loads). The purpose of the model is not to reproduce the watershed and utility processes with scientific precision, but to better understand the interdependence of the processes and their sensitivity to future decisions. Therefore, the model has been tested only inasmuch as the input can be shown to reproduce current or historic patterns or trends and respond appropriately to changes. There are no parameters to calibrate, and the testing of the model relies on expert judgment to determine if the system responses are representative of actual and expected conditions.

4.2 Harpeth River

4.2.1 Historical Flow Record

Conditions for modeled scenarios were defined for the project's 30-year planning period, with utility demands corresponding to projections from 2010 through 2040. These demand levels were applied to a wide range of flow conditions over the hydrologic period of record to calculate the system's average response to the different management conditions. By simulating any given demand level over the historical hydrologic record, the frequency of specific conditions (e.g., limited water withdrawals due to low-flow conditions) can be quantified and interpreted as the probability of occurrence in any given year, given the specified demand levels. This superimposing of lengthy historical hydrologic patterns over any future year of forecasted demand facilitates an understanding of increasing risks with time, without the need to forecast future hydrology.

4.2.2 USGS Gauge Records

The hydrologic period of record for the model is January 1, 1975 through December 31, 2009 (35 years). This is the period of available historic data for the USGS Gauge on the Harpeth River at Franklin, TN (gauge #03432350). The USGS gauging station at Franklin is located downstream of the Franklin Water

Treatment Plant intake and upstream of the Franklin Sewage Treatment Plant (WWTP) discharge. This period includes a sufficient variation in hydrologic conditions including 3 weeks with average flows less than the 7Q10 (0.5 cubic feet per second (cfs) or 0.3 million gallons per day (mgd)) and 3 weeks with average flows greater than the 99th percentile average daily high flow (3,700 cfs or 2,400 mgd). The annual average flows in this time period range from a 93.3 cfs or 60 mgd (1981) to 575 cfs or 370 mgd (1979). Future conditions are evaluated by applying the historical data from the hydrologic period of record over any given future year of projected demand.

There is a second USGS gauging station on the Harpeth River downstream of the City, #03432400, that has streamflow data available from October 1, 1988 through December 31, 2009. This gauge was used in calculating runoff contributions to streamflow, which are discussed in Section 4.5.

4.2.3 Water Supply Withdrawal Records

Monthly operating reports (MORs) from the Franklin Water Treatment Plant (WTP) contain the daily flow through the plant for the hydrologic period of record, 1975 through 2009. There are no records of the actual withdrawal from the river for this period, so the daily volume of treated water delivered by the water plant was used as a proxy for the river withdrawals. This time series was added back into the USGS gauge records in order to develop a naturalized upstream flow boundary condition to the model.

4.2.4 WWTP Discharge Records

MORs from the WWTP containing discharge records are available only for 1999 through 2009. Precipitation records from the WWTP rain gauge are available for 1928 through 2009. Based on Franklin population, seasonal rainfall, and recent WWTP discharge records, the time series of effluent flow to the river was extended back in time to cover the entire hydrologic period of record. An observed relationship between per capita wastewater discharge to the river and annual rainfall volume was used, along with observed seasonal variation in wastewater discharge, to calculate estimated seasonal WWTP discharge for the years prior to 1999. The artificial WWTP discharge record was an input to the stormwater calculations discussed in Section 4-5 and a factor in the Harpeth River model mass balance. **Figure 4-2** shows the synthetic WWTP discharge time series (1975-1998) and the actual MOR data (1999-2009).

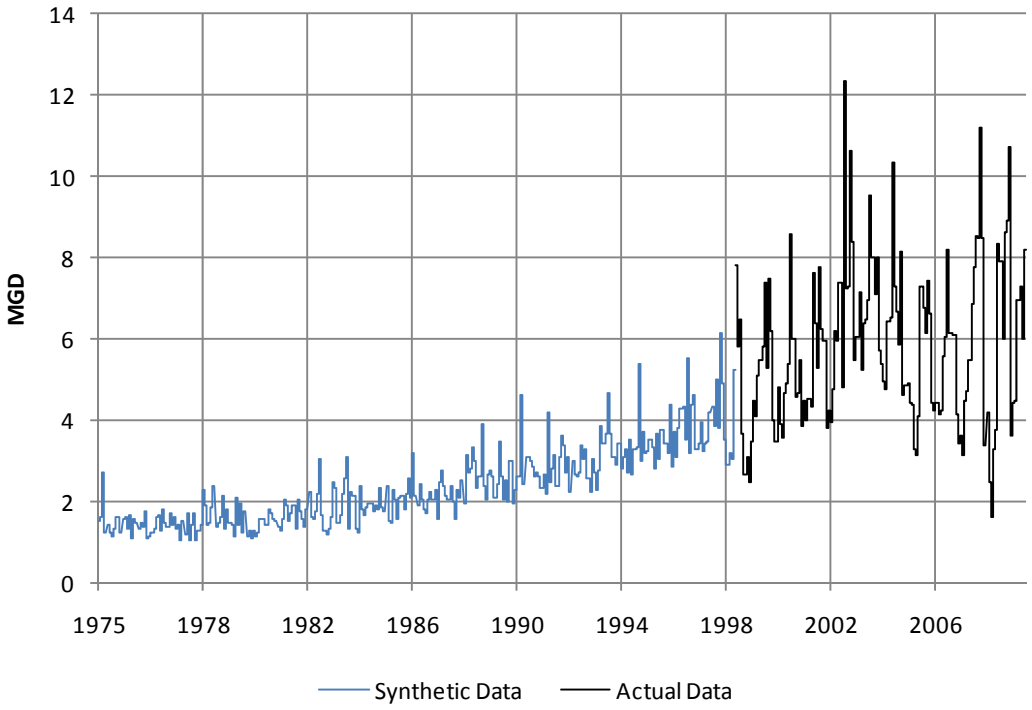


Figure 4-2
Calculated and Actual WWTP Discharges

4.2.5 Mass Balance

Figure 4-3 shows the mass balance diagram for the Harpeth River sector of the Franklin integrated model. For any given scenario, the sum of all inflow and outflow volumes must be equal to zero for over the hydrologic period of record. That is, all water entering the river as an upstream boundary condition, WWTP discharge, stormwater flow, or base flow return must exit the system via withdrawal or simulated flow downstream of the modeled city boundary. The river model balances with respect to total volume in million gallons after each model run covering the entire hydrologic period of record. Historical input data is used as a boundary condition to the model, whereas model calculations are variable with each scenario and depend upon the management conditions applied to the model (e.g., WWTP capacity and stormwater control measures)

Inflows to the Harpeth River model sector include the following:

- Historical USGS gauge records upstream of Franklin WTP
- Historical WTP withdrawals (added to normalize historic USGS gage records)
- Modeled WWTP discharges
- Modeled stormwater flow from managed collection measures (BMPs)

- Modeled stormwater flow from unmanaged conveyance and collection
- Modeled base flow return from irrigation and septic systems

Outflows from the Harpeth River model sector include the following:

- Modeled water treatment plant withdrawal
- Modeled flow downstream of the modeled city boundary

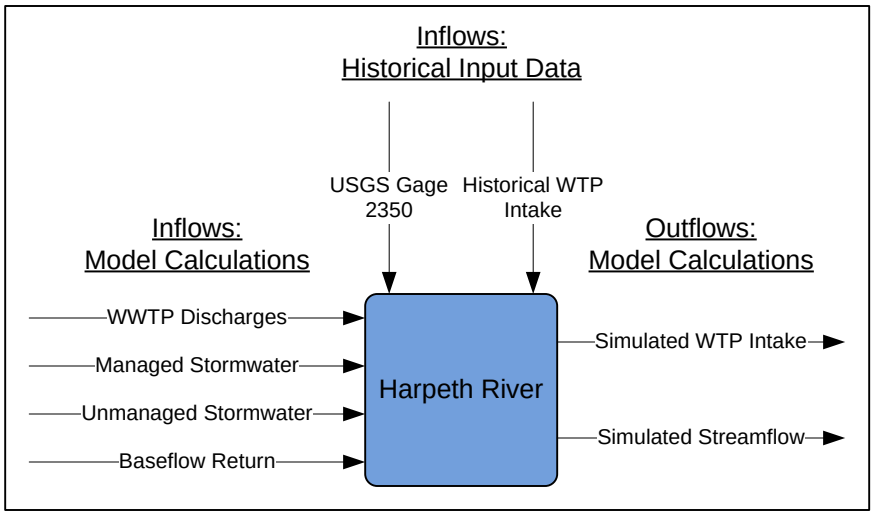


Figure 4-3
Harpeth River Model Sector Mass Balance

4.2.6 Spatial Orientation

Figure 4-4 shows the sequence of inflows into and withdrawals from the modeled Harpeth River system. The inputs are labeled with letters which correspond to how they are added across the system. This summation is shown in the downstream flow equation. Input A is the upstream boundary condition. Inputs B and C are wholly dependent upon the options selection under the scenario being modeled. WTP withdrawal (D) is a result of various factors including, but not limited to, the demand for potable water, the low-flow limitation on river withdrawals, the intake pump capacity, the raw water reservoir level and capacity, and the WTP capacity. Input E, stormwater volume, is represented by a single addition to the river flow, rather than a continuous input along the length of the river. The single point represents the total volume of stormwater flowing to the river within the modeled city area. (Note that the aggregation of stormwater flows is appropriate; because the information required by stakeholders was related to bulk effects of stormwater at downstream locations, rather than the distributed effects throughout the river, which will be examined with more detailed river models.) This input is discussed in more detail in Section 4-5. Though much smaller by comparison, base flow return to

the river (G) is also quantified at a single point, and includes only irrigation and septic system recharge. Precipitation infiltration changes are indirectly quantified in stormwater calculations. Input F is the result of various factors including, but not limited to, potable water use, sewering, WWTP capacity, and reclaimed water use.

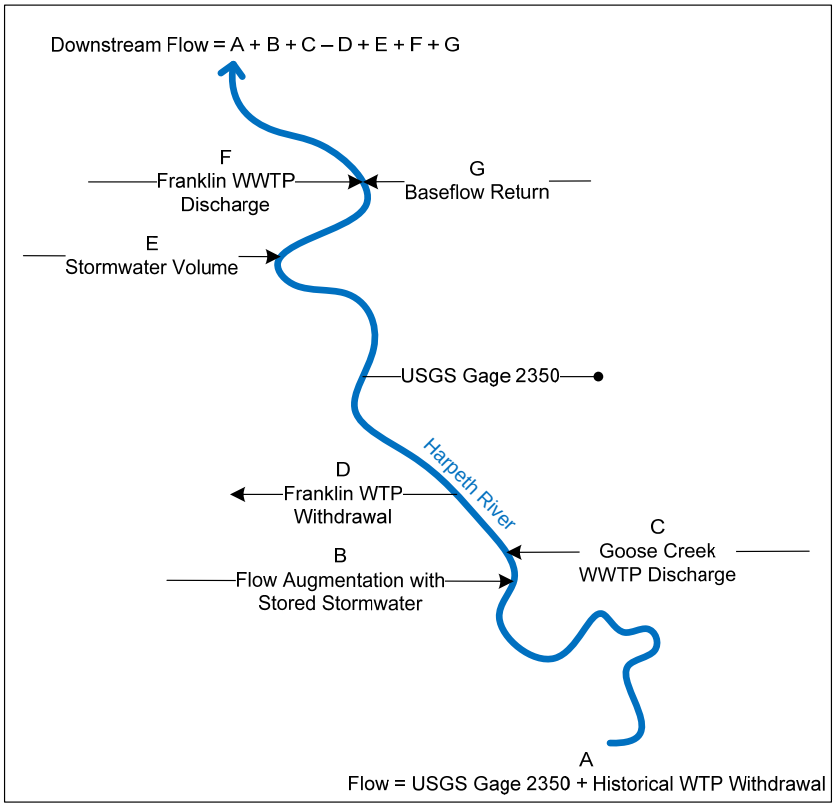


Figure 4-4
Spatial Orientation of Modeled Harpeth River Inflows and Outflows

4.3 Potable Water Supply

4.3.1 Water Demand Projections

Water demands were forecasted in six 5-year increments for the planning period beginning in 2015 and ending in 2040. Water demand projections had previously been developed for the City of Franklin for several development scenarios and reported in the *Jackson Thornton Utilities' Independent Evaluation of Feasibility Study,* conducted by Metcalf & Eddy (June 2008). The annual demands for a moderate growth scenario were disaggregated into water demands by use sector for model input: residential essential, residential irrigation, commercial essential, commercial irrigation, recreational essential, recreational irrigation, and industrial essential. Essential demands are used as an estimate for what is needed for drinking, bathing, necessary industrial processes, and moderate lawn watering. The moderate growth scenario was based on the City's *2004 Land Use*

Plan that specified the average water demand in 2005 at 6.25 mgd, a projected growth of 0.18 mgd per year from 2005 to 2020, and a projected growth of 0.09 mgd from 2020 to 2040. The percentages of total water demand fitting into the seven water use sectors listed above were estimated using the City's recent billing records and were not varied by projected demand year. Recreational demands include the irrigation water use of golf courses. **Table 4-2** lists the annual total water demand projections, and **Table 4-3** lists the percentage of demand partitioned to each of the water use sectors.

Table 4-2
Annual Total Water Demand Projections

Demand Year	Total Annual Demand, MG
2015	2,752
2020	3,081
2025	3,246
2030	3,411
2035	3,576
2040	3,741

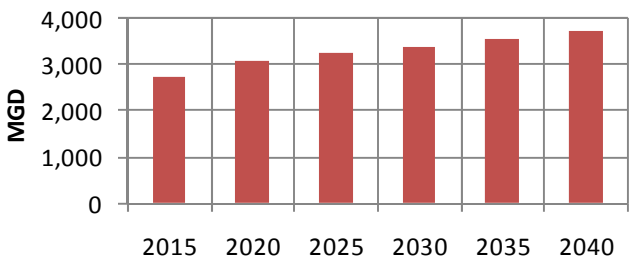
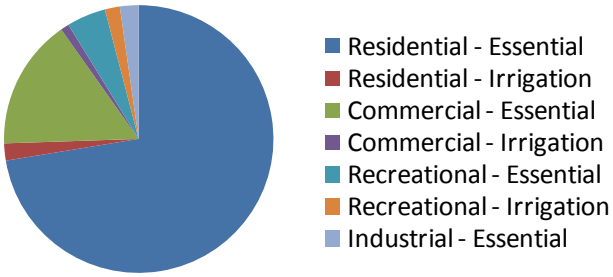


Table 4-3
Demand Partitioning by Water Use Sector

Water Use Sector and Type	Percent of Total Demand
Residential - Essential	72.4%
Residential - Irrigation	2.1%
Commercial - Essential	15.8%
Commercial - Irrigation	1.0%
Recreational - Essential	4.7%
Recreational - Irrigation	1.8%
Industrial - Essential	2.3%



To capture the seasonality of water demands, monthly multipliers were used to develop a monthly average water demand for each use sector. The multipliers were calculated from the City's billing records obtained from 2000 through 2009. **Table 4-4** shows the monthly multipliers for the different use sectors (residential, commercial, recreational, and industrial) and use types (essential or irrigation). The average monthly values were used as the demand for each day in that month.

Table 4-4
Monthly Multipliers for Water Demands

Month	Essential Demand	Irrigation Demand	Industrial Demand
January	0.86	0.12	1.0
February	0.83	0.08	1.0
March	0.83	0.08	1.0
April	0.85	0.15	1.0
May	0.92	0.46	1.0
June	1.04	1.07	1.0
July	1.16	1.77	1.0
August	1.23	2.20	1.0
September	1.21	2.13	1.0
October	1.11	1.71	1.0
November	1.05	1.62	1.0
December	0.91	0.60	1.0

Figures 4-5 through **4-8** show the resulting monthly water demands, by use type and water use sector, used as model input for each of the demand years from 2015 to 2040. The monthly variation is shown in the bar charts, with the darker color bands representing the increasing demand for each 5-year increment.

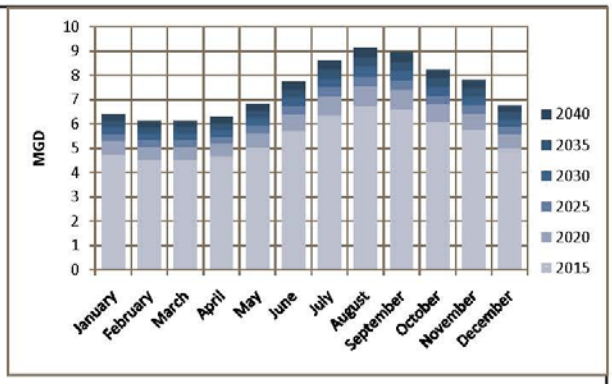
Water distribution system leakage also effectively places a demand on the system. If the City's water users need 10 mgd, for example, but the system will leak 1 mgd, the City must provide 11 mgd to the distribution system. The current leakage rate was estimated at 1 mgd. This estimate is based on data from 2008-2009 records that showed 0.07 mgd of known leaks in the system and a total volume of unaccounted for water of 1.5 mgd. The actual leakage rate is somewhere between these two values, assuming not all leaks are known and not all unaccounted for water is leakage (paper losses, for example). The leakage rate was not escalated for the next 30 years but was reduced by 50 percent when the option to address distribution system leakage was activated.

4.3.2 Water Supply Model Sector

The Franklin IWRP explores multiple pathways for obtaining the water needed to meet the City's demands. They are the following:

- Harpeth River raw water
- Large regional wholesaler – Harpeth Valley Utility District (HVUD)
- Cumberland River raw water

Residential - Essential						
Demand in MGD	Demand Year					
	2015	2020	2025	2030	2035	2040
January	4.71	5.27	5.55	5.83	6.12	6.40
February	4.51	5.05	5.32	5.59	5.86	6.13
March	4.51	5.05	5.33	5.60	5.87	6.14
April	4.63	5.19	5.47	5.74	6.02	6.30
May	5.02	5.62	5.92	6.22	6.52	6.82
June	5.70	6.38	6.72	7.06	7.40	7.74
July	6.34	7.10	7.48	7.86	8.24	8.62
August	6.72	7.53	7.93	8.33	8.74	9.14
September	6.59	7.38	7.77	8.17	8.56	8.96
October	6.06	6.78	7.15	7.51	7.87	8.23
November	5.74	6.42	6.77	7.11	7.45	7.80
December	4.98	5.58	5.88	6.18	6.47	6.77
Average	5.46	6.11	6.44	6.77	7.09	7.42



Residential - Irrigation						
Demand in MGD	Demand Year					
	2015	2020	2025	2030	2035	2040
January	0.02	0.02	0.02	0.02	0.02	0.03
February	0.01	0.01	0.02	0.02	0.02	0.02
March	0.01	0.01	0.01	0.02	0.02	0.02
April	0.02	0.03	0.03	0.03	0.03	0.03
May	0.07	0.08	0.09	0.09	0.09	0.10
June	0.17	0.19	0.20	0.21	0.22	0.23
July	0.28	0.31	0.33	0.34	0.36	0.38
August	0.34	0.39	0.41	0.43	0.45	0.47
September	0.33	0.37	0.39	0.41	0.43	0.45
October	0.27	0.30	0.32	0.33	0.35	0.36
November	0.25	0.29	0.30	0.32	0.33	0.35
December	0.09	0.11	0.11	0.12	0.12	0.13
Average	0.16	0.18	0.18	0.19	0.20	0.21

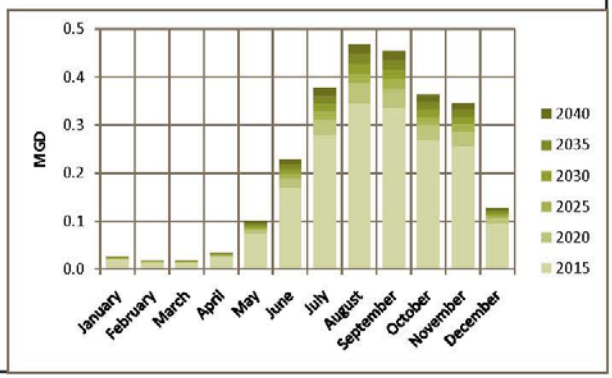
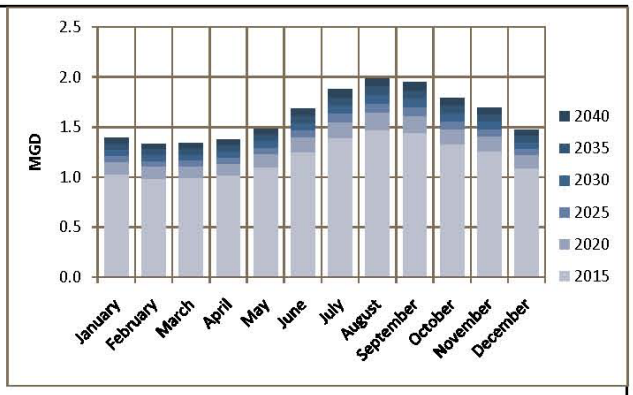


Figure 4-5
Modeled Residential Water Demands 2015 to 2040

Commercial - Essential						
Demand in MGD	Demand Year					
	2015	2020	2025	2030	2035	2040
January	1.02	1.15	1.21	1.27	1.33	1.39
February	0.98	1.10	1.16	1.22	1.27	1.33
March	0.98	1.10	1.16	1.22	1.28	1.34
April	1.01	1.13	1.19	1.25	1.31	1.37
May	1.09	1.22	1.29	1.35	1.42	1.48
June	1.24	1.39	1.46	1.54	1.61	1.69
July	1.38	1.55	1.63	1.71	1.79	1.88
August	1.46	1.64	1.73	1.81	1.90	1.99
September	1.43	1.61	1.69	1.78	1.86	1.95
October	1.32	1.48	1.56	1.63	1.71	1.79
November	1.25	1.40	1.47	1.55	1.62	1.70
December	1.08	1.21	1.28	1.34	1.41	1.47
Average	1.19	1.33	1.40	1.47	1.54	1.62



Commercial - Irrigation						
Demand in MGD	Demand Year					
	2015	2020	2025	2030	2035	2040
January	0.01	0.01	0.01	0.01	0.01	0.01
February	0.01	0.01	0.01	0.01	0.01	0.01
March	0.01	0.01	0.01	0.01	0.01	0.01
April	0.01	0.01	0.01	0.01	0.02	0.02
May	0.04	0.04	0.04	0.04	0.05	0.05
June	0.08	0.09	0.10	0.10	0.11	0.11
July	0.14	0.15	0.16	0.17	0.18	0.18
August	0.17	0.19	0.20	0.21	0.22	0.23
September	0.16	0.18	0.19	0.20	0.21	0.22
October	0.13	0.15	0.15	0.16	0.17	0.18
November	0.12	0.14	0.15	0.15	0.16	0.17
December	0.05	0.05	0.05	0.06	0.06	0.06
Average	0.08	0.09	0.09	0.10	0.10	0.10

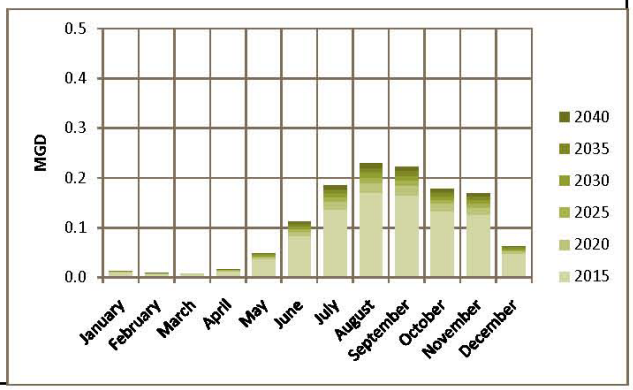
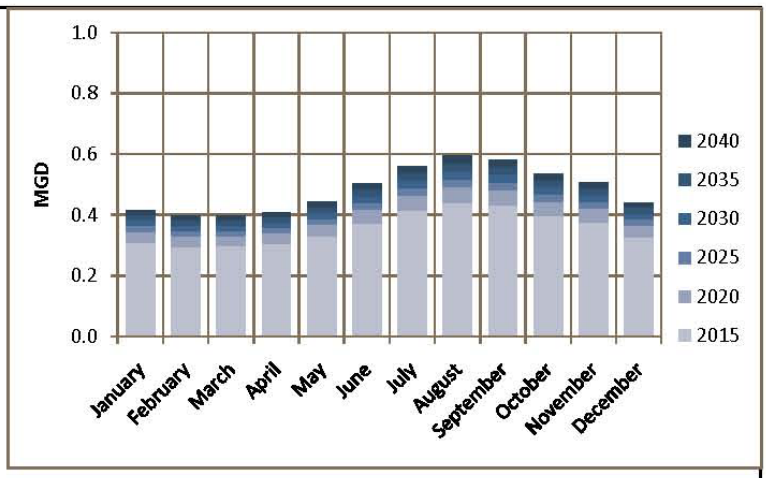


Figure 4-6
Modeled Commercial Water Demands 2015 to 2040

Recreational - Essential						
Demand in MGD	Demand Year					
	2015	2020	2025	2030	2035	2040
January	0.31	0.34	0.36	0.38	0.40	0.42
February	0.29	0.33	0.35	0.36	0.38	0.40
March	0.29	0.33	0.35	0.36	0.38	0.40
April	0.30	0.34	0.36	0.37	0.39	0.41
May	0.33	0.37	0.38	0.40	0.42	0.44
June	0.37	0.41	0.44	0.46	0.48	0.50
July	0.41	0.46	0.49	0.51	0.54	0.56
August	0.44	0.49	0.52	0.54	0.57	0.59
September	0.43	0.48	0.51	0.53	0.56	0.58
October	0.39	0.44	0.46	0.49	0.51	0.54
November	0.37	0.42	0.44	0.46	0.48	0.51
December	0.32	0.36	0.38	0.40	0.42	0.44
Average	0.35	0.40	0.42	0.44	0.46	0.48



Recreational - Irrigation						
Demand in MGD	Demand Year					
	2015	2020	2025	2030	2035	2040
January	0.02	0.02	0.02	0.02	0.02	0.02
February	0.01	0.01	0.01	0.01	0.01	0.01
March	0.01	0.01	0.01	0.01	0.01	0.01
April	0.02	0.02	0.02	0.03	0.03	0.03
May	0.06	0.07	0.07	0.08	0.08	0.08
June	0.14	0.16	0.17	0.18	0.19	0.19
July	0.24	0.27	0.28	0.29	0.31	0.32
August	0.30	0.33	0.35	0.37	0.38	0.40
September	0.29	0.32	0.34	0.36	0.37	0.39
October	0.23	0.26	0.27	0.28	0.30	0.31
November	0.22	0.24	0.26	0.27	0.28	0.30
December	0.08	0.09	0.10	0.10	0.10	0.11
Average	0.13	0.15	0.16	0.17	0.17	0.18

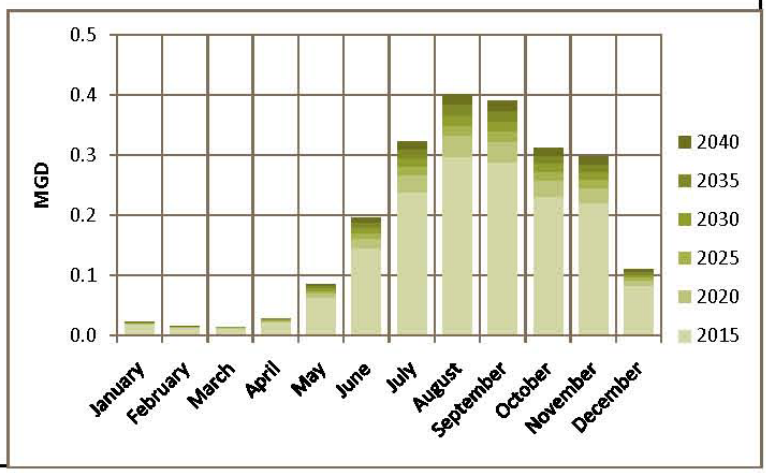


Figure 4-7
Modeled Recreational Water Demands 2015 to 2050

Industrial - Essential						
Demand in MGD	Demand Year					
	2015	2020	2025	2030	2035	2040
January	0.17	0.19	0.20	0.21	0.22	0.23
February	0.17	0.19	0.20	0.21	0.22	0.23
March	0.17	0.19	0.20	0.21	0.22	0.23
April	0.17	0.19	0.20	0.21	0.22	0.23
May	0.17	0.19	0.20	0.21	0.22	0.23
June	0.17	0.19	0.20	0.21	0.22	0.23
July	0.17	0.19	0.20	0.21	0.22	0.23
August	0.17	0.19	0.20	0.21	0.22	0.23
September	0.17	0.19	0.20	0.21	0.22	0.23
October	0.17	0.19	0.20	0.21	0.22	0.23
November	0.17	0.19	0.20	0.21	0.22	0.23
December	0.17	0.19	0.20	0.21	0.22	0.23
Average	0.17	0.19	0.20	0.21	0.22	0.23

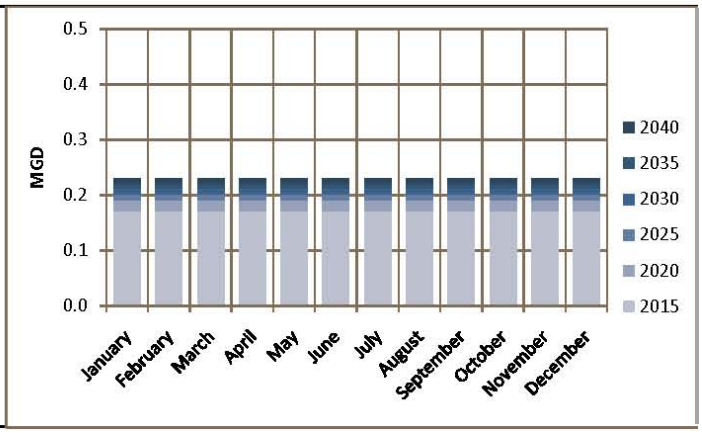


Figure 4-8
Modeled Industrial Water Demands 2015 to 2040

The water supply sector uses user-specified input parameters, along with natural and imposed constraints on the system, to draw water from one or a combination of the sources listed above. **Figure 4-9** shows a representation of the flow of water into Franklin's distribution system. Raw water, either from the Harpeth River or other regional sources, enters the system through the raw water reservoir. Basic reservoir inflows and outflows are included in the model, including direct rainfall, evaporation, leakage, and backwash. Water from the reservoir flows, as demanded, into the Franklin WTP and is then combined with regional treated sources to meet total potable water demands. Different scenarios modeled for the Franklin IWRP used different combinations of water sources to evaluate the cost and performance of Franklin's various water supply options. The following sections discuss the assumptions and specifications of the modeled sources of water.

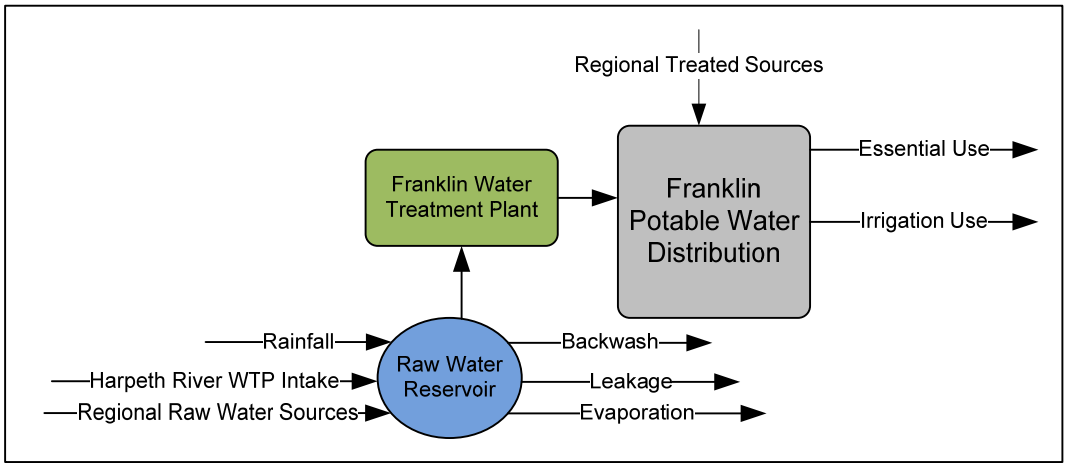


Figure 4-9
Water Supply Sector Schematic

4.3.3 Franklin Water Treatment Plant

The treatment capacity of Franklin WTP presents a constraint for raw water coming into the water supply system. The existing capacity is 2.1 mgd, and three options are included in the model for future capacities: maintain the existing 2.1 mgd, upgrade the plant to 4.0 mgd, and upgrade the plant to a capacity that would treat all of the City's demand for the next 30 years. The modeled demand projections include a maximum monthly average demand of 13 mgd in August 2040. The energy required to operate the WTP was estimated with the equation[1,2]:

[1] One of the measures against which all alternatives were compared, in accordance with stakeholder requests for information, was the amount of energy consumed by each major project.
[2] Carlson, Steven W. and Adam Walburger, 2007. Energy Index Development for Benchmarking Water and Wastewater Utilities. AwwaRF

$$E = e^{8.2+\ln(Q)}$$

Equation 1

E: energy required
Q: flow through the plant

4.3.4 Harpeth River as a Supply Source

Under several modeled IWRP alternatives and under existing conditions, water is drawn from the Harpeth River at maximum allowable rates, as governed by pump capacity and low flow withdrawal constraints. The water available for withdrawal in the Harpeth River is specified by the 2007 Aquatic Resource Alteration Permit (ARAP), which states the following two criteria:

1. Flow in the Harpeth River shall not be reduced below 10 cfs (6.46 mgd) as a result of the withdrawal.

2. Water shall be withdrawn at a rate of no more than 20 percent of the flow in the river at the intake.

The current reported existing capacity of the raw water intake pump is 8.1 mgd but will reportedly discharge only about 7 mgd according to the *2006 Design Report for the Franklin Water Treatment Plant*. The pump capacity is a constraint only when water is available for withdrawal from the Harpeth River at a rate of greater than 7 mgd and when the raw water reservoir requires more than 7 million gallons to meet current demand and maintain its water level.

The raw water reservoir was dredged and a new liner put in around the time that the Franklin IWRP Phase I study and modeling were conducted. The reported reservoir capacity in the 2006 design report was 96.7 million gallons, with only 39.3 million gallons of usable volume. The reservoir leakage rate was estimated at 1 mgd in the 2006 Design Report and assumed to be reduced by 75 percent with the new liner. The reservoir repairs are included as a no-cost option in the integrated model, and when the option is activated the greater reservoir capacity and lesser leakage rate are applied.

4.3.5 Regional Potable Sources

Regional potable sources, namely HVUD, are included in the model to supply remaining City demand that cannot be met with water from the Harpeth River. It is also possible to configure the model to supply all of the City's demand by purchasing treated water from HVUD. The agreement between the City and HVUD stipulates a minimum purchase requirement, or minimum cost to the City to buy any water from the wholesaler. To simulate cost efficient operating procedures, the minimum purchase volume is satisfied before other, cheaper sources are utilized to prevent having the City pay for water that it never uses from HVUD. The current minimum purchase requirement in the agreement

between the City and HVUD is 99.7 million gallons per month. In the model, a daily rate of 3.63 mgd is used, which is the estimated minimum purchase volume in 2020 according to the *Jackson Thornton Utilities' Independent Evaluation of Feasibility Study*, conducted by Metcalf & Eddy (June 2008). The capacity of the HVUD pipeline delivering water to Franklin is assumed to be large enough to meet all of the City's demand through 2040.

The energy required to deliver the water via HVUD (originating in the Cumberland River) is estimated using the Darcy-Weisbach friction loss equation and net elevation change to calculate the total hydraulic head:

$$H = f\frac{L}{D} * \frac{V^2}{2g} + z$$

Equation 2a

H = total head loss
f: Darcy friction factor (0.014)
L: pipe length (19 miles)
D: pipe diameter (3 feet)
V: water velocity (flow/area)
g: gravitational acceleration
z: net elevation change of pipeline (360 feet)

Then, using the following equation, the hydraulic head is converted to energy. A pump efficiency of 75 percent was assumed and appropriate conversion factors applied to the equation.

$$E = H * Q * SG$$

Equation 2b

E: energy required
Q: flow through the pipe
SG: specific gravity of water (1.0)

4.3.6 Regional Raw Water Sources

The Franklin integrated model includes an option for the City to meet all of its potable water demand by constructing a pipeline to transport raw water from the Cumberland River. As part of this option, the Franklin WTP would be upgraded to treat all of the City's water supply. A preliminary study was done for the City on various versions of this supply line in 1989 (Franklin Water Facilities and Supply Report). The specifications of the line that are included in the model represent an averaging of the various alternative preliminary designs proposed in this report. **Table 4-5** lists the specifications used in the model. The energy required to transport the water from the Cumberland River to the Franklin WTP is calculated using Equations 2a and 2b above.

Table 4-5
Approximate Cumberland River Supply Line Specifications

Parameter	Model Specification
Length	19 miles
Diameter	3 feet
Darcy friction factor (f)	0.014
Net elevation change	363 feet

4.3.7 Supply Redundancy

An important distinction between Franklin's alternative sources of water is how the City will get water, should its main source be compromised. As part of the integrated modeling analysis, this scenario was simulated for each of the alternatives by turning off the largest source (by volume) and relying only on secondary sources (if any are available). The resulting performance measure was the level to which the City's essential demands could still be met. **Table 4-6** lists the assumptions made as part of this analysis.

Table 4-6
Supply Redundancy Analysis Assumptions

Supply Option	Largest Source by Volume	Secondary Available Source
Withdraw and treat all available water from Harpeth River, satisfy remaining demand with HVUD water purchase	HVUD wholesale purchase	Harpeth River, withdrawal restrictions applied
Shut down WTP, withdraw no water from Harpeth River, satisfy all demand with HVUD water purchase	HVUD wholesale purchase	None (Franklin WTP would be shut down)
Withdraw, transport, and treat water from Cumberland River to satisfy all demand	Cumberland River withdrawal	Harpeth River withdrawal (Assume that this source, while not utilized except in an emergency, would be available, should the City need it. Harpeth River withdrawal restrictions are applied.)

4.4 Wastewater Treatment

The integrated model includes several options for meeting Franklin's long-term wastewater treatment capacity needs. The City's wastewater capacity needs are projected to grow with the increasing water demand, as well as the result of potentially accepting regional wastewater for treatment at the Franklin WWTP. The integrated model compares the daily total demand for treatment with the specified capacity of the wastewater treatment system to calculate the total effluent generated by the plant. The effluent is made available for reclaimed distribution or storage, and that which is not reused is discharged to the Harpeth River. Pollutant loading to the river is calculated based on the discharge volume and permitted concentrations.

Figure 4-10 shows a schematic of the wastewater sector in the Franklin integrated model. The projected demands on wastewater treatment are discussed in the following sections.

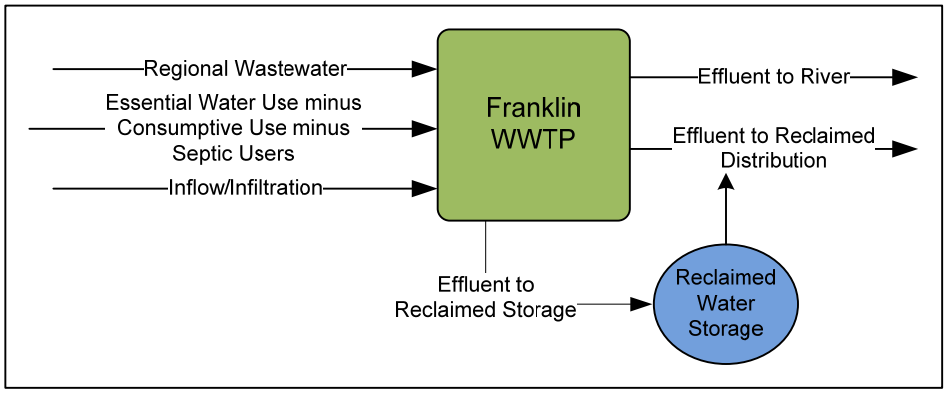


Figure 4-10
Wastewater Treatment Sector Schematic

4.4.1 Wastewater Demand Projections

Demand projections for Franklin's wastewater treatment capacity over the next 30 years are necessarily linked to the City's water supply demand projections. The mass balance of the water resources system depends upon equal volumes of water entering and exiting the system. Wastewater demand projections for use in the integrated model are based on two sources of wastewater: that which is generated within the City as a direct result of water use and that which is imported from outside communities that rely on separate sources of potable water.

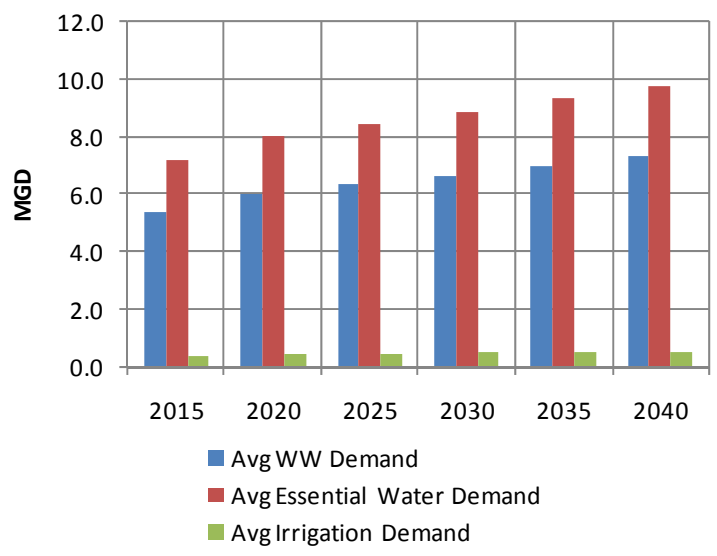
Wastewater demand from City of Franklin water supply customers is calculated using the total essential demand for potable water (see Section 4.3 for water demand values and explanation). The total essential demand is reduced by a factor to represent only usage of water travelling to the WWTP. The wastewater generation factors are estimated monthly averages based on a typical rate of 90 percent of water that is used indoors. Essential use estimates were developed from City billing records that specify some billing for outdoor use. It is assumed that not all outdoor uses (particularly individual residential irrigation) are billed as such. This is apparent in the large increase in indoor residential water demands in the summer months. For wastewater demand purposes, monthly essential water demands were reduced to 90 percent of normal winter essential use. **Table 4-7** shows the wastewater generation factors that were applied to essential potable demand totals to calculate wastewater demand. The percentages are variable throughout the year because, for the purposes of this IWRP, essential demands are greater in the summer than in the winter. **Table 4-8** and the accompanying graph show the in-City wastewater demand projections for 2015-2040.

Table 4-7
Wastewater Generation Factors

Month	Wastewater Generation, as % of Total Essential Demand
January	87
February	90
March	90
April	88
May	81
June	72
July	65
August	61
September	62
October	68
November	72
December	82

Table 4-8
Projected In-City Wastewater Demands

Projection Year	Average Wastewater Demand, mgd
2015	5.5
2020	6.0
2025	6.3
2030	6.7
2035	7.0
2040	7.3



Additional wastewater inflow is added to the total demand on the Franklin system when the option is activated to accept regional wastewater from neighboring communities. Based on discussions with the City, the additional wastewater demand on the system is estimated at 1 mgd.

Inflow and infiltration (I/I) also effectively place a demand on the wastewater treatment system. The volume of I/I entering the collection system is greatly dependent upon rainfall and has not been quantified extensively within the Franklin wastewater collection system. The planning-level estimates of I/I used in the integrated model are based on seasonal rainfall trends and recent wastewater inflow data. The total average monthly rainfall, average wastewater

inflow, and estimated sanitary wastewater generation for 2001 through 2009 were compared to develop an average I/I estimate for input into the model. **Table 4-9** shows the monthly values used for I/I into the collection system. The values were not escalated over the next 30 years, but were reduced by 50 percent, if the option to address I/I was activated.

Table 4-9
Estimated Inflow and Infiltration

Month	Inflow and Infiltration (mgd)
January	3.1
February	4.4
March	3.5
April	3.5
May	3.0
June	1.0
July	0.6
August	0.6
September	1.1
October	1.5
November	1.7
December	3.8
Average	2.3

4.4.2 Franklin Wastewater Treatment Plant

The capacity of the existing plant is set to the current design capacity of 12 mgd. The future capacity, when the option to upgrade the WWTP is activated, is set to be 15 mgd. This value was used in cost and energy calculations and would be sufficient to handle the projected seasonal peak demands in 2040. **Figure 4-11** shows the highest average monthly wastewater demands projected for 2015—2040 along with the existing and upgraded WWTP capacities.

Pollutant loading to the Harpeth River related to WWTP effluent was calculated based on the waste load allocation given to the WWTP in its 2009 NPDES permit. Data from the plant's MORs from 2001 through 2009 shows that the actual loadings are less, but for the purposes of relative comparison of alternatives, the NPDES values are appropriate. **Table 4-10** shows the values in pounds per million gallons used, along with the modeled effluent discharged to the river to calculate the total pollutant loading from the plant in pounds per day.

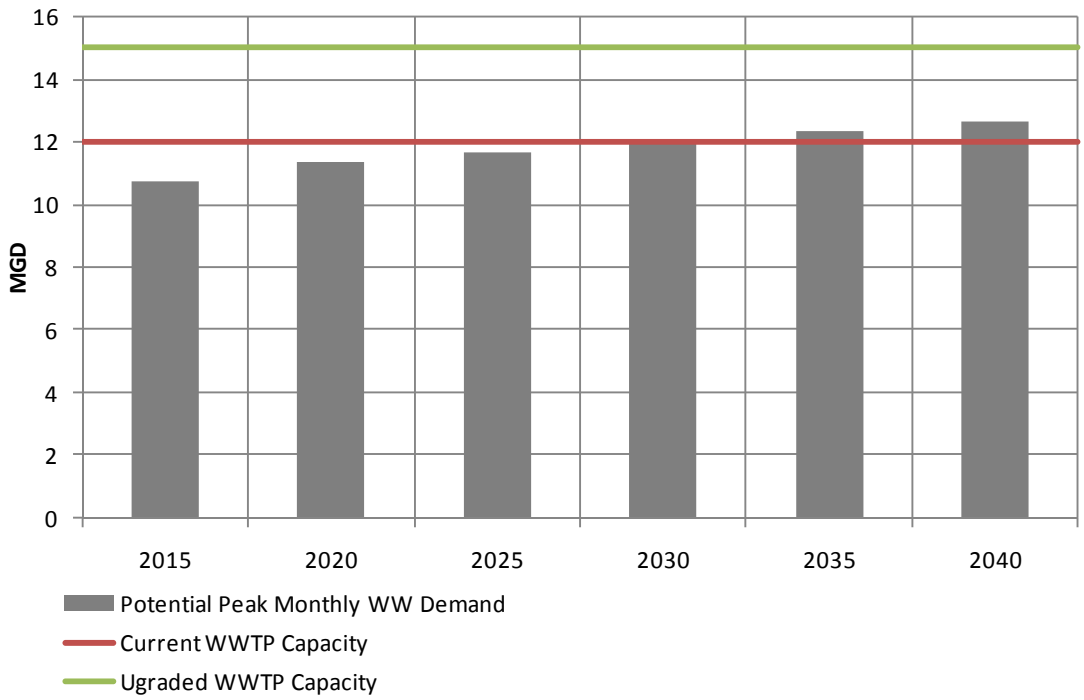


Figure 4-11
Potential Peak Monthly Wastewater Demands and Treatment Capacities (Franklin WWTP)

Table 4-10
Allowable NPDES WWTP Effluent Pollutant Loading Rates

Season	Loading, lbs/MG	
	BOD[3]	Nitrogen
January – April and November – December	83.4	60.0
May – October	33.4	41.7

The integrated model includes an option to reduce the pollutant loading to the river in the summertime by introducing a more advanced method of treatment, which was assumed to be reverse osmosis (RO). The RO system would only be run in the summer months (May-October) and would result in lower nitrogen and BOD loads in those months, which are listed in **Table 4-11**, and derived from RO studies and projects in Tucson, AZ and Miami-Dade County, FL.

[3] Stakeholders identified BOD (Biological Oxygen Demand) and Nitrogen as the two most important indicators of water quality in the Harpeth River. Hence, the model was developed to track these two pollutants.

Table 4-11
Reduced Franklin WWTP Effluent Pollutant Loading Rates

Season	Loading, lbs/MG	
	BOD	Nitrogen
January – April and November – December	83.4	60.0
May – October	10.0	14.6

Energy requirements to run the WWTP were estimated based on the following equation[4]:

$$E = e^{15.8+\ln(Q)}$$

Equation 3

E: energy required
Q: flow through plant

Energy requirements to run the RO system were estimated at approximately 4,000 kWh per million gallons[5].

4.4.3 Goose Creek Wastewater Treatment Plant

A potential solution for meeting Franklin's future wastewater treatment needs is to build a new plant at the location known as Goose Creek. The City has previously acquired this land, which is located on the Harpeth River upstream of the Franklin WTP. The proposed Goose Creek plant capacity was estimated to be 2 mgd. **Figure 4-12** shows the highest average projected wastewater demands for 2015—2040 along with the existing Franklin WWTP capacity and the additional capacity that would be added with the Goose Creek WWTP.

In order to meet anticipated stringent effluent limits and garner public support for a new wastewater plant discharge upstream of a water supply intake, the proposed Goose Creek plant would include advanced treatment processes such as membrane bioreactor (MBR) and tertiary polishing wetlands. The model integrates the advanced treatment into the system by sending 2 mgd of wastewater demand to the Goose Creek plant and reducing the pollutant loads in the effluent. **Table 4-12** shows the reduced pollutant loads that can be expected from using MBR and tertiary polishing wetlands.

[4] Carlson, Steven W. and Adam Walburger, 2007. Energy Index Development for Benchmarking Water and Wastewater Utilities. AwwaRF
[5] Voutchkov, Nikolay. Seawater Reverse Osmosis Design and Optimization. Advance Membrane Technologies, Stanford University 2008.

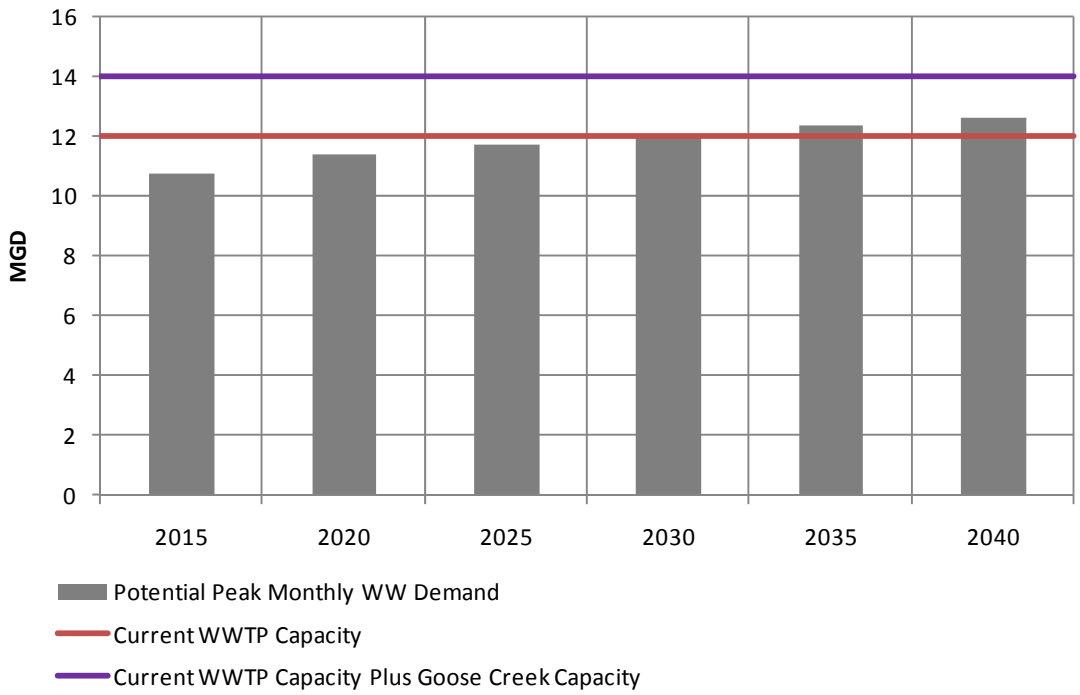


Figure 4-12
Potential Peak Monthly Wastewater Demands and Treatment Capacities (Franklin and Goose Creek WWTPs)

Table 4-12
Estimated Goose Creek WWTP Effluent Pollutant Loading Rates

Season	Loading, lbs/MG	
	BOD	Nitrogen
January – April and November – December	20.9	60.0
May – October	20.9	41.7

4.4.4 Biosolids Management

The integrated model takes into account biosolids management only as far as estimating the volume of solids generated, the cost of each option, and the energy required or generated by the processes. The volume of biosolids generated was estimated based on the total wastewater treated and whether or not processes to generate higher total solids content were activated in the model. The basis of these calculations was an assumption of 11,000 tons of biosolids generated per year under a flow of 10 mgd through the plant, and a 40-percent reduction in weight, if high total solids processes are employed[6]. By using the total WWTP flow to calculated biosolids generation, the total volume of biosolids escalates

[6] Hallsdale-Powell Utility District Beaver Creek WWTP Phase 3 Solids Train Upgrade, Draft Preliminary Engineering Report, November 2009 (CDM)

with wastewater demand over the planning period. The energy required or generated in processing the biosolids varies among the different management options included in the model. **Table 4-13** offers a summary of how the energy estimates are calculated in the model. Energy required to transport biosolids is included in these net estimates, and is based on 15 kWh per gallon of gasoline needed.

Table 4-13
Biosolids Energy Requirements and Generation Estimates

Option	Net Energy Estimate in kWh/ton
Current process with landfill disposal	13 (required)
Current process with Metro Nashville disposal	5 (required)
Upgrade to Class A biosolids	1,800 (required)
Upgrade to ash disposal	1,000 (required)
Upgrade to higher total solids content	650 (required)
Upgrade to Class A and composting	1,000 (required)
Upgrade to biogas	1,000 (generated)
Land application	(no additional energy required)

4.5 Stormwater

The performance of various stormwater management options was evaluated in the model using a simple representation of the Harpeth River drainage basin and Franklin's stormwater system. The stormwater sector of the integrated model is not a hydrologic model of the watershed, nor is it a parameterized model of the City's stormwater collection system. **Figure 4-13** shows a schematic of how the integrated model stormwater sector is set up. Estimated stormwater flows and loads from different land use types are routed either directly to the river (through collection and conveyance), through BMPs, or to localized storage for reuse. Stormwater flowing to the river is quantified at a single point representing the City's aggregated runoff contribution to streamflow. The point of stormwater quantification and measurement of the impact of stormwater is at the downstream end of the modeled City area.

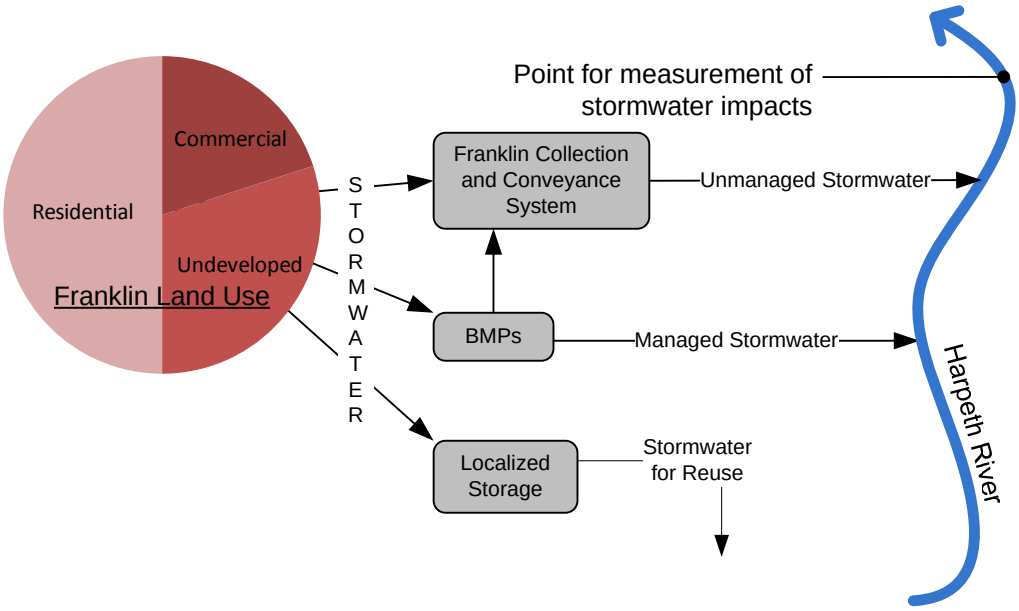


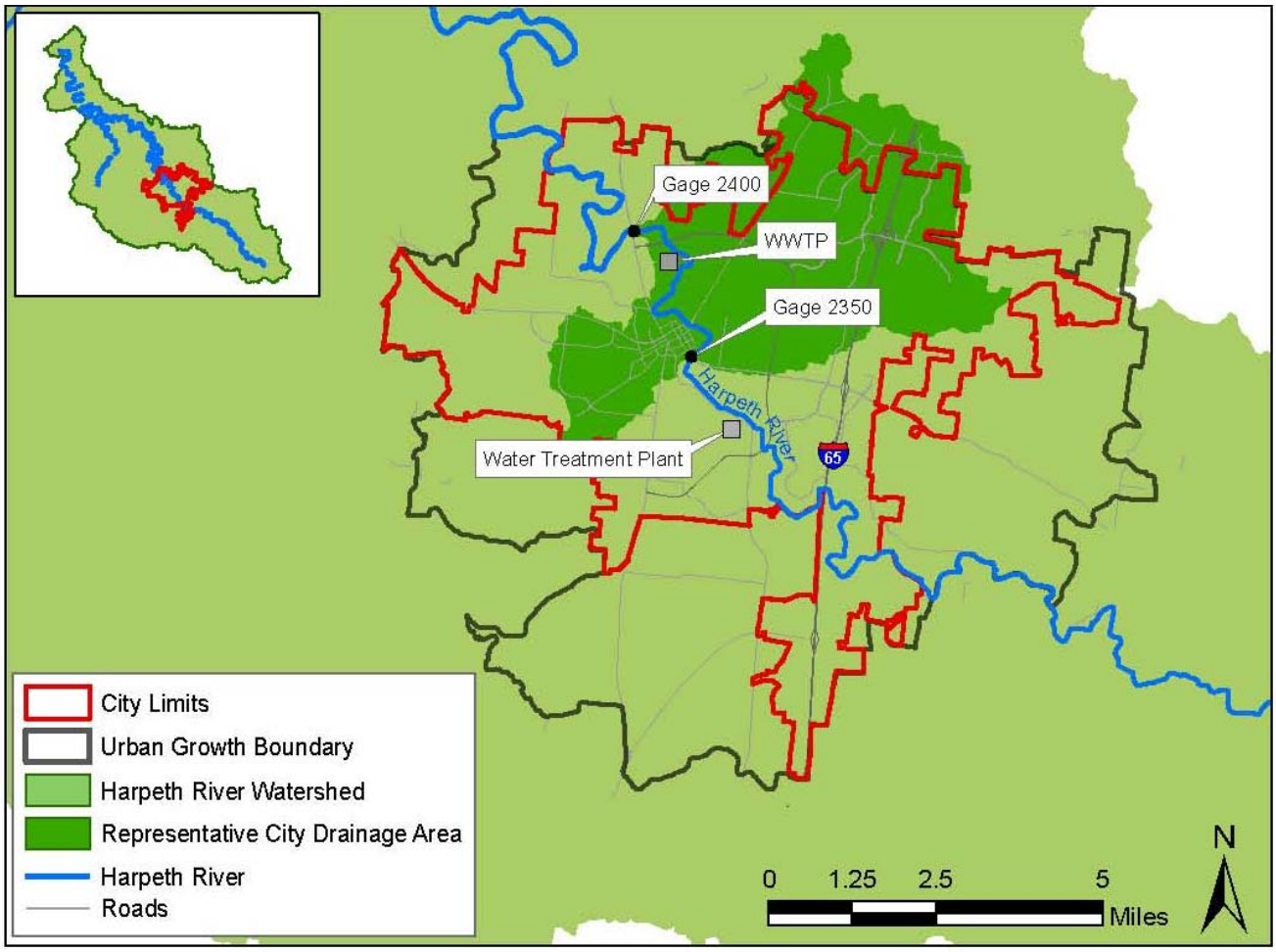
Figure 4-13
Stormwater Sector Schematic

4.5.1 Estimated Stormwater Flow

The model uses a representative volume of stormwater generated by three broad land use types within the City based on streamflow data and river withdrawals and discharges over the hydrologic period of record (1975 through 2009). There are two USGS streamflow gauges on the Harpeth River in Franklin, referred to herein as gauge 2530 and gauge 2400. Gauge 2350 is upstream of most of the City, located just downstream of the WTP intake. Gauge 2400 is located downstream of most of the City and downstream of the wastewater treatment plant discharge. The difference in drainage area of these two gauges (19 square miles) is used as a representative subset of the Harpeth River watershed within the City of Franklin for the purposes of the integrated modeling. **Figure 4-14** shows the stream gauges and the representative drainage area within the City[7]. Streamflow data are available for gauge 2350 for the entire hydrologic period of record, but only incomplete gauge data are available for gauge 2400 from 10/1/1988 through 12/31/2009. A linear relationship between the two gauges was observed (Equation 4 and **Figure 4-15**) and used to calculate the flow at the downstream gauge for days when no data were available.

[7] Note that the representative drainage area for stormwater does not cover the entire watershed nor the entire city of Franklin. However, it was used to study representative stormwater contributions to the Harpeth River because it was bounded by extensive data from which stormwater flow could be directly calculated, it bounds a geographic area that is sensitive to all key decisions on water, wastewater, and reclaimed water, and is a reasonable cross section of land use types in the Franklin community. The model also includes the effects of runoff upstream of this drainage area, as well as projects that may be sited beyond its boundaries, such as the Goose Creek WWTP, for example.

Figure 4-14
Representative City Stormwater Drainage Area



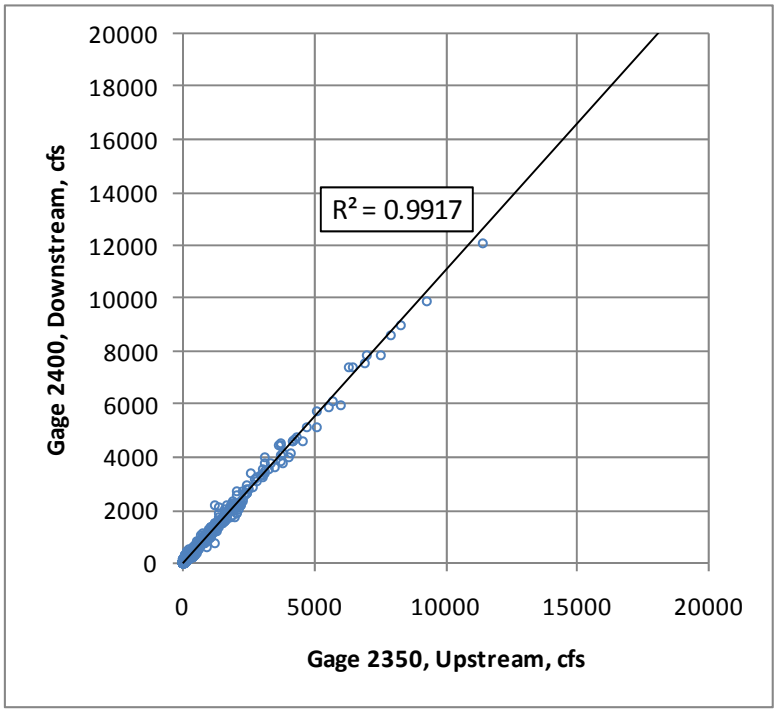


Figure 4-15
Streamflow Gauge Correlation

$$Q_{2400} = 1.105Q_{2350}$$

Equation 4

Q_{2400}: flow at downstream gauge
Q_{2350}: flow at upstream gauge

The flow difference between the two gauges (and without the estimated WWTP discharge discussed in Section 4.2) is used in the model as a daily time series of historical stormwater volume from the representative drainage area. This time series represents the stormwater that is available for collection and reuse, has potential to be managed through BMPs, and may eventually be quantified as stormwater flow to the river.

4.5.2 Land Use Types

The stormwater volume is partitioned into daily flow from three landuse types to facilitate calculations of typical runoff loads and estimate stormwater availability for land use specific BMPs and reuse strategies. The development of Franklin's land over the planning period is derived from the 2004 Land Use Inventory and discussions with the City. **Table 4-14** shows the fraction of the City's land that falls into the three relevant land use categories. Runoff from each of the land use categories is not directly proportional to that land cover fractions in the table but, rather, is a function of the percentage of land cover and the typical runoff

coefficient for that land use. For example, runoff from commercial land in 2015 would be higher than 11 percent of the total, and this would be balanced by runoff from undeveloped land being less than 55 percent of the total. But the runoff coefficients are only applied to the fraction of land to which they are relevant. The effective runoff coefficients for the three land use types are Residential 1.0, Commercial 1.8, and Undeveloped 0.51.

Table 4-14
Franklin Projected Land Use

Projection Year	Residential	Commercial	Undeveloped	
2015	34%	11%	55%	
2020	37%	12%	51%	
2025	41%	14%	46%	
2030	44%	15%	41%	
2035	48%	17%	36%	
2040	51%	18%	31%	

4.5.3 Pollutant Loads

Pollutant loading to the Harpeth River due to stormwater was estimated using typical values of BOD and nitrogen concentrations found in stormwater from the USGS Nationwide Urban Runoff Program (NURP) database[8]. The concentrations from Knoxville, TN were averaged with the national average concentrations to estimate the pollutant loads in Franklin. It is likely that Franklin's stormwater contains more or less BOD and nitrogen than the values used in the model; but for comparative purposes, the estimations of stormwater pollutants developed

[8] Stakeholders identified BOD (Biological Oxygen Demand) and Nitrogen as the two most important indicators of water quality in the Harpeth River. Hence, the model was developed to track these two pollutants.

from the NURP database are sufficient. **Table 4-15** lists the concentrations used in the model by land use type. These concentrations are multiplied by the modeled stormwater flow from each land use type to calculate loading to the river in pounds per day.

Table 4-15
Estimated Pollutant Concentrations in Stormwater

Land Use Type	BOD, mg/L	Nitrogen, mg/L
Residential	11	2.3
Commercial	11	2.6
Undeveloped	8.3	1.6

Several options included in the IWRP involved stormwater BMPs, ordinance, and other strategic controls to reduce the negative impacts of stormwater on the Harpeth River. As the model is not a representation of watershed or stream channel processes, it is not possible to quantify the effects of these various options on flood levels or flood frequency (i.e., hydraulic response to stormwater runoff). Therefore, flooding impacts have been included as qualitative performance measures: negative impacts of stormwater reduced and change in 100-year flood elevation. The total volume of stormwater, as it affects river flow, is simulated in the model hydrologically, but not hydraulically.

Projects and policies in the IWRP that aim to reduce stormwater pollutant loading to the river are considered in the model by reducing the volume of stormwater or the concentration of pollutants in stormwater that reaches the river. For example, residential rain barrels intercept stormwater flow from pervious surfaces, therefore reducing the volume that flows unabated to the river. Constructed wetlands also reduce volume by facilitating infiltration, but also reduce levels of pollutants in stormwater that eventually flows to the river. In the model, estimates of BMP sizes and pollution reduction capabilities are used to modify the basic flow concentration equation to calculate the resulting, reduced load when BMPs are activated. **Table 4-16** lists the assumptions of size and pollutant reduction capabilities used in the model.

Table 4-16
Stormwater BMP Assumptions

Flow Captured by BMPs	**% of Flow from Total Drainage Area**		
Constructed Wetlands	5%		
Pervious Pavement	5%		
Rain Gardens	7%		
Reductions Performed on Flow Captured by BMPs -			
BMP Pollutant Reductions[1]	**BOD**	**Nitrates**	**TKN**
Constructed Wetlands	20%	30%	30%
Pervious Pavement	20%	65%	65%
Rain Gardens	20%	50%	50%
Runoff Lost through ET, Infiltration, etc.	**% of Inflow to BMP**		
Constructed Wetlands	20%		
Pervious Pavement	80%		
Rain Gardens	10%		

1: State of Georgia Stormwater Manual

4.6 Reclaimed Water Distribution

The Franklin integrated model represents the City's reclaimed water system by comparing the water available for reuse—wastewater effluent and collected stormwater—with the demand for reclaimed water and the infrastructure available to store and transport the water. Several key assumptions were made in model development and will be explained in this section:

- The City predicts that there is an untapped demand for reclaimed water. Historical non-essential use patterns are therefore not completely explanatory in the development of future demand projections. The City believes that the demand for reclaimed water will increase with improved and increased infrastructure to deliver the water to customers. In other words, extrapolating historical use of reclaimed or non-essential water use into the future would likely under predict the actual demand, once the infrastructure and the resource itself are fully available.

- Projected demand for water use sectors is based on the City's 2009 Reclaimed Water System Master Plan.

- Water reuse will offset the demand for potable water for irrigation, but this offset must be limited to the volume of potable water that customers would actually purchase to meet their irrigation needs. In the model, this limit is based on recent billing data for irrigation uses (See Section 4.3 for discussion and values of irrigation water demands).

Figure 4-16 shows a schematic of the modeled reclaimed water distribution system. Similarly to potable water use, reclaimed water use is segmented into 4 sectors: residential, commercial, recreational, and industrial. There are two sources of water available for reuse: WWTP effluent as a large, centralized source requiring major infrastructure for distribution, and collected stormwater as a smaller, decentralized source requiring individual customers to initiate collection and reuse. Both sources would otherwise flow to the Harpeth River.

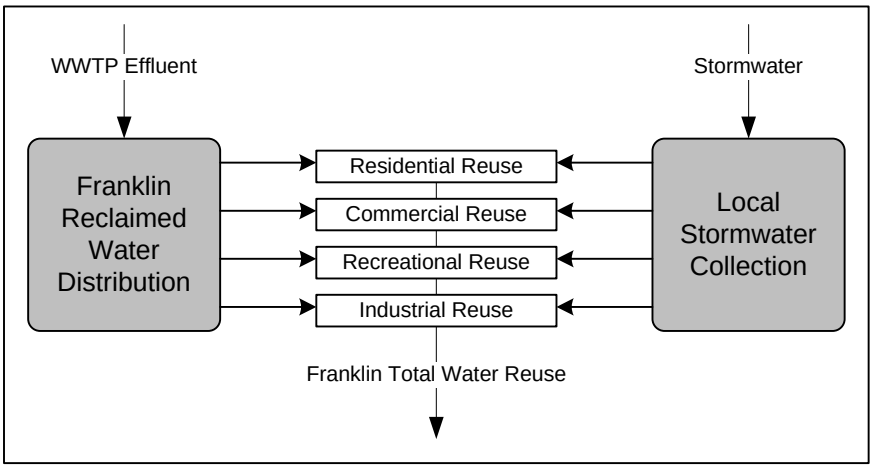


Figure 4-16
Franklin Reclaimed Water Model Schematic

4.6.1 Stormwater Reuse

Stormwater reuse volumes are difficult to estimate with accuracy without further study into potential collection technologies and locations. The model uses estimated rates of capture for residential, commercial, and recreational users to calculate how much stormwater could be made available for reuse on a localized scale. When stormwater reuse options are activated, the demand on the reclaimed wastewater system is reduced by the appropriate amount. **Table 4-17** lists the assumptions that define the amount of available stormwater for reuse. The land use sectors are discussed with the stormwater model sector, Section 4.5, and correspond to the fraction of the land area included in the representative City stormwater drainage basin that could contribute runoff to the respective stormwater collection for reuse. These fractions represent the percentage of stormwater runoff from each land use that is collected and available for reuse. The land fractions change throughout the planning period according to land development projections set forth in the City of Franklin's 2004 Land Use Inventory. The modeled captured runoff is equal to the rainfall volume over the fractional land area, not exceeding the maximum capture volume. Daily rainfall totals for the hydrologic period of record were obtained from the Franklin WWTP rain gauge. The maximum capture rate is small in order to represent the volume of water that would actually be available for use after potential

infiltration, evaporation, storage limitations and other factors that may prevent rain water from being reused.

Table 4-17
Assumptions Defining Stormwater Available for Reuse

Sector	Runoff Fraction Available (%)	Maximum Capture Rate (Inches of Rainfall per Day)
Residential	2	0.1
Commercial	20	0.1
Recreational	20	0.1

4.6.2 Reclaimed Wastewater Effluent

The total volume of wastewater effluent available for distribution is decided in the wastewater treatment sector of the integrated model and is ultimately the result of total potable water use within the City (Section 4.4). The wastewater effluent is pumped into the reclaimed water distribution system. The current capacity of the pump is 7.5 mgd, and the modeled future capacity with upgrades is 12 mgd. Reclaimed water distribution options in the model represent the various projects discussed in the *Reclaimed Water System Master Plan* (2009) and establish how much reclaimed water can be transported to customers. Through discussions with the City, the capacities and target sectors for each of the reclaimed water infrastructure projects were established and are listed in **Table 4-18**.

Table 4-18
Reclaimed Water Infrastructure Capacities

Infrastructure Option	Line Capacity (mgd)	Customer Type, Res/Com/Rec/Ind
Current Distribution Lines	1.0	Residential
	2.0	Commercial
	3.0	Recreational
12" Long Lane Line	3.5	All
Horton Lane Force Main	1.6	Residential, Recreational
12" Columbia Ave/SE Pkwy Line	3.6	All
Total	**14.7**	

An option to incorporate significant reclaimed water storage to alleviate the seasonality of demands is also included in the model. The modeled volume of this new storage is 30 million gallons. This would allow alternative plans to manage effluent volumes into the river.

Based on current demands, the recreational sector is the largest user of reclaimed water (principally, golf courses). The model logic is set to supply the recreational sector with reclaimed water to meet its demands first and then split the remaining supply between the commercial and residential sectors. Currently, the industrial sector does not provide a large demand for reclaimed water, though

the potential for installing lines to provide a less seasonally variable flow of reclaimed water to industrial users will be explored in Phase II of the Franklin IWRP study.

The energy required to deliver reclaimed water to customers is related to the pumping of wastewater effluent from the treatment plant into the distribution system. The following equation was used to estimate the pumping energy required[9]:

$$E = e^{12+\ln(1.25Q)}$$

Equation 5

E: energy required
Q: flow to reclaimed water distribution

4.6.3 Reclaimed Water Demand Projections

Demand projections for reclaimed water were calculated based on values in the 2009 *Reclaimed Water System Master Plan*. This document lists various potential users and potential demands which were aggregated into the water use sectors (residential, commercial, and recreational) and projected out over the planning period. The escalation of reclaimed water demand is related to the time at which the City anticipates infrastructure could be built to supply those customers. **Table 4-19** lists the projected average reclaimed water demands by projection year and water use sector. The seasonal variation in reclaimed water demands is large, considering that most reclaimed water usage is for irrigation. The monthly variations were developed using records of wastewater effluent sent to reuse and discussions with stakeholders and the City. **Table 4-20** shows the monthly multipliers used for reclaimed water demand for the water use sectors. **Figures 4-17** through **4-19** show the calculated reclaimed water demands used in the model.

Table 4-19
Projected Average Reclaimed Water Demands

Projection Year	Residential (mgd)	Commercial (mgd)	Recreational (mgd)	Total (mgd)
2015	0.49	0.72	1.89	3.10
2020	0.77	0.89	2.00	3.65
2025	1.15	1.18	2.08	4.40
2030	1.93	1.96	2.08	5.97
2035	2.71	2.74	2.08	7.53
2040	3.49	3.52	2.08	9.09

[9] Carlson, Steven W. and Adam Walburger, 2007. Energy Index Development for Benchmarking Water and Wastewater Utilities. AwwaRF

Table 4-20
Projected Monthly Multipliers for Reclaimed Water Demands

Month	Residential and Commercial	Recreational
January	0.22	0.03
February	0.19	0.01
March	0.39	0.01
April	0.70	0.10
May	0.85	0.15
June	2.07	1.03
July	1.69	2.23
August	2.00	3.32
September	1.73	2.18
October	1.24	1.38
November	0.71	1.11
December	0.22	0.46

Residential Reclaimed						
Demand in MGD	Demand Year					
	2015	2020	2025	2030	2035	2040
January	0.11	0.17	0.25	0.42	0.59	0.76
February	0.10	0.15	0.22	0.37	0.53	0.68
March	0.19	0.30	0.44	0.74	1.05	1.35
April	0.34	0.54	0.80	1.35	1.89	2.44
May	0.42	0.65	0.97	1.64	2.30	2.97
June	1.02	1.59	2.37	3.99	5.61	7.23
July	0.83	1.30	1.94	3.25	4.57	5.89
August	0.99	1.54	2.29	3.85	5.41	6.98
September	0.85	1.33	1.98	3.33	4.67	6.02
October	0.61	0.95	1.42	2.38	3.35	4.32
November	0.35	0.55	0.82	1.37	1.93	2.48
December	0.11	0.17	0.25	0.42	0.59	0.76
Average	0.49	0.77	1.15	1.93	2.71	3.49

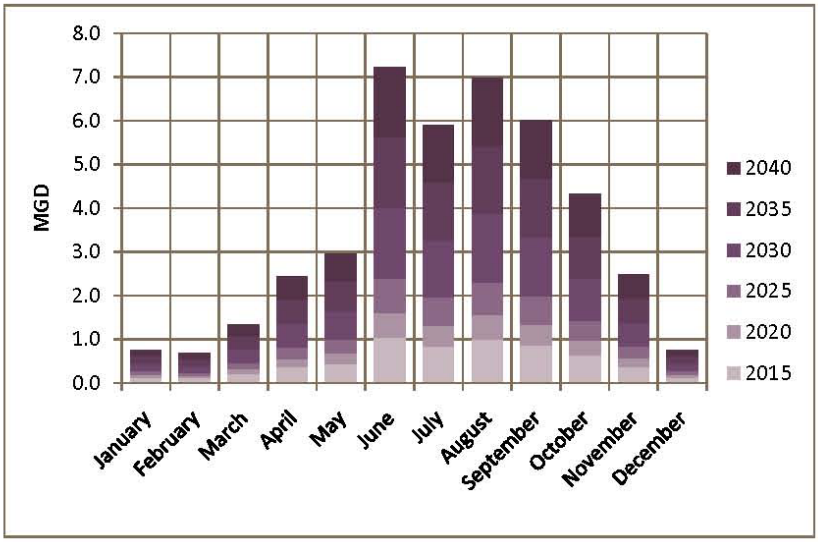


Figure 4-17
Modeled Residential Reclaimed Water Demands 2015 to 2040

Commercial Reclaimed						
Demand in MGD	Demand Year					
	2015	2020	2025	2030	2035	2040
January	0.16	0.19	0.26	0.43	0.60	0.77
February	0.14	0.17	0.23	0.38	0.53	0.68
March	0.28	0.34	0.45	0.76	1.06	1.36
April	0.50	0.62	0.82	1.37	1.91	2.46
May	0.61	0.75	1.00	1.66	2.33	2.99
June	1.49	1.84	2.44	4.05	5.67	7.29
July	1.21	1.50	1.99	3.31	4.62	5.95
August	1.44	1.77	2.35	3.91	5.47	7.04
September	1.24	1.53	2.03	3.38	4.73	6.08
October	0.89	1.10	1.45	2.42	3.39	4.35
November	0.51	0.63	0.84	1.39	1.95	2.51
December	0.16	0.19	0.26	0.43	0.60	0.77
Average	0.72	0.89	1.18	1.96	2.74	3.52

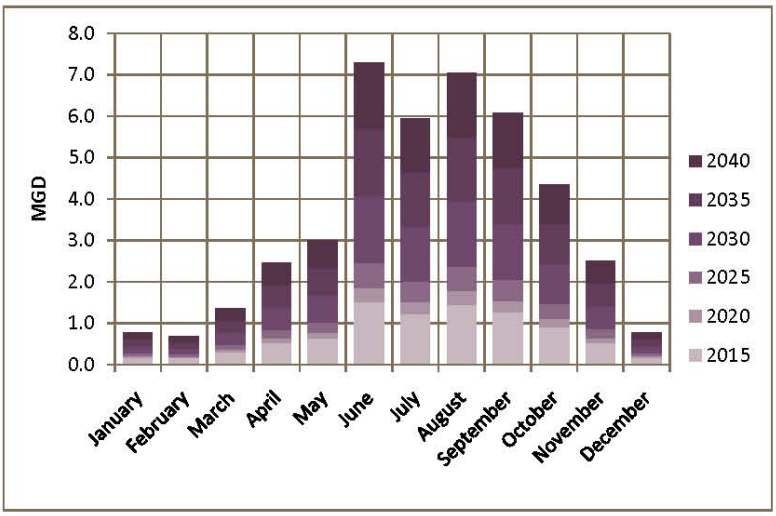


Figure 4-18
Modeled Commercial Reclaimed Water Demands 2015 to 2040

Recreational Reclaimed			
Demand in MGD	Demand Year		
	2015	2020	2025 - 2040
January	0.05	0.05	0.05
February	0.02	0.02	0.02
March	0.01	0.01	0.01
April	0.18	0.19	0.20
May	0.28	0.29	0.30
June	1.95	2.06	2.15
July	4.22	4.45	4.64
August	6.28	6.63	6.91
September	4.13	4.36	4.54
October	2.61	2.75	2.87
November	2.10	2.21	2.31
December	0.87	0.92	0.96
Average	1.89	2.00	2.08

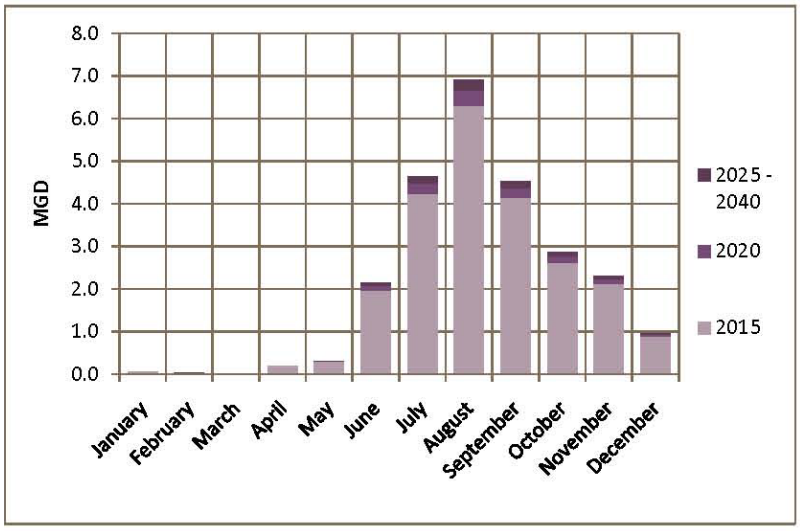


Figure 4-19
Modeled Recreational Reclaimed Water Demands 2015 to 2040

4.6.4 Streamflow Augmentation

In addition to localized stormwater collection and reclaimed wastewater effluent reuse for irrigation, the IWRP includes an option to use Robinson Lake to augment low flows in the Harpeth River. Robinson Lake is located upstream of the Franklin WTP and currently has an unmanaged rock weir structure controlling flow discharge to the river. The project option involves constructing an outlet structure to release flow from the lake when the river flow is extremely low. The specifications of the model representation of this scenario are listed in **Table 4-21**.

Table 4-21
Robinson Lake Streamflow Augmentation Specifications and Assumptions

Parameter	Model Specification
Lake Area	11 acres
Lake Drainage Area	100 acres
Usable Storage Volume	1 foot over lake area, total 11 ac-ft
River Low Flow Trigger for Robinson Lake Release	Median September flow: 3.78 mgd (5.85 cfs)
Release Rate	Variable with volume, up to 1 mgd

If the drainage area to Robinson Lake was entirely impervious—1.3 inches of rain would fill the 11 acre-feet of usable storage. Considering pervious surfaces, it is assumed that 3.0 inches of rain would be needed to fill the usable storage. Under this assumption, a constant multiplier was calculated to estimate the volume of storage replenishment from rainfall in the 100-acre drainage area and used, along with the historical rainfall time series, to calculate the inflow to the lake from runoff. The release of water from Robinson Lake is triggered when the streamflow in the river falls below the historical September median flow and is released at a rate that varies with lake volume.

4.7 Modeled Option Costs

Preliminary life-cycle costs for each option included in the Franklin IWRP Phase 1 analysis were developed through engineering estimates, using available existing plans, and discussions with the City. The costs are appropriate as planning-level estimates that can be used to compare options and alternatives to help the City see the general tradeoffs between performance and cost. **Table 4-22** lists each option, the estimated capital and annual operating costs, and the source of the estimate. Many costs depend on variables such as volume pumped or flow through a treatment plant, so it is not possible to report a single value for annual operating cost as the value changes with different alternatives.

Category	Options	Capital Cost	Annual or Operating Cost	Source
Water Supply	Upgrade existing 2.1 MGD WTP and purchase remaining water from HVUD	$4,841,500	$2.55 per 1000 gallons purchase, $1.72 per 1000 gallons produced	AECOM - Design Report for Franklin WWTP
	Expand existing WTP to 4.0 MGD, upgrade WTP intake structure and purchase remaining water from HVUD	$6,739,000	$2.55 per 1000 gallon	
	Shut down existing WTP and purchase all water from HVUD	$1,293,350		
	Construct raw water transmission line from the Cumberland River and upgrade water treatment plant to supply all City demand	$67,500,000		Consoer, Townsend & Associates - Franklin Water Facilities and Supply Report
Conservation	Indoor and outdoor conservation (public education, etc)		$20,000 per year	Five Mile Creek Watershed Management Plan
	Low flow incentives		$50,000 per year, for 6 years	
Water Distribution	Remove outdated tanks	$2 per gallon capacity		Engineering estimate
	Address water loss	$350,000	$1.0 to 1.2 million per year	City of Franklin
	Install advanced metering	$3,000,000		
Wastewater Treatment	Construct new WWTP at Goose Creek	$150,000,000	$0.86 per 1000 gallon	Engineering estimate and City of Franklin
	Collect and treat wastewater from adjacent communities or other small systems (e.g., Lynwood, Cartwright Creek)		$0.86 per 1000 gallon	City of Franklin
	Upgrade and rerate existing WWTP	$2.50 per gallon capacity		CDM - Prelim Design Report for Kingsport WWTP Improvements
Collection System	Address inflow and infiltration	$1,350,000		City of Franklin
	Hook up septic users to sewer	$5040 per hook-up		Five Mile Creek Watershed Management Plan
Biosolids	Upgrade biosolids facilities for biogas to energy	$23,950,000	$563,000 per year	CDM - Hallsdale-Powell Prelim Engineering Report
	Upgrade solids handling facilities to produce Class A solids	$21,430,000	$739,000 per year	
	Upgrade solids handling facilities to drying/ERS (ash disposal)	$20,190,000	$845,000 per year	
	Upgrade solids handling facilities to produce higher TS content sludge	$18,760,000	$824,000 per year	
	Class A biosolids to Franklin's composting facility	$21,430,000	$739,000 per year	
	Solids trucked to Metro Nashville for disposal/processing		$37.20 per ton	City of Franklin
	Solids disposal at BFI (108 miles/trip)		$39.00 per ton	
	Land application (Switch grass production)	$55,000		John Buchanan (UT) - Dispersal System Study
Reclaimed Water	Complete the 12" Long Lane line and retrofit the existing 500,000 gallon Long Lane water reservoir for reclaimed water service	$410,000		Smith Seckman Reid - Reclaimed Water System Master Plan
	Complete the distribution loop around the city by constructing the 12" Columbia Avenue/Southeast Parkway reclaimed line and construct a 500,000 gallon storage tank in the vicinity of Winstead Hill	$2,320,000		
	Identify and establish dedicated reclaimed water sites	$2,500,000		
	Increased storage	$14,800,000		
	Convert the Franklin Green/Horton Lane sanitary force main for reclaimed water distribution	$85,000		
	Install additional pumps to increase the station capacity to approximately 12 million gallons per day	$1000 per million gallons capacity		
	Establish additional reclaimed water storage facilities/ convert existing water storage tanks to reclaimed storage tanks	$2 per gallon capacity		
Stormwater	Rain gardens	$25,000/ unit	$25,000 per year	Five Mile Creek Watershed Management Plan
	Constructed wetlands	$625,000 per acre		CDM - Dry Branch Bid Tabulation
	Conveyance upgrades	$4,480,000		CDM - Stormwater Master Plan
	Residential rain barrels	$25 (rain barrel) per cubic foot water savings		Center for Watershed Protection - Urban Stormwater Retrofit Practices
	Pervious pavement	$24,500 per acre		
	Use of Robinson Lake to provide enhanced base flow in the Harpeth River during dry periods	$1,000,000		Engineering estimate
Water Quality & Ecological Health	Treat discharged effluent to higher standard during summer months	$26,250,000	$1,305 per million gallons	UNEP Sourcebook
	Removal of low head dam at the water treatment plant intake	funded		U.S. Fish and Wildlife
	Widespread stream and bank restoration	$66 per linear foot		Five Mile Creek Watershed Management Plan

Table 4-22
Estimated Costs for Project Options

Section 5
IWRP Phase I Results

The IWRP is based upon a holistic evaluation of Franklin's water resources. This concept is demonstrated in Figure 4-1**.** The process diagram demonstrates the water cycle for Franklin's water resources and utilities and how a single change may affect multiple systems. A combination of model results and qualitative scores for each performance measure resulted in a weighted composite score for each of the five alternatives initially developed by the stakeholders. These results allowed the stakeholders to understand the tradeoffs and drivers of the scores and combine options into hybrid alternatives aimed at satisfying many objectives broadly and being recommended for further analysis in Phase II.

5.1 Initial Alternative Scores

Results of the model are provided in the form of performance measure scores, which can be converted from the respective units related to each performance measure into a standardized scale. Section 3.6 includes a description of the process of converting raw scores (e.g., costs, frequencies, pollutant loads) into standardized scores, weighting the scores with the stakeholder developed weights, and aggregating the scores into a single composite score for each alternative. Once the scores for each performance measure were converted to a standardized scale, they were added into a composite score for each alternative developed by the stakeholders aimed at meeting the five most heavily weighted objectives:

- Reliability
- Efficiency
- Water Quality (and Ecological Health)
- Service at a Reasonable Cost (Cost)
- Safety and Security

Table 5-1 lists the performance measure scores for each alternative, **Figure 5-1** shows the standardized and weighted scores for the individual nine objectives for each of the alternatives, and **Figure 5-2** shows the comprehensive score as a stacked bar chart, with each color representing an objective. In addition to the five alternatives, a do-nothing alternative is shown on the graph. The raw scores are listed with their respective units. Qualitative scores were agreed upon by the steering committee for each alternative, and were assigned based on a relative scale of 0 to 5 that generally corresponded to a range of worse than current conditions, no change, and better than current conditions. The scores in the graphs are standardized (see Section 3.6), so a higher composite score for the cost objective means that the cost is more preferable, based on the performance measures for that objective, not higher.

The efficiency alternative was the overall best scoring alternative, and the composite scores demonstrates that doing nothing is not an effective plan for Franklin. Closer review of results reveals that while safety and security and low cost have similar scores overall, these composite scores differ in their components. Low cost does not

score as well under the performance measures of safety and security (5) and sustainable biosolids handling (7) objectives; and safety and security does not score as well in the cost (4) objective. Generally, the five alternatives scored well with respect to the objectives they were targeting (i.e., the reliability alternative scored the best in the reliability performance measures).

The composite scores shown in **Figure 5-2** are not intended to rank the initial alternatives for inclusion in the final IWRP. Rather, they serve to help the stakeholders understand the tradeoffs involved with selecting different sets of options. The safety and security alternative and the efficiency alternative both scored well, overall, but for different reasons. A potential hybrid alternative would be to combine the projects that resulted in high scores in those two alternatives. Another example is the water quality alternative,aimed exclusively at improving water quality, which does not score as well in the efficiency objective. This is likely due to the selections of project options included in the alternative, which do not include building reclaimed water distribution infrastructure, addressing inflow and infiltration, or conservation. Augmenting the water quality alternative with these types of projects would likely result in a hybrid that scores better than the original.

5.2 Hybrid Alternative Development

During Workshop 4 the stakeholders reviewed the results from the initial five alternatives and discussed possibilities for improving the alternatives into hybrids aimed at meeting multiple objectives. The stakeholders developed four hybrid alternatives and agreed to recommend that each of them be studied further in Phase II of the IWRP process. The recommended hybrid alternatives are:

- **Efficiency + Safety and Security –** Through the analysis and discussion of the separate alternatives (efficiency and safety and security), a combination of the options in these two alternatives was selected with the intent of maximizing the performance of the resulting hybrid alternative.

- **Water Quality –** The evaluated water quality alternative was improved by selecting projects in the distribution system, water conservation, and reclaimed water sectors, since it had the second lowest efficiency score of the five alternatives.

- **Low Cost –** The low cost alternative was modified in its wastewater treatment plant option, switching from building a new WWTP at Goose Creek to upgrading and rerating the existing plant.

- **Reliability –** The reliability alternative was modified to include water conservation projects, since its initial score for efficiency was the lowest of the five alternatives.

Table 5-2 shows the four recommended alternatives and the project options that were selected for each. Discussion of the recommended alternative scores follows in Section 6.

Table 5-1
Performance Measure Scores for Initial Alternatives

Objectives		Weight	Performance Measures	Units	Water Quality	Low Cost	Efficiency	Reliability	Safety & Security	Do Nothing
1	Reliability	31.1	% time all demands met	% time (all days)	27.7	33.2	56.1	57.9	24.7	24.7
			Avg magnitude of deficits (all uses)	MG	8.06	6.87	8.82	9.36	7.84	7.84
			Vol of WW capacity surplus or shortfall	mgd	4.19	5.83	3.56	2	3.56	0.29
			Supply redundancy	% volume	0	19.9	36.1	44	19.9	19.3
2	Efficiency	15.5	Volume of stormwater put to beneficial use	MGD (all days)	0.1	0	0.50	0.50	0	0.00
			% total reuse demand satisfied	% volume	38.2	52.4	60.7	60.1	37.2	37.3
			% demand reduction	% volume	0	5	5	0	5	0
			Reduction in inflow and infiltration	qualitative	5	4	5	2	5	2
			% reduction in unaccounted for water	% volume	0	50	50	0	50	0
3	Water Quality & Ecological Restoration	13.5	Frequency of low flow < September median	% time (all days)	7.37	9.11	0.81	0.92	0.81	9.11
			Average summer BOD load	LB/day (summer only)	960	1,030	1,020	1,030	1,100	1,130
			Average summer nitrogen load	LB/day (summer only)	240	250	280	280	390	380
			Ecological indicators	qualitative	4.5	3	4.5	3.5	3	3
			Negative impacts of stormwater reduced	qualitative	3.5	3	3	3	3.5	3
4	Service at a Reasonable Cost	13.2	Life-cycle cost of projects and policies	million $	566	405	605	759	677	360
			Combined change in water and sewer rates	qualitative	2.5	2.3	2	1.8	1.5	3
			Meet secondary drinking water standards	qualitative	2.5	3.5	5	4	3	3.5
5	Safety & Security	8.3	% of total wastewater on septic	% volume	0	4	0	0	0	4
			Change in 100 year flood elevation	qualitative	4	3	3	3	5	3
			Vulnerability of infrastructure & facilities	qualitative	1.5	4	4.5	4	4	1.5
			Emerging water quality concerns	qualitative	4	3.5	5	4	3.5	4
6	Achieve Regional Acceptance	5.7	Extent of regional focus	qualitative	4.5	3	4	3	3	3
			Likelihood of public acceptance	qualitative	3	4	3.5	2.5	3	1
7	Sustainable Biosolids Mgmt	4.7	Biosolids handled sustainably	qualitative	1	4	4.5	2	5	1
8	Improved Access & Aesthetics	4.5	% of streamflow that is WWTP effluent	% volume (Sept. only)	36	5	22	22	36	35
			Extent of bank stabilization	qualitative	5	1	5	1	5	1
			Erosion potential	qualitative	4.5	3	3.5	3	4	3
			Public accessibility	qualitative	3	3	3	3	2	3
9	Carbon Footprint	3.5	Average energy requirements	average kWh/day	95,800	35,200	30,500	134,900	57,600	72,600

Raw scores are planning-level estimates based on existing information and used only for initial comparison – they are subject to revision with more detailed evaluation in Phase II.

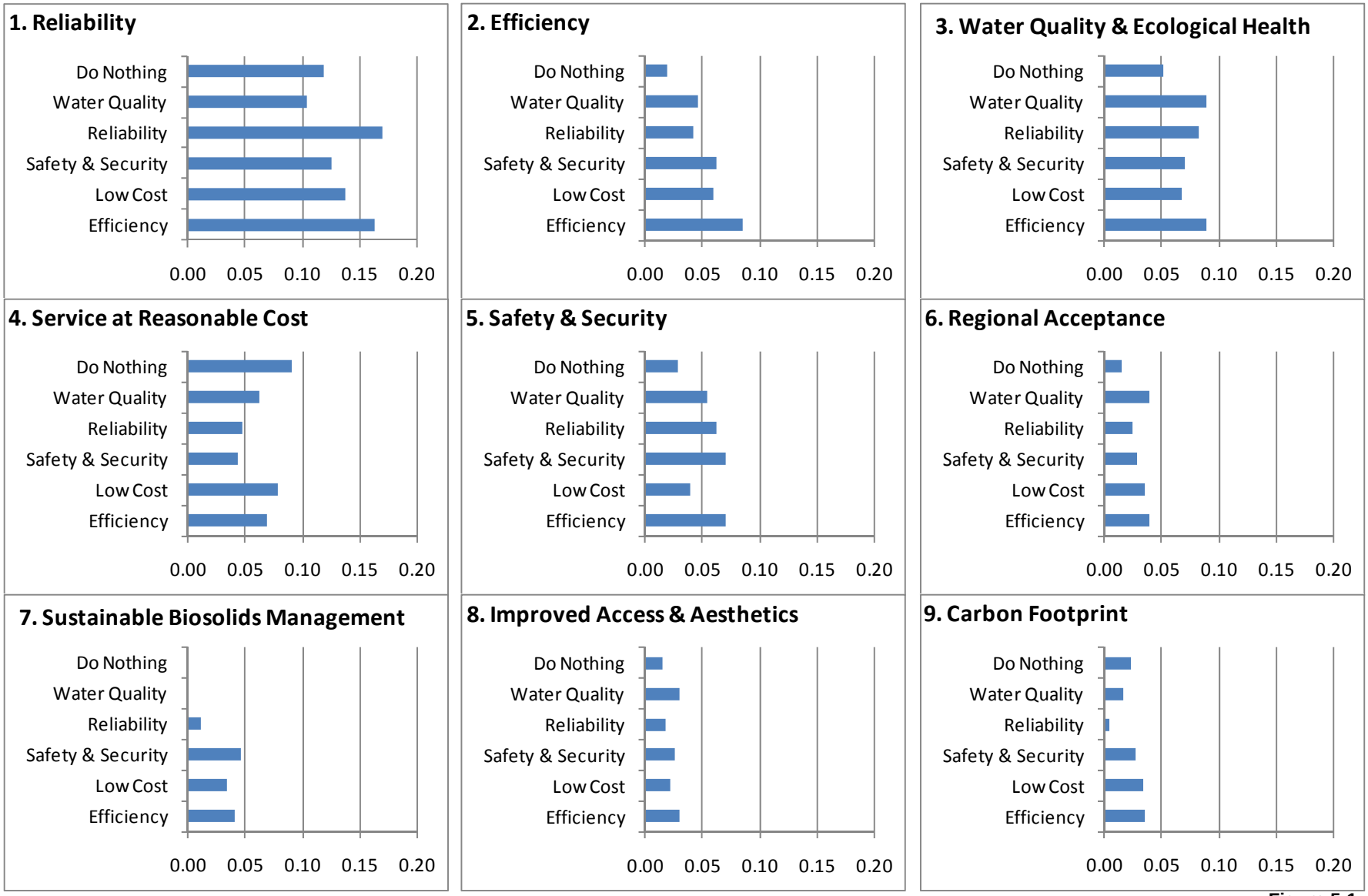


Figure 5-1
Weighted Objective Scores for the Initial Alternatives

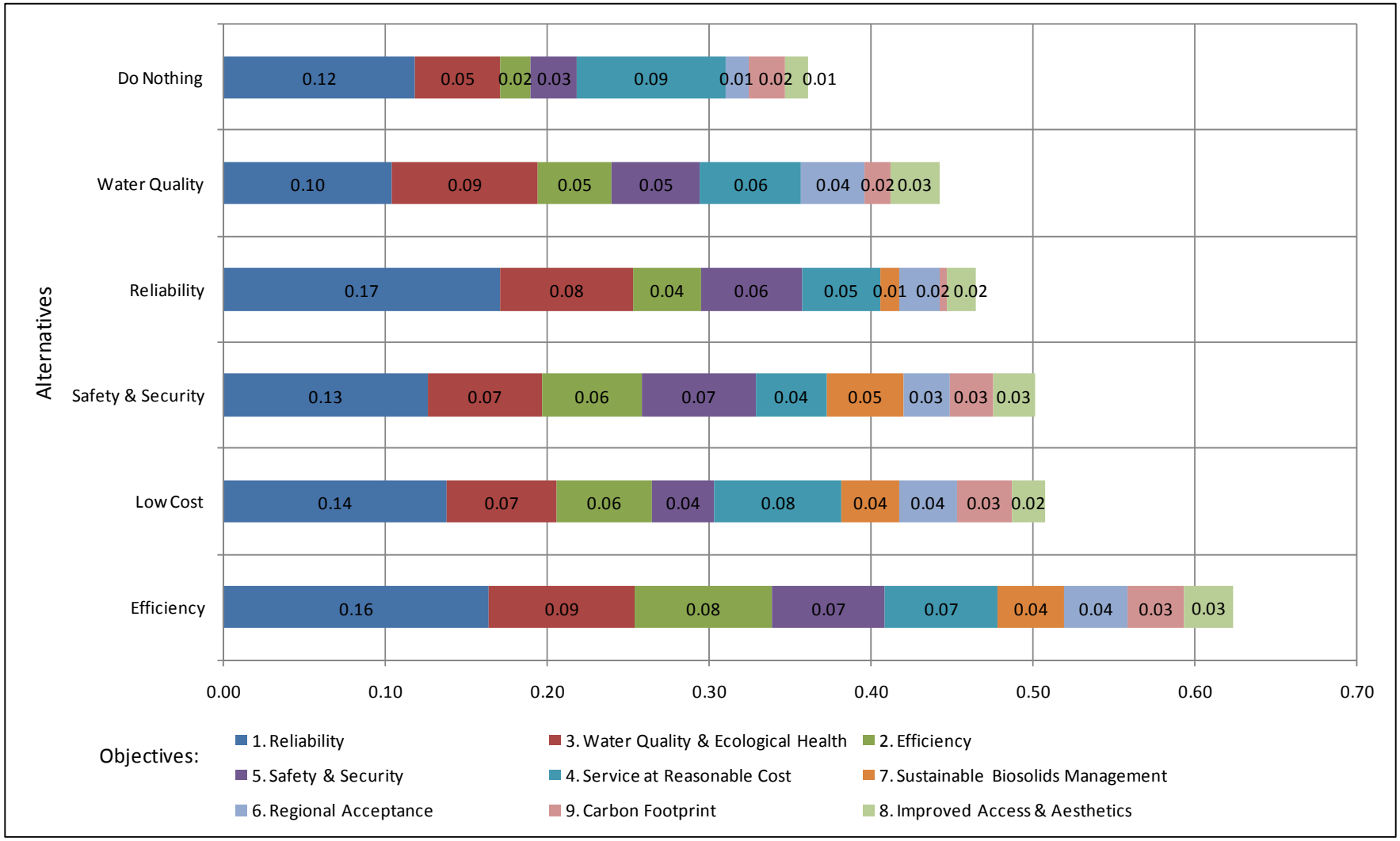


Figure 5-2
Composite Scores for the Initial Alternatives

Category	Options	Efficiency + Safety & Security	Water Quality Plus	Revised Low Cost	Revised Reliability
Stormwater Options	Residential rain barrels	X	X		X
	Commercial stormwater reuse	X	X		X
	Recreational stormwater reuse	X	X		X
	Rain gardens	X	X		
	Pervious pavement	X	X		
	Constructed wetlands	X	X		
	Conveyance upgrades	X	X		
	Increased storage	X	X		
Water Treatment Plant	Upgrade existing 2.1 mgd WTP and purchase remaining water from HVUD			X	
	Expand existing WTP to 4.0 mgd, upgrade WTP intake structure and purchase remaining water from HVUD	X			
	Repair water reservoir (ongoing)	X		X	
	Shut down existing WTP and purchase all water from HVUD		X		
	Construct raw water transmission line from the Cumberland River and upgrade water treatment plant to supply all City demand				X
Distribution System	Address water loss	X	X	X	
	Install advanced metering	X	X	X	X
	Remove outdated tanks	X	X		
	System management practices	X	X	X	X
Conservation Options	Indoor and outdoor conservation (public education, etc)	X	X	X	X
	Conservation ordinances	X	X	X	X
	Low flow incentives	X	X	X	X
	Rate block structure, etc	X	X	X	X
Wastewater Treatment Plant	Upgrade and rerate existing WWTP		X	X	X
	Construct new WWTP at Goose Creek	X			X
	Collect and treat wastewater from adjacent communities or other small systems (e.g., Lynwood, Cartwright Creek)	X	X		
	Treat discharged effluent to higher standard during summer months		X		
Collection System	Address inflow and infiltration	X	X	X	
	Hook up septic users to sewer	X	X		X
	System management practices	X	X		
Ecological Restoration Options	Removal of low head dam at the water treatment plant intake	X	X	X	
	Address old dump site (from downtown to Liberty Creek) and convert to Harpeth River access area				
	Use of Robinson Lake to provide enhanced base flow in the Harpeth River during dry periods	X	X		
	Cattle exclusion	X	X		
	Widespread stream and bank restoration	X	X		
Reclaimed Water Options	Complete the 12" Long Lane line and retrofit the existing 500,000 gallon Long Lane water reservoir for reclaimed water service	X	X		
	Complete the distribution loop around the city by constructing the 12" Columbia Avenue/Southeast Parkway reclaimed line and construct a 500,000 gallon storage tank in the vicinity of Winstead Hill	X	X		X
	Convert the Franklin Green/Horton Lane sanitary force main for reclaimed water distribution	X	X	X	
	Increase City-wide reuse by increasing customer base	X	X	X	
	Install additional pumps to increase the station capacity to approximately 12 million gallons per day	X	X		X
	Establish additional reclaimed water storage facilities/ convert existing water storage tanks to reclaimed storage tanks	X	X		X
	Identify and establish dedicated reclaimed water sites	X	X	X	
	System management practices	X	X	X	
Biosolids Options	Upgrade solids handling facilities to produce Class A solids				X
	Upgrade solids handling facilities to drying/ERS (ash disposal)				X
	Upgrade solids handling facilities to produce higher TS content sludge				
	Solids disposal at BFI (108 miles/trip)				
	Solids trucked to Metro Nashville for disposal/processing			X	
	Class A biosolids to Franklin's composting facility		X		
	Land application (Switch grass production)	X			
	Upgrade biosolids facilities for biogas to energy	X	X	X	

Table 5-2
Recommended Alternatives

Section 6
Conclusions and Recommendations

Phase I of the IWRP project concludes with recommendations of alternatives to carry into Phase II. The recommendations are made by the steering committee through the input and support of the stakeholder advisory group.

6.1 Stakeholder Recommended Alternatives

The scorecard results of the alternatives analysis were discussed with stakeholders during Workshop 4. Based on the performance of individual options, preliminary results of hybrid alternatives, and interactive analysis of hybrid alternatives, the stakeholders selected four alternatives which they recommended to carry forward. These four hybrid alternatives are defined in Table 5-2, and may be compared with Table 3-3 containing the initial alternatives.

Table 6-1 is the performance measure scorecard of the recommended hybrid alternatives, and **Figure 6-1** shows the standardized scores for each objective and hybrid alternative (as well as the do-nothing alternative). The following is a summary of the four alternatives recommended to be studied further in Phase II. Each of the four recommended alternatives has a different water supply option, and a focus of Phase II will be to study those options in greater detail to refine assumptions and make more educated comparisons. **Table 6-2** lists the key features of each recommended alternative.

- **Efficiency + Safety and Security –** This alternative includes all stormwater, distribution system, collection system, conservation, reclaimed water, and ecological restoration options (except addressing the dump site, which is not included in any alternative due to lack of information defining the project, namely cost, at this time). Water supply options include repairing the raw water reservoir, upgrading the existing WTP to 4 mgd, and purchasing the remaining water needed to meet demands from HVUD. Wastewater will be accepted from other small communities and a new WWTP will be constructed at Goose Creek. Biosolids handling will be improved by upgrading the facility for biogas to energy and processing biosolids for land application.

- **Water Quality –** This alternative includes all stormwater, distribution system, collection system, conservation, reclaimed water, and ecological restoration options. All of the water supply for the City will be purchased from HVUD. No water will be withdrawn from the Harpeth River. The existing WWTP will be upgraded to sufficient capacity to treat Franklin's and neighboring small communities' wastewater and to treat effluent in the summer months to a higher

standard. Biosolids handling includes upgrading to produce Class A solids, biogas to energy, and sending Class A solids to Franklin's composting facility.

- **Low Cost –** This alternative includes all conservation options. No stormwater options are included. Select, low cost options for the distribution and collection systems are included. Only the removal of the low-head dam (which is not considered to be funded by the City) is included from ecological restoration options. Reclaimed water options that do not involve building new lines or converting tanks are included. The water supply option is to update the existing 2.1 mgd plant, repair the reservoir, and purchase the remaining water needed from HVUD. The wastewater option is to upgrade and rerate the existing plant only. Biosolids handling will include upgrading to biogas for energy (to recover energy costs) and transport the solids to Metro Nashville for disposal and processing.

- **Reliability –** This alternative includes all conservation options, stormwater options that focus on reuse, collection and distribution system options that aim to provide customers with a more reliable service, and no ecological restoration options. The water supply option is to build a new transmission line from the Cumberland River and upgrade the Franklin WTP to treat all of the City's water. The existing WWTP will be upgraded and rerated and a new plant will be constructed at Goose Creek. Several, though not all, reclaimed water options are included. Biosolids handling will be improved by upgrading both to produce Class A solids and ash disposal.

Table 6-1
Performance Measure Scores for Recommended Hybrid Alternatives

Objectives		Weight	Performance Measures	Units	Efficiency + Safety & Security	Water Quality Plus	Revised Low Cost	Revised Reliability	Do Nothing
1	Reliability	31.1	% time all demands met	% time (all days)	56.1	58.1	33.2	53	24.7
			Avg magnitude of deficits (all uses)	MG	8.82	7.27	6.87	9.23	7.84
			Vol of WW capacity surplus or shortfall	MGD	3.56	4.56	5.83	7.56	0.29
			Supply redundancy	% volume	36.1	0	20.3	46.3	19.3
2	Efficiency	15.5	Volume of stormwater put to beneficial use	MGD (all days)	0.50	0.50	0.00	0.50	0.00
			% total reuse demand satisfied	% volume	60.7	70.3	52.4	55.3	37.3
			% demand reduction	% volume	5	5	5	5	0
			Reduction in inflow and infiltration	qualitative	5	5	4	4	2
			% reduction in unaccounted for water	% volume	50	50	50	0	0
3	Water Quality & Ecological Restoration	13.5	Frequency of low flow < September median	% time (all days)	0.81	7.37	9.11	0.92	9.11
			Average summer BOD load	LB/day (summer only)	960	910	1,030	1,020	1,130
			Average summer nitrogen load	LB/day (summer only)	260	170	250	270	380
			Ecological indicators	qualitative	4.5	4.5	4	3.5	3
			Negative impacts of stormwater reduced	qualitative	3.5	3.5	3	3	3
4	Service at a Reasonable Cost	13.2	Life-cycle cost of projects and policies	million $	631	645	404	804	360
			Combined change in water and sewer rates	qualitative	2	2.5	2.2	1.8	3
			Meet secondary drinking water standards	qualitative	5	2.5	3.5	4	3.5
5	Safety & Security	8.3	% of total wastewater on septic	% volume	0	0	4	0	4
			Change in 100 year flood elevation	qualitative	5	4	3	3	3
			Vulnerability of infrastructure & facilities	qualitative	4	1.5	4	4	1.5
			Emerging water quality concerns	qualitative	3	4	3.5	4	4
6	Achieve Regional Acceptance	5.7	Extent of regional focus	qualitative	3.5	4.5	3	3	3
			Likelihood of public acceptance	qualitative	2	3	4	2.5	1
7	Sustainable Biosolids Mgmt	4.7	Biosolids handled sustainably	qualitative	5	5	4	2	1
8	Improved Access & Aesthetics	4.5	% of streamflow that is WWTP effluent	% volume (Sept. only)	22.1	0.0	4.7	21.5	35
			Extent of bank stabilization	qualitative	5	5	1	1	1
			Erosion potential	qualitative	4	4.5	3	3	3
			Public accessibility	qualitative	3	3	3	3	3
9	Carbon Footprint	3.5	Average energy requirements	average kWh/day	30,500	90,800	35,000	112,100	72,600

Raw scores are planning-level estimates based on existing information and used only for initial comparison – they are subject to revision with more detailed evaluation in Phase II.

Table 6-2
Key Features of Final Four Alternatives

	Key Features				
Recommended Alternatives	**Water Supply**	**Wastewater**	**Stormwater**	**Reclaimed Water**	**Harpeth River**
Efficiency + Safety and Security	Upgrade Franklin WTP to 4 mgd, purchase remaining water from HVUD	Goose Creek plant and WW from other communities	All major projects	All major projects	All ecological restoration options
Water Quality	Shut down Franklin WTP and purchase all water from HVUD	Upgrade existing WWTP, accept WW from other communities, treat effluent to higher standard in summer	All major projects	All major projects	All ecological restoration options
Low Cost	Maintain 2.1 MGD at Franklin WTP, purchase remaining water from HVUD	Upgrade existing WWTP	None	Options that do not include new lines or tanks	Remove low head dam
Reliability	Upgrade Franklin WTP to supply all City demand, transport raw water form Cumberland River	Upgrade existing WWTP and build Goose Creek	Options that focus on reuse	Some major, all minor projects	None

Figure 6-2 shows the aggregated weighted scores for each alternative described above. The efficiency + safety and security hybrid alternative has the highest composite score, though all four recommended alternatives scored comparably in the most heavily weighted objectives. While tradeoffs were observed, especially in regards to cost performance measures, generally alternatives that included more options scored higher based on the objectives and weights developed by the stakeholders. **Figure 6-3** shows a comparison of the composite scores without the cost scores included, versus the estimated cost of each alternative. This comparison, while based on preliminary cost data and estimations that need to be refined, shows that there may be a value of expenditure at which stakeholder-defined benefits level off. Phase II will include further analysis of this type of observation, so that the final IWRP is a logical group of projects that is designed to meet the City's objectives for water resource management, while recognizing constraints such as cost and permitting.

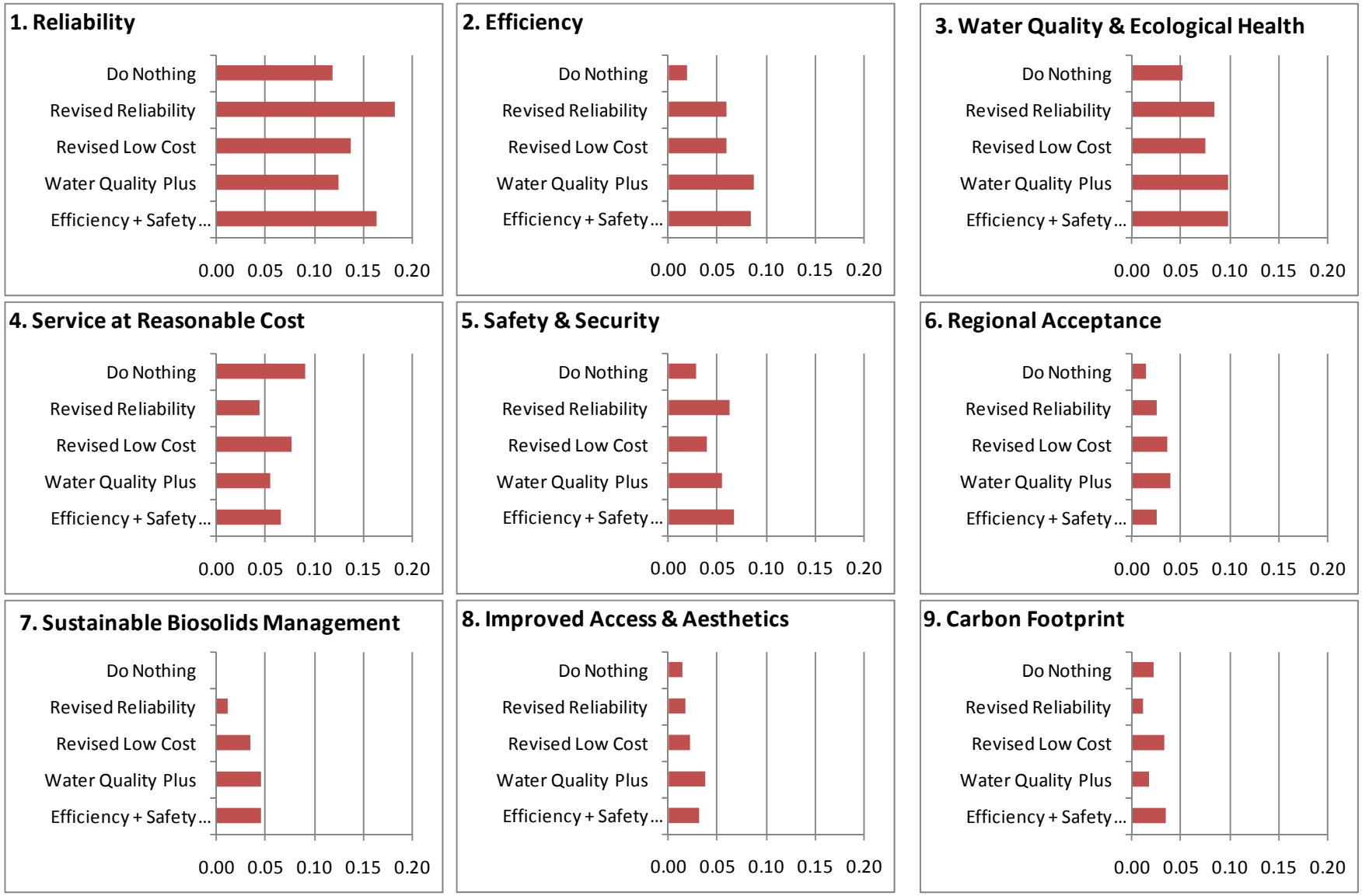


Figure 6-1
Weighted Objective Scores for Recommended Hybrid Alternatives

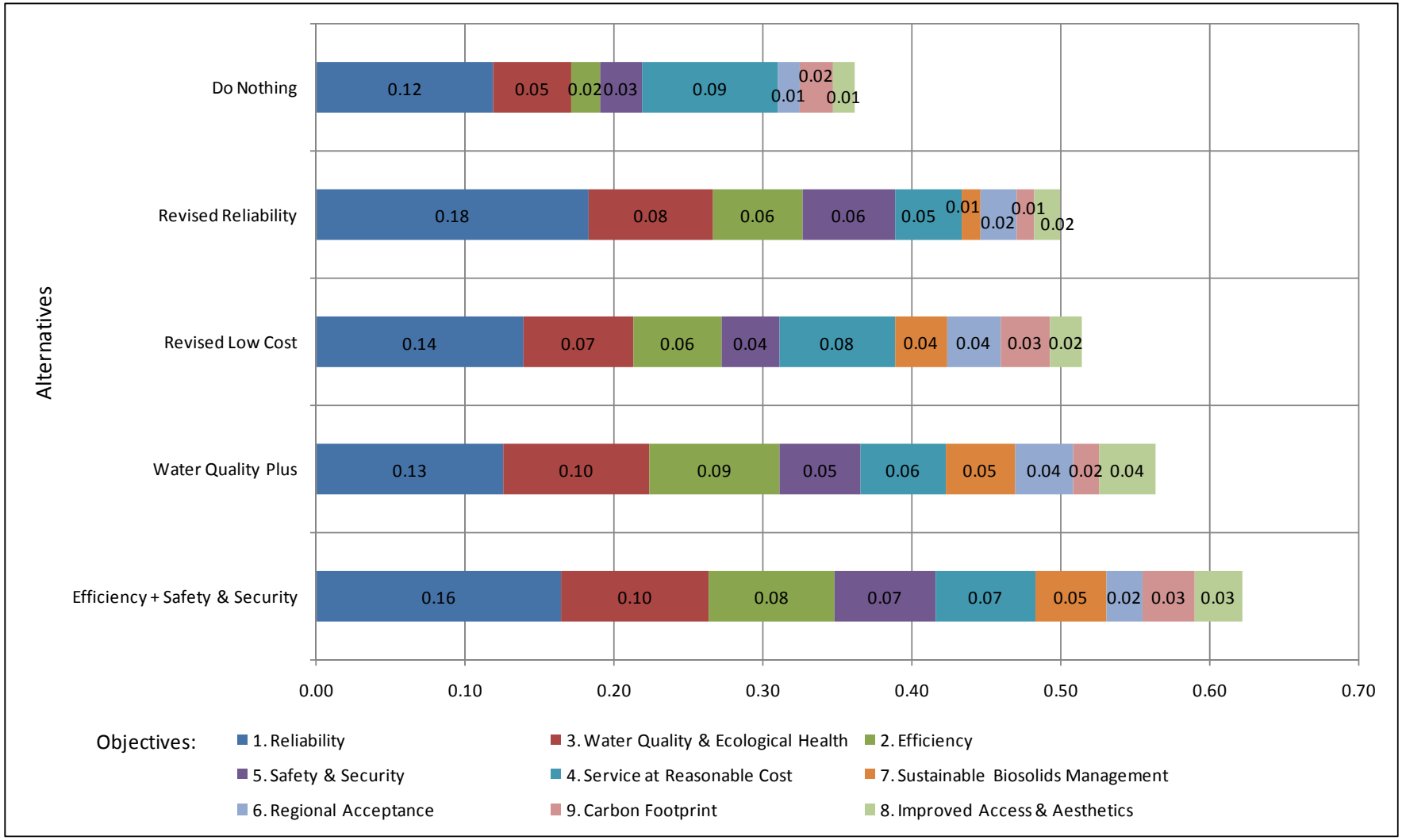


Figure 6-2
Composite Scores for Recommended Hybrid Alternatives

6.2 IWRP Phase II

This report is the culmination of Phase I activities and documents the stakeholder-driven process, as well as development of the integrated system model, technical data and assumptions, and other information used to screen alternatives that are carried into Phase II of the IWRP. The IWRP will be completed during Phase II of this planning process. As a result of the outcomes of Phase I of the IWRP, the Phase II process, at a minimum, will include the following:

- Detailed technical analysis of the recommended alternatives.
- Detailed cost analysis.
- Continued modeling and screening of the plans to compare and rank them.
- Continued interaction with stakeholders.
- Conceptual design as necessary to support cost and performance estimation (siting, sizing, performance, needs, etc.).
- Identification of a single preferred plan (the IWRP) from among the alternatives (a blend of the recommended alternatives.
- Permitting plans for identified projects.
- Financing plan for the implementation of the IWRP.
- Phasing plan for project implementation.

The purpose of Phase I was to convene a stakeholder advisory group and steering committee, formulate a list of objectives that the IWRP would address, conduct a preliminary evaluation of potential projects to be included in the IWRP, and develop recommended groupings of projects based on a framework of weights and performance measures agreed upon by the stakeholders. An integrated system model and decision support methodology were used to assist the stakeholders in understanding how decisions made about one aspect of the water resources system would affect the system as a whole. Phase I concludes with a greater understanding of the City of Franklin's water resources systems, consensus amongst stakeholders on the objectives of the IWRP, and a refined list of alternatives to be studied further in Phase II.

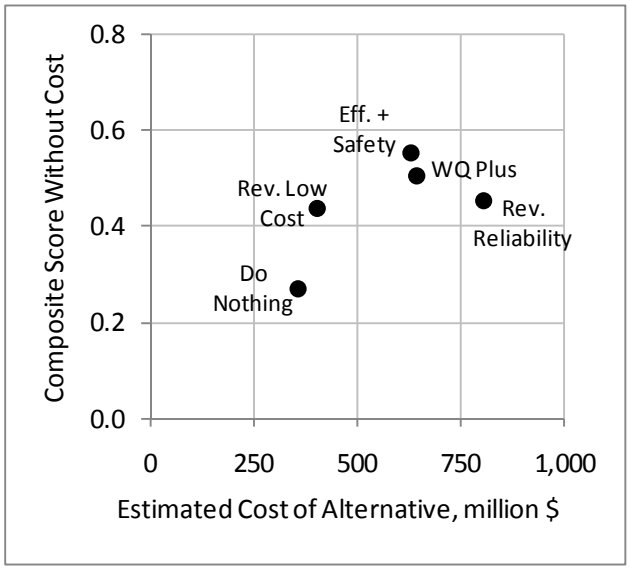


Figure 6-3
Preliminary Comparison of Possible Benefits and Costs of Recommended Alternatives

Section 7
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City of Franklin, Tennessee

Integrated Water Resources Plan

Appendix B:

Potable Water

- Review of CTE/AECOM
  Design Report: Franklin Water Treatment Plant
- City of Franklin Integrated Water Resources Plan (IWRP)
  Water Distribution System Improvements Modeling

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Technical Memorandum

To: *City of Franklin IWRP Team*

From: *CDM*

Date: *August 10, 2011*

Subject: *Review of CTE/AECOM Design Report: Franklin Water Treatment Plant dated July 2006*

Executive Summary

As part of its Integrated Water Resources Plan (IWRP) the City of Franklin (City) has undertaken an evaluation of its water treatment plant (WTP). CDM has conducted reviews of previous work focused on addressing regulatory requirements and facility upgrades at the WTP that will be considered as projection options during development of the IWRP. Review of the previous work identified two pending regulatory drivers that would impact decisions regarding project options for the WTP. The first regulatory driver is the Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) which requires additional treatment for Cryptosporidium removal/treatment to be implemented by October 2012. Implementation of UV disinfection at the WTP is the most straightforward method of meeting this regulatory deadline. In addition, compliance with new Stage 2 Disinfectants and Disinfection Byproducts Rules (Stage 2 D/DBPR) requirements is also required by October 2012. Management and control of DBPs requires a two-pronged approach that includes addressing initial formation of the DBPs at the WTP and managing water age in the distribution system. The City is currently working with its wholesale water provider to evaluation methods for addressing the DBPs in the purchased water.

In addition updating the previous cost estimates for WTP upgrades required to maintain current plant operations, this technical memorandum provides estimates for implementing UV disinfection at the City's WTP which would aid in minimization of formation of DBPs as well as meet the requirements of the LT2ESWTR. Concurrently, CDM is working with City Staff to evaluate distribution system options to reduce water age; results of this analysis will be provided in a separate technical memorandum.

1.0 Introduction

As part of its Integrated Water Resources Plan (IWRP) the City of Franklin (City) has undertaken an evaluation of its water treatment facility. This evaluation includes a CDM review of the July 2006 CTE/AECOM report entitled Design Report for the Franklin Water Treatment Plant (WTP Report) that was previously produced for the City. Review of this report identified two important issues related to regulatory compliance that are discussed within this technical memorandum. Additionally, comments regarding recommendations for individual process improvements at the

water treatment plant and updated construction cost estimates for these improvements are provided herein. Comments in this memorandum have been organized in the order of the CTE/AECOM report, with regulatory compliance being discussed first, followed by process recommendations and an updated cost estimate for each process alternative.

In addition to review of the WTP Report, CDM reviewed the July 2009 draft Drinking Water Quality Evaluation and Recommendations (draft WQ Evaluation) that were developed by Metcalf & Eddy/AECOM in conjunction with Hazen and Sawyer. The purpose of this report was to identify the most appropriate methods to reduce the potential for water quality issues in the City's distribution system and to identify appropriate system modifications to reduce the risk of non-compliance with future regulations. Specifically, the new disinfection by-product (DBP) regulations were considered; and, data that was not yet available at the time of the development of the WTP Report was also analyzed. CDM has also provided comments on this draft WQ Evaluation document.

2.0 Regulatory Compliance

Sections I and II of the WTP Report describe the Stage 2 Disinfectants and Disinfection Byproducts Rule (Stage 2 D/DBPR) and the Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR). Potential compliance issues for Franklin Water Treatment Plant (WTP) are not discussed in detail. This may be attributed to the fact that the study was conducted in 2006, before some details related to compliance with this rule were fully defined: specifically, the LT2ESWTR Bin classification for *Cryptosporidium* treatment requirements, as described in the following Sections.

2.1 Stage 2 D/DPBR Compliance

Many water systems treat their water with a chemical disinfectant to inactivate pathogens that cause disease. The public health benefits of chemical disinfection are significant and well-recognized; however, disinfection poses risks of its own. While disinfectants are effective at controlling harmful microorganisms, they react with organic and inorganic matter in the water and form DBPs, some of which pose health risks when present above certain levels. Since the discovery of chlorination byproducts in drinking water in 1974, numerous toxicological studies have been conducted that show some DBPs to be carcinogenic and/or cause reproductive or developmental effects in laboratory animals. As a result, the US Environmental Protection Agency (EPA) promulgated a set of rules to set maximum contaminant limits for DBPs that are protective of human health.

The most recent changes to the Disinfectants and Disinfection Byproducts Rules are the Stage 2 D/DBPR. This rule, promulgated in January 2006, required systems to first conduct an Initial Distribution System Evaluation (IDSE) to identify compliance monitoring sites for the Disinfection Byproduct Maximum Contaminant Levels (DBP MCLs). The IDSE sample locations are shown in **Figure 1;** sampling results from the IDSE are summarized in **Table 1.** The Stage 2

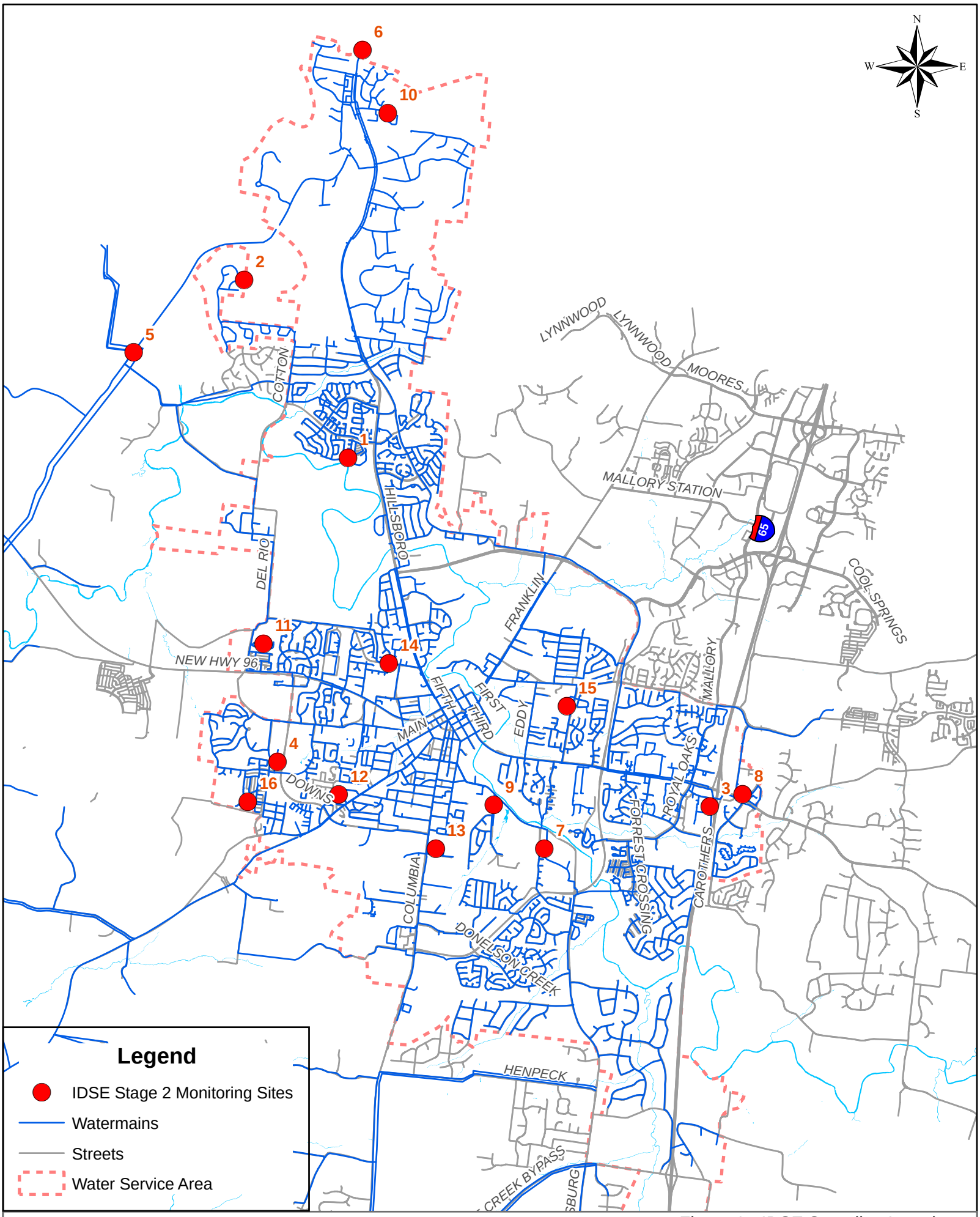


Figure 1 - IDSE Sampling Locations
Water Distribution System
City of Franklin

CDM

D/DBPR will apply the current compliance standards of running annual average (RAA) of 80 micrograms per liter (µg/L) for total trihalomethanes (TTHMs) and 60 µg/L of the sum of the five regulated haloacetic acids (HAA5) to individual locations in the distribution system. The City of Franklin falls in the category of a Schedule 2 system, which requires compliance with the new Stage 2 D/DBPR requirements by October 2012.

The change to a location-specific basis for regulating DBPs will have a significant effect on the measures required to achieve compliance, because each location must comply with the DBP MCLs; the ability to average results from sample locations that have elevated TTHM and HAA5 concentrations with those that have lower concentrations will no longer be possible. As shown in **Figures 2** and **3,** there are multiple locations that have average values that exceed the regulatory limits for both TTHM and HAA5 when sample results are averaged on a locational basis. Consequently, changes may be required in the treatment process, in addition to optimization of the distribution system operation, to comply with the Stage 2 D/DBPR.

One important consideration with regard to the overall facility upgrades, in light of DBP management, is the Harpeth Valley Utility District (HVUD) water supply. A cursory review of seasonal variations in DBP concentrations indicates that both TTHM and HAA5 concentrations are higher during seasons when water demands are high and the majority of water is purchased from HVUD. When no water can be withdrawn from the Harpeth River, the City operates the WTP such that a minimum flow of 1 mgd of the water purchased from HVUD is rerouted through the WTP, primarily to address low distribution system pressures in the southern portion of the City's service area. While the primary purpose of this mode of operation is to feed the distribution system to maintain system pressures, it also results in improved water quality with regard to DBPs in the distribution system because it aids in managing water age in the southern portion of the service area. Additional details on the water distribution system will be provided in a separate technical memorandum.

The WTP Report does not include an alternatives analysis for helping the City meet Stage 2 D/DBPR compliance, nor does it include recommendations that would adequately lower DBP concentrations at the compliance sampling locations. The City is already aware of the DBP issues and has already performed some alternative process testing, such as with magnetic ion exchange resin (MIEX), and other alternatives. CDM continues to support the City to evaluate reasonable and feasible options for DBP control both at the WTP and in the distribution system, making use of data already available, to identify the most economical approaches for DBP control.

Table 1
Summary of Stage 2 D/DBPR IDSE Site Data

Site ID	Description	10/8/2007		12/10/2007		2/12/2008		4/8/2008		6/9/2008		8/11/2008		LRAA	
		TTHM mg/L	HAA5 mg/L	TTHM mg/L	HAA5 mg/L	TTHM mg/L	HAA5 mg/L	TTHM mg/L	HAA5 mg/L	TTHM mg/L	HAA5 mg/L	TTHM mg/L	HAA5 mg/L	TTHM mg/L	HAA5 mg/L
1		0.0783	0.0561	0.0492	0.0334	0.0387	0.0324	0.0474	0.0455	0.0648	0.0458	0.0977	0.0528	0.0627	0.0443
2		0.0607	0.0404	0.0493	0.0346	0.0354	0.0289	0.0463	0.0505	0.0615	0.0344	0.1350	0.0602	0.0647	0.0415
3		0.0758	0.0410	0.0654	0.0732	0.0413	0.0443	0.0452	0.0652	0.0901	0.0393	0.1050	0.0566	0.0705	0.0533
4		0.0800	0.0554	0.0628	0.0701	0.0800	0.0408	0.0515	0.0729	0.0850	0.0422	0.1070	0.0588	0.0777	0.0567
5		0.0552	0.0581	0.0394	0.0326	0.0246	0.0212	0.0291	0.0290	0.0602	0.0292	0.0694	0.0354	0.0463	0.0343
6		0.0805	0.0161	0.0327	0.0349	0.0243	0.0261	0.0307	0.0344	0.0537	0.0274	0.0646	0.0411	0.0478	0.0300
7		0.0731	0.0750	0.0334	0.0325	0.0272	0.0255	0.0243	0.0235	0.0619	0.0358	0.0506	0.0580	0.0451	0.0417
8		0.1120	0.0276	0.0853	0.0202	0.0544	0.0410	0.0617	0.0598	0.0876	0.0675	0.1380	0.0396	0.0898	0.0426
9		0.1180	0.0725	0.0476	0.0202	0.0360	0.0317	0.0366	0.0348	0.0855	0.0528	0.1070	0.0554	0.0718	0.0446
10		0.0624	0.0501	0.0404	0.0489	0.0445	0.0289	0.0530	0.0489	0.0777	0.0513	0.0793	0.0514	0.0596	0.0466
11		0.0923	0.0828	0.0635	0.0303	0.0496	0.0446	0.0332	0.0363	0.0644	0.0295	0.0694	0.0388	0.0621	0.0437
12		0.1340	0.0194	0.0534	0.0440	0.0358	0.0420	0.0699	0.0796	0.1500	0.0926	0.0964	0.0655	0.0899	0.0572
13		0.1620	0.1170	0.1630	0.0712	0.0604	0.0566	0.0780	0.0813	0.1710	0.0594	0.1360	0.0076	0.1284	0.0655
14		0.0742	0.0710	0.0573	0.1000	0.0394	0.0325	0.0374	0.0345	0.0548	0.0483	0.0832	0.0497	0.0577	0.0560
15		0.0933	0.0068	0.0602	0.0410	0.0591	0.0472	0.0528	0.0299	0.0853	0.0444	0.1330	0.0548	0.0806	0.0374
16		0.1150	0.0065	0.0877	0.0535	0.0547	0.0444	0.0013	0.0011	0.0058	0.0018	0.0127	0.0000	0.0462	0.0179

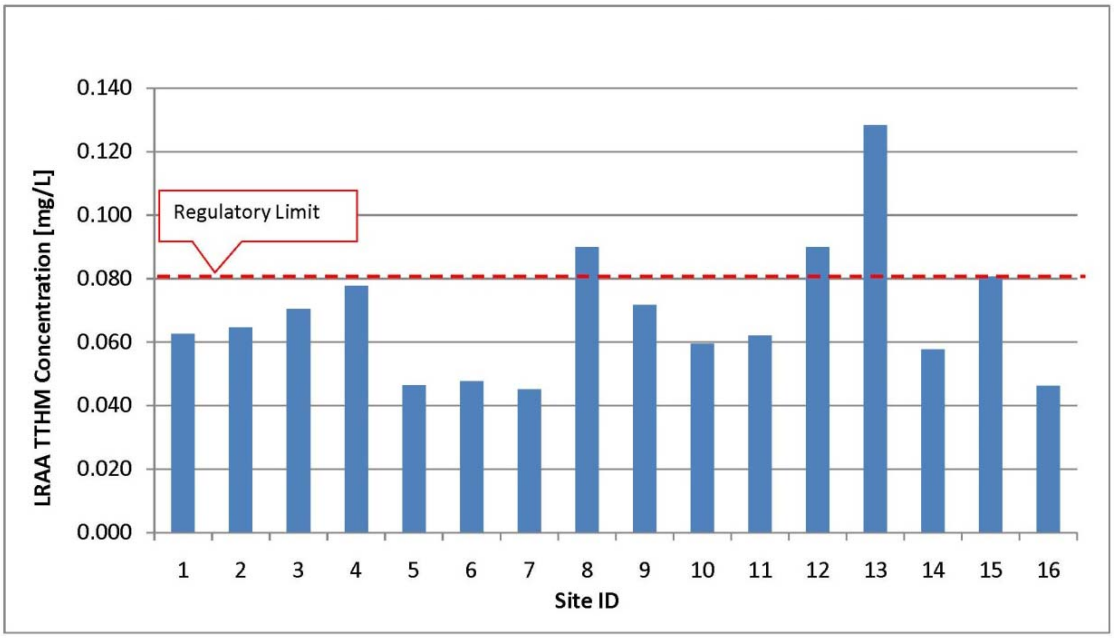


Figure 2
LRAA of TTHM Concentrations at IDSE Sites

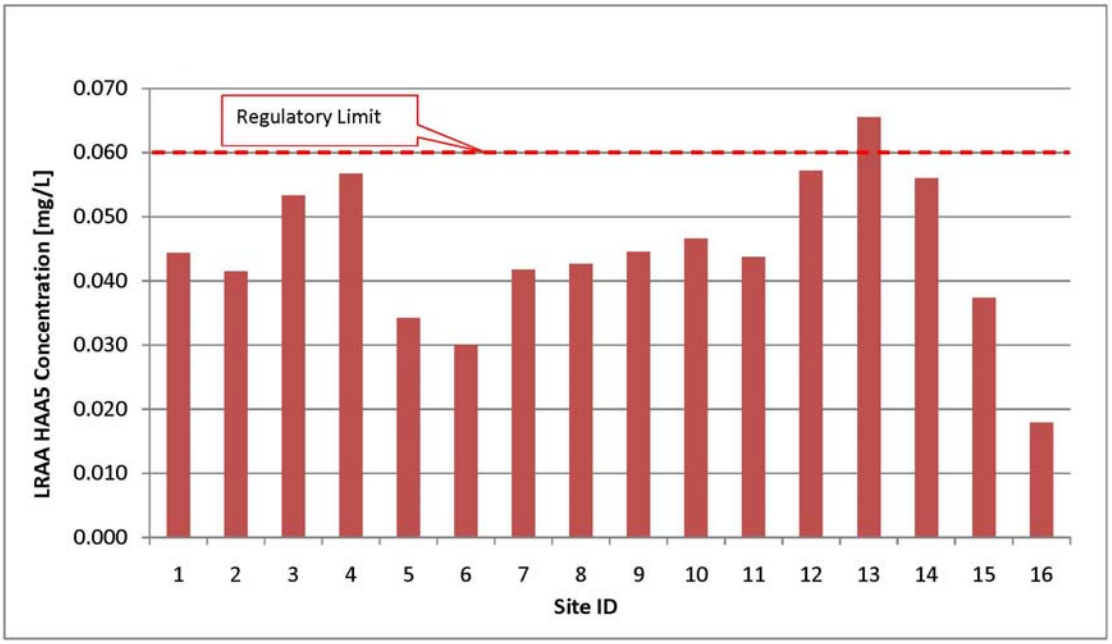


Figure 3
LRAA of HAA5 Concentrations at IDSE Sites

2.2 LT2ESWTR Compliance

The EPA published the Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) on January 5, 2006 which was promulgated to improve control of microbial pathogens. The LT2ESWTR is expected to reduce drinking water related exposure to *Cryptosporidium* substantially, thereby reducing both illness and death associated with cryptosporidiosis through source water monitoring, additional treatment techniques, and higher standards for drinking water quality. The LT2ESWTR requires additional treatment for *Cryptosporidium* in cases where source water average concentrations exceed 0.075 oocysts per liter. The City of Franklin source water testing results were slightly above this limit, requiring an additional 1-log removal and/or inactivation for *Cryptosporidium* (e.g., placing the City of Franklin in Bin 2 under the LT2ESWTR).

The previous WTP Report assumed that Franklin would be Bin 1 with no additional treatment for *Cryptosporidium* required, although the WTP Report does discuss potentially pursuing a membrane treatment option, if *Cryptosporidium* concentrations are determined to be higher than assumed. Consequently, the WTP Report does not provide an evaluation of the most cost-effective approach for LT2ESWTR compliance. However, complying with this rule is less complicated than meeting Stage 2 D/DBPR compliance. There are fewer options for meeting LT2ESWTR, requiring less study to compare options; and a full study is not necessary, although treatment options and safety factors should be established for this facility. The alternatives for meeting LT2ESWTR compliance include implementation of modified operations management practices, UV disinfection, or membrane filtration. Operational practices involve maintaining turbidity under 0.15 NTU in all filters, over 95 percent of the time every month. This is a low-cost approach but leaves no safety factor and places the plant in the position of being at risk of a Notice of Violation due to a relatively small plant upset. Additional options are provided in the LT2ESWTR Toolbox (http://www.epa.gov/safewater/disinfection/lt2/pdfs/guide_lt2_toolboxguidancemanual.pdf).

While not specifically a part of this review, a cost estimate for the option of a UV disinfection process, that is expandable to a UV-advanced oxidation process (AOP), was developed for the WTP. This information has been developed in conjunction with other ongoing IWRP evaluations. The option was developed such that the initial installation of UV disinfection equipment would address LT2ESWTR compliance requirements, discussed above, and the system would be sized so that it could be upgraded to a UV-advanced oxidation process (UV-AOP), in the future.

Initially, the concept of applying a UV-AOP was considered as an option for implementation at a potential new wastewater treatment plant (WWTP), proposed to be located upstream of the existing WTP. The process was proposed as a method of addressing compounds that would potentially cause taste and odor, as well as other currently unregulated and emerging constituents. Initial UV-AOP equipment capital cost proposals were obtained for the proposed new WWTP; equipment proposals from two vendors were in excess of $18M. This information was presented and discussed with the IWRP Steering Committee; and, based on this discussion the decision was made to evaluate the UV-AOP process option at the WTP. Capital cost proposals

were solicited for UV equipment that would be expandable to UV-AOP at the WTP. Equipment cost proposals were substantially lower in cost than for similar equipment at the proposed new WWTP because of the quality of the water to be treated. The vendor supplied information for UV-AOP at the WTP was used to develop construction cost, which is summarized along with other cost updates in Section 4.0.

3.0 Facility Process Upgrades

Sections III and IV of the WTP Report briefly, but adequately, describe minimum requirements for upgrading the existing water treatment facilities and expanding the treatment capacity from 2.1 to 4.0 million gallons per day (mgd). If the plant capacity is not increased, upgrades to the existing facilities should be prioritized with an implementation schedule and corresponding budget. The City has continued to support facility maintenance and other process improvements, such as installation of a variable speed drive on the river intake pump, to maximize plant efficiency. However, if there is a large project at the plant, such as a treatment capacity expansion, economies of scale would suggest correcting major deficiencies concurrently, as recommended in the WTP Report. CDM comments on recommended upgrades and alternatives for each process are presented in the following Sections.

3.1 Raw Water Intake and Pump Station

The WTP Report covers options for upgrading the traveling screen versus providing new intake screens in the river. Both options are feasible as presented in the WTP Report; however, smaller raw water pumps may merit consideration, if the WTP capacity is to remain at 2.1 mgd.

3.2 Raw Water Reservoir

The WTP Report addresses the importance of repairing the major leak in the raw water reservoir and addressing the buildup of spent backwash waste solids. To date, the reservoir rehabilitation project has been completed with a new storage volume of 113 million gallons.

3.3 Mixing

The WTP Report describes the existing mixers and chemical feed points but makes no recommendations or evaluations regarding changes or improvements. A brief review of mixing intensity and the optimal feed points for chemicals is warranted. For example, powdered activated carbon would be more effective on a per-unit-weight basis, if fed further upstream instead of co-currently with coagulant.

3.4 Flocculation

The age of the existing flocculators justifies consideration of improvement options, but the WTP Report recommends a complete process change from walking beam flocculators to paddle wheel flocculators, without a comparison of alternatives. The WTP Report states, “the equipment is in satisfactory condition but probably needs to be replaced, due to its age and parts availability.”

However, there are no letters from suppliers of walking beam flocculators to support this claim. Unless City staff has further investigated parts availability, other than that reflected in the WTP Report, contacting alternate manufacturers to obtain quotes on upgrading the existing equipment and comparing alternatives is suggested.

3.5 Settling Basins

The WTP Report presents two options for the settling basins: 1) upgrading and expanding with more conventional basins employing tube settlers or 2) abandoning the conventional basins and retrofitting one of the basins with a dissolved air flotation (DAF) system. This drastic change to a high-rate process is inadequately justified in terms of both cost and performance. There are no pilot testing results demonstrating that one alternative is better through seasonal water quality changes. Plant staff and the Tennessee Department of Environment and Conservation (TDEC) need to be comfortable that any process change can be successfully implemented and that the change would positively result in improved treatment performance. As such, if a DAF process alternative is to be further pursued, then testing should be considered. Also, the economics of proposed alternatives would change, if the existing walking beam flocculators only require minor upgrades or if State requirements result in changes in the design loading rates.

3.6 Filters

The recommendation to upgrade the valves and replace the filter underdrains is not unusual. Filter underdrain inspection could be performed to verify the need for replacement. CDM concurs with the conclusion that conversion to a backwash system that uses air scour alone, as well as in combination with a concurrent water backwash, would reduce finished water usage and improve filter cleaning.

3.7 Transfer Pumps and High Service Pumps

The WTP Report recommends upgrades as needed under the alternatives that expand plant capacity. CDM concurs with the additional recommendation that the exact horsepower and hydraulic conditions should be checked in final design.

3.8 Chemical Systems

CDM is in agreement that chemical systems should be upgraded, as needed, if plant capacity is increased. The City has already upgraded the facility to address issues associated with the use of gas chlorine by switching to use of delivered bulk sodium hypochlorite solution. However, as noted above, optimal chemical feed locations were not evaluated.

4.0 Cost Estimates

The construction cost estimates in the WTP Report were escalated to current dollars and evaluated for reasonableness. Minimal documentation was provided in the WTP Report regarding development of cost estimates, making it difficult to assess the adequacy of

contingencies and other capital cost factors. Given this uncertainty and the early stage of the project, a 25-percent contingency has been provided for other items that will become apparent as the design progresses. Costs have been revised to reflect 2011 dollars, using a 3 percent per year factor. Detailed cost estimate tables for the conventional and DAF options are provided as attachments to this memorandum. The membrane filtration option cost was also updated, because the worst-case, in terms of cost for LT2ESWTR compliance would be adding membrane filtration technology. Updated costs for these project options have been summarized in Table 2 with supporting documentation provided as an attachment to this memorandum.

Less costly options, such as operational modifications and implementation of UV disinfection that meet LT2ESWTR requirements should have been investigated in the WTP Report. CDM has provided an opinion of probable construction costs for UV disinfection, as well as an adder for upgrading for UV-AOP, which is summarized along with cost information in Table 2. Further, it is important to note that the WTP Report did not evaluate DBP control options for future compliance, nor did it include costs for advanced treatment for DBP control. CDM is currently working with City staff to identify distribution system improvements that could aid in managing DBPs, however, additional investigation into other treatment and management options at the WTP are merited.

Table 2
Updated Construction Cost Estimates (Year 2011 Dollars)

Item	Upgrade Existing 2.1 mgd WTP	4.0 mgd Treatment Alternative		
		No. 1 (Conventional)	No. 2 (DAF)	No. 3 (Membranes)
Raw Water P.S. w/ Travelling Water Screen	$ 1,330,000	$ 1,330,000	$ 1,330,000	$ 1,330,000
Replace or Upgrade Existing Flocculation Equipment	$ 50,000	$ 50,000	-	$ 50,000
Upgrade Existing Settling Basin Nos. 1, 2, & 3	$ 340,000	$ 340,000	-	-
Construct Settling Basin Nos. 4 & 5	-	$ 2,040,000	-	-
Add DAF to Existing Settling Basin No. 3	-	-	$ 2,480,000	-
Upgrade Existing Filters	$ 1,530,000	$ 1,530,000	$ 1,530,000	-
Replace 2 mgd Clearwell Transfer Pump with 4 mgd Pump	-	$ 260,000	$ 260,000	-
Convert Existing Filters to Membranes	-	-	-	$ 6,500,000
Replace 2 mgd High Service Pumps with 4 mgd Pump	-	$ 760,000	$ 760,000	$ 760,000
Upgrade Existing Chemical Feeders	$ 40,000	$ 70,000	$ 70,000	$ 70,000
Sitework and Buried Piping	-	$ 750,000	$ 750,000	$ 750,000
Subtotal	**$ 3,290,000**	**$ 7,130,000**	**$ 7,180,000**	**$ 9,460,000**
Optional UV Disinfection System	$ 684,000	$ 684,000	$ 684,000	$ 684,000
Optional AOP Adder	$ 750,000	$ 750,000	$ 750,000	$ 750,000
Project Total with UV and AOP Option	**$4,724,000**	**$ 8,564,000**	**$ 8,614,000**	**$10,894,000**
Legal, Technical and Other Costs (15%)	$ 709,000	$ 1,285,000	$ 1,292,000	$ 1,634,000
Project Total	**$ 5.4 M**	**$ 9.8 M**	**$ 9.9 M**	**$ 12.5 M**

5.0 Drinking Water Quality

CDM reviewed the July 2009 draft Drinking Water Quality Evaluation and Recommendations (draft WQ Evaluation) that were developed by Metcalf & Eddy/AECOM in conjunction with Hazen and Sawyer. The purpose of this WQ Evaluation was to identify appropriate methods to reduce the potential for water quality issues in the City's distribution system and to identify additional distribution system modifications to reduce the risk of water quality issues in the future. This draft WQ Evaluation and related work focused on Franklin's distribution system model to simulate chlorine decay and using those results to develop recommendations regarding maintaining chlorine residuals and reducing TTHMs. In summary, CDM is in concurrence with the following findings:

- Reducing water age and reducing chlorine residual can help reduce TTHMs,
- Use of booster chlorine can allow lower residuals earlier in the system and lower TTHMs, and
- Automatic flushing devices can help lessen TTHMs.

However, the draft WQ Evaluation presents and discusses mainly THM data with limited review of the HAA5s. Both classes of these DBPs are regulated and Franklin's IDSE data suggest compliance will be difficult for both THMs and HAAs, as previously described in **Section 2.1**. The WQ Evaluation does not discuss the fact that addition of automatic flushers which can be used to lower water residence time and hence DBPs at the locations feeding the flushers only; and that adding flushers after the IDSE evaluation alters the residence times of specific locations and therefore where maximum TTHM and HAA5 site are located in the system. The end result is that TDEC could request additional TTHM and HAA5 sampling at other locations without flushers or repeating the IDSE. Further, the draft WQ Evaluation does not present all of the TTHM and HAA5 data from the IDSE study and does not describe the four TTHM sites and one HAA5 site that had locational running annual averages (LRAAs) exceeding the Stage 2 DBP compliance limits. It is important to note that several of the higher TTHM and HAA5 sites from the IDSE study are in the portions of the distribution system that are primarily fed from the City of Franklin WTP, thus it is recommended that treatment optimization options at the WTP also be evaluated along with the ongoing efforts to identify distribution system improvements to manage water age.

The draft WQ Evaluation does however, describe that HVUD will be required to assist with TTHM and HAA5 compliance per the State of Tennessee rule that states "parent systems designated by the department that routinely sell water to consecutive systems with MCL violations for TTHM or HAA5 shall meet 0.048 mg/L TTHM and 0.036 mg/L HAA5 at the entry point and master meter for the consecutive system in order to demonstrate enhanced coagulation." This is an important consideration in the overall management of DBPs.

Memorandum

To: *City of Franklin IWRP Team*

From: *CDM Smith*

Date: *January 2012*

Subject: *City of Franklin Integrated Water Resources Plan (IWRP)*
Water Distribution System Improvements Modeling (79935.T1.WDIST)

Executive Summary

As part of the development of its Integrated Water Resources Plan (IWRP), the City of Franklin (COF) has undertaken an evaluation of its water distribution system. This evaluation included a review of the existing water distribution system hydraulic model, primarily focusing on reducing the high water age in portions of the COF system, as well as a long-term review of the hydraulic capacity of the system under build-out demand conditions.

As part of the water age review, both operational and infrastructure recommendations have been included in this technical memorandum. CDM Smith has recommended a total of nine (9) projects for implementation to reduce the water age to less than a week, based on 2010 average day demand conditions, in efforts to maintain compliance with disinfection by-product (DBP) regulations. The estimated cost of implementation of these recommended improvements is $2.1M. In addition to these improvements, CDM Smith installing a new, comprehensive SCADA system, which would allow the COF to improve the monitoring and operation of the distribution system. The estimated cost of a new system-wide SCADA system is approximately $830,000.

Distribution system improvements have also been recommended to provide adequate transmission for long-term demands in the southern section of the service area. The total estimated cost of these projects is $5.9M. The timing of these hydraulic improvements will be based on the continued development and population growth in the southern portion of the City.

Finally, in addition to providing a more economical water supply, an upgrade to the existing water treatment plant (WTP) from a maximum capacity of 2.1 million gallons per day (MGD) to 4 MGD, in conjunction with implementing the treatment upgrades recommended in the CDM Smith technical memorandum dated August 2011, *Review of CTE Design Report: Franklin Water Treatment Plant dated July 2006*, would improve water quality in the central and southern extents of the distribution system. This would also provide the operators greater flexibility when operating the distribution system. The increased capacity of the WTP would provide a greater level of in the event the Harpeth Valley Utility District (HVUD) water supply was not available.

1.0 Introduction

As part of the development of its Integrated Water Resources Plan (IWRP), the City of Franklin (COF), with the assistance of CDM Smith, has undertaken an evaluation of its water distribution system. The evaluation consisted of two primary tasks:

- Review of the existing water distribution system, primarily focusing on reducing the high water age within the COF system. The COF water system has exceeded limits for haloacetic acids at sampling locations in more than one quarterly sample over the past two years and has until October 2012 to reduce these concentrations to remain compliant with Stage 2 Disinfectants and Disinfection Byproducts Rule (Stage 2 D/DBPR). The Stage 2 D/DBPR was promulgated in January 2006 and required systems to first conduct an Initial Distribution System Evaluation (IDSE) to identify compliance monitoring sites for the Disinfection Byproduct Maximum Contaminant Levels (DBP MCLs). The Stage 2 D/DBPR will apply the current compliance standards of running annual average (RAA) of 80 micrograms per liter (µg/L) for TTHMs and 60 µg/L of the sum of the five regulated haloacetic acids (HAA5) to individual locations in the distribution system. The change to a location-specific basis for regulating DBPs will have a potential effect on the measures required to achieve compliance, because each location must comply with the DBP MCLs. The water distribution system experiences high water age throughout the year contributing to the formation of excessive DBPs. The Stage 2 D/DBPR is discussed in greater detail in the technical memorandum *Review of CTE/AECOM Design Report: Franklin Water Treatment Plant, dated July 2006*, from CDM Smith,

- Assess the long-term hydraulic capacity of the distribution system for future projected demands.

This technical memorandum presents the findings and recommendations of both tasks. These evaluations were conducted using an existing computer hydraulic model of the water distribution system, previously calibrated by CTE in 2009 to tracer testing. The existing model is represented in the WaterGEMS V8i platform. WaterGEMS is a proprietary software produced by Bentley Systems, Incorporated.

2.0 Distribution System Overview and Operation

The COF water distribution system is located in Williamson County, Tennessee, and spans approximately 19,300 acres. The 2010 average day and maximum day water demands were 6.3 and 8.5 MGD, respectively. The system is comprised of nearly 500 miles of ¾-inch to 36-inch diameter mains. The Franklin water treatment plant, rated at a maximum capacity of 2.1 MGD, is located near the center of the distribution system, along the Harpeth River on Lewisburg Highway (State Road 431). To meet the demands of the system, water is also provided by a wholesale utility, Harpeth Valley Utility District (HVUD), through two connection points located

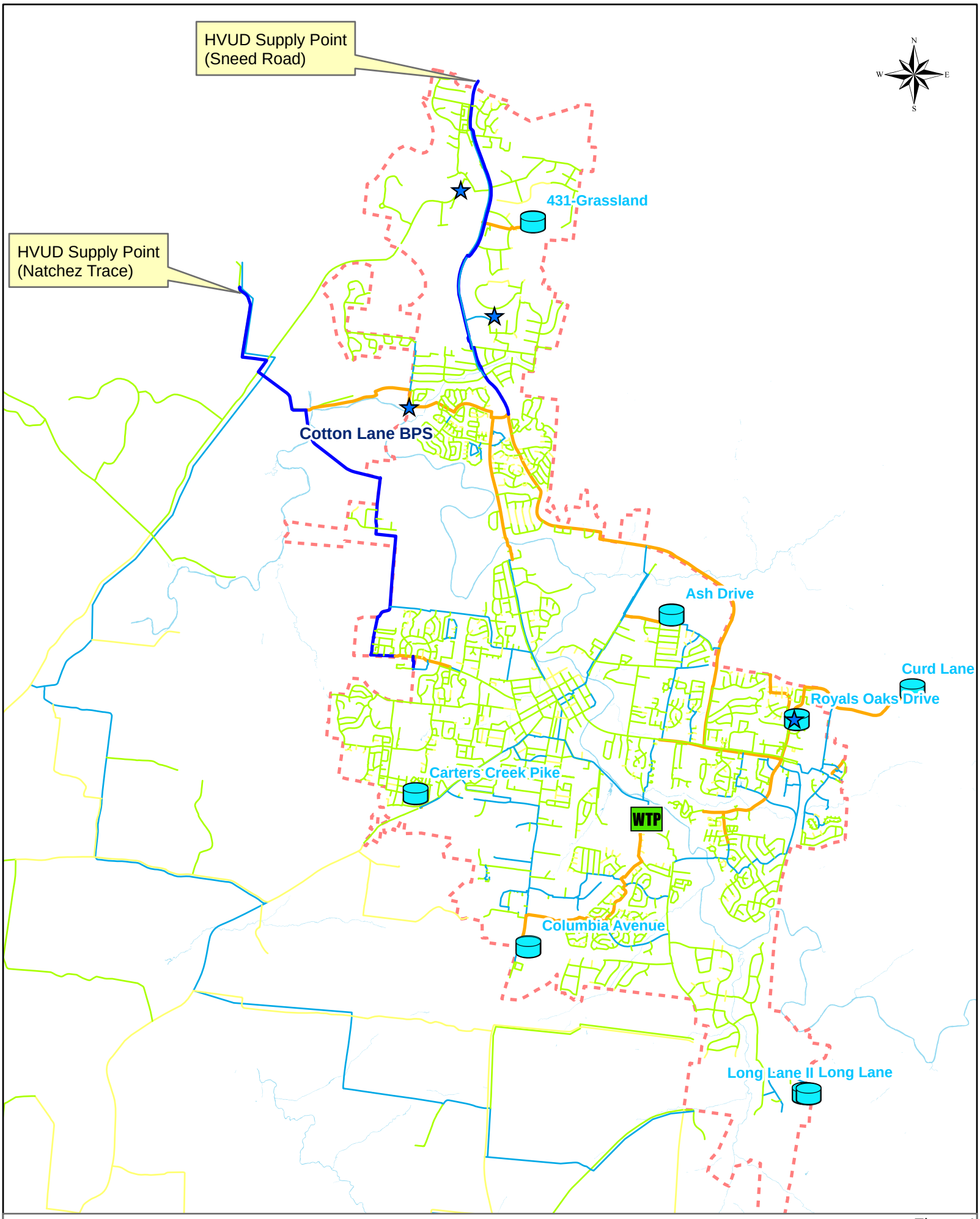
at the northern end of the system. An overview of the COF distribution system as well as a hydraulic profile of the system is provided in **Figures 1** and **2**, respectively.

The majority of the COF distribution system is well looped with 12-inch to 36-inch diameter trunk transmission mains. The central portion of the system contains smaller diameter trunk mains, mainly 12-inch and 16-inch, which convey water from the Franklin WTP. The northern portion of the system contains the larger diameter trunk mains (20-inch to 36-inch) which convey water from the HVUD connection points into the remaining portions of the CoF system. The Cotton Lane booster pump station (BPS) is utilized to boost flow from the western HVUD connection point (Natchez Trace connection point) to the rest of the distribution system; the Cotton Lane BPS is primarily used during summer months when the COF is experiencing high demand periods.

There are 7 ground storage tanks with a total system storage capacity of 10 million gallons (MG). There are plans to replace the existing Long Lane 0.5-million-gallon storage tank with the new Long Lane II 2-million-gallon storage tank in early 2012, which will result in a total system storage capacity of 11.5 MG.

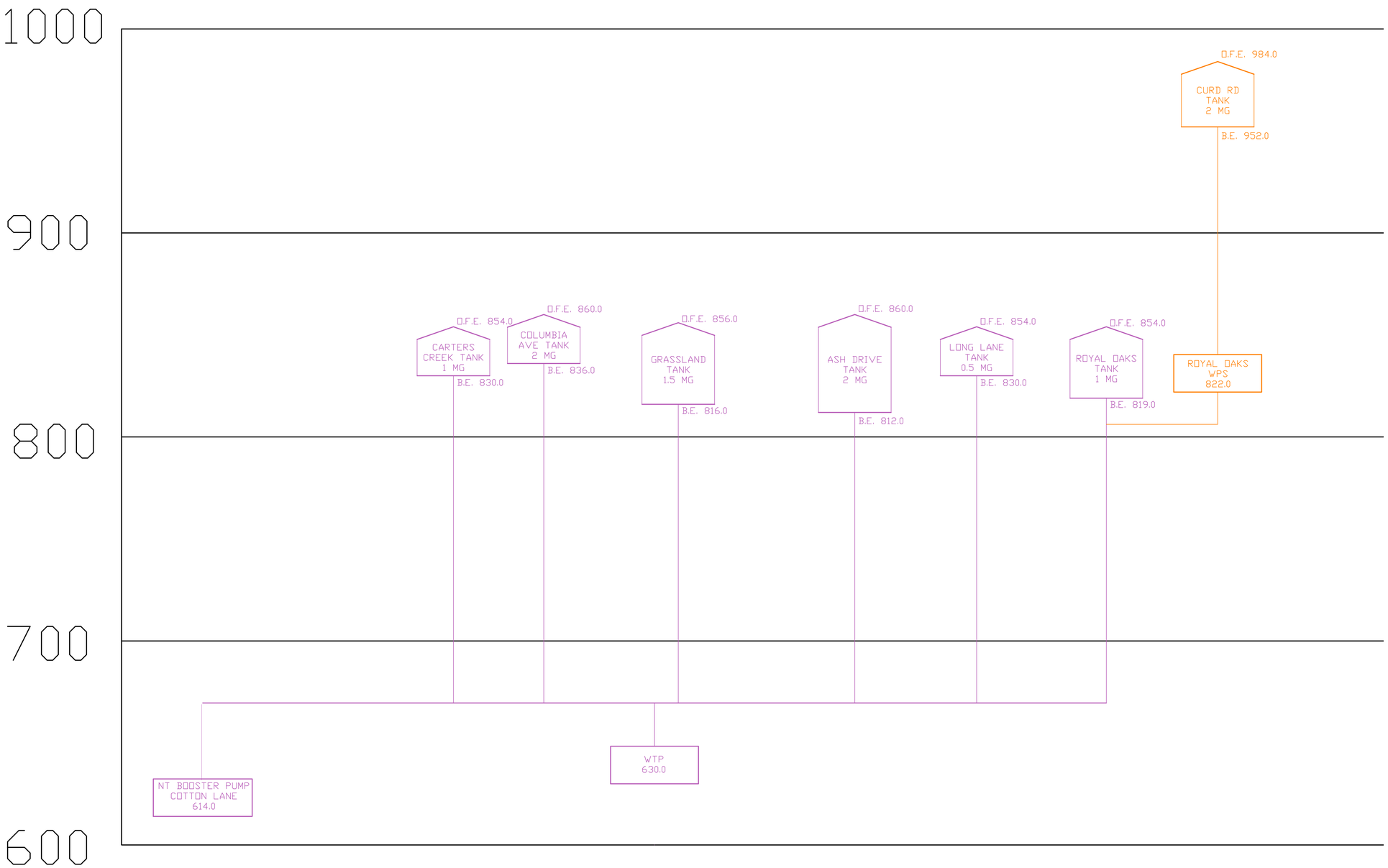
The COF system has two primary pressure zones: a low pressure zone and high pressure zone. The majority of the system is maintained on the same hydraulic gradeline in the low pressure zone; six storage tanks with a combined capacity of 8 MG are located in the low pressure zone. The storage tanks have overflow elevations ranging from 854 feet to 860 feet.

The high pressure zone is served by a single pump station, Royal Oaks Drive, which is used to lift flow from the low pressure zone to the Curd Road Tank. The 2 MG Curd Road Tank serves the eastern portion of the COF service area has an overflow elevation of 984 feet. Two smaller pump stations, Legends Ridge BPS and Hidden River BPS, are located toward the northern portion of the system and are used to boost flow from the low pressure zone to isolated neighborhoods. There are no storage tanks are used in these areas.



CDM

Figure 1
Water Distribution System
City of Franklin



REV. NO.	DATE	DRWN	CHKD	REMARKS

DESIGNED BY:
DRAWN BY: C. Grissom
SHEET CHK'D BY: J. Lind
CROSS CHK'D BY:
APPROVED BY:
DATE: 10-4-10

CDM Camp Dresser & McKee Inc.
consulting engineering construction operations

HYDRAULIC GRADIENT CHART
FRANKLIN WATER DISTRIBUTION SYSTEM

PROJECT NO.
FILE NAME:

SHEET NO.
1

3.0 System Water Age Conditions

High water age is one of the conditions, and most directly correlated parameters leading to excessive DBP formation. This factor is considered the primary indicator for evaluating DBP formation potential in a water distribution system. In the COF distribution system, high water age is due to several related factors:

- Purchased water from HVUD is delivered to the COF distribution system at a higher hydraulic grade line elevation than the COF tank overflow elevations. Discharge pressures at both HVUD metered supply points are controlled by a partially closed, manually operated valve to increase headloss between the two utilities to reduce pressure. The amount of headloss resulting from this operation is not systematic or measured. The current valve setting results in a downstream hydraulic grade line elevation that is higher than the operating level of the storage tanks, thus hindering tank turnover.

- The COF system has a high storage to demand ratio (11.5 MG of storage compared to 6.3 MGD average daily demand) which limits tank turnover and little usage of the stored volume. This, along with aged water entering the system from HVUD combine to cause high water age in portions of the system and an increased DBP formation potential.

Information on the HVUD distribution system was limited at the time of this study; therefore the specific age of the water supplied from that system is unknown. Based on field testing results conducted in 2007, and 2008 cited in the July 2009 draft report titled Drinking Water Quality Evaluation and Recommendations, developed by Metcalf & Eddy/AECOM, in conjunction with Hazen and Sawyer, the total trihalomethanes (TTHM) and HAA5 concentration measured at the HVUD supply points are near or exceed the 60-percent Rule established by the Tennessee Department of Environment and Conservation (TDEC), in November 2008. The 60-percent Rule applies to utilities supplying water to neighbors through wholesale connection points to limit the TTHM and HAA5 concentration to 60 percent of the maximum contaminant limit (MCL), or 0.048 mg/L TTHM and 0.036 mg/L HAA5. Because the TTHM and HAA values have been measured close to, or exceeding 60 percent of the MCL, it was assumed water from HVUD had already aged by the time it reached the COF connection points. It is recommended that regular testing be conducted at these two connection points to better define the DBP concentrations entering the COF distribution system from HVUD.

With regard to tank storage, water age depends on turnover and mixing in the tank. To minimize water age, storage tanks are typically operated such that they feed peak demands during the day and are replenished during the night with a minimum of 20 percent to 30 percent of their tank volume cycled daily. To accomplish this, the storage volume in the system should approximate the average daily demand (ADD) in the system. Storage of at least one day of ADD is required by TDEC and stated in TDEC Community Public Water Design Criteria [Section 8.3.1],

> "The purpose of system storage is to have sufficient water available to provide adequate flow and pressure at peak demand as well as to provide for fire flows when needed. For most water systems, a satisfactory rule-of-thumb to meet these needs is to provide at least the average 24-hour demand in elevated storage. In the absence of an acceptable engineering study of the amount of water the system needs to meet the customer demand and provide for fire emergencies, the projected 24-hour demand at the end of the planning period will be the minimum requirement for elevated storage. This requirement may be reduced when the source, treatment facilities, and pumps have sufficient capacity with standby power capability to supplement peak demands of the system."

The current COF distribution system allows flow from HVUD to directly feed a majority of the system demands. This mode of operation reduces the need for tanks to satisfy demand, thus resulting in minimal turnover. Because the storage to demand ratio is already high, this results in high water age in the storage tanks and their corresponding demand zones.

Figures 3 through 8 illustrate SCADA tank levels recorded in February, April, and August of 2010. These graphs show the limited occurrence of the drain/fill cycle in each tank. Altitude valves are currently used in most tanks to control tank volume. This is evident on the graphs because some tanks never achieve a maximum level (100-percent full) because the altitude valves effectively create a reduced overflow elevation which the tank level should not exceed.

As shown on Figures 3 through 8, the existing achieve minimal turnover over a 30-day period. Based on similar distribution systems in Tennessee, tank levels are typically operated to fluctuate enough to turn over the entire tank volume in 3 to -5 days, equating to approximately 20 - 30 percent daily turnover of the tank maximum volume. However, because of the large volume of aged water the COF system receives from HVUD, tanks in the COF system should be operated to achieve a recommended turnover rate closer to 40 percent, in a given 24-hour period, based on modeling results.

Figures 3 through 8 show gradually increasing tank levels during the month of February and portions of August, indicating the tanks did not empty or drain during that time. Additionally, some of the figures show a sharp drop in tank levels which is caused by the manually induced turnover emptying the tanks.

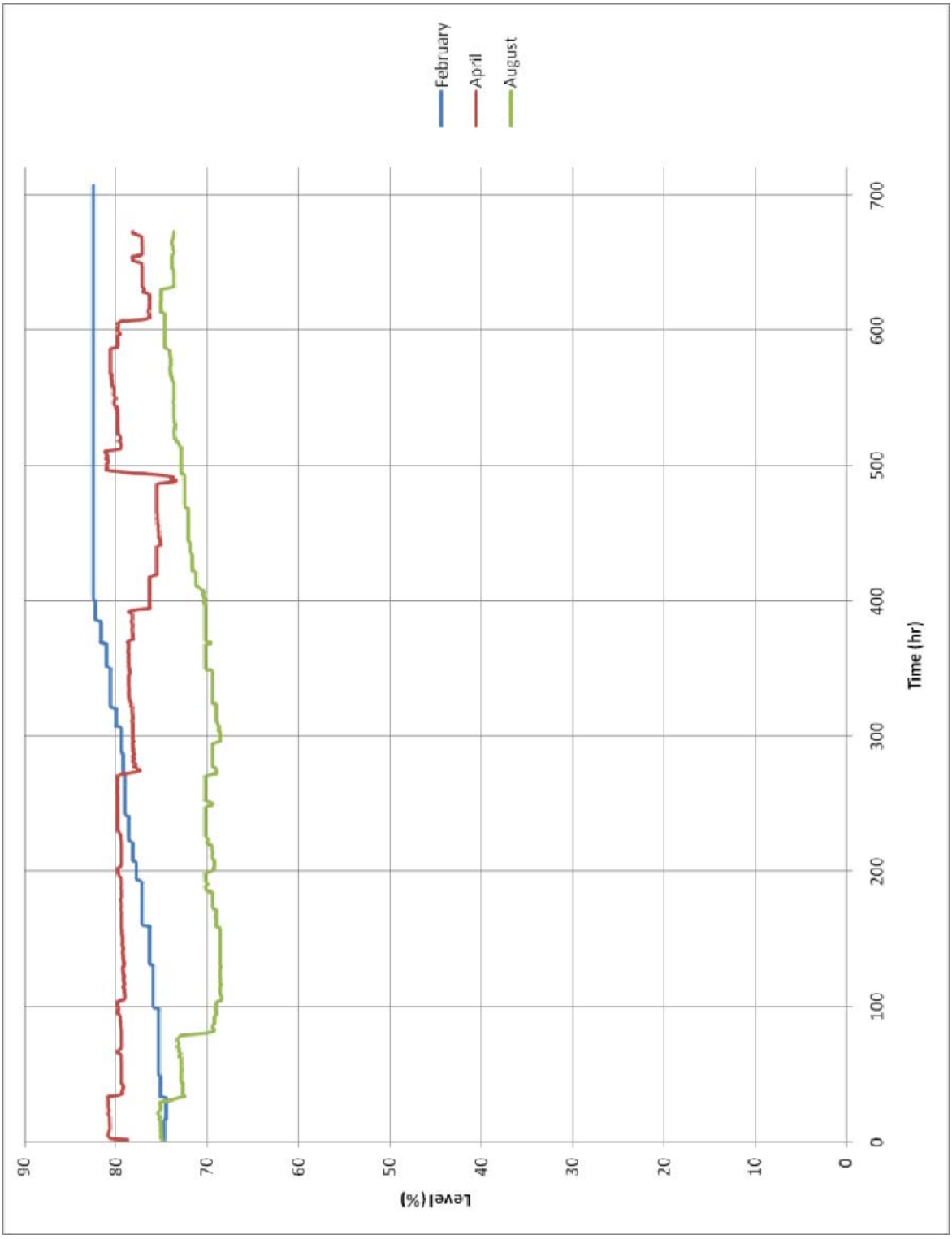


Figure 3
Ash Drive SCADA Recorded Tank Levels

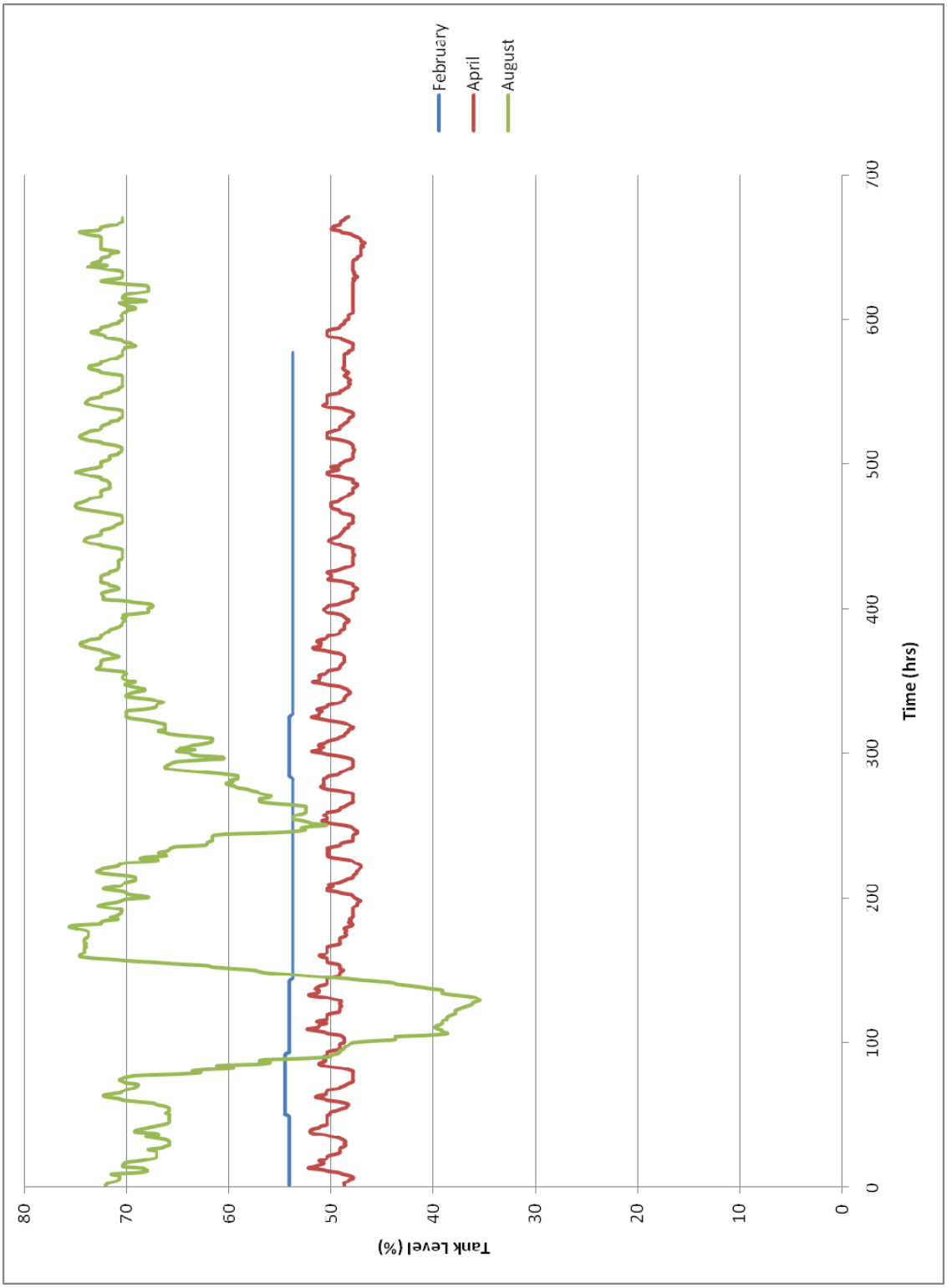


Figure 4
Grassland SCADA Recorded Tank Levels

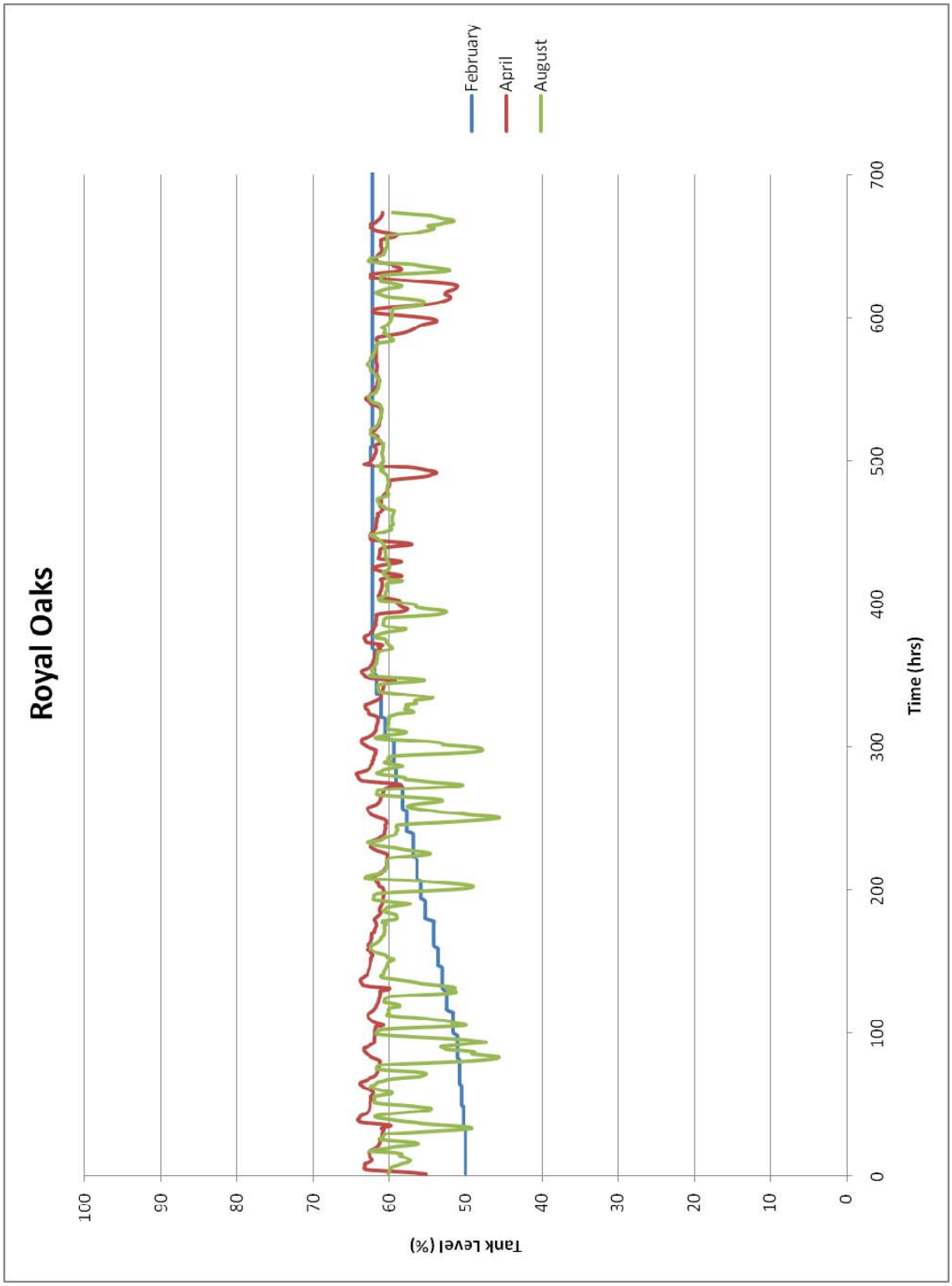


Figure 5
Royal Oaks SCADA Recorded Tank Levels

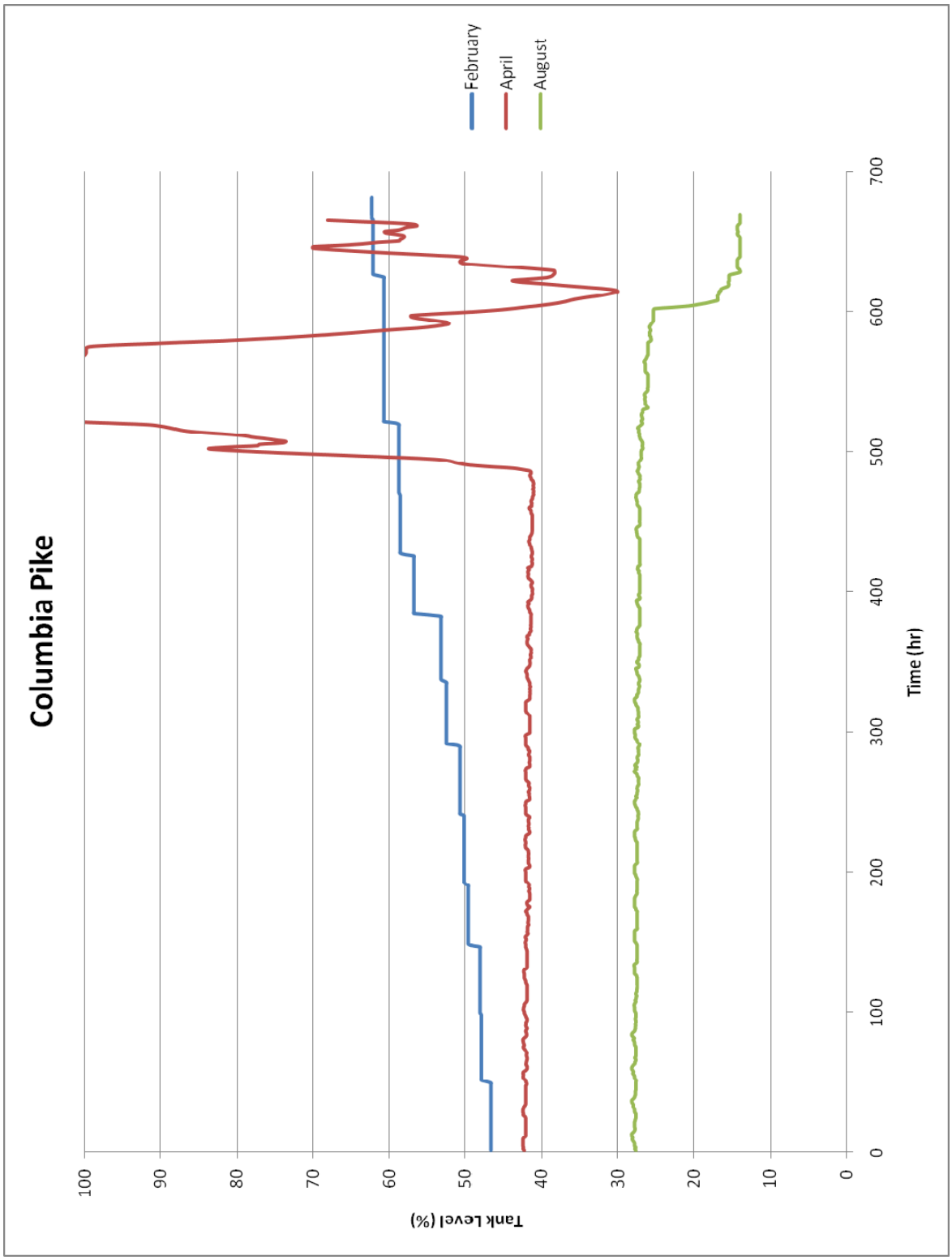


Figure 6
Columbia Avenue SCADA Recorded Tank Levels

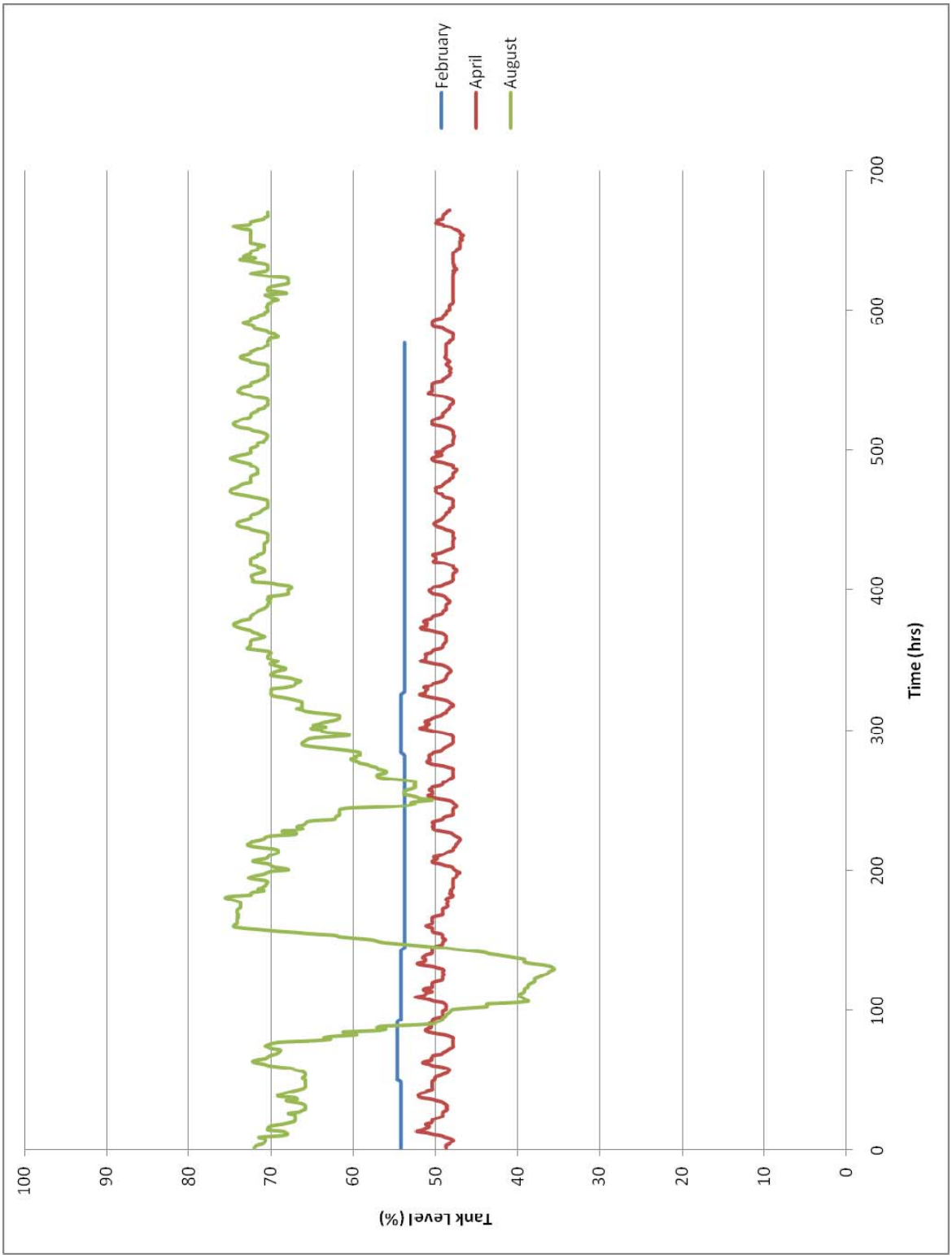


Figure 7
Carters Creek SCADA Recorded Tank Levels

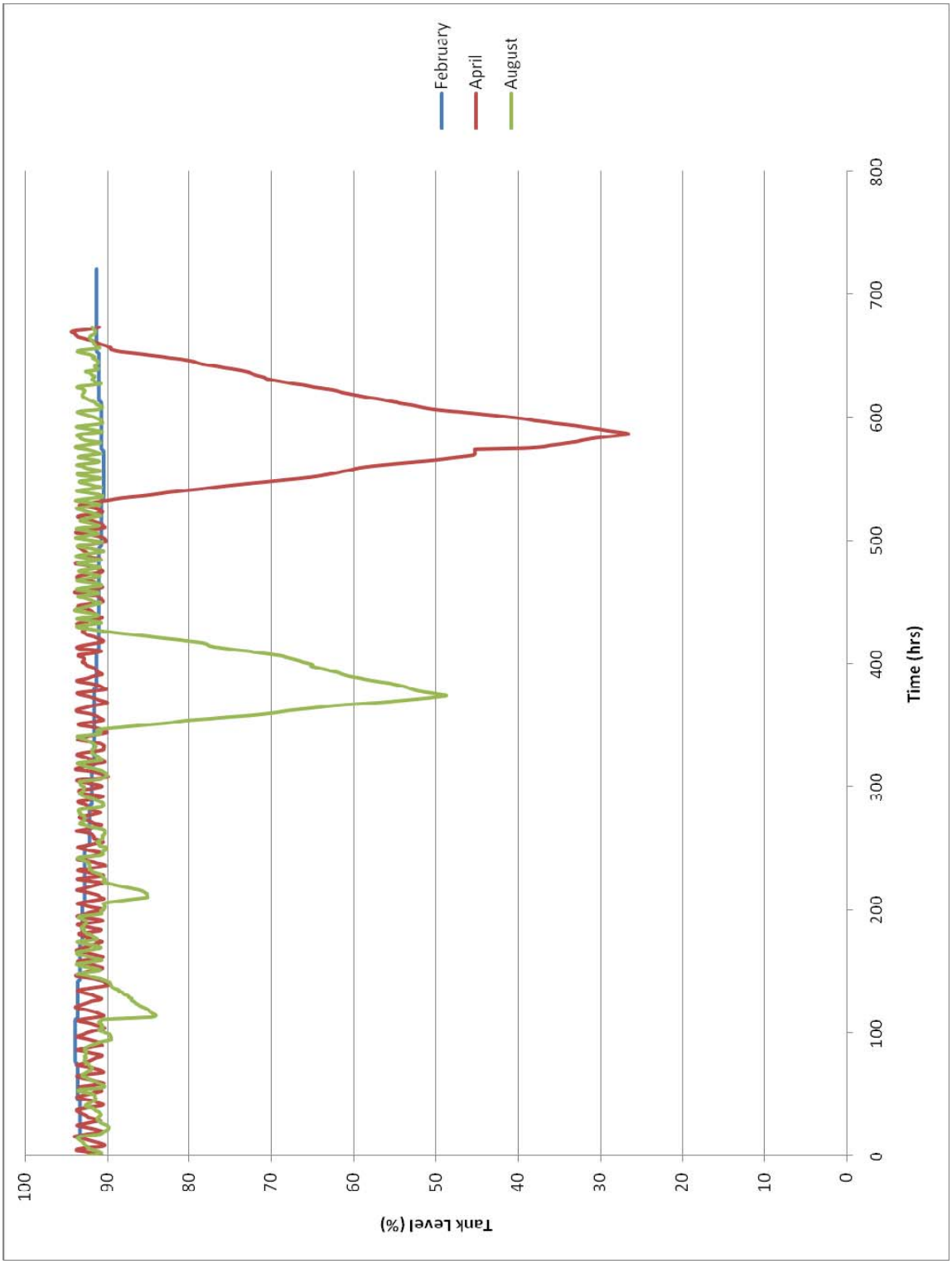


Figure 8
Curd Lane SCADA Recorded Tank Levels

4.0 Existing Water Age Analysis

The Franklin water distribution system model, in Bentley WaterGEMS software platform, was provided to CDM Smith by the COF for modeling the water age reduction alternatives. The model was originally developed by CTE for 2008 conditions and was used in the 2009 Report, *Drinking Water Quality Evaluation and Recommendations*. The COF hydraulic model was developed and calibrated by CTE in 2009. Based on customer billing data and recent SCADA data, demands in the system have not significantly changed since the 2009 calibration. It was assumed, as part of this study, that this model is accurate and no testing or calibration would be performed as directed by the client and documented in the IWRP project scope. However, several newly installed mains were added to the model in February 2011 because COF staff was able to provide information regarding these distribution system improvements, completed since 2008.

The water age analysis was conducted based on the existing (2010) average daily demand (ADD) of 6.3 MGD, assuming that water in the storage tanks is completely mixed, i.e., new water entering the tank is mixed with older water already existing in the tank. A review of the tank inlet/outlet configuration is recommended to ensure short circuiting is not occurring. An analysis of water age within the storage facilities was not included in this scope of work. It was also assumed that water entering the COF system from the HVUD supply points is 3-days old at the time of entry. This assumption may be low based on the testing results discussed in the 2009 CTE report *Drinking Water Quality Evaluation and Recommendations,* which indicated DBPs recorded during 2007 and 2008 at the HVUD supply points were close to, or exceeded 60 percent of the MCL. It was assumed, in the modeling analysis, that HVUD has taken steps to address the aging water and has reduced the water age since the 2007 and 2008 sampling, in an effort to comply with the 60-percent Rule.

Model simulations were conducted to simulate a 2-week period of average demand. Tank levels were compared to SCADA during ADD periods to verify that model tank and pump station operation is similar to field operation. Existing system water age was then analyzed based on the results of these simulations. Metrics utilized to evaluate existing water age included average system water age, number of pipes with maximum water age over 168 hours (7 days), ratio of storage to demand, and percent daily turnover in the storage tanks. Seven days was used as the criteria for maximum water age based on an industry standard for maximum DBP formation potential as described in Section 5710 of *Standard Methods for the Examination of Water and Wastewater (1998).*

Figure 9 illustrates the model predicted water age in the existing COF distribution system. As shown, areas of water age exceeding 1-week are located in the vicinity of the Ash Drive, Long Lane, Curd Lane, and Columbia Avenue storage tanks. As stated previously, the likely cause of high water age in these areas is already aged water entering the system from HVUD, higher hydraulic grade line from HVUD limiting tank turnover, and the large storage to demand. Additionally, isolated mains are shown with high water age in Figure 9; these are typically

dead-end mains and may have very low demands applied to them in the model. This may not be representative of the actual demands in these areas and it is recommended to review recent billing data and re-allocate the demands in the model, if necessary, to verify that the correct parcel/meter demands are assigned to the correct main.

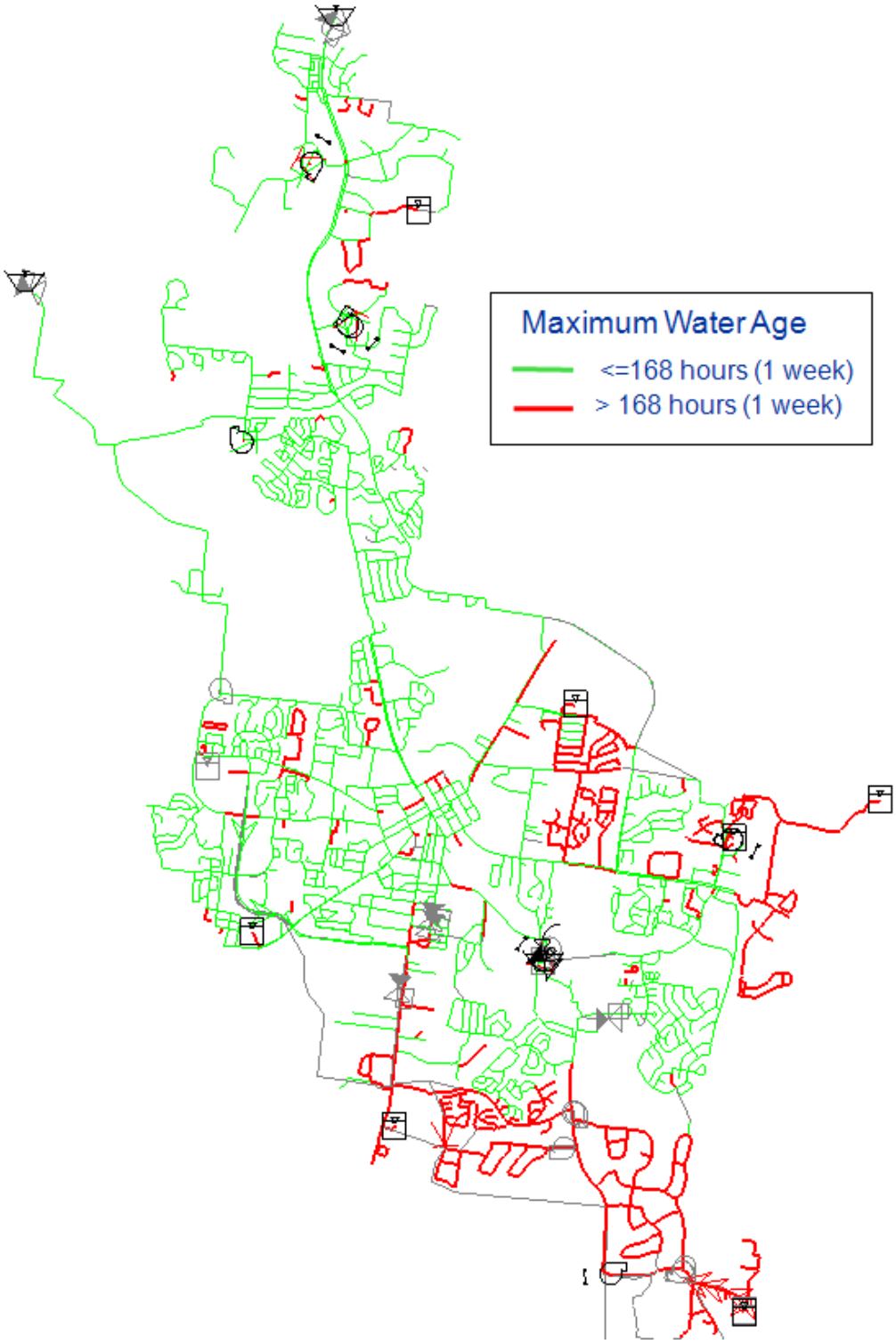


Figure 9 – Existing System Model Predicted Water Age

5.0 Water Age Reduction Modeling and Methodology

The COF's existing hydraulic water distribution system model was used to assess the estimated water age in the distribution system under various demand scenarios as well as develop potential solutions to reduce the estimated age. In addition to model evaluation, a review of the CTE water age evaluation titled "Drinking Water Quality Evaluation and Recommendations," dated July 2009 was conducted. Recommendations were developed from both analyses. The analyses were based on evaluation criteria, which were established based on discussions with the COF:

a. Maintain system pressure greater than 20 psi under all flow conditions TDEC Community Public Water Design Criteria [Section 9.0.1.c].

b. Maintain fire flows of at least 500 gpm with system pressure at minimum 20 psi, based on TDEC requirements (Chapter 324, Paragraph (18) of Rule 1200-5-1-17 Operations and Maintenance).

c. Maintain storage capacity equal to or exceeding 6.3 MG (equal to seasonal existing average daily demand). Based on the TDEC Community Public Water Design Criteria [Section 8.3.1], system storage shall equal or exceed the ADD. This provides emergency storage for one full day of ADD while minimizing hydraulic residence time and water age less than 168 hour.

d. Water age less than 168 hours in all pipes based on 7-day standard for maximum DBP formation potential [Section 5710 of *Standard Methods for the Examination of Water and Wastewater (1998]).*

e. Maintain average pipe velocity less than 5 feet per second (fps) and peak velocity less than 10 fps, based on AWWA guidelines which recommend a maximum design velocity of 10 fps, with velocities less than 5 fps as the desirable range.

To improve water age in the areas shown on Figure 9, the following goals were established:

a. Reduce the hydraulic grade line elevation from the two HVUD feeds in a controlled and measurable fashion to improve the hydraulic effectiveness of the existing storage facilities

b. Decrease the storage to demand ratio by either removing tanks which do not receive adequate turnover, operating them at lower levels (decrease storage), or by increasing the zone of influence of existing tanks (increase demand on individual tanks)

c. Increase the volume of new water to the southern extents of the COF system

The following options were evaluated in effort to reduce the water age in the COF's water distribution system without compromising other system parameters (pressure, fire flows):

a. Removal of tanks from service during low demand periods to reduce the storage to demand ratio. With seasonal demands varying on average from 79 percent of ADD in April to 135 percent ADD in the peak summer season, storage volumes could be regulated proportionally. Because demands are lower in winter, storage capacity could be reduced during winter months low and increased accordingly during late summer when demand is highest. This could be accomplished by removing select tanks from service and/or substantially reducing the maximum level in the tanks by lowering the system pressure head. These tanks could be temporarily returned to service during seasonal high demand periods as needed; however, this would require re-disinfecting prior to returning the tanks to service

b. Installation of pressure reducing valves at the two HVUD connection points to reduce downstream hydraulic grade line elevation to increase turnover and improve the hydraulic effectiveness of the existing storage facilities

c. Control of flow entering distribution system from HVUD connection points to allow the tanks to satisfy the peak demands, forcing them to drain and fill

d. Add transmission mains to minimize headloss in flow conveyance to tanks in the southern extent of the system; and install connector pipes to close localized loops at existing dead-end pipes

e. Add flushing at distal ends of dead-end pipes which would require greater than 100 ft of new pipe to connect back to the system via a loop

f. Install additional mains or add booster pump to improve tank filling on the southern end of the system and serve the elevated area around the Royal Oaks tank

g. Create pressure zones along the north-south split to allow operational pressure zones to isolate HVUD supply source from Franklin WTP supply source and vary tank operation based on supply pressures

6.0 Water Age Improvement Recommendations

Based on a combination of recommended improvements that were previously identified (Section 5.0) to improve water age in the system, it is anticipated that the COF can make significant improvements to the overall water age in the distribution system. The following improvements, shown on **Figure 11**, need to be implemented in sequential order, as presented, to effectively decrease water age:

1. *Reduce pressures from HVUD supply points.* Replace existing valves with pressure reducing valves (PRV) at the two HVUD supply points to regulate pressures in a controlled and measurable fashion. The PRVs should be adjustable so that the downstream pressure can be varied depending on the time of day. A reduction in the hydraulic grade line elevation of the HVUD supply improves the hydraulic effectiveness of the storage facilities. By decreasing HVUD pressures during the day (high demand period) and increasing HVUD pressures at night (low demand period), the PRVs allow storage tanks to naturally drain a percentage of storage during the day and fill from HVUD at night. An example of the PRV pressure variation over a day with the recommended PRV improvement implemented, is shown in **Figure 10**.

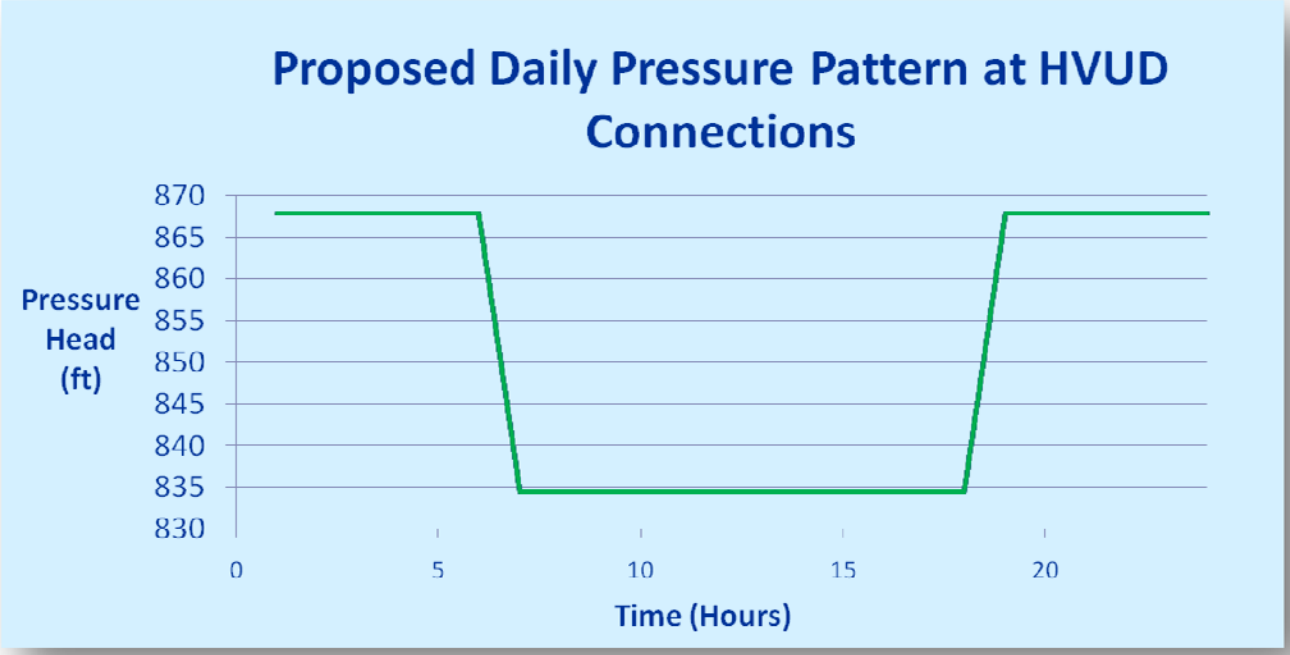


Figure 10
PRV Pressure Variation

2. *Remove the Grassland storage tank from service*. Due to the close proximity of Grassland to the Sneed Road (also known as Hillsboro Road HVUD supply point), the tank will not experience adequate turnover and will distribute aged water to the area immediately around the tank once HVUD pressures are reduced. Due to these concerns and lack of operational benefit from the tank, the Grassland storage tank should be removed from service.

3. *Remove the Royal Oaks tank from service*. This tank shows a limited zone of influence in the model, due to its close proximity to the Ash Drive and Curd Lane tanks. The Royal Oaks BPS draws flow from this tank and therefore, induces turnover. However, because it is pumping directly to the Curd Lane tank, it causes high water age in the Curd Lane repump zone. By removing the Royal Oaks tank from service, the Royal Oaks Drive BPS can draw flow as an in-line BPS and reduce water age in the Curd Lane repump zone. It may be possible to utilize the Royal Oaks tank in the reclaimed water system because as a major main is planned to loop near the tank along Interstate 65.

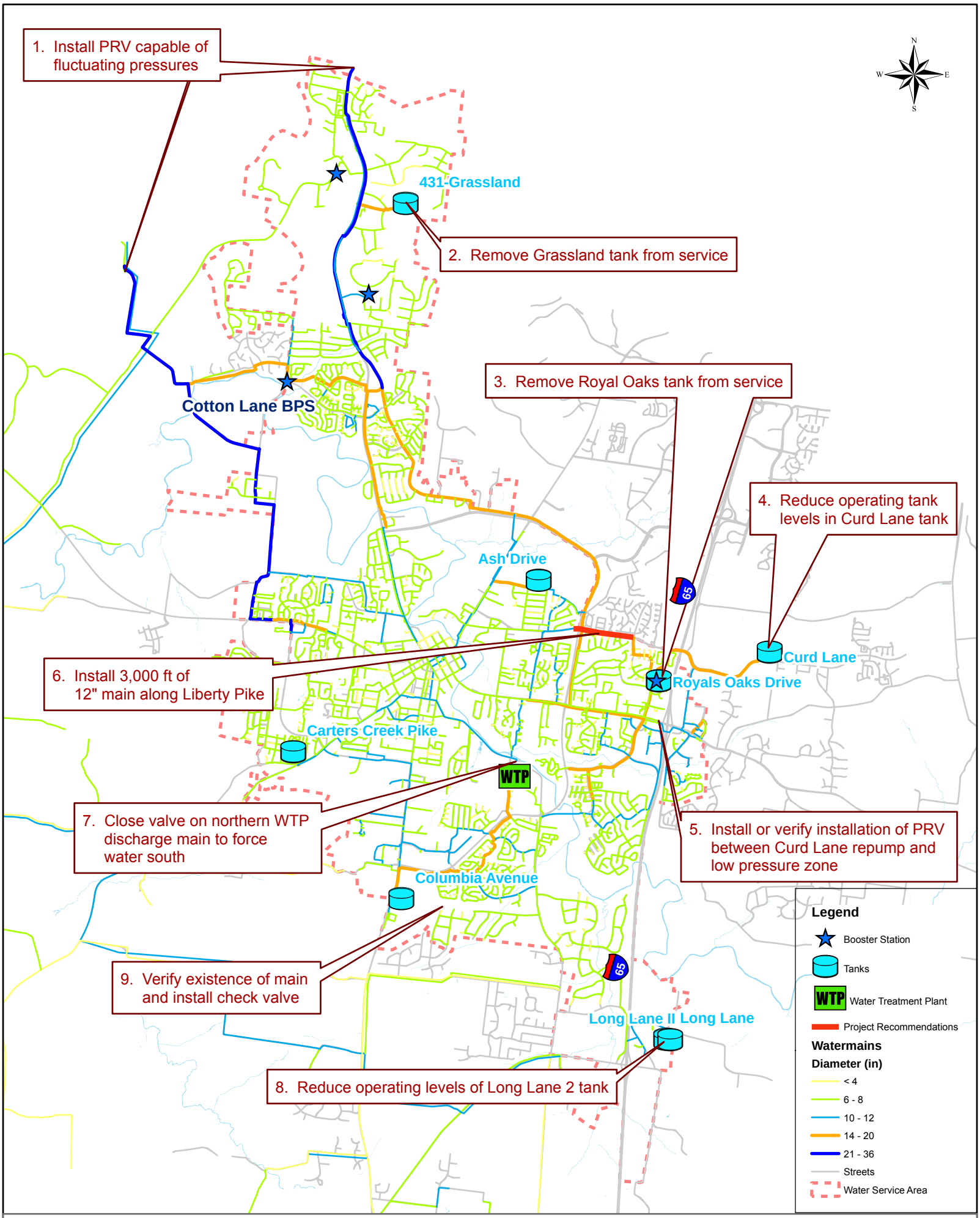
4. *Reduce tank levels in the Curd Lane tank*. While removing the Royal Oaks tank decreases the water age to the Curd Lane repump zone, it does not reduce the water age to less than a week. It is necessary to achieve higher turnover in the Curd Lane tank to decrease the water age in the area and as a result, it is recommended to maintain this tank at no more than 50-percent full, and let the tank drain to about 23-percent full before the Royal Oaks BPS is programmed to fill up to 50-percent again. Because this recommended improvement reduces the amount of available emergency storage to this repump area, which serves the hospital, it is recommended to maintain the existing emergency fire pump and install backup power generation at the Royal Oaks BPS for emergencies.

   It is also recommended that the COF identify and install emergency interconnections to Milcrofton Utilities on the eastern border of this repump zone in an effort to supply more water to this area in the event of an emergency. This recommendation is not anticipated to reduce the system pressures at the hospital to unacceptable pressures. When the tank is at 23-percent full, the pressure at the ground floor of the hospital is anticipated to be close to 75 psi; however, a pressure drop to approximately 60 psi is anticipated when Recommendation 5 is implemented and the PRV opens to circulate flow. Lowering the tank levels may also lower system pressures at the apartment buildings along Royal Oaks, north of Murfreesboro Road. The pressure at the ground elevation at the apartment site is expected to close to 50 psi. If the apartment complex requires higher pressures, it may be desirable to install a private pump to boost pressure for tenants.

5. *Install or verify installation of PRV connecting Curd Lane repump zone to Low Pressure zone*. In order to expand the service area of the Curd Lane storage tank, it is necessary to allow it to feed the low pressure zone, as well as the existing Curd Lane repump zone. To do this, a PRV must be installed between the Curd Lane repump zone and low pressure zone boundaries. COF staff believes there is an existing PRV (currently not in operation) near the Royal Oaks BPS. It is recommended that staff verify existence of this PRV and model its effects on this area to ensure it allows the needed circulation of water between pressure zones. If this PRV does not exist or does not allow provide circulation, it is recommended to install a PRV along Highway 96 between the two zones. Operational controls should allow the PRV to close when the Royal Oaks BPS is in operation to reduce flow recirculation.

6. *Install main near the Ash Drive tank to enlarge demand zone of the tank*. Approximately 3,000 feet of 12-inch diameter main is necessary along Liberty Pike to tie into the existing 10-inch diameter main at Hillhaven Lane and the existing 16-inch diameter main at Liberty Hills Drive. This main allows circulation of water and limits the amount of stagnant water within that area.
7. *Direct new WTP production south.* Close valve on WTP northern discharge main to force flow from the WTP south to the Columbia Avenue and Long Lane tanks. Closing this valve increases the volume of newly produced water in the southern extent of the service area. It also expands the service area of the Ash Drive tank by eliminating a competing source of water for the customers north located north of the WTP and south of the Ash Drive tank.
8. *Reduce levels and expand initial service area of new Long Lane II tank.* The COF is planning to replace the current Long Lane tank (500,000-gallon capacity) with the Long Lane II tank (2- million-gallon capacity) at a higher elevation (overflow elevation of 915 feet) than the existing tank. The new tank will be filled using an in-line BPS along Old Peytonsville Road near the intersection of Lewisburg Pike, essentially creating a new pressure zone. It is recommended that until more development results in increased demands on the tank, this tank should be maintained between 7 feet and 14 feet, out of a maximum operating range of zero feet to 35 feet in order to reduce the volume of water aging in the tank.
9. *Install a check valve to limit Long Lane II tank service area*. Install a check valve on the 8-inch diameter main along Lewisburg Pike between Holly Hill Drive and Donelson Creek Parkway. This will allow the new Long Lane II tank to serve a larger service area and to drain and fill more efficiently. The model indicates there is a 16-inch diameter main running north and south between Oakwood Drive and Wisteria Drive; however, it is not identified in the COF geographic information system (GIS) database. This is a cross country main that does not follow a street path and if this main does not exist, it is recommended to install this 1,400-foot stretch of 16-inch diameter main. This main allows Long Lane II some ability to feed back into the COF Low Pressure zone to increase the demand on the tank. Its distance from the tank will limit flow but it will be able to service a portion of the zone. This main will be necessary in the future as demand increases in the southern portion of the service area.

Locations of the recommended improvements are illustrated in **Figure 11;** model results (**Figure 12**) indicate that implementing these nine recommendations reduces the average water age throughout the system to less than 1-week except for a minimal number of dead-end mains, based on 2010 ADD conditions. In the case of the dead-end mains, it is recommended that looping these dead-ends back into the system be incorporated in the long-term goals of the COF. In the short-term, periodic flushing of these mains is recommended to limit the water age.



CDM

Figure 11
Water Age Project Recommendations
City of Franklin

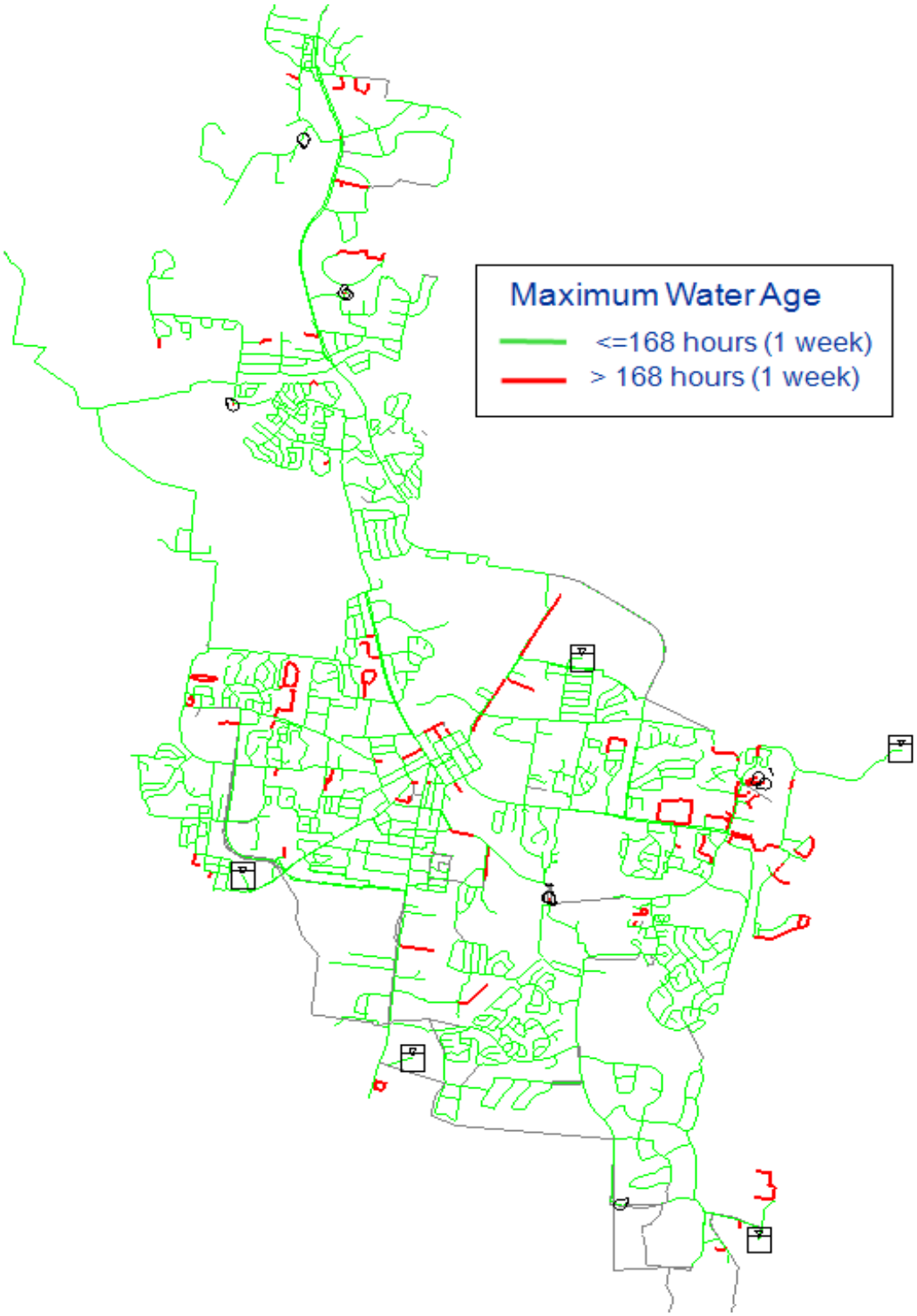


Figure 12
Modeled Water Age After Recommendations

In addition to these operational and infrastructure improvements, it is recommended that COF install a new, comprehensive SCADA system to improve monitoring and control the new PRVs, tank levels, BPS and WTP high service pump (HSP) flow and pressure, as well as system pressures and any other actuated valves within the system. This is critical to maintaining an efficiently-operated and controlled distribution system.

From a water quality perspective, it would be beneficial for the COF to increase the maximum capacity of the WTP from 2.1 MGD to the maximum possible capacity of 4 MGD based on the limitation of the Harpeth River supply source, effectively doubling the current capacity of the WTP. By increasing the water produced from the COF WTP, the system would naturally decrease the amount of aging water required to be purchased from HVUD. This would decrease the water age in the COF system, particularly in the southern extents of the system. By decreasing the overall age of the water entering the system, the operational constraints and tank turnover recommendations listed above can be relaxed in some areas because they are no longer forced to compensate for the high amount of aging water from HVUD. The closer proximity of the COF supply to the demand reduces the water age. Conceptual cost estimates for upgrading the WTP as well as greater discussion regarding increasing the capacity of the WTP is found in the CDM Smith technical memorandum dated August 2010, *Review of CTE/AECOM Design Report: Franklin Water Treatment Plant dated July 2006*.

Water Age Recommendation Cost Estimate

In general, the recommendations have been focused on operational changes rather than infrastructure capital expenditures. Planning level costs were developed for the infrastructure recommendations and unit costs were developed by the CDM Smith construction division, CCI, unless quoted directly from a vendor. In the case of the recommended PRVs, CDM Smith requested a quote from Southern Sales, Inc., in which an electronic actuated pressure reducing valve with manual hydraulic bypass (Cla-Val model 390-07BW) was recommended. Costs for the PRVs include installation of a vault to house the units and SCADA integration. The planning-level cost estimates (**Table 1**) include a 30-percent construction contingency and a 25-percent allowance for legal, administration, and engineering fees.

Table 1 – Water Age Improvements Planning-Level Cost Estimates

Recommendation Number	Item Description	Unit	Quantity	Unit Cost	Material Cost
1	24" PRV at HVUD supply vault	LS	1	$ 200,000	$ 200,000
1	36" PRV at HVUD supply vault	LS	1	$ 350,000	$ 350,000
2	Demolition of Grassland Tank	LS	1	$ 200,000	$ 200,000
3	Remove Royal Oaks Tank from Service†	n/a	n/a	n/a	n/a
4	Reduce Tank Levels in Curd Lane Tank	n/a	n/a	n/a	n/a
5	Install PRV between Curd Lane repump and Low Pressure zone	LS	1	$ 100,000	$ 100,000
6	12-inch DIP on Liberty Pike	LF	3,000	$130	$ 390,000
7	Close valve on northern discharge from WTP	n/a	n/a	n/a	n/a
8	Reduce Tank Levels in Long Lane II Tank	n/a	n/a	n/a	n/a
9	8" Check Valve on Lewisburg Pike	LS	1	$ 10,000	$ 10,000
Material Subtotal					$ 1,300,000
30 percent Construction Contingency					$ 390,000
25 percent Engineering, Legal, Administration Fees††					$ 420,000
Total Cost					**$ 2,100,000**

† Additional costs would be incurred if the tank was incorporated into the reclaim water system.
††Engineering, Legal, and Administrative fees are applied to the Material Subtotal plus the Construction Contingency

* All costs are rounded to two significant figures and expressed in 2011 dollars.

7.0 Growth Related Capacity Analysis

A long-term capacity analysis was also conducted to evaluate future demand projections of the COF system. This section provides a discussion of the methodology for developing the future water demands, model analysis under future demand conditions, and project recommendations for the system to adequately meet future demands.

7.1 Demand Projections

CDM Smith's teaming partner, Smith Seckman Reid, Inc (SSR), assisted with development of long-term demands of the COF system. Because the water distribution system service area is bounded on all sides by existing utilities, the COF does not anticipate a geographical expansion of the service area. Potential increases in demand are projected through development of open land and the possibility of infill within the existing service area boundaries. Population projections and assumptions made during the development process were based on the 2011 Development Report from the COF Planning Department. Where developments were previously defined and approved, the actual proposed number of residential units was used to develop the number or density of units within a development. Where open land, excluding parks or designated recreation areas, was found with no current plan for development, raw acreage, less 15 percent for roads and public areas, was assumed with a density of 2.5 units per acre, based on discussions with the Planning Department. Additionally, open land with slopes in excess of 15 percent, or land within the flood plain was assumed undevelopable and was eliminated from this analysis.

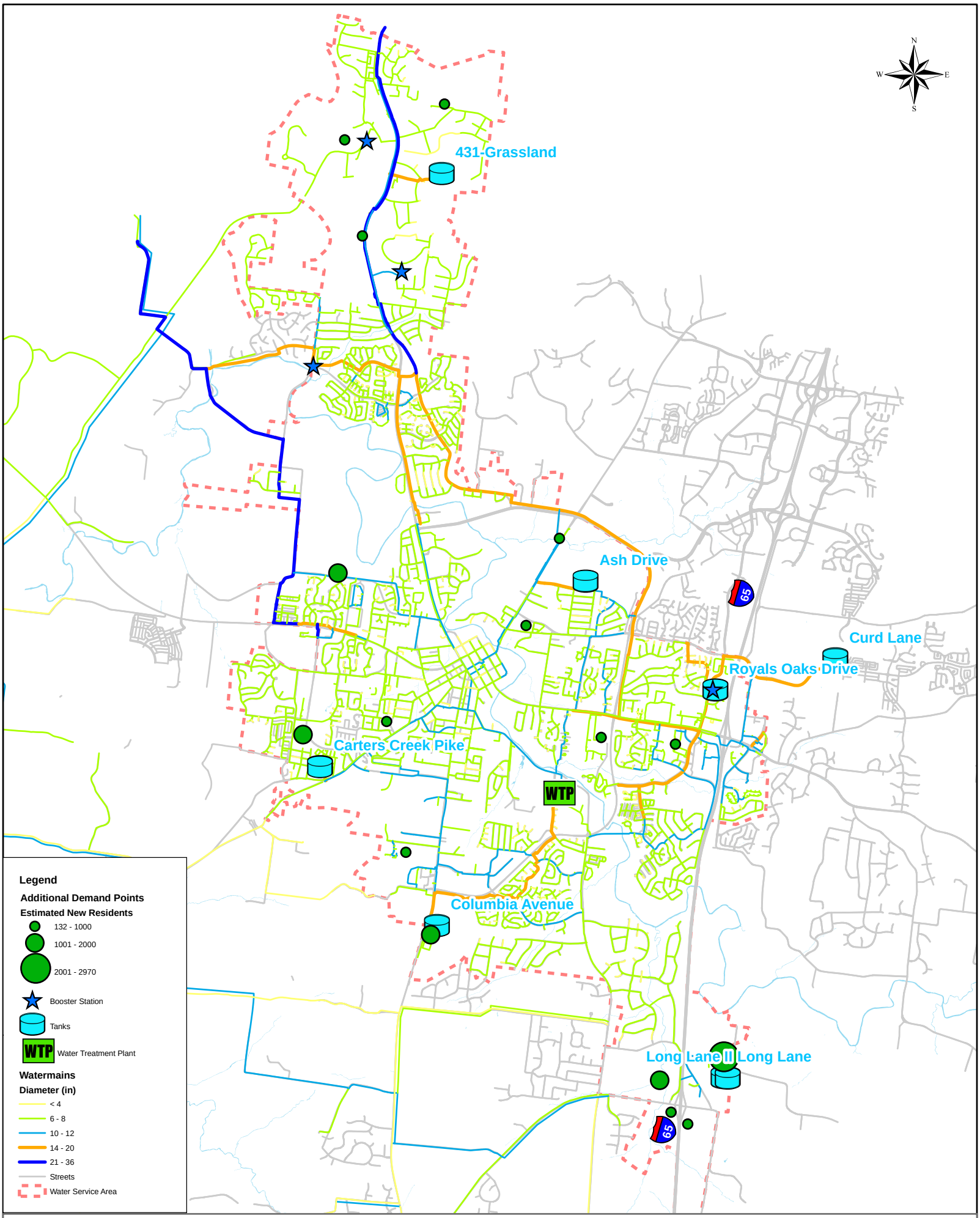
Once the total number of residential units was developed for a build-out condition within the COF service area, future population was developed by assuming a conversion factor of 2.95 people per unit; this number was verified by COF personnel. A per capita demand factor was applied to develop the future average daily demand for the service area, based on current total WTP production and HVUD purchased water in the existing COF service area, divided by the total number of people within the service area. The unit demand factor was applied to the future population to determine the future average daily demand. To develop a maximum day demand (MDD) under build-out conditions, a peaking factor of 1.5, based on the 10 State Standards for demand peaking factors, was applied to the average daily demand. **Table 2** illustrates the current and build-out demand conditions developed using the above described process and utilized during the model analysis.

Table 2 – CoF Water System Customers and Water Demand Estimate

Scenario	Customers	ADD (MGD)	MDD (MGD)
2010 (Existing)	38,200	6.3	9.45
2040 (Build-Out)	50,700	8.3	12.45

After demand projections were developed, a point shapefile was created utilizing GIS software to geographically locate the demand resulting from future developments. The demand point shapefile was applied to the WaterGEMS model in a separate scenario to represent build-out

ADD. The MDDs were also assigned under build out conditions and future demands were not assigned to smaller neighborhood mains including 6-inch or 8-inch in diameter. This is critical because smaller mains could create a bottleneck and a restriction that would not effectively illustrate the total hydraulic capacity of the distribution system. Therefore, demands were assigned to the closest large diameter distribution or transmission main in order to assess the overall hydraulic capacity of the system and its ability to convey these future flows. **Figure 13** illustrates the assigned locations of the build-out average daily demands within the hydraulic model.



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0 0.45 0.9 1.8 2.7 Miles

Figure 13
Build-Out Demand Allocation
City of Franklin

7.2 Hydraulic Capacity Analysis

After future ADD and MDD were assigned in the model, it was run for an extended period simulation. The following criteria were used to assess the ability of the existing distribution system to service the customer base under future demand conditions:

a. Maintain system pressure greater than 20 psi under all flow conditions TDEC Community Public Water Design Criteria [Section 9.0.1.c].

b. Maintain fire flows of at least 500 gpm with system pressure at minimum 20 psi, based on TDEC requirements (Chapter 324, Paragraph (18) of Rule 1200-5-1-17 Operations and Maintenance).

c. Maintain storage capacity equal to or exceeding 6.3 MG (equal to seasonal existing average daily demand). Based on the TDEC Community Public Water Design Criteria [Section 8.3.1], system storage shall equal or exceed the ADD. This provides emergency storage for one full day of ADD while minimizing hydraulic residence time and water age less than 168 hour.

d. Water age less than 168 hours in all pipes based on 7-day standard for maximum DBP formation potential [Section 5710 of *Standard Methods for the Examination of Water and Wastewater (1998])*.

e. Maintain pipe velocity less than 5 feet per second (fps) and peak velocity less than 10 fps, based on AWWA guidelines which recommend a maximum design velocity of 10 fps, with velocities less than 5 fps as the desirable range.

In general, the model indicates the system has difficulty conveying flow from HVUD supply points in the north to the southern extents of the service area. To increase conveyance capacity to the southern extents of the system, the following projects are recommended:

a. Install approximately 6,000 feet of 24-inch main along Columbia Avenue which would connect the 24-inch main on Downs Boulevard that feeds into the 16-inch main tying into Columbia Avenue tank. This main is necessary under 2040 ADD conditions to convey flow to maintain necessary tank levels in the Columbia Avenue tank; it is, however adequate under 2040 MDD, peak hour conditions.

b. Install approximately 10,000 feet of 16-inch main along Oakwood Drive, Henpeck Lane, and Lewisburg Pike to connect into the 16-inch main running north and south between Oakward Drive and Wisteria Drive, (water age recommendation 9). This 16-inch main should feed the suction side of the new BPS along Lewisburg Pike, feeding the Long Lane II tank. The discharge side of the Lewisburg Pike BPS should also be a 16-inch main until it reaches the Long Lane II tank; however, this main is already in the planning stages by the COF and is planned to be a 16-inch main.

c. Install larger flow meters and associated piping in the HVUD vaults. These meters and short lengths of associated 10-inch and 12-inch mains may be creating excessive headloss under future maximum day scenarios. This recommendation should be reevaluated as demands in the COF system increases over time.

These three projects, recommended to improve hydraulic capacity under long-term demand projections, are shown in **Figure 14**. It should be noted that the projects focus on major transmission mains required to meet projected future demands. In some areas, including Goose Creek, additional distribution mains will be necessary as development is planned and becomes active within the system. It is recommended to assess these developments utilizing the hydraulic model as information becomes available; as a result the COF should maintain, update, and calibrate the hydraulic model so that it continues to represent accurate system conditions.

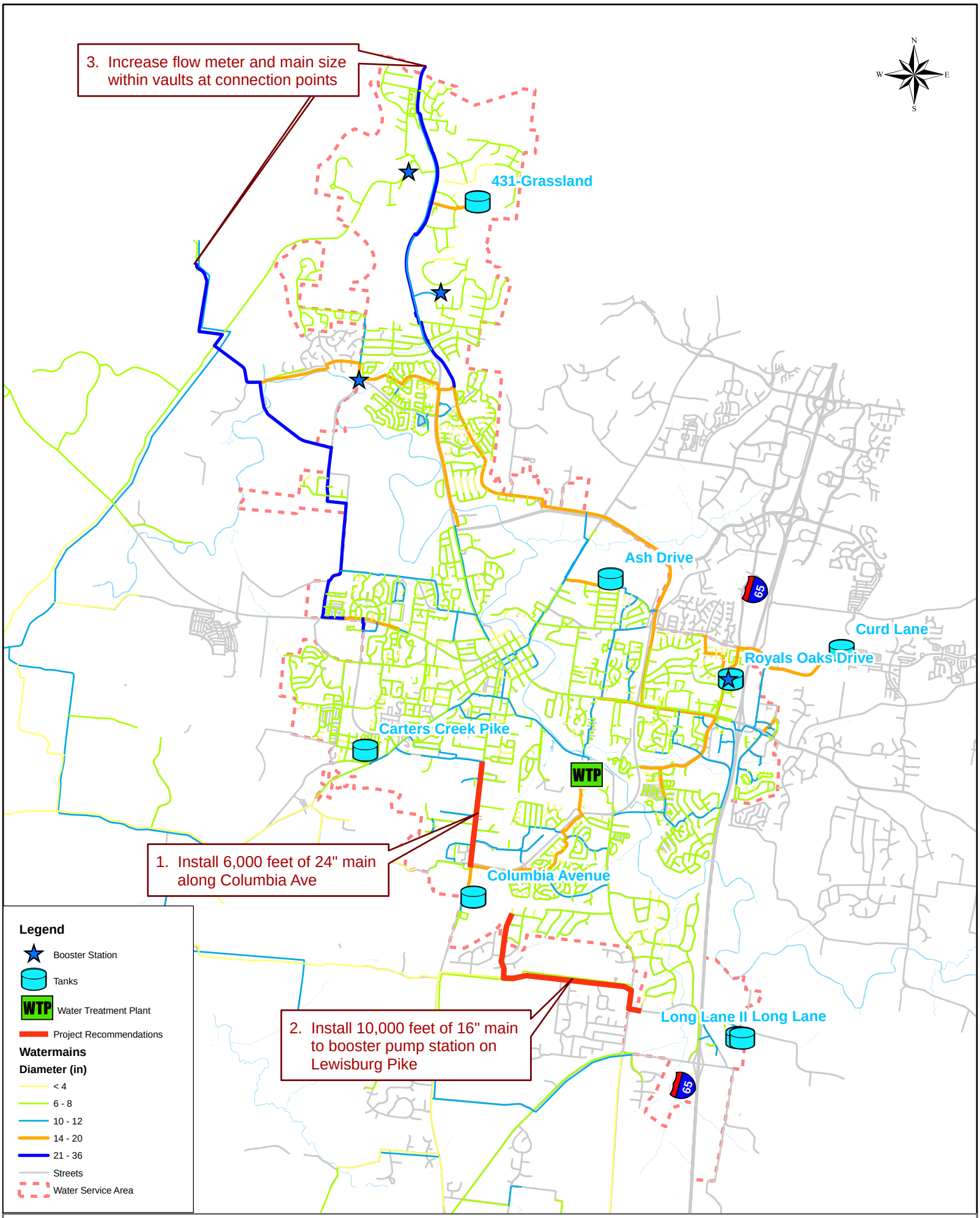

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Figure 14
Long-Term Project Recommendations
City of Franklin

7.4 Build-Out Capacity Recommendation Cost Estimate

Planning-level costs were developed for the infrastructure recommendations necessary to meet projected future demands. Unit costs were developed by the CDM Smith construction division, CCI, unless quoted directly from a vendor. The planning-level cost estimates include a 30-percent construction contingency and a 25-percent allowance for legal, administration, and engineering fees which is applied to the sum of the materials subtotal and construction contingency. **Table 3** lists the planning-level costs for the long-term improvements recommended in the COF system.

Table 3 – Long-Term Improvements Cost Estimate

Recommendation Number	Item Description	Unit	Quantity	Unit Cost	Material Cost
1	20" DIP - Columbia Ave	LF	6,000	$ 220	$ 1,300,000
2	16" DIP - Long Lane Connector	LF	10,000	$ 175	$ 1,800,000
3	Upsize HVUD Vaults	LS	2	$ 250,000	$ 500,000
Material Subtotal					$ 3,600,000
30 percent Construction Contingency					$ 1,100,000
25 percent Engineering, Legal, Administration Fees††					$ 1,200,000
Total Cost					**$ 5,900,000**

*All costs are rounded to two significant digits and presented in 2011 dollars.
††Engineering, Legal, and Administrative fees are applied to the Material Subtotal plus the Construction Contingency.

8.0 Summary

As part of the IWRP, CDM Smith was tasked with assessing the COF distribution system with regard to water age in the existing system and hydraulic capacity under build-out conditions. Operational changes, as well as multiple infrastructure capital projects, are recommended to decrease the water age throughout the COF system. The total cost to implement these projects is approximately $2.1M. CDM Smith also recommends the COF upgrade their existing SCADA system to improve overall operation and maintenance of the distribution system. It is estimated the cost of this upgrade is approximately $830,000.

Finally, it is recommended that the COF upgrade their existing WTP from a maximum capacity of 2.1 MG to a maximum capacity of 4 MG. This would improve water quality in the system by increasing the volume of water from the WTP; and therefore, decreasing the volume of aging water the system purchases from HVUD. The total estimated cost to implement the water age recommendations listed in this document including SCADA integration is approximately $2.9M.

Infrastructure projects were recommended to address the long-term growth needs and hydraulic capacity of the distribution system. These improvements will help convey water to the southern extents of the COF service area, where a large majority of future customer demand will be located. The total cost of the three recommended system improvements is estimated at $5.9M. The project recommendations and associated costs reflect the analysis that can be conducted from the distribution system model and do not account for modeling of water ages within tanks. These recommendations are intended to increase tank turnover to reduce the overall system water age. It is possible that additional projects are needed to reduce water age in selected tanks; however, this is unknown without implementation of the recommended projects.

It is recommended that the COF conduct regular testing at the HVUD supply points to better quantify the quality of water entering their system. It is recommended to conduct testing to determine the age at which the COF and HVUD water are expected to exceed the regulatory limits on DBPs to better correlate model data with field data.

City of Franklin, Tennessee

Integrated Water Resources Plan

Appendix C:

Conservation

- Case Study: Evaluation of Toilet Replacement Conservation Program
- Water Conservation Technical Analysis: Evaluation of Available Water Conservation Strategies
- Evaluation of Potential Irrigation Ordinance Water Savings

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Technical Memorandum

To: *City of Franklin IWRP Team*

From: *CDM*

Date: *July 27, 2011*

Subject: *Integrated Water Resources Plan – Case Study - Evaluation of Toilet Replacement Conservation Program*

In most communities, toilets account for the greatest amount of indoor residential water use. Implementing a toilet rebate or toilet replacement program has shown to be a worthwhile approach to conserving water in many cities. Offering customers financial incentives to replace older, less efficient toilets with newer and more efficient models saves water with every flush. This hypothetical case study allows examination of the potential benefits (water savings) that could be achieved and costs associated with the implementation of a toilet rebate program in the City of Franklin, Tennessee. The suggested rebate program would provide City water customers with vouchers to be used for the purchase of 1.28 gallon per flush (gpf) High Efficiency Toilets (HETs).

As discussed in Section 2.0 of the Water Conservation Strategies memorandum, the 1992 Energy Act requires that all toilets manufactured after January 1, 1994 are required to be within a maximum flush volume of 1.6 gpf (Ultra Low Flush Toilets or ULFTs). Consequently, it can be assumed that all housing units built after January 1, 1994 have installed ULFTs. The 3.5 gpf toilets were introduced in the early 1980s. Homes built prior to 1980 were furnished with toilets using 5 to 7 gpf. Therefore, the primary target of an HET toilet replacement program would be installation in housing units built before 1994, because they are most likely to have older and less efficient toilets. However, some customers whose homes already have ULFTs may also participate in the rebate program. For the purposes of this analysis it was assumed that all housing units (as identified by the U.S. Census Bureau) in the City of Franklin are potential candidates for toilet rebates and replacement[1,2].

[1] Note that the number of housing units in the City of Franklin is defined by U.S. Census data and does not necessarily correspond to the precise geographical boundaries of the City's water service area.

1.0 Data and Assumptions

Determining the age of the housing stock in Franklin is necessary in order to estimate the number of homes likely to have older, less efficient toilets and the number of homes likely to have newer, more water efficient toilets. Estimating the number of housing units by toilet volume per flush type allows for an estimation of the number of toilets that may be replaced and, consequently, the potential water savings of a toilet rebate program. U.S. Census data for Franklin, Tennessee provides the number of housing units by the age of the structure. According to the 2009 U.S. Census Bureau American Community Survey (ACS) for Franklin, in 2009 there were 22,206 total housing units. 2009 is the last year of available data for age of the housing stock in Franklin and was used as the baseline for this analysis.

Approximately 65 percent of the total housing units in the City have been built in the last 20 years (1990-2009). The ACS only estimates the year a structure is built in 10-year increments (e.g., 1980-1989, 1990-1999, etc.) [3]. The 2000 decennial census provides this data in 5-year increments after 1990 (e.g., 1990-1994, 1995-1999, etc.). To better understand the number of housing units built in Franklin since 1994, year 2000 Census data for Franklin was used to provide the estimate of housing units by year built in the 1990s rather than interpolating a value from 2009 ACS data.

Table 1 shows the number of housing units in Franklin by the age of the structure. The table also shows the assumptions for the gallons per flush of toilets in the housing unit based on the age of the structure. Housing units built before 1980 are assumed to have toilets that use 5 gallons per flush. Housing units built between 1980 and 1994 are assumed to have toilets that use 3.5 gallons per flush and units built after 1994 are assumed to have toilets that use 1.6 gallons per flush.

Table 1 - Franklin, Tennessee Housing Units by Age of Structure and Assumed Gallon/Flush Toilets

Period Built	# of Units	% of Total	Gallons/Flush
1994-2009*	11,965	54%	1.6
1980-1994*	5,414	24%	3.5
Pre-1980	4,827	22%	5

Source: 2009 U.S. Census American Community Survey

*2000 U.S. Census data is used to provide the estimate of number of housing units built during the 1990s

[2] The U.S. Census Bureau defines housing units as a house, an apartment, a mobile home, a group of rooms, or a single room that is occupied (or if vacant, is intended for occupancy) as separate living quarters. Separate living quarters are those in which the occupants live and eat separately from any other persons in the building and which have direct access from the outside of the building through a common hall.

[3] Note that reporting at this level of detail from the 2010 Census has not yet been released.

2.0 Scenarios and Assumptions

An unknown factor in any voluntary water conservation program is the level of participation by customers in the water service area. Thus, for the purposes of this analysis three scenarios related to the level of participation in a toilet rebate program are evaluated, and form low, medium, and high levels of potential conservation savings. The three scenarios are:

Low Conservation Savings – Assumes that 25 percent of housing units in the City of Franklin service area purchase and install HETs as part of a rebate program.

Medium Conservation Savings – Assumes that 50 percent of the housing units in the City of Franklin service area purchase and install HETs as part of a rebate program.

High Conservation Savings – Assumes that 75 percent of the housing units in the City of Franklin service area purchase and install HETs.

Several assumptions were made in order to calculate potential water savings and program costs from a toilet rebate program. Included among the assumptions used to derive potential water savings are the number of flushes per housing unit per day and the percent of housing units that have retrofitted toilets since the implementation of the mandates of the 1992 Energy Act in 1994. Included among the assumptions used to derive the potential costs of the rebate program are the rebate amount and any administrative costs associated with operating the program. **Table 2** presents the assumptions for estimating potential water savings and program costs.

Table 2 - Baseline Toilet Rebate Program Assumptions for Franklin, TN

Factor	Baseline Assumption
Water Savings Assumptions	
# Flushes per Housing Unit per Day*	12.4
Annual rate of retrofit to 1.6 gpf since 1994	0.5%
Program Cost Assumptions	
Toilet Rebate Amount	$100/toilet
Administrative Costs (Low Conservation Savings Scenario)	$5 per rebate
Administrative Costs (Medium Conservation Savings Scenario)	$10 per rebate
Administrative Costs (High Conservation Savings Scenario)	$15 per rebate

**Residential End Uses of Water*, AWWARF 1999, p. 96

A $100 rebate per 1.28 gpf toilet is a commonly used as a rebate value. For example, rebate programs in Charlottesville, Virginia, Miami, Florida, Madison, Wisconsin, and Douglas County, Georgia have all offered $100 to customers purchasing and installing an HET. The assumed cost of program administration is different for each of the conservation savings scenarios. Administrative and marketing costs per rebate issued are assumed to increase as the rate of customer participation increases. In particular the higher participation rates will require more marketing effort. For example, a program with a 75 percent participation rate issues three times the number of rebates as a program with a 25 percent participation rate and therefore is assumed to incur administrative and marketing costs that are three times as high. While it is likely that costs associated with developing a rebate program would be similar under each conservation savings scenario, staff time and marketing materials would likely be proportional to the number of rebates issued. Thus program costs are assumed to increase proportionally to the level of participation.

3.0 Water Savings Calculations

The first step in calculating the potential savings for a toilet rebate program in Franklin is to determine how many housing units exist by gpf category. Table 1 shows the number of housing units by the period built and gpf category. Table 2 presents the assumption that since 1994, housing units in Franklin built prior to 1994 have been retrofitting older toilets with 1.6 gpf ULFTs at a rate of 0.5 percent per year (1 in 200 housing units annually). This rate of retrofitting may vary among communities but is a reasonable assumption when assessing potential savings associated with toilet replacements. The water savings from retrofits are sometimes referred to as natural replacement or "passive savings".

Table 3 shows the number of housing units by period built and gpf category both with and without the assumption of a 0.5 percent annual rate of replacement of toilets in housing units built prior to 1994. The period of annual toilet replacements analyzed is from 1994 to 2009 (i.e., 3.5 and 5.0 gpf toilets were replaced with 1.6 gpf toilets between 1994 and 2009). Assuming that 8 percent of the housing units built before 1994 replaced their 3.5 or 5 gpf toilets with ULFTs between 1994 and 2009 (0.5 percent annually multiplied by 16 years), the current (i.e., 2009) estimated number of housing units with ULFTs shown in Table 3 increases from 11,965 to 12,784, going from 54 percent to 58 percent of the total housing units in Franklin.

The number of housing units built before 1980 and the number of housing units built between 1980 and 1994 assumed to have undergone a toilet retrofit between 1994 and 2009 was calculated based on the percent of housing units in each category as a percent of total housing units in the city built before 1994. For example, if 30 percent of the housing units built before 1994 were built before 1980 it was assumed that 30 percent of the units that were retrofitted between 1994 and 2009 were built before 1980 and 70 percent were built between 1980 and 1994.

Table 3 - Franklin, Tennessee Housing Units by Age of Structure and Assumed Gallon per Flush Toilets: Comparison of with and without Natural Replacement Assumptions

No Assumption of Natural Replacement				0.5% Annual Natural Replacement, 1994-2009		
Period Built	**# of Units**	**% of Total**	**Gallons /Flush**	**# of Units**	**% of Total**	**% of Total**
1994-2009*	11,965	54%	1.6	12,784	58%	1.6
1980-1994*	5,414	24%	3.5	4,981	22%	3.5
Pre-1980	4,827	22%	5	4,441	20%	5
Total	22,206	100%		22,206	100%	

Source: 2009 U.S. Census American Community Survey
*2000 U.S. Census data is used to provide the estimate of number of housing units built during the 1990s

Table 4 shows the estimated number of households by toilet gpf category and the estimated annual water use estimated for the current mix of toilets and for the three conservation savings scenarios discussed above. *Under the estimated current mix, it is assumed that no housing units have converted to HETs.* Estimated total annual toilet water use for the assumed current mix of housing units is 272 million gallons (about 28 percent of total residential water use in Franklin[4,5]). Using the assumption of a 25 percent rate of participation in a toilet rebate program by all households in Franklin, annual toilet water use decreases by 13 percent (35.9 million gallons). Under the 50 percent market penetration scenario annual toilet water use decreases by 26 percent (71.7 million gallons), and under the 75 percent market penetration scenario annual toilet water use decreases by nearly 40 percent (108 million gallons). The estimated toilet water use by toilet volume type is shown in **Figure 1.** Note, this analysis only applies to changes within the current housing stock and does not evaluate potential savings in new construction.

[4] Total residential water use is defined as the total billed water use for single-family and multifamily residential accounts inside the City during the last calendar year of available billing data (April 2009 – March 2010).

[5] For comparison, the American Water Works Association Research Foundation estimates that toilet water use represents 26.7% of the daily residential indoor water use.

Table 4 - Number of Households by Toilet Flush Volume and Annual Water Use for Current and Market Penetration Scenarios

Conservation Savings Scenario	Toilet Type	# Housing Units by Toilet Type	Annual Water Use (MG)**
Current Mix*	1.28 gpf	0	0.0
Current Mix*	1.6 gpf	11,965	92.6
Current Mix*	3.5 gpf	5,414	78.9
Current Mix*	5.0 gpf	4,827	100.5
Current Mix Total		**22,206**	**272.0**
Low	1.28 gpf	5,552	32.2
Low	1.6 gpf	9,588	69.4
Low	3.5 gpf	3,736	59.2
Low	5.0 gpf	3,331	75.4
Low Total		**22,206**	**236.1**
Medium	1.28 gpf	11,103	64.3
Medium	1.6 gpf	6,392	46.3
Medium	3.5 gpf	2,490	39.5
Medium	5.0 gpf	2,220	50.2
Medium Total		**22,206**	**200.3**
High	1.28 gpf	16,655	96.5
High	1.6 gpf	3,196	23.1
High	3.5 gpf	1,245	19.7
High	5.0 gpf	1,110	25.1
High Total		**22,206**	**164.5**

*Current mix assumes 0.5% annual rate of replacement with 1.6 gpf toilets since 1994.

**Assumes 12.4 flushes per housing unit per day; MG is million gallons

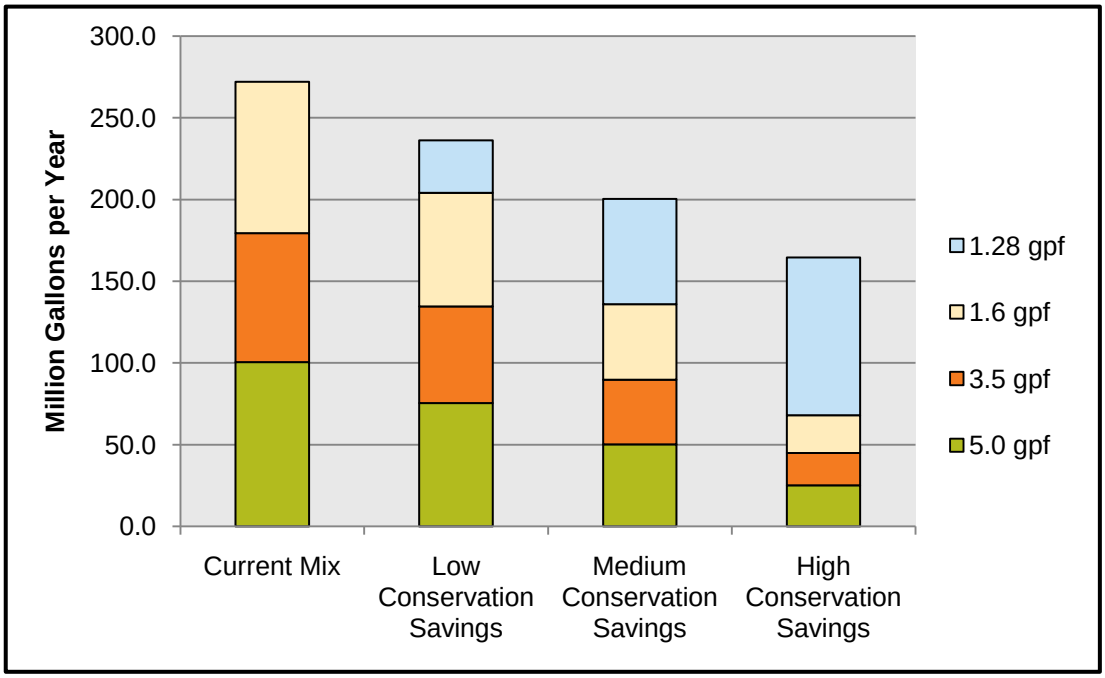

Figure 1 - Franklin, Tennessee Estimated Annual Toilet Water Use by Conservation Scenario

Table 5 provides the water savings calculations by conservation scenario, based on the water use presented in Table 4. The total savings over the current mix of toilets by housing units as well as the savings by account are shown in Table 5. The total estimate of accounts is 12,917 and is the number of residential accounts inside the city during the last month of available water billing data (March 2010). The potential savings estimates are also shown in **Figure 2**.

Table 5 - Estimated Toilet Water Use Savings

Conservation Savings Scenario	Total Annual Toilet Water Use (MG*)	Annual Savings from Current Mix (MG*)	# of Accounts	Toilet Water Use Per Account (GPD**)	% Difference from Current Mix
Current Mix (assuming retrofits)	272.0	0.0	13,026	57.2	0%
Low Conservation Savings	236.1	35.8	13,026	49.7	-13%
Medium Conservation Savings	200.3	71.7	13,026	42.1	-26%
High Conservation Savings	164.5	107.5	13,026	34.6	-40%

*MG is million gallons
**GPD is gallons per day

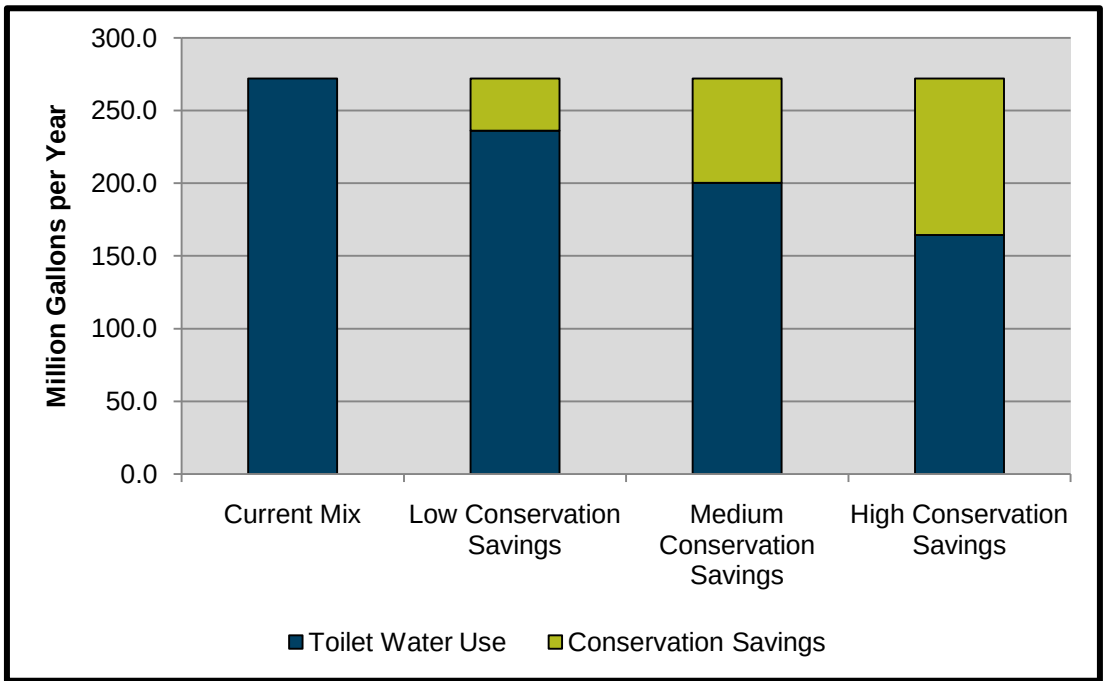

Figure 2 - Estimated Toilet Replacement Conservation Savings Comparison with Current Use

4.0 Cost Calculations

Table 2 shows program cost assumptions for this analysis. In addition to the assumptions shown in Table 2, it is necessary to make an assumption regarding the number of toilets that will be replaced per housing unit participating in the program. National 2010 Census data provides estimates of the number of bathrooms in new single-family homes built from 1973 through 2010 (U.S. Census Bureau 2011). Within the Census dataset, housing units are categorized by units with 1.5 baths or less, 2 baths, 2.5 baths, and 3 or more baths. Assuming that half of the houses categorized as 1.5 baths or less have one bathroom and half have 1.5 bathrooms and assuming that houses categorized as having 3 or more baths have 3 bathrooms, the average U.S. house built between 1973 and 2010 has 2.33 bathrooms. Therefore, for the purposes of this analysis, it is assumed that the average number of bathrooms per housing unit in Franklin is 2.33 and that the average household participating in the rebate program will replace all toilets within their home (i.e., for each participating housing unit, 2.33 rebates are issued).

Using the assumptions discussed above, the cost of a toilet rebate program under three conservation savings scenarios are presented in **Table 6**. The costs presented in the table reflect the costs of the rebates themselves ($100 per toilet) and administrative costs such as program planning and development, maintaining data records, accounting, public outreach, marketing, and education. Costs associated with installing the new toilets are not factored in to this analysis. As the table shows the most costly scenarios are those that involve providing rebates and replacing the greatest number of toilets. However, these scenarios also result in the greatest amount of potential water savings.

Conservation scenario costs are proportional to potential water savings. For example, total program costs under the High Conservation Savings scenario are 200 percent higher than the estimated costs associated with the Low Conservation Savings scenario and estimated water savings are also 200 percent higher than the Low scenario. The program cost per 1,000 gallons saved varies by about 9.5 percent between the High and Low Conservation Savings scenarios. The program cost per 1,000 gallons saved should also be calculated for the cumulative gallons saved over the life of the new fixture. With proper maintenance, a toilet fixture may last more than 20 years. For illustrative purposes, Table 6 shows the cost per 1,000 gallons saved over 10 years.

Table 6 - Estimated Toilet Rebate/Replacement Program Costs for the City of Franklin

Conservation Savings Scenario	# Housing Units Participating	# Toilets Assumed to be Replaced	Estimated Program Cost	Program Cost per 1,000 Gallons Saved	Program Cost per 1,000 Gallons Saved Over 10 Years*
Low	5,552	12,935	$1,358,202	$37.90	$3.79
Medium	11,103	25,871	$2,845,757	$39.71	$3.97
High	16,655	38,806	$4,462,665	$41.51	$4.15

*Program costs should be spread out over cumulative savings over the lifespan of the fixture.

Figures 3 and **4** show the savings and cost estimates for Low, Medium, and High Conservation Savings scenarios over one year and over 10-years, respectively. Estimated water savings are shown on the left vertical axis while the cost per 1,000 gallons saved are shown on the right axis.

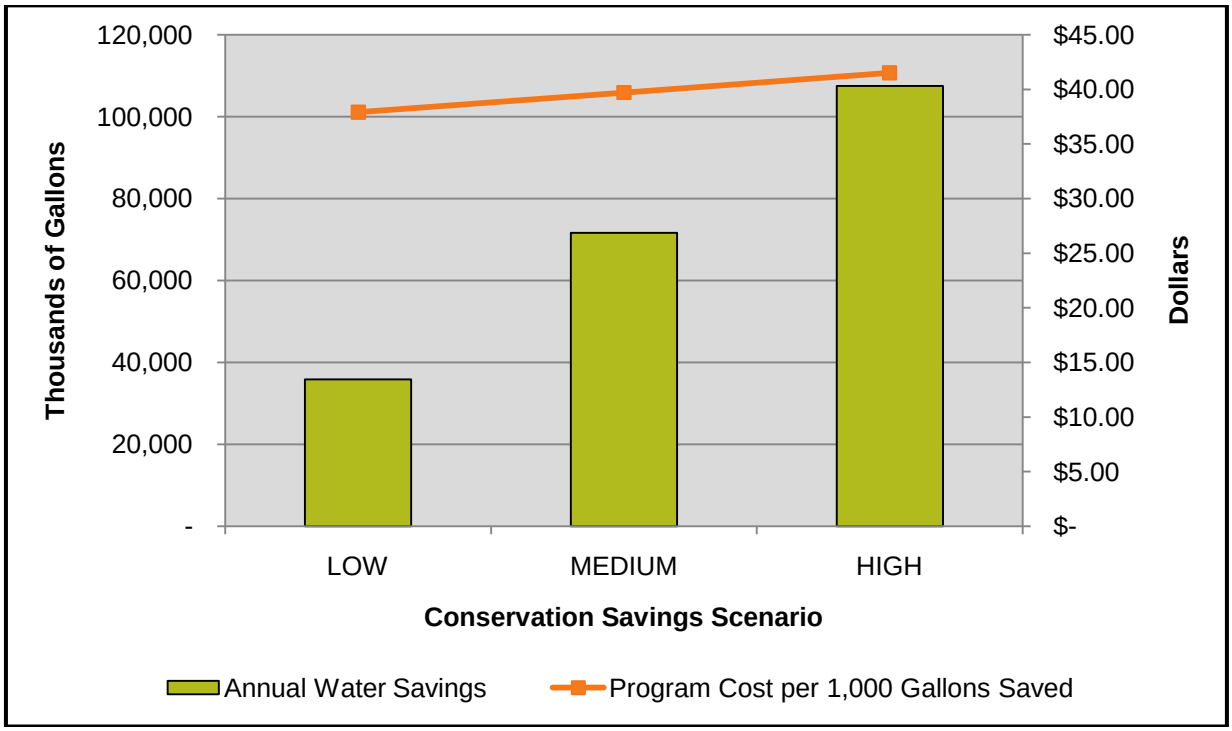

Figure 3 - Estimated Annual Water Savings and Cost per 1,000 Gallons Saved

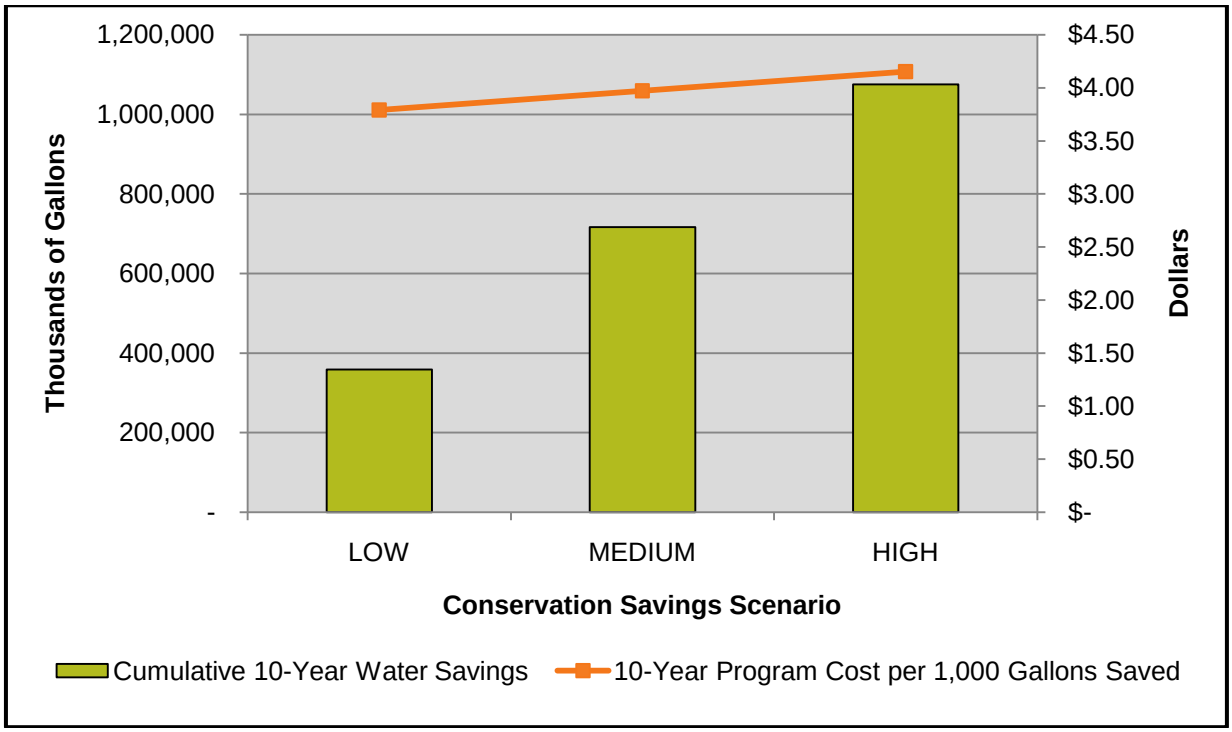

Figure 4 - Estimated 10-Year Cumulative Water Savings and Cost per 1,000 Gallons Saved

5.0 Summary and Conclusions

The case study presented in this document represents one example of a potential water conservation program that could be implemented by the City. Similar programs have been implemented in similar southeastern communities with beneficial results. However, the city-wide benefits and the costs of such a program should be evaluated in comparison with other alternatives being considered to determine the feasibility of the program. According to data provided by City staff, the cost to produce 1,000 gallons of treated water at the City's WTP is approximately $1.72. The cost to purchase 1,000 gallons of water from the Harpeth Valley Utilities District is approximately $2.55. By comparison, the toilet replacement program evaluated for the City of Franklin is estimated to cost approximately $4.00 for every 1,000 gallons saved. This suggests that the proposed toilet replacement program, while beneficial from an environmental standpoint, may be most costly than the other options being considered as a part of the Integrated Water Resources Plan. Further evaluation of the cost vs benefits of these options will be performed with the STELLA model.

6.0 References

United States Census Bureau. (2011, July 26). Number of Bathrooms in New Single-Family Houses Completed. Retrieved from: http://www.census.gov/const/C25Ann/sftotalbaths.pdf.

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Technical Memorandum

To: *City of Franklin IWRP Team*

From: *CDM*

Date: *July 27, 2011*

Subject: *Integrated Water Resources Plan – Water Conservation Technical Analysis - Evaluation of Available Water Conservation Strategies*

Realizing desired savings through water conservation measures requires understanding the options available and how they relate to the planning goals and unique characteristics of the utility. This memorandum provides a discussion of components of a an effective water conservation program as well as common water conservation program options available to to Franklin, Tennessee. In addition, costs and savings realized by other agencies and utilities that have implemented these programs will be presented where possible to illustrate economic impacts of particular water conservation program options.

Integrating water conservation goals and programs into utility water planning is emerging as a priority for communities outside of the traditional water-short regions of the western United States (US) Catalysts for implementing water conservation programs include growing competition for limited supplies, increasing costs and difficulties with developing new supplies, increasing demands that stress existing infrastructure, and growing public support for resource protection and environmental stewardship. Many utilities are also beginning to understand the value of water conservation as a way of saving on costs both to the utility and to its customers. Throughout the US, utilities have experienced quantifiable benefits associated with long-term water conservation programs including:

- Reduction in operation and maintenance costs resulting from lower use of energy for pumping and less chemical use in treatment and disposal;
- Reduced purchases from wholesalers;
- Delaying of capital facilities projects.

1.0 Effective Water Conservation

Selecting the appropriate conservation program includes understanding water use habits of customers, service area demographics, and the water efficiency goals of the utility, among other considerations.

Water conservation has been defined as "any beneficial reduction in water use or in water losses" (Baumann, Boland, and Sims 1984). Consequently, one component of an effective water conservation program is that reduction in water use or water losses produces a net benefit (i.e., the value of savings are greater than the costs required to achieve the savings). Another component of an effective program is that the unit cost of water saved is less than the unit cost of additional new supply. Additionally, an effective water conservation program meets the goals set by the implementing entity. There are a variety of metrics that may determine whether goals have been met and approaches to program evaluation fall into three general categories: 1) process evaluation, 2) impact evaluation, and 3) economic evaluation. The following paragraphs will briefly describe each of these categories of conservation effectiveness evaluation.

Process evaluation is a term applied to tracking and measuring the operational efficiency of a water conservation program. This method of evaluation typically uses tools such as customer satisfaction surveys, fixture retention surveys, and market penetration surveys to gather data on participant attitudes related to their satisfaction with both the water saving devices or behaviors they have adopted and a program's administrative process. This process evaluates program implementation methods and determines whether or not a water conservation program has been successfully and efficiently implemented.

Impact evaluation quantifiably examines water savings that are clearly attributed to a specific water conservation program. Undertaking this method of effectiveness evaluation requires the collection and analysis of water use data over a period of time. Sometimes the water use data is combined with participant level information such as household characteristics, types of water using fixtures, and water use behaviors. Results of impact evaluation indicate the volume of water saved by a specific water conservation program.

Economic evaluation often focuses on the unit cost of water saved (e.g., dollars per gallon per day saved). The cost of water conservation programs varies widely. From the utility perspective, program costs are mostly related to the design and implementation of the program. Cost components of program design and implementation to consider include staff time, rebates and incentives given to customers, length the program, etc. There may also be costs to the utility such as lost revenue from decreased sales and benefits such as reduced pumping and treatment costs.

Cost-benefit analysis of water conservation programs provides an evaluation of an array of options related to water conservation and classifies these items as costs or benefits, depending upon the perspective. For example, a toilet rebate is a cost item to the utility, but counts as a

benefit item when calculating the cost of a toilet rebate program from the customer's perspective. A cost-benefit analysis typically evaluates the cost of water conservation programs from different perspectives: utility, customer, rate-payer, and society. Thus, program costs may vary by the perspective used to evaluate a conservation program.

Although two separate utilities may spend the same dollar amount on similar water conservation programs, the participation rates and water savings achieved by the two utilities may differ. As a result, the dollar value of a program may vary by provider based on the customer response to a particular program. Furthermore, separate utilities may face different circumstances such as the cost of source water and feasible alternatives such that a conservation program may be cost-effective for one utility but not another. The following section describes many of the common water conservation programs that have been implemented in different parts of the country and provides cost and savings estimates obtained from case studies of water providers where possible.

2.0 Program Descriptions

Hardware and Rebates

Many utilities have realized significant water savings by implementing programs that offer economic incentives such as rebates to customers for converting older and less efficient water fixtures such as toilets, showerheads, faucets, and clothes washers to newer more efficient models. There is usually a high start-up cost to utilities to initiate these programs ($25,000 to $100,000); making the cost-benefit analysis of such a strategy an important consideration (Alliance for Water Efficiency 2011b).

Some water agencies have also invested in device distribution programs where a water efficient device that can be easily installed (e.g., low-flow showerheads, flow restrictors, low-flow aerators for faucets) are provided to customers. Device distribution has been most commonly used in drought or supply crisis situations and is often targeted to customers, sectors, or areas that present the most cost-effective option for implementation such as areas with older housing, low-income communities, and commercial restaurants and kitchens.

Toilets

Following the passage and implementation of the 1992 U.S. Energy Policy Act (Energy Act), all new and remodeled homes in the U.S. are mandated to install toilets with a maximum flush rate of 1.6 gallons per flush. These toilets are commonly referred to as Ultra Low Flush Toilets or ULFTs. Replacement of less efficient toilets with ULFTs has proven to be a beneficial method of water conservation to both the residential and non-residential water use sectors. For example, in Tampa, Florida, the Tampa Water Department (TWD) offered rebates as high as $100 for replacement of toilets in single-family and multi-family homes, as well as for commercial customers. A 38 gallons per capita per day (gpcd) reduction in water use between 1993 and

2008 has been attributed to the toilet rebate program (EPA 2011a). The cost of the rebates awarded by the TWD from 1993 to 2008 is estimated at about $3.3 million with over 37,000 toilets replaced during that period (City of Tampa 2011).

Urinals

The 1992 Energy Act included mandates for urinals installed in new and remodeled buildings to use 1 gallon per flush (gpf) or less. Older urinals installed prior to the implementation of the 1992 Energy Act typically use between 1.5 and 5 gpf (Vickers 2001). More recently introduced High Efficiency Urinals (HEUs) use 0.5 gpf or less. Non-flush urinals, which use a dry drainage system, are also on the market. Savings from urinal replacement would be predominately realized by commercial and industrial customers. The EPA WaterSense® program estimates that the potential water savings of replacing a 1.5 gpf urinal with a 0.5 gpf model could be more than 4,600 gallons per year per urinal at a cost savings of more than $850 over the useful life of the fixture (EPA 2011b).[1]

Showers and Faucets

Showers account for nearly 17 percent of indoor residential water use, or about 30 gallons per household per day (EPA 2011d). The 1992 Energy Act requires that all showerheads installed in new and remodeled buildings have flow rates of 2.5 gallons per minute (gpm) or less. Showerheads manufactured prior to 1980 had an average flow rate of 4.3 gpm while models designed from 1980-1994 had average flow rates of 3.0 gpm. Efficient models on the market today have typical flow rates of 2.0 gpm. Research by Biermayer (2006) estimates that replacing a 4.0 gpm showerhead with a 2.2 gpm showerhead would result in savings of 14.8 gallons per shower.[2]

Kitchen and bath faucets are the next largest source of indoor residential water use behind showers, accounting for approximately 15.7 percent of use (Mayer et al. 1999). The 1992 Energy Act requires 2.5 gpm flow rates for all newly installed faucets. Recently developed high efficiency faucets have reduced flow rates of 1.0 to 2.2 gpm.

Case studies of utilities that have successfully implemented showerhead and faucet replacement programs have been documented. A combination showerhead and toilet replacement program in Ashland, Oregon contributed to a 16 percent reduction in winter water usage. The program is also attributed with saving an estimated 514,000 kilowatt-hours of electricity annually, mostly through reduced water heating energy use (EPA 2011a).

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[1] Savings estimated based on the assumption that the average urinal is flushed 18 times per day and is in use 260 days per year.
[2] Assuming an estimated national average shower time of 8.2 minutes.

Device distribution programs have been implemented in several U.S. cities. In Houston more than 10,000 high efficiency showerheads have been distributed as part of conservation kits. The conservation kits, which also included faucet aerators, were estimated to produce average water savings of 18 percent per household (EPA 2011a). In Goleta, California, a city of just over 50,000 people, 35,000 low-flow showerheads were installed between 1987 and 1991. Goleta's water efficiency program, which emphasized showerhead replacement and toilet rebates, resulted in a 50 percent reduction in per capita residential water use (EPA 2011a).

Dishwashers

According to 2005 statistics from the U.S. Census Bureau, 63% of households nationwide utilize a dishwashing machine. The average household uses a dishwasher 3 days per week, leading to a substantial amount of potential water savings from water efficient models. Dishwashers manufactured during 1980-1990 used approximately 14 gallons per load (gpl), while models manufactured in 1990-1995 reduced this amount by 3 gallons to 11 gpl. While the 1992 Energy Act did not mandate restrictions on dishwashers, more efficient models continue to be developed with recent models using only 7.0 gpl, as much as a 50% reduction in water use from earlier models. While water savings from dishwashers is significant, this end use comprises one of the smallest portions of indoor water use. Simple education programs, aimed at behavioral changes such as operating the dishwasher with full loads only, are also an important part of enhancing water efficiency for dishwashers.

Non-residential dishwashers use significantly more water than residential versions, often using over two-thirds of overall water use in commercial kitchens (Alliance for Water Efficiency, 2011a). Water usage among these units generally ranges from 0.33 gallons per rack (gpr) to over 20 gpr. Additionally, energy usage by these units is exceptionally high given the amount of energy required to heat water to the required temperature. The life of these machines is generally longer than residential models as well, averaging 20-25 years. Advancements in water efficient technologies, combined with this extended life span, leads to a substantial amount of potential water savings.

Pre-Rinse Spray Valves

Pre-rinse spray valves are handheld devices used in many commercial kitchens and restaurants designed to loosen food and debris from dishes prior to placing them into dishwashers. Dishwashing represents about two-thirds of all the water used in a typical restaurant, with nearly one-half of that water used to rinse dishes before the actual washing. In addition, significant energy is required to heat the rinse water. Most pre-rinse spray valves use 3 to 7 gpm depending on the age of the device. By contrast, low-flow pre-rinse spray valves use 1.6 gpm or less. The 2005 Energy Policy Act requires that pre-rinse spray valves installed in new or remodeled commercial kitchens use these more efficient low-flow spray valves.

Potential water and economic savings will vary by facility based upon patterns of use. A California Urban Water Conservation Council (CUWCC) pilot study estimates water savings of 100 to 300 gpd per valve head when converting from a less efficient to a more efficient device, depending on the size of the establishment. The same study estimates average energy savings as 0.92 therms per head per day for heads with gas water heating and 20.9 kilowatt-hours per head per day for heads with electric water heating (SBW Consulting, Inc. 2004).[3]

Clothes Washers

Significant advances in clothes washer water use efficiency have been made in the past few decades. Models manufactured prior to 1980 typically used about 56 gallons per load (gpl) while those produced between 1980 and 1990 used approximately 51 gpl. More modern top-loading models being produced use about 43 gpl (about 16 percent less water per load than older models). Front-loading high efficiency washers (HEWs) use an average of 27 gpl, require less energy than standard top-loading washers, significantly reduce drying time, and require less detergent.

According to 2005 U.S. Census Bureau statistics, 82 percent of households have a clothes washer. Mayer et al. (1999) found that the average household washes about seven loads of laundry per week. Based on these statistics, the typical household could save an average of 5,824 gallons of water per year by switching from a 43 gpl top-loading washer to a front-loading HEW[4].

A pilot program in Toronto, Ontario, Canada installed high efficiency front-loading washers at six different apartment buildings and tracked hot and cold water use for eight weeks. Results showed a 44 percent reduction in total water consumption with the front-load washers and a 61 percent decrease in hot water consumption. A customer satisfaction evaluation conducted after the eight week period showed very strong satisfaction ratings with two-thirds of tenants stating they were very satisfied (CMHC 2002).

3.0 Irrigation Technologies and Ordinances

Rain Sensors & Weather-based Controllers

The EPA estimates that nationwide about 30 percent of residential water use is for outdoor uses (EPA 2011c). This estimate can vary significantly depending on the particular climatic characteristics of an area. Of this 30 percent, it is estimated that 50 percent is wasted due to poor irrigation practices such as overwatering, improper system design, evaporation, and wind (The Saving Water Partnership 2003).

Traditional irrigation systems use a clock timer with a preset schedule. In contrast, weather-based irrigation control technology uses local weather and landscape conditions to modify

[3] Assumes gas efficiency of 70% and electric efficiency of 90%.
[4] Assumes an average savings of 16 gpl, 7 times per week for 52 weeks.

irrigation schedules to actual conditions on the site allowing irrigation to more closely match the water requirements of plants. Rain sensors are one example of this technology. These sensors save water by eliminating unnecessary watering once there has been sufficient rainfall.

The U.S. Bureau of Reclamation compiled a summary of available research on "smart" irrigation control devices. Literature suggests that typical residential savings following installation of such devices range from about 15 to 25 percent, however, it is important to note that the majority of the studies compiled are reporting observations from programs in the western U.S. (USBR 2008). Residential weather-based irrigation controllers cost $100-250 on average depending on the specific features. Commercial weather-based irrigation controllers can cost several thousand dollars. Vickers (2001) estimates that rain sensors can save 5 to 10 percent of the outdoor water used. Rain sensors have proven to be a cost effective method of saving water. Average prices are $15 to $50 per sensor with the benefits of increased water efficiency and lower water bills quickly outweighing the cost of the device.

Rain sensor giveaway programs and installation ordinances have been implemented in several cities including Austin, Texas and St. Petersburg, Florida. Follow-up on these efforts revealed that rain sensor giveaways were more successful and widely adopted when installation was offered in addition to the free devices.

Cary, North Carolina (a mid-sized utility system) instituted an ordinance in 1997 requiring rain sensors on all automatic irrigation systems. The purpose of the ordinance was to decrease water usage during peak demand months. The City reports that 80 percent of residential customers and nearly 100 percent of commercial customers are in compliance with the ordinance. Other irrigation-related ordinances that have been adopted by Cary include a year-round alternate day irrigation schedule and a water waste ordinance. It is estimated that the landscape/irrigation codes in Cary will save between 0.02 and 0.04 million gallons per day (mgd) between 2009 and 2019 at a unit cost of water saved of $276 per mgd.

Education, Information & Awareness

Customer education (such as the City of Franklin's Water Hog program) is a critical component of any water conservation program. However, determining the direct effect on water use from education, information, and awareness efforts can be difficult because they are often bundled with other conservation programs and tend to vary by region. While water savings directly attributable to education are difficult to estimate, providing information that could change behaviors and water use habits can produce considerable savings. Baumann et al. (1998) and the EPA (1998) have found that public awareness campaigns can be expected to reduce demand by 2 to 5 percent.

Actual reported unit cost of water saved varies considerably. In one study of the costs and benefits of various conservation programs to water suppliers in the Portland, Oregon

metropolitan area, the unit cost of information, education, and awareness programs to the utilities and society were $122 per acre-foot saved ($374 per million gallons) (RWPC 2004). Platt and Delforge (2001) estimate a unit cost of water saved through public education in Cary, North Carolina to be $401 per mgd and a benefit cost ratio of 1.53. The Cary, North Carolina example has a goal of reducing per capita consumption by 20 percent by 2020.

4.0 Audits & Accountability Measures

Submetering

Submetering is a useful tool that targets new and/or existing multi-family accounts through ordinances or regulations requiring submeters for individual units. In many cases, multi-family water use is billed to the owner of the complex based on the reading of one master meter. Tenants of the complex pay a flat rate for water use, generally factored into their rent. As a result, the renter receives no direct price signal associated with their water use and has no economic incentive to reduce excess use. A nationwide survey conducted as part of the National Multi-family Submetering and Allocation Billing Program Study in 2004 found that over 85 percent of multi-family residents pay for water use as part of their rent (Mayer et al. 2004). This same study found that submetering of multi-family complexes can reduce water use by 15.3 percent compared to properties that bill water use as part of rent. Many submetering service companies claim water savings of 10 to 30 percent, or more (AWWA 2000).

The costs of implementing a submetering program are varied depending on the level of implementation. Because costs of installation are generally lowest during construction, ordinances requiring submetering only on newly constructed multi-family dwellings tend to be the least costly. The average cost of a submeter system installed during construction is $200 while retrofitting existing properties have an average cost of $250 per unit (AWWA 2000). Costs of installing submeters on existing properties can be the responsibility of the property owner; however, incentive programs implemented by utilities will likely accelerate this process.

Residential and Commercial, Industrial, and Institutional Audits

The purpose of a water audit is to assess water use practices and identify methods to improve water efficiency. Residential audits generally include measurements of fixture flow rates, evaluation of irrigation systems and landscape, and leak detection. Commercial, Industrial, and Institutional (CII) audits are often related to engineering modifications of site-specific water uses such as process water use and cooling use. Water efficiency improvements for CII customers can also include replacement of plumbing fixtures and leak reduction.

Residential water audits can be a precursor to larger, more focused water conservation programs. For example, the city of Ashland, Oregon conducted water audits of almost 1,900 residents from 1992 to 2001, and following the audits, nearly 85 percent of homes audited

participated in the city's showerhead and/or toilet replacement program (EPA 2011a). Vickers (2001) found that an audit's typical potential demand reduction ranges from 15 to 35 percent.

Costs can be covered either by the provider, or by the customer receiving the audit. Average costs for a CII audit are $1,000 while residential audits cost significantly less at an average of $75 per audit (Davis and Christiansen 2003). The Cary, North Carolina residential water audit program was reported to have a unit cost of water saved of $547 with a benefit cost ratio of 1.13 (Platt and Delforge 2001).

5.0 Conservation Rate Structures

Increasing Block Rates

An increasing block rate structure, similar to the City's current rate structure, is an advanced method used to allocate costs by the quantity of water used. The concept is meant to be a catalyst for customers to implement voluntary water conservation practices to reduce water use and, as a result, reduce their overall cost for water. The most effective rate structure is one in which the cost per unit of water increases as the customer uses more water. This approach to rates helps water providers achieve the goal of reduced daily peak and seasonal peak usage and overall reduced system demand. Usually, 3 to 4 tiers are adequate for an effective residential rate system.

The savings associated with conservation rates are dependent on customers' responses to increased utility bills. City staff has suggested that the current rate structure has a relatively minor impact on excessive use due to the affluent nature of the customer base. Therefore, a more aggressive rate structure may be necessary to initiate a change in use.

The Irvine Ranch Water District (IRWD) in southern California is an example of a successful and aggressive conservation rate structure program. The IRWD calculates rates for each account based on landscape square footage, number of residents, any additional needs of individual customers, and daily evapotranspiration rates. IRWD observed nearly immediate impacts following implementing their new rate structure in 1991. Water use declined 19 percent in 1991/1992 compared to 1990/1991 with high customer satisfaction ratings (EPA 2011a). This case may be the exception, however, as one study by Little and Gallup (2011) of three utility systems only identified one case where the quantified benefits exceeded the quantified costs from both the utility and participant perspective over the 20 year assumed lifespan of the rates.

Utilities considering a pricing structure that reduces demand should also consider the potential for a decrease in revenue. A nationwide survey of 23 utilities that have implemented conservation rates by Wang et al. (2005) found that only 9 percent reported increased revenues, 26 percent reported decreased revenues, 30 percent believed conservation rates were revenue neutral, and 35 percent did not know or did not respond.

Seasonal Rates

Seasonal water rates are a pricing structure whereby the cost of water per unit is higher during peak seasonal use times, typically summer months, to encourage efficient outdoor water use. The rates should be designed to send a price signal to customers to reduce excess water use and become more conscious of lawn and landscaping water use. The structure can be applied to both residential and non-residential water accounts.

Seasonal water use costs very little for a utility to implement. The primary cost is dedicated to staff time and effort to design an appropriate rate structure. Successful seasonal rate structure programs have been implemented in Phoenix, Arizona as well as Seattle, Washington, where the program is credited as having saved 5 mgd since 1990. A case study analysis from Sandy City, Utah found that two years after the implementation of a seasonal rate structure, savings were 9.1 percent of the pre-seasonal rate water use and average annual savings were about 2.5 mgd for a service area with a population of 100,000.

6.0 Summary and Conclusion

This memorandum provides a high-level discussion of various water conservation program options available to utilities and appropriate for consideration in the City of Franklin. Selecting the appropriate suite of water conservation activities requires understanding a utility's goals for conservation and the demographics and water use characteristics of the service area, among many other considerations. Case study findings have provided examples of both successful and unsuccessful programs throughout the country proving that what works in one location may not necessarily work in another.

This memorandum also discusses three post-implementation approaches to evaluating the effectiveness of a conservation program: 1) process evaluation, 2) impact evaluation, and 3) economic evaluation. These methods of evaluation provide a metric for utilities to cross-sectionally determine whether or not a particular conservation program is effective. A program's effectiveness can be measured quantitatively by calculating water saved, customer participation rates, or the costs vs. the benefits of the program. Programs can also be evaluated qualitatively with surveys of customer satisfaction.

While many of the case studies discussed offer examples from the western U.S., successful and progressive water conservation programs have been implemented in the eastern portion of the country as well. Cities such as Cary, North Carolina and Tampa, Florida have experienced positive results through the implementation of conservation programs tailored to their particular goals and objectives. These examples provide evidence of the broader geographical relevance of water conservation as a useful tool in water demand management.

As a supplement to this document, CDM has prepared a case study specific to the City of Franklin that discusses the potential costs and benefits of a toilet replacement/rebate program. The case

study calculates the approximate cost of the program per 1,000 gallons of water saved. Information such as this can be used within the STELLA model to compare against the cost/benefit ratios of other alternatives being considered (such as Water Treatment Plan expansion or new water purchases from the Harpeth Valley Utility District). This will help the City and the Project Team make the best decisions among the options available. Additional water conservation case studies will follow, including the benefits of an Irrigation Control Program.

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Technical Memorandum

To: *City of Franklin IWRP Team*

From: *CDM*

Date: *September 27, 2011*

Subject: *Integrated Water Resources Plan - Evaluation of Potential Irrigation Ordinance Water Savings*

Executive Summary

This analysis examines the potential outdoor irrigation water savings in Franklin, Tennessee achieved through the implementation of a water service area ordinance requiring "smart" controllers on all automatic irrigation systems for single-family residential and commercial customers. Water savings are estimated under three savings scenarios based on assumptions developed for the current automatic irrigation system penetration rate, customer compliance rate, and level of savings. Savings are presented for low, medium, and high scenarios.

The potential savings estimated based on the scenario assumptions indicate that implementing a smart controller ordinance in Franklin could reduce single-family residential and commercial peak season (i.e., May through November) outdoor water demands by as much as 15 percent and as little as 1.5 percent depending on the savings scenario evaluated. The estimate of the cost per unit of water saved ranges from $0.15 per 1,000 gallons saved under the high savings scenario to $1.45 per 1,000 gallons saved under the low savings scenario. Under each of the savings scenarios the cost of implementing the ordinance and achieving the estimated potential savings is less than the cost of acquiring water through either of Franklin's two water supply options; direct withdrawals from the Harpeth River or purchasing water from Harpeth Valley Utilities. Therefore, the results of this analysis show, based on the assumptions presented in this memorandum, that an irrigation control ordinance is a cost-effective method of conserving water in Franklin.

1.0 Introduction

This memorandum presents the results of an analysis estimating the potential water savings and program costs associated with implementing an irrigation ordinance for single-family residential and commercial water customers in the Franklin, Tennessee water service area. The process

represents a high-level planning approach designed to provide a range of potential future water demand savings associated with implementing and enforcing a system-wide irrigation control policy. The City of Franklin's *Draft Resolution Authorizing the Adoption of an Irrigation Ordinance for Potable and Reuse/Reclaimed Water* (*Draft Resolution*) serves as the basis for the types of activities evaluated. The draft ordinance includes requirements for irrigation system design and installation. For the purposes of this analysis, only the potential savings associated with requiring "smart" automatic irrigation controllers on all new and existing irrigation systems will be evaluated[1]. Other ordinance stipulations would likely contribute to potential outdoor water use savings, however, data limitations, including the absence of metered outdoor water use for many customers as well as a lack of an understanding of the current level of irrigation system penetration and customer attitudes toward different irrigation system types and methods, including graywater systems and drip irrigation systems, prevent an informed analysis of those requirements.

The approach to estimating potential water savings from irrigation ordinances first requires an estimate of the volume of water used for outdoor irrigation that would be targeted by the ordinance. Then a range of assumptions can be made regarding market penetration, enforcement, compliance, and other factors that influence the effectiveness of the ordinance. Finally, the costs and savings associated with the range of estimates can be developed.

2.0 Estimating Outdoor Water Use

Total annual billed residential water demands in the Franklin service area are about 3.2 million gallons per day (mgd) with the highest demands typically occurring in August and September. According to recent water billing data in Franklin, there are nearly 13,000 total residential water accounts inside the city and about 2,300 residential water accounts outside of the city limits. Franklin also has approximately 60 single-family residential metered irrigation accounts inside the city and about 70 residential metered irrigation accounts outside of the city. The annual billed residential irrigation demands in the Franklin service area are about 0.08 mgd, with the highest demands typically occurring in August and September.

There are nearly 1,000 total commercial water accounts inside the city and about 60 commercial water accounts outside the city limits. Total annual billed commercial water demands in Franklin are about 1.00 mgd with the highest demand typically occurring from August through October. Franklin also has approximately 90 commercial metered irrigation accounts inside the city and one commercial metered irrigation account outside the city limits. The annual billed commercial irrigation demands in the Franklin service area are about 0.12 mgd, with the highest demands typically occurring in July through September.

[1] "Smart" automatic irrigation system controllers have the purpose of eliminating wasteful irrigation water use by being equipped with sensors that measure humidity and/or soil moisture content and utilize local weather data to limit irrigation water use to optimal times and conditions.

The average (fiscal years 2000 to 2009) annual pattern of billed water demand in Franklin for both the single-family residential and commercial customer classes are shown in **Figure 1**. The non-irrigation accounts' average billed use is also depicted in Figure 1 and these data are referenced to the left vertical axis while the irrigation accounts' average billed use are depicted using the right vertical axis. The data in Figure 1 shows that average single-family residential and commercial non-irrigation billed demand peaks annually in September. The irrigation accounts' average billed water use peaks in August for both customer classes. Metered irrigation demands are relatively minor compared to demand associated with non-irrigation accounts. Average annual irrigation demands are about 3 percent of average annual non-irrigation demands for the single-family residential customer class and about 11 percent of total average non-irrigation demands for the commercial customer class.

The demand patterns shown in Figure 1 indicate that each sector's water use increases during summer months. June through November typically experiences the highest single-family residential and commercial water demands in the Franklin service area. Historical weather patterns in Franklin may help to explain the pattern in water demand pattern.

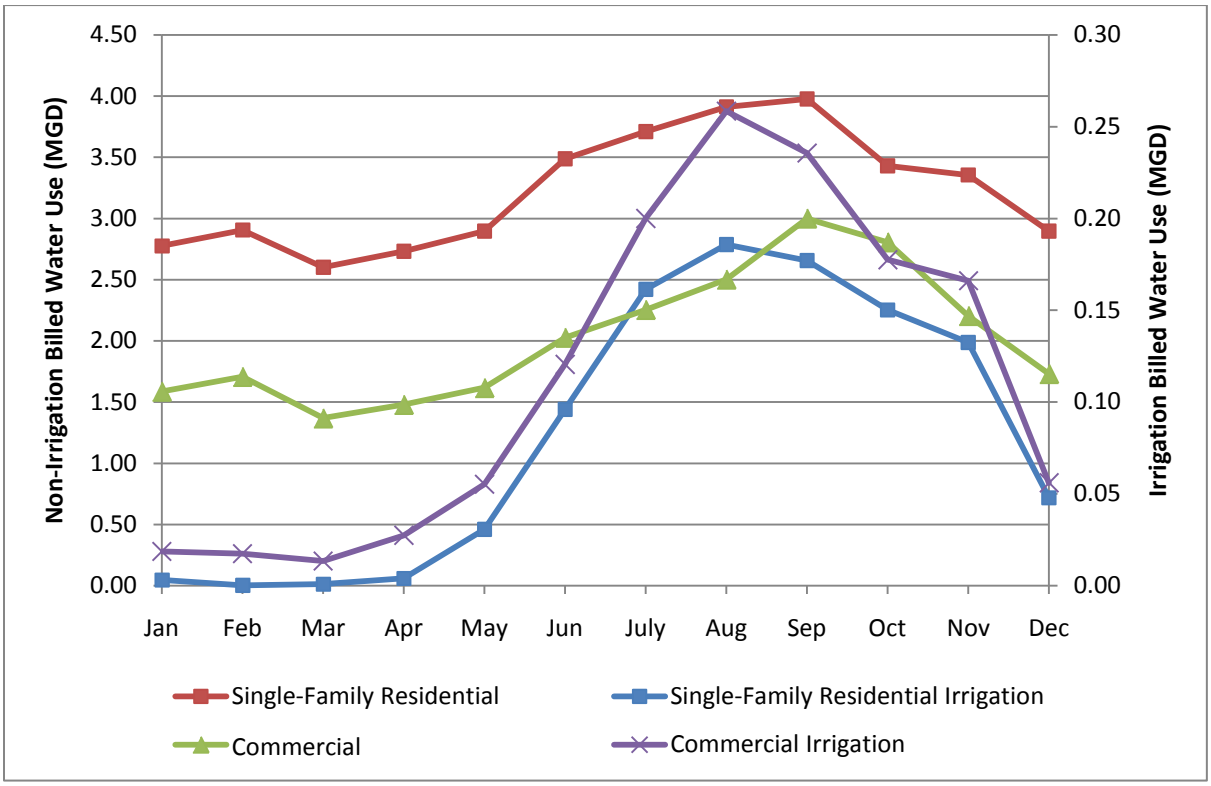


Figure 1 - Single-Family Residential and Commercial Demand Patterns: Fiscal Year 2000 through Fiscal Year 2009

Franklin receives an annual average of approximately 54 inches of precipitation, with March being the average wettest month and October being the average driest month. The average monthly precipitation and average daily high temperatures by month observed in Franklin are shown in **Figure 2,** which also shows June through October being the driest part of the year. This period also corresponds to the time of year when single-family residential water demand is highest. Therefore, the majority of the difference between the peak water demand season (i.e., May through November) and off-peak water demand season (i.e., December through April) is likely attributable to outdoor water use, when normal precipitation is at its lowest.

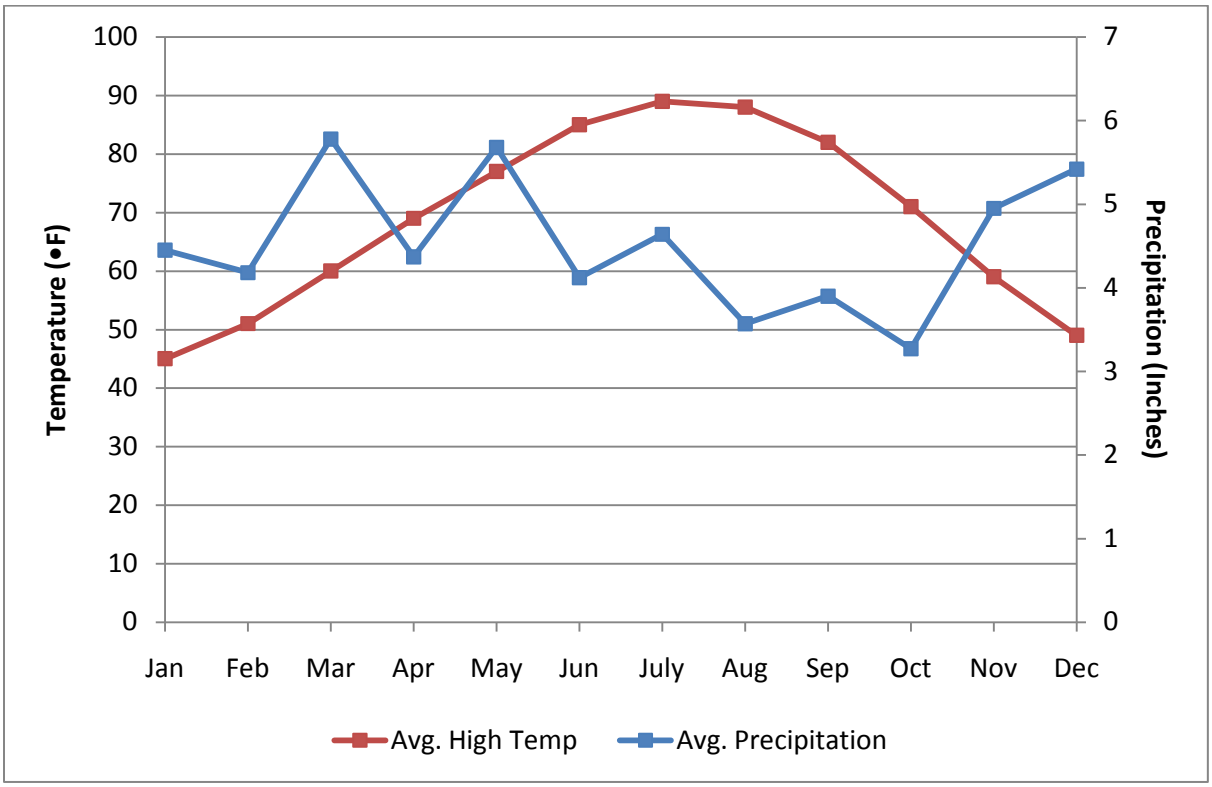


Figure 2 - Franklin, Tennessee Average Monthly Precipitation and High Temperature

The average historical peak season and off-peak season monthly residential demands in Franklin are provided in **Table 1**. The average seasonal monthly demands are the average of the historical monthly billed demand for all months in each particular season, with each month's demand weighted by the number of accounts active in each particular year. The average historical peak season and off-peak season monthly commercial demands in Franklin are provided in **Table 2**.

The irrigation accounts are expected to have little or no demand in the off-peak season and nearly all of the water use during the peak season. The standard (non-irrigation) accounts show a pattern

of normal water use, often associated with indoor water use, during the off-peak season. Thus, the difference between peak and off-peak water use may be associated with summer irrigation.

Table 1 - Average Single-Family Residential Historical Demand by Season

Metered Residential				Metered Residential Irrigation		
Month	**Avg. Residential Billed Demand, in mgd**	**Difference in Demand, in mgd**	**Avg. % Difference, Peak vs. Off-Peak**	**Avg. Residential Billed Demand, in mgd**	**Difference in Demand, in mgd**	**Avg. % Difference, Peak vs. Off-Peak**
Avg. Off-Peak Season Month (Dec-April)	2.78	0.76	27.4%	0.01	0.12	1200%
Avg. Peak Season Month (May -Nov)	3.54			0.13		

Table 2 - Average Commercial Historical Demand by Season

Metered Commercial				Metered Commercial Irrigation		
Month	**Avg. Commercial Billed Demand, in mgd**	**Difference in Demand, in mgd**	**Avg. % Difference , Peak vs. Off-Peak**	**Avg. Commercial Billed Demand, in mgd**	**Difference in Demand, in mgd**	**Avg. % Difference, Peak vs. Off-Peak**
Avg. Off-Peak Season Month (Dec-May)	0.86	0.23	26.7%	0.03	0.14	466.7%
Avg. Peak Season Month (Jun-Nov)	1.09			0.17		

The following section provides a discussion of the method used to determine the portion of peak season water demand attributed to outdoor water use, and thus, the water use from which potential savings can be achieved.

3.0 Minimum Month Methodology

The methodology to calculate minimum month outdoor water use is based on the premise that during wet months outdoor water use is minimal and during dry months outdoor water use is at its peak. For any given year or years, the average billed water use for each customer classification is determined for each month. Resultant values are plotted to determine the month with the lowest average consumption per customer, or the period when the majority of a customer's bill is for indoor water use. This lowest value is the minimum and represents the minimum consumption month. This method assumes that a percentage of the consumption may include outdoor water use regardless of whether or not the month is wet. **Table 3** provides the assumed percentage of outdoor water use during the minimum water consumption month, for the single-family residential and commercial customer classes. While it is likely that outdoor water use is zero in the minimum month, a value of 5 percent is assumed in this analysis to error on the conservative side. These percentages can readily be adjusted for each customer class.

Table 3 - Outdoor Water Use Assumptions for Minimum Month

Billing Classification	Percent of Outdoor Water Use
Single-Family Residential	5%
Commercial	5%

Using historical average water use, as described in Section 2.0 and the outdoor water use assumptions for the minimum month in Table 3, the minimum month analysis can be performed for the single-family residential and commercial customer classes in the Franklin, Tennessee service area. For this methodology, billing data beginning in July 2000 and ending in March 2010 for single-family residential and commercial customer classes were used in conjunction with outdoor water use assumptions for the minimum month. **Figure 3** shows results for the single-family sector, and **Figure 4** shows results for the commercial sector.

In Figure 3, data represent billed single-family residential consumption for non-irrigation accounts. During March, the minimum consumption month, 0.13 mgd or 5 percent of the average water consumed during that month is attributed to outdoor water use. For the remainder of the year, outdoor water use fluctuates month-to-month and is represented by any billed water use greater than 2.47 mgd (estimated minimum month indoor water use), as indicated by the green shaded area in Figure 3. Indoor water use is assumed to remain constant at 2.47 mgd, as indicated by the blue shaded area.

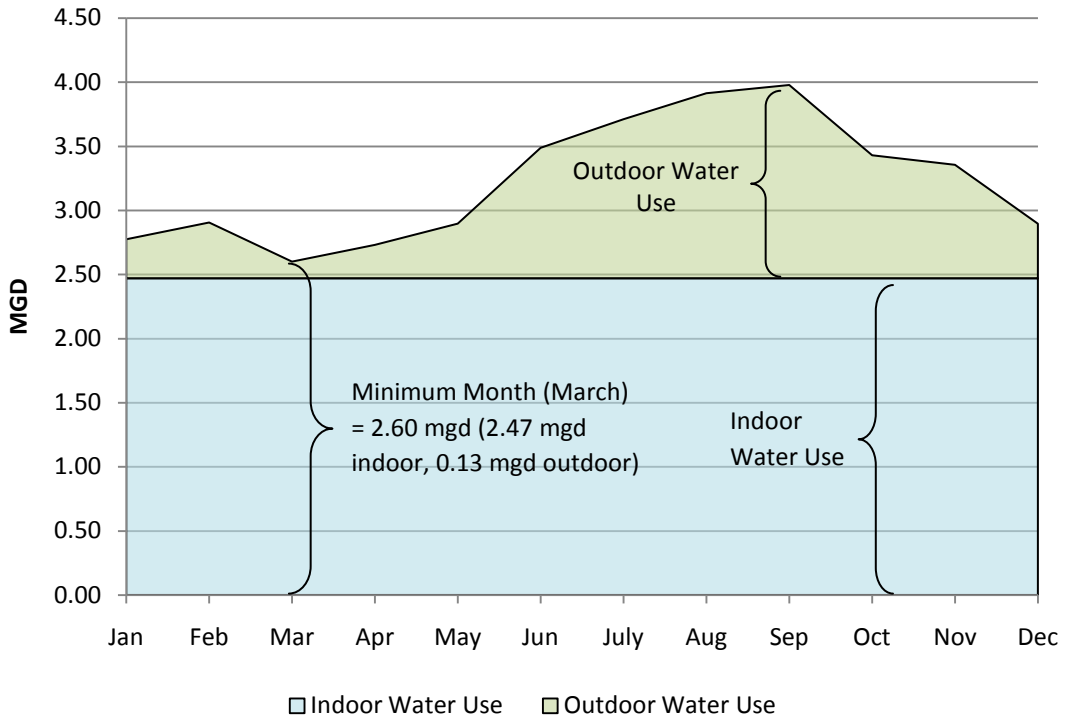


Figure 3 - Minimum Month Methodology for the Single-Family Residential Customer Class

Figure 4 shows results of the minimum month methodology for the commercial customer class for all non-irrigation accounts. Again, March is the month with the lowest overall billed water use for non-irrigation accounts. For the commercial sector, it is estimated that 0.04 mgd or 5 percent of the total water consumed during March is attributed to outdoor water use. For all other months of the year, any billed water use greater than 0.76 mgd, or 95 percent of the minimum month billed water use, is assumed to be outdoor water use, as indicated by the green shaded area in Figure 4.

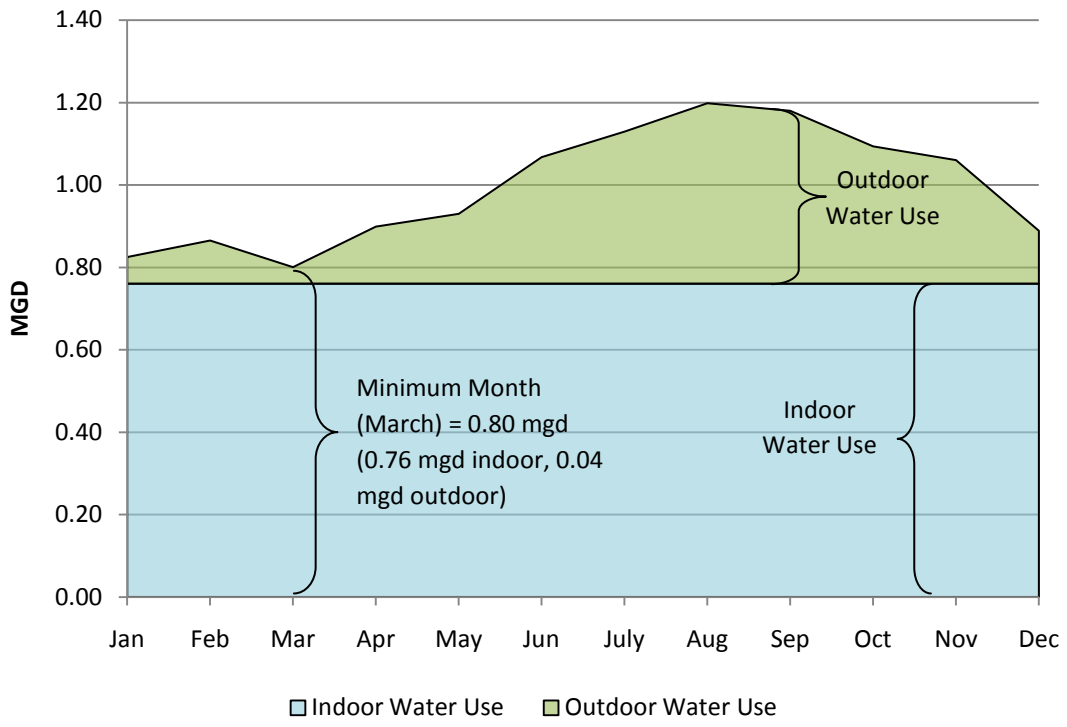


Figure 4 - Minimum Month Methodology for the Commercial Customer Class

The breakdown of outdoor water use for the single-family residential and commercial customer classes based on average annual billed water use for the period July 2000-March 2010 and the results of the minimum month methodology are presented in **Table 4**. Single-family residential outdoor water use is approximately 25 percent of average annual billed use and 33 percent of peak season use. Commercial outdoor water use is approximately 31 percent of average annual billed use and 40 percent of peak season use.

Table 4 - Single-Family and Commercial Average Annual Indoor and Outdoor Water Use Estimates

Customer Class	Total Average Billed, in MGD[1]	Indoor Estimate, in MGD	Outdoor Estimate, in MGD	Outdoor % of Total	Avg. Peak Season Outdoor Estimate, in MGD	Avg. Peak Season Outdoor % of Total
Single-Family	3.30	2.47	0.84	25%	1.20	33%
Commercial	1.11	0.76	0.35	31%	0.51	40%

[1] Total includes average billed use inside and outside of the City as well as metered irrigation use.
[2] Peak season is May through November

The minimum month methodology presented above demonstrates the typical breakdown of monthly indoor versus outdoor billed water use for single-family and commercial customer classes in Franklin. It is assumed that a portion of outdoor water use could be saved through the implementation of an irrigation ordinance. For this analysis, the rate of potential savings is inferred from case study findings of outdoor water use reductions, as a result of implementing an irrigation control ordinance, and is discussed in Section 4.0.

4.0 Key Assumptions

It is difficult to estimate the portion of peak season outdoor water demand used solely for irrigation, because it is expected that activities other than landscape irrigation contribute to peak season outdoor water use (e.g., car washing and swimming pools). Further, not all irrigation is provided through irrigation systems, as all houses and commercial facilities in the service area may not have such systems. However, it is reasonable to assume that the majority of that outdoor use is irrigation. For planning purposes, it is assumed that all of the peak season (i.e., May through November) outdoor water use, estimated following the minimum month methodology, is irrigation use eligible for potential irrigation ordinance program savings.

The assumption of 5 percent outdoor water use in the minimum month (Table 3) accounts for the portion of outdoor use that is not irrigation use. It is also assumed that estimated outdoor water use during the off-peak season is not eligible for potential irrigation ordinance program savings. It is important to note that assuming all of the estimated peak season outdoor water use is for landscape irrigation may result in overestimation of annual residential irrigation demands and consequently, of potential water savings achieved through an irrigation ordinance. It is likely that not all landscape irrigation in the service area is applied by means of a controlled irrigation system. Thus, not all water used for landscaping would be affected by an ordinance requiring smart controllers.

There is also some uncertainty associated with estimating potential water savings achieved with an irrigation ordinance. A major unknown, regarding adoption of an irrigation ordinance, is the level of compliance. Realized savings from the ordinance is dependent on water customers being aware, understanding requirements, and feeling incentivized to follow the ordinance, as well as the level of enforcement of the ordinance. Some important questions regarding compliance with an ordinance include:

1) What portion of residential water customers has automatic irrigation systems?
2) What percent of residential customers will comply with the ordinance?
3) What percent of residential customers will ignore the ordinance?
4) How will incentives or dis-incentives influence compliance?
5) How will enforcement of the ordinance occur?

4.1 Documented Savings

Documented savings from installation of irrigation control devices and implementation of irrigation ordinances throughout the U.S. can help to understand potential savings achievable for this analysis. It is important to note that various geographic and climatic variables can impact the level of savings observed from documented case studies. Therefore, generalizing these savings to other communities should be done with caution. Consequently, for this analysis, a range of potential savings are developed in order to understand the high and low boundaries of potential savings in Franklin, as illustrated in the literature.

It has been estimated that approximately half of outdoor residential water use is wasted due to poor irrigation practices such as overwatering, improper system design, evaporation, and wind (The Saving Water Partnership 2003). The U.S. Bureau of Reclamation (2008) found that installation of "smart" irrigation controls, mainly in the western U.S., resulted in 15 to 25 percent savings. The EPA has stated that for the average irrigation system in the U.S. sensor-based irrigation control technology has the potential to provide about 20 percent savings compared to conventional clock-driven controllers (EPA, 2011).

In Irvine Ranch, California, evapotranspiration (ET) controllers in test homes were able to convert almost 85 percent of the conservation potential into achieved savings. These savings represented an approximately 16 percent reduction in household outdoor water use (Hunt and Lessick, 2001). Additional analyses showed that outdoor water use savings of 24 percent could be achieved by single-family homes in the top third of high water users.

While much of the smart irrigation controller savings analysis and research has been conducted in drier climates where ET rates are higher, there is evidence of a significant potential for savings in climates that experience greater amounts of precipitation. For instance, Dukes (2008) found that smart irrigation controllers in Florida have the potential to significantly reduce irrigation water use. The results of the analysis indicate that by implementing smart irrigation controllers including

time clock adjustments, rain sensors, and ET controllers, systems in central Florida have the potential to reduce irrigation water use by 30 to 60 percent under normal to rainy weather conditions.

The level of potential irrigation water use savings are influenced by multiple factors including the length of the irrigation season, the type of grass and vegetation being irrigated, precipitation patterns, and customers' attitudes and perceptions. The savings identified in the literature in climates that range from dry to wet illustrate irrigation system smart controller potential savings from 15 to 60 percent. For the purposes of this analysis, based on the documented case study savings, the rate of potential savings is assumed to range from 15 to 25 percent and is estimated under three savings scenarios discussed below. These assumptions could be refined with further rigorous analysis.

4.2 Irrigation System Penetration Rates

Another factor in calculating potential irrigation ordinance water savings is the portion of single-family homes and commercial facilities that have irrigation systems (also referred to as market penetration rate of irrigation systems). According to the Draft Resolution, under the proposed irrigation control ordinance, automatic controllers shall be required for all irrigation systems and automatic systems shall have, at minimum, a device that measures humidity and/or soil moisture content or utilizes recent local weather data. All homes and facilities with irrigation systems would be subject to the ordinance requirements unless they have already installed smart irrigation controllers.

The irrigation system penetration rate is not known for Franklin. Therefore, assumptions regarding the penetration rate of irrigation systems for single-family homes and commercial facilities are required to estimate potential water savings. Thus, for the purposes of this analysis, three automatic irrigation system penetration rates are evaluated: 25 percent, 50 percent, and 75 percent.

4.3 Compliance Rates

After developing assumptions regarding irrigation system penetration rates in Franklin, it is necessary to create assumptions on the rate of compliance with an irrigation ordinance requiring retrofits to existing systems. There is little information available documenting irrigation ordinance participation rates. In Cary, North Carolina, a mid-sized city with approximately 35,000 water customers, compliance rates of 80 percent for residential customers and nearly 100 percent for commercial customers have been reported following a city-wide ordinance requiring rain sensors on all automatic irrigation systems (Platt, 2011). Cary boasts one of the more aggressive and well known water conservation programs in the country. Therefore, it is likely that the participation rates reported there would be at the high end of the spectrum. For the purposes of this analysis, three single-family residential and commercial compliance rates are evaluated: 40 percent, 60 percent, and 80 percent.

4.4 Savings Scenarios Assumptions

The irrigation ordinance assumptions, discussed above, form the basis for calculating potential irrigation water savings in Franklin. In total, three savings scenarios are evaluated, representing low, medium, and high potential savings. **Table 5** below summarizes these scenarios and assumptions. For this analysis, the irrigation system penetration rates, compliance rates, and savings rates, shown in Table 5, apply to both single-family residential and commercial customer classes.

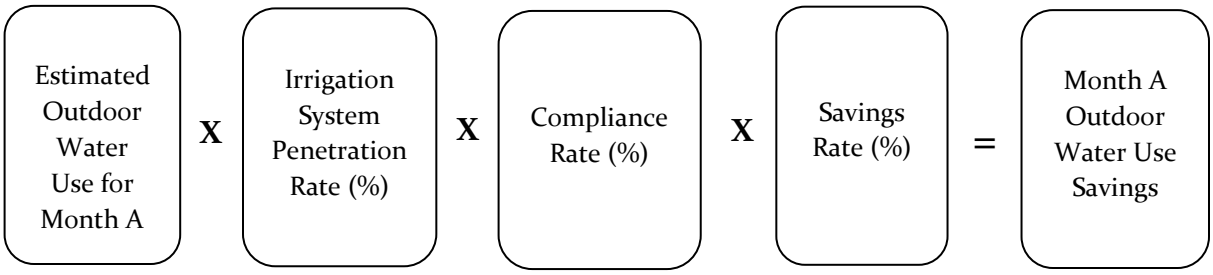
Table 5 - Irrigation Ordinance Savings Scenarios

Scenario	Irrigation System Penetration Rate	Compliance Rate	Outdoor Water Use Savings Rate
Low Savings	25%	40%	15%
Medium Savings	50%	60%	20%
High Savings	75%	80%	25%

5.0 Estimating Potential Water Savings

The potential water savings from implementation of an irrigation ordinance can be calculated using the monthly outdoor water use estimates and the scenario assumptions presented in Section 4.4. Potential irrigation ordinance savings are calculated as shown in **Figure 5**. Using the formula shown in Figure 5, the calculated outdoor water use savings are summed for each month in the peak season to derive the total annual irrigation control ordinance program savings potential.

Figure 5 - Estimate of Peak Season Month Irrigation Ordinance Water Use Savings



Potential future residential and commercial savings could be derived by applying the estimate of outdoor water use approach and the savings rate assumptions described above to a disaggregated water demand forecast for the City of Franklin water service area. That is, a forecast disaggregated by sector and month for future years. However, a disaggregated water demand forecast is not

available for this analysis. Thus, the estimated savings are based upon current service area characteristics (i.e., estimated seasonal irrigation use) with no growth in the number of accounts.

Using the method and assumptions described above, **Table 6** shows the estimated irrigation ordinance savings for the three savings scenarios. The savings vary significantly depending on the scenario. Under the low savings scenario, estimated savings from implementing an irrigation ordinance are only 1.5 percent of the average peak season outdoor water use. Conversely, under the high savings scenario, 15 percent of estimated outdoor water use during the peak season could be saved. **Figure 6** and **Figure 7** show results of the savings analysis for single-family residential and commercial customer classes, respectively.

Table 6. Estimated Irrigation Ordinance Water Savings

Scenario	Avg. Peak Season Month Outdoor Water Use, in mgd	Total Peak Season Outdoor Water Use, in mgd	Total Savings as % of Total Peak Season Outdoor Use	Estimated Total Peak Season Savings, in mgd
Single-Family Residential				
Low Savings	1.20	8.41	1.5%	0.13
Medium Savings	1.20	8.41	6.0%	0.50
High Savings	1.20	8.41	15.0%	1.26
Commercial				
Low Savings	0.51	3.55	1.5%	0.05
Medium Savings	0.51	3.55	6.0%	0.21
High Savings	0.51	3.55	15.0%	0.53

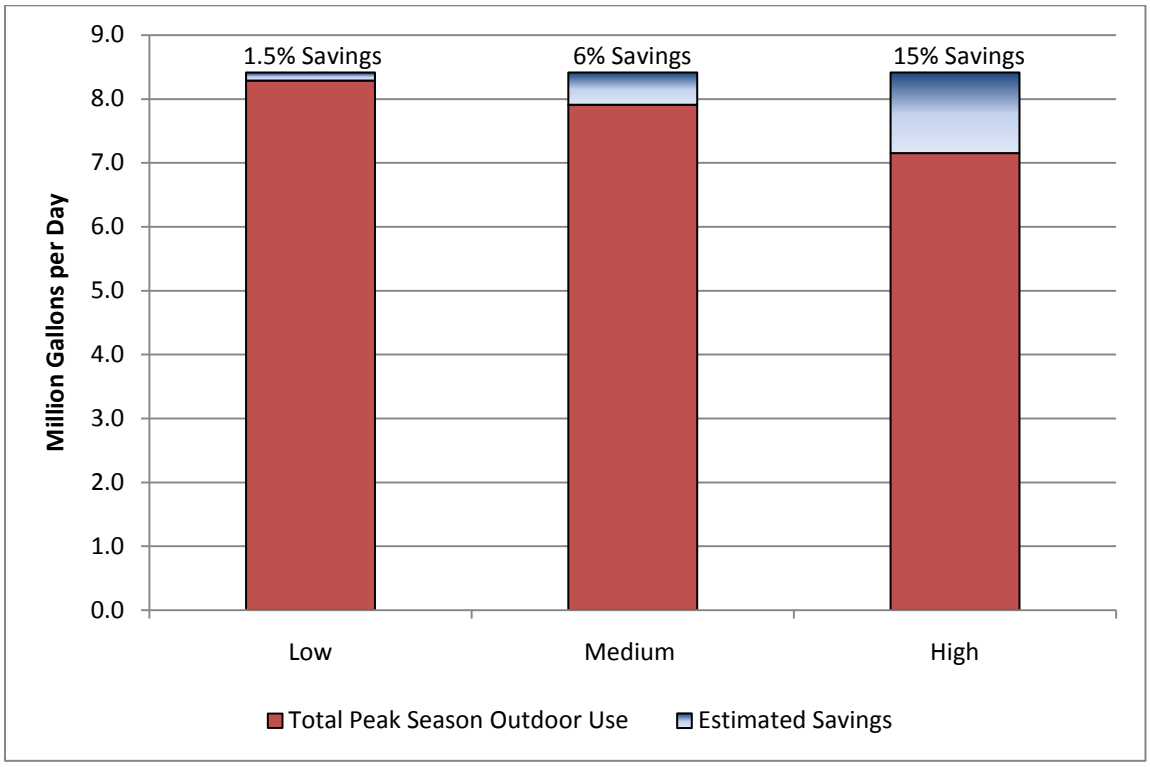


Figure 6 - Single-Family Residential Irrigation Ordinance Savings Estimates

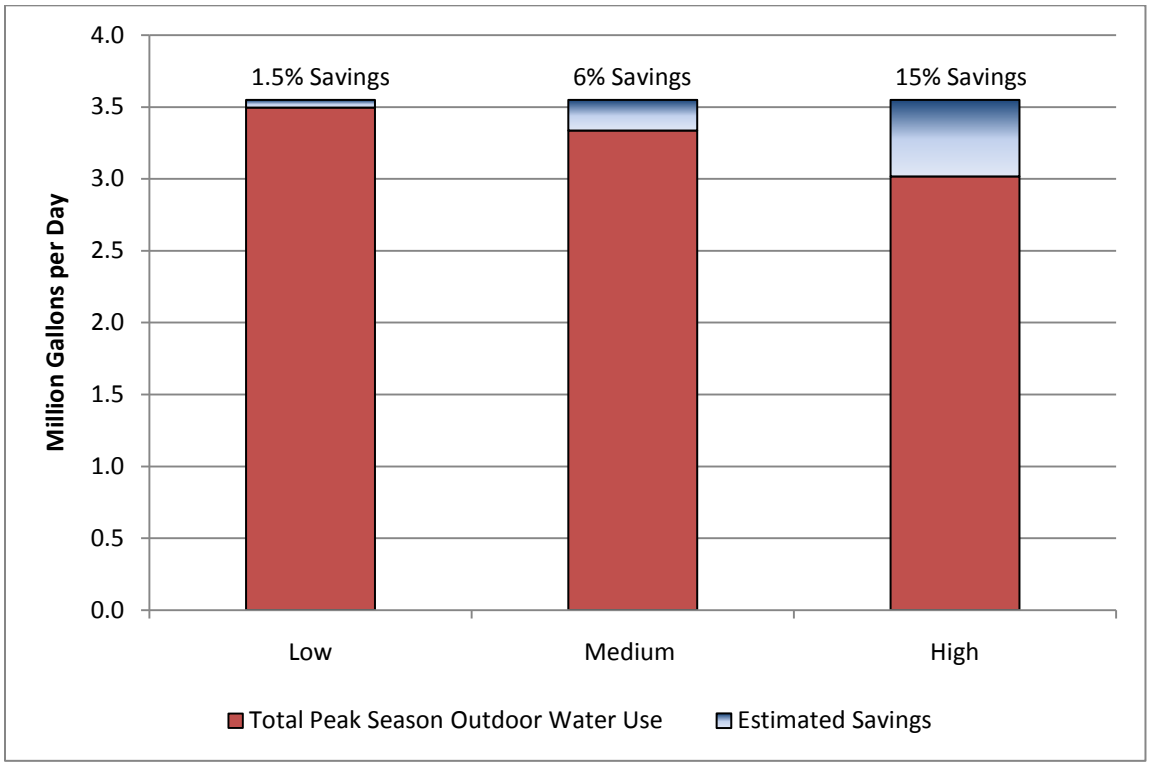


Figure 7 - Commercial Irrigation Ordinance Savings Estimates

6.0 Estimating Potential Program Costs

There are several cost considerations associated with implementing a system-wide irrigation ordinance. Program costs include staff time and materials to develop the ordinance, conducting outreach and education to water customers, enforcing the ordinance, and monitoring and tracking program performance. Costs may vary substantially from one utility to another. Furthermore, there is little information in the literature that identifies program costs for different system sizes, program objectives, and levels of enforcement.

Water conservation program budget information reported by the City of Cary, North Carolina provides a reference for costs required to implement and maintain a water conservation program in a mid-sized city. It should be noted that budget items listed for Cary reflect a conservation program that includes more activities than just an irrigation ordinance. However, the more aggressive a water conservation program is, the higher the costs associated with staff time to ensure compliance and consistency in the use of efficiency standards. **Table 7** presents some costs identified by Cary that may apply to an irrigation ordinance program in Franklin (Platt, 2011). For the purposes of this analysis, the Cary, North Carolina annual cost estimates shown in Table 7 are assumed to apply to Franklin; where there are a range of costs, the midpoint is used.

It should also be noted that the draft City of Franklin irrigation ordinance identifies a $150 penalty to be imposed for the first ordinance compliance violation. Therefore, it can be expected that money collected through the issuance of fines may be available to offset some program costs as presented in Table 7.

Table 7 - Selected Cary, North Carolina Conservation Annual Program Budget Items

Budget Item		Cary, NC Estimated Cost	Cost Assumptions Used for Franklin
Water Conservation Coordinator (Overall program manager)		1.0 FTE ($36,525 - $52,957)	$44,741
Water Conservation Assistant (Coordinate education and enforcement)		0.75 FTE ($30, 035 - $43,555)	$36,795
Field Technician (Conduct field enforcement of ordinance)		$8 - $10/hr during summer months ($4,160 - $5,200 per season)	$4,680
Supplies	Paper for Brochures	$3,000	$3,000
	Promotional Items	$5,000	$5,000
	Irrigation Supplies	$500	$500
Total Annual Budget		**$79,220 - $110,212**	**$94,716**

7.0 Estimate of Costs per Unit of Water Saved

Using the assumptions regarding the irrigation ordinance program costs shown in Table 7 and estimated program savings shown in Table 6, the annual program costs per unit of water saved can be calculated. The cost per unit of water saved provides decision makers with information necessary to understand the value of the program, in relation to other conservation programs or other sources of water supply.

The cost per unit of water saved is expressed on an annual peak season basis and is calculated by dividing the annual program costs by the estimate of annual water savings attributable to implementing and operating the program. The cost per unit of water saved is calculated as illustrated in **Figure 8**.

Figure 8 - Calculation of Cost per Unit of Water Saved

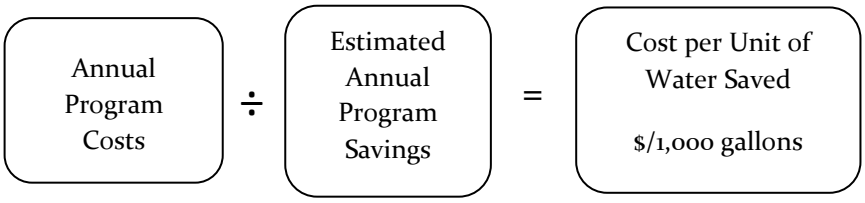


Table 8 shows results of the cost per unit of water saved analysis. The irrigation ordinance program becomes more cost effective as the savings scenario goes from low to high. Under the low savings scenario, it costs $1.45 to save 1,000 gallons, whereas under the high savings scenario, it costs only 14 cents to save 1,000 gallons.

Table 8 - Total Annual Irrigation Control Ordinance Program Cost per Unit of Water Saved

Savings Scenario	Annual Savings, in Thousands of Gallons	Cost/Thousand Gallons Saved
Low	65,484	$1.45
Medium	261,936	$0.36
High	654,841	$0.14

Note: Savings and cost per 1,000 gallons saved includes both single-family residential and commercial estimated savings.

8.0 Cost per Unit of Water Saved vs. Cost of Additional Supplies

In order to make an informed decision regarding various water supply options, the cost to obtain additional water supplies in Franklin can be compared to the costs and benefits of implementing an irrigation ordinance program. The City of Franklin has two options for procuring water supplies: purchase treated water from Harpeth Valley Utilities at a cost of $2.55 per 1,000 gallons or withdrawals from the Harpeth River at a cost of $1.72 per 1,000 gallons. A comparison of the irrigation ordinance savings scenario, costs per 1,000 gallons of water saved and the two water acquisition costs for Franklin, is shown in **Figure 9**. The figure shows that the irrigation ordinance program is a more cost-effective means for Franklin to obtain additional water supplies, compared to purchasing from Harpeth Valley Utilities or direct Harpeth River withdrawals.

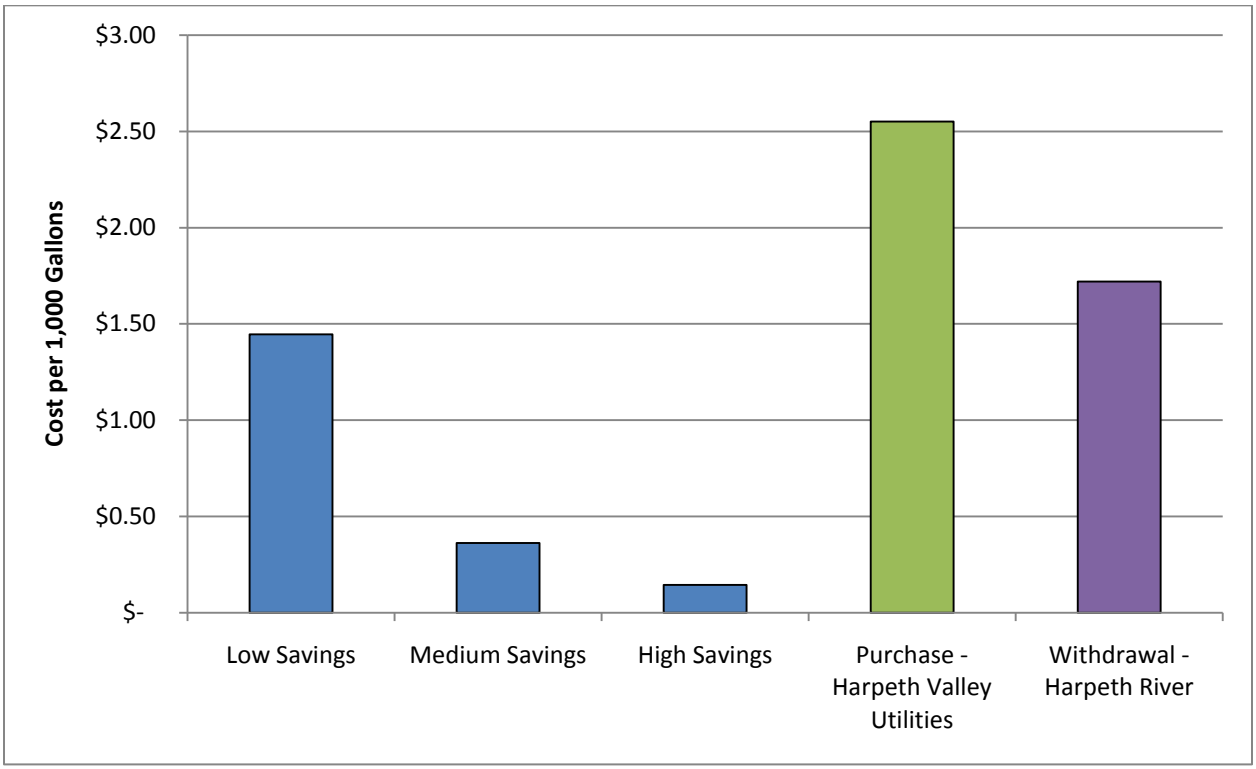


Figure 9 - Cost per 1,000 Gallons Saved/Acquired Comparison

9.0 Conclusion

This memorandum examines the potential single-family residential and commercial outdoor water use savings attributable to an irrigation control ordinance in the City of Franklin water service area. Potential savings are calculated using historical customer class billing data to develop an average year demand and assumptions regarding the current penetration rate of automatic irrigation systems, customer compliance, and efficiency gains by converting an automatic irrigation system to one with "smart" controllers that measure humidity and/or soil moisture and utilize recent local weather data in order to eliminate wasteful irrigation water use. The savings assumptions that form the basis for this analysis are informed by documented case studies throughout the U.S., where available. Potential irrigation water use savings are estimated for a low, medium, and high savings scenario.

Program cost assumptions are derived from documented cost estimates reported for the City of Cary, North Carolina. Because of its similar climate and service area characteristics, these program costs are assumed for Franklin. Furthermore, the estimated costs per unit of water saved from implementing an irrigation control ordinance are compared to known water purchase and treatment costs for Franklin.

Results of this analysis show, based on the assumptions presented in this memorandum, that an irrigation control ordinance is a cost-effective method of conserving water in Franklin. The combined single-family residential and commercial customer class peak seasonal savings range from 1.5 to 15 percent of total peak season outdoor water use depending on the savings scenario. These savings translate into program costs of $0.14 to $1.45 per 1,000 gallons saved, depending on the savings scenario. By comparison, it costs the City of Franklin between $1.72 and $2.55 per 1,000 gallons to obtain additional water supplies.

10.0 References

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City of Franklin, Tennessee

Integrated Water Resources Plan

Appendix D:

Wastewater

- Development of Sanitary Sewer Flows for the City of Franklin IWRP
- Existing WWTP Technical Analysis Evaluation of Existing Equipment
- Collection System Technical Memorandum
- New WWTP Technical Analysis: Technical Memorandum
- Existing WWTP Technical Analysis Biological and Hydraulic Capacity Analysis

CDM Smith

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Reid, Inc.

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Nashville, TN 37204
(615) 383-1113
FAX (615) 386-8469
www.ssr-inc.com

Memorandum

DATE: January 19, 2011

TO: File

CC:

FROM: Bo Butler

RE: **FRANKLIN INTEGRATED WATER RESOURCE PLAN**

Development of Sanitary Sewer Flows for the City of Franklin IWRP
SSR No. 10-41-025.0

Nashville
Houston
Fort Lauderdale
Sarasota
Memphis
Phoenix
New Orleans
Dallas

MAYES CREEK DRAINAGE BASIN

The Mayes Creek Drainage Basin is characterized by mainly undeveloped land to the far eastern side within the city of Franklin. Approximately half of this drainage basin lies within the urban growth boundary. However, the remainder outside the UGB drains to areas within the UGB so that sanitary sewer service should take into account those areas that could drain to it. The total acreage is approximately 9,850 acres. There are approximately 640 existing single-family units within the basin. There is a total undeveloped acreage within the basin of 7,701 acres, representing an additional 6,968 lots that could be developed in the basin. The total projected number of homes within the basin is 7,608, with a projected population of approximately 19,400. The total average flow anticipated from this basin within the 20-year study period is 2.7 MGD with a total peak flow of 8.7 MGD.

Flow from the Mayes Creek Drainage Basin is routed through preliminary layouts of interceptor sewers ranging from 12” in diameter to 36” in diameter. The low point in the basin is just east of the intersection of S. Caruthers Road and Arno Road. It is anticipated that flow from this basin will be collected and transported west toward the Goose Creek Basin by either deep sanitary interceptors or pump station.

GOOSE CREEK DRAINAGE BASIN

The Goose Creek Drainage Basin lies in the southern portion of the city, essentially bisected by Interstate 65 north of S.R. 840 and south of Forrest Crossing subdivision. The Goose Creek Drainage Basin is comprised of approximately 10,300 acres, of which 5,200 are currently undeveloped. There are approximately 1,986 existing developed lots within the basin. Land development in the area could add up to 14,385 lots using current zoning densities in addition to 7,800,000 square feet of office retail space. The projected population in the area within the 20-year study period, assuming full build-out of the basin, is 41,746 residents. The average flow from the basin is anticipated to be 6.7 MGD, with a total peak flow of 22 MGD.

Preliminary routing of sanitary sewer flow within the Goose Creek Drainage Basin has been completed and utilizes interceptor sewers in the range of 10" through 36" in diameter. This pump station transports sanitary sewage through force mains around the Forrest Crossing golf course to an existing gravity sewer, behind Moore Elementary School. From this point to the Franklin Wastewater Treatment Plant, sewage flows by gravity where it is treated and disposed of.

WESTERN DRAINAGE BASIN

The Western Drainage Basin is located in the far western section of the Franklin urban growth boundary. It is essentially bisected by Highway 96 W. and includes Franklin Green, West Haven and subdivisions along Carlisle Lane. The Western Drainage Basin is comprised of approximately 5,500 acres, of which 2,500 acres are currently undeveloped. There are currently 1,231 existing developed lots. Using current zoning densities, an additional 6,195 lots, and an additional 600,000 SF of possible office/retail space are possible in the 20-year study period. The projected population within the area over the planning period would be 18,936 residents. The average sewer flow in the area is anticipated to be 2.7 MGD, with a total peak flow of 8.8 MGD. Sanitary sewer flow from the Western Drainage Basin will be routed through interceptor sewers ranging from size from 12" to 30". Sewer flow will be collected at the lower end of the basin where a pump station will deliver flow to the Franklin Wastewater Treatment Plant. An existing pump station was built a few years ago that serves essentially the southern portion of the drainage basin where flow is delivered to the wastewater plant. However, for the development to continue in the northern half of the drainage basin, that pump station will have to be moved or an additional pump station built in an area near the meeting of the waters, or the confronts of the Big Harpeth and West Harpeth rivers.

CENTRAL FRANKLIN SEWER BASIN

The Central Franklin Sewer Basin is located within the center portion of the city of Franklin, largely in an area west of the Harpeth River, east of Downs Boulevard and extending from Fieldstone Farms to the north to Winstead Hill on the south. The basin is comprised of approximately 4,000 acres and the basin is seen as essentially fully developed. However, in-field development is sure to take place over time. There are approximately 5,162 existing developed lots with an additional 102 lots currently planned according to the Franklin Planning Department. There are also 2.7 million SF of retail office space planned for the Central Franklin Drainage Basin. Over the 20-year study period, it is anticipated that the population in this area will be approximately 13,423 residents. The average daily flow is projected for this area at 1.8 MGD, with a total peak flow of 6.0 MGD.

The Central Drainage Basin is currently served by interceptor sewers and pump stations. The sewers range in size from 10" to 54", and there are currently six pump stations operating within the drainage basin. This is where the Franklin Water Management Department is currently directing their efforts toward rehabilitation of old cracked and failing pipeline, where it is felt that a substantial amount of inflow and infiltration are entering the sanitary sewer system.

WATSON BRANCH DRAINAGE BASIN

The Watson Branch Drainage Basin is located in the central eastern section of the city of Franklin. The basin is comprised of approximately 8,120 acres, of which 911 acres are undeveloped. There are currently 9,013 existing developed lots within the basin, with a number of planned lots according to the Planning Department, to be 805 units. During the study period, it is anticipated that the population in the area would reach 25,036 residents. There is currently 13,255 million SF of office/retail space planned for this drainage basin. The total average daily flow from the basin is anticipated to be approximately 7.2 MGD, with peak flows reaching 23 MGD during the study period.

This drainage basin, while essentially built out, will experience in-field development and also additional development east of Interstate 65 and south of the McEwen Drive corridor. The basin is currently served by interceptors ranging in size from 12” to 54”. Sanitary sewer infrastructure at the lower end of this basin intercepts sewer flow from the Goose Creek area, the Lawton Creek interceptor area, and the Liberty Creek interceptor area.

SPENCER CREEK DRAINAGE BASIN

The Spencer Creek Drainage Basin is located in the northeastern section of the city of Franklin urban growth boundary. The basin is comprised of approximately 5,982 acres, of which 860 acres are undeveloped. There are currently 8,587 existing developed lots with approximately 3,357 additional lots possible under current zoning densities. There is additional 21 million SF of office/retail space that is planned for this area. During the study period, it is anticipated the population in the fully developed basin will reach 30,500 residents. The total average sewer flow from this basin is anticipated to be 7.7 MGD, with peak flows in excess of 25 MGD.

The basin is currently served by interceptors ranging in the size from 10” to 36”. There are currently four pump stations in the outlying areas of the basin that are needed to direct flow into the gravity flow section of this interceptor system. Flow from this basin enters the Harpeth River interceptor just south of the existing Franklin Wastewater Treatment Plant.

SOUTHWEST DRAINAGE BASIN

The Southwest Drainage Basin is comprised of Four sub-basins located in the southern portion of the Franklin Urban Growth Boundary. Generally bisected by Columbia Pike, the area is largely undeveloped with the exception that a portion of Oakwood Estates and subdivisions along Henpeck Lane are located within the drainage area. There are approximately 7,900 acres in the basin, and there are approximately 880 existing single family homes in the area. During the study period, it is anticipated that a total of 7,140 single family homes will be located in the basin, representing a population of 18,200. The total average daily sewer flow from this basin is projected to be 2.5 MGD with peak flows approaching 8.25 MGD.

The basin is currently served exclusively by on-site septic systems, and no sanitary sewers have been planned for these areas. Due to the geography of these basins, pump stations will be required along with gravity sewers for future sanitary sewer service.

SUMMARY

The City of Franklin will include approximately 173,000 people and in excess of 68,000 homes in the study area according to these projections. There will be approximately 45.5 million square feet of commercial space within the Franklin Urban Growth Boundary. The total average daily sanitary sewer flow is projected to be 32.3 MGD with peak flows reaching 105 MGD.

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Technical Memorandum No. 1

To: *City of Franklin IWRP Steering Committee*

From: *CDM*

Date: *September 22, 2011*

Subject: *Integrated Water Resources Plan – Existing WWTP Technical Analysis Evaluation of Existing Equipment*

Executive Summary

CDM staff visited the WWTP facility on January 26, 2011 to perform a condition assessment of the existing facility that included an assessment of the mechanical equipment, structures, and electrical components. The information collected during the condition assessment was used to develop a Criticality Analysis for the processes at the WWTP. This scoring of the processes will impact decisions regarding the need for potential upgrades to the WWTP if existing equipment is nearing the end of its useful life or requires upgrades prior to any additional future expansions. A summary of the findings from the Criticality Analysis are shown in the below table.

In this table, higher criticality scores equate to longer lasting and more efficient equipment, whereas lower scores equate to unreliable equipment that is nearing its service life. Scores presented in this table indicate that most equipment at the WWTP is nearing the expected service life and, either equipment replacement and/or upgrade is required in the near future. It is important to note that this table is an estimation tool based on conversations with WWTP staff and visual observations.

CDM recommends the City develop a Computerized Maintenance Management System (CMMS) in addition to the City's overall IMS (Information Management System). A CMMS will summarize and create historical tracking for vehicle maintenance, work orders for process equipment, spare parts inventories, and necessary staff training for maintenance certifications as required. A CMMS will store all of this information, creating readily accessible files for staff while organizing the overall maintenance of the WWTP to create an additional level reliability.

Equipment Group	Equipment	Average Criticality Score
Influent Pump Station	Influent pumps	14
Headworks	Screens, Grit Removal, Odor Control	67
Oxidation Basins	Anoxic mixers, surface aerators, course air diffusers, submersible pumps, blowers	27
Secondary Clarifiers	Clarifier drive	15
Secondary Clarifiers #5 - #8 Pump Station	RAS pumps, WAS pumps, scum pumps	49
Secondary Clarifiers #9 & #10 Pump Station	RAS pumps, WAS pumps, scum pumps	20
Filters	Denitrification filters, backwash air blowers, backwash water pumps, methanol system, dry well sump	24
Post Aeration	Post aeration basin, blowers, fine air diffusers	21
Disinfection	UV system	24
Water Reclamation	Reclaimed pumps[1]	38
Thickening	DAF tanks, floatator drive, recycle pump, air compressor, thickened sludge pump, pressurization tank	71
Storage Tank	BFP feed tank, blowers, course air diffusers	52
Solids Processing Building	Belt filter presses, hydraulic units, wash water pumps, belt filter press feed pumps, conveyors, polymer system, odor control	68

[1] Reclaimed pump 3 was not in service at the time of the condition assessment, therefore no assessment and numerical value was assigned to this category.

A structural analysis was also performed on the WWTP structures. Routine structural maintenance and inspection is a necessary requirement for all components of a WWTP. CDM's structural engineer provided recommendations based upon the site visit for necessary general structural maintenance of the WWTP. The recommended maintenance described in this memorandum is summarized below along with a recommended structural maintenance schedule for the facility.

Improvement	Year			
	2011 - 2015	2015 - 2020	2020 - 2025	2025 - 2030
Headworks				
Removal of all spalled and/or cracked concrete	X			
Locate all active leaks, repair with grout as necessary	X	X	X	X
Inspection of interior of empty structure	X	X	X	X
Oxidation Ditch Structures				
Repair expansion joint filler material	X			
Repair minor leaks	X	X	X	X
Distribution Boxes				
Drain, clean, and inspect interior of structures	X	X	X	X
Clarifiers				
Drain, clean, and inspect interior of structures	X	X	X	X
Final Clarifier Pump Station #5 - #8				
Repair spalled concrete	X			
Apply surface dampproofing/ waterproofing to clarifier walls	X			
Clean and inspect pumps, piping, and structure	X	X	X	X
Blower Building				
Continue inspections on regular basis	X	X	X	X
Biosolids Facility				
Inspect, repair spalled concrete as necessary	X	X	X	X
UV Disinfection				
Replace rusted rebar	X			
Repair spalled concrete	X			
Drain channels/inspect structure	X	X	X	X

1.0 Introduction

As part of its Integrated Water Resources Plan (IWRP), the City of Franklin is developing a plan to address the issues at the existing wastewater treatment plant (WWTP) that would allow the facility to consistently meet NPDES permit compliance at its design flow. Additionally, the plan will provide a summary of the necessary WWTP improvements that would be required to maximize the capacity at the existing facility. For this analysis, CDM performed a facility condition assessment and recommended improvements for the existing WWTP to maintain the existing capacity of the WWTP.

1.1 Background of Existing WWTP

The City of Franklin WWTP was originally constructed in 1975, and was expanded in 1990, 1996, 1999, and again in 2005. The original plant, constructed in 1975 for an initial capacity of 2.5 million gallons per day (mgd), was entirely abandoned in favor of a later design. The current WWTP facilities were constructed in 1999 to include a new headworks facility with mechanical bar screens and two Pista grit chambers, two new oxidation basins, three secondary clarifiers, filters, post-aeration and a step aerator. The 1999 upgrade was designed and constructed to expand the plant capacity from the original 2.5 mgd to 6.5 mgd. In 2005, a second expansion included the addition of a third oxidation basin, two additional secondary clarifiers, new denitrification filters, an ultraviolet (UV) disinfection facility, and a new step aerator; these facilities increased the plant capacity to 12 mgd.

1.1.1 WWTP Discharge Limits

The current NPDES permit allows the existing WWTP to discharge into the Harpeth River; a summary of the permit requirements are summarized in **Table 1**. The City's current permit was issued on September 30, 2010 and will expire on November 30, 2011. This short permit cycle was implemented to allow the Tennessee Department of Environment and Conservation (TDEC) to schedule the permit renewal to be concurrent with other discharges that are within the same watershed to be consistent with the State's watershed management approach.

Table 1
Current NPDES Permit Requirements

Parameters	Effluent Limits						
	Monthly Average (mg/L)	Monthly Average Load (lb/day)	Weekly Average (mg/L)	Weekly Average Loading (lb/day)	Daily Maximum Concentration (mg/L)	Daily Minimum Percent Removal (%)	Daily Minimum (mg/L)
CBOD5 (summer)[1]	4.0	400	6.0	600	8.0	40	-
CBOD5 (winter)[1]	10	1,001	15	1,500	20	40	-
Ammonia as N (summer)[1]	0.4	40	0.6	60	0.8	-	-
Ammonia as N (winter)[1]	1.5	150	2.3	230	3.0	-	-
Total Nitrogen (summer)[1]	5.0	377	-	-	-	-	-
Total Phosphorus as P (summer)[1]	5.0	-	-	-	-	-	-
Suspended Solids (summer)[1]	10	1,001	15	1,501	20	40	-
Suspended Solids (winter)[1]	30	3,002	40	4,003	45	40	-
E. coli (cfu/100mL)	126	-	-	-	941 cfu/100	-	-
Settleable Solids (ml/L)	-	-	-	-	1.0	-	-
Dissolved Oxygen	-	-	-	-	-	-	8.0
pH (standard units)	-	-	-	-	9.0	-	6.0

[1] Summer = May 1 through October 31 and Winter = November 1 through April 30.

1.1.2 Existing WWTP Process

Wastewater enters the WWTP through the influent pump station that is sited on the northern side of the plant. Flow is pumped through the mechanical bar screens to remove floatables and through the vortex grit chambers to remove inorganic solids. From the grit chambers, flow is distributed through the splitter box to the three oxidation basins; however, one of the basins is currently not in use. A schematic of the existing WWTP is provided in **Figure 1**. It is important to note that, although the original WWTP buildings and structures remain onsite, they are currently not in use. Potential reuse of these existing structures will be evaluated as part of a potential plant expansion; additional discussion of this will be provided in Section 2.0.

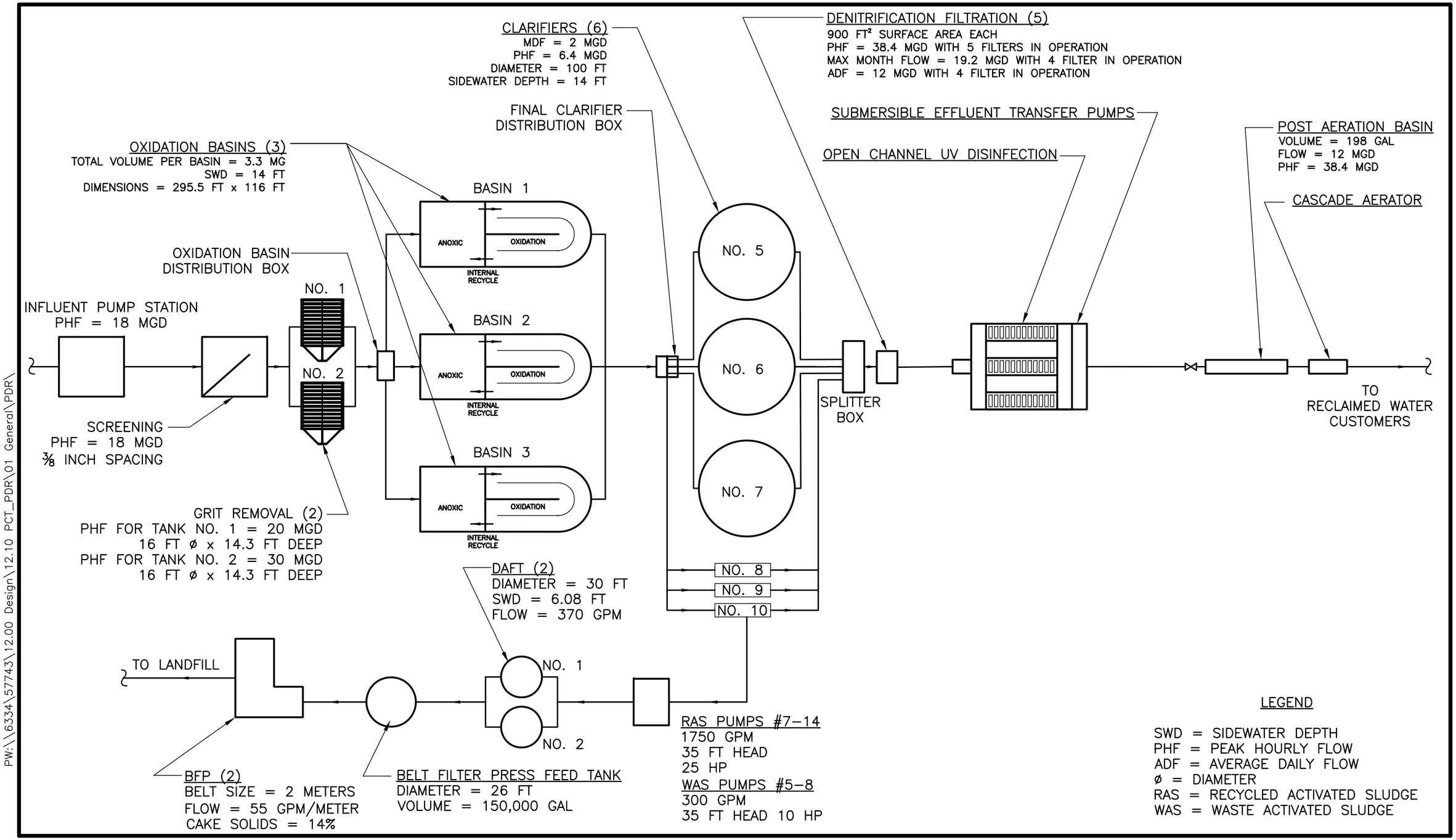

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Figure No. 1
Existing Franklin WWTP
Process Flow Diagram

From the oxidation ditches, wastewater flows by gravity to the clarifier distribution box where it is distributed among the six clarifiers. Treated wastewater from the clarifiers flows to the filter junction box, where it is distributed among the five denitrification filters. Return activated sludge (RAS) from the clarifiers, is pumped back to the oxidation basins where it is mixed with influent wastewater. Treated wastewater from the denitrification filters flows by gravity to the ultraviolet (UV) disinfection process, before flowing to the post-aeration tank and then the step aerator, prior to discharge to the Harpeth River.

Waste activated sludge (WAS) from the process is pumped from the clarifiers to the dissolved air floatation thickeners. After solids are thickened, they are stored in the belt filter press feed tank during non-operational hours; during operational hours, waste solids are dewatered with belt filter presses (BFPs) to approximately 14 percent solids and taken by truck for disposal in the Camden landfill, a one-way distance of approximately 100 miles from the WWTP .

2.0 Condition Assessment – Existing WWTP

Evaluation of the existing WWTP included a physical condition assessment to determine the potential for future use of the facilities. On January 26, 2011, a team of CDM engineers visited the WWTP to perform a facility inspection. During the site visit, individual unit process components were inspected for normal wear and tear during operation; additionally, inspections of process structures and electrical components at the WWTP were conducted. The information collected during the condition assessment will impact decisions regarding the need for potential upgrades to the WWTP if existing equipment is nearing the end of its useful life or requires upgrades prior to any additional future expansions.

2.1 Procedure

This assessment has been focused to determine the condition of equipment under typical, local operating conditions, with an emphasis on the longevity and operational intent of the equipment, and the ability of the equipment to continue to provide uninterrupted service for a specified service life. The generally accepted service life of WWTP equipment is as follows:

- Structures at 50-60 years
- Pipe at 70 years
- Major electromechanical equipment at 15 to 20 years
- Small electromechanical systems at 10 to 15 years

A visual inspection of the suitability and condition of the existing treatment facilities and other major equipment was conducted in a manner that would not disrupt plant operations. This also included an analysis of the physical condition and operation of the existing equipment and facilities.

Each of the WWTP processes were divided into their own category and separately evaluated based on equipment. The initial WWTP, which has since been abandoned, was also evaluated with the other components to the new WWTP, which is currently being used to process the flows. Various components of each of the process were constructed at different times during expansions and have been evaluated for remaining useful life.

2.1.1 Abandoned Plant

The abandoned plant includes equipment and buildings for processes for chlorine addition, micro-screening, return sludge pumping, blowers, clarifiers 1 through 4, excess flow basins 1 and 2, aeration basins, chlorine contact tanks, and tertiary filters. The facility was abandoned in place, approximately 15 years ago. The existing equipment is also far beyond its anticipated service life of 15 to 20 years and corrosion with corrosive loss is prevalent throughout the facility, thus limiting its return to service.

Figure 2: Pumping equipment in sludge pump room

Figure 3: Entrance into the sludge pump room

The abandoned structures have evidence of cracking, efflorescence (calcified water seepage through cracks that dries to a white chalk), and spalling (fragmented, chipped concrete) and would require further structural evaluation prior to making recommendations regarding the ability and potential costs to place this equipment in use.

2.1.2 Influent Pumping

The influent pump station was reconstructed during the 2004 expansion and consists of five influent pumps driven by variable frequency drives, valves, piping and other pumping appurtenances; odor control equipment is also installed at the pump station.

2.1.2.1 Findings

Because this system was newly constructed in 2004, the pumps and pumping appurtenances are in good condition. The pumps and piping have a life expectancy of approximately 20 years and the structure has an expectancy of approximately 50 years.

Figure 4: Influent pump station

Figure 5: Influent pump station

2.1.3 Headworks

The headworks, constructed in 1997, consists of influent channels, adjustable sluice gates, rotating drum screens with a manual bypass, a Pista-grit vortex grit removal system with a manual bypass, influent flow monitoring and laboratory sampling equipment. The headworks are intended to trap and remove inorganic debris from the influent wastewater to minimize potential for damage to downstream equipment and improve the performance of the treatment process.

2.1.3.1 Findings

The headworks are approaching the end of its expected useful life. This equipment is operated at its peak capacity and will likely begin to have frequent breakdowns. Observations of hoses and braces in the facility indicate that maintenance staff must open hatches and wash-down equipment frequently to maintain operation of the equipment. Signs of corrosive losses on the grit washers, leaking flanges around the cyclones, and oil around the gearbox gaskets are also typical indications that these components will soon reach the end of their useful life. Other indications that equipment may begin to fail include the rated capacity of the equipment; the current headworks system is rated at 20 and 30 mgd peak flow per chamber.

Figure 6: Corrosive loss in the flanges and shroud in the grit classifier.

Figure 7: Spalling with rust emanating through the wall indicates rebar corrosion in the wall.

2.1.4 Oxidation Basins

Wastewater is treated biologically to remove carbon and nitrogen in the oxidation basins. The oxidation system includes the influent distribution box, recycled activated sludge (RAS) distribution box, three oxidation basins, mixers and aeration blowers. Initial construction of two of the oxidation basins was in 1996; the third basin was constructed in 2004.

Each oxidation basin contains an anoxic zone for denitrification and an aeration zone that removes the biological oxygen demand and provides for ammonia nitrification of the influent. Raw influent is routed to the anoxic zone of the basins from the headworks. RAS is pumped from the underflow of the final clarifiers and recycled into the anoxic zone from the RAS pump station. The mixed liquor flows from the oxidation basins to the secondary clarifiers.

2.1.4.1 Findings

Overall, the oxidations basins are in good condition for continued use. Mechanically, equipment in the two original basins will approach the expected service life within the next five years. Inspection of equipment indicates that most equipment is in an acceptable condition. There are no excessive vibrations, noise, or other apparent problems. The newest basin, number three, is anticipated to have a continued useful service life of approximately 15 years. It is notable that at the time of the facility inspection, basin two, was out of service and filled with process water. While this practice is acceptable, future flows may warrant use of this third basin for increasing the treatment capacity of the process.

Figure 8: Blowers for basins 1 and 2

Figure 9: Piping to diffusers and basin wall for channels in Reactor 1

2.1.5 Secondary Clarification

The secondary clarification process receives mixed liquor from the oxidation basins and allows biomass to settle to the bottom of the clarifier, scum to float to the top, and clarified effluent to pass through the effluent launders. Mixed liquor from the oxidation basins that is periodically wasted from the clarifiers is called waste activated sludge (WAS). Sludge is pumped from the clarifiers either as RAS or WAS, and scum is pumped directly to solids storage or processing. The secondary clarifier system includes the clarifiers and equipment, and the sludge and scum pumps.

2.1.5.1 Findings

There are 8 RAS pumps and 4 WAS pumps in RAS/WAS pump station No. 1, that were installed during the 1996 construction; however, since their installation two of the motors have been replaced. The pumps in pump station No. 1 appear to need maintenance, including belt tightening and alignment. These pumps are anticipated to have approximately five years of remaining service life.

There are four RAS pumps and two WAS pumps in RAS/WAS pump station No. 2. Pumps in RAS/WAS pump station No. 2 were installed during the 2004 upgrade construction and are anticipated to have a continued useful life of approximately 15 years.

Figure 10: Overview of Clarifier 8

Figure 11: WAS pumps for clarifiers 5 – 8

Figure 12: RAS/WAS pump station for clarifiers 9 & 10

Figure 13: WAS pump station for clarifiers 9 & 10

2.1.6 Filtration

Denitrification is the process that converts nitrate produced in the aeration basins to nitrogen gas to complete the cycle of nitrogen removal from wastewater. This process may require carbon in excess of what is contained in the wastewater and an external source of carbon is often necessary. The denitrification system consists of the filter influent flow meter, five filter beds with media in five graduated layers, and a methanol feed system. The filter underdrain consists of a layer of specially formed plastic jacketed concrete air/water distribution blocks. Each filter has two filter troughs. There is one backwash pump for all of the filters, with one redundant

backwash pump in standby mode. Three positive displacement backwash air blowers are in use with one blower used for standby mode. The methanol dosing facility consists of a methanol storage tank, two diaphragm methanol pumps, sample pumps, and a nitrate analyzer.

2.2.6.1 Findings

The filtration system was constructed in 2004 and is in good condition and is anticipated to have a remaining useful life of approximately 15 years.

Figure 14: Filters

Figure 15: Methanol storage tank

2.1.7 Disinfection

The WWTP uses UV light to disinfect wastewater prior to discharge. The disinfection system consists of the UV equipment, post aeration, and the cascade aerator. The post aeration tanks and cascade aerator provide additional dissolved oxygen to the filtered effluent. Treated wastewater flows through the UV system, through the post aeration basins, and discharges into the Harpeth River through the cascade aerator.

2.1.7.1 Findings

The disinfection system was constructed in 1990 and is in fair condition, but nearing the end of its useful service life. The post aeration basin was constructed in 2004 and appears from inspection to evenly distribute the air throughout the basin. The equipment for this process is in fairly good condition; the cascade aeration system currently in use was constructed in 2004 and is in good condition.

Figure 16: Single rack of UV bulbs

Figure 17: Post aeration channel

2.1.8 Plant Service Water and Water Reclamation

The plant service water and water reclamation systems consist of eight vertical turbine pumps that pump water from the treated effluent wet well for in-plant uses and for distribution to the reclaimed water distribution system. The bulk sodium hypochlorite disinfection system, also included in this evaluation, includes all associated appurtenances (tanks, pumps, etc.) Four of the pumps in the system are used for plant service water and pump unchlorinated water, and the remaining four pump chlorinated water to the reclaimed water distribution system.

2.1.8.1 Findings

Equipment in the reclaimed water pump station is from the 1995 WWTP expansion, and is nearing the end of its expected life. At the time of inspection, a motor was removed from the reclaimed water pumps, indicating necessary repairs for this critical system. The chlorine disinfection pumps and system are also nearing their anticipated service life.

Figure 18: Reclaimed water system pumps

Figure 19: Plant water pumps

2.1.9 Biosolids Facilities

The biosolids facilities at the WWTP include dissolved air floatation thickeners, storage tanks, belt filter presses, and associated odor control equipment. For a complete description of these facilities, additional information has been provided in the Biosolids TM No. 1 distributed on May 4, 2011.

2.2 Criticality Analysis

A Criticality Analysis was conducted as part of this facility assessment and was based on the judgment and extensive experience of the CDM field inspector, in conjunction with plant operations and maintenance staff. The criticality rating is determined from the adjusted effective service life discussed in Section 2.3. Direct visual inspections to determine the condition and estimated remaining service life, assignment of a reliability rating, determination of the age-based probability of failure, and assignment of the asset critical failure mode (i.e. scores worker and public safety, potential regulatory violation, and potential fiscal impact for remediation).

The general condition ratings used for direct visual inspection to determine the system condition and estimate the remaining useful life based on the system condition, physical demands, and environmental conditions in which each system is expected to perform is summarized in **Table 2**. **Attachment A** to this document provides a table with scoring for each of the WWTP processes as described in categories that are described in Sections 2.2.1 through 2.2.5. The general condition ratings for each of the pieces of equipment at the WWTP are summarized in Attachment A, (column 5).

Table 2
General Condition Rating Definitions

Condition (column 5)	General Rating Definitions
(1) Excellent	New or almost new equipment in excellent condition. Asset is fully functional as designed with no visible defects or wear.
	Asset requires only normal preventive maintenance.
	Asset is assumed to have 100 - 75% of its effective life remaining.
(2) Good	Asset is fully functional for current operating conditions, shows signs of only minor wear.
	Asset requires normal preventive maintenance and only minor corrective maintenance.
	Asset may have been very recently overhauled or rebuilt.
	Asset is assumed to have approximately 75 - 25% of its effective life remaining.
(3) Fair	Asset exhibits normal or slightly excessive wear but is functionally sound.
	Asset requires normal preventive maintenance and occasional major corrective maintenance.
	Asset is assumed to have approximately 25% or less of its effective life remaining.
(4) Poor	Asset functions but only with a high degree of maintenance or does not function as needed for current operating conditions, indicating that the asset is near the end of its design life.
	Asset requires significant rehabilitation.
	Asset is assumed to be beyond its effective life.
(5) Failed, Imminent, Failure, Obsolete	Asset has failed or will fail imminently and is virtually unserviceable.
	Asset consistently fails to perform at or near its design capacity and no replacement parts are available.
	Asset can be assumed to have reached the end of its effective life and requires immediate replacement.
(0) Abandoned	Asset is abandoned in place; this equipment may only need minimal maintenance to be placed back into service.

2.2.1 Asset Reliability Ratings

The WWTP was quantitatively rated on reliability for the ability of the process to provide adequate service when required for WWTP operations. This rating is generally concluded from a review of maintenance records or discussion with WWTP staff. A summary of the asset reliability ratings and the associated reliability based probability of failure are provided in Table 3. The ratings for each piece of equipment have been summarized in columns 6 and 7 of Attachment A.

Table 3
Asset Reliability Ratings

Reliability Rating (column 6 of Attachment A)	Description	Failure Timing	Reliability Based Probability of Failure (column 7 of Attachment A)
1	As Specified by	Never	0
2	Random Breakdown	Every 10 Years	1
3	Occasional Breakdown	Every 5 Years	2
4	Periodic Breakdown	Every 2 Years	5
5	Continuous Breakdown	≤1 Year	10

2.2.2 Effective Service Life

The next step in determining the criticality rating is the determination of the age-based probability of failure by utilizing the percent of the effective life consumed as determined from direct visual inspection. **Table 4** provides a summary of the anticipated service life for the process equipment at the WWTP. Because this service life is determined under manufacturer conditions, the service life is then adjusted to represent the actual field conditions in which the equipment of the system is expected to operate.

Table 4
Base Effective Service Life

Asset Group	Base Effective Service Life, Years	Adjusted Effective Service Life, Years (column 8 of Attachment A)
Structures	75	50
Major Electromechanical Systems	30	15 – 20
Small Electromechanical Systems	25	10 - 15

2.2.3 Age-Based Probability of Failure

The age based probability of failure is a quantitative rating used to determine the how likely the equipment is going to fail based on the amount of useful life remaining. The age-based probability of failure was rated by applying the percentage of the estimated useful life of the equipment already consumed. The estimated remaining useful life was determined from the inspector and is shown in column 9 of Attachment A. The estimated remaining useful life was calculated by dividing the estimated remaining useful life by the adjusted effective service life. This value is correlated to a useful life remaining for the equipment. The aged-based probability of failure rating is then applied to the equipment based on the estimated useful life already consumed. A summary of the estimated useful life consumed and the age-based probability of failure rating are summarized in **Table 5**. Data for each piece of equipment at the WWTP has been provided in columns 8 through 11 of Attachment A.

Table 5
Age-Based Probability of Failure

Estimated Useful Life Already Consumed (%) (column 9 of Attachment A)	Age-Based Probability of Failure Rating (column 12 of Attachment A)
0%	1
10%	2
20%	3
30%	4
40%	5
50%	6
60%	7
70%	8
80%	9
90% - 100%	10

2.2.4 Redundancy Ratings

The redundancy rating is a quantitative rating used to determine the likeliness of equipment to fail based on the level of redundancy that is provided. Using the age-based probability of failure from Table 5 and reliability-based probability as shown in Table 6, the higher of the two probabilities was selected to determine the worst-case probability of failure for the equipment.

The level of redundancy was determined by the inspector for the equipment and is provided in column 14 of Attachment A. Based on the level of redundancy, the level of reduction for the probability was determined from **Table 6**.

Table 6
Redundancy Ratings

Level of Redundancy (column 14 of Attachment A)	Reduced Probability of Failure
0% Backup	No Reduction
50% Backup	50%
100% Backup	90%
200% Secondary Backup	98%

2.2.5 Asset Critical Failure Mode

The asset critical failure mode is used to determine a quantitative rating relating the reliability of the equipment to critical functions of the process. The redundancy rating values were used to determine the probability of failure from a redundancy standpoint.

Based on the function of the asset, an Asset Critical Failure Mode from **Table 7** was assigned to each piece of equipment. The Asset Critical Failure Mode rating for each piece of equipment at the WWTP was assigned and has been summarized in columns 16 through 19 of Attachment A.

Table 7
Asset Critical Failure Mode

Critical Criteria	Assign 1	Assign 2	Assign 3
Worker and Public Safety	Poses no threat	Poses some threat	Poses significant threat
Regulatory Violation	No violation likely to result if remedial action undertaken as planned	Violation might occur even if remedial action is undertaken	Violation is likely
Fiscal Impact	Remediation can be done at minimal cost	Remediation would be costly	Remediation would be a major burden on staff and/or be at considerable cost

2.2.6 Overall Asset Criticality

The Overall Asset Criticality is then calculated by multiplying the probability of failure by the asset critical failure mode.

Asset Criticality = Probability of Failure x Asset Critical Failure Mode

A summary of the overall Asset Criticality factor is presented in column 20 of Attachment A for the individual equipment.

2.3 Summary of Condition Assessment and Criticality Analysis

The complete Condition Assessment and Criticality Analysis is provided in Attachment A, which details all of the factors and ratings for equipment at the WWTP. A summary of each of the major systems at the WWTP is provided in **Table 8**. The average criticality score shown in Table 8 is the average of each all of the individual pieces of equipment within each process system.

Higher criticality scores equate to longer lasting and more efficient equipment, whereas lower scores equate to unreliable equipment that is nearing its service life. Scores presented in Table 8 indicate that most equipment at the WWTP is nearing the expected service life and, either equipment replacement and/or upgrade is required in the near future. It is important to note that this table is an estimation tool based on conversations with WWTP staff and visual observations.

Table 8
Summary of Criticality Analysis

Equipment Group	Equipment	Average Criticality Score
Influent Pump Station	Influent pumps	14
Headworks	Screens, Grit Removal, Odor Control	67
Oxidation Basins	Anoxic mixers, surface aerators, course air diffusers, submersible pumps, blowers	27
Secondary Clarifiers	Clarifier drive	15
Secondary Clarifiers #5 - #8 Pump Station	RAS pumps, WAS pumps, scum pumps	49
Secondary Clarifiers #9 & #10 Pump Station	RAS pumps, WAS pumps, scum pumps	20
Filters	Denitrification filters, backwash air blowers, backwash water pumps, methanol system, dry well sump	24
Post Aeration	Post aeration basin, blowers, fine air diffusers	21
Disinfection	UV system	24
Water Reclamation	Reclaimed pumps[1]	38
Thickening	DAF tanks, floatator drive, recycle pump, air compressor, thickened sludge pump, pressurization tank	71
Storage Tank	BFP feed tank, blowers, course air diffusers	52
Solids Processing Building	Belt filter presses, hydraulic units, wash water pumps, belt filter press feed pumps, conveyors, polymer system, odor control	68

[1] Reclaimed pump 3 was not in service at the time of the condition assessment, therefore no assessment and numerical value was assigned to this category.

2.4 General Structural Assessment

A general structural assessment was performed by a structural engineer during the site visit. In addition to the general condition of the structures, recommendations for repairs and/or improvements have been included and are summarized in the following sections.

2.4.1 Headworks Structure

The headworks structure was operational at the time of the assessment and cover plates were securely fastened. Because the system was in operation, access to the inside of the structure was

not obtained. The exterior of the structure showed signs of spalling and cracked concrete in several areas; specifically, there are two horizontal construction joints a few feet above grade in the Pista structures that show spalling around the joints, the areas are also cracked and have several active leaks. Additionally, concrete in the vicinity of the pipes entering the structure near the Pista grit system is cracked and has active leaks. There was a minor leak near a construction joint in the screenings dumpster room.

Figure 20: Headworks structure, Pista structure cracked and leaking walls at horizontal joints.

2.4.1.1 Recommendations

- Removal all spalled and/or cracked concrete.
- Locate all active leaks and repair with a flexible expanding polyurethane grout such as Sika Fix HH, by Sika Corporation.
- Repair all concrete with a cementitious repair product.
- A thorough inspection of the interior of the structure is recommended; ideally, the structure should be drained, pressure cleaned, and inspected and recommendations for interior repairs (if any) can be made following the inspection.

2.4.2 Oxidation Ditch Structures

All of the oxidation ditches were filled with water at the time of this inspection, although only basins 1 and 3 were being used to process wastewater. The exterior of the structure and the interior wall surfaces above the water level appeared to be without significant signs of distress. There were no obvious signs of cracks or spalls noted at the time of inspection. There were several minor cracks in the walls, some which were damp; it is likely that these cracks close when temperatures increase during summer months. The expansion joint filler material is cracked and falling out.

Figure 21: Oxidation basin, typical expansion joint condition.

2.4.2.1 Recommendations

- If the minor leaks are an issue for the owner, they can be repaired with a flexible expanding polyurethane grout such as Sika Fix HH, by the Sika Corporation.

- Repair expansion joint filter material with flexible sealant.

2.4.3 Distribution Boxes

The oxidation basin distribution box and the RAS distribution box were in operation at the time of the inspection and cover plates on the top of the structure were securely fastened, therefore the interior of the structures were not inspected. The exterior of the structures appeared to be in good condition with no obvious signs of distress.

2.4.3.1 Recommendations

While not an immediate need, the interior of the structures should be drained and cleaned, and the interior of the structures should be inspected. These activities should be scheduled as routine maintenance within the next five years.

2.4.3 Clarifiers

The secondary clarifiers were in service at the time of inspection. The structures are mostly below grade and the exposed and above grade portions of the structures appeared to be in very good condition with no obvious signs of distress.

2.4.4 Final Clarifier Pump Station

The final clarifier pump station consists of an above grade masonry building with a below grade concrete vault. Visible portions of the final clarifier pump station appeared to be in very good condition with no obvious signs of distress.

2.4.5 RAS/WAS Pump Station (Final Clarifiers No. 5 – No. 8)

The RAS/WAS pump station is a cast in place structure sited between the clarifiers and is primarily below grade. There are some areas of spalled concrete on the above grade walls adjacent to the clarifiers; the below grade walls (which are the exterior surface of the clarifiers) had significant dampness, but otherwise appeared to be in good condition.

Figure 22: RAS/WAS pump station, spalled concrete.

2.4.5.1 Recommendations

- Repair spalled concrete with a cementitious repair mortar.
- Apply surface dampproofing/waterproofing to clarifier walls to reduce below grade dampness with a product such as xypex.

2.4.6 Blower Building

The blower building is a pre-engineered metal building. The structural steel is in very good condition with only small areas exhibiting signs of minor corrosion. However, building insulation is crumbling and falling off the structure.

2.4.6.1 Recommendations

Inspect the structure on a regular basis to monitor the condition of the structural steel.

2.4.7 Biosolids Facility

The two story biosolids facility is constructed of masonry with precast slabs. The building appeared to be in very good condition with no obvious signs of structural distress.

2.4.8 UV Disinfection Structure

The cast in place concrete UV disinfection structure was in service at the time of the inspection and is primarily below grade. Overall, the visible portions of the structure appeared to be in very good condition. There was one area on the interior of a wall with exposed rebar and spalled concrete.

Figure 22: UV disinfection structure, exposed rebar with spalled concrete.

2.4.8.1 Recommendations

- Remove rusted rebar back to sound rebar and attach to existing sound rebar using a mechanical bar splicer.
- Repair spalled concrete with a cementitious repair mortar.

2.4.9 Summary of Structural Assessment

Routine structural maintenance and inspection is a necessary requirement for all components of a WWTP. CDM's structural engineer has also provided recommendations based on the site visit that was made in early 2010 for necessary general structural maintenance of the WWTP. The recommended maintenance described in this memorandum is summarized in **Table 10** along with a recommended structural maintenance schedule for the facility. It should be noted that the maintenance recommended in this schedule is in addition to the routine maintenance already incurred by the WWTP.

Table 10
WWTP Structural Improvements Schedule

Improvement	Year			
	2011 - 2015	2015 - 2020	2020 - 2025	2025 - 2030
Headworks				
Removal of all spalled and/or cracked concrete	X			
Locate all active leaks, repair with grout as necessary	X	X	X	X
Inspection of interior of empty structure	X	X	X	X
Oxidation Ditch Structures				
Repair expansion joint filter material	X			
Repair minor leaks	X	X	X	X
Distribution Boxes				
Drain, clean, and inspect interior of structures	X	X	X	X
Clarifiers				
Drain, clean, and inspect interior of structures	X	X	X	X
Final Clarifier Pump Station #5 - #8				
Repair spalled concrete	X			
Apply surface dampproofing/ waterproofing to clarifier walls	X			
Clean and inspect pumps, piping, and structure	X	X	X	X
Blower Building				
Continue inspections on regular basis	X	X	X	X
Biosolids Facility				
Inspect, repair spalled concrete as necessary	X	X	X	X
UV Disinfection				
Replace rusted rebar	X			
Repair spalled concrete	X			
Drain channels/inspect structure	X	X	X	X

2.5 General Electrical Assessment

A general electrical assessment was performed by an electrical engineer during the site visit. In addition to the general condition of the equipment, recommendations for improvements have been developed as part of this assessment.

The existing electrical equipment throughout the facility is in good to fair condition based on initial visual observations. To provide reliable power distribution, it is recommended that the electrical system be designed with two independent sources of power and protection from common mode failures. These sources are generally two totally independent utility services, or a utility service and sufficient standby generator power. This approach is further outlined in the EPA Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability (EPA-430-99-74-001) paragraph 231.

Meeting the requirements of NFPA (National Fire Protection Association) 820 (Standard for Fire Protection in Wastewater Treatment and Collection Facilities) are also recommended for upgrades to the facility. NFPA 820 is a nationally recognized standard related to safeguarding against fire and explosion hazards specific to wastewater treatment plants, and the standard is also recognized by the National Electric Code. Based on initial visual observations, this facility does not currently meet the requirements for this standard. CDM recommends that any facility upgrades be designed and constructed to meet these standards. Work that is required for existing equipment and existing areas can be determined on a case by case basis in accordance with NFPA 820 and the National Electric Code.

Additionally, it is recommended that all electrical rooms be environmentally conditioned to extend the life of the electrical equipment and reduce maintenance. Process areas and structures containing electrical distribution equipment should be partitioned and air conditioned wherever possible; however, electrical improvements in existing areas will be evaluated on a case by case basis.

Once a final determination is made of the necessary process equipment upgrades and expansions, a more comprehensive evaluation of the electrical equipment is recommended. This assessment should be focused to determine the specific capabilities of the existing electrical distribution equipment with regard to meeting power requirements and throughout the facility.

3.0 Summary and Recommendations

This document provides a summary of information that was used to develop high level mechanical, structural, and electrical assessments of the equipment and facilities at Franklin's existing WWTP facility. A general Criticality rating for each piece of mechanical equipment has been provided that describes the general condition and criticality for replacement and this

information is provided in Attachment A and a summary of the recommended structural maintenance has also been provided and summarized in Section 2.5.9.

This analysis has developed the information necessary to assign criticality ratings and identification of equipment replacement needs necessary for continued operation of the WWTP. Recommended structural maintenance activities have also been identified. Detailed maintenance checklists for individual pieces of equipment have not been reviewed as part of this evaluation, but this process is recommended to aid the operations staff in guiding the overall maintenance of the WWTP. Development of a Computerized Maintenance Management System (CMMS) is recommended for addition to the City's overall IMS (Information Management System). A CMMS will summarize and create historical tracking for vehicle maintenance, work orders for process equipment, spare parts inventories, and necessary staff training for maintenance certifications as required. A CMMS will store all of this information, creating readily accessible files for staff while organizing the overall maintenance of the WWTP to create an additional level reliability.

Attachment A
City of Franklin, TN
Condition Assessment and Criticality Analysis

Column	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Major Facility Mane	Major Equipment Group	System	EQUIPMENT NUMBER	EQUIPMENT NAME	Condition Rating (0-5)	Reliability Rating	Probability of Failure - Reliability	Adjusted Effective Service Life	Estimated Remaining Useful Life	Estimated Remaining Useful Life (%)	Estimated Useful Life Already Consumed(%)	Aged Based Probability of Failure - Useful Life	Actual Probability of Failure	Level of Redundancy	Probability of Failure Based on Redundancy	Safety	Regulatory Violation	Fiscal Impact	Asset Critical Failure Mode	Criticality
CWWTP	Old Plant (Abandoned)	Clarifiers	NA	Clarifier-1	0	5	10	50	0	0%	100%	10	10	0%	10	1	0	0	1	10
			NA	Clarifier-2	0	5	10	50	0	0%	100%	10	10	0%	10	1	0	0	1	10
			NA	Clarifier-3	0	5	10	50	0	0%	100%	10	10	0%	10	1	0	0	1	10
			NA	Clarifier-4	0	5	10	50	0	0%	100%	10	10	0%	10	1	0	0	1	10
		Disinfection	NA	Disinfection	0	5	10	50	0	0%	100%	10	10	0%	10	1	0	0	1	10
		Tertiary Filters	NA	Tertiary Filters	0	5	10	30	0	0%	100%	10	10	0%	10	1	0	0	1	10
		Aeration Basins	NA	Aeration Basins	0	5	10	50	0	0%	100%	10	10	0%	10	1	0	0	1	10
		Return Sludge Pumping	NA	Return Sludge Pumping-1	0	5	10	20	0	0%	100%	10	10	0%	10	1	0	0	1	10
			NA	Return Sludge Pumping-2	0	5	10	20	0	0%	100%	10	10	0%	10	1	0	0	1	10
		Micro Screening	NA	Micro Screening	0	5	10	20	0	0%	100%	10	10	0%	10	1	0	0	1	10
		ATAD	NA	ATAD-1	0	5	10	20	0	0%	100%	10	10	0%	10	1	0	0	1	10
			NA	ATAD-2	0	5	10	20	0	0%	100%	10	10	0%	10	1	0	0	1	10
			NA	ATAD-3	0	5	10	20	0	0%	100%	10	10	0%	10	1	0	0	1	10
		Sludge Storage	NA	Sludge Storage-1	0	5	10	40	0	0%	100%	10	10	0%	10	1	0	0	1	10
			NA	Sludge Storage-2	0	5	10	40	0	0%	100%	10	10	0%	10	1	0	0	1	10
		Excess Flow Basins	NA	Excess Flow Basins-1	0	5	10	50	0	0%	100%	10	10	0%	10	1	0	0	1	10
			NA	Excess Flow Basins-2	0	5	10	50	0	0%	100%	10	10	0%	10	1	0	0	1	10
		Blower Building	NA	Blower Building-1	0	5	10	20	0	0%	100%	10	10	0%	10	1	0	0	1	10
		Filter Pump Station	NA	Filter Pump Station	0	5	10	20	0	0%	100%	10	10	0%	10	1	0	0	1	10
		Chlorine Contact Basins	NA	Chlorine Contact Basins-1	0	5	10	20	0	0%	100%	10	10	0%	10	1	0	0	1	10
			NA	Chlorine Contact Basins-2	0	5	10	20	0	0%	100%	10	10	0%	10	1	0	0	1	10
	Headwork's	Screening	DKN	Fine Screen No. 1	3	3	2	20	4	20%	80%	9	9	0%	9	2	2	3	7	63
			DKN	Fine Screen No. 2	3	3	2	20	4	20%	80%	9	9	0%	9	2	2	3	7	63
			DKN	Fine Screen Conveyor No. 1	3	3	2	20	4	20%	80%	9	9	0%	9	2	2	3	7	63
			DKN	Fine Screen Conveyor No. 2	3	3	2	20	4	20%	80%	9	9	0%	9	2	2	3	7	63
			DKN	Fine Screen Belt Conveyor	3	3	2	20	4	20%	80%	9	9	0%	9	2	2	3	7	63
		Grit Removal	DKN	Turbo Pump N0.1	4	4	5	20	2	10%	90%	10	10	0%	10	2	2	3	7	70
			DKN	Turbo Pump N0.2	4	4	5	20	2	10%	90%	10	10	0%	10	2	2	3	7	70
			DKN	Paddle Drive No.1	4	4	5	20	2	10%	90%	10	10	0%	10	2	2	3	7	70
			DKN	Paddle Drive No.2	4	4	5	20	2	10%	90%	10	10	0%	10	2	2	3	7	70
			DKN	Grit Washer No. 1	4	4	5	20	2	10%	90%	10	10	0%	10	2	2	3	7	70
			DKN	Grit Washer No. 2	4	4	5	20	2	10%	90%	10	10	0%	10	2	2	3	7	70
		Headworks Odor Control		Odor Control	3	3	2	15	3	20%	80%	9	9	0%	9	2	3	2	7	63
	Oxidation Basins	Oxidation Ditch No.1	AZM-1.1	Anoxic Mixer No.1	3	2	1	20	5	25%	75%	8.5	8.5	50%	4.25	2	3	3	8	34
			AZM-1.2	Anoxic Mixer No.2	3	2	1	20	5	25%	75%	8.5	8.5	50%	4.25	2	3	3	8	34
			AZA-1.1	AFD Surface Aerator No.1	3	2	1	20	5	25%	75%	8.5	8.5	50%	4.25	2	3	3	8	34
			AZA-1.2	Surface Aerator No.2	3	2	1	20	5	25%	75%	8.5	8.5	50%	4.25	2	3	3	8	34
			AZA-1.3	AFD Surface Aerator No.3	3	2	1	20	5	25%	75%	8.5	8.5	50%	4.25	2	3	3	8	34
			DKN	Course Air Diffusers	4	2	1	20	5	25%	75%	8.5	8.5	50%	4.25	2	3	3	8	34
			DKN	Submersible Pump	3	3	2	15	1	7%	93%	10	10	50%	5	2	1	1	4	20
		Oxidation Ditch No.2	AZM-2.1	Anoxic Mixer No.1	3	2	1	20	5	25%	75%	8.5	8.5	50%	4.25	2	3	3	8	34
			AZM-2.2	Anoxic Mixer No.2	3	2	1	20	5	25%	75%	8.5	8.5	50%	4.25	2	3	3	8	34
			AZA-2.1	AFD Surface Aerator No.1	3	2	1	20	5	25%	75%	8.5	8.5	50%	4.25	2	3	3	8	34
			AZA-2.2	Surface Aerator No.2	3	2	1	20	5	25%	75%	8.5	8.5	50%	4.25	2	3	3	8	34
			AZA-2.3	AFD Surface Aerator No.3	3	2	1	20	5	25%	75%	8.5	8.5	50%	4.25	2	3	3	8	34
			DKN	Course Air Diffusers	4	2	1	20	5	25%	75%	8.5	8.5	50%	4.25	2	3	3	8	34
			DKN	Submersible Pump	3	3	2	15	1	7%	93%	10	10	50%	5	2	1	1	4	20
		Oxidation Ditch No.3	AZM-3.1	Anoxic Mixer No.1	2	1	0	20	13	65%	35%	4	4	50%	2	2	3	3	8	16
			AZM-3.2	Anoxic Mixer No.2	2	1	0	20	13	65%	35%	4	4	50%	2	2	3	3	8	16
			AZA-3.1	AFD Surface Aerator No.1	2	1	0	20	13	65%	35%	4	4	50%	2	2	3	3	8	16

Attachment A
City of Franklin, TN
Condition Assessment and Criticality Analysis

Column	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Major Facility Mane	Major Equipment Group	System	EQUIPMENT NUMBER	EQUIPMENT NAME	Condition Rating (0-5)	Reliability Rating	Probability of Failure - Reliability	Adjusted Effective Service Life	Estimated Remaining Useful Life	Estimated Remaining Useful Life (%)	Estimated Useful Life Already Consumed(%)	Aged Based Probability of Failure - Useful Life	Actual Probability of Failure	Level of Redundancy	Probability of Failure Based on Redundancy	Safety	Regulatory Violation	Fiscal Impact	Asset Critical Failure Mode	Criticality
			AZA-3.2	Surface Aerator No.2	2	1	0	20	13	65%	35%	4	4	50%	2	2	3	3	8	16
			AZA-3.3	AFD Surface Aerator No.3	2	1	0	20	13	65%	35%	4	4	50%	2	2	3	3	8	16
			BDP-1	Submersible Pump	2	1	0	15	8	53%	47%	5	5	50%	2.5	2	1	1	4	10
		Aeration Blowers	BL-201	Centrifugal Blower	2	2	1	20	10	50%	50%	6	6	0%	6	1	1	2	4	24
			BL-202	Centrifugal Blower	2	2	1	20	10	50%	50%	6	6	0%	6	1	1	2	4	24
	Secondary Clarification	Clarifiers	Clarifier 5	Clarifier Drive 5	4	4	5	20	5	25%	75%	8.5	8.5	50%	4.25	1	2	2	5	21
			Clarifier 6	Clarifier Drive 6	4	4	5	20	5	25%	75%	8.5	8.5	50%	4.25	1	2	2	5	21
			Clarifier 7	Clarifier Drive 7	4	4	5	20	5	25%	75%	8.5	8.5	50%	4.25	1	2	2	5	21
			Clarifier 8	Clarifier Drive 8	2	2	1	20	15	75%	25%	3.5	3.5	50%	1.75	1	2	2	5	9
			Clarifier 9	Clarifier Drive 9	2	2	1	20	15	75%	25%	3.5	3.5	50%	1.75	1	2	2	5	9
			Clarifier 10	Clarifier Drive 10	2	2	1	20	15	75%	25%	3.5	3.5	50%	1.75	1	2	2	5	9
		Pump Station 5-8	S-7	RAS Pump S-7	4	4	5	20	5	25%	75%	8.5	8.5	0%	8.5	1	3	2	6	51
			S-8	RAS Pump S-8	4	4	5	20	5	25%	75%	8.5	8.5	0%	8.5	1	3	2	6	51
			S-9	RAS Pump S-9	4	4	5	20	5	25%	75%	8.5	8.5	0%	8.5	1	3	2	6	51
			S-10	RAS Pump S-10	4	4	5	20	5	25%	75%	8.5	8.5	0%	8.5	1	3	2	6	51
			S-11	RAS Pump S-11	4	4	5	20	5	25%	75%	8.5	8.5	0%	8.5	1	3	2	6	51
			WAS-5	WAS Pump -5	4	4	5	20	5	25%	75%	8.5	8.5	0%	8.5	1	3	2	6	51
			WAS-6	WAS Pump -6	4	4	5	20	5	25%	75%	8.5	8.5	0%	8.5	1	3	2	6	51
			SC-1	Scum Pump-1	4	4	5	20	5	25%	75%	8.5	8.5	0%	8.5	1	3	2	6	51
			SC-2	Scum Pump-2	4	4	5	20	5	25%	75%	8.5	8.5	0%	8.5	1	3	2	6	51
			FC-5	Sump Pump FC-5	4	4	5	15	1	7%	93%	10	10	0%	10	1	1	1	3	30
		Pump Station 9-10	S-12	RAS Pump 12	2	2	1	20	15	75%	25%	3.5	3.5	0%	3.5	1	3	2	6	21
			S-13	RAS Pump 13	2	2	1	20	15	75%	25%	3.5	3.5	0%	3.5	1	3	2	6	21
			S-14	RAS Pump 14	2	2	1	20	15	75%	25%	3.5	3.5	0%	3.5	1	3	2	6	21
			WAS-7	WAS Pump 7	2	2	1	20	15	75%	25%	3.5	3.5	0%	3.5	1	3	2	6	21
			WAS-8	WAS Pump 8	2	2	1	20	15	75%	25%	3.5	3.5	0%	3.5	1	3	2	6	21
			SC-3	Scum Pump 3	2	2	1	20	15	75%	25%	3.5	3.5	0%	3.5	1	3	2	6	21
			SC-4	Scum Pump 4	2	2	1	20	15	75%	25%	3.5	3.5	0%	3.5	1	3	2	6	21
			FC-9	Sump Pump FC-9	2	2	1	15	8	53%	47%	5	5	0%	5	1	1	1	3	15
	Influent Pumping	Influent Pump Station	DKN	Influent Pump 1	1	2	1	20	15	75%	25%	3.5	3.5	50%	1.75	2	3	3	8	14
			DKN	Influent Pump 2	1	2	1	20	15	75%	25%	3.5	3.5	50%	1.75	2	3	3	8	14
			DKN	Influent Pump 3	1	2	1	20	15	75%	25%	3.5	3.5	50%	1.75	2	3	3	8	14
			DKN	Influent Pump 4	1	2	1	20	15	75%	25%	3.5	3.5	50%	1.75	2	3	3	8	14
			DKN	Influent Pump 5	1	2	1	20	15	75%	25%	3.5	3.5	50%	1.75	2	3	3	8	14
	Process Water	Process Water Pluming		Process Water Pump-1	3	3	2	20	5	25%	75%	8.5	8.5	0%	8.5	1	1	1	3	26
				Process Water Pump-2	3	3	2	20	5	25%	75%	8.5	8.5	0%	8.5	1	1	1	3	26
				Process Water Pump-3	3	3	2	20	5	25%	75%	8.5	8.5	0%	8.5	1	1	1	3	26
				Process Water Pump-4	3	3	2	20	5	25%	75%	8.5	8.5	0%	8.5	1	1	1	3	26
	Filtration	TETRA Filters		Dentirification Filter-1	1	1	0	50	40	80%	20%	2	2	50%	1	2	3	3	8	8
				Dentirification Filter-2	1	1	0	50	40	80%	20%	2	2	50%	1	2	3	3	8	8
				Dentirification Filter-3	1	1	0	50	40	80%	20%	2	2	50%	1	2	3	3	8	8
				Dentirification Filter-4	1	1	0	50	40	80%	20%	2	2	50%	1	2	3	3	8	8
				Dentirification Filter-5	1	1	0	50	40	80%	20%	2	2	50%	1	2	3	3	8	8
		Backwash Air Blowers	BAB-1	Backwash Air Blower-1	2	2	1	20	15	75%	25%	3.5	3.5	50%	1.75	2	3	3	8	14
			BAB-2	Backwash Air Blower-2	2	2	1	20	15	75%	25%	3.5	3.5	50%	1.75	2	3	3	8	14
			BAB-3	Backwash Air Blower-3	2	2	1	20	15	75%	25%	3.5	3.5	50%	1.75	2	3	3	8	14
		Backwash Water Pumping	BWP-1	Backwash Water Pump-1	2	2	1	20	15	75%	25%	3.5	3.5	0%	3.5	2	3	3	8	28
			BWP-2	Backwash Water Pump-2	2	2	1	20	15	75%	25%	3.5	3.5	0%	3.5	2	3	3	8	28
		Methanol System		Methane Storage Tank	3	2	1	15	8	53%	47%	5	5	0%	5	3	3	3	9	45
			MMP-1	Methane Feed Pump-1	4	3	2	15	8	53%	47%	5	5	0%	5	3	3	3	9	45

Attachment A
City of Franklin, TN
Condition Assessment and Criticality Analysis

Column	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Major Facility Mane	Major Equipment Group	System	EQUIPMENT NUMBER	EQUIPMENT NAME	Condition Rating (0-5)	Reliability Rating	Probability of Failure - Reliability	Adjusted Effective Service Life	Estimated Remaining Useful Life	Estimated Remaining Useful Life (%)	Estimated Useful Life Already Consumed(%)	Aged Based Probability of Failure - Useful Life	Actual Probability of Failure	Level of Redundancy	Probability of Failure Based on Redundancy	Safety	Regulatory Violation	Fiscal Impact	Asset Critical Failure Mode	Criticality
			MMP-2	Methane Feed Pump-2	4	3	2	15	8	53%	47%	5	5	0%	5	3	3	3	9	45
		Dry Well Sump	SU-1A	Dry Well Sum Pump-1	2	2	1	15	8	53%	47%	5	5	0%	5	2	3	3	8	40
			SU-2A	Dry Well Sum Pump-2	2	2	1	15	8	53%	47%	5	5	0%	5	2	3	3	8	40
	Post Aeration	Post Aeration Basin		Post Aeration Basin	1	2	1	50	40	80%	20%	2	2	0%	2	2	3	2	7	14
		Post Aeration Blowers	BL-101	Post Aeration Blower-1	2	2	1	20	15	75%	25%	3.5	3.5	50%	1.75	2	3	3	8	14
			BL-102	Post Aeration Blower-2	2	2	1	20	15	75%	25%	3.5	3.5	50%	1.75	2	3	3	8	14
			BL-103	Post Aeration Blower-3	2	2	1	20	15	75%	25%	3.5	3.5	50%	1.75	2	3	3	8	14
				Fine Air Diffusers	2	3	2	10	5	50%	50%	6	6	0%	6	2	3	3	8	48
	Disinfection	UV Disinfection System		UV Disinfection-1	3	4	5	10	5	50%	50%	6	6	50%	3	2	3	3	8	24
				UV Disinfection-2	3	4	5	10	5	50%	50%	6	6	50%	3	2	3	3	8	24
				UV Disinfection-3	3	4	5	10	5	50%	50%	6	6	50%	3	2	3	3	8	24
				UV Disinfection-4	3	4	5	10	5	50%	50%	6	6	50%	3	2	3	3	8	24
	Water Reclamation	Reclamation Pumping		Reclamation Pump-1	4	4	5	20	2	10%	90%	10	10	0%	10	1	2	2	5	50
				Reclamation Pump-2	4	4	5	20	2	10%	90%	10	10	0%	10	1	2	2	5	50
				Reclamation Pump-3	0	NIS		20	0	0%	100%	10				1	2	2	5	0
				Reclamation Pump-4	4	4	5	20	2	10%	90%	10	10	0%	10	1	2	2	5	50
	Thickening	DAFT	DAFT-1	Dissolved Air Floatation Tank-1	3	4	5	50	10	20%	80%	9	9	0%	9	2	3	3	8	72
				Floatator Drive-1	3	4	5	20	3	15%	85%	9	9	0%	9	2	3	2	7	63
			DAFT-2	Dissolved Air Floatation Tank-2	3	4	5	50	10	20%	80%	9	9	0%	9	2	3	3	8	72
				Floatator Drive-2	3	4	5	20	3	15%	85%	9	9	0%	9	2	3	2	7	63
				Recycle Pump-1	4	3	2	15	3	20%	80%	9	9	0%	9	2	3	3	8	72
				Recycle Pump-2	4	3	2	15	3	20%	80%	9	9	0%	9	2	3	3	8	72
				Air Compressor-1	3	3	2	15	8	53%	47%	5	5	0%	5	3	3	3	9	45
				Air Compressor-2	4	3	2	15	3	20%	80%	9	9	0%	9	3	3	3	9	81
			TSP-1	Thickened Sludge Pump-1	4	3	2	15	3	20%	80%	9	9	0%	9	2	3	3	8	72
			TSP-2	Thickened Sludge Pump-2	4	3	2	15	3	20%	80%	9	9	0%	9	2	3	3	8	72
				Pressurization Tank-1	3	3	2	15	3	20%	80%	9	9	0%	9	3	3	3	9	81
				Pressurization Tank-2	3	3	2	15	3	20%	80%	9	9	0%	9	3	3	3	9	81
	Belt Filter Press	BFP Feed Tank		BFP Feed Tank	4	3	2	15	3	20%	80%	9	9	0%	9	3	3	3	9	81
			BL-201	Blower-1	3	3	2	20	4	20%	80%	9	9	50%	4.5	2	2	3	7	32
			BL-202	Blower-2	3	3	2	20	4	20%	80%	9	9	50%	4.5	2	2	3	7	32
			BL-203	Blower-3	3	3	2	20	4	20%	80%	9	9	50%	4.5	2	2	3	7	32
				BFP Feed Tank Course Air Diffusers	4	3	2	15	3	20%	80%	9	9	0%	9	3	3	3	9	81
		Belt filter Press	BFP-1	Belt Filter Press-1	4	3	2	20	5	25%	75%	8.5	8.5	0%	8.5	3	3	3	9	77
				Hydraulic Unit	4	3	2	20	5	25%	75%	8.5	8.5	0%	8.5	3	3	3	9	77
				Wash Water Pump	4	3	2	20	5	25%	75%	8.5	8.5	0%	8.5	2	3	3	8	68
				BFP Feed Pump	4	3	2	20	5	25%	75%	8.5	8.5	0%	8.5	3	3	3	9	77
			BFP-2	Belt Filter Press-2	4	3	2	20	5	25%	75%	8.5	8.5	0%	8.5	3	3	3	9	77
				Hydraulic Unit	4	3	2	20	5	25%	75%	8.5	8.5	0%	8.5	3	3	3	9	77
				Wash Water Pump	4	3	2	20	5	25%	75%	8.5	8.5	0%	8.5	2	3	3	8	68
				BFP Feed Pump	4	3	2	20	5	25%	75%	8.5	8.5	0%	8.5	3	3	3	9	77
				Filter Press Conveyor	3	3	2	20	5	25%	75%	8.5	8.5	0%	8.5	2	2	2	6	51
		Polymer System		Polymer System	4	3	2	10	3	30%	70%	8	8	0%	8	3	3	3	9	72
		Solids Odor Control		Odor Control	3	3	2	15	3	20%	80%	9	9	0%	9	2	3	2	7	63
		ATAD Odor Control		Odor Control	0	3	2	15	3	20%	80%	9	9	0%	9	1	1	1	3	27

Attachment B
City of Franklin, TN
WWTP Equipment Replacement Prioritization

Column	1	2	3	4	5	6
Process Area	System	Equipment Name	Condition Rating	Remaining Useful Life	Criticality	Replacement Priority
Headworks	Screening	Fine Screen No. 1	3	4	63	189
Headworks	Screening	Fine Screen No. 2	3	4	63	189
Headworks	Screening	Fine Screen Conveyor No. 1	3	4	63	189
Headworks	Screening	Fine Screen Conveyor No. 2	3	4	63	189
Headworks	Screening	Fine Screen Belt Conveyor	3	4	63	189
Headworks	Grit Removal	Turbo Pump N0.1	4	2	70	280
Headworks	Grit Removal	Turbo Pump N0.2	4	2	70	280
Headworks	Grit Removal	Paddle Drive No.1	4	2	70	280
Headworks	Grit Removal	Paddle Drive No.2	4	2	70	280
Headworks	Grit Removal	Grit Washer No. 1	4	2	70	280
Headworks	Grit Removal	Grit Washer No. 2	4	2	70	280
Headworks	Odor Control	Odor Control	3	3	63	189
Oxidation Basins	Oxidation Ditch No.1	Anoxic Mixer No.1	3	5	34	102
Oxidation Basins	Oxidation Ditch No.1	Anoxic Mixer No.2	3	5	34	102
Oxidation Basins	Oxidation Ditch No.1	AFD Surface Aerator No.1	3	5	34	102
Oxidation Basins	Oxidation Ditch No.1	Surface Aerator No.2	3	5	34	102
Oxidation Basins	Oxidation Ditch No.1	AFD Surface Aerator No.3	3	5	34	102
Oxidation Basins	Oxidation Ditch No.1	Course Air Diffusers	4	5	34	136
Oxidation Basins	Oxidation Ditch No.1	Submersible Pump	3	1	20	60
Oxidation Basins	Oxidation Ditch No.2	Anoxic Mixer No.1	3	5	34	102
Oxidation Basins	Oxidation Ditch No.2	Anoxic Mixer No.2	3	5	34	102
Oxidation Basins	Oxidation Ditch No.2	AFD Surface Aerator No.1	3	5	34	102
Oxidation Basins	Oxidation Ditch No.2	Surface Aerator No.2	3	5	34	102
Oxidation Basins	Oxidation Ditch No.2	AFD Surface Aerator No.3	3	5	34	102
Oxidation Basins	Oxidation Ditch No.2	Course Air Diffusers	4	5	34	136
Oxidation Basins	Oxidation Ditch No.2	Submersible Pump	3	1	20	60
Oxidation Basins	Oxidation Ditch No.3	Anoxic Mixer No.1	2	13	16	32
Oxidation Basins	Oxidation Ditch No.3	Anoxic Mixer No.2	2	13	16	32
Oxidation Basins	Oxidation Ditch No.3	AFD Surface Aerator No.1	2	13	16	32
Oxidation Basins	Oxidation Ditch No.3	Surface Aerator No.2	2	13	16	32
Oxidation Basins	Oxidation Ditch No.3	AFD Surface Aerator No.3	2	13	16	32

Attachment B
City of Franklin, TN
WWTP Equipment Replacement Prioritization

Process Area	System	Equipment Name	Condition Rating	Remaining Useful Life	Criticality	Replacement Priority
Oxidation Basins	Oxidation Ditch No.3	Submersible Pump	2	8	10	20
Oxidation Basins	Aeration Blowers	Centrifugal Blower	2	10	24	48
Oxidation Basins	Aeration Blowers	Centrifugal Blower	2	10	24	48
Secondary Clarification	Clarifiers	Clarifier Drive 5	4	5	21	84
Secondary Clarification	Clarifiers	Clarifier Drive 6	4	5	21	84
Secondary Clarification	Clarifiers	Clarifier Drive 7	4	5	21	84
Secondary Clarification	Clarifiers	Clarifier Drive 8	2	15	9	18
Secondary Clarification	Clarifiers	Clarifier Drive 9	2	15	9	18
Secondary Clarification	Clarifiers	Clarifier Drive 10	2	15	9	18
Secondary Clarification	Clarifiers	RAS Pump S-7	4	5	51	204
Secondary Clarification	Pump Station 5-8	RAS Pump S-8	4	5	51	204
Secondary Clarification	Pump Station 5-8	RAS Pump S-9	4	5	51	204
Secondary Clarification	Pump Station 5-8	RAS Pump S-10	4	5	51	204
Secondary Clarification	Pump Station 5-8	RAS Pump S-11	4	5	51	204
Secondary Clarification	Pump Station 5-8	WAS Pump -5	4	5	51	204
Secondary Clarification	Pump Station 5-8	WAS Pump -6	4	5	51	204
Secondary Clarification	Pump Station 5-8	Scum Pump-1	4	5	51	204
Secondary Clarification	Pump Station 5-8	Scum Pump-2	4	5	51	204
Secondary Clarification	Pump Station 5-8	Sump Pump FC-5	4	1	30	120
Secondary Clarification	Pump Station 9-10	RAS Pump 12	2	15	21	42
Secondary Clarification	Pump Station 9-10	RAS Pump 13	2	15	21	42
Secondary Clarification	Pump Station 9-10	RAS Pump 14	2	15	21	42
Secondary Clarification	Pump Station 9-10	WAS Pump 7	2	15	21	42
Secondary Clarification	Pump Station 9-10	WAS Pump 8	2	15	21	42
Secondary Clarification	Pump Station 9-10	Scum Pump 3	2	15	21	42
Secondary Clarification	Pump Station 9-10	Scum Pump 4	2	15	21	42
Secondary Clarification	Pump Station 9-10	Sump Pump FC-9	2	8	15	30
Influent Pumping	Influent Pump Station	Influent Pump 1	1	15	14	14
Influent Pumping	Influent Pump Station	Influent Pump 2	1	15	14	14
Influent Pumping	Influent Pump Station	Influent Pump 3	1	15	14	14
Influent Pumping	Influent Pump Station	Influent Pump 4	1	15	14	14
Influent Pumping	Influent Pump Station	Influent Pump 5	1	15	14	14

Attachment B
City of Franklin, TN
WWTP Equipment Replacement Prioritization

Process Area	System	Equipment Name	Condition Rating	Remaining Useful Life	Criticality	Replacement Priority
Process Water	Process Water Pluming	Process Water Pump-1	3	5	26	78
Process Water	Process Water Pluming	Process Water Pump-2	3	5	26	78
Process Water	Process Water Pluming	Process Water Pump-3	3	5	26	78
Process Water	Process Water Pluming	Process Water Pump-4	3	5	26	78
Filtration	TETRA Filters	Dentirification Filter-1	1	40	8	8
Filtration	TETRA Filters	Dentirification Filter-2	1	40	8	8
Filtration	TETRA Filters	Dentirification Filter-3	1	40	8	8
Filtration	TETRA Filters	Dentirification Filter-4	1	40	8	8
Filtration	TETRA Filters	Dentirification Filter-5	1	40	8	8
Filtration	Backwash Air Blowers	Backwash Air Blower-1	2	15	14	28
Filtration	Backwash Air Blowers	Backwash Air Blower-2	2	15	14	28
Filtration	Backwash Air Blowers	Backwash Air Blower-3	2	15	14	28
Filtration	Backwash Water Pumping	Backwash Water Pump-1	2	15	28	56
Filtration	Backwash Water Pumping	Backwash Water Pump-2	2	15	28	56
Filtration	Methanol System	Methane Storage Tank	3	8	45	135
Filtration	Methanol System	Methane Feed Pump-1	4	8	45	180
Filtration	Methanol System	Methane Feed Pump-2	4	8	45	180
Filtration	Dry Well Sump	Dry Well Sum Pump-1	2	8	40	80
Filtration	Dry Well Sump	Dry Well Sum Pump-2	2	8	40	80
Post Aeration	Post Aeration Basin	Post Aeration Basin	1	40	14	14
Post Aeration	Post Aeration Blowers	Post Aeration Blower-1	2	15	14	28
Post Aeration	Post Aeration Blowers	Post Aeration Blower-2	2	15	14	28
Post Aeration	Post Aeration Blowers	Post Aeration Blower-3	2	15	14	28
Post Aeration	Post Aeration Blowers	Fine Air Diffusers	2	5	48	96
Disinfection	UV Disinfection System	UV Disinfection-1	3	5	24	72
Disinfection	UV Disinfection System	UV Disinfection-2	3	5	24	72
Disinfection	UV Disinfection System	UV Disinfection-3	3	5	24	72
Disinfection	UV Disinfection System	UV Disinfection-4	3	5	24	72
Water Reclamation	Reclamation Pumping	Reclamation Pump-1	4	2	50	200
Water Reclamation	Reclamation Pumping	Reclamation Pump-2	4	2	50	200
Water Reclamation	Reclamation Pumping	Reclamation Pump-3	4	2	50	200
Water Reclamation	Reclamation Pumping	Reclamation Pump-4	4	2	50	200

Attachment B
City of Franklin, TN
WWTP Equipment Replacement Prioritization

Process Area	System	Equipment Name	Condition Rating	Remaining Useful Life	Criticality	Replacement Priority
Thickening	DAFT	Dissolved Air Floatation Tank-1	3	10	72	216
Thickening	DAFT	Floatator Drive-1	3	3	63	189
Thickening	DAFT	Dissolved Air Floatation Tank-2	3	10	72	216
Thickening	DAFT	Floatator Drive-2	3	3	63	189
Thickening	DAFT	Recycle Pump-1	4	3	72	288
Thickening	DAFT	Recycle Pump-2	4	3	72	288
Thickening	DAFT	Air Compressor-1	3	8	45	135
Thickening	DAFT	Air Compressor-2	4	3	81	324
Thickening	DAFT	Thickened Sludge Pump-1	4	3	72	288
Thickening	DAFT	Thickened Sludge Pump-2	4	3	72	288
Thickening	DAFT	Pressurization Tank-1	3	3	81	243
Thickening	DAFT	Pressurization Tank-2	3	3	81	243
Belt Filter Press	BFP Feed Tank	BFP Feed Tank	4	3	81	324
Belt Filter Press	BFP Feed Tank	Blower-1	3	4	32	96
Belt Filter Press	BFP Feed Tank	Blower-2	3	4	32	96
Belt Filter Press	BFP Feed Tank	Blower-3	3	4	32	96
Belt Filter Press	BFP Feed Tank	BFP Feed Tank Coarse Air Diffusers	4	3	81	324
Belt Filter Press	Belt filter Press	Belt Filter Press-1	4	5	77	308
Belt Filter Press	Belt filter Press	Hydraulic Unit	4	5	77	308
Belt Filter Press	Belt filter Press	Wash Water Pump	4	5	68	272
Belt Filter Press	Belt filter Press	BFP Feed Pump	4	5	77	308
Belt Filter Press	Belt filter Press	Belt Filter Press-2	4	5	77	308
Belt Filter Press	Belt filter Press	Hydraulic Unit	4	5	77	308
Belt Filter Press	Belt filter Press	Wash Water Pump	4	5	68	272
Belt Filter Press	Belt filter Press	BFP Feed Pump	4	5	77	308
Belt Filter Press	Belt filter Press	Filter Press Conveyor	3	5	51	153
Belt Filter Press	Solids Odor Control	Polymer System	4	3	72	288
Belt Filter Press	ATAD Odor Control	Odor Control	3	3	63	189
Belt Filter Press		Odor Control	0	3	27	0

Memorandum

To: *City of Franklin IWRP Team*

From: *CDM Smith, Inc.*

Date: *February 2012*

Subject: *Wastewater Collection System Technical Analysis*

Executive Summary

The Wastewater Collection System technical analysis for the Integrated Water Resource Plan (IWRP) includes evaluation of the system inflow and infiltration (I/I) used for identification of collection system areas of concern, evaluation of accepting wastewater that is currently routed to other wastewater treatment plants outside of the City's current service area, planning for connection of septic users within the urban growth boundary, and evaluation of options that will allow wastewater to be conveyed to the treatment plant or plants through the IWRP planning period.

The I/I analysis included flow monitoring throughout the collection system; data analysis allowed identification of areas of the collection system that were further investigated. Additional testing has been recommended for these areas to determine the best improvement option, such as rehabilitation or replacement of the infrastructure. Cost estimates for these improvements depend on the extents required, but may be in the range of $9 to $31 million, plus an additional $1.7 million annually, for maintenance.

The IWRP process includes an evaluation of accepting wastewater flows from the Berry's Chapel Utility Incorporated (BCUI, formerly Lynwood Utility Corporation) as well as from the Cartwright Creek, LLC system are included as part of this study; these options evaluate construction of a lift station and force main directly to the Franklin WWTP. The cost for providing the necessary infrastructure to capture wastewater flows currently routed to BCUI has been estimated at $5,150,000; and the cost for the Cartwright Creek is $5,460,000 (Butler).

Several areas within Franklin's urban growth boundary include residences utilizing septic systems; there are approximately 2000 residential units that could be connected to the City's collection system. Residences with septic systems that voluntarily connect to the sewer must finance the connection. Therefore, the cost to the city should be negligible (Butler).

The IWRP also includes evaluation of wastewater treatment plant (WWTP) options to meet projected, future wastewater flows. In order to fully evaluate the WWTP options, it is important to define the options and associated costs for conveyance of wastewater flows to the plants. If a second WWTP is not constructed in the southern portion of the City's service area, additional infrastructure would be required to convey future wastewater flows from the south the existing WWTP. The cost estimate of for construction of infrastructure to accommodate flows in excess of 10 mgd from the southern part of the service area is $25,290,000 (Butler).

1.0 Introduction

The primary focus and objective of Phase II of the City of Franklin's Integrated Water Resources Plan (IWRP) is to identify a plan that includes a portfolio of projects allowing the City to meet future water resources demands. Part of meeting wastewater demands includes identifying the long-term needs of the system within the Urban Growth Boundary (UGB) so that efficient conveyance of wastewater to treatment facilities can be provided over the planning period. In order to develop a long-term plan for the collection system, the IWRP includes development of information with regard to four subtasks comprising the collection system evaluation:

- Identification of collection system areas of concern by means of collection system inflow and infiltration analysis
- Development of a plan and costs for conveyance improvements to collect wastewater currently routed to the existing plants at Cartwright Creek and BCUI WWTPs, if these options are identified as part of the final IWRP
- Analysis of the potential for connecting septic users to the City's sewer collection system, within the urban growth boundary
- Preliminary plan for collection system improvements and projects if the existing WWTP is upgraded, and a new WWTP is not constructed in the southern portion of the service area.

2.0 Collection System Assessment

The objective of the collection system assessment is to determine if, where, and to what extent a reduction in wet-weather wastewater flows may be possible. In any collection system, a certain amount of inflow and infiltration (I/I) enters the sewers in addition to wastewater being conveyed. Some of this flow is constant inflow from groundwater or surface water sources, and some flow increases dramatically following rain events. These additional flows must be conveyed to the WWTP, which adds expense at lift stations and at the WWTP. In addition, the excess flow volume in the collection system can cause wet-weather overflows, which are a violation of the Clean Water Act. This section provides a description of the process for characterizing the City's sewer flows and presents areas prioritized for further investigation and for improvements.

2.1 Collection System Description

The City of Franklin's wastewater collection system has approximately 300 miles of gravity pipe, ranging in diameter from 6-inch to 54-inch. Associated with the gravity system are 7,500 manholes. In addition, the system has 18 miles of force main, ranging in diameter from 4-inch to 16-inch, which convey the wastewater flow from 24 lift stations located throughout the collection system. The City of Franklin's sewered area covers approximately 14,000 acres. An overview of the collection system is shown in **Figure 2-1**.

In order to identify problem areas with overflows or capacity issues that should be addressed with any future wastewater system improvements, CDM has reviewed existing studies, planning documents, conducted interviews with staff, and conducted an analysis of information collected during a recent flow monitoring study.

2.2 Collection System Flows

The purpose of monitoring flow within a collection system is to determine the characteristics of the sewer flow. Flow monitors measure the depth and velocity of flow in the gravity pipes, and from these measurements and knowledge of pipe geometry, flow is calculated and reported for the duration of the monitoring period.

In general, wastewater flows can be divided into three components: base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall dependent infiltration/inflow (RDII). The wet weather component, RDII, is of particular importance because it is the increased sewer system flow that occurs during a rainfall event. Consequently, hydrograph decomposition is performed on the flow data to distinguish BWF and GWI from RDII. Results of the hydrograph decomposition are utilized to evaluate existing conditions within sewersheds and to make comparisons to determine areas of concern. The components of the hydrograph are described below and illustrated in **Figure 2-2**.

2.2.1 Base Wastewater Flow

Base wastewater flow is domestic wastewater from residential, commercial, and institutional (schools, churches, hospitals, etc.) sources, as well as industrial wastewater sources. It is affected by the population and land uses in an area and varies throughout the day, in response to personal habits and business operations.

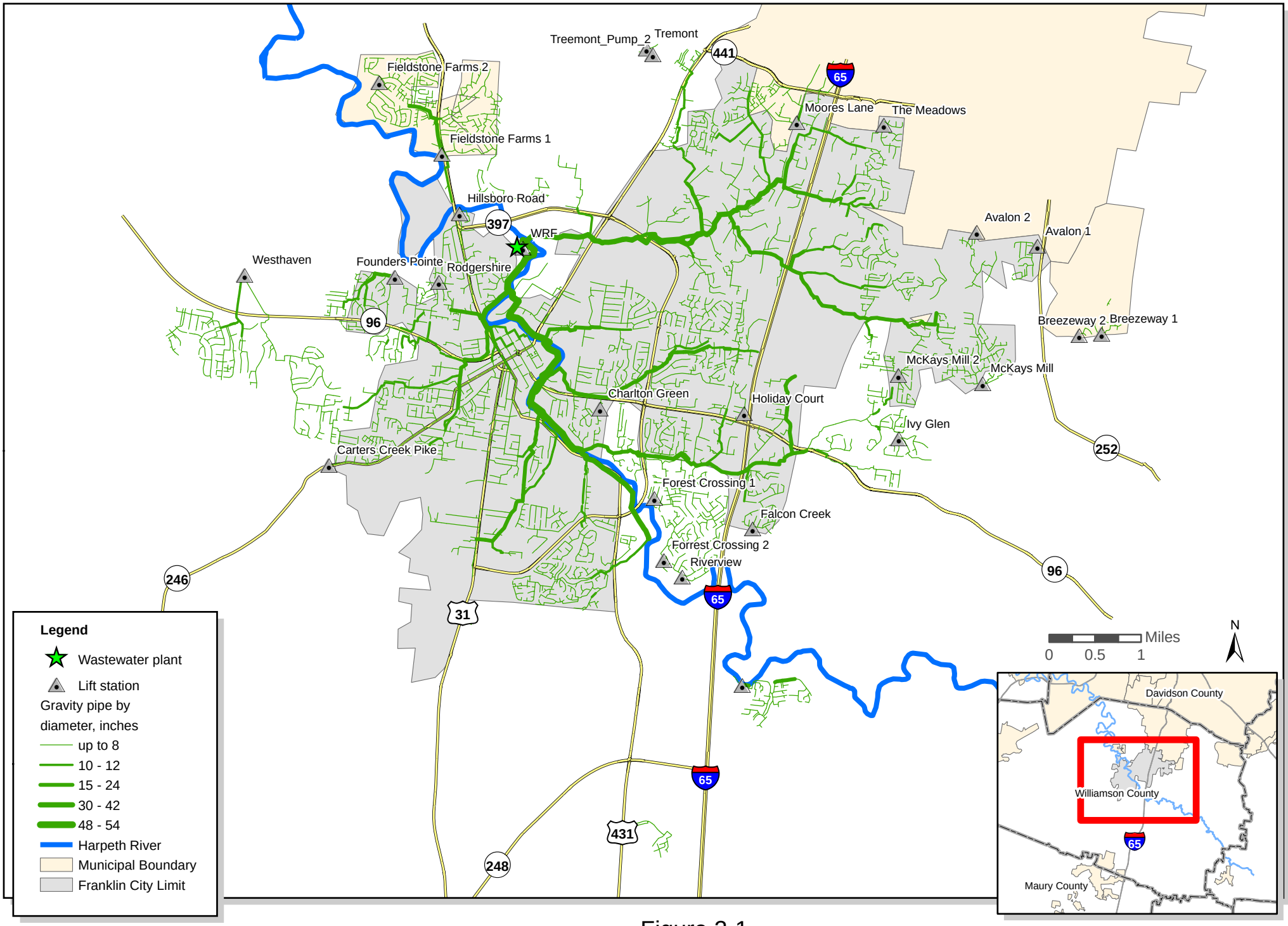

Figure 2-1
Franklin Collection System Overview

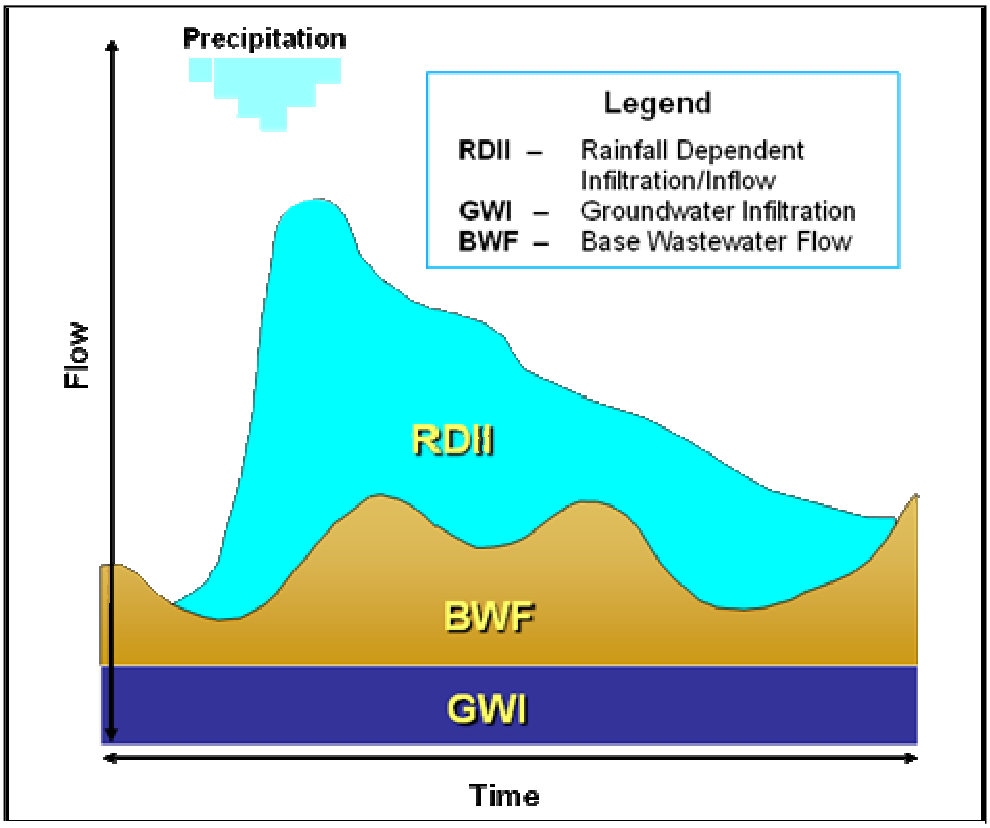

Figure 2-2
Wastewater Flow Hydrograph Components

2.2.2 Groundwater Infiltration

Groundwater infiltration is defined as groundwater entering the collection system through defective pipes, pipe joints, and manhole walls. The magnitude of GWI depends on the depth of the groundwater table above the pipelines, the percentage of the system that is submerged, and the physical condition of the sewer system. Variation in groundwater levels is seasonal, which typically results in seasonal variation in GWI. Groundwater level is affected by rainfall and other hydrologic factors and typically responds gradually; therefore, influence on GWI due to individual rainfall events is typically not evaluated. GWI is evidenced by a general increase in wastewater flow that persists for periods of many days or weeks.

2.2.3 Rainfall Dependent Infiltration/Inflow

Rainfall dependent I/I (RDII) refers to stormwater that enters the sanitary sewer system in direct response to the intensity and duration of rainfall events. RDII includes stormwater entering the collection system through direct connections, such as roof downspouts illegally connected to the

sanitary sewers, yard and area drains, holes in manhole covers, cross-connections with storm drains, or catch basins. RDII also includes the flow that results from a delayed response of stormwater that enters the collection system through defective pipes, pipe joints, and manhole walls after percolating through the soil.

2.3 Flow Monitoring Study

Compliance Envirosystems, LLC (CES) was contracted to collect flow monitoring data in the Franklin collection system from February 26th through April 28th, 2011. Thirty-one flow monitors and 11 rain gauges were installed at strategic locations within the collection system; these locations are shown in **Figure 2-3**. This figure also shows delineated sewershed areas, that is, the sewered area contributing flow to each installed flow monitor.

Data collected from flow monitors during the monitoring period was analyzed to separate the RDII component from the base wastewater flow and groundwater infiltration via hydrograph decomposition. This separation analysis was performed using the SSOAP toolbox, which is accessible as freeware from the US Environmental Protection Agency (EPA) website at the following location: http://www.epa.gov/nrmrl/wswrd/wq/models/ssoap/.

Five rain events were selected for analysis, as summarized in **Table 2-1**. These rain events, which occurred during the 60-day flow monitoring period, ranged from 1.10 to 1.47 inches, averaged across all installed rain gauges (Figure 2-3). The RDII pattern is a function of the rainfall volume and intensity, and therefore varies with each rainfall event. The R-value is used to represent the RDII volume as a function of the rainfall and contributing sewered area; it is defined as the percent of rainfall entering the collection system as RDII. Using SSOAP, the R-value was calculated for each flow monitor, for each of the 5 rain events. A rainfall-weighted R-value was then found for each flow monitor by using the R-values and rainfall volume for these five events. Finally, the incremental R-value was calculated to identify the R-value of the area isolated by each respective flow monitor. **Table 2-2** lists the flow monitors, their contributing sewered areas, and the resulting incremental R-value. **Figure 2-4** shows the color coded R-value for each flow monitor and associated sewershed.

Table 2-1: Selected Rainfall Events

Rain event date	Average total rainfall volume, inches	Average peak rainfall intensity, in/hr
February 28, 2011	1.10	0.78
March 5, 2011	1.20	0.48
April 4, 2011	1.38	0.34
April 11, 2011	1.23	0.66
April 15, 2011	1.47	0.72

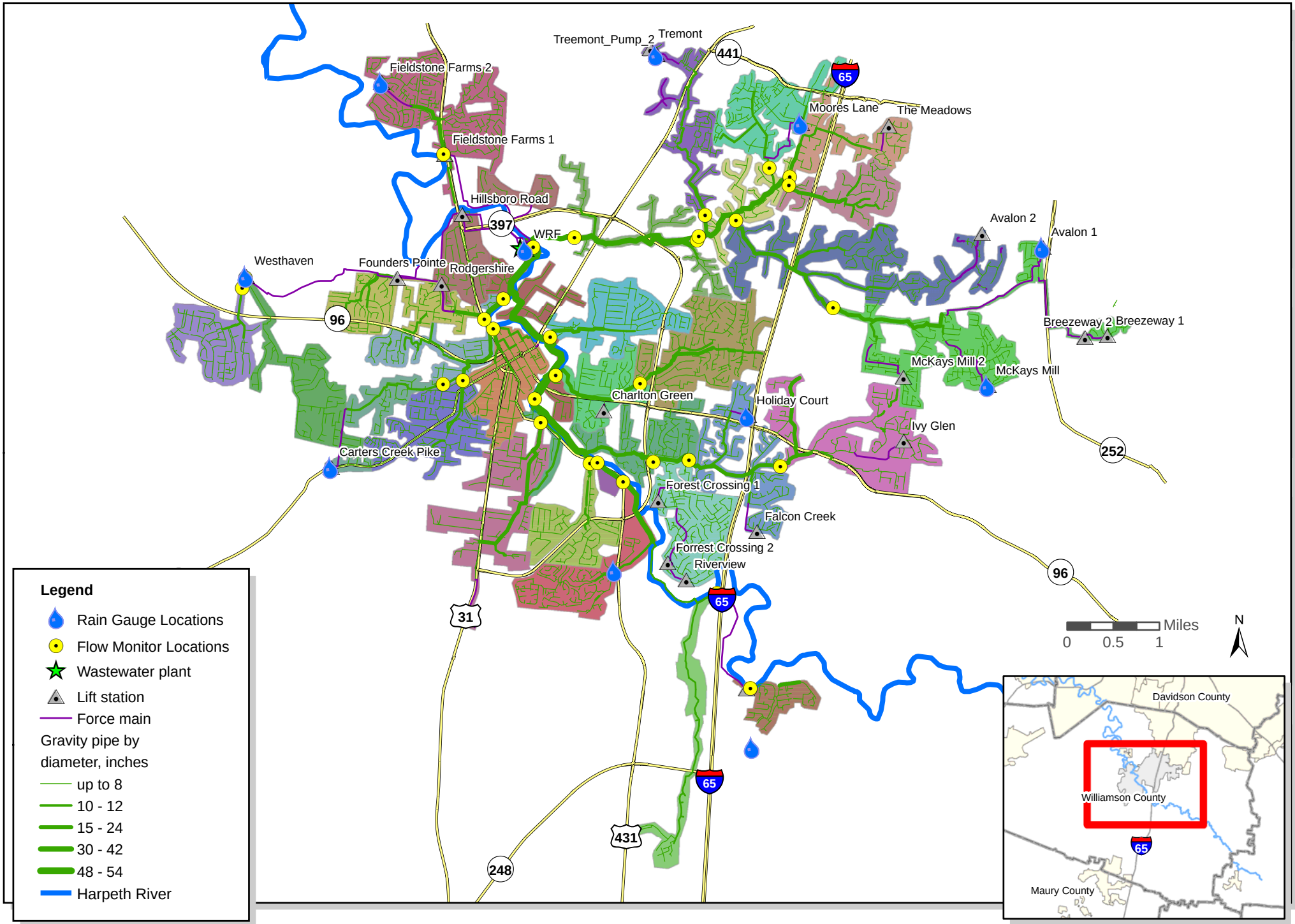

Figure 2-3
Franklin Sewersheds, Flow Monitors, and Rain Gauges

Table 2-2: Flow Monitors and Incremental Areas and R-Values

Flow Monitor	Incremental Area, Acres	Incremental R-value
FM1	710	1.3%
FM2	492	1.8%
FM3	510	0.2%
FM4	491	15.2%
FM5	625	9.1%
FM6A	374	0.7%
FM6B*	41	-
FM6C	483	1.2%
FM7	331	2.8%
FM8	327	3.0%
FM9	414	7.3%
FM10	444	2.5%
FM11	431	10.6%
FM12	354	21.2%
FM13	837	0.3%
FM14	886	0.5%
FM15	487	5.3%
FM16A	349	1.1%
FM16B	325	0.9%
FM17	359	0.9%
FM19	300	4.1%
FM20a	386	0.3%
FM21	746	2.4%
FM22	239	9.2%
FM23	679	6.1%
FM24a	313	3.6%
FM25	491	8.6%
FM26-18	261	5.4%
FM20b	463	1.2%
FM24b*	183	-
WWTP	450	20.8%

**These areas did not have sufficient flow data for analysis.*

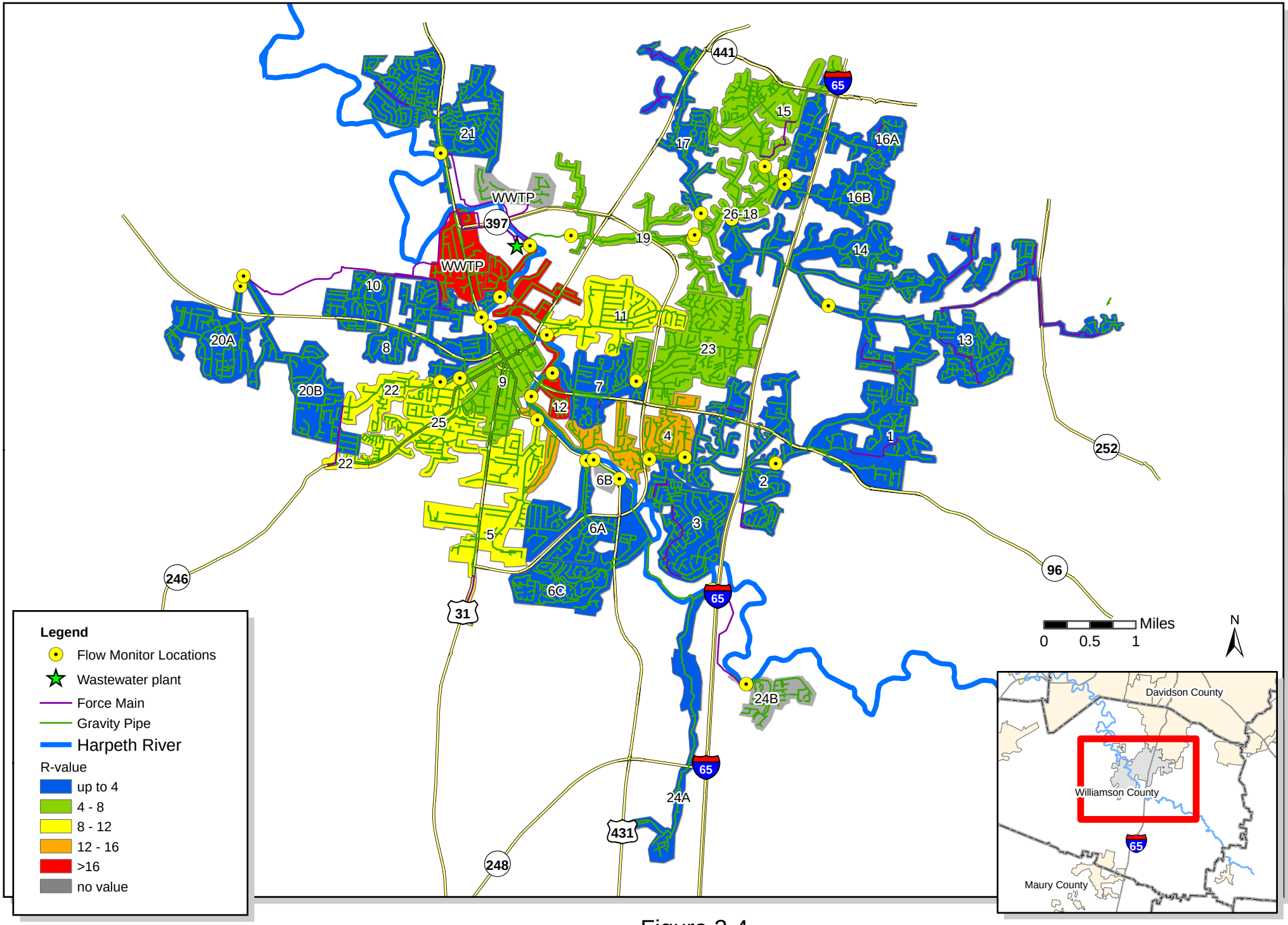

Figure 2-4
Franklin Collection System R-values

2.4 Results

The resulting R-values shown in Table 2-2 and Figure 2-4 were discussed with City of Franklin wastewater staff, SSR, and CDM. Using the calculated R-values and anecdotal information, areas within the Franklin collection system were prioritized for Sanitary Sewer Evaluation Surveys (SSES) work. The red areas in Figure 2-4 have R-values greater than 16%, which means that 16% or more of the rainfall in the associated area is entering the sewer system. One of these areas is FM12, which is primarily along the large trunkline leading to the WWTP; this line follows the Harpeth River, which could also be a source of inflow. The other area of concern is north of the WWTP, along Hillsboro Road in the areas in and around the Rebel Meadows neighborhood. Discussions with City of Franklin staff reveal that this area is comprised of older sewer and should be a priority area for improvements. As a first step, city staff will investigate the sewer river crossings in these areas; following this investigation, they will walk and inspect the remainder of the lines to identify any potential problems.

The orange area is sewershed FM4; examples of the neighborhoods within the area are Maplewood and Henley. One neighborhood in this area is relatively new construction, thus in this area, city staff will check the system for proper installation of new laterals. For the remainder of this area, city staff will walk the lines, followed by either closed circuit television (CCTV) or smoke testing to determine the best improvement option. The yellow areas include sewersheds FM5, FM11, FM22, and FM25. These sewersheds are comprised of areas of older construction, and the City of Franklin agreed that the sewers in these areas should be inspected for improvements. These areas will undergo CCTV inspection.

2.5 Improvement Options

Based on the information collected and analyzed from this preliminary flow monitoring study, several areas have been prioritized for further investigation via SSES. In addition to these measures, a hydraulic model of the collection system is recommended. A calibrated hydraulic model would provide a more accurate representation of the sewer system which could be used to simulate a range of hydraulic conditions and to analyze how the system would respond to various improvement options. Data collected during the flow monitoring period is of a quality sufficient to use in the calibration of a hydraulic model. It is further recommended that, if possible, lift station runtime data be retrieved for the flow monitoring period, to better refine a model and/or to provide a more in-depth understanding of the collection system and its potential limitations.

Following the SSES, the City of Franklin would have more accurate knowledge of their collection system, which would aid with the selection of planned rehabilitation and replacement improvements. In general, the possible collection system improvements are:

- Conveyance Improvements: replacement or relief sewers are typically required to convey projected dry-weather flows that exceed existing trunk sewer capacity. In addition, SSOs can

be eliminated by constructing replacement or relief sewers designed with increased capacity to effectively convey peak wet-weather flows.

- Equalization Storage: flow equalization facilities offer a means of reducing or eliminating wet-weather overflows by storing peak flows in excess of the sewer capacity.

- Rehabilitation: sewer rehabilitation and inflow source correction can be an effective means of reducing RDII and peak wet-weather flows that may result in sewer overflows.

At this point in the collection system analysis, selection of improvements is neither effective nor efficient. Based on the discussions held with City staff regarding the system, the most likely improvement options at this point are rehabilitation and/or pipe replacement. Although pipe replacement may be considered as a form of rehabilitation (by reducing the RDII), it may also be a convenient means to simultaneously increase pipe size to account for additional capacity to meet population growth and increased sewer flows. Rehabilitation may be performed using less obtrusive trenchless technology methods, such as cured-in-place pipe (CIPP) lining. The planning-level cost estimates presented in **Table 2-3** are for pipe replacement and a composite of trenchless technology rehabilitation methods. This table includes costs associated with manhole replacement and rehabilitation, as manholes can also be a source of RDII in the sewer system.

Table 2-3: Planning Level Cost Estimates

Diameter, Inches	Replacement	Rehabilitation
Gravity Pipe	**Cost*, $ per linear foot**	
8	130	55
10	136	64
12	142	74
18	166	109
24	204	151
30	244	192
36	279	242
48	346	360
60	434	482
Manhole	**Cost*, $ per vertical foot**	
48	229	156
60	335	175
72	417	190

*Note that these costs are construction costs only, and do not include any design/engineering fees or construction contingencies.

For purposes of evaluation within the IWRP modeling process, the following levels of improvements were considered, based on the "color-coded" areas in Figure 2-4, and the costs in **Table 2-3.**

Table 2-3: Planning Level Cost Estimates

Areas of Improvement (see Figure 2-4)	Cost, $
Red areas	9,300,000
Orange and red areas	16,400,000
Yellow, orange and red areas	31,100,000

*Note that these costs are construction costs only, and do not include any design/engineering fees or construction contingencies.

In addition, an annual cost of $1,700,000 will be included; this is based on the standard practice maintaining of 1% (in linear footage) of a collection system each year.

3.0 Plan for Conveyance of Wastewater from Cartwright Creek and Berry's Chapel Utility WWTPs

As previously described, the IWRP process includes an evaluation of accepting wastewater flows from the Berry's Chapel Utility as well as from the Cartwright Creek system as part of this study. These options evaluate construction of a lift station and force main directly to the Franklin WWTP. The project costs for providing the necessary infrastructure to capture wastewater flows currently routed to these facilities is provided in this section.

3.1 Cartwright Creek

The Cartwright Creek WWTP is permitted to discharge up to 250,000 gpd to the Harpeth River. Average daily flows have been observed to be approximately 455,000 gallons per day, with peak flows approaching 1.36 mgd. There are currently approximately 500 existing residential customers and 35 non-residential customers.

The plan to serve Cartwright Creek as a bulk sewer customer would include construction of a lift station and force main to convey wastewater directly to the existing Franklin WWTP. Routing for this force main is primarily along Hillsboro Road with the northernmost portion of the line running across undeveloped areas near the Harpeth River to reach the Cartwright Creek WWTP site. The line would require of a 1,050 gpm lift station along with 31,000 feet of 10-inch ductile iron pipe. One major river crossing and one major highway crossing are included in the estimate of costs. The estimated construction costs for the required infrastructure are summarized in **Table 3-1.**

Table 3-1 Cartwright Creek Connection Infrastructure Cost

Cartwright Creek Transmission System	
Pump Station	$1,360,000
Force Main	$2,140,000
Project Costs (Engineering, Legal) @ 20%	$700,000
Contingency @ 30%	$1,260,000
Total Project Cost	$5,460,000

These costs do not include any costs to rehabilitate the Cartwright Creek collection system. This project would benefit from the development of a system-wide collection system model to determine if there is capacity in the existing system to take the flows from Cartwright Creek further upstream in the collection system which could potentially reduce the transmission system construction costs.

3.2 Berry's Chapel Utility Incorporated

The BCUI WWTP is permitted to discharge up to 400,000 gpd to the Harpeth River. The plan to serve the BCUI as a bulk sewer customer would be to build a lift station and force main to convey the wastewater directly to the Franklin WWTP. The routing for this force main is primarily along property lines and existing rights of way as show in **Figure 3-1** to reach the BCUI WWTP site. The line would require a 1,400 gpm lift station along with 23,000 feet of 12 inch ductile iron pipe. The estimated construction costs for the required transmission system improvements are summarized in **Table 3-2.**

Table 3-2 BCUI Connection Infrastructure Cost

BCUI Transmission System	
Pump Station	$1,500,000
Force Main	$1,800,000
Project Costs (Engineering, Legal) @ 20%	$660,000
Contingency @ 30%	$1,190,000
Total Project Cost	$5,150,000

These costs do not include any costs to rehabilitate the BCUI collection system. As with the Cartwright Creek project, this project would benefit from the development a system-wide

collection system model to determine if there is capacity in the existing system to collect flows from BCUI further upstream and thus potentially reducing the transmission system construction costs.

4.0 Option for Connecting Septic Users within the UGB

In addition to potentially capturing wastewater that currently flows to Cartwright Creek and BCUI, it is possible that the City could develop a long-term program to identify and eliminate septic systems within the City limits, and ultimately the UGB. To evaluate the option of connecting these users, the existing sewer system studies previously conducted for the City of Franklin were reviewed. According to Butler, most of the unsewered systems are located in the Mayes Creek, Goose Creek and Southwest basins. Some of the areas within the Southwest basin are currently served by decentralized wastewater treatment facilities owned and operated by regulated utility companies. It is not anticipated that these newly installed treatment systems will need replacement within the next 30 years.

The City of Franklin has operated under a policy that requires homeowners in an existing neighborhood to pay for a collection system that would be installed to serve their homes through special assessment districts, set up by the City. It is not anticipated that the City will change this policy in the future. As a result, installation of collection systems to serve existing neighborhoods on septic systems is not anticipated to cost the City additional funds. Homeowners would be responsible for purchasing, installing and maintaining their own grinder pumps in the case of low pressure collection systems.

In a previous study conducted for the City, Goose Creek Study, (SSR 2005), several areas that could be served by City sewer were identified (SSR 2005). The areas identified in this study all flow towards the proposed Goose Creek WWTP and include the following:

- Redwing Farms: 188 lots
- Ellington Park area: 152 lots
- Walnut Winds: 137 lots
- Green Valley: 209 lots
- Goose Creek Subdivision: 85 lots
- Other areas (not subdivisions): 255 lots

Additional areas which are currently served by septic tanks were identified in the Southwest Area Study (SSR 2000), as shown in Figure 4-1. The existing homes in sub-basin SW4 would be most likely to be served by the City due to proximity to the existing UGB and City-owned

collection system. These areas flow by gravity to the existing WWTP, but could be served by pumping to the proposed WWTP site in the southern portion of the City.

- SW4: 414 lots

- SW2: 244 lots

Though not likely to be served in the future due to their outlying location in relation to the existing infrastructure two other areas with a substantial number of septic users have been identified Southwest Area Study, (SSR 2000). This includes two areas; the first area is SW1 which contains 70 lots; flow from this area could be conveyed by gravity to the existing WWTP. The second area is SW3 which includes 150 lots of which most are currently on an existing decentralized wastewater treatment system. Additionally, there are approximately 446 existing homes in the Mayes Creek basin that are on septic systems. These customers, if connected to the collection system would drain by gravity to the proposed new WWTP site near the confluence of the Harpeth River and Fivemile Creek at the Goose Creek Site.

In summary, there are approximately 2,130 lots within the Franklin UGB; 1,472 of these sites would drain toward the existing WWTP and 658 would drain toward the potential new WWTP site in the southern portion of the service area. This accounting does not include those 220 lots in SW1 and SW3, as previously discussed. There are remaining homes within the UGB that are on septic systems, but there are few and they are widely distributed. Connection of these users should be evaluated on a case by case basis.

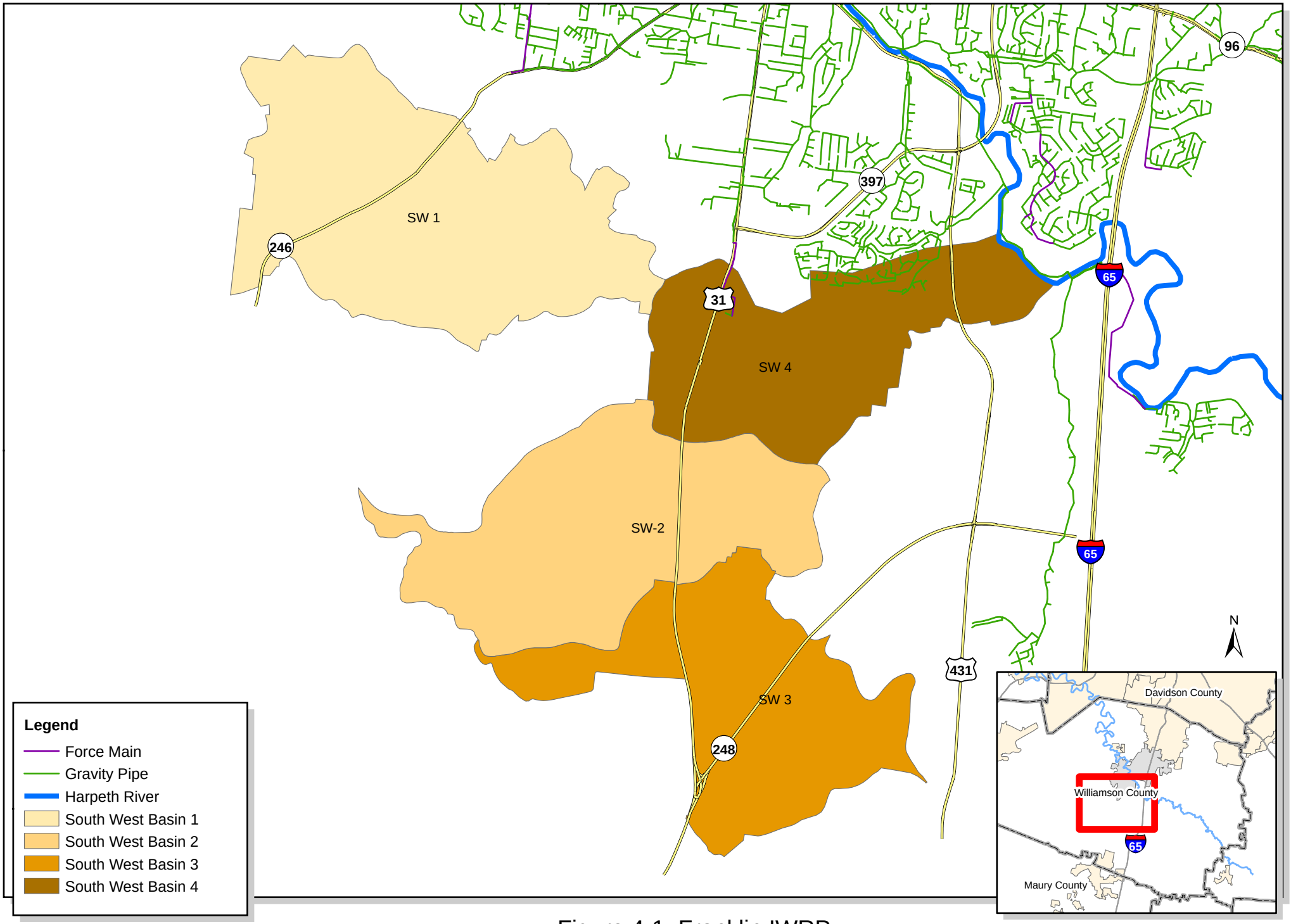

Figure 4-1: Franklin IWRP
Location of Septic Sewer Systems; Southwest Drainage Basins

5.0 Collection System Plan for Future Wastewater Demands without a second WWTP

The current infrastructure that provides wastewater collection in the southern portion of the City's service area is sized to convey up to 10 mgd of wastewater from the southern portion of the service area to the existing WWTP. If the City should decide that a second WWTP not be constructed, then a parallel system would need to be constructed to convey the any additional flows above 10 mgd. Based on the construction costs for the original transmission force mains (Harpeth River and Goose Creek Interceptors) and projecting them to 2011 construction dollars using the ENR Index, the estimated cost to construct the necessary collection and transmission system from the southern portion of the service area to the existing WWTP is approximately $25M, as summarized in **Table 5-1.**

Table 5-1 Southeast Interceptor Costs

Cost For Duplicating Southeast Interceptor Lines	
Harpeth River Interceptor	$14,770,000
Goose Creek Interceptor	$6,305,000
Project Costs (Engineering, Legal) @ 20%	$4,215,000
Total Project Cost	$25,290,000

6.0 Conclusions

This technical memorandum presents collection system options considered as part of the IWRP study, along with conceptual level cost estimates for implementing these options. The IWRP process will evaluate these options to determine which options should be included in the final, comprehensive plan.

In addition to the cost information provided in this memorandum, it is recommended that the City develop a hydraulic model of the collection system. While the SSES investigation of the selected areas of concern in the collection system will provide valuable information as to what improvements may be most beneficial, a fully calibrated model would provide the City a long-term management tool. The flow monitoring data analyzed as part of this study can easily be used to calibrate the model, which would then be able to predict how the collection system reacts under various conditions. This process will better enable the City to monitor and identify areas of existing and future concern. As discussed, a comprehensive model would aid in determining if additional flows, such as those from BCUI and Cartwright Creek, could be more easily connected to the system than by a direct transmission main to the WWTP.

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79935.WWTP.NEW - Technical Memorandum 1

To: *City of Franklin IWRP Team*

From: *Nicole Kolankowsky, Kati Bell, and Brian Karmasin*

Date: *February 21, 2012*

Subject: *Integrated Water Resources Plan – New WWTP Technical Analysis Technical Memorandum*

Executive Summary

The basis of this TM is to recommend a future treatment option for a new wastewater treatment plant (WWTP) in Franklin, TN. This TM describes in further detail the treatment technologies suggested and the three potential process trains developed at the New WWTP Process Workshop that was help with the Steering Committee and WWTP staff on February 4, 2011. A summary of the three potential process trains are shown below in **Table ES-1.**

Table ES-1
Summary of AWT Process Alternatives for a Proposed New WWTP

Process Train	Biological Process	Tertiary Treatment	Disinfection
Option 1	Conventional Plug-Flow Activated Sludge	Denitrification Filter	UV & UV-AOP
Option 2	Oxidation Ditch	Denitrification Filter	UV & UV-AOP
Option 3	Conventional Plug-Flow Activated Sludge	Membrane Bioreactor	UV & UV-AOP

It is also important to note that the initial concept of the new WWTP included UV disinfection, expandable to a UV-advanced oxidation process (AOP) for all process options. Initial equipment proposals were obtained for the UV-AOP process at the new WWTP with proposals from two vendors being in excess of $18M. This information was presented to, and discussed with the Steering Committee. Based on this discussion, the decision was made to evaluate the UV-UV/AOP process option at the WTP for addressing taste and odor and other emerging constituent issues associated with constructing a WWTP upstream of the existing water treatment plant (WTP). Thus, the equipment capital cost for addition of UV equipment that would be expandable to

UV/AOP to address these treatment objectives has been addressed in the cost estimates developed for improvements at the existing WTP and is summarized in the April 2011 Technical Memorandum, Review of CTE/AECOM Design Report: Franklin Water Treatment Plant dated July 2006, along with other cost updates for improvements and the WTP; there are other ancillary benefits of this approach as described in the April 2011 memorandum. This further justifies the selection of the MBR process to provide additional treatment reliability.

Finally, concurrent to this analysis, integrated modeling and evaluation of the existing WWTP capacity resulted in evaluation of two capacity options for the new MBR facility. The facility was evaluated for 4 mgd capacity with the ability to expand to an ultimate capacity of 8mgd to meet potential future wastewater demands. A summary of the costs of the 4 mgd facility and 8 mgd expansion are provided in **Table ES-2.**

Table ES-2
MBR Process Option Cost Summary

	4 mgd Option	8 mgd Option
Capital Cost	$59,864,000	$78,115,000
Annual O&M	$1,184,000	$1,536,000
Present Worth (20-year)	$47,189,000	$71,124,044

1.0 Background

During Phase I of the Integrated Water Resources Plan (IWRP), several of the Alternatives defined and selected by the Stakeholders identified a new wastewater treatment plant (WWTP) in the southern portion of Franklin's sewer service area as one of the City's long-term options for meeting its wastewater demands. During Phase II of this IWRP, the integrated model developed during Phase I will be used to evaluate the various options for the new facility as well as the costs and benefits of upgrading and rerating the existing WWTP. This memorandum presents the information is necessary to develop the analyses to define the proposed alternative treatment facility sizing, process options, and costs for an advanced WWTP on the Goose Creek site, a property previously purchased by the City.

In order for CDM to complete the conceptual engineering necessary to evaluate costs of constructing a new WWTP, CDM worked with the Steering Committee and WWTP staff to identify criteria that would be critical for success of a new WWTP project. These criteria were developed in the New WWTP Process Workshop held on February 4, 2011. Additional details on this Workshop are included in the Meeting Minutes of the New WWTP Process Workshop From this working meeting, three major issues were identified as critical success factors including: 1). address public perception, 2). provide odor control measures, and 3). include MBR process options. As a result, three generic biological processes were identified for further evaluation for the proposed WWTP conceptual design.

The process options developed in that Workshop have been evaluated in two stages. The first stage included a conceptual comparison of treatment processes that were identified for a single flow condition. The process options included an anaerobic/anoxic/ aerobic (A2O) activated sludge process followed by a denitrification filter and, a membrane bioreactor (MBR) configuration with a secondary anoxic reactor and a separate membrane tank. All of these biological process options produce advanced wastewater treatment (AWT) quality effluent. Conventional ultraviolet (UV) disinfection and a UV-advanced oxidation process (UV-AOP) were both evaluated for disinfection, with the UV-AOP proposed for addressing potential taste and odor compounds as well as other emerging contaminants, which are not currently regulated. The cost of implementing this process has been compared with implementing this process at the water treatment plant downstream of the proposed discharge location for the new WWTP.

Costs during the first stage of this analysis were developed based on the general requirements of each system, including major equipment requirements. The American Association of Cost Engineers (AACE) defines three levels of cost estimates—1) order-of-magnitude, 2) budgetary, and 3) definitive—each of which is applicable at a different stage of a project. The comparative costs represent order-of-magnitude estimates as defined by AACE, with the order-of-magnitude estimates being made without detailed engineering data. The estimates rely on the use of previous estimates and historical data from comparable work, estimating guides, handbooks, and costing curves and are intended for planning purposes and comparing alternatives. Costs are given in current (2011) dollars without escalation.

The preferred process identified from stage one of this analysis was advanced to provide more detailed information and for flows that were refined during the integrated modeling process used for development of the IWRP. The final costs presented in this memorandum include budgetary costs for equipment capital and installation; 10-percent contractor overhead and profit, 25-percent construction contingency, and 10-percent engineering, administration, and implementation. The actual cost of any project will depend on actual labor and material costs for competitive bids, project complexity, competitive market condition, actual site conditions, final scope of work, implementation schedule, continuity of personnel, and engineering.

1.1 Address Public Perception

Because the proposed location of a new WWTP is at the Goose Creek site, already acquired by the City, there may be issues with respect to public perception of impacts to the water quality of the Harpeth River both from an ecological perspective as well as that with regard to the proximity to the intake of the water treatment plant (WTP). Thus, any plan should include a process strategy that provides advanced wastewater treatment (AWT) to address biochemical oxygen demand (BOD), total suspended solids (TSS), and nutrient removal. Additionally, the overall plan should address treatment options for emerging compounds of concern, particularly with respect to its

location upstream of WTP. Although there are public perception issues with respect to water quality, there is also the desire to consider WWTP treatment processes that will supply water to enhance ecological flows and potentially allow supplementation of the water supply through planned indirect potable reuse, as many other communities in the nation are already implementing.

1.2 Provide Odor Control Measures in Design

The City of Franklin is a highly desirable location for residential development. Construction of any new WWTP in the City should provide measures to continue to protect the aesthetic values of the community. As a result, it was recommended to provide a wastewater treatment process that could be developed with no primary treatment which is sometimes a source of odor. Further, the Steering Committee wanted to capture information that provides documentation of the costs for providing high level odor control at any new facility.

1.3 Provide Flow Equalization for MBR Options

It is assumed that advanced treatment processes will be required to meet more stringent permit limits that in the existing WWTP permit. Thus, in addition to garnering public acceptance necessary for permitting and regulatory support, the proposed plant will include consideration of a MBR. Liquid process options were evaluated at the level of conceptual design so that a selection of the most viable liquid process train could be advanced to approximately a conceptual design and layout so that costs estimates for equipment and construction could be completed. Because there are trade-offs between costs of membranes and flow equalization structures, the working group opted to provide flow equalization at the new facility to limit the peaking factor to 2 for MBR process options. Flow equalization is important for MBR processes because there is a fixed area of flow provided within the MBR and for cost effectiveness, the MBR is sized only for a peak factor of 2.

2.0 Wastewater Treatment Plant Design Criteria

To develop conceptual site plans that would be used for costs estimates for equipment and construction, the WWTP size and processes were first be defined. Design flows and loadings provided the foundation for detailed designs of facilities. These criteria were generally derived from a combination of available plant data, specific requirements mandated by the regulatory agencies, policy decisions by the facility owners, and the engineering judgment of the designers. As such, it was necessary to define the wastewater flow and quality that will serve as the basis of design for the proposed WWTP.

Future wastewater demand projections were previously developed and described in a Technical Memorandum to the Steering Committee dated December 8, 2010, using the most recent ten

years of wastewater influent data with population to estimate wastewater generated per person per day. That estimate considered the growth rate of sewer customers as proportional to the growth rate of the City's population. This estimate was developed in conjunction with SSR, who has been the City's consultant throughout the planning and development of major sewer projects to collect and route wastewater to the existing WWTP. In recent years, it has become clear, as discussed in the December 8, 2010 Water Resources Demands Projections Technical Memorandum, that the existing WWTP is quickly reaching capacity and additional flows must be accommodated in the very near future.

With respect to the future projections of wastewater quality, historical data were used as the basis for extrapolation to future loads using population projections and conservative adjustments to influent wastewater quality characteristics that varied significantly from typical municipal WWTP characteristics.

2.1 Wastewater Treatment Plant Flows

Current flows to the existing WWTP average 9.6 million gallons per day (mgd), which is approximately 80-percent of the rated capacity of the plant. The total projected wastewater demands for the City have been previously presented in a December 8, 2010 Water Resources Demand Technical Memorandum. Since development of that document, SSR has compiled information regarding the wastewater demands at "build-out" for the sewersheds that flow to the City's service area. In order to correlate the flows from "build-out" to the end of the IWRP planning period in 2040 that would be routed to either the existing plant, located on the north end of the City, or to a new facility, located in the south of the City, SSR used population projections to ratio the wastewater demands in each sewer basin.

In summary, the estimated flows to the proposed new treatment facility, by the year 2040, could range between 3.6 million gallons per day (mgd) and 6.6 mgd, with a total service area build-out of 8 mgd depending on the rate and pattern of population growth over the planning period. Based on SSRs experience, and additional information from a collection system study conducted in early 2011, the initial planning flows to the facility were established at an annual average daily flow of 6 mgd.

The design capacity is stated in terms of annual average daily flow (AADF or ADF) or maximum monthly average flow, that the facility is anticipated to treat. In reality, the quantity of flow received at a wastewater facility varies continuously in response to daily, weekly, and seasonal (and long-term) cycles in the service area, as well as seasonal climatic changes, particularly the amount of rainfall. As a result, facilities must be designed to handle a range of flows. Of particular concern are the maximum or peak hydraulic flow rates that the facility must be capable to pass hydraulically.

The purpose of the initial analysis is to identify the preferred process that can provide treatment objectives of this project for an annual average daily flow of 6 mgd. Once the preferred process was identified, additional information was developed for 4 and 8 mgd scenarios, which spans the range of flows that would be anticipated for the new WWTP in the southern portion of the service area. The peaking factors that were be applied to this analysis have been developed based on historical operating data from the existing WWTP as well as feedback from the Steering Committee as noted in Section 1.3.

2.2 Wastewater Treatment Plant Loadings

In addition to flow information, loadings for the new Franklin WWTP were estimated from calculated values from historical influent data at the existing Franklin WWTP. Each facility component or process has been designed for the design flow and/or load that is critical for that specific component. The aeration process must also have the turndown capability for efficient treatment of minimum flows expected at facility start-up. As such, the CDM Smith standard practice for developing criteria for basis of design was applied to the proposed new Franklin WWTP as follows:

- Maximum monthly (also called average day maximum month or ADMM) load to size the biological reactors for BOD and nutrient removal facilities,
- Maximum day (MD) loads to size the aeration system,
- Minimum day loads to check turndown for the aeration system,
- Maximum day and peak hour (PH) flows for the secondary clarifiers, and
- Peak hour flow for hydraulic elements.

Analysis of historical data from the Franklin WWTP showed that the BOD:TKN (total Kjelhdal Nitrogen) ratio at the plant is approximately 11:1. The TKN has been adjusted in the historical data to provide a BOD:TKN of 6:1 as a conservative approach to design with this value being selected to be representative of what is typically observed at municipal facilities. Carbonaceous Biochemical Oxygen Demand (CBOD) is measured by plant staff and is currently the basis for oxygen demand at the facility. CBOD is converted to BOD for identifying future loads. A summary of the future loads to a new WWTP, based on a 6 mgd flow as the initial basis of design, are provided in **Table 2-1**.

Table 2-1
Summary of WWTP Influent Conditions

Parameter	Flow Condition				
	Minimum Day	Average Day	Average Daily Maximum Month	Maximum Day	Peak Hour
Flow (mgd)	4.1	6.0	7.8	9.6	12/18[1]
BOD_5 (lb/d)	7,211	10,604	13,467	16,966	-
BOD_5 (mg/L)	-	212	-	-	-
TSS (lb/d)	5,334	8,207	9,520	13,541	-
TSS (mg/L)	-	164	-	-	-
TKN (lb/d)	1,131	1,767	1,962	2,492	-
TKN (mg/L)	-	35.3	-	-	-
Total Phosphorus (TP) (lb/d)	279	410	533	657	-
TP (mg/L)	-	8.2	-	-	-

[1] Peak flow of 8 mgd used for MBR alternative. 12 mgd used for conventional alternatives.

2.3 WWTP Effluent Requirements

The Harpeth River is impaired due to nutrients and it has significant low flows during summer months upstream of the existing WWTP, thus construction of an advanced WWTP upstream of this river segment could result in an enhancement of the low flow conditions to positively impact ecological flows, as well as the ability of the existing water treatment plant to withdraw water from the Harpeth River. Thus, for evaluation of the proposed new WWTP, based on discussions with TDEC, technology based limits will be assumed. A summary of the design criteria that will be used for the proposed new WWTP are provided in **Table 2-2**.

Table 2-2
Summary of WWTP Effluent Design Conditions

Effluent Conditions	Limit (mg/L)
$CBOD_5$	4.0
TSS	5.0
Ammonia (NH_4-N)	0.8
Total Nitrogen (TN)	3.0
TP	0.3

While these limits are somewhat restrictive, there are innovative solutions being evaluated as part of the IWRP that would allow permitting of a new WWTP. Discussions with TDEC staff, in conjunction with modeling the water quantity and water quality of the Harpeth River have shown that it is possible to meet the current waste load allocations for the river under the scenarios that include the addition of a new WWTP. The key to obtaining a permit would be a design to provide

advanced wastewater treatment that results in very high effluent quality, coupled with enhanced reuse and other potential nutrient offsets within the watershed. Options for providing this level of treatment at the proposed new WWTP are described in Section 3.

3.0 Process Options

Technologies that meet advanced water treatment (AWT) and have been demonstrated to reliably provide this high level of treatment were discussed with the City of Franklin in the February 4 New WWTP Process Workshop. One of the results of the workshop was a short list of treatment process options to be evaluated for the proposed new WWTP and a summary of these processes is provided in **Table 3-1.** This section provides a description and summary of the cost impacts for each of the process alternatives developed during that workshop.

Table 3-1
Summary of AWT Process Alternatives for a Proposed New WWTP

Process Train	Biological Process	Tertiary Treatment	Disinfection
Option 1	Conventional Plug-Flow Activated Sludge	Denitrification Filter	UV & UV-AOP
Option 2	Oxidation Ditch	Denitrification Filter	UV & UV-AOP
Option 3	Conventional Plug-Flow Activated Sludge	Membrane Bioreactor	UV & UV-AOP

CDM Smith developed process flow diagrams for these scenarios; conventional spreadsheet calculations have been developed, and were supplemented with steady-state process modeling, using BIOWIN, an industry-standard software package, to evaluate the optimum internal recycle flows that would minimize operating costs and meet the anticipated treatment requirements for the proposed facility. Each of the process options presented in Table 3-1 is described in the following sections.

3.1 Option 1 – Activated Sludge/Denitrification Filter

Option 1 is a conventional plug-flow, activated sludge process that is also referred to as an "A2O" process, with tertiary filtration and UV disinfection. The A2O process consists of a fermentation zone, pre-anoxic zone, and aeration zone. Following these three zones is secondary clarification, tertiary filtration employing denitrification filters, and UV disinfection. A schematic of the process is provided in Figure 3-1 for the initial scenario of an average daily flow of 4 mgd and peak hour flow of 12 mgd for this process.

In this process, raw wastewater enters a fermentation zone and is mixed with the return activated sludge. With the presence of neither nitrate nor oxygen, microorganisms become stressed and release phosphorus into the wastewater. The released phosphorus, along with additional soluble phosphorus, is subsequently assimilated in new biomass produced in the aeration basin. The biomass and the phosphorus it has taken up (in the aeration basin), is

removed from the biological process through the wasting process as waste activated sludge (WAS). The amount of phosphorus (P) released in the fermentation zone is a function of the dissolved oxygen and nitrate concentrations in the basin as well as the readily biodegradable organics in the wastewater. In the event the readily biodegradable organics in the wastewater are not adequate, biological phosphorus removal capacity decreases significantly. Phosphorus accumulating organisms (PAOs) require a minimum BOD:P ratio of 25 to 1. The current BOD:P ratio used in this evaluation was 26 to 1, meeting the minimum requirements.

For a 6 mgd facility, total volume required for the fermentation basins is 0.33 million gallons and the pre-anoxic basins require a total volume of 0.62 million gallons. At an aerobic solids retention time of 9 days, roughly 3.12 million gallons of operating aeration volume is required. Three process trains are proposed to provide redundancy and flexibility; the proposed layout of each aeration basin provides a length to width ratio of over 5 to 1. Baffling internal to the aeration basin can also assist in providing a plug-flow regime. Four new 200 horsepower blowers would be required to provide air to this process train during maximum day loading conditions, assuming a fine-pore diffused aeration system.

The final clarifier design is based on a sludge volume index of 130 mL/g at average day conditions. The clarifier units were evaluated using EPA "Design Criteria for Mechanical, Electrical, and Fluid System and Component Reliability" at peak hour flow, which requires 75 percent of the peak hour flow to be treated with one clarifier out of service or maximum day flow of 9.6 mgd with a settling safety factor of 1.3 (EPA, 1978). Three, eighty (80)-foot diameter clarifiers are recommended. The denitrification filters were sized based on a proposal from ITT Water and Wastewater Leopold, Inc. Four filters at 60 feet long by 14 feet wide are proposed; the design hydraulic loading rate at average and peak hour flows are 1.24 gpm/ft^2 and 3.72 gpm/ft^2, respectively.

3.2 Option 2 – Oxidation Ditch/Denitrification Filter

Option 2 would utilize a biological treatment process similar to Option 1. However, instead of using rectangular basins with diffused aeration, Option 2 provides new oxidation ditches, similar to the process used at the existing WWTP. The proposed configuration consists of a fermentation zone and pre-anoxic zone followed by an oxidation ditch with internal recycle back to the pre-anoxic zone. The oxidation ditch process is a variation of suspended activated sludge that is typically characterized by circular or oval channels and mechanical surface aeration although oxidation ditches have also been installed with diffused aeration and mixers. Variations on the oxidation ditch process have been developed and patented by various manufacturers including Ovivo's Carrousel™ and Envirex's Orbal™ process.

Oxidation ditches are usually run at long solids retention times (SRT), which reduces biosolids production. Mechanical surface aerators are often installed to promote a motive force to move mixed liquor through the channels, in addition to providing oxygen to the mixed liquor. The majority of flow in the oxidation ditch is recycled internally, around passes which promotes a

complete-mix regime in the bioreactor. Due to the configuration of the oxidation ditch, spatially varying aerobic and anoxic zones develop in the tanks. Facilities that employ oxidation ditches frequently report simultaneous nitrification and denitrification (SND) as well as biological phosphorus removal. However, care must be taken in operation of oxidation ditches because excessive anoxic volume in the oxidation ditch could result in depletion of dissolved oxygen and nitrates, and subsequent release of phosphorus into the mixed liquor.

For a 6 mgd design flow, oxidation ditch sizing was based on a proposal from Ovivo. The total volume for the fermentation basins is 0.38 million gallons and the pre-anoxic basins have a total volume of 0.66million gallons. At an aerobic SRT of 12.3 days, roughly 4.5 million gallons of operating aeration volume is required. Four 125-horsepower mechanical surface aerators are also provided. The final clarifier design is based on a sludge volume index of 130 mL/g at average day conditions. The units were configured to meet EPA Class I Reliability at peak hour flow (75 percent of the peak hour flow to be treated with one clarifier out of service). Three eighty (80)-foot diameter clarifiers are recommended. Denitrification filters were sized based on a proposal from ITT Water and Wastewater Leopold, Inc.; four filters at 60 feet long by 14 feet wide are proposed. The hydraulic loading rates at average and peak hour flows are 1.24 gpm/ft^2 and 3.72 gpm/ft^2, respectively. A process flow diagram for the Option 2 configuration is provided in **Figure 3-2.**

3.3 Option 3 – Activated Sludge/Membrane Bioreactor

The third biological treatment process option evaluated during phase one of this analysis included a 5-stage Bardenpho process with an integrated membrane. The membrane bioreactor (MBR) process is provided with flow equalization and UV disinfection. This option would have three process trains consisting of one fermentation tank, one pre-anoxic tank, one aeration tank, and one post-anoxic basin per train. The fermentation stage acts the initial phosphorus and BOD removal and denitrification occurs in the pre-anoxic tank, where nitrate is converted to molecular nitrogen. In the third basin, oxygen is introduced in the aeration basin where nitrification occurs; ammonia and nitrite are oxidized and form nitrate. An internal recycle flow of 19.8 mgd is provided to return flows from the pre-anoxic basin back into the fermentation basin. Internal recycle is an important component to the treatment process because it is directly related to MLSS concentration. The greater the internal recycle, the greater the allowable MLSS concentration, which reduces equipment size and footprint requirements. A return activated sludge (RAS) flow of 3.6 mgd is included to provide flow from the MBR to the pre-anoxic basin. The fourth basin in this process, post-anoxic, is required to remove any remaining nitrogen and supplemental carbon feed is planned to enhance nitrogen removal. A schematic of Option 3 is provided in Figure 3-3.

Following the post-anoxic process, mixed liquor flows by gravity to submerged microfiltration (MF) or ultrafiltration (UF) membrane tanks, where treated water is separated from suspended biomass. Biomass contained in the mixed liquor must be returned from the membrane tanks to the beginning of the process at flow rates between 4 to 6 times the influent flows to remove the

solids that accumulate in the membrane tanks. These solids are returned to the main biological treatment process and waste activated solids (WAS) are intermittently removed from the MBR process using a dedicated waste sludge pump.

Option 2 does not require secondary clarifiers and filters because the membranes perform the solids separation function of both these unit processes. Pumps are most commonly used to draw effluent (also called filtrate or permeate in an MBR process) through the membranes; although with some systems, gravity can be used to provide the necessary suction pressure. MBR systems offer several advantages over conventional activated sludge systems. Because secondary clarifiers are replaced by membranes, sludge settling characteristics no longer affect plant capacity or final effluent quality. This allows the biological processes to operate at much higher suspended solids concentrations and longer solids retention time (SRT), thereby providing greater treatment capacity per unit tank volume. A well-operated MBR will provide effluent quality, surpassing all conventional secondary and BNR treatment processes by producing effluent with BOD, TSS and pathogen concentrations at or below analytical limits of detection.

The relatively small footprint of MBR systems can significantly reduce overall site requirements by removing the need for final clarifiers and filtration units. Also, because clarifier solids loading is not a concern, design mixed liquor concentrations can be significantly higher than in conventional activated sludge processes thereby allowing a further reduction in footprint by reducing bioreactor volume. At an average MLSS concentration of 7,500 mg/L and aerobic SRT of 9 days, approximately 1.5 million gallons of aeration volume are required for the 6.0 mgd design condition. Option 3, has a much higher MLSS concentration than Option 1 (MLSS concentration of between 3,000 and 3,500 is assumed), as a result a lower alpha value was selected for design to represent a greater air requirement due to a decrease in oxygen transfer efficiency at the higher MLSS concentrations.

4.0 Project Options Costs

Conceptual cost estimates for all three of the process options have been developed. These costs include both capital and operations; this information is used to aid in identifying the preferred process option for further analyses, which was conducted in parallel with other IWRP modeling efforts.

4.1 Capital Costs for Process Options

Costs associated with the headworks concrete, alum feed system, reuse storage and pumping, and biosolids handling were not estimated for phase one of this evaluation because it was assumed these costs would be similar for each option. The purpose of this first phase of cost estimates was to provide comparative costs among the process options. Thus, planning level estimates of capital costs were prepared for the proposed WWTP using unit costs for cast-in-place concrete, manufacturer's quotations, and industry accepted percentages for civil/site work, piping/valves and fittings, electrical and instrumentation work. A summary of the conceptual cost estimates for a 6 mgd facility under each of the process scenarios is provided in the following sections.

4.1.1 Option 1 Costs (Activated Sludge/Denitrification Filter)

Conceptual costs for process Option 1 included all main components of headworks, bioreactors, secondary clarifiers, filters, disinfection, and reaeration. The estimated conceptual cost for this process option at 6 mgd was $36,040,000 with component details as described below and summarized in **Table 4-1**:

- **Headworks**: Costs associated with the headworks equipment includes 6mm screens and grit removal system.
- **Bioreactors**: Costs include concrete and rock excavation for fermentation, pre-anoxic, and aerobic tanks, mixers, three blowers, fine bubble diffusers, and six internal recycle (IR) pumps.
- **Secondary Clarifiers**: costs include concrete and rock excavation, internal mechanisms, four RAS pumps and two WAS pumps.
- **Filters**: Costs associated with the filters include three denitrification filters and concrete and rock excavation.
- **Disinfection**: UV disinfection costs include concrete, rock excavation, and UV equipment.
- **Reaeration**: Costs include concrete and rock excavation for the reaeration tank, blowers and diffusers.

Table 4-1
Option 1 Conceptual Capital Cost

Component	TOTAL COST
Headworks	$2,291,000
Bioreactors	$4,383,000
Secondary Clarifiers	$2,000,000
Denitrification Filters	$2,952,000
Disinfection System	$849,000
Reaeration Tank	$360,000
SUBTOTAL	**$12,835,000**
Civil Allowance	$1,283,000
Mechanical Piping Allowance	$1,925,000
Instrumentation & Electrical Allowance	$3,209,000
CONSTRUCTION SUBTOTAL	**$19,252,000**
Permits	$193,000
Builder's Risk	$96,000
General Liability	$193,000
GC Bonds	$289,000
CONSTRUCTION SUBTOTAL	**$20,023,000**
Contractor's Overhead and Profit	$2,002,000
General Conditions	$2,002,000
Contingency	$6,006,000
Escalation (2011$)	$0
TOTAL ESTIMATED CONSTRUCTION COST (ROUNDED)	**$30,033,000**
Design & Construction Services	$4,505,000
City Project Administration	$601,000
Legal/Finance	$901,000
TOTAL ESTIMATED CAPITAL COST (ROUNDED)	**$36,040,000**

4.1.2 Option 2 Costs (Oxidation Ditch/Denitrification Filter)

Conceptual costs for Option 2 included headworks equipment, bioreactors, secondary clarifiers, filters, disinfection, and reaeration tanks. The estimated conceptual cost for this process option at 6 mgd was $35,536,000 with component details as described below and summarized in **Table 4-2:**

- **Headworks:** Costs associated with the headworks include 6mm screens and grit removal system.
- **Bioreactors:** Costs include concrete and rock excavation for fermentation and oxidation ditch tanks and oxidation ditch components.
- **Secondary Clarifiers:** costs include concrete and rock excavation, internal mechanisms, four RAS pumps and two WAS pumps.
- **Filters:** Costs associated with the filters include three denitrification filters and concrete and rock excavation.
- **Disinfection:** UV disinfection costs include concrete, rock excavation, and UV equipment.
- **Reaeration:** Costs include concrete and rock excavation for the reaeration tank, blowers and diffusers.

Table 4-2
Option 2 Conceptual Capital Cost

Component	TOTAL COST
Headworks	$2,138,000
Bioreactors	$4,744,000
Secondary Clarifiers	$1,898,000
Denitrification Filters	$2,730,000
DisinfectionSystem	$809,000
Reaeration Tank	$337,000
SUBTOTAL	**$12,656,000**
Civil Allowance	$1,265,000
Mechanical Piping Allowance	$1,898,000
Instrumentation & Electrical Allowance	$3,164,000
CONSTRUCTION SUBTOTAL	**$18,983,000**
Permits	$189,000
Builder's Risk	$95,000
General Liability	$190,000
GC Bonds	$285,000
CONSTRUCTION SUBTOTAL	**$19,742,000**
Contractor's Overhead and Profit	$1,974,000
General Conditions	$1,974,000
Contingency	$5,923,000
Escalation (2011$)	$0
TOTAL ESTIMATED CONSTRUCTION COST (ROUNDED)	**$29,613,000**
Design & Construction Services	$4,442,000
City Project Administration	$592,000
Legal/Finance	$889,000
TOTAL ESTIMATED CAPITAL COST (ROUNDED)	**$35,536,000**

4.1.3 Option 3 - Costs (Activated Sludge/MBR)

Conceptual costs for Option 2 included all main components of headworks, flow equalization, MBR, disinfection, and reaeration. The conceptual capital cost for this option at 6 mgd was estimated at $46,645,000. Component details are described below and costs are summarized in **Table 4-3:**

- **Headworks**: Costs associated with the headworks include coarse and fine screens and grit removal system.
- **Flow Equalization:** Costs include equalization tank and mixing system.
- **Bioreactors**: Costs include concrete and rock excavation for fermentation and pre-anoxic tanks, mixers, blowers, fine bubble diffusers, and six internal recycle (IR) pumps, four RAS pumps and two WAS pumps.
- **MBR**: Costs include concrete and rock excavation for the MBR tanks, and the MBR filters.
- **Disinfection**: UV disinfection costs include concrete, rock excavation, and UV equipment.
- **Reaeration**: Costs include concrete and rock excavation for the reaeration tank, blowers and diffusers.

Table 4-3
Option 3 Capital Costs

Component	TOTAL COST
Headworks	$2,604,000
Flow Equalization	$1,196,000
Bioreactors	$4,592,000
MBR	$7,274,000
Disinfection System	$647,000
Reaeration Tank	$299,000
SUBTOTAL	**$16,112,000**
Civil Allowance	$1,661,000
Mechanical Piping Allowance	$2,492,000
Instrumentation & Electrical Allowance	$4,153,000
CONSTRUCTION SUBTOTAL	**$24,917,000**
Permits	$249,000
Builder's Risk	$125,000
General Liability	$249,000
GC Bonds	$374,000
CONSTRUCTION SUBTOTAL	**$25,914,000**
Contractor's Overhead and Profit	$2,591,000
General Conditions	$2,591,000
Contingency	$7,774,000
Escalation (2011 $)	$0
TOTAL ESTIMATED CONSTRUCTION COST (ROUNDED)	**$38,871,000**
Design & Construction Services	$5,831,000
City Project Administration	$777,000
Legal/Finance	$1,166,000
TOTAL ESTIMATED CAPITAL COST (ROUNDED)	**$46,645,000**

4.2 Operations Costs for Process Options

Operations costs have been estimated for each alternative based on major power requirements and major chemical costs. **Table 4-4** lists the major assumptions used to prepare the estimates of operations costs.

Table 4-4
Operations Cost Assumptions

Item	Unit Costs	Units
Electric Power	0.095	$/kWh
Methanol	1.43	$/gallon
Alum (as Al3+)	4,900	$/ton
Sodium Hypochlorite	1.57	$/gallon (12%)
Citric Acid	1.65	$/lb
Design Period	20	years
Interest	6.0%	rate/year
Operation schedule	52	weeks/year
Motor Efficiency	95	percent
Horizontal Pump Efficiency	75	percent
Alum Feed	2.0	Mol Al/Mol P
Density of Methanol	6.6	lb/gal
Density of Citric Acid	10.3	lb/gal
Citric Acid strength	50	percent

Based on the cost assumptions provided in **Table 4-5**, conceptual level annual operations cost estimates were developed for the three process options at 6 mgd. These costs do not include labor or maintenance, which is anticipated to be similar for each of the process options.

Table 4-5
Operations Cost Summary

	Option 1	Option 2	Option 3
Power Costs ($/year)	$295,000	$249,000	$501,000
Power Cost ($/MG)	$135	$114	$229
Chemical Costs ($/year)	$117,000	$117,000	$253,000
Chemical Cost ($/MG)	$53	$53	$116
	Option 1	**Option 2**	**Option 3**
Total ($/year)	$412,000	$366,000	$754,000
Diffuser Replacement Costs ($/10 years)	$34,000	$34,000	$48,000

4.3 Present Worth Costs

Present worth costs were calculated for each Process Option. Costs are presented in the following summary table, over a 20 year lifecycle at a six-percent interest rate. Power cost inflation was not accounted for in the evaluation.

Table 4-6
Summary of AWT Process Alternatives for a Proposed New WWTP

Process Train	Average Daily Flow (mgd)	Peak Hourly Flow (mgd)	Capital Cost	Annual Operations Cost	20 Year Present Worth Cost
Option 1 (Activated Sludge/Denitrification Filter)	6.0	18.0	$36.0M	$412,000	$41.0M
Option 2(Oxidation Ditch/Denitrification Filter)	6.0	18.0	$35.5M	$366,000	$40.0M
Option 3 (Activated Sludge/MBR)	6.0	12.0	$46.6M	$754,000	$57.2M

5.0 Process Summary and Recommendation

Based on the comparative information for the three process options, the Steering Committee discussed which of the options is the most appropriate for including in the final analysis for the IWRP. While the costs between the three process options varied slightly, the decision was based on the non-cost benefits of the treatment technologies. The advantages of the MBR option are:

- Secondary clarifiers and tertiary filtration processes are eliminated, thereby reducing plant footprint.
- Increased mixed liquor concentrations are possible, therefore reducing the aeration basin volume, further reducing the plant footprint.
- Solids separation is increased in an MBR than in a clarifier.
- High quality of wastewater effluent/reclaimed water.
- Membranes provide a physical barrier to prevent solids carryover in the effluent. As such, operational conditions such as sludge bulking and high solids loading do not impact effluent quality in an MBR as it would in a secondary clarifier.

The Steering Committee indicated that these process advantages outweighed the slightly higher costs associated with the MBR process. As a result Option 3, the activated sludge/MBR process was recommended for evaluation during the second stage of this analysis.

6.0 Planning Level Layout and Costs for Selected Option

As previously described in Section 1.0, during Phase I of the IWRP, Stakeholders identified continued use, and possible expansion of the existing WWTP, in addition to a new WWTP in the southern portion of town as a potential long-term solution to meeting water resources needs. During Phase II of the IWRP, CDM Smith developed conceptual costs necessary to compare three major process alternatives that could be implemented to meet the advanced water quality treatment requirements that are anticipated for any new plant that would discharge to the Harpeth River. Based on the advantages of an MBR facility, as described in Section 5.0, this treatment process was selected for further evaluation.

Additional integrated modeling was conducted during Phase II of the IWRP to further evaluate the relationships among the City's water supply, wastewater, reclaimed water, and stormwater. As described in the September 15, 2010 final *Phase I Results* of the IWRP, these analyses were modeled with the software STELLA. The model is a representation of the interconnections between the major water resources systems and included planning-level calculations of flow, pollutant loads, energy requirements, and operational costs. During this Phase II of the IWRP, the integrated model was refined and used in conjunction with a detailed water quality model to evaluate the overall impacts of Stakeholder derived project alternatives on the Harpeth River.

STELLA was also used to evaluate the distribution of treatment capacity between the proposed new MBR in the southern portion of the City's service area and that which could be achieved at the existing WWTP. In order to support this analysis, conceptual level design costs were developed for the new MBR under two different flow scenarios which would complement the potential capacity at the existing WWTP. Two flow scenarios were proposed for the new treatment facility; 4 mgd and 8 mgd capacities were evaluated to address the range of anticipated flows that could be observed to the new facility by the year 2040. These flows could range between 3.6 million gallons per day (mgd) and 6.6 mgd, with a total service area build-out of 8 mgd depending on the rate and pattern of population growth over the planning period.

6.1 Conceptual Costs for MBR Facility

The basis of design for the MBR facility option for 4 mgd and 8mgd flow conditions was the loadings to the facility as summarized in **Tables 6-1** and **6-2.** These design criteria were derived as previously described in Section 2.

Table 6-1
Summary of WWTP Influent Conditions for 4 mgd Design Capacity

Parameter	Flow Condition				
	Minimum Day	Average Day	Average Daily Maximum Month	Maximum Day	Peak Hour
Flow (mgd)	2.7	4.0	5.2	6.4	8/12[1]
BOD_5 (lb/d)	4,807	7,069	8,978	11,311	-
BOD_5 (mg/L)	-	212	-	-	-
TSS (lb/d)	3,556	5,471	6,346	9,027	-
TSS (mg/L)	-	164	-	-	-
TKN (lb/d)	855	1,335	1,508	1,913	-
TKN (mg/L)	-	35.3	-	-	-
TP (lb/d)	247	363	491	630	-
TP (mg/L)	-	8.2	-	-	-

[1] Peak flow of 8 mgd used for MBR alternative. 12 mgd used for conventional alternatives.

Table 6-2
Summary of WWTP Influent Conditions for 8 mgd Design Capacity

Parameter	Flow Condition				
	Minimum Day	Average Day	Average Daily Maximum Month	Maximum Day	Peak Hour
Flow (mgd)	5.4	8.0	10.4	12.8	16/24[1]
BOD_5 (lb/d)	9,614	14,138	17,956	22,621	-
BOD_5 (mg/L)	-	212	-	-	-
TSS (lb/d)	7,112	10,942	12,693	18,054	-
TSS (mg/L)	-	164	-	-	-
TKN (lb/d)	1,709	2,671	3,015	3,826	-
TKN (mg/L)	-	35.3	-	-	-
TP (lb/d)	337	495	760	1,048	-
TP (mg/L)	-	8.2	-	-	-

[1] Peak flow of 16 mgd used for MBR alternative. 24 mgd used for conventional alternatives.

The conceptual plan for the facility, under both the 4 and 8 mgd scenarios (where the 8 mgd scenario is presented as a modular design that is upgradable from the 4 mgd system) includes: an influent pump station, headworks, flow equalization, odor control system, biological process building (including the 5-stage Bardenpho process with integrated membrane), UV disinfection trains, solids thickening, and a reaeration tank. Cascade aeration could also be utilized; however, the cost of a reaeration basin is more conservative and is carried forward. The biological process

includes two process trains (for the 4-mgd initial construction) consisting of one fermentation tank, one pre-anoxic tank, one aeration tank, and one post-anoxic basin per train. Expansion to 8-mgd would double the size of the biological process, adding two more trains for a total of four process trains. Additionally, the following components will be included when expanding to 8-mgd:

- A second equalization tank
- Two additional process trains (including biological treatment, MBR and reaeration)
- Third train for UV disinfection
- Additional pumping for the reuse system and plant transfer system
- Additional storage, pumping, and thickener for the biosolids process.

6.2 Cost

Detailed conceptual costs were calculated for the selected option (activated sludge/MBR process) at a flow of 4 mgd and 8 mgd (anticipated expansion) using the same cost structure that was previously used for the 6 mgd option.

To construct the first phase of the WWTP to 4 mgd, the following components were included in the conceptual construction cost estimate. The estimated conceptual cost for this process option at 4 mgd was $59,864,000 with component details as described below Site Work: Costs include rock excavation, landscaping, access roads, and site improvements.

- Influent Pump Station and Headworks: Costs associated with the influent pump station and headworks include two influent pumps, one 6mm (coarse) bar screen, one 2mm (fine) drum screens and grit removal system.

- Odor Control: Costs for the odor control system includes one biofilter.

- Splitter Boxes: Costs include concrete and slide gates for construction of the splitter boxes.

- Equalization Tank: Costs associated with the EQ tank include one prestressed concrete tank, two pumps, and one mixing system.

- Aeration Basins: Costs include concrete for fermentation and pre-anoxic tanks, eight mixers, three blowers, fine bubble diffusers, and six IR pumps.

- MBR: Costs include concrete and MBR filters.

- Chemical Feed System: Costs associated with the chemical feed system include concrete, three chemical metering pumps, and three chemical storage tanks.

- Disinfection: UV disinfection costs include concrete, covered canopy, and UV trains.
- Reaeration: Costs include concrete for the reaeration tank, two blowers and diffusers.
- Transfer Pumps: Costs include five transfer and plant water pumps.
- Reuse: Costs associated with the reuse system include one prestressed concrete tank, pump and piping.
- Biosolids: Costs include one sludge holding tank, four RAS pumps, two WAS pumps, sludge thickener building, rotary drum thickener, polymer system, two tanker trucks, and associated biosolids piping.
- Process Building: The process building is approximately 67,000 square feet in size and will include a concrete foundation, steel framing and roof, HVAC, piping, plumbing, windows, and doors.
- Administration Building: The process building is approximately 4,500 square feet in size and will include a concrete foundation, steel framing and roof, HVAC, piping, plumbing, windows, and doors.
- On-site Generators: The costs include one fuel tank and one 750kW generator.
- Electrical System: These costs include all electrical and instrumentation components at the WWTP.

The cost breakdown for the 4 mgd scenario is provided in Table 6-1. Information for the 8 mgd expansion is also presented and includes the following additional equipment components:

- A second equalization tank
- Two additional process trains (including biological treatment, MBR and reaeration)
- Additional equipment for the chemical feed system
- Third train for UV disinfection
- Additional pumping for the reuse system and plant transfer system
- Additional storage, pumping, and thickener for the biosolids process.

Table 6-3
MBR Capital Costs

COMPONENT	4 mgd Costs	8 mgd Costs
Site work	$1,768,000	$2,066,000
Influent Pump Station	$3,471,000	$3,993,000
Odor Control	$448,000	$448,000
Splitter Box	$476,000	$475,000
EQ Tank	$2,217,000	$4,318,000
Aeration Basins	$4,990,000	$8,669,000
MBR	$9,913,000	$16,211,000
Chemical Feed System	$373,000	$578,000
Disinfection	$842,000	$1,548,000
Reaeration	$486,000	$980,000
Transfer Pumps	$323,000	$509,000
Reuse	$430,000	$875,000
Biosolids	$3,693,000	$4,703,000
Process Building	$22,422,000	$22,417,000
Admin Building	$910,000	$910,000
Generators	$863,000	$1,273,000
Electrical	$6,239,000	$8,145,000
Total	**$59,864,000**	**$78,115,000**

6.2.2. Operations Costs

Operations costs are estimated for both flow conditions based on major power requirements, major chemical costs, and operator labor rates. **Table 6-4** lists the major assumptions used to prepare the estimates of operations costs.

Table 6-4
Operations Cost Assumptions

Item	Unit Costs	Units
Electric Power	0.095	$/kWh
Methanol	1.43	$/gallon
Alum (as Al3+)	4,900	$/ton
Sodium Hypochlorite	1.57	$/gallon (12%)
Citric Acid	1.65	$/lb
Design Period	20	years
Interest Rate	6.0%	/ year

Operating weeks per year	52	
Number of Operators (4 mgd)	6	/day
Number of Operators (8 mgd)	9	/day
Loaded Labor Rates	$38.05	/hour
Working hours per day	24	/day
Motor Efficiency	0.95	
Horizontal Pump Efficiency	75%	
Molar Ratio Alum Feed	2.0	
Molar Ratio Ferric Chloride Feed	1.3	
Methanol Feed Ratio	3.0	
Density of Methanol	6.6	lb/gal
Density of Citric Acid	10.3	lb/gal
Citric Acid strength	50%	

Based on the cost assumptions provided in Table 6-4, conceptual level annual operations cost estimates were developed for all three MBR process options.

Table 6-5
Operations Cost Summary

	4 mgd	6 mgd	8 mgd
Power Costs ($/year)	$303,000	$318,000	$333,000
Power Cost ($/MG)	$208	$145	$114
Chemical Costs ($/year)	$267,000	$274,000	$281,000
Chemical Cost ($/MG)	$183	$125	$96
Labor Costs ($/year)	$614,000	$768,000	$922,000
Total ($/year)	$1,184,000	$1,360,000	$1,536,000
Diffuser Replacement Costs ($/10 years)	$33,000	$48,000	$65,000
Membrane Replacement Costs ($/10 years)	$910,000	$1,362,000	$1,649,000

Based on the analysis provided above, CDM Smith recommends that the proposed MBR facility layout accommodate expansion so that process equipment can be added on an as needed basis. While the MBR process is robust, it can be prone to issues if the facility is underloaded. The high dissolved oxygen concentrations (5 to 6 mg/L), combined with a high recycle rate from the MBR to the anoxic zone, can be detrimental in nitrogen removal. Care should be taken during final design to mitigate this through the use of VFDs (which will allow turndown capability) on the recycle pumps and, potentially, providing carbon at this tank. The potential for not providing this operational flexibility to turndown recycle flows or add carbon to the first anoxic zone could result in permit limits not being met.

6.2.3 Cost Summary

A summary of costs, capital as well as operations for the proposed new WWTP at initial and ultimate design are summarized in **Table 6-6**. Present worth costs, shown in this table, have been calculated over a 20 year period, assuming a 6 percent interest rate. The present worth costs include capital and operations costs, as well as diffuser and MBR replacement occurring every 10 years.

Table 6-6
MBR Process Option Cost Summary

	4 mgd Option	8 mgd Option
Capital Cost	$59,864,000	$78,115,000
Annual O&M	$1,184,000	$1,536,000
Present Worth (20-year)	$47,189,000	$71,124,044

CDM Smith has evaluated the process options and sized the new WWTP facilities for the flows anticipated for the planning area and period. The proposed design accommodates the advanced treatment processes necessary to meet the stringent permit limits anticipated. The design is also robust and will provide the basis for garnering public acceptance and regulatory support necessary for its permitting. And, ultimately, this design is consistent with meeting the long-term wastewater needs of the City of Franklin within the plan that has been developed to meet total system demands and capacity based on the ultimately growth of the City and its wastewater system.

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Technical Memorandum No. 2

To: *City of Franklin IWRP Team*

From: *CDM Smith*

Date: *June 27, 2012*

Subject: *Integrated Water Resources Plan – Existing WWTP Technical Analysis Biological and Hydraulic Capacity Analysis*

Executive Summary

In this evaluation, a hydraulic and biological treatment capacity assessment of the existing City of Franklin WWTP was conducted. This included a review of future flow projections, analysis of influent wastewater characteristics, and discussion and evaluation of the current NPDES permit requirements, as well as the anticipated future permit requirements. An analysis of the biological and hydraulic treatment capacities at the WWTP were performed to determine where potential constraints currently exist and where process improvements could and should be made.

The biological analysis demonstrated that the addition of fermentation basins ahead of the existing oxidation basins, in conjunction with addition of another denitrification filter, could allow the existing WWTP to biologically treat 16.0 mgd. However, the hydraulic analysis showed that the WWTP cannot hydraulically pass the peak hour flows associated with a 12.0 mgd design; as a result there are hydraulic improvements that are necessary to meet both the peak flows associated with the current rated capacity, as well as the higher peak hydraulic flows associated with a proposed increase in biological capacity.

In summary, the overall cost of improvements to maintain the existing capacity at 12.0 mgd at the existing WWTP is $8.1M. To expand the facility to 16 mgd capacity, the associated costs are estimated at $18.6M. The unit costs for each process have been shown in the event that the City chooses to implement these changes in a phased plan for construction; however, the City must note some economy of scale may be lost with phasing construction.

1.0 Introduction

As part of its Integrated Water Resources Plan (IWRP), the City of Franklin is developing a plan which will address issues associated with aging infrastructure at its existing wastewater treatment plant (WWTP) and address the required capacity to meet future wastewater demands. The facility improvements that are necessary to consistently meet NPDES permit requirements at the design flow of 12 million gallons per day (mgd) have been previously described in Technical Memoranda (TM) No. 1 - *Existing WWTP Technical Analysis Evaluation of Existing Equipment*, which was previously distributed to the IWRP Steering Committee on September 22, 2011. In addition to addressing issues associated with aging equipment nearing the end of its useful life, CDM Smith evaluated the existing WWTP to determine if additional improvements could be implemented at the facility to increase capacity to meet some or all of the future wastewater demands that have been projected for the IWRP planning period. The purpose of this TM is to:

- Evaluate the biological treatment capacity and hydraulic capability that could be achieved at the existing WWTP;
- Identify facility improvements that would be required to allow the facility to meet future anticipated permit limits at the projected capacity; and,
- Provide planning level capital costs associated with these improvements.

2.0 NPDES Permit Requirements

The National Pollution Discharge Elimination System (NPDES) permit for the City of Franklin WWTP authorizes the discharge of treated municipal wastewater from Outfall 001 to the Harpeth River at river mile 85.2. The permit also authorizes unrestricted non-potable reuse of its treated municipal wastewater. The City's current permit which specifies a treatment capacity of 12 mgd was issued on September 30, 2010 and expired on November 30, 2011. This short permit cycle was implemented to allow the Tennessee Department of Environment and Conservation (TDEC) to schedule the permit renewal to be concurrent with other discharges that are within the Harpeth River watershed to be consistent with the State's watershed management approach. The permit renewal application has been submitted to TDEC and a new permit is expected in the coming months. The current NPDES permit requires that the discharge from the WWTP be limited and monitored as specified in **Table 1**.

Table 1
Current NPDES Permit Requirements

Parameters	Effluent Limits						
	Monthly Average Conc. (mg/L)	Monthly Average Amount (lb/day)	Weekly Average Conc. (mg/L)	Weekly Average Amount (lb/day)	Daily Maximum Conc. (mg/L)	Daily Minimum Removal (%)	Daily Minimum
CBOD5 (summer)[1]	4.0	400	6.0	600	8.0	40	-
CBOD5 (winter)[1]	10	1,001	15	1,500	20	40	-
Ammonia as N (summer)[1]	0.4	40	0.6	60	0.8	-	-
Ammonia as N (winter)[1]	1.5	150	2.3	230	3.0	-	-
Total Nitrogen (summer)[1]	5.0	377	-	-	-	-	-
Total Phosphorus as P (summer)[1]	5.0	-	-	-	-	-	-
Suspended Solids (summer)[1]	10	1,001	15	1,501	20	40	-
Suspended Solids (winter)[1]	30	3,002	40	4,003	45	40	-
E. coli (cfu/100mL)	126	-	-	-	941	-	-
Settleable Solids (ml/L)	-	-	-	-	1.0	-	-
Dissolved Oxygen (mg/L)	-	-	-	-	-	-	8.0
pH (standard units)	-	-	-	-	9.0	-	6.0

[1] Note: Summer = May 1 - Oct. 31 and winter = Nov. 1 - Apr. 30.

With increasing focus on addressing nutrients loads to the Harpeth River, it is anticipated that more stringent permit requirements for phosphorus will be forthcoming in future permit cycles. Based on CDM Smith discussions with TDEC and limits that are being published in other middle Tennessee NPDES permits, it is anticipated that a monthly average summer phosphorus limit of less than 1.0 mg/L would be required along with a monitoring requirement for winter discharges. The anticipated changes to permit requirements are summarized in **Table 2**; and, these more restrictive limits anticipated for phosphorus were considered along with limits for other parameters, presented in Table 1used for establishing design criteria for evaluation of the existing WWTP.

Table 2
Additional Design Criteria for Proposed Franklin WWTP Upgrades

Parameters	Monthly Average Concentration (mg/L)
Total Phosphorus as P (summer)	1.0
Insoluble Total Phosphorus as P (summer)	Report
Total Phosphorus as P (winter)	Report

3.0 Existing Treatment Facilities

The existing Franklin WWTP consists of a series of unit processes to remove solids, carbonaceous biochemical oxygen demand (CBOD) and nutrients before the treated wastewater is disinfected for either discharge to the Harpeth River or reuse throughout the City. The following series of unit processes comprise the treatment system at the Franklin WWTP:

- Influent screens
- Vortex grit chambers
- Oxidation ditches
- Final clarifiers
- Denitrification filters
- UV disinfection system
- Post aeration
- Cascade aeration
- Outfall

The influent screen channels at the headworks facility receive the influent flow from the influent pumping station. The screen area consists of channels for two mechanically operated drum screens and an associated bypass channel. From the screens, flow is diverted to the two vortex grit removal units, with associated separate bypass channels provided around these units. Flow enters one of three oxidation ditches; two of which are currently in operation. Following biological treatment, flow is split among six final clarifiers for settling. The clarified effluent from the final clarifiers is pumped to the denitrification filters for final polishing, this process is fed by the filter feed pumps. Filter effluent flows through the UV disinfection system, into the post aeration basin and through the cascade aeration structures before final discharge into the Harpeth River outfall. In addition, a portion of the treated effluent is chlorinated and distributed through the reclaimed water system following pumping from the post aeration basin.

The heart of the treatment system is a biological treatment process configured in a Modified Ludzack-Ettinger (MLE) process to meet the NPDES permit requirements. This configuration uses an anoxic stage followed by an aerobic stage for biological nitrogen removal; the anoxic stage utilizes influent CBOD as a carbon source for denitrification. Biomass as mixed liquor suspended solids (MLSS) is constantly recycled from the aerobic to the anoxic stage to provide nitrate for denitrification. The aerobic zone converts the remaining CBOD and ammonia nitrogen to carbon dioxide, water, nitrate, and biomass; air is provided in the aerobic zone through both mechanical surface aerators and diffused aeration.

The MLSS from the MLE process flows to secondary clarifiers, which provide gravity liquid/solid separation. The majority of the settled mixed liquor, denoted as return activated sludge, is recycled to the anoxic stage of the MLE process to provide a constant source of biomass for the process. The remaining settled mixed liquor is removed from the process as

waste activated sludge and is conveyed to the biosolids processing facilities. Wasting of solids is required so that the bacterial population in the MLE process can be managed; this provides adequate biomass for the treatment process while avoiding the possibility of overloading the secondary clarifiers.

Final biological treatment of the wastewater is accomplished through the denitrification filters. These filters remove both suspended solids and serve as a polishing step to remove nitrate that is not removed in the MLE process. Methanol is added as a carbon source to the denitrifying bacteria that grow on the filter media; this allows the bacteria to respire and grow. A summary of the biological unit processes and associated facilities is provided in **Table 3**.

Table 3
Existing Facility Biological Treatment Process Components

Unit Process	Unit	Value
Oxidation Basin #1		
Anoxic volume	MG[1]	0.72
Aerobic volume	MG	2.58
Mechanical Surface Aerators	hp[2]	3 @ 100 hp each
Supplemental Diffused Air	scfm[3]	5000
Oxidation Basin #2		
Anoxic volume	MG[1]	0.72
Aerobic volume	MG	2.58
Mechanical Surface Aerators	hp	3 @ 100 hp each
Supplemental Diffused Air	scfm	5000
Oxidation Basin #3		
Anoxic volume	MG[1]	0.72
Aerobic volume	MG	2.58
Mechanical Surface Aerators	hp	3 @ 150 hp each
Final Clarifiers		
Number	--	6
Diameter	ft	100
Denitrification Filters		
Number	---	5
Depth of gravel media support	ft	1.5
Depth of sand media	ft	6.0
Surface area, each	ft^2	898

[1] MG = million gallons
[2] hp = horsepower
[3] scfm = standard cubic feet per minute

3.1 Flow Characterization

The design capacity of any WWTP is stated in terms of the annual average daily flow (AADF or ADF). For the years 2008, 2009, and 2010, the ADF at Franklin's WWTP were 8.7, 10.1 and 9.4 mgd, respectively; the average of all three years was 9.6 mgd, which is approximately 80% of the design capacity of the facility. In reality, the flow received at any wastewater facility varies in response to daily, weekly, and seasonal (and long-term) changes in demands in the service area,

as well as seasonal climatic changes, particularly the amount of rainfall. As a result, facilities must be designed to accommodate a range of flows and must be able to pass, hydraulically, the maximum or peak hour flow.

Average wastewater flow projections for the service area were previously developed and presented in the Technical Memorandum, *Water Resources Demand Projections*, dated December 8, 2010. A summary of the projected wastewater demands is presented in **Figure 1.**

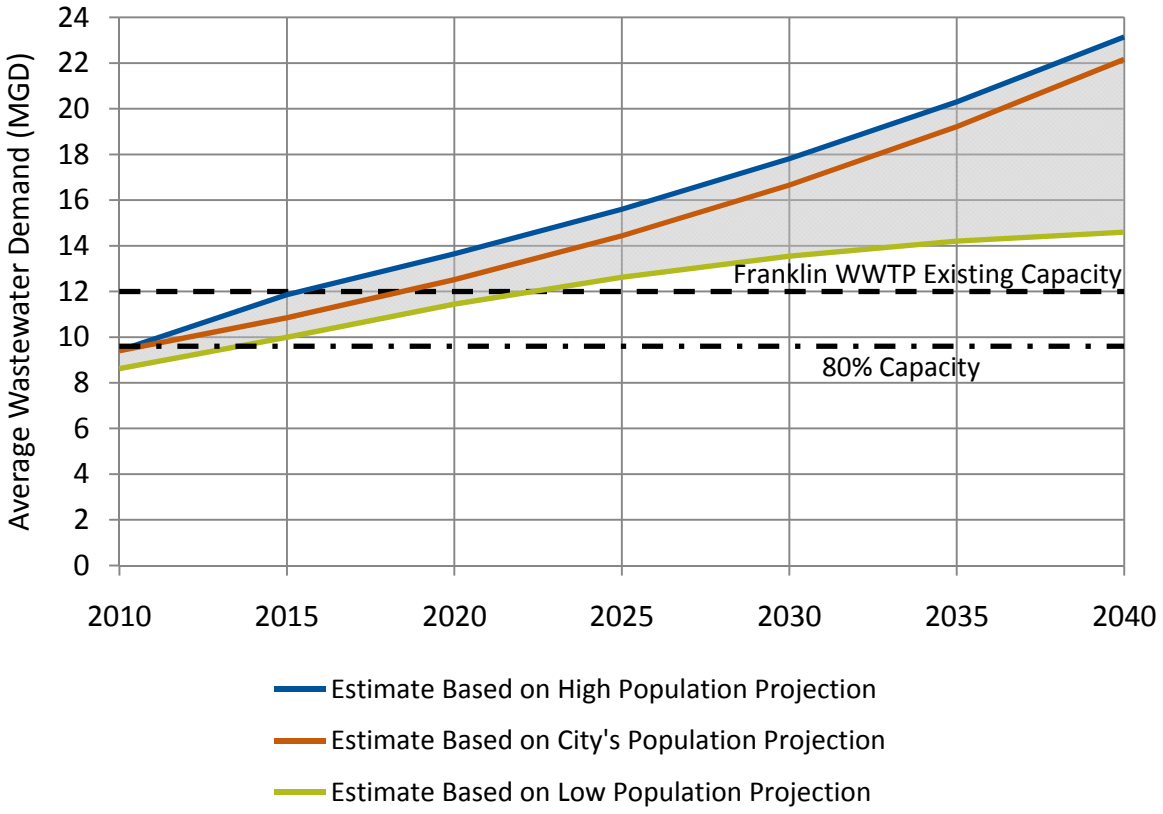


Figure 1
Average Day Wastewater Flow Projections

3.2 Mass Load Characterization

CDM Smith performed an analysis of the influent wastewater using monthly operating report (MOR) data from 2008 to 2010 to define influent characteristics including flow, carbonaceous biochemical oxygen demand CBOD, total suspended solids (TSS), and ammonia nitrogen. The data was analyzed and both the daily and 30-day moving average mass loads were calculated. The average day condition was determined using the average flows and average mass loads found from the data set; average day concentrations were generated from the flow and mass

load data. The maximum month flow and loading conditions were evaluated using the 95th percentile of the 30-day moving average flow and loading data; and the maximum daily conditions were calculated on the 95th percentile of all daily flow and loading data. A summary of the existing loading conditions is provided in **Table 4**.

Table 4
Influent Conditions Based on Historical Data

Data	Unit	Average Daily	Average Day Maximum Month	Maximum Daily
Flow	mgd	9.42	12.2	15.1
	Peaking factor	1.00	1.30	1.60
CBOD5	mg/L	178	-	-
	lbs/day	14,000	17,800	22,400
	Peaking factor	1.00	1.27	1.60
TSS	mg/L	164	-	-
	lbs/day	12,900	15,000	21,300
	Peaking factor	1.00	1.16	1.65
Ammonia	mg/L	12	-	-
	lbs/day	940	1,040	1,330
	Peaking factor	1.00	1.11	1.41

4.0 Future Facility Capacity

In order to determine the potential for expanding the capacity of the existing facility, it is important to clearly define the anticipated flows and loads that must be treated at the facility. Average and peak flows are used to assess the hydraulic capacity of each unit process while wastewater mass loading characterization is used to confirm the capacity of the biological treatment processes. **Table 5** summarizes the design conditions used to evaluate the capacity of the unit treatment processes.

Table 5
Design Conditions for Individual Unit Processes

Unit Process or Operation	Design Condition
Screening	Peak hour flow
Grit removal	Peak hour flow
Anaerobic reactors	Maximum month phosphorus load
Anoxic reactors	Maximum month nitrogen mass load
Aeration tanks	Maximum month BOD_5 mass load
Aeration system	Maximum day oxygen demand
Secondary settling tanks	Max day/peak hour flow
Denitrification Filters	Peak hour flow/maximum month nitrate mass load
Disinfection	Peak hour flow

4.1 Hydraulic Capacity Analysis

In recent years, there have been growing concerns among staff at the City's WWTP with the lack of ability of the plant to pass high flows, particularly during and following heavy rain events. While the WWTP is rated for a 12.0 mgd capacity, that capacity is based on an average daily flow. The most recent design documents from the last facility upgrade (**Figure 2**), showed a design peak flow of 38.4 mgd. CDM Smith performed a hydraulic analysis to determine the actual hydraulic capacity of the facility, as well as identify any hydraulic restrictions that may be preventing the facility from passing the 38.4 mgd flow.

4.1.1 Hydraulic Model

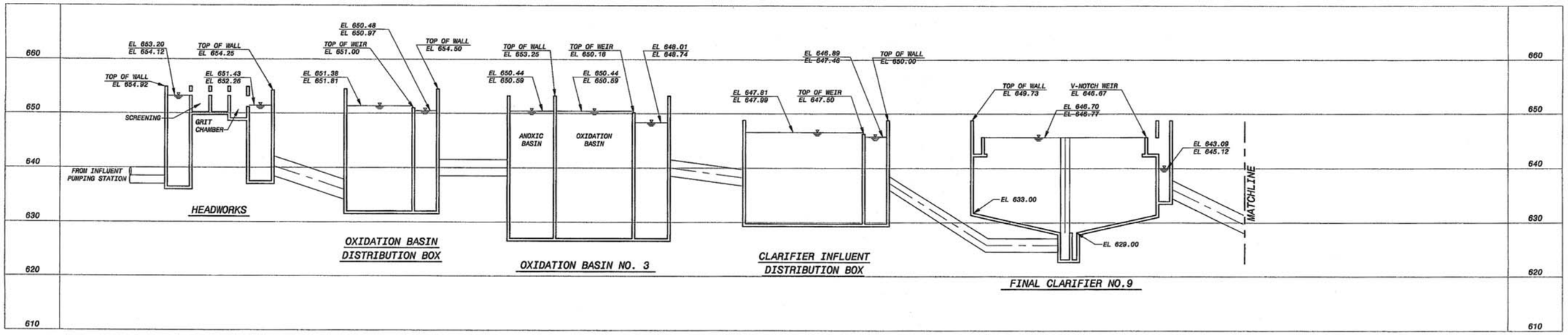
The WWTP hydraulic analysis included an evaluation of a combination of flows ranges throughout the closed conduits, open channels, and hydraulic controls that encompass the entire WWTP (i.e. weirs, slide gates, piping, etc.). The hydraulic analysis began at the most downstream control point, i.e., the water surface elevation at the Harpeth River; hydraulic losses were added to this elevation moving backward through the WWTP processes while incorporating intermediate hydraulic controls and devices. Friction losses in piping and conduits were obtained using the Hazen-Williams equation and open-channel conduit losses were obtained using Manning's equation. Minor head losses at all fittings, transitions, openings, gates, valves, and open channel bends were computed by multiplying the appropriate standardized coefficient by the losses calculated from the flow velocity. The following assumptions and simplifications were made for this analysis:

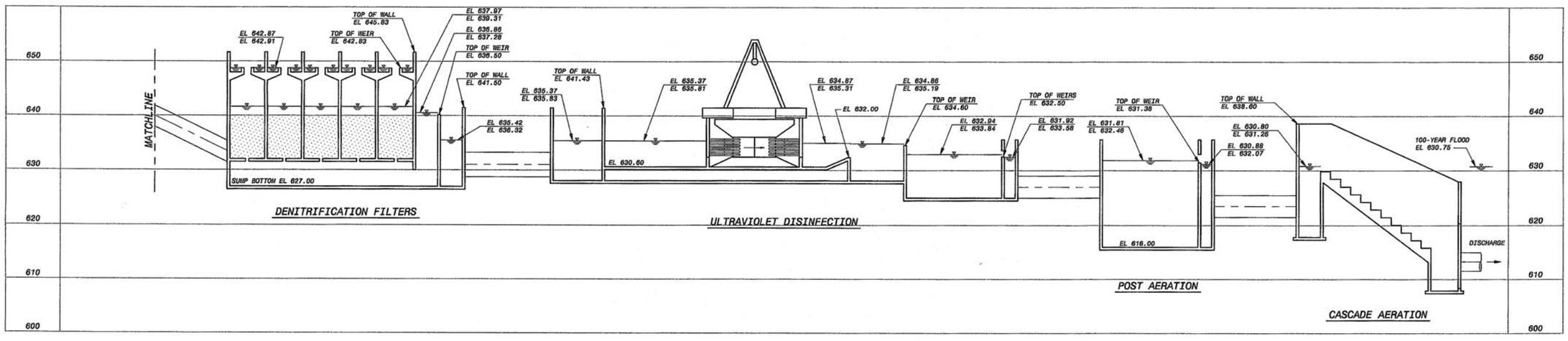
- Steady state flow conditions were assumed throughout the WWTP
- Dimensions and flow paths were constructed as per as-built drawings (1997 and 2005)
- The 100-year flood elevation was assumed for the water surface at the outfall; and
- Sidestream flows were not significant and not included.

4.1.2 Field Verification of the Hydraulic Model

To verify the hydraulic model that was constructed based on as-built drawings and City staff's knowledge of the WWTP field testing was performed at 21.0 mgd on April 5, 2011 and 17.0 mgd on April 19, 2011. Water surface elevations at key hydraulic control points were measured from known elevations, *i.e.*, top of wall or channel. A field survey was also conducted by a professional surveyor as part of the field tests in May 2011 to confirm structural elevations corresponding with drawings.

Data collected from these two sampling events were used to verify the model. Overall, the results of the desk top analysis and field measurements showed good agreement, as shown in **Figure 3**. In almost all cases, the difference between field-measured values and model calculated values were within 2-inches; a very few locations showed a difference of 3 to 5-inches





HYDRAULIC PROFILE

HORZ : NO SCALE
VERT : 1" = 10'-0"

LEGEND:

EG ..... EXISTING GRADE
FG ..... FINISH GRADE
EL ..... ELEVATION
AVERAGE DAILY FLOWS (12 mgd)
PEAK DAILY FLOWS (38.4 mgd)

Figure 2
Hydraulic Grade at Existing Facility - Record Drawings

10/22/04	CONFORMED TO CONSTRUCTION RECORDS	E	SCM TRO MSL
10/16/02	BID SET	D	LRG REO RDL
01/25/02	TDEC REVIEW	C	SCM LRG
11/02/01	LEVEL 2 SUBMITTAL	B	SCM MSL
08/08/01	LEVEL 1 SUBMITTAL	A	KGM GEM
DATE	REVISIONS AND RECORD OF ISSUE	NO.	BY OK APP

CrGNET ID: 97374-3000-325-C-H00001XQ XREF1 ID:
SAVED: MIC11538, 9/8/2005 4:06:47 PM XREF2 ID:
DWG VER #: XREF3 ID:
PLOTTED: MIC11538, 12/30/2004 4:38:38 PM XREF4 ID:
USER: MIC11538 SIF: ACAD-20001 XREF5 ID:

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BLACK & VEATCH Corporation
Nashville, Tennessee

CITY OF FRANKLIN, TENNESSEE
WWTP RENOVATION
CONTRACT 3 - LONG TERM IMPROVEMENTS

HYDRAULIC PROFILE

DESIGNED: GEM, LRG
DETAILED: RDE, SCM
CHECKED: REO
APPROVED: RDL
DATE: 10/16/02

PROJECT NO.
97374

G15

SHEET 15 OF 160

MICHAEL SHANNON LAMBERT REGISTERED ENGINEER AFRICA DIVE SUMMITVILLE NO. 106908 STATE OF TENNESSEE
4/05/05

RECORD DRAWING

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BY: 4/05/05
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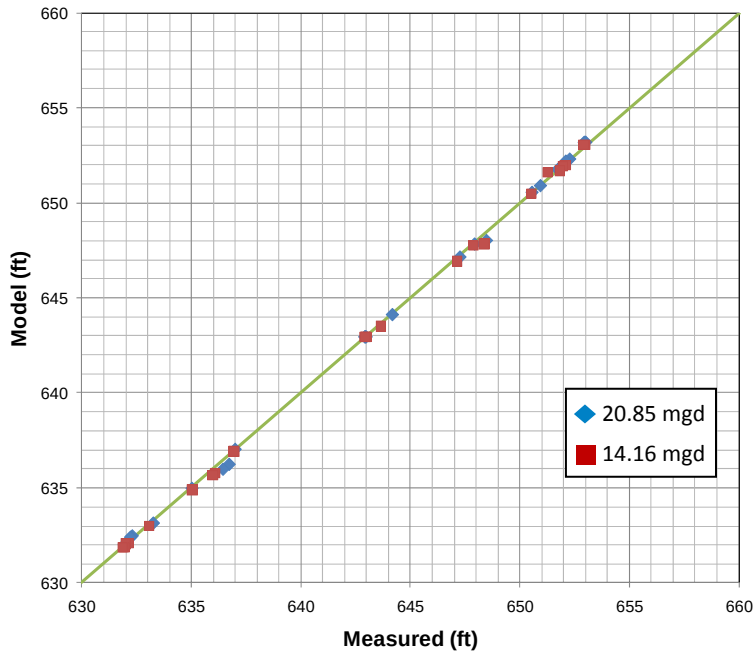

Figure 3
Hydraulic Model Verification using Measured Elevations

between field measurement and model prediction. Because of good agreement between model values and field measurements, no further adjustments were made to the model.

4.1.3 Results of Hydraulic Capacity Analysis

Several factors were considered when determining hydraulic capacities and limits of treatment processes; these included submergence of weirs, freeboard and overflows at structures, capacities of pumps and measurement range of flow metering devices. Reduced freeboard and occurrence of overflows are an indication of severe hydraulic limitations through the treatment components. Weirs and other devices provide hydraulic control for WWTP operations, aid in maintaining minimum depths, and facilitate even flow split between treatment units; however, they must be operated properly to control the plant flow.

Submerged weirs do not provide hydraulic control and can disturb even flow split between process units; this condition can also result in short-circuiting of flow paths which also impacts treatment. With respect to the measurement range of flow metering devices, depth of flow may impact the ability of particular devices to accurately measure flow. While this may not impact the ability of the treatment unit to physically pass the flow, it can have significant impacts if the measurements control other plant processes such as chemical feed rates. Considering these factors, a summary of results of the hydraulic analysis are provided below.

- During the 100-year flood, flooding/overtopping of the cascade aeration system is expected,

- At 27 mgd, submergence of weirs at the final clarifier influent distribution box is expected,
- At 29 mgd, the settled water junction box is expected to be overtopping,
- At 32 mgd, submergence of the overflow weirs at the final clarifiers is expected,
- At 34 mgd, submergence of the weirs at the influent distribution box for the oxidation ditches is expected, and
- At 38.4 mgd, there is a high potential for overflow at the headworks

4.1.4 Recommendations for Hydraulic Capacity Improvements

In order to address the hydraulic limitations, such that the existing hydraulic peak hour flow capacity of 38 mgd could be achieved at the WWTP at the existing 12 mgd rated biological treatment capacity, the following improvements are required at the existing WWTP:

- Construct flow equalization facility and allow 23-mgd of flow through the plant, and divert 15-mgd through flow equalization
- Modify the settled water junction box (raise walls approximately 1.5 feet)
- Continued use of all three oxidation ditches
- Maintain 12-mgd RAS flow

If it is desired to increase the overall treatment capacity of the facility, it is also necessary to increase hydraulic capacity for peak flows. Additional improvements could be made to allow up to 33 mgd to hydraulically pass through the facility, and a description of these upgrades follow:

- Add at least one more UV channel or provide a new UV system with lower headloss
- Provide additional denitrification filter
- Modify the settled water junction box (raise walls approximately 2.5 feet)
- Upsize the denitrification filter influent flow meter and all associated 36-inch piping (new piping needs to be at least 42-inch diameter)
- Raise the weirs in the clarifier distribution box by 1 foot; walls need to be raised at least 1 foot
- Raise the weirs in the oxidation ditch distribution by 0.5 feet
- Raise the walls in all three oxidation ditches by 0.5 feet, and all three ditches should be used for continuous treatment

- Provide an additional screen and grit unit with a pump station between the headworks and oxidation ditches; or provide an entirely new headworks structure to address headloss issues

- Miscellaneous piping and other minor hydraulic modifications throughout the existing plant

4.2 Biological Treatment Capacity Analysis

The mass load characterization presented in Section 3.2 was used to develop design loads for evaluation of the biological treatment capacity at the existing plant. The first step of this process includes converting CBOD and ammonia concentrations, as summarized in Table 4, to BOD and total Kjehdahl nitrogen (TKN) by dividing by 0.84 and 0.66, respectively. Because phosphorus was not monitored for compliance from 2008 through 2010, the value for total phosphorus (TP) concentrations was assumed to be 8.2 mg/L, based on limited data collected at the facility. Finally, the maximum month and maximum day load peaking factors of 1.30 and 1.60, previously determined, were used for resulting calculations. The influent characteristics utilized to develop criteria for all other flow conditions were scaled to known flows as presented in Table 6.

Table 6
Influent Mass Loads – Existing Conditions

Data	Unit	Average Daily	Average Day Maximum Month	Maximum Daily
Flow	mgd	9.42	12.2	15.1
	Peaking factor	1.00	1.30	1.60
BOD5	mg/L	212	-	-
	lbs/day	16,700	21,200	26,700
	Peaking factor	1.00	1.27	1.60
TSS	mg/L	164	-	-
	lbs/day	12,900	15,000	21,300
	Peaking factor	1.00	1.16	1.65
TKN	mg/L	18	-	-
	lbs/day	1,420	1,560	2,000
	Peaking factor	1.00	1.11	1.41
TP	mg/L	8.2	-	-
	lbs/day	640	830	1,020
	Peaking factor	1.00	1.30	1.60

In addition to the influent characteristics presented in Table 6, there were several site and design constraints that were considered in the evaluation to determine the practical limit of increasing the facility capacity as follows:

- A fermentation tank could be added on the site as a detached structure from the existing oxidation tanks,
- Additional denitrification filters could be added on the existing site,
- Additional anoxic or aerobic volume could not be added and the existing MLE configuration would not be modified,
- Additional air through larger mechanical surface aerators or additional supplemental blowers could not be added, and
- Additional final clarifiers could not be added.

After reviewing the influent design flows and loads at several different conditions, coupled with the constraints listed above, CDM Smith determined that the biological capacity at the existing facility could be expanded to 16 mgd ADF by adding new fermentation basins and a new denitrification filter. Supporting components, such as flow equalization and hydraulic upgrades, will also be needed and would need to be sized during final design of the upgraded facilities**.** **Table 7** summarizes the design conditions at 16 mgd ADF.

Table 7
Influent Design Conditions for 16 mgd ADF

Data	Unit	Average Daily	Average Day Maximum Month	Maximum Daily
Flow	mgd	16.0	20.8	25.6
	Peaking factor	1.00	1.30	1.60
BOD	mg/L	212	-	-
	lbs/day	28,300	36,000	45,300
	Peaking factor	1.00	1.27	1.60
TSS	mg/L	164	-	-
	lbs/day	21,900	25,400	36,100
	Peaking factor	1.00	1.16	1.65
TKN	mg/L	18	-	-
	lbs/day	2,400	2,700	3,400
	Peaking factor	1.00	1.11	1.41
TP	mg/L	8.2	-	-
	lbs/day	1,100	1,400	1,800
	Peaking factor	1.00	1.30	1.60

While the capacity of the existing facility could be expanded to 16 mgd, if other process improvements such as anaerobic digestion and additional filtration capacity are implemented, the liquid sidestream from this process could result in additional nitrogen and phosphorus loads to the main treatment process. As a result, it is an important factor to consider this when evaluating the overall treatment capacity of the biological treatment process. **Table 8** summarizes the impacts of the blended influent flow (with recycled sidestreams); it was

assumed that the flow, BOD5, and suspended solids loads were negligible when compared to the impacts of the additional nitrogen and phosphorus.

Table 8
Design Conditions for 16 mgd ADF including Anaerobic Digester Sidestream

Data	Unit	Average Daily	Average Day Maximum Month	Maximum Daily	Peak Hour Flow
Flow	mgd	16.0	20.8	25.6	48
	Peaking factor	1.00	1.30	1.60	
BOD5	mg/L	212	-	-	
	lbs/day	28,300	36,000	45,300	
	Peaking factor	1.00	1.27	1.60	
TSS	mg/L	164	-	-	
	lbs/day	21,900	25,400	36,100	
	Peaking factor	1.00	1.16	1.65	
TKN	mg/L	22	-	-	
	lbs/day	3,000	3,400	4,300	
	Peaking factor	1.00	1.13	1.43	
TP	mg/L	9	-	-	
	lbs/day	1,160	1,500	1,900	
	Peaking factor	1.00	1.30	1.64	

The 16 mgd analysis was evaluated using a qualitative desktop approach and verified with steady-state process model demonstrating that the facility could meet potential future effluent permit limits, previously presented in Table 2. In order to maximize the efficiency of the WWTP, a revised process configuration was proposed and evaluated. The modified process included provisions to facilitate biological removal of phosphorus, by adding anaerobic fermentation basins upstream of the existing MLE process, resulting in an anaerobic/anoxic/aerobic or A2O configuration.

The A2O process allows for simultaneous nitrogen and phosphorus removal. As noted above, the process consists of an anaerobic zone, an anoxic zone, and an aerobic zone. An internal recycle stream returns nitrates from the aerobic zone to the anoxic zone, similar to the MLE process. RAS is recycled to the head of the anaerobic zone along with the influent. The existing activated-sludge basin can be modified to include the anaerobic and anoxic zones, assuming sufficient volume remains in the aerobic zone to perform nitrification. With two recycle streams, some piping and pumping are needed and with only one anoxic zone, if VFAs are sufficient, there might not be a need to supplement the carbon. With A2O, as with all combined nitrogen-phosphorus biological systems, some phosphorus is taken up in the anoxic zone and the sludge residence time in each zone must be sufficient to allow complete phosphate release or uptake. The clarifier also must be operated to regularly waste solids to avoid release of phosphate by the endogenous respiration of the microorganisms that are responsible for phosphorus uptake.

Even with biological phosphorus removal, it was assumed (to provide conservative cost estimates) that chemical phosphorus removal in the clarifiers and/or filters may be necessary to

polish effluent phosphorus to meet anticipated future permit limits. An additional denitrification filter is needed to supplement the existing five filters to treat the resulting higher hydraulic flows; however, it should be noted that even at the expanded 16 mgd capacity, the denitrification filters are only at less than 20% of their nitrate removal capacity based on approximately 75 pounds per day per 1000 cubic feet of media (Metcalf & Eddy, 4th edition; Table 9-20) for a design temperature of 14C. This provides additional flexibility to remove nitrate that could bleed through the secondary treatment process.

The evaluation was run using a 9-day aerobic solids retention time which will promote nitrification during the winter months at a mixed liquor temperature of 14C; a design summary of the desktop evaluation is presented herein in **Table 9**.

While the biological treatment capacity could be increased to 16 mgd with the upgrades described in this section, it will also be necessary to increase the hydraulic capacity to accommodate a higher peak flow of 48 mgd, associated with the 16 mgd biological capacity (3.0 peaking factor for a 16 mgd capacity). This could be accomplished by implementing the following improvements:

- At peak flows, process 33 mgd through the WWTP and divert up to 15 mgd to a new flow equalization basin (as described previously)
- Maintain a 16 mgd RAS flow rate to maintain the appropriate SRT

Table 9
Design Summary for A2O Process at City of Franklin Existing WWTP

Design Consideration	Parameter	Value	
Environmental Conditions	Bioreactor pH	7.2	
	Maximum Water Temperature	25°C	
	Minimum Water Temperature	14°C	
	Maximum Air Temperature	38°C	
	Minimum Air Temperature	0°C	
	Maximum Relative Humidity	100%	
Process Summary	Design Capacity (MGD) Flow	16.00	
	Aerobic Solids Retention Time (days)	9.0	
	Net Yield	0.81	
	MLSS (mg/L)	3,900	
	WAS (lb/day at Average Daily Monthly Maximum)	29,000	
Reactor Volumes	New Fermentation Basin Volume (MG)	0.65	
	Existing Anoxic Basin Volume (MG)	2.16	
	Existing Aerobic Volume (MG)	7.74	
Aeration Requirements	Surface aerators required (existing/new)	9/0	
	Blowers required (existing/new)	1/0	
	Design DO at max day load (mg/L)	1.0	
	Alpha (mechanical aeration/ course bubble diffused air)	0.90/0.70	
	Standard aeration efficiency (lb hp/hr)	3.4	
Secondary Clarifiers	Clarifiers required (existing/new)	6/0	
	Flow Scenario	Average	Peak
	Hydraulic Loading Rate (gpd/sf)	255	764
	Solids Loading Rate (lb/day/sf)	14.0	29.8
Denitrification Filters	Filters required (existing/new)	5/1	
	Flow scenario	Average	Design Peak[1]
	Hydraulic Loading Rate (gpm/sf)	2.1	4.3
	Nitrate Loading Rate (lb/day/kcf)	5.6	10.71
Pumped Recycles	RAS Ratio	0.79	
	Required Internal Recycle Flow (% of Average Daily Monthly Maximum)	99%	
Alum Dosing	% Phosphorus Removed by ChemP (After BioP)	84%	
	Molar Ratio Al:P	1.72	
	Alum Sludge Produced (lb/day)	2,295	

Design peak rate for hydraulic loading is peak hour flow; design peak rate for nitrate load is maximum month nitrate load.

5.0 Cost Analysis

Costs for the improvements to the WWTP that were determined to be required for maintaining the existing biological capacity of 12 mgd with a 38 mgd peak hydraulic flow were developed; similar costs were developed for upgrading the facility to a rated capacity of 16 mgd with a hydraulic peak of 48 mgd. Costs presented in this section are based on the analyses presented in previous section of this document and costs represent conservative estimates. A summary of the facility upgrades that are necessary to meet potentially more stringent permit limits for phosphorus removal are presented in Table 10. A summary of facility improvements that could be implemented to allow the existing facility to meet the anticipated permit requirements at a rated capacity of 16 mgd are provided in Table 11. Assumptions that used in these cost estimates included:

- In either case, a 5.0 million gallon equalization tank is proposed to be installed downstream of the headworks which will attenuate peak hour flows by 15 mgd allowing unit processes downstream to be sized for smaller flows and mitigate the requirement for additional hydraulics upgrades required in order to expand the facility
- The concrete slab for the fermentation tank is 24 inches thick, with 16 inches thick walls that are 18 feet high
- The baffles are assumed to be 12 inches thick and rock excavation is based on similar surrounding regional projects
- The costs assume that half of the reactor is buried, and that there will be three fermentation zones (one for each basin), requiring a total of six mixers
- The new denitrification filters are assumed to have a concrete slab 24 inches thick and walls that are 16 inches thick and 10 feet high; in addition, the assumption is that half of the reactor height is buried
- The manufacturer's proposal for the denitrification filter included costs for additional methanol storage, feed pumps, backwash pumps, and scour pumps for the filters
- The disinfection system is assumed to have a 24 inch thick concrete slab with 16 inch thick concrete walls that are 7 feet high, utilizing common wall technology and assuming that half of the reactor height is buried
- Construction costs include permits, contractor's costs and profit, with all estimated contingencies associated with each task

Table 10
Engineer's Opinion of Probable Construction Cost
for Improvements at Franklin WWTP for 12 mgd Capacity

Component	Unit	Quantity	Unit Cost	Material Cost	Labor, Construction Equipment & Misc. Material (%)	Labor, Construction Equipment & Misc. Material Cost	Sales Tax	COST
Fermentation Tank								
Concrete Slab	CY	536	$350	$187,743	0%	$0	$17,366	$205,000
Concrete Walls	CY	227	$650	$147,502	0%	$0	$13,644	$161,000
Concrete Baffles	CY	108	$650	$70,284	0%	$0	$6,501	$76,800
Rock Excavation	CY	287	$100	$28,685	0%	$0	$2,653	$31,400
Fermentation Mixers	EA	6	$15,000	$90,000	30%	$27,000	$8,325	$125,000
Denitrification Filters								
Concrete Slab	CY		$350	$0	0%	$0	$0	$0
Concrete Walls	CY		$800	$0	0%	$0	$0	$0
Rock Excavation	CY		$100	$0	0%	$0	$0	$0
Denitrification Filter	LS			$0	30%	$0	$0	$0
Flow Equalization								
Equalization Tank	EA	1	$1,565,000	$1,565,000	30%	$469,500	$144,763	$2,180,000
Hydraulic Upgrades								
Clarifier Distribution Box Modifications	EA	0	$204,142	$0	0%	$0	$0	$0
Oxidation Ditch Distribution Box Modifications	EA	0	$158,099	$0	0%	$0	$0	$0
Filter Influent Flow Meter Modifications	EA	0	$140,739	$0	0%	$0	$0	$0
Settled Water Junction Box Modifications	EA	1	$99,707	$99,707	0%	$0	$9,223	$109,000
SUBTOTAL								**$2,880,000**
Civil Allowance		10%						$288,000
Mechanical Piping Allowance		15%						$432,000
Instrumentation & Electrical Allowance		25%						$720,000
CONSTRUCTION SUBTOTAL								**$4,320,000**
Permits		1%						$43,200
Builder's Risk		1%						$21,600
General Liability		1%						$43,200
GC Bonds		2%						$64,800
CONSTRUCTION SUBTOTAL								**$4,500,000**
Contractor's Overhead and Profit		10%						$450,000
General Conditions		10%						$450,000
Contingency		30%						$1,350,000
TOTAL ESTIMATED CONSTRUCTION COST								**$6,740,000**
Design & Construction Services		15%						$1,010,000
City Project Administration		2%						$135,000
Legal/Finance		3%						$202,000
TOTAL ESTIMATED CAPITAL COST								**$8.09M**

Table 11
Engineer's Opinion of Probable Construction Cost
for Improvements at Franklin WWTP for 16 mgd Capacity

Component	Unit	Quantity	Unit Cost	Material Cost	Labor, Construction Equipment & Misc. Material (%)	Labor, Construction Equipment & Misc. Material Cost	Sales Tax	COST
Headworks								
Screens	EA	1	$152,000	$152,000	30%	$45,600	$14,060	$212,000
Pump Station	LS	1	$600,000	$600,000	30%	$180,000	$55,500	$836,000
Grit Unit	EA	1	$124,000	$124,000	30%	$37,200	$11,470	$173,000
Fermentation Tank								
Concrete Slab	CY	536	$350	$187,743	0%	$0	$17,366	$206,000
Concrete Walls	CY	227	$650	$147,502	0%	$0	$13,644	$162,000
Concrete Baffles	CY	108	$650	$70,284	0%	$0		$71,000
Rock Excavation	CY	287	$100	$28,685	0%	$0	$2,653	$32,000
Fermentation Mixers	EA	6	$15,000	$90,000	30%	$27,000	$8,325	$126,000
Filters								
Concrete Slab	CY	67	$350	$23,275	0%	$0	$2,153	$25,500
Concrete Walls	CY	88	$800	$70,400	0%	$0	$6,512	$77,000
Rock Excavation	CY	166	$100	$16,600	0%	$0	$1,536	$18,200
Denitrification filter	LS	1	$348,900	$348,900	30%	$104,670	$32,273	$486,000
Flow Equalization								
Tank	EA	1	$1,565,000	$1,565,000	30%	$469,500	$144,763	$2,180,000
Disinfection								
Concrete Slab	CY	18	$350	$6,300	0%	$0	$0	$6,300
Concrete Walls	CY	25	$800	$20,000	0%	$0	$0	$20,000
Rock Excavation	CY	32	$100	$3,200	0%	$0	$0	$3,200
UV	LS	1	$960,000	$960,000	30%	$288,000	$88,800	$1,340,000
Hydraulic Upgrades								
Clarifier Distribution Box Modifications	EA	1	$204,142	$204,142	0%	$0	$18,883	$224,000
Oxidation Ditch Distribution Box Modifications	EA	1	$158,099	$158,099	0%	$0	$14,624	$173,000
Filter Influent Flow Meter Modifications	EA	1	$140,739	$140,739	0%	$0	$13,018	$154,000
Settled Water Junction Box Modifications	EA	1	$99,707	$99,707	0%	$0	$9,223	$109,000
SUBTOTAL								**$6,640,000**
Civil Allowance		10%						$664,000
Mechanical Piping Allowance		15%						$996,000
Instrumentation & Electrical Allowance		25%						$1,660,000
CONSTRUCTION SUBTOTAL								**$9,960,000**
Permits		1%						$99,600
Builder's Risk		1%						$49,800
General Liability		1%						$99,600
GC Bonds		2%						$150,000
CONSTRUCTION SUBTOTAL								**$10,400,000**
Contractor's Overhead and Profit		10%						$1,040,000
General Conditions		10%						$1,040,000
Contingency		30%						$3,120,000
TOTAL ESTIMATED CONSTRUCTION COST								**$15,600,000**
Design & Construction Service		15%						$2,340,000
City Project Administration		2%						$312,000
Legal/Finance		3%						$468,000
TOTAL ESTIMATED CAPITAL COST								**$18.7M**

6.0 Summary

In this evaluation, a hydraulic and biological treatment capacity assessment of the existing City of Franklin WWTP was conducted. This included a review of future flow projections, analysis of influent wastewater characteristics, and discussion and evaluation of the current NPDES permit requirements, as well as the anticipated future permit requirements. An analysis of the biological and hydraulic treatment capacities at the WWTP were performed to determine where potential constraints occur and where improvements can and should be made.

The hydraulic analysis, discussed in Section 4.1 was conducted in parallel to the biological treatment capacity analysis, discussed in Section 4.2. The hydraulic analysis identified several bottlenecks at the current rated plant capacity. Assuming that all of the basins at the existing WWTP are in service with no flow bypassed to the old WWTP, improvements are recommended and necessary for the WWTP to maintain the existing biological capacity of 12 mgd with a hydraulic peak flow capacity of 38 mgd.

The biological analysis demonstrated that the addition of fermentation basins ahead of the existing oxidation basins in conjunction with the addition of another denitrification filter could allow the existing WWTP to biologically treat 16.0 mgd. And, the hydraulic analysis showed that the WWTP could already not hydraulically pass the peak hour flows associated with a 12.0 mgd design; as a result there are additional hydraulic improvements necessary to meet the higher peak hydraulic flows associated with the increased capacity.

In summary, the overall cost of improvements to maintain the existing capacity at 12.0 mgd at the existing WWTP is $8.1M. To expand the facility to 16 mgd capacity, the associated costs are estimated at $18.6M. The unit costs for each process have been shown in the event that the City chooses to implement these changes in a phased plan for construction.

City of Franklin, Tennessee

Integrated Water Resources Plan

Appendix E:

Biosolids

- Biosolids Technical Analysis:
  Evaluation of Existing Equipment and Sludge Production Forecast
- Biosolids Technical Analysis:
  TM 2A - Evaluation of One Versus Two Biosolids Facilities
- Biosolids Technical Analysis:
  TM No. 2 - Evaluation of Biosolids Alternatives and Technologies
- Biosolids Report

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Technical Memorandum No. 1

To: *City of Franklin IWRP Steering Committee*

From: *CDM*

Date: *April 14, 2011*

Subject: *Integrated Water Resources Plan – Biosolids Technical Analysis*
Evaluation of Existing Equipment and Sludge Production Forecast

Executive Summary

As part of its Integrated Water Resources Plan (IWRP), the City of Franklin is developing a plan to improve the solids handling facilities at its wastewater treatment plant (WWTP). This technical memorandum (TM) describes the existing conditions identified during site visits to the WWTP, reviews the historical monthly operating reports (MORs), and provides a preliminary analysis of the current operation and maintenance (O&M) requirements and costs of solids treatment and an overview of the solids projections based on current conditions and the availability of existing equipment to meet future needs of the WWTP. The following summarizes the findings of this evaluation.

The facility assessment conducted to evaluate the existing solids process revealed the following regarding equipment condition:

- All equipment is in poor condition, requiring a significant amount of maintenance.
- All equipment is near the end of its service life.
- Substantial rehabilitation and maintenance will be required to keep the equipment at functioning capacity.

A review of the historical records on biosolids operations in conjunction with estimates of future wastewater capacity demand, as described in the "City of Franklin IWRP, Phase II Water Resources Demand Projection Technical Memorandum" (December 8, 2010) were used to estimate solids production for developing engineering estimates for biosolids alternatives. A summary of the biosolids production and costs includes the following:

- The average waste activated sludge (WAS) production is 820 pounds per million gallons (mg) of wastewater treated.
- By 2040, the Franklin WWTP will produce and be required to treat an average of 18,200 pounds of WAS per day.

- The current cost to treat solids is approximately $79 per wet ton or $547 per dry ton.
- These costs will be used to compare new treatment and disposal alternatives to the existing treatment and disposal process.

The existing facility capacity compared with future demands shows that the existing thickening and dewatering processes lack the capacity required to treat the quantity of solids projected to be produced by 2040, and additional process units or replacement with new treatment technologies will be required in order to handle the anticipated solids production.

1.0 Introduction

As part of its IWRP, the City of Franklin is developing a plan to improve the solids handling facilities at its wastewater treatment plant WWTP. This TM describes the existing conditions identified during site visits to the WWTP, reviews the historical MORs, and provides a preliminary analysis of the current O&M requirements and costs of solids treatment and an overview of the solids projections based on current conditions and the availability of existing equipment to meet future needs of the WWTP.

Biosolids Workshop No. 1 was held on February 2, 2011 to determine the preliminary alternatives (individual treatment technologies and potential solids process trains) that will be further analyzed during the alternatives selection process. The results of this analysis, including planning level costs and a life cycle cost analysis, will be discussed in future TMs and presented at Workshop No. 2.

2.0 Condition Assessment – Existing Solids Processes

In the 1990s, the City implemented a Class A biosolids stabilization process called autothermal thermophilic aerobic digestion (ATAD). Because the ATAD process, as implemented, was extremely odorous, this method of stabilization was soon abandoned. Currently, WAS produced at the WWTP is thickened by dissolved air flotation thickeners (DAFT) and dewatered by belt filter presses (BFPs). Solids are continuously thickened, stored during off times of dewatering operation, and dewatered 24 hours per day, 4 days per week. The dewatered sludge is transported approximately 108 miles (one way) to a private landfill in Camden, TN for disposal. The City uses three City-owned trucks and employs three full-time drivers in its disposal operations.

CDM completed an evaluation and condition assessment of the existing facilities at the WWTP as part of the work completed for the Integrated Water Resources Plan – Task 2. Although the analysis was conducted for the entire WWTP, only the solids handling facilities are discussed in this TM. Condition assessments have been performed using a rating scale for the existing equipment. **Table 1** provides a summary of the general condition rating system. A more detailed analysis and rating system of the existing WWTP can be found in the "Existing Facilities" TM.

Table 1
General Condition Rating Definitions

Condition	General Rating Definitions
(1) Excellent	New or almost new equipment in excellent condition. Asset is fully functional as designed with no visible defects or wear.
	Asset requires only normal preventive maintenance.
	Asset is assumed to have 100 - 75% of its effective life remaining.
(2) Good	Asset is fully functional for current operating conditions, shows signs of only minor wear.
	Asset requires normal preventive maintenance and only minor corrective maintenance.
	Asset may have been very recently overhauled or rebuilt.
	Asset is assumed to have approximately 75 - 25% of its effective life remaining.
(3) Fair	Asset exhibits normal or slightly excessive wear but is functionally sound.
	Asset requires normal preventive maintenance and occasional major corrective maintenance.
	Asset is assumed to have approximately 25% or less of its effective life remaining.
(4) Poor	Asset functions but only with a high degree of maintenance or does not function as needed for current operating conditions, indicating that the asset is near the end of its design life.
	Asset requires significant rehabilitation.
	Asset is assumed to be beyond its effective life.
(5) Failed, Imminent, Failure, Obsolete	Asset has failed or will fail imminently and is virtually unserviceable.
	Asset consistently fails to perform at or near its design capacity and no replacement parts are available.
	Asset can be assumed to have reached the end of its effective life and requires immediate replacement.
(0) Abandoned	Asset is abandoned in place; this equipment may only need minimal maintenance to be placed back into service.

2.1 Waste Activated Sludge Pumps

The WAS pumps send sludge from the secondary clarifiers to the dissolved air flotation system for thickening. There are two WAS pumping stations associated with the secondary clarifiers. The first station receives flow from clarifiers 5 through 8 (**Figure 1**), and the second station receives flow from clarifiers 9 and 10 (**Figure 2**). Each WAS pump is dedicated to a single clarifier and is a horizontal self-priming centrifugal pump.

Figure 1
WAS Pumps for Clarifiers 5 through 8

Figure 2
WAS Pumps for Clarifiers 9 and 10

2.1.1 Findings – WAS Pumps

The WAS pumps have received a condition rating of 4. During the inspection, the pumps appeared to be original from the 1996 WWTP expansion with the addition of the new clarifiers, and various belt drives appeared to need tightening and alignment. These pumps are nearing the end of their expected service life and will require frequent routine maintenance for consistent operation. The pumps in the second station between clarifiers 9 and 10 look new. However, they will require routine maintenance to allow for consistent operation.

2.2 Thickening – Dissolved Air Flotation Thickeners

The components associated with the DAFTs include two DAFT tanks, scum skimmers, and sludge rakes driven by two ¾ hp flotator drives, two 30 hp recycle pumps, two 10 hp thickened sludge pumps, and two 15 hp air compressors. As WAS is pumped from the secondary clarifiers and enters the DAFT tanks, the lighter solids float and heavier solids sink. Solids are removed from the tank and pumped to the thickened sludge tank (**Figure 3**), sludge storage tanks, or the BFP feed tank (**Figure 4**). Typically, these tanks are used during the week as temporary storage, as the BFPs are continuously operating. On the weekends, however, solids are stored in these tanks until Monday, when the BFPs begin operation and these tanks are incrementally emptied and bled back into the solids stream in preparation for the next weekend.

2.2.1 Findings – Thickening System

The DAFT system has received a condition rating of 4. The DAFT pumping components, switchgear, compressors, and pressurization tanks are approaching their expected service life of 20 years. The efficiency of this equipment should be further evaluated. Plant staff indicated that this equipment was installed during the 1996 upgrade.

Figure 3
Thickened Sludge Tank

Figure 4
BFP Feed Tank

2.3 Dewatering - Belt Filter Press

There are two BFPs used to dewater sludge before ultimate disposal in a landfill. The system consists of two Ashbrook-Klampress extended-gravity, eight-roller BFPs, two BFP feed pumps, BFP feed tanks, sludge storage tanks, a polymer injection system, and a BFP conveyor system. Sludge from the DAFT units is pumped to holding tanks, before it is conditioned with a polymer solution (**Figure 5**). The conditioned sludge is then pumped to the BFPs by two progressive-cavity BFP feed pumps (**Figure 6**). The top section of the BFP is a gravity drainage section, where sludge is raked into ridges and furrows to enhance drainage. The two belts come together and roll through eight progressively smaller rollers (**Figure 7**) where excess water is squeezed out. These units were installed in 1996 and are approximately 15 years old. The BFPs are controlled by a local programmable logic controller (PLC).

Figure 5
Polymer Injection

Figure 6
BFP Feed Pumps

Figure 7
Polymer Injection

2.3.1 Findings – Dewatering System

The BFP system has received a condition rating of 4. Discussions with WWTP staff revealed unequal distribution of sludge between the two BFPs due to inadequate splitting of the sludge in the piping (**Figure 8**). It was also noticed that, in some places, sludge sticks to the belt. Sludge sticking to the belt is an indication of uneven mixing or overdosing of polymer. There is also limited additional storage space to store thickened sludge in an emergency or if pressing operations are stopped.

The sludge also contains grit and has been found to wear on the rubber helical lobe impellers of the feed pumps. On these pumps, the rotors and stators wear at a quicker pace than expected. If grit is passing through the system to solids processing, the headworks and internal piping should be thoroughly investigated for potential contributions of grit to the system. There is storage in the building of additional spare impellers, rotors, and stators for easy access during routine maintenance.

2.4 Summary of Condition Assessment

The complete WWTP condition assessment will be included in the "Existing WWTP" TM that will be published at a later date. Overall, the solids equipment has been rated poor and currently functions with a high rate of maintenance and attention. Significant rehabilitation is required for the BFPs, as both units run constantly and no redundancy is provided. The City expects to perform a major overhaul of the BFPs during the coming fiscal year and is accepting bids for the overhaul work. All equipment is nearing the expected service life, and significant rehabilitation and maintenance will be required to keep the equipment operating at functioning capacity.

Figure 8
Thickened Sludge Distribution Piping

3.0 Existing Conditions and Solids Mass Balance

3.1 Historical Data Analysis

CDM analyzed plant influent flow and wastewater quality data from 2008 through 2010. Historical trends for flow, 5-day carbonaceous biochemical oxygen demand (CBOD5) and total suspended solids (TSS) are summarized in **Table 2**.

Table 2
Summary of Historical Influent Data

Year	Average Influent Flow (mgd)	Average Influent $CBOD_5$ (mg/L)	Average Influent TSS (mg/L)
2008	8.7	212	190
2009	10.1	188	170
2010	9.4	180	163
Average	*9.4*	*193*	*174*

The 2008 through 2010 data were analyzed, and mass loadings and peaking factors were developed. The average day maximum month (ADMM) peaking factor was generated by taking the 95th percentile 30-day moving average load and dividing it by the average daily load from 2008 through 2010. The maximum day (MD) peaking factor was generated by taking the 95th percentile daily load and dividing it by the average daily load from 2008 through 2010. **Table 3** summarizes the mass loadings and peaking factors for the WWTP.

Table 3
Raw Influent Mass Loading and Peaking Factors

Influent Data	Annual Average Day	Average Day Maximum Month	Maximum Day	Average Day Maximum Month Peaking Factor
Flow (mgd)	9.4	-	-	
$CBOD_5$ (mg/L)	178	-	-	
$CBOD_5$ (lbs/day)	14,025	17,680	23,178	1.7
TSS (mg/L)	164	-	-	
TSS (lbs/day)	12,872	14,876	21,109	1.6

Figure 9 shows a process flow diagram and the mass balance for the solids handling process, along with quantities for each process. These quantities were calculated from reviews of the MORs from 2008 to 2010, as well as daily flow calculations tabulated by WWTP staff.

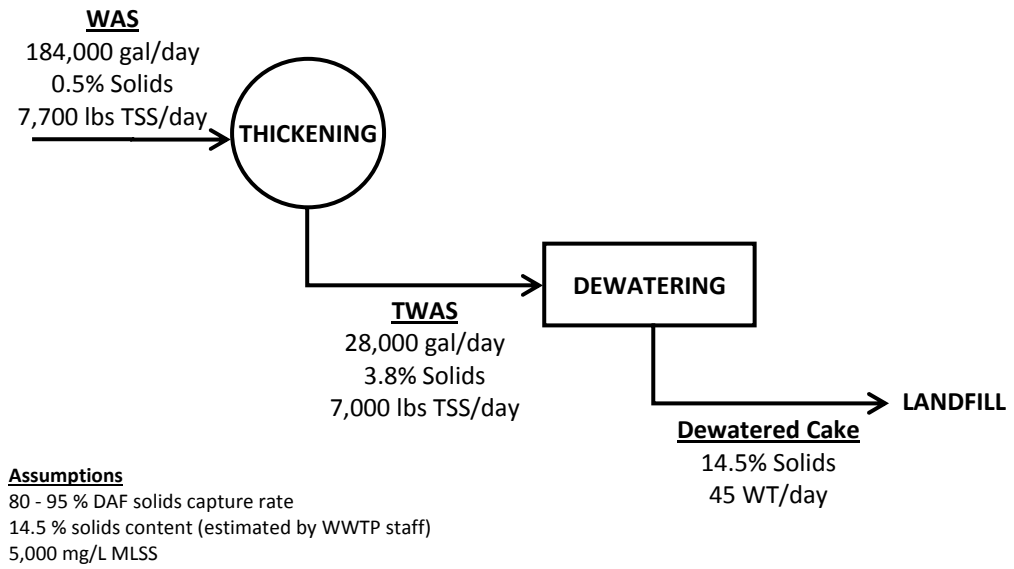

Figure 9
Existing Solids Process Train

Table 4 summarizes the historical daily production of sludge at various stages during the solids processing train.

Table 4
Historical Solids Processing and Disposal Information

Year	WAS to DAFT		TWAS to BFPs		BFP Sludge to Landfill[1]		
	Flow (gal/day)	TSS (lbs/day)	Flow (gal/day)	TSS (lbs/day)	WT/ day	DT/ day	Trucks/ day
2008	169,208	7,056	22,235	6,350	43.1	6.25	1.96
2009	209,441	8,734	30,362	7,860	45.4	6.59	2.06
2010	174,625	7,282	31,141	6,554	44.6	6.43	2.03
Average	*184,222*	*7,682*	*27,895*	*6,914*	*44.4*	*6.43*	*2.02*

[1] Per dewatering operating day (four days per week).

An average of approximately 45 WT of sludge is produced at the WWTP per operating day. Currently, the sludge does not need to be stabilized for landfill disposal. However, all sludge is required to pass the paint filter test, which determines that it is not a liquid waste. Testing has been performed for metals and toxicity characteristics in addition to the paint filter test. Conversations with operators indicate that the dewatered cake is maintained around 14.5 percent when it reaches the truck for disposal. Because thickened sludge is dewatered 4 days per week, the remaining sludge is stored and incrementally bled back into the solids train throughout the following week. Solids disposal requires approximately three trucks per day for transporting and disposing of sludge at the landfill and maintaining a truck for loading at the WWTP.

3.2 Design Criteria

Based on the solids production and influent CBOD5 mass loading, the biological growth yield was calculated for average day conditions at the WWTP. These calculations are presented in **Table 5**. Daily solids production values were also compared to average daily flow. Therefore, the average WAS production-to-flow ratio was calculated as 820 lbs WAS for every million gallons of wastewater (lbs/mg) treated at the WWTP, and the maximum month WAS production to flow ratio was 1,056 lbs/mg. These values were used, along with projected wastewater flows, to create the solids production forecasts that are discussed in the next section.

Table 5
Summary of Solids Production and Yield

Year	WAS Production		Average Daily $CBOD_5$ Mass Loading (lbs/day)	Yield (lbs WAS/ lb $CBOD_5$ removed)	WAS Production to Flow Ratio		
	Avg Day (lbs/day)	Max Month (lbs/day)			Avg Day (lbs/mg)	Max Month (lbs/mg)	Peaking Factor
2008	7,056	8,144	14,246	0.50	814	940	1.15
2009	8,734	10,287	14,716	0.60	865	1,019	1.18
2010	7,282	9,777	13,147	0.56	777	1,044	1.34
Average	*7,682*	*9,902*	*14,025*	*0.55*	*820*	*1,056*	*1.29*

While reviewing these solids production calculations, CDM noted that the CBOD5 yield is lower than typically observed. The WWTP should continue monitoring the solids stream to ensure that the data provided are consistent with the additional monitoring results.

4.0 Future Solids Processing

In order to determine the planning criteria for the WWTP, the design solids treatment capacity will be described in terms of the maximum month ADF (i.e., ADMM) that the solids processes are expected to treat. As shown in Table 5, the average day and maximum month WAS production-to-flow ratios are 820 and 1,060 lbs/mg, respectively. These numbers were multiplied by the

average daily wastewater flow projections provided in the "Water Resources Demand Projections" TM (December 2010) to obtain the average day and maximum month solids production (lbs/day) through 2040.

These solids projections were performed based on the existing WWTP processes that create the solids that must be treated. Although these projections are subject to change based on process changes, they represent a good base point for forecasting future solids production.

Table 6 summarizes the projected wastewater flows and WAS production through 2040. This table uses the average and maximum month flow ratio values (820 lb/mg and 1,060 lb/mg seen in Table 5). These average and maximum month values are used to estimate the WAS production and are based off of actual operating data for the WAS production from the clarifiers observed from the WWTP during the records review.

Table 6
Projected Wastewater Flows and Estimated WAS Production, 2010 to 2040

Year	Projected Average Daily Flow (mgd)	Estimated WAS Production (lbs/day)	
		Average Day (@ 820 lbs/mg)	Maximum Month (@1,060 lbs/mg)
2010	9.4	7,700	9,900
2015	10.8	8,900	11,400
2020	12.5	10,200	13,200
2025	14.4	11,800	15,200
2030	16.7	13,700	17,600
2035	19.2	15,700	20,300
2040	22.2	18,200	23,500

5.0 Existing WWTP Solids Operating Costs

CDM also conducted a review of the WWTP's existing utility and operations records to determine the current cost to treat the sludge. For input into these calculations, costs typically observed for routine maintenance have been estimated. Estimated costs also incorporate labor, polymer usage, odor control chemical usage, equipment electrical usage, and hauling and landfill fees. Typically, O&M costs are also included in this calculation for routine maintenance. However, the WWTP currently does not maintain a comprehensive maintenance management system (CMMS), and historical trending of the O&M requirements is difficult to obtain. In lieu of O&M cost data, a conservative amount has been added into the calculations to account for routine maintenance. **Table 7** provides a summary of the existing cost for treatment and disposal per ton of sludge produced at the WWTP.

Table 7
Existing WWTP - Current Solids Handling Operating Costs

Solids Processing Personnel		
Labor (WWTP & Hauling)		
WWTP Labor - # of employees dedicated to solids process	2.3	
Driver Labor - # of drivers for solids hauling	3	
Hourly Rate	$35.08	
Total Labor Cost	**$386,722**	**per year**
Solids Processing Equipment		
Power		
Annual power consumption	1,164,226	kWh/year
Annual power cost	**$66,063**	per year
Polymer		
Estimated polymer consumption	1.5	drums per week
Estimated polymer unit cost	$1.57	per pound
Annual polymer consumption	35,100	lbs polymer consumed
Total polymer cost	**$55,107**	per year
Odor Control		
12.5% Hypochlorite solution	$1.57	per gallon
25% Sodium hydroxide solution (caustic)	$1.54	per gallon
Muriatic acid	$2.38	per gallon
Estimated usage of hypo for solids odor control	2,000	gallons per year
Estimated usage of NaOH for solids odor control	1,500	gallons per year
Estimated usage of muriatic acid for solids odor control	1,400	gallons per year
Total odor control chemical cost	**$8,782**	**per year**
Other Costs		
Other O&M costs	$20,000	per year
Heating tank parts & labor	$6,000	per year
Total Other Costs	**$26,000**	**per year**
Total Solids Processing Costs	**$155,952**	**per year**
Solids Disposal		
Hauling		
Wet sludge disposed by truck	920	tons per month
Landfill Fees		
Tipping fee at landfill	*$22.24*	*per wet ton*
Total Landfill Fees	***$245,530***	***per year***
Mileage		
Truck capacity	*22*	*wet tons*
Fuel cost	*$4.00*	*per gallon*
Miles to Camden landfill	*108*	*per way*
Truck fuel consumption rate	*8*	*miles/gallon*

Table 7 (cont'd)
Existing WWTP - Current Solids Handling Operating Costs

Annual Fuel Cost	***$54,196***	***per year***
Insurance	$440	per year
Fleet maintenance	$6,480	per year
Truck Replacement		
Truck replacement timeline	*5*	*Years*
New truck cost	*$130,000*	
Annual Truck Replacement Cost	***$26,000***	***per year***
Total Solids Disposal Costs	**$332,646**	**per year**
TOTAL O&M COSTS	**$875,319**	**per year**
Cost per ton of processed sludge	**$79 per wet ton**	**$547 per dry ton**

Estimations and assumptions were made for certain cost items that were not provided or are not solely dedicated to solids processing. For example, the WWTP has six electrical meters, but all six of them are routed to varying parts of the WWTP; there is no single meter providing power to a single process. Power usage was assumed for the DAFTs, sludge pumps, and GBTs based on the total motor horsepower associated with each of these processes. Similar estimations were also made for the odor control chemicals, since varying parts of the WWTP are supplied by the odor control system.

Electrical usage was estimated—specifically, from the motor horsepower size—and calculated based on the amount of usage during operations. For example, the belt filter presses have a total motor horsepower (BFP feed pumps, belt drive, hydraulic power unit, wash water pump, conveyor, and polymer mix pump) draw of 65 hp combined for both units when in operation. These units are powered 24 hours per day, 4 days per week. Multiplying by a conversion factor of 0.746 kW hr per horsepower, CDM determined a total electrical usage of 242,062 kW hr per year for the dewatering process. Electrical usage for each unit in the solids processing train (DAFTs, BFP feed tank, and BFPs) was calculated, and a total annual electrical usage was determined for the system. Electrical usage for the WAS pumps and odor control unit was assumed to remain constant, despite process changes, and was not considered in these calculations.

From this table, the O&M cost is approximately $79 per WT, or $547 per DT of sludge produced at the WWTP on an annual basis. This cost will be used during the design process to compare new treatment and disposal alternatives to the existing WWTP.

6.0 Summary

This TM has provided a condition assessment of the existing facilities, summarized the mass balance that has been performed around the solids treatment facilities, determined the current cost to treat sludge produced at the WWTP, and projected solids production for the design period of the project.

Based on CDM's analysis of the existing data and calculation of solids projections for the planning period, a significant increase in solids production is anticipated for the planning period. **Tables 8** through **10** summarize the existing solids treatment process and the projected demands on the equipment throughout the planning period.

Table 8
Existing Solids Treatment Train - Thickening Capacity Analysis

Year	Operation Schedule		Max Month Solids Loading (lbs/day)	Number of Units		Solids Loading Rate (lbs/hr/ft^2)	
	Hours/ Day	Days/ Week		Duty	Standby	Desired	Effective
2008 to 2010	24	7	9,933	1	1	0.4 to 1	0.59
2020			13,300	1	1		0.78
2030			17,800	2	1		0.52
2040			23,600	2	1		0.70

Table 8 shows that operating one of the two existing DAFT tanks will be sufficient through 2020. However, by 2030, two duty tanks will be required in order to maintain an acceptable solids loading on the thickeners. Therefore, the thickening facilities will need to be expanded via the addition of a third DAFT tank or replaced with a different process.

Table 9
Existing Solids Treatment Train - Thickened WAS Storage Analysis

Parameter	Value
Current TWAS storage capacity (gallons)	150,000
Desired TWAS storage (days)	3
Storage capacity required at maximum month conditions	
2020	130,000
2030	173,000
2040	230,000

Table 9 shows that the existing TWAS storage capacity will be exceeded during the 2020 to 2030 planning period. It should be noted that the above calculations were determined based on the present operating schedule of the BFPs. If additional days of BFP downtime are required (i.e., due to holidays, equipment failure, etc.), then additional storage capacity will be needed sooner.

Table 10
Existing Solids Treatment Train - Dewatering Capacity Analysis

Year	Op. Schedule		Max Month Solids Loading (lbs/day)	Number of Units		Solids Loading Rate (lbs/hr/m)	
	Hours/ Day	Days/ Week		Duty	Standby	Desired	Effective
2008 to 2010	16	4	9,030	2	0	600 to 800	247
2020	8	5	17,700	2	1		774
2030	12	5	23,600	2	1		688
2040	14	5	31,300	2	1		783

As shown in Table 10, the current operation of both existing BFPs is at a solids loading rate that is well below the manufacturer's recommended loading rate. The City's planned refurbishment of the BFPs should help to improve dewatering efficiency and reduce the operating schedule that is currently in place. It was assumed that the BFPs would be operating at the manufacturer's recommended loading rates through 2040. At the recommended loading rates and a 5-day, 8-hour-per-day operating schedule, a second duty BFP will be required in 2020.

A 5-day dewatering schedule, from 8 to 14 hours per day, was selected to coincide with the anticipated weekday-only operations at the landfill to which the cake will be hauled. More frequent and longer operating times for the BFPs equate to a consistent loading process as shown in the table; however, longer operation times may require additional dewatered cake handling and storage facilities to account for cake production outside of normal business hours, when the landfill is not accepting deliveries. Although the above table shows solids loadings remaining consistent throughout the planning period with only modifications to the operating schedule, it should be noted that operating this equipment for more hours each operating day will require the City to engage in a significant maintenance effort in order to stay on schedule.

Table 11 shows that the existing sludge pumps will require the addition of redundant backups during the 2020 to 2030 planning period. These additions parallel the previously discussed additions of DAFT tankage and a BFP. Without the addition of a backup pump, equipment failure during the 2020 to 2030 planning period (based on these conditions) poses potential serious consequences for the treatment and conveyance of the sludge from one treatment train to the next.

Table 11
Existing Solids Treatment Train - Sludge Pumping Analysis

Type of Pump	Number of Units (Present)		Capacity Each (gpm)	Number of Duty Units Required at Max Month Conditions		
	Duty	Standby		2020	2030	2040
TWAS	1	1	120	1	2	2
BFP Feed	1	1	200	2	2	2

It should be noted that, while some of these pieces of equipment have been observed to possess adequate capacity; the projected service life of the structure, equipment, and mechanical appurtenances may need significant maintenance and/or overhauls to maintain their proper operating capacities. Life cycle cost calculations will be performed and presented in subsequent TMs to determine the future costs of equipment throughout their anticipated lifetimes.

6.1 Next Steps

During the February 2, 2011 Biosolids Workshop No. 1, a solids processing matrix was developed. This matrix defines four distinct solids treatment trains that include processes the City was interested in defining further. The matrix can be seen in **Table 12**.

Table 12
Solids Treatment Trains Developed at Biosolids Workshop No. 1

Process Train	Thickening	Stabilization	Dewatering	Drying	Biosolids Class
Option 1 (Existing)	DAF	None	BFP	None	N/A
Option 2	Drum Thickener	Anaerobic	Screw Press	Solar	A
Option 3	Screw Thickener	Anaerobic	Centrifuge	Rotary Drum/ Belt Dryer	A
Option 4	Gravity Belt Thickener	None	Centrifuge	Belt Dryer with ERS	N/A

Using this matrix, conceptual construction costs will be developed for further consideration. The results of this analysis will be presented to City staff and stakeholders at Biosolids Workshop No. 2. Participants at this upcoming workshop will select the combination of treatment technologies that will be further evaluated for selection as the City's desired solids treatment train.

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Technical Memorandum No. 2A

To: *City of Franklin IWRP Team*

From: *CDM*

Date: *April 29, 2011*

Subject: *Integrated Water Resources Plan – Biosolids Technical Analysis*
TM 2A - Evaluation of One versus Two Biosolids Facilities

Executive Summary

This Technical Memorandum (TM) was created concurrently with TM No. 2 (Evaluation of Biosolids Alternatives and Technologies). It explores the relative capital and operation and maintenance (O&M) costs associated with the following biosolids management strategies:

- Alternative A: No new wastewater treatment plant (WWTP) is constructed, and all wastewater and resultant biosolids are treated at Franklin WWTP.

- Alternative B: A new WWTP is constructed to accept flows between 2 and 6 mgd. In addition to a new solids process at Franklin WWTP, varying degrees of solids treatment occur at the new WWTP.

Variations on Alternative B (Alternatives B-1 through B-3) were studied in order to develop an understanding of the relative costs. For this analysis, CDM used the Option 3 treatment train (screw thickening, mesophilic anaerobic digestion, centrifuge dewatering, rotary drum drying) as a representative set of processes that achieves the City's goal of producing biosolids that can be beneficially reused. More detailed descriptions of the Option 3 treatment technologies are included in TM No. 2. At the time of this writing, no treatment train option has been either selected or eliminated from further consideration. The costs presented in this TM are preliminary, planning-level costs that are presented solely to illustrate the relative expense of constructing and operating the alternatives discussed herein.

Based on the economic analysis of Alternatives A and B presented in Table ES-1, CDM has concluded that the lowest capital and O&M costs are to be realized if all solids are generated and treated at Franklin WWTP.

Table ES-1
Summary of Preliminary Economic Analysis for Alternatives A and B

Alternative	Capital Cost	Annual O&M Cost (2040)	Present Worth	O&M Cost Per DT
Alternative A: All Wastewater and Solids Treatment at Franklin WWTP	$50.0MM	$1.03MM	$65.8MM	$309
Alternative B-1: Dewatering at New WWTP, Drying at Franklin WWTP	$66.6MM	$1.36MM	$87.6MM	$408
Alternative B-2: Thickening at New WWTP, Transfer to Franklin WWTP	$54.6MM	$1.15MM	$72.4MM	$346
Alternative B-3: Pump WAS to Franklin WWTP	$63.5MM	$1.08MM	$80.2MM	$324
Current O&M Cost	N/A	N/A	N/A	$547

1.0 Introduction

In support of the City of Franklin's Integrated Water Resources Plan (IWRP), CDM has been evaluating the solids handling facilities at the City's wastewater treatment plant (WWTP). CDM has conducted site visits to the WWTP to assess the condition of the existing equipment, as well as reviewing the historical monthly operating reports (MORs), a performing preliminary analysis of the current operation and maintenance (O&M) requirements and costs of solids treatment, and developing an overview of the solids projections based on current conditions and the availability of existing equipment to meet future needs of the WWTP. The results of these activities were presented in Technical Memorandum (TM) No. 1, "Evaluation of Existing Equipment and Sludge Production Forecast."

In TM No. 1, CDM concluded that the existing solids handling facilities were near the end of their useful life; and that if the existing facilities were not replaced and expanded, the anticipated future increases in solids production would exceed the plant's solids treatment capacity between 2020 and 2030. Upgrades to Franklin WWTP's solids handling facilities are needed, regardless of whether a second WWTP is constructed. CDM and the City expect that some kind of solids handling will always be needed at Franklin WWTP, unless the City elects to consolidate solids treatment at another location, such as a second WWTP.

In preparing TM No. 2 ("Evaluation of Biosolids Alternatives and Technologies"), CDM developed preliminary capital, O&M, and present worth costs of the four solids treatment trains discussed during Biosolids Workshop No. 1. Concurrent with this economic analysis, CDM analyzed the

relative capital construction and O&M costs to operate one biosolids treatment facility at Franklin WWTP or two biosolids facilities—one at Franklin WWTP and one at a new WWTP. The purpose of TM 2A is to present and discuss the results of capital construction and O&M costs analysis.

CDM is developing planning level capital costs of the four biosolids treatment train options that were developed at Biosolids Workshop No. 1, held on February 2, 2011. One of the objectives of this part of CDM's work is to evaluate the feasibility and cost of the following biosolids management strategies:

- Alternative A: No new WWTP is constructed, and all wastewater and resultant biosolids are treated at Franklin WWTP.

- Alternative B: A new WWTP is constructed to accept flows between 2 and 6 mgd. In addition to a new solids process at Franklin WWTP, varying degrees of solids treatment occur at the new WWTP.

2.0 Discussion of Alternatives

Variations on Alternative B were studied in order to develop an understanding of the relative costs. For this analysis, CDM used the Option 3 treatment train as a representative example of a complete treatment process that includes all four solids processing steps—thickening, stabilization, dewatering, and drying—and achieves the City's goal of producing biosolids that can be beneficially reused. More detailed descriptions of the Option 3 treatment technologies (screw thickening, anaerobic digestion, centrifuge dewatering, and rotary drum drying) are included in TM No. 2. At the time of this writing, no treatment train option has been either selected or eliminated from further consideration.

The methodologies and assumptions presented in Section 4 of TM No. 2 apply to the development of the costs presented in this memorandum. These costs, summarized in **Table 2-1,** are preliminary, planning-level costs and are presented solely to illustrate the relative expense of constructing and operating the alternatives discussed herein.

2.1 Alternative A: No New WWTP; All Wastewater and Solids Treated at Franklin WWTP

In this alternative, a new WWTP is not constructed, and all wastewater flows to Franklin WWTP for treatment. All waste activated sludge (WAS) produced by the wastewater treatment process is generated and treated on site prior to disposal. A solids handling building would combine all of the new solids handling technologies—three screw thickeners, support equipment for two mesophilic anaerobic digesters, three dewatering centrifuges, and one rotary drum dryer—under one roof.

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Table 2-1
Summary of Alternatives

Process	Alternative A: All Solids Treatment at Franklin WWTP			Alternative B-1: Dewatering at New WWTP, Drying at Franklin WWTP						Alternative B-2: Thickening at New WWTP, TWAS to Franklin WWTP						Alternative B-3: WAS Force Main to Franklin WWTP					
				Franklin WWTP			New WWTP			Franklin WWTP			New WWTP			Franklin WWTP			New WWTP		
	No. of Units in 2040[1]	Unit Capacity[2]	Op. Schedule (hrs/week)[3]	No. of Units in 2040[1]	Unit Capacity[2]	Op. Schedule (hrs/week)[3]	No. of Units in 2040[1]	Unit Capacity[2]	Op. Schedule (hrs/week)[3]	No. of Units in 2040[1]	Unit Capacity[2]	Op. Schedule (hrs/week)[3]	No. of Units in 2040[1]	Unit Capacity[2]	Op. Schedule (hrs/week)[3]	No. of Units in 2040[1]	Unit Capacity[2]	Op. Schedule (hrs/week)[3]	No. of Units in 2040[1]	Unit Capacity[2]	Op. Schedule (hrs/week)[3]
WAS Storage	1	1.2 MG	n/a	1	0.9 MG	n/a	1	0.3 MG	n/a	1	0.85 MG	n/a	1	0.3 MG	n/a	1	1.2 MG	n/a	1	0.3 MG	n/a
Thickening (Screw Thickener)	3	1,100 lbs/hour	75	2	1,100 lbs/hour	109	2	1,100 lbs/hour	40	3	1,100 lbs/hour	55	2	1,100 lbs/hour	40	3	1,100 lbs/hour	75	n/a	n/a	n/a
TWAS Storage	1	160,000 gal	n/a	1	120,000 gal	n/a	1	50,000 gal	n/a	1	120,000 gal	n/a	1	50,000 gal	n/a	1	160,000 gal	n/a	n/a	n/a	n/a
Stabilization (Anaerobic Digestion)	2	0.6 MG	n/a	2	0.4 MG	n/a	1	0.4 MG	n/a	2	0.6 MG	n/a	n/a	n/a	n/a	2	0.6 MG	n/a	n/a	n/a	n/a
Dewatering (Centrifuge)	3	1,280 lbs/hour	43	2	1,280 lbs/hour	62	2	1,280 lbs/hour	23	3	1,280 lbs/hour	43	n/a	n/a	n/a	3	1,280 lbs/hour	43	n/a	n/a	n/a
Drying (Rotary Drum Dryer)	1	825 lbs/hour	132	1	825 lbs/hour	132	0	825 lbs/hour	n/a	1	825 lbs/hour	132	n/a	n/a	n/a	1	825 lbs/hour	132	n/a	n/a	n/a
Planning Level Construction Cost	**$49,976,000**			**$66,616,000**						**$54,639,000**						**$63,546,000**					
Annual O&M Cost	**$1,027,000**			**$1,356,000**						**$1,149,000**						**$1,077,000**					
Present Worth	**$65,800,000**			**$87,600,000**						**$72,400,000**						**$80,200,000**					
O&M Cost per DT	**$309**			**$408**						**$346**						**$324**					

[1] Includes standby unit where applicable.
[2] Capacities expressed as pounds per hour or pounds per day are dry solids
[3] In the maximum month.
[4] O&M costs are presented for the design year of 2040

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The planning-level Opinions of Probable Construction costs (OPCC) for Alternative A is $50.0 million**.** The annual O&M cost in 2040 is estimated to be $1.03 million**,** or $309 per dry ton (DT) of solids treated**.** The 30-year present worth of Alternative A is $65.8 million**.**

2.2 Alternative B: Construct New WWTP and Treat Solids at Both WWTPs

The possibility of constructing a second WWTP is currently being investigated. For the purposes of this analysis, the capacity of this new WWTP was assumed to be 2 mgd at startup, with an ultimate capacity of 6 mgd in 2040.

One approach to solids treatment would be to construct a completely separate, independent solids treatment train at the new WWTP. A more practical approach, however, would be to transfer solids from the new WWTP to Franklin WWTP for treatment. Many solids treatment technologies benefit from continuous running, and consolidating solids treatment at one plant lengthens process run times, while minimizing the amount of redundant equipment, O&M, and staffing requirements. Furthermore, the proximity of this potential new WWTP to Franklin WWTP makes transfers of sludge feasible. Three alternatives, each involving different degrees of solids treatment at the new WWTP, are discussed below.

2.2.1 Alternative B-1: Dewatering at New WWTP, Drying at Franklin WWTP

In this alternative, a solids handling building at Franklin WWTP would house two screw thickeners, support equipment for two anaerobic digesters, two dewatering centrifuges, and a rotary drum dryer. A smaller solids handling building at the new WWTP would contain two screw thickeners, support equipment for one anaerobic digester, and two dewatering centrifuges. Because even the smallest drum dryer would be oversized for a 2 to 6 mgd WWTP, CDM did not include a dryer at the new WWTP. Instead, it was assumed that the dewatered biosolids from the new WWTP would be hauled via truck to Franklin WWTP for drying and subsequent disposal.

All of this redundant equipment and facilities results in a planning level OPCC of $66.6 million for Alternative B-1. The annual O&M cost in 2040 is estimated to be $1.36 million, or $408/DT of solids treated, due to the additional staffing and maintenance needed for the new WWTP**.** The 30-year present worth of Alternative A is $87.6 million**.**

2.2.2 Alternative B-2: Partial Solids Treatment at New WWTP; Transfer of TWAS to Franklin WWTP

This alternative assumes that WAS from the new WWTP is transported to Franklin WWTP via tanker truck. The WAS could be hauled, untreated at 0.5-percent solids, or be thickened by mechanical means to approximately 5-percent solids prior to transport. As **Table 2-2** demonstrates, because thickened WAS (TWAS) occupies 1/10 the volume of unthickened WAS, it is more economical to transport than unthickened WAS.

Table 2-2
Comparison of Costs for Hauling WAS and TWAS from a New WWTP

Waste-Water Flow (mgd)	Unthickened WAS				Thickened WAS			
	Solids (%)	Volume Produced (gal/day)	Truckloads per Year[1]	Annual Hauling Cost[2]	Solids (%)	Volume Produced (gal/day)[3]	Truckloads per Year[1]	Annual Hauling Cost[2]
2	0.5	39,400	2,400	$278,000	5.0	3,800	240	$33,000
6		118,000	7,180	$818,000		11,300	690	$84,000

[1] Each tanker truckload is 6,000 gallons.
[2] Includes insurance, maintenance, fuel, and labor costs.
[3] Assumes 90-percent solids capture in the thickeners.

For Alternative B-2, CDM assumed that WAS produced at the new WWTP would be thickened from 0.5- to 5-percent solids using a screw thickener. TWAS would be pumped into a tanker truck and hauled to Franklin WWTP for further treatment and ultimate disposal. The hauling distance used for this analysis was assumed to be 6.8 miles, which is approximately equal to the length of a theoretical sludge transfer pipeline discussed in Alternative B-3, Section 2.2.3.

When compared to Alternative A, it is evident that Alternative B-2 will have a higher construction cost due to the need to construct separate thickening facilities at each WWTP. Alternative B-2 requires the procurement of two more screw thickeners, construction of a thickening building at the new WWTP, and WAS and TWAS storage tanks with mixing systems. The City does not currently own any sludge tanker trucks, so the City would have to purchase at least two tanker trucks. A tanker truck unloading facility would also have to be constructed at the Franklin WWTP to receive the thickened sludge.

The planning level OPCC for Alternative B-2 is $54.6 million. The annual O&M cost in 2040 is estimated to be $1.15 million, or $346/DT of solids treated. The 30-year present worth of Alternative A is $72.4 million.

2.2.3 Alternative B-3: WAS Force Main from New WWTP to Franklin WWTP

Pumping of WAS from the new WWTP to Franklin WWTP was examined in Smith Seckman & Reid, Inc.'s (SSR) January 2011 technical memorandum. A 36,000-foot-long force main would be required to transmit raw WAS at 0.5-percent solids to Franklin WWTP. The conceptual cost estimate presented in SSR's memo is $2.7 million to $3 million, which does not include acquisition of rights-of-way. The use of a force main between the plants eliminates the expense of hauling liquid sludges via tanker truck, and it also eliminates the need to construct solids handling facilities at the new WWTP.

In developing preliminary costs for this alternative, CDM assumed that a 6-inch, 36,000-foot-long force main would be constructed from the new WWTP to Franklin WWTP, along with WAS storage and pumping facilities at the new WWTP. The planning level OPCC for Alternative B-3 is

$63.5 million. The annual O&M cost in 2040 is estimated to be $1.08 million, or $324/DT of solids treated. The 30-year present worth of Alternative B-3 is $80.2 million. These costs are likely to increase after the cost of right-of-way acquisition is calculated.

3.0 Conclusions and Recommendations

Alternative A, where all solids are generated and treated at Franklin WWTP, carries the lowest construction and O&M costs of the four alternatives presented in this memorandum. Alternative B-2, in which two separate solids handling treatment trains are constructed, is the most expensive. CDM recommends that, in order to minimize capital and O&M costs, the focus of the IWRP should be on consolidation of solids handling facilities at Franklin WWTP.

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Technical Memorandum No. 2

To: *City of Franklin IWRP Team*

From: *CDM*

Date: *October 12, 2011*

Subject: *Integrated Water Resources Plan – Biosolids Technical Analysis*
TM No. 2 – Evaluation of Biosolids Alternatives and Technologies

Executive Summary

Building upon the findings in Technical Memorandum (TM) No. 1 and the decisions made at Biosolids Workshop No. 1, this TM discusses in further detail the solids treatment technologies and the four potential process trains developed at the Workshop:

Process Train	Thickening	Stabilization	Dewatering	Drying	Biosolids Class
Option 1 (Existing)	DAF	None	Belt Filter Press	None	N/A
Option 2	Drum Thickener	Anaerobic	Screw Press	Solar	A
Option 3	Screw Thickener	Anaerobic	Centrifuge	Rotary Drum/ Belt Dryer	A
Option 4	Gravity Belt Thickener	None	Centrifuge	Belt Dryer with ERS	N/A

In evaluating these process trains, CDM performed conceptual sizing of equipment and facilities to meet treatment needs in 2040 at the existing WWTP, developed planning level capital and O&M costs and life cycle costs, and compared these costs to those of the WWTP's existing solids treatment process. The estimated costs of these four trains are as follows:

Process Train	Capital Cost	Annual O&M Cost (2040)	Present Worth	O&M Cost Per DT
Option 1	$23 M	$2.2 M	$42 M	$361
Option 2	$81 M	$1.0 M	$93 M	$166
Option 3	$67 M	$1.5 M	$81 M	$239
Option 4	$77 M	$1.9 M	$94 M	$306
Current O&M Cost	N/A	N/A	N/A	$547

1.1 Introduction

As part of its Integrated Water Resources Plan (IWRP), the City of Franklin is developing a plan to improve the solids handling facilities at its wastewater treatment plant (WWTP). At Biosolids Workshop No. 1, held on February 2, 2011, participants determined the preliminary alternatives (individual treatment technologies and four potential solids process trains) that would be further analyzed during the alternatives selection process.

Technical Memorandum (TM) No. 1, submitted in May, included an assessment of the condition of the WWTP's existing solids handling facilities; a review of the plant's historical operating data; a preliminary analysis of the current operation and maintenance (O&M) requirements and costs to treat the sludge; and an overview of the solids projections based on current conditions and the capacity of the existing equipment to meet the needs of the WWTP in the design year of 2040.

CDM used the data compiled in TM No. 1 and the results of Workshop No. 1 to develop this TM, which discusses the selected solids treatment technologies and the four potential process trains in further detail. These process trains have been evaluated to identify the conceptual sizing of equipment and facilities, as well as to develop planning level capital and O&M costs and life cycle costs. The planning level costs for each process train were compared to the O&M costs for the existing solids treatment process which were developed in TM No. 1.

2.1 Updates to Solids Production Forecast

Following submittal of TM No. 1, CDM revised the Franklin WWTP's solids production forecast. The revision was needed because CDM's WWTP process modeling work showed that the anticipated solids production would be higher than the current solids production cited in TM No. 1. **Table 1** lists the new design parameters and the recalculated solids loadings.

Table 1
Updated Solids Design Criteria and Production Forecast

Parameter		Value
Yield (lbs WAS/lb BOD_5 removed)		0.8
BOD_5 Loading	Influent	212 mg/L (1,768 lbs/MG)
	Effluent	5 mg/L (42 lbs/MG)
BOD_5 Removed		1,726 lbs/MG
Solids Production (lbs WAS/MG wastewater)[1]	Average Day	1,381 lbs/MG
	Max Month	1,795 lbs/MG

[1] Includes chemical sludge.

3.1 Evaluation Criteria for Technologies and Process Trains

During Biosolids Workshop No. 1, evaluation criteria were established for developing effective solids management strategies that will meet the long-term needs of the City. Workshop participants identified the following priorities for solids management strategies:

- Efficiency of operations: Equipment and operating trains that provide efficient solids processing with little effect upon other treatment processes, consume less energy, and require less maintenance.
- Decreased energy consumption: Processes that require reduced quantities of fossil fuel-derived energy, employ high-efficiency equipment, beneficially reuse waste energy, or sequester carbon dioxide tend to have lower carbon footprints.
- Sustainability: Processes that will sustain themselves through various disposal options including lower energy consumption, efficient operations, and high quality solids.
- Diverse portfolio of product use/disposal options: Class B biosolids use is limited to agricultural land application, but the regulations place fewer restrictions on the use of Class A biosolids. Class A biosolids have a larger portfolio of options and may be land applied or used in home gardens and lawns.
- Reliability: Redundant equipment that will allow for continuous solids processing operations while other equipment is taken out of service.
- Risk reduction: Single use/disposal options, like private landfills, determine what type and how much solids they can accept from a municipality. Private landfills can also eliminate biosolids disposal at a moment's notice, leaving the municipality without disposal options. Risk reduction would include the potential to provide more than one end use/disposal alternative.
- Environmental/public acceptance: Public buy in of solids processing effects and the resulting minimal impact to the environment are important to the community's achievement of sustainable goals.
- Odor control: Because the Franklin WWTP is located near several residential neighborhoods and a school, processes with a lower potential to generate odors are preferred.
- Automated processes: A new process that is automated will require less training of staff and less of a learning curve immediately after it is implemented.
- Class A biosolids: A quality product with a variety of use/disposal options.

- Expandability strategy for growth: A solids train upgrade with a compact layout leaves more space available for future expansion of the facility, thus reducing or eliminating the need for building expansion or additional land acquisition. Also favorable are processes whose solids treatment capacity can be expanded simply, such as by installing additional pieces of equipment.

As part of the evaluation process that will be conducted during an upcoming Steering Committee meeting, a decision matrix containing these evaluation criteria will be developed that will rate each solids process for its ability to meet the City's needs.

4.1 Process Alternatives

Table 2 shows the solids processing matrix that was developed at Biosolids Workshop No. 1. This matrix defines the four distinct solids treatment trains that consist of the treatment technologies the City wishes to investigate further.

Table 2
Summary of Unit Processes and System Alternatives

Process Train	Thickening	Stabilization	Dewatering	Drying	Biosolids Class
Option 1 (Existing)	DAF	None	Belt Filter Press	None	N/A
Option 2	Drum Thickener	Anaerobic	Screw Press	Solar	A
Option 3	Screw Thickener	Anaerobic	Centrifuge	Rotary Drum/ Belt Dryer	A
Option 4	Gravity Belt Thickener	None	Centrifuge	Belt Dryer with ERS	N/A

These four process trains were evaluated for this TM. Budgetary cost proposals and sizing information for major equipment were obtained from vendors. Planning level capital, O&M, and life cycle costs for each process train were developed. The evaluation was structured so that the City could have the option of mixing and matching individual unit process technologies as additional information is obtained. The relative capital and O&M costs of a single solids treatment facility (at Franklin WWTP) versus two facilities (at Franklin WWTP and a potential new WWTP) were discussed separately in TM No. 2A, which was issued in May, and is further discussed in this TM.

In addition to developing planning level capital, O&M, and life cycle costs to suit the Franklin WWTP at up to a 24 mgd average daily flow (ADF), CDM developed the same costs for the Franklin WWTP at up to 16 mgd ADF, plus a new WWTP at 4 and 8 mgd ADF. It was assumed that the new WWTP would include facilities for thickening the WAS, and the thickened WAS would

then be transported via tanker truck to the Franklin WWTP for further treatment. As explained in TM No. 2A, this strategy would allow the City to consolidate most of the treatment at the Franklin WWTP, while keeping the amount of redundant equipment, O&M, and staffing requirements to a minimum. These single- and two-plant relative costs are explored further in Section 5.

The following sections discuss the benefits, requirements, and relative costs of the thickening, stabilization, dewatering, and drying technologies that were identified for further evaluation.

4.1.1 Thickening Alternatives

Thickening is often used to remove water and increase the solids content of the waste activated sludge (WAS) prior to stabilization. Thickening improves the efficiency and minimizes the size of downstream treatment processes. The optimum thickened solids concentration varies depending on the actual process chosen. At the Franklin WWTP, thickening is currently performed by dissolved air flotation thickening (DAFT). The four thickening technologies examined in this TM are DAFT, rotary drum thickening, screw thickening, and gravity belt thickening.

4.1.1.1 Thickening Design Criteria and Assumptions

Similar to existing thickening operations, it was assumed that the thickening equipment will be operated 7 days per week, with sludge storage provided after the thickening process. The thickening design parameters shown in **Table 3** were applied to all of the thickening technologies examined.

Table 3
Thickening Design Parameters

Parameter	Initial Loadings (9.4 mgd ADF)		Design Loadings (24 mgd ADF)	
	Avg. Day	Max. Month	Avg. Day	Max. Month
Hydraulic Loading (gal/day)	130,000	169,000	331,000	431,000
Solids Loading (lbs/day)	13,000	16,900	33,100	43,100
Feed Sludge Solids Content (%)	1.2			
Thickened Sludge Solids Content (%)	5.0			
Thickening Polymer Consumption (lbs/DT)	7.5			
Solids Capture (%)	95			

[1] This is currently the operational pattern of the WWTP for solids processing.

If thickening occurs on a shorter schedule, WAS storage would need to be provided before the thickening equipment.

4.1.1.2 Dissolved Air Flotation (Option 1)

System Description

The City of Franklin currently utilizes DAFTs for its thickening process. WAS from the clarifiers flows into the DAFTs, where it is mixed with a water mixture. Air is introduced into the water-saturated solid mixture that is being held at an elevated pressure. When the mixture is depressurized, the dissolved air is released as finely divided bubbles that carry the sludge to the top, where it is removed by skimmers.

The DAFT tanks at Franklin WWTP are circular tanks that are similar in appearance to clarifiers. DAFTs require minimal energy and can typically produce solids in the range of 3 to 5 percent. Odor can be associated with DAFTs, and some plants have enclosed DAFTs to combat odors. However, the City has operated its outdoor DAFTs with minimal odors. The existing DAFT tanks at Franklin WWTP are shown in **Figure 1.**

Figure 1
DAFT at Franklin WWTP

Conceptual System Design

A DAFT facility for the current plant at existing plant flows would consist of two 30-foot-diameter tanks. For thickening of the WAS produced at design conditions, five tanks would be needed—four duty units and one standby unit—so an upgraded DAFT footprint should include space to accommodate this future growth. In a two-plant configuration, four DAFT units (three duty, one standby) would be required at Franklin WWTP at design conditions, and three DAFT units (two duty, one standby) would be required at the new WWTP, for a total of seven units. Additional WAS pumps may not be needed at design flows, but the distribution piping at Franklin WWTP would require modification to provide adequate solids loading to each of the DAFT tanks with appropriate sludge quantities. Franklin WWTP achieves a 3 to 5 percent thickened solids concentration without the aid of a polymer. The addition of polymer could improve performance, but it would also increase annual operation costs of the WWTP.

The planning level costs to construct the Option 1 DAFT system equipment and structure are approximately $15.3 million (one plant) or $26.4 million (two plants). The average power consumption would be approximately 230 kilowatt-hours (kWh) per dry ton (DT).

4.1.1.3 Rotary Drum Thickener (Option 2)

System Description

A rotary drum thickening system consists of a conditioning system (including a polymer feed system) and rotating drums that are covered with filter fabric. Polymer is mixed with dilute sludge in the flocculator, and the conditioned sludge is fed into rotating-screen drums that separate the flocculated solids from the water. Thickened sludge rolls out the end of the drums, while separated water decants through the screens. A schematic of a rotary drum thickener is shown in **Figure 2.**

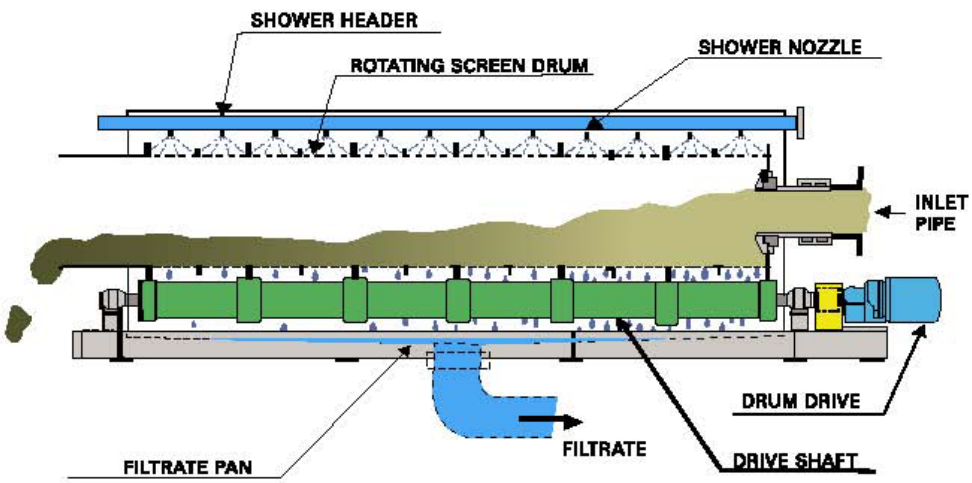


Figure 2
Schematic of a Drum Thickener
(from http://www.fkcscrewpress.com/crst.html)

The thickener's horizontal position may be inclined so that the discharge side is higher. The major advantage of drum thickeners, compared with conventional belt thickeners, is the drum's rotary motion that increases the gravitational pressure on the screening surface and forms rolling flocs, significantly improving water separation and filtration.

Cleaning of the drum surface is carried out by pressurized water jets located inside the drum thickener housing. Washing is controlled by a timer. The drum thickener has a compact design, is easy to enclose (**Figure 3**), and can nearly eliminate odors and splashing water. Drum thickeners are available with varying capacities, ranging from 5 to 400 gpm. O&M costs are low due to minimal maintenance and low power consumption.

Conceptual System Design

A conceptual rotary drum thickening facility for Franklin WWTP would include two thickeners at current conditions, with one standby unit. Three duty units and one standby would be required for the maximum month solids production in 2040, so the thickener facility should be designed for addition of a fourth unit. In a two-plant configuration, Franklin WWTP would require three units (two duty, one standby), and the new WWTP would require three units (two duty, one standby), for a total of six installed units. Each unit would be sized for a throughput of 700 pounds per hour. Feed pumps, one per thickener, would be provided to feed thin sludge to the process. Additionally, a polymer feed system would consist of a packaged liquid polymer batching system, a mixing/aging tank for the diluted polymer solution, and positive-displacement polymer feed pumps for injection of the polymer solution into the sludge feed pipes. One polymer feed system would be installed for each thickener. Because these thickeners are closed systems, odor control ductwork could easily be connected to each unit, or they could be housed inside a biosolids building connected to odor control ductwork.

Figure 3
Drum Thickener with Enclosure at
Gilder Creek WWTP, Greenville, South Carolina

The planning level costs to construct the Option 2 rotary drum thickener equipment and structures are approximately $8.3 million (one plant) or $17.3 million (two plants). The average power draw of a rotary drum thickener and its support equipment would be approximately 80 kWh/DT.

4.1.1.4 Rotary Screw Thickener (Option 3)

System Description

The rotary screw thickener consists of a conditioning system (including a polymer feed system) and a rotating thickener similar to a rotary drum thickener. A schematic of a rotary screw thickener is shown in **Figure 4.**

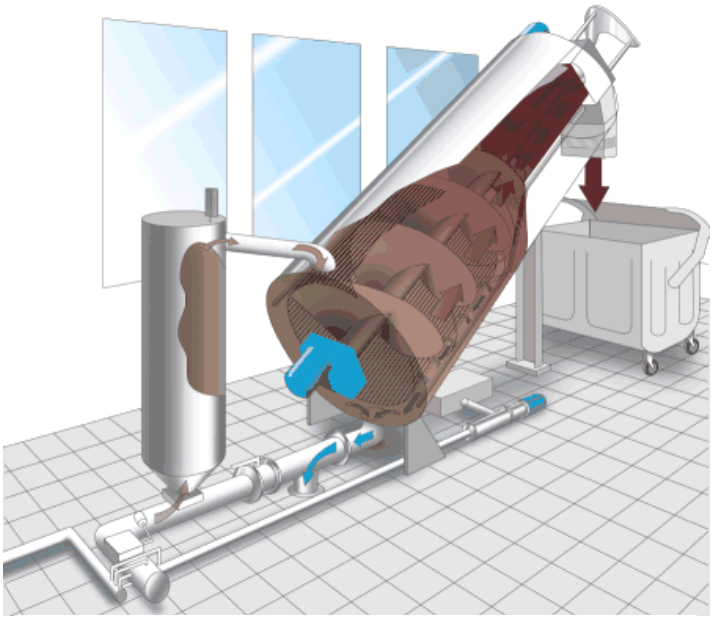


Figure 4
Schematic of a Rotary Screw Thickener
(from http://www.huber-technology.com/hp59/index.htm)

Feed sludge is pumped to a flocculation reactor, and a dilute polymer solution is introduced through a dosing ring into the feed sludge. The injected polymer is intensively mixed with the sludge in a static inline mixer. Strong sludge flocs are formed in the agitated flocculation reactor where the flocculated sludge overflows into the screw thickener. The screw thickener is comprised of an inclined wedge section basket with a narrow spacing. A screw, slowly rotating with variable speed, conveys the sludge gently upward through the inclined basket, allowing water to drain by gravity through the basket.

The screw flights generally have a brush for continuous internal cleaning of the wedge section basket. Periodically, the wedge section basket is also cleaned with spray water from the outside. Spray bars rotate around the basket but within the machine. The screw pushes the thickened sludge to the upper end of the wedge section basket, where it drops through a chute into a storage tank or the throat of a thickened sludge pump that forwards it to further treatment.

Advantages of using rotary screw thickeners include a compact footprint and low weight with a complete enclosure which prevents emission of odor, vapors, and spray water. Operation and maintenance requirements are relatively low, and the process can be fully automated enabling simplification of continuous operations, if desired. Screw thickening can achieve up to a 4- to 8-percent solids concentration in the thickened sludge.

Conceptual System Design

A conceptual screw thickening facility design for Franklin WWTP would include three thickeners—two duty and one standby—each sized for a throughput of approximately 1,100 lbs/hr. In a two-plant configuration, three units (two duty, one standby) would be installed at

Franklin WWTP, and three units (two duty, one standby) would be installed at the new WWTP, for a total of six installed thickeners. One flocculation tank serves two duty thickeners. Positive displacement pumps would be provided to feed thin sludge to the process. The polymer feed system would consist of two packaged dry- or liquid-polymer batching systems for redundancy, a mixing/aging tank for the diluted polymer solution, and positive-displacement polymer feed pumps for injection of the polymer solution into the sludge feed pipes.

The planning level costs to construct the Option 3 screw thickening system equipment and structures are $7.5 million (one plant) or $17.5 million (two plants). The average power draw of a screw thickening system would be approximately 50 kWh/DT.

4.1.1.5 Gravity Belt Thickener (Option 4)

System Description

A gravity belt thickener (GBT) employs gravity drainage through a filter bed to thicken polymer-conditioned sludge. Dilute sludge, typically 0.5- to 1.0-percent solids, is introduced at the feed end of a wide, horizontal filter belt constructed of porous fabric. As the slurry makes its way down the moving belt, free water drains through the porous belt. Plows break up the sludge and form furrows to enhance drainage of more water. Equipment manufacturers employ different variations on this process to hold back the solids while producing movement throughout the sludge blanket to allow drainage of free water. Sludge is discharged at the end of the horizontal filter belt as a thickened sludge that can be pumped. An example of a GBT installation is shown in **Figure 5**.

Figure 5
Gravity Belt Thickener at Charles River Pollution Control District WWTF, Franklin & Medway, Massachusetts

Systems can be designed for up to 5- to 7-percent thickened solids. Solids capture can range between 90 and 98 percent depending upon process operations.

Conceptual System Design

A conceptual GBT process design for Franklin WWTP would include three 2-meter units (two duty and one standby), sized for a throughput of approximately 1,100 pounds per hour. In the two-plant configuration, Franklin WWTP would require three 2-meter units (two duty, one standby), and the new WWTP would require three 2-meter units (two duty, one standby) for a total of six two-meter GBTs.

The planning level costs to construct the Option 4 GBT system equipment and structures are $7.8 million (one plant) or $18.3 million (two plants). The estimated average power draw of a GBT system would be approximately 60 kWh/DT.

4.1.1.6 Summary of Thickening Alternatives

A summary of the unit process requirements for these thickening alternatives is provided in **Table 4**. This table provides conceptual design criteria for the one- and two-plant configurations based on the forecasted solids production discussed in Section 2 and provides planning level estimates of equipment costs along with ancillary requirements, estimated annual operations, and maintenance costs (including labor, energy, and chemicals) and a list of installation locations.

4.2 Stabilization Alternatives

Stabilization processes are designed to reduce pathogens, eliminate odors, and/or reduce the volume of sludge to be disposed. A stabilization process can be a beneficial and cost-saving precursor to other downstream processes. For example, stabilization reduces the amount of solids fed to a dewatering process and, in turn, reduces the size, capital cost, power and polymer requirements, and operating hours of the dewatering equipment. Dewatering of stabilized biosolids can result in a drier cake. If a drying process is part of the treatment train, stabilization can help to keep the size and capital cost of the dryer to a minimum.

On the other hand, there are certain downstream processes that do not benefit from stabilization. Processes designed to derive useful energy from the biosolids, including the energy recovery system proposed for the Option 4 belt dryer, tend to be optimized for biosolids that have not been stabilized and thus have a higher energy content.

Table 4

Comparison of Thickening Technologies - Existing WWTP at 24 MGD

Parameter		Option 1	Option 2	Option 3	Option 4
Technology		Dissolved Air Flotation (DAF)	Rotary Drum Thickener (RDT)	Screw Thickener	Gravity Belt Thickener (GBT)
Conceptual Model Selection		EIMCO/Ovivo 30' diameter or equal	Andritz 12x3 RST or equal	Huber Technology RoS2 4L or equal	Komline-Sanderson or equal
Solids Loading Rate (lbs/day)[1]	Initial	13,000	13,000	13,000	13,000
	Design	33,000	33,000	33,000	33,000
Unit Capacity (lbs/hr)		495	705	1,100	1,100
Feed Solids		1.2%	1.2%	1.2%	1.2%
Discharge Solids		5.0%	5.0%	5.0%	5.0%
Polymer Consumption (lbs/DT)		7.5	7.5	7.5	7.5
Number of Units	Initial	2	2	1	1
	Design	4	3	2	2
	Standby	1	1	1	1
Proposed Operating Schedule (days/hours)	Initial	7 / 13	7 / 9	7 / 12	7 / 12
	Design	7 / 17	7 / 16	7 / 15	7 / 15
Ancillaries Required					
Polymer Feed Tank		X	X	X	X
Flocculation Tank		X	X	X	X
Wash Water			X	X	X
Odor Control			X	X	X
Upstream Grinder			X	X	X
Feed Sludge Mixer		X	X	X	X
Estimated Capital Cost					
Process Equipment		$1,625,000	$730,000	$562,500	$600,000
Equipment with Ancillaries		$2,375,000	$1,350,000	$1,127,500	$1,183,000
Estimated Annual O&M Cost (Initial ADF)		$151,906	$106,127	$117,706	$123,191
Planning Level Cost[2]		$15,322,000	$8,318,000	$7,457,000	$7,824,000
Approx. Power Consumption (kWh/DT)		228	83	47	61
Installed Locations/References		City of Franklin WWTP, Franklin, TN	Arbennie Pritchett WRF, Okaloosa County, FL	Winter Haven WWTP, Winter Haven, FL	Fourth Creek WWTP, Statesville, NC

[1] Average day basis.

[2] Planning level cost includes process equipment and appurtenances, structures and demolition (where applicable), labor, construction equipment and material, sitework, piping, permits, tax, bonds, design and construction services, and design and construction contingencies. See Table 18 for additional details.

Table 5

Comparison of Thickening Technologies - Existing WWTP at 16 MGD

Parameter		Option 1	Option 2	Option 3	Option 4
Technology		Dissolved Air Flotation (DAF)	Rotary Drum Thickener (RDT)	Screw Thickener	Gravity Belt Thickener (GBT)
Conceptual Model Selection		EIMCO/Ovivo 30' diameter or equal	Andritz 12x3 RST or equal	Huber Technology RoS2 4L or equal	Komline-Sanderson or equal
Solids Loading Rate (lbs/day)[1]	Initial	130,000	130,000	130,000	130,000
	Design	221,000	221,000	221,000	221,000
Unit Capacity (lbs/hr)		495	705	1,100	1,100
Feed Solids		1.2%	1.2%	1.2%	1.2%
Discharge Solids		5.0%	5.0%	5.0%	5.0%
Polymer Consumption (lbs/DT)		7.5	7.5	7.5	7.5
Number of Units	Initial	2	2	1	1
	Design	3	2	2	2
	Standby	1	1	1	1
Proposed Operating Schedule (days/hours)	Initial	7 / 13.1	7 / 9.2	7 / 11.8	7 / 11.8
	Design	7 / 15	7 / 15.7	7 / 10	7 / 10
Ancillaries Required					
Polymer Feed Tank		X	X	X	X
Flocculation Tank		X	X	X	X
Wash Water			X	X	X
Odor Control			X	X	X
Upstream Grinder			X	X	X
Feed Sludge Mixer		X	X	X	X
Estimated Capital Cost					
Process Equipment		$1,300,000	$547,500	$562,500	$600,000
Equipment with Ancillaries		$1,850,000	$952,500	$967,500	$1,023,000
Estimated Annual O&M Cost (Initial ADF)		$151,906	$106,127	$117,706	$123,191
Planning Level Cost[2]		$11,570,000	$5,244,000	$5,302,000	$5,669,000
Approx. Power Consumption (kWh/DT)		228	83	47	61
Installed Locations/References		City of Franklin WWTP, Franklin, TN	Arbennie Pritchett WRF, Okaloosa County, FL	Winter Haven WWTP, Winter Haven, FL	Fourth Creek WWTP, Statesville, NC

[1] Average day basis.

[2] Planning level cost includes process equipment and appurtenances, structures and demolition (where applicable), labor, construction equipment and material, sitework, piping, permits, tax, bonds, design and construction services, and design and construction contingencies. See Table 19 for additional details.

Table 6

Comparison of Thickening Technologies - New WWTP at 8 MGD Capacity

Parameter		Option 1	Option 2	Option 3	Option 4
Technology		Dissolved Air Flotation (DAF)	Rotary Drum Thickener (RDT)	Screw Thickener	Gravity Belt Thickener (GBT)
Conceptual Model Selection		EIMCO/Ovivo 30' diameter or equal	Andritz 12x3 RST or equal	Huber Technology RoS2 4L or equal	Komline-Sanderson or equal
Solids Loading Rate (lbs/day)[1]	Initial	6,000	6,000	6,000	6,000
	Design	11,000	11,000	11,000	11,000
Unit Capacity (lbs/hr)		495	705	450	1,100
Feed Solids		1.2%	1.2%	1.2%	1.2%
Discharge Solids		5.0%	5.0%	5.0%	5.0%
Polymer Consumption (lbs/DT)		7.5	7.5	7.5	7.5
Number of Units	Initial	1	1	1	1
	Design	2	2	2	2
	Standby	1	1	1	1
Proposed Operating Schedule (days/hours)	Initial	5 / 15.6	5 / 11	3 / 11.7	4 / 8.8
	Design	5 / 15.6	5 / 11	3 / 11.7	4 / 8.8
Ancillaries Required					
Polymer Feed Tank		X	X	X	X
Flocculation Tank		X	X	X	X
Wash Water			X	X	X
Odor Control			X	X	X
Upstream Grinder			X	X	X
Feed Sludge Mixer		X	X	X	X
Estimated Capital Cost					
Process Equipment		$650,000	$365,000	$375,000	$400,000
Equipment with Ancillaries		$1,220,000	$965,000	$975,000	$1,012,000
Estimated Annual O&M Cost (Initial ADF)[2]		$428,028	$397,950	$591,961	$460,732
Planning Level Cost[3]		$14,794,000	$12,080,000	$12,158,000	$12,647,000
Approx. Power Consumption (kWh/DT)		228	83	47	61
Installed Locations/References		City of Franklin WWTP, Franklin, TN	Arbennie Pritchett WRF, Okaloosa County, FL	Winter Haven WWTP, Winter Haven, FL	Fourth Creek WWTP, Statesville, NC

[1] Average day basis.

[2] Cost includes labor and fule costs for hauling thickened sludge from new WWTP to existing WWTP.

[2] Planning level cost includes process equipment and appurtenances, structures and demolition (where applicable), labor, construction equipment and material, sitework, piping, permits, tax, bonds, design and construction services, and design and construction contingencies. See Table 19 for additional details.

Aerobic and anaerobic digestion are two of the most commonly used stabilization processes. The stabilization alternatives selected at Biosolids Workshop No. 1 were mesophilic anaerobic digestion (MAD) and no stabilization. Aerobic digestion was not considered, because it has a higher energy demand than anaerobic digestion.

4.2.1 Stabilization Design Criteria and Assumptions

As previously stated, two of the City's priorities are to increase its disposal options and to produce biosolids that meet the Class A pathogen reduction criteria in the Federal 40 CFR Part 503 regulations. Biosolids meeting the Class A criterion are nearly pathogen free and may be used by the general public for agronomic purposes, including fertilizing feed and food crops, lawns, and home gardens.

The MAD system described routinely produces biosolids that meet Class B pathogen reduction criteria. Although thermophilic phased anaerobic digestion (TPAD), an adaptation of MAD, is capable of producing Class A biosolids, the energy requirements for TPAD are higher due to the need to maintain a higher process temperature (140°F to 160°F) compared to MAD (90°F to 100°F). The knowledge base for operation of TPAD is also limited. Lastly, in Options 2 and 3, where MAD is used, subsequent solar or thermal drying of the Class B digested biosolids is expected to produce the desired Class A pathogen reduction.

The design parameters for stabilization are provided in **Table 7**. These parameters apply to the two treatment train options (Options 2 and 3) where stabilization is included. For the two-plant configuration, the Option 2 and 3 digesters were conceptually sized to process the thickened sludge produced at Franklin WWTP, as well as the thickened sludge brought over from the new WWTP.

4.2.2 No Stabilization Step (Options 1 & 4)

In Option 1, Franklin WWTP's existing solids treatment process (DAF thickening, BFP dewatering, landfill disposal) is duplicated. Therefore, no stabilization step was considered for this option. In addition, no stabilization was considered for Option 4; because belt drying with energy recovery does not require or benefit from stabilization. The volatile solids that are normally destroyed by stabilization are desirable to the energy recovery process because they contribute to the heating value of the dried biosolids.

Table 7
Stabilization Design Parameters (Options 2 & 3)

Parameter	Initial Loadings (9.4 MGD ADF)		Design Loadings (24 MGD ADF)	
	Avg. Day	Max. Month	Avg. Day	Max. Month
Hydraulic Loading (gal/day)	30,000	38,000	76,000	98,000
Total Suspended Solids (lbs/day)	12,300	16,000	31,500	41,000
Volatile Suspended Solids (lbs/day)	8,400	10,900	21,400	27,800
Volatile Solids Destruction, minimum (%)	40			
SRT, design / minimum (days)	20 / 15			

4.2.3 Mesophilic Anaerobic Digestion (Options 2 & 3)

System Description

Anaerobic digestion is a widely used stabilization method that uses anaerobic microbes to perform a series of biochemical transformations. These transformations break down complex organic compounds in wastewater sludges into methane and carbon dioxide.

Numerous variations on the anaerobic digestion process, from standard rate to 2-stage digestion, have been developed over decades of practice. Conventional MAD was selected for this concept-level analysis due to its relatively low energy consumption, the potential for beneficial use of the biogas produced by the digestion process, and well-established operating procedures.

A typical MAD system consists of one or more closed, insulated digester tanks, with a mixing system, a heating system, and biogas handling equipment. Biogas produced by the anaerobic microbes is captured from the headspace of the tank; the biogas can be treated and stored for reuse, or it can be disposed via flare. Energy from the biogas may be used to heat the digester, generate electricity, or serve other heating needs around a WWTP. **Figure 6** shows an example of an anaerobic digester installation.

Figure 6
Anaerobic Digesters at Gilder Creek WWTP, Greenville, South Carolina

Conventional MAD operates at temperatures between 90°F and 100°F. This temperature is maintained by an automated digester heating system, which uses a hot water loop and a heat exchanger to transfer heat to the sludge. A mixing system keeps the contents of the digester tank well-mixed, preventing accumulation of solids at the bottom of the tank and ensuring a uniform temperature throughout the tank. This mixing system may be a gas mixing system, which uses biogas to create an upflow pattern in the tank, or a pumped mixing system, which recirculates sludge through specially designed mixing nozzles.

Biosolids produced by MAD meet Class B pathogen reduction standards; in order to meet Class A pathogen reduction standards, it is necessary to include a subsequent treatment method capable of achieving Class A pathogen reduction. CDM's conceptual design for MAD was based on a minimum design solids retention time (SRT) of 15 days at the design condition. It was assumed that MAD would achieve a volatile solids destruction of 40 percent.

Conceptual System Design

A conceptual configuration for Options 2 and 3 mesophilic digestion at Franklin WWTP consists of two anaerobic digesters, each with a volume of approximately 0.8 MG. Under this conceptual configuration, the biogas would be captured and used in the boilers to heat the solids in the digesters. Excess biogas would be flared. It is anticipated that the biogas produced at initial and design conditions would be sufficient to supply the heating needs of the digesters, with natural gas or fuel oil available as an auxiliary fuel for the boilers. Energy use for recirculating and mixing the digester contents would be approximately 230 to 300 kWh/DT. The estimated planning level capital cost to construct the MAD system equipment and structures is $11.3 million.

4.2.4 Summary of Stabilization Alternatives

A summary of the unit process requirements for the stabilization alternatives considered is provided in **Table 8**. Table 8 provides design criteria for the Franklin WWTP based on the solids mass balance discussed in TM No. 1 and also provides planning level cost estimates of equipment costs, along with ancillary requirements, estimated annual operations and maintenance costs (including energy) and a list of installation locations.

Table 8

Comparison of Stabilization Technologies - Existing WWTP at 24 MGD

Parameter		Option 1	Option 2	Option 3	Option 4
Technology		None	Mesophilic Anaerobic Digestion (MAD)	Mesophilic Anaerobic Digestion (MAD)	None
Solids Loading Rate (lbs/day)[1]	Initial	n/a	12,000	12,000	n/a
	Design	n/a	31,000	31,000	n/a
Volatile Solids Destruction		n/a	40%	40%	n/a
Design SRT (days)		n/a	20	20	n/a
Feed Solids		n/a	5.0%	5.0%	n/a
Discharge Solids		n/a	3.6%	3.6%	n/a
Unit Dimensions (ft) & Capacity (MG)		n/a	55 ft diameter & 40 ft SWD 0.77 MG	55 ft diameter & 40 ft SWD 0.77 MG	n/a
Number of Units	Initial	n/a	1	1	n/a
	Design	n/a	2	2	n/a
	Standby	n/a	0	0	n/a
Ancillaries Required					
Mixing System		n/a	X	X	n/a
Aeration System		n/a			n/a
Heating System		n/a	X	X	n/a
Odor Control		n/a			n/a
Biogas Collection		n/a	X	X	n/a
Estimated Capital Cost					
Process Equipment		n/a	$1,410,000	$1,410,000	n/a
Equipment with Structures		n/a	$3,417,120	$3,417,120	n/a
Estimated Annual O&M Cost (Initial ADF)		n/a	$57,364	$263,088	n/a
Planning Level Cost[1]		n/a	$11,250,000	$11,250,000	n/a
Approx. Power Consumption (kWh/DT)		n/a	294	294	n/a
Installed Locations/References		n/a	Palm Beach County, Florida		n/a
Comments		n/a	Potential for biogas capture for green energy		n/a

[1] Planning level cost includes process equipment and appurtenances, structures and demolition (where applicable), labor, construction equipment and material, sitework, piping, permits, tax, bonds, design and construction services, and design and construction contingencies. See Table 18 for additional details.

4.3 Dewatering Alternatives

4.3.1 Dewatering Design Criteria & Assumptions

Due to their age and condition, the Franklin WWTP's two existing BFPs operate for four days each week on a near-continuous basis. For this analysis, CDM has assumed that dewatering would be performed seven days per week in order to coordinate with a seven-day drying schedule in Options 2, 3 and 4. The dewatering schedule can be modified to accommodate a five day work week, with longer dewatering hours and possibly additional shifts. **Tables 9 and 10** summarize the dewatering design parameters used to compare the technologies during the maximum month design condition. If a drying process is ultimately selected for implementation, the dewatering process operation schedule must be clearly coordinated with the drying operations.

Table 9
Dewatering Design Parameters – Options 1 & 4
Existing WWTP at 24 MGD

Parameter	Initial Loadings (9.4 MGD ADF)		Design Loadings (24 MGD ADF)	
	Avg. Day	Max. Month	Avg. Day	Max. Month
Hydraulic Loading (gal/day)	30,000	38,000	76,000	98,000
Solids Loading (lbs/day)	12,300	16,000	31,500	40,900
Feed Sludge Solids Content (%)	5.0			
Dewatered Sludge Solids Content (%)	20			
Dewatering Polymer Consumption (lbs/DT)	20			
Solids Capture (%)	95			

Table 10
Dewatering Design Parameters – Options 2 & 3
Existing WWTP at 24 MGD

Parameter	Initial Loadings (9.4 MGD ADF)		Design Loadings (24 MGD ADF)	
	Avg. Day	Max. Month	Avg. Day	Max. Month
Hydraulic Loading (gal/day)	30,000	38,000	76,000	98,000
Solids Loading (lbs/day)	9,000	11,700	22,900	29,800
Feed Sludge Solids Content (%)	3.6			
Dewatered Sludge Solids Content (%)	20			
Dewatering Polymer Consumption (lbs/DT)	20			
Solids Capture (%)	95			

One of the City's objectives for the solids treatment process is to produce a drier cake than the current process provides, thus reducing the amount that must be landfilled or dried. The City also prefers an enclosed system due to its ability to capture and treat potential odors.

Solids conditioning is critical to dewatering performance. Operational experience with each of the selected dewatering alternatives indicates that dewatering operations are substantially aided by a consistent solids feed and the application of polymer.

4.3.2 Belt Filter Press (Option 1)

System Description

Introduced to North America in the 1970s and widely used, the BFP employs two and as many as three moving belts, as well as a combination of gravity drainage and mechanical compression, to dewater solids. Polymer-conditioned solids are first uniformly deposited on the belt in the gravity drainage section, where free water is removed. Next, low- and high-pressure sections squeeze the solids between two tensioned belts and into a filtrate drain at the bottom of the unit. Doctor blades scrape and remove the dewatered cake from the belts. BFPs typically produce cake solids ranging from 12 to 20 percent.

The City currently uses two two-meter Ashbrook Simon-Hartley Klampress model BFPs to dewater thickened WAS from the DAF process. The thickened sludge from the BFP feed tanks is typically around 3.5-percent solids. The BFPs achieve a cake solids content of approximately 14.5 percent. The dewatered solids are stored in trucks prior to disposal at Camden Landfill, and the

dewatering filtrate is returned to the headworks of the WWTP. The City intends to have the BFPs refurbished this year in the hopes of improving their performance and reliability.

The BFP is an open operating environment. Without the presence of a hood to capture odors, the entire BFP building (or the entire room containing the presses) will require ventilation to the odor control system. Also, safety barriers must be provided in order to protect plant staff from various moving parts. Plant operators must be diligent in maintenance procedures, as maintaining the proper tension in the belt rollers is vital to the performance of the press. The press also requires a relatively large amount of wash water that must be sent back to the head of the plant. An example of a BFP installation is shown in **Figure 7.**

Figure 7
Two-Meter Andritz Belt Filter Press at the Dallas SWWTP, Dallas, TX

Conceptual System Design

A conceptual design for Franklin WWTP consists of four 2-meter BFPs (three duty, one standby) at design conditions. Each BFP would be sized for a solids loading of 1,400 pounds per hour. Polymer dosing stations and feed pumps would also be provided.

The planning level costs to construct the Option 1 BFP system equipment and structures are $8.1 million, and a complete BFP dewatering train is expected to consume approximately 42 kWh/DT.

4.3.3 Screw Press (Option 2)

System Description

The screw press operates in a manner similar to the presses used to dewater screenings removed at the headworks of a WWTP and the rotary screw thickeners described in Section 4.1.4. The feed solids, after being conditioned with polymer in a flocculation reactor, are introduced at the bottom end of an inclined trough that contains a perforated bucket surrounding a slowly, continuously rotating screw. The solids first lose their free water via gravity drainage. As the screw conveys the solids higher along the tube, the screw flights become narrower and the solids are squeezed against a cone at the top of the unit to force additional water out. Having reached the top of the unit, the dewatered cake drops out of the unit and onto a conveyor, into the inlet of a positive-displacement pump, or into a storage vessel. The filtrate flows out through the basket and drops out through a pipe at the bottom end of the unit. Inside the unit, the screw flights are continuously brushed clean, and the basket is periodically flushed with a small quantity of wash water.

Figure 8
Mobile Screw Press used for Demonstration at the Beaver Creek WWTP, Powell, Tennessee

Simple in construction and operation, the screw press is compact and completely enclosed. Energy consumption and noise are low due to the low speed and low horsepower of the variable-speed, screw drive motor. The screw press requires less water than a BFP, and the amount of polymer required for effective dewatering is low. The typical solids capture rate is 95 percent or more. If a progressive cavity or rotary lobe pump is used to feed solids to the screw press, then the flocculation tank may not be required. Double disc, hose, and diaphragm pumps, if used for solids feed, would require the flocculation tank in order to achieve proper mixing and to dampen the flow pulsations inherent to these types of pumps.

Conceptual System Design

A screw press installation at Franklin WWTP would require a total of four presses (three duty, one standby) in the design year. Each press has a maximum solids loading of 900 pounds per hour. As stated above, the use of rotary lobe or progressive cavity pumps would eliminate the need for a flocculation tank. A polymer dosing station and sludge feed pump would be provided along with each screw press.

The Option 2 planning level costs to construct the screw press system equipment and structures are $15.3 million. A screw press dewatering train would consume approximately 50 kWh/DT.

4.3.4 Solid-Bowl Centrifuge (Options 3 & 4)

System Description

In a dewatering centrifuge, solids are subjected to a centrifugal force of more than 3,000 times the force of gravity. This centrifugal force facilitates separation of solids from liquids based on the density difference between the solid and liquid fractions. Two types of centrifuges have been primarily used for dewatering solids: the imperforate-basket centrifuge and the solid-bowl centrifuge. Continuous improvements to solid-bowl centrifuges, paired with limitations inherent to imperforate-basket centrifuges, have made solid-bowl centrifuges the preferred technology in

municipal wastewater. Therefore, only solid-bowl centrifuges will be considered in this evaluation. A solid-bowl dewatering centrifuge installation is shown in **Figure 9.**

Figure 9
Andritz (Bird) CP-3084 Dwatering Centrifuges
at R.L. Sutton WRF, Smyrna, Georgia

Regardless of whether the centrifuge is co- or counter-current design, the main components of the solid-bowl centrifuge are the bowl and the scroll. Each component is driven by a separate

electrical motor. The bowl spins at high speeds with the scroll spinning at a lower speed. Solids from the feed material settle against the inner wall of the bowl and are continuously swept to the solids discharge end by the scroll. An adjustable weir at the end of the bowl controls the flow of the removed liquid, or centrate, discharged from the centrifuge. Dewatered cake falls out of the bottom of the centrifuge through a discharge chute that can be connected directly to a storage container, such as a silo, or to a screw or belt conveyor. Centrate flows out through a separate discharge chute that can be piped to the head of the plant. Typical solids capture rates can easily exceed 90 percent.

Centrifuges are completely enclosed. Odorous air is typically collected at the centrate chute for treatment by an odor control system. In addition to producing a drier cake than BFPs, centrifuges tend to require less water, less space, and less operator attention; whereas BFPs tend to require less energy and polymer per ton of solids.

Conceptual System Design

A conceptual Option 3 centrifuge facility at Franklin WWTP would consist of three centrifuges (two duty and one standby), each sized for a throughput of 1,280 lbs/hr for Option 3. The system would include positive-displacement sludge feed pumps, polymer application systems, and connection of the liquid discharge chutes to a building odor control system. For maximum control of odors, a screw conveyor would be the likely choice for conveyance of the dewatered cake to any further processes. The planning level cost to construct the Option 3 centrifuge system equipment and structures is approximately $4.3 million, and the average power consumption would be approximately 90 kWh/DT.

Because Option 4 lacks a stabilization process, the conceptual centrifuge equipment selection must account for a larger quantity of solids than Option 3. Therefore, a centrifuge with a capacity of 1,500 lbs/hr was selected. Four of these centrifuges (three duty, one standby) would be required in the design year, including the ancillary equipment mentioned above. The planning level costs to construct the Option 4 centrifuge system equipment and structures are $6.2 million, and its average power consumption would be approximately 95 kWh/DT.

4.3.5 Summary of Dewatering Alternatives

A summary of the unit process requirements for these dewatering alternatives is provided in **Table 11.** Table 11 also provides a summary of design criteria for the Franklin WWTP based on the solids mass balance discussed during TM No. 1 and provides planning level estimates of equipment costs, along with ancillary requirements, estimated annual operations and maintenance costs (including labor, energy, and chemicals), and a list of installation locations.

Table 11

Comparison of Dewatering Technologies - Existing WWTP at 24 MGD

Parameter		Option 1	Option 2	Option 3	Option 4
Technology		Belt Filter Press (BFP)	Screw Press	Centrifuge	Centrifuge
Conceptual Model Selection		Komline-Sanderson GRSL-2 Kompress	Huber Technology RoS3 Q800 Screw Press	Andritz D4LL High Solids Centrifuge	Andritz D4LL High Solids Centrifuge
Solids Loading Rate (lbs/day)[1]	Initial	12,000	9,000	9,000	12,000
	Design	31,000	23,000	23,000	31,000
Unit Capacity (lbs/hr)		1,400	900	1,280	1,500
Feed Solids		5.0%	3.6%	3.6%	5.0%
Discharge Solids		20%	20%	20%	20%
Polymer Consumption (lbs/DT)		20	20	20	20
Number of Units	Initial	2	2	1	1
	Design	3	3	2	3
	Standby	1	1	1	1
Proposed Operating Schedule (days/hours)	Initial	7 / 4.4	7 / 5	7 / 7	7 / 8.2
	Design	7 / 7.5	7 / 8.5	7 / 9	7 / 7
Ancillaries Required					
Polymer Feed Tank		X	X	X	X
Flocculation Tank		X			
Wash Water		X	X	X	X
Odor Control		X	X	X	X
Upstream Grinder		X	X	X	X
Feed Sludge Mixer		X	X	X	X
Estimated Capital Cost					
Process Equipment		$1,060,000	$1,280,000	$795,000	$1,252,000
Equipment with Structures		$1,540,000	$1,750,000	$1,085,000	$1,582,000
Estimated Annual O&M Cost (Initial ADF)		$141,000	$145,000	$313,000	$478,000
Planning Level Cost[2]		$8,119,000	$15,281,000	$4,245,000	$6,163,000
Approx. Power Consumption (kWh/DT)		42	50	87	94
Installed Locations/References		City of Franklin WWTP, Franklin, TN	Fleming Island Regional WWTP, Orange Park, FL	R. L. Sutton WRF, Smyrna, GA	

[1] Average day basis.

[2] Planning level cost includes process equipment and appurtenances, structures and demolition (where applicable), labor, construction equipment and material, sitework, piping, permits, tax, bonds, design and construction services, and design and construction contingencies. See Table 18 for additional details.

4.4 Drying Alternatives

Thermal drying processes are intended to further reduce the volume of solids to be managed. The drying process exposes the dewatered cake to heat, evaporating the water and reducing the moisture content to 10 percent or less. The heat transfer methods used in drying are typically categorized by one of the following methods:

- Radiation: Radiant energy transfers to the solids and evaporates the moisture. The source of the radiant energy can be the sun, infrared lamps, heating elements, gas-heated refractorys, or microwaves.
- Conduction: The solids are separated from the heat-transfer medium (steam or other hot fluid) by a solid wall. This method is also known as indirect drying.
- Convection: Hot gases are typically used as the heat-transfer medium. The solids make direct contact with the hot gases to evaporate moisture. This method is termed direct drying.
- A combination of the above.

Drying is not currently practiced at Franklin WWTP. The technologies that were short-listed for further evaluation included rotary drum drying, belt drying with and without the proprietary energy recovery system, and solar drying.

4.4.1 Drying Design Criteria and Assumptions

Longer operating schedules, with minimal shutdowns and minimal time between shutdowns, are beneficial to thermal drying processes. Not only does minimizing the number of startups and shutdowns save the equipment from excessive heat cycling and wear and tear, it also saves energy by reducing the amount of heating required to bring the process up to operating temperature from shutdown or standby mode. Significant capital cost reductions can also be realized when a drying unit is sized for 24/5 or 24/7 operation, compared to 8/5 operation. **Tables 12 and 13** summarize the design parameters used to compare the drying processes.

Table 12
Drying Design Parameters – Options 2 & 3
Existing WWTP at 24 mgd

Parameter	Initial Loadings (9.4 MGD ADF)		Design Loadings (24 MGD ADF)	
	Avg. Day	Max. Month	Avg. Day	Max. Month
Solids Loading (lbs/day)	8,500	11,100	21,800	28,300
Feed Sludge Solids Content (%)	20			
Dried Sludge Solids Content (%)	80 (Option 2 solar dryer); 90 (Option 3 belt dryer)			

Table 13
Drying Design Parameters – Option 4 (Belt Dryer with ERS)
Existing WWTP at 24 MGD

Parameter	Initial Loadings (9.4 MGD ADF)		Design Loadings (24 MGD ADF)	
	Avg. Day	Max. Month	Avg. Day	Max. Month
Solids Loading (lbs/day)	11,700	15,200	29,900	38,900
Feed Sludge Solids Content (%)	20			
Dried Sludge Solids Content (%)	N/A[1]			

[1] Dried solids from the ERS system are classified as ash.

The facilities for cake storage and the means of transporting cake to the drying process must be sized to provide the process with a continuous supply of cake. If dewatering operations only perform during intermittent cycles, the cake storage silo must have sufficient volume to contain the solids produced during the shift, plus the previous day's cake production that has yet to be fed to the dryer. Progressive cavity pumps may be used to transport cake to the inlet of the drying process.

Consistency of the cake feed is also important for a stable thermal drying process. A slug of unusually wet cake can disrupt drying operations by sticking to or clogging internal components of the dryer. The sudden appearance of combustible material or solvents in the feed cake can also cause operational problems in the thermal process.

4.4.2 Rotary Drum Dryer

System Description

The rotary drum drying system is an established method of direct drying in the United States. The system consists of a horizontally mounted, slowly rotating cylindrical steel shell into which cake and hot gases are fed. The wet cake feed is mixed with dried solids returned from the outlet of the dryer to reduce the moisture content of the feed and to prevent the wet cake from sticking to the

inside of the unit. Axial flights welded to the interior of the dryer cause the material to cascade continuously and dry rapidly.

Rotary dryer configurations can be single or triple pass. However, most modern units are the triple-pass design. Dried solids are typically discharged to a cyclone separator for cooling and air pollution control. Venturi or tower scrubbers are usually provided for particulate control, and odor control measures can include chlorine or permanganate-based scrubbers or afterburners for thermal oxidation. The process is completely enclosed, with few external moving parts.

Conceptual System Design

The DDS 20 system is the smallest Andritz drum dryer installed in the United States. This dryer evaporates up to 4,400 pounds of water per hour and dries dewatered cake to a solids content of 90 percent. However, based on the anticipated solids loading to the dryer, the DDS 20 would operate on a relatively short schedule — an average of 69 hours per week in the design year. As previously mentioned, thermal drying can be most economical at longer operating schedules. Therefore, in order to achieve a longer operating schedule, CDM selected a smaller belt dryer for Option 3.

4.4.3 Belt Dryer (Options 3 and 4)

System Description

Belt drying is becoming established in Europe, but only a handful of installations have been completed in the U.S. Inside the belt dryer, cake is uniformly deposited upon a slowly moving belt made of polypropylene or a metal mesh. As the belt travels through the dryer, hot gases evaporate the moisture in the cake. The dryer is a completely enclosed process, and like the rotary dryer, the exhaust gases from the dryer are treated to remove particulates and odors. Commercially available belt drying systems from Andritz and Krüger can utilize a variety of energy sources including oil, natural gas, steam, and biogas from a digestion process. Andritz's BDS belt drying system recycles some of the dried material to the head of the process in order to reduce the moisture content of the feed. Krüger's BioCon system does not. The dried product is air cooled at the end of the belt, then transferred via screw or pneumatic conveyor to silos for storage and ultimate disposal or sale.

Manufacturers of belt drying systems tout the system's safety features, such as the enclosed nature of the process and the relatively low operating temperature, low speed of the belt, the reduced explosion hazard due to very low dusting, and the fully automated operation.

Belt dryer manufacturers offer a variety of energy recovery methods that can be used to convert the dried sludge into useful energy, or to capture and reuse the heat generated during the drying process. For example, Krüger offers two optional recovery systems: the BioCon Energy Exchange System (BEES) and the BioCon Energy Recovery System (BERS). BERS was selected during Biosolids Workshop No. 1 for inclusion in Option 4.

The BERS option employs a furnace at the end of the dryer to convert the dried product into thermal energy for the drying process; thus reducing the requirement for natural gas, oil, or other sources of heat energy. Flue gases from the furnace are sent to a heat exchanger or steam boiler, where the heat is transferred to the air used to dry the sludge inside the belt dryer. The manufacturer reports that the final product can be 5 percent of the original quantity. The ash from the furnace can be sent to a landfill or reused as construction filler material.

Krüger advises that the addition of BERS is better suited to undigested sludge, which has a higher volatile solids content and a correspondingly higher heating value than digested sludge. The digested sludge could provide roughly one-half to two-thirds of the total heating energy for the dryer. Correspondingly, the amount of natural gas or oil consumed could be reduced by one-half to two-thirds. According to Krüger, the BERS option would approximately double the amount of system maintenance required. Furthermore, it would also require the addition of activated carbon and hydrated lime (51 and 214 tons per year in 2040, respectively) to treat the flue gases from the furnace. Annual fuel consumption and dried solids storage and disposal costs, however, would be reduced. A photo of a Krüger BioCon belt dryer with BERS is shown in **Figure 10.**

Figure 10
Kruger BioCon Belt Dryer with Energy Recovery System
(from: Krugerusa.com/en/press/20090325,12652.htm)

Conceptual System Design

For Option 3, CDM selected the Andritz BDS 15 drying system as the basis of conceptual design. One dryer, capable of evaporating up to 3,300 pounds of water per hour, would be required at initial solids loadings, and a matching second dryer would be added to handle solids in the design year. No standby unit would be provided. The planning capital equipment cost of the BDS 15 belt drying system is approximately $43.8 million. Its approximate power consumption is 362 kWh/DT.

For Option 4, the Kruger BioCon with BERS was selected as the basis for conceptual design. This dryer is capable of evaporating up to 4,900 pounds of water per hour. Similar to Option 3, one dryer would be required at initial loadings, and a second dryer would be added in the future to

accommodate the design solids loading. No standby unit would be installed. The planning level cost to construct the BioCon dryer with BERS, including equipment and structures, is approximately $62.9 million. The approximate power consumption is 331 kWh/DT.

These capital costs include the dryer and ancillary equipment such as a cake dosing screw conveyor, dried product recycle (where applicable), exhaust gas treatment, heating equipment, and controls. Both Krüger and Andritz offer various optional extras, including dried product cooling prior to silo storage, dried product storage silos and offloading equipment, and nitrogen energy from the drying process. An odor control system was included in the planning level equipment cost.

4.4.5 Solar Drying (Option 2)

System Description

In a solar drying system, the cake is transferred, manually or automatically, to large greenhouse structures where it is spread uniformly on the floor to dry. Some solar drying systems, such as Krüger's Solia system, encourage aerobic fermentation of the sludge by arranging the material in windrows. Solar radiation evaporates the moisture. Machines automatically till the solids, exposing moist solids to the air for further drying. The moisture-laden air is removed from the greenhouses and can be treated for odors before being released to the atmosphere. The air flow through the greenhouses is automatically controlled. Overall, the system requires minimal operator attention.

Continuous, year-round operation can be achieved with solar drying. If there is not enough incident sunlight to support the solar drying operation, or if there is not enough space for the drying beds, supplemental equipment can be added to speed the drying process. For example, the addition of radiant heating elements in the floor of the drying beds can facilitate heat transfer into the drying solids. Perforated flooring enables air to circulate better and remove more moisture. The circulated air can also be heated to further facilitation evaporation.

Energy efficiency and low operations and maintenance costs are the obvious advantages of solar drying. The main disadvantage of solar drying is the space required for the facility.

Figure 11 shows an example of a solar dryer installation. The land to the west of the Franklin WWTP could be used for the solar drying bays.

Conceptual System Design

CDM obtained preliminary sizing and budgetary costs from Parkson (Thermo-System) and Krüger (Solia system). Both systems would require approximately 5 acres of drying area in 2040. The planning level cost to construct a solar drying system is approximately $46.0 million. The anticipated power consumption of the system, with odor control equipment included, is 1,100 to

1,400 kWh/DT. Elimination of the odor control system would reduce the power consumption to approximately 260 to 340 kWh/DT.

Figure 11
Parkson Thermo-System solar dryer installation in Okeechobee, Florida

4.4.6 Summary of Drying Processes

A summary of the unit process requirements for these drying alternatives is provided in **Table 14.** Table 14 provides design criteria for the Franklin WWTP based on the solids mass balance discussed in TM No. 1 and provides planning level estimates of equipment costs along with ancillary requirements, estimated annual operations and maintenance costs (including labor, energy, and chemicals) and a list of installation locations. Because energy use by the dryer is such an important part of the long-term costs of these technologies, it is difficult to compare costs at the conceptual level without a thorough life-cycle cost analysis, which will be performed in later sections of this TM.

5.1 Preliminary Economic Analysis

5.1.1 Methodology

After developing preliminary equipment sizing, CDM performed a preliminary economic analysis consisting of planning level capital construction and O&M cost estimates, as well as calculation of basic present worth life cycle costs for each option, for both one and two plants. These present worth analyses will be used at an upcoming Steering Committee meeting to evaluate each alternative comparatively and to assist the City in selecting the most suitable treatment train.

Table 14

Comparison of Drying Technologies - Existing WWTP at 24 MGD

Parameter		Option 1	Option 2	Option 3	Option 4
Technology		None	Solar Dryer	Belt Dryer	Belt Dryer with Energy Recovery System
Conceptual Model Selection		n/a	Solar Dryer - Kruger Solia, Parkson Thermo-System or equal	Belt Dryer - Andritz BDS 1.5 or equal	Belt Dryer with ERS - Kruger BioCon or equal
Solids Loading Rate (lbs/day)[1]	Initial	n/a	9,000	9,000	12,000
	Design	n/a	22,000	22,000	30,000
Unit Capacity (lbs/hr)		n/a			
Feed Solids		n/a	20%	20%	20%
Discharge Solids		n/a	80%	90%	Ash
Number of Units	Initial	n/a	8	1	1
	Design	n/a	16	2	2
	Standby	n/a	0	0	0
Proposed Operating Schedule	Initial	n/a	7 / 24	7 / 10.3	7 / 9.6
	Design	n/a	7 / 24	7 / 13.2	7 / 12.3
Estimated Capital Cost					
Process Equipment		n/a	$9,360,000	$7,600,000	$6,822,000
Equipment with Structures		n/a	$9,860,000	$8,050,000	$10,806,000
Estimated Annual O&M Cost (Initial ADF)		n/a	$311,705	$333,628	$482,273
Planning Level Cost[2]		n/a	$46,037,000	$43,779,000	$62,887,000
Approx. Power Consumption (kWh/DT)		n/a	1,411	362	331

[1] Average day basis.

[2] Planning level cost includes process equipment and appurtenances, structures and demolition (where applicable), labor, construction equipment and material, sitework, piping, permits, tax, bonds, design and construction services, and design and construction contingencies. See Table XX for additional details.

5.1.2 Preliminary OPCCs

In performing these analyses, CDM developed cost criteria based on recent experience in the design, bidding, and construction of WWTP improvements and quotations received from suppliers and contractors. The opinions of probable construction cost (OPCCs) presented here include the clarifications and assumptions listed below and in **Table 15.**

- These are planning level OPCCs only, based on construction of complete solids treatment trains for the design year of 2040.
- For the two-plant options, it was assumed that Franklin WWTP and the new WWTP would use identical thickening machines.
- The capital cost of the two-plant options includes procurement of two new tanker trucks, as well as construction of truck loading and receiving facilities at the new WWTP and Franklin WWTP, respectively.
- Rock excavation is not included.
- Only nominal dewatering is needed for new structures.
- No contaminated soil or hazardous materials will be encountered.
- Construction costs are based on a normal 40-hour work week.
- Construction costs are based on 2011 dollars (no escalation applied).

5.1.3 Annual O&M Costs

Planning level annual O&M cost calculations were based on the parameters and assumptions listed in **Table 16.**

In addition to these assumptions of unit costs, CDM's O&M specialists estimated the labor that would be required to operate and maintain each solids treatment train. These estimates were based on the relative complexity of each technology and its anticipated operating schedule at initial and design conditions.

For a direct comparison to the Franklin WWTP's existing solids treatment process, these annual O&M costs were also converted to treatment costs per dry ton (DT) of solids processed.

5.1.4 Life Cycle Cost Assumptions

Table 17 presents the assumptions that were used to calculate the life cycle costs for the single- and two-plant options.

Table 15
Planning Level Capital Cost Assumptions

Cost Item	Value
Allowances Applied to Estimated Construction Cost	
Labor & Material	30 percent
Piping	15 percent
Instrumentation & Electrical	25 percent
Markups Applied to Estimated Contractor's Cost	
Permits	1.0 percent
Sales Tax	9.25 percent
Builder's Risk	0.5 percent
General Liability	1.0 percent
GC Bonds	1.5 percent
General Conditions	10 percent
Overhead & Profit	10 percent
Markups Applied to Estimated Construction Cost	
Construction Contingency	30 percent
Design & Construction Services	15 percent
City Project Admin.	2.0 percent
Legal/Finance	3.0 percent

Table 16
O&M Cost Assumptions

Cost Item	Value
Labor	
Fully Loaded Labor Rate	$35.08 per hour
Utilities	
Electricity	$0.06/kWh
Natural Gas	$10/MMBtu
Chemicals	
Thickening/Dewatering Polymers	$1.60/lb delivered
Sodium Hypochlorite Solution	$1.57/gal delivered
Caustic Soda	$1.54/gal delivered
Muriatic Acid	$2.38/gal delivered
Activated Carbon	$1,200/ton delivered
Hydrated Lime	$150/ton delivered
Biosolids Hauling	
Solids Disposal Locations	
Option 1	Dewatered Cake (17%) to Landfill
Option 2	Dried Biosolids (75%) to Farms
Option 3	Dried Biosolids (90%) to Farms
Option 4	Ash to Landfill
Truck Capacity	20 CY
Landfill Tipping Fee	$22.24/WT
Fuel Cost	$4.50/gal diesel
Truck Fuel Economy	5 miles/gallon
Round Trip Distances	To Landfill: 216 miles Between WWTPs: 13.6 miles To Farms: 100 miles
Driver's Labor Per Load	To Landfill: 13.5 hours Between WWTPs: 3 hours To Farms: 6 hours
Fleet Maintenance	$6,480 per year
Insurance	$440 per year

Table 17
Life Cycle Cost Assumptions

Cost Item	Value
Interest Rate	5 percent
Period	20 years

5.2 Discussion

Tables 18 and 19 present the preliminary OPCCs, O&M costs, and present worth cost of each option for the single- and two-plant configurations.

5.2.1 Preliminary OPCCs

Figures 12 and 13 compare the relative capital cost of each option. Option 1, where the existing treatment train is retained and expanded to suit 2040 conditions, is clearly the least expensive because it only consists of DAF thickening and BFP dewatering; it does not include any stabilization or drying steps.

The remaining three options are closely matched. Although Option 4 lacks a stabilization step, the added expense of the belt dryer with energy recovery brings this option's cost in line with those of Options 2 and 3.

Figure 12
Comparison of Preliminary OPCCs for Options 1 through 4 (One Plant)

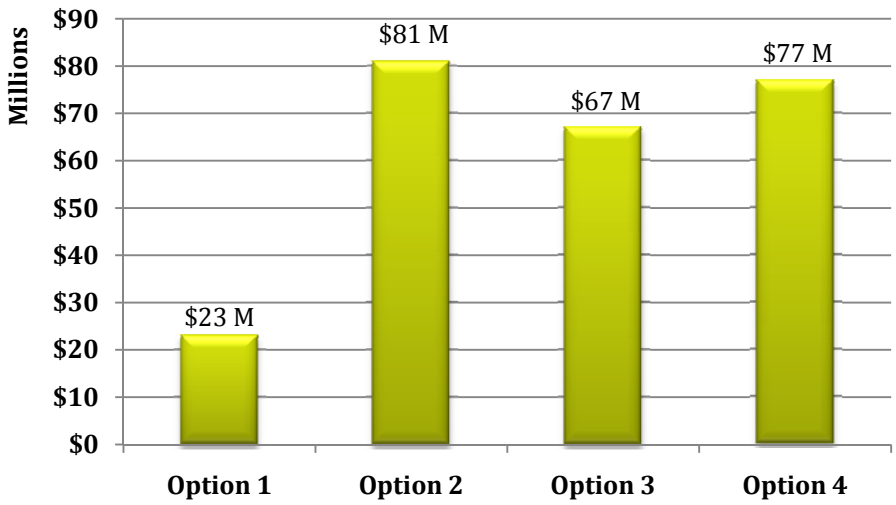
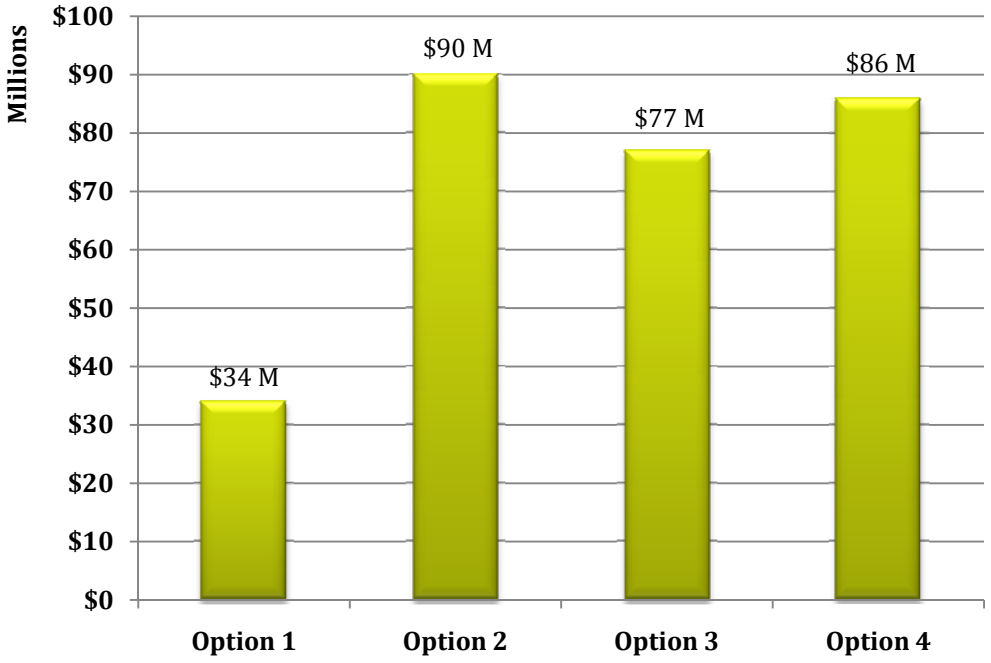


Figure 13
Comparison of Preliminary OPCCs for Options 1 through 4 (Two Plants)



5.2.2 Annual O&M Costs

The calculated annual O&M costs are presented in **Figures 14 and 15.** Separate costs were developed for the initial and design plant conditions in order to illustrate the rise in O&M costs as plant flows increase to 2040.

Although Option 1 and the existing plant have the same process, CDM assumed that the cake solids produced by the BFPs would have a slightly higher solids content (17 percent) compared to the cake that is currently produced (approximately 14.5 percent). This drier cake occupies less volume which, in turn, costs less to transport and dispose of at the Camden landfill. CDM also assumed that the new BFPs in Option 1 would require significantly less operator attention and maintenance, compared to the two existing BFPs, which have reached the end of their service lives.

A comparison of the remaining treatment train options shows that all three have comparatively lower annual O&M costs. Because it was assumed that the Class A biosolids produced by Options 2 and 3 would be beneficially reused, landfill tipping fees are eliminated. Shorter distances to local farms also help to reduce the labor and fuel costs of these two options. Although ash is

hauled to the landfill in Option 4, the volume of ash hauled is considerably smaller than that of the dewatered cake hauled in Option 1.

Heating energy for the rotary drum dryer is a major component of the Option 3 annual O&M cost. Even if it is assumed that excess biogas from the anaerobic digesters can be used as a fuel source for the dryer, there is still a heating deficit that has to be made up by a supplemental fuel source such as natural gas. In Option 4, the belt dryer's entire energy requirements can theoretically be supplied by the energy recovery system, and thus, no supplemental fuel is required.

Figure 14
Comparison of Preliminary Annual O&M Costs for Options 1 through 4 (One Plant)

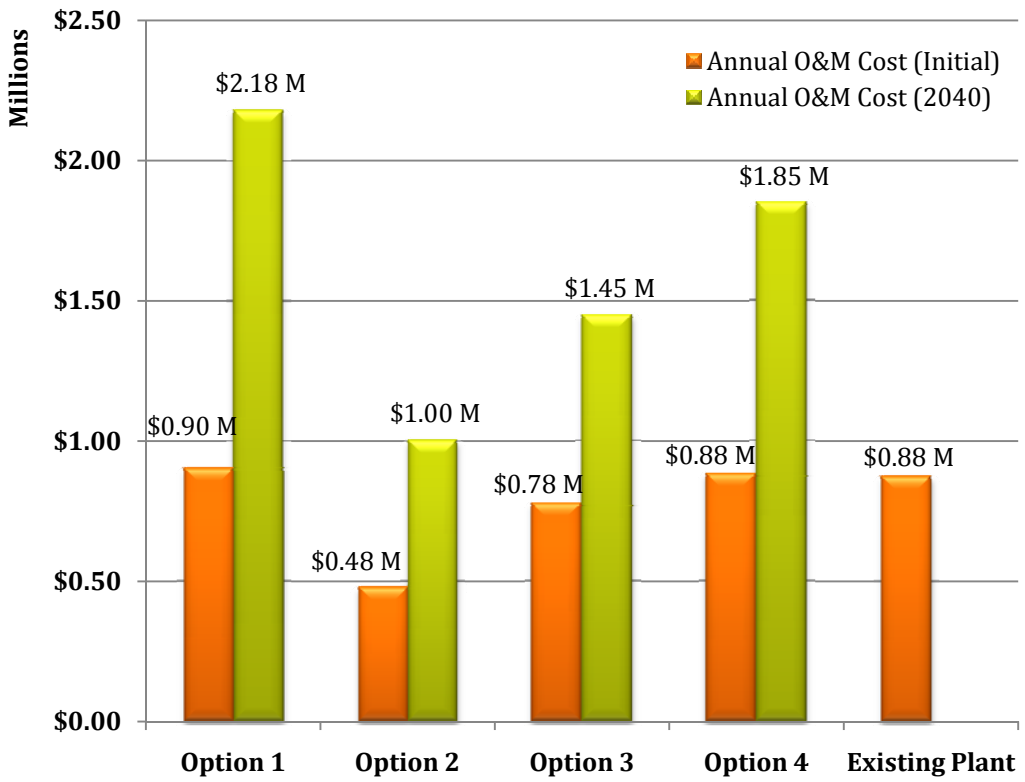
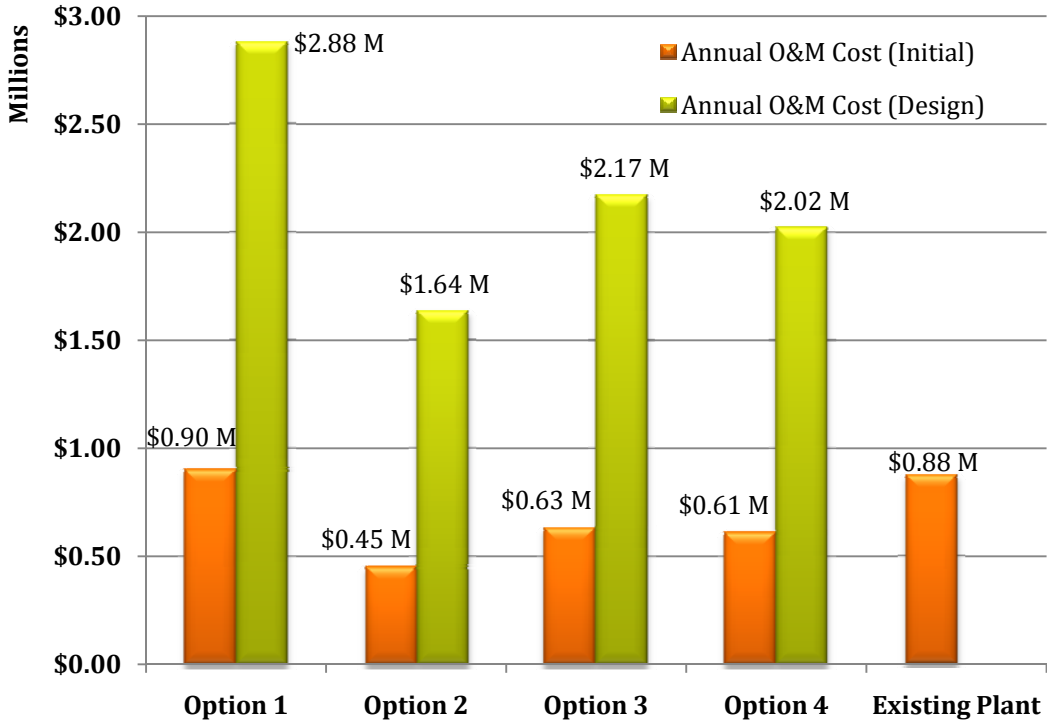


Figure 15
Comparison of Preliminary Annual O&M Costs for Options 1 through 4 (Two Plants)



5.2.3 Life Cycle Costs

The total present worth of each option is shown in **Figures 16 and 17.** With the highest OPCC and the second-highest annual O&M cost, Option 3 has the highest present worth. Despite having the highest annual O&M cost, Option 1 has the lowest total present worth because it also has the lowest OPCC.

Figure 16
Comparison of Preliminary Total Present Worth for Options 1 through 4 (One Plant)

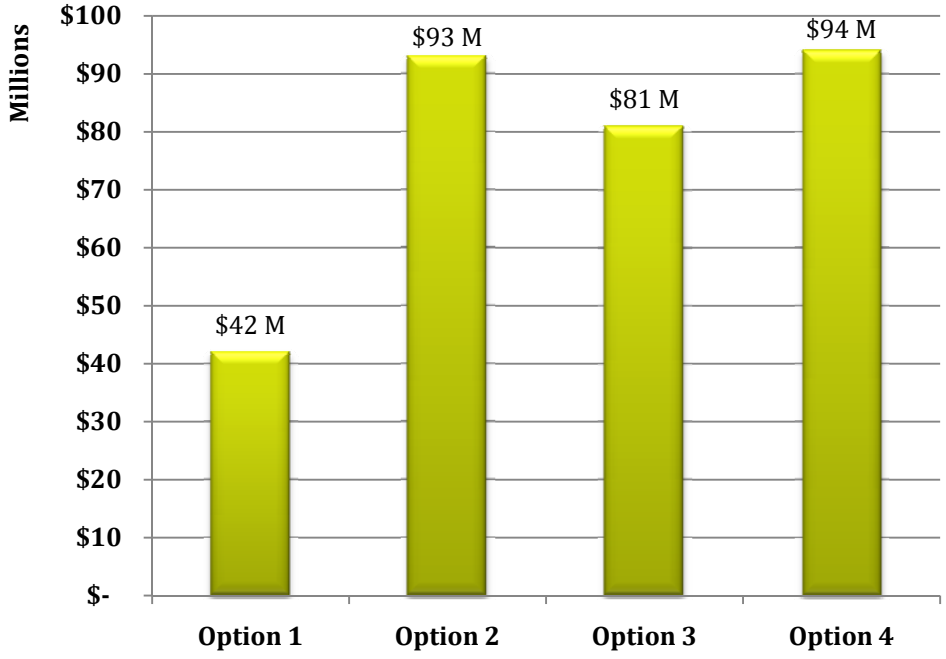
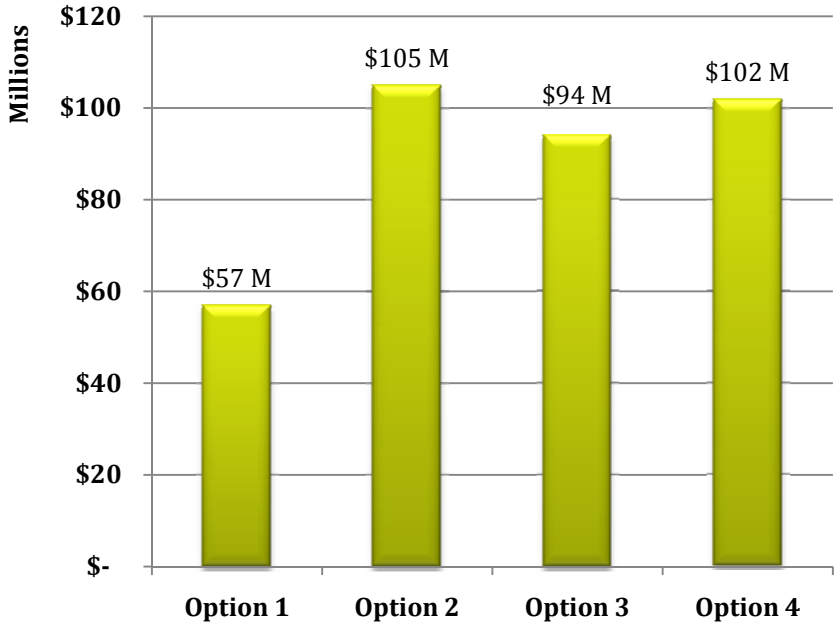


Figure 17
Comparison of Preliminary Total Present Worth for Options 1 through 4 (Two Plants)



5.2.4 Treatment Cost

In TM 1, CDM estimated that the City's current O&M cost to treat solids was $547 per DT of solids processed. This cost was calculated as the first step towards comparing the current process to the four solids treatment trains discussed in this TM. **Figures 18 and 19** compare this current O&M cost to the calculated O&M cost for each option. Just as with the annual O&M costs presented in Section 5.2.2, two O&M costs – one for initial conditions and one for 2040 conditions – are listed for each option because treatment unit costs generally decrease as capacity increases. All four options have treatment costs that are lower than that of the current process.

Option 2 has the lowest calculated treatment cost. Not surprisingly, Option 1 has the highest treatment cost because it is simply a continuation of the City's current practice of hauling large quantities of relatively wet material to the landfill.

Figure 18
Comparison of O&M Cost per Dry Ton Processed (One Plant)

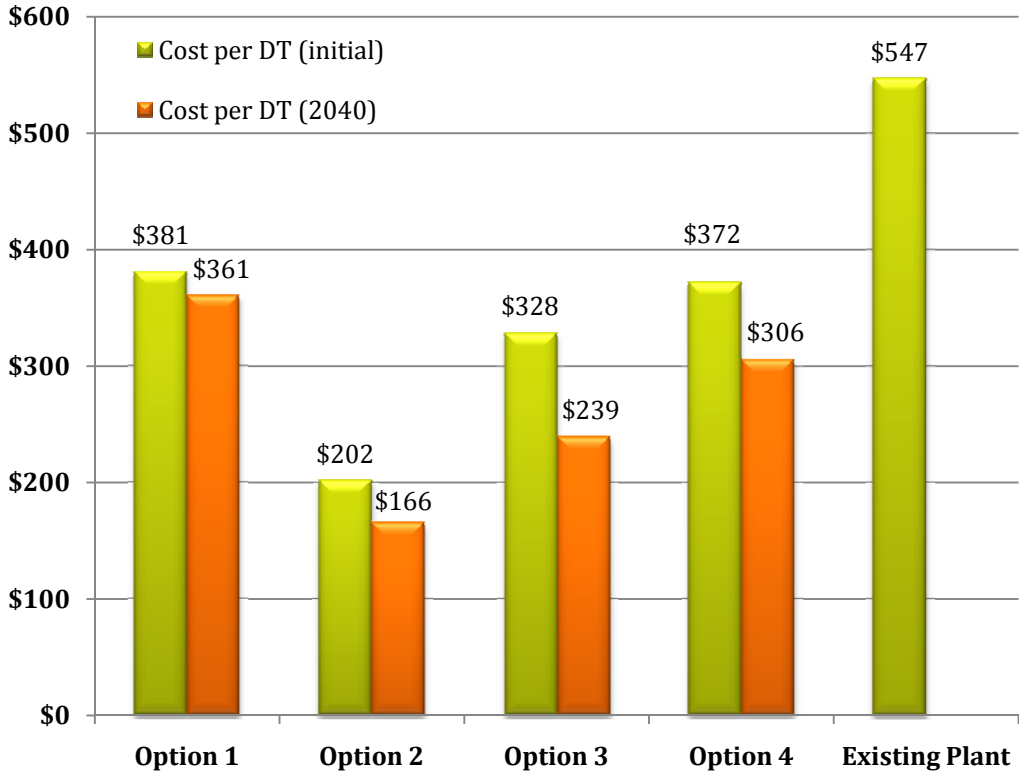
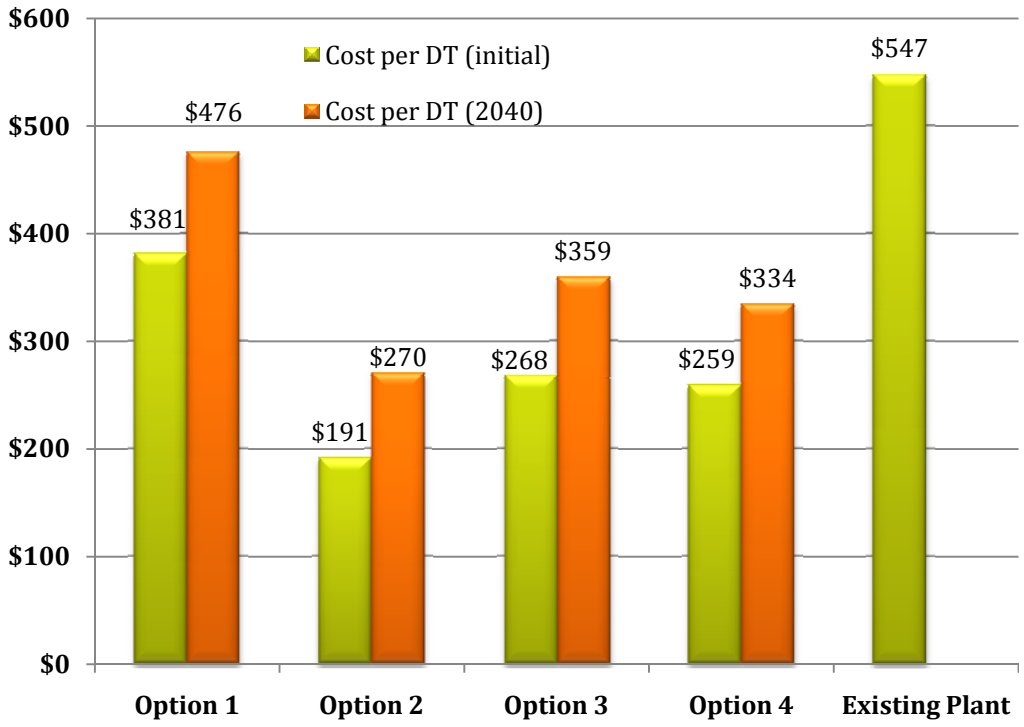


Figure 19
Comparison of O&M Cost per Dry Ton Processed (Two Plants)



5.2.5 Phased Improvements

At this level of cost analysis, CDM has not yet examined phasing of the solids treatment train improvements. It should be possible for the City to reduce its initial capital cost by building the new solids treatment train in two phases. The first phase would consist of the process units needed to meet solids treatment needs in the near future. In a second phase of work, the remaining components of the treatment train, designed to meet treatment needs through 2040, would be added to the first phase.

CDM intends to explore the timing and cost breakdown of phased improvements in a subsequent report.

6.1 September 28, 2011, Steering Committee Meeting

Members of the Steering Committee and CDM met on September 28, 2011, to review the information in this TM and select a biosolids process for conceptual design. Decisions made at the meeting and results of the conceptual design are presented in the Biosolids Report in **Appendix E.**

Table 18
Preliminary OPCCs, O&M Costs and Present Worth - Options 1 through 4

Construction Cost Component	% of Cost	Option 1 - Continue with Existing Process				Option 2 - Solar Dryer				Option 3 - Rotary Drum Dryer				Option 4 - Belt Dryer w/Energy Recovery			
		Thickening	Stabilization	Dewatering	Drying	Thickening	Stabilization	Dewatering	Drying	Thickening	Stabilization	Dewatering	Drying	Thickening	Stabilization	Dewatering	Drying
		Dissolved Air Flotation	None	Belt Filter Press	None	Rotary Drum Thickener	Mesophilic Anaerobic Digestion	Screw Press	Solar Dryer	Screw Thickener	Mesophilic Anaerobic Digestion	Centrifuge	Rotary Drum Dryer	Gravity Belt Thickener	None	Centrifuge	Belt Dryer with ERS
Facilities & Equipment																	
Process Equipment	n/a	$2,375,000	$0	$1,540,000	$0	$1,350,000	$1,410,000	$1,750,000	$9,860,000	$1,127,500	$1,410,000	$1,085,000	$8,050,000	$1,183,000	$0	$1,582,000	$10,806,000
Structure (where applicable)	n/a	$2,140,000	$0	$750,000	$0	$1,060,000	$2,007,120	$3,000,000	$2,824,533	$1,060,000	$2,007,120	$0	$4,500,000	$1,114,000	$0	$0	$7,500,000
Demolition (where applicable)	n/a	$40,000	$0	$20,000	$0	$40,000	$50,000	$20,000	$0	$40,000	$50,000	$20,000	$0	$40,000	$0	$20,000	$0
Labor, Construction Equipment & Misc Material	30%	$712,500	$0	$462,000	$0	$405,000	$423,000	$525,000	$2,958,000	$338,250	$423,000	$325,500	$2,415,000	$354,900	$0	$474,600	$3,241,800
Allowances																	
Sitework	5%	$263,375	$0	$138,600	$0	$142,750	$194,506	$264,750	$782,127	$128,288	$194,506	$71,525	$748,250	$134,595	$0	$103,830	$1,077,390
Piping	15%	$790,125	$0	$415,800	$0	$428,250	$583,518	$794,250	$2,346,380	$384,863	$583,518	$214,575	$2,244,750	$403,785	$0	$311,490	$3,232,170
Instrumentation & Electrical	25%	$1,316,875	$0	$693,000	$0	$713,750	$972,530	$1,323,750	$3,910,633	$641,438	$972,530	$357,625	$3,741,250	$672,975	$0	$519,150	$5,386,950
CONSTRUCTION SUBTOTAL		**$7,637,875**	**$0**	**$4,019,400**	**$0**	**$4,139,750**	**$5,640,675**	**$7,677,750**	**$22,681,673**	**$3,720,338**	**$5,640,675**	**$2,074,225**	**$21,699,250**	**$3,903,255**	**$0**	**$3,011,070**	**$31,244,310**
Permits	1.0%	$76,379	$0	$40,194	$0	$41,398	$56,407	$76,778	$226,817	$37,203	$56,407	$20,742	$216,993	$39,033	$0	$30,111	$312,443
Sales Tax	9.25%	$241,656	$0	$156,695	$0	$137,363	$143,468	$178,063	$1,003,255	$114,723	$143,468	$110,399	$819,088	$120,370	$0	$160,969	$1,099,511
Bonds & Insurance																	
Builder's Risk	0.5%	$38,189	$0	$20,097	$0	$20,699	$28,203	$38,389	$113,408	$18,602	$28,203	$10,371	$108,496	$19,516	$0	$15,055	$156,222
General Liability	1.0%	$76,379	$0	$40,194	$0	$41,398	$56,407	$76,778	$226,817	$37,203	$56,407	$20,742	$216,993	$39,033	$0	$30,111	$312,443
GC Bonds	1.5%	$114,568	$0	$60,291	$0	$62,096	$84,610	$115,166	$340,225	$55,805	$84,610	$31,113	$325,489	$58,549	$0	$45,166	$468,665
SUBTOTAL #1		**$8,185,046**	**$0**	**$4,336,871**	**$0**	**$4,442,703**	**$6,009,769**	**$8,162,923**	**$24,592,195**	**$3,983,874**	**$6,009,769**	**$2,267,593**	**$23,386,308**	**$4,179,755**	**$0**	**$3,292,481**	**$33,593,593**
General Conditions/OH&P																	
General Conditions	10%	$818,505	$0	$433,687	$0	$444,270	$600,977	$816,292	$2,459,220	$398,387	$600,977	$226,759	$2,338,631	$417,976	$0	$329,248	$3,359,359
Overhead & Profit	10%	$818,505	$0	$433,687	$0	$444,270	$600,977	$816,292	$2,459,220	$398,387	$600,977	$226,759	$2,338,631	$417,976	$0	$329,248	$3,359,359
SUBTOTAL #2		**$9,822,056**	**$0**	**$5,204,245**	**$0**	**$5,331,243**	**$7,211,723**	**$9,795,507**	**$29,510,634**	**$4,780,649**	**$7,211,723**	**$2,721,111**	**$28,063,569**	**$5,015,707**	**$0**	**$3,950,978**	**$40,312,311**
Contingency																	
Construction Contingency	30%	$2,946,617	$0	$1,561,274	$0	$1,599,373	$2,163,517	$2,938,652	$8,853,190	$1,434,195	$2,163,517	$816,333	$8,419,071	$1,504,712	$0	$1,185,293	$12,093,693
SUBTOTAL #3		**$12,768,672**	**$0**	**$6,765,519**	**$0**	**$6,930,616**	**$9,375,240**	**$12,734,159**	**$38,363,825**	**$6,214,844**	**$9,375,240**	**$3,537,445**	**$36,482,640**	**$6,520,419**	**$0**	**$5,136,271**	**$52,406,005**
Escalation	0.0%	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**
CONSTRUCTION TOTAL		**$12,769,000**	**$0**	**$6,766,000**	**$0**	**$6,931,000**	**$9,375,000**	**$12,734,000**	**$38,364,000**	**$6,215,000**	**$9,375,000**	**$3,537,000**	**$36,483,000**	**$6,520,000**	**$0**	**$5,136,000**	**$52,406,000**
Design & Construction Services	15%	$1,915,000	$0	$1,015,000	$0	$1,040,000	$1,406,000	$1,910,000	$5,755,000	$932,000	$1,406,000	$531,000	$5,472,000	$978,000	$0	$770,000	$7,861,000
City Project Administration	2.0%	$255,000	$0	$135,000	$0	$139,000	$188,000	$255,000	$767,000	$124,000	$188,000	$71,000	$730,000	$130,000	$0	$103,000	$1,048,000
Legal/Finance	3.0%	$383,000	$0	$203,000	$0	$208,000	$281,000	$382,000	$1,151,000	$186,000	$281,000	$106,000	$1,094,000	$196,000	$0	$154,000	$1,572,000
CAPITAL COST - EACH TECHNOLOGY		**$15,322,000**	**$0**	**$8,119,000**	**$0**	**$8,318,000**	**$11,250,000**	**$15,281,000**	**$46,037,000**	**$7,457,000**	**$11,250,000**	**$4,245,000**	**$43,779,000**	**$7,824,000**	**$0**	**$6,163,000**	**$62,887,000**
TOTAL OPTION CAPITAL COST		**$23,000,000**				**$81,000,000**				**$67,000,000**				**$77,000,000**			
Total Present Worth		**$42,000,000**				**$93,000,000**				**$81,000,000**				**$94,000,000**			
Annual O&M Cost																	
Initial ADF		**$902,000**				**$478,000**				**$778,000**				**$882,000**			
Design ADF		**$2,181,000**				**$1,003,000**				**$1,448,000**				**$1,850,000**			
O&M Cost Per Dry Ton Processed																	
Initial ADF		**$381**				**$202**				**$328**				**$372**			
Design ADF		**$361**				**$166**				**$239**				**$306**			

Notes
1. Engineering cost includes preliminary and final design, construction administration, and field services.
2. Life Cycle Cost Parameters
   Interest Rate 5%
   Life Cycle 20 years

ENR Construction Cost Index 8938.30 for January 2011

Table 19
Preliminary OPCCs, O&M Costs and Present Worth - Options 1 through 4

		Option 1 - Continue with Existing Process					Option 2 - Solar Dryer					Option 3 - Rotary Drum Dryer					Option 4 - Belt Dryer w/Energy Recovery				
		Franklin WWTP				New WWTP	Franklin WWTP				New WWTP	Franklin WWTP				New WWTP	Franklin WWTP				New WWTP
		Thickening	Stabilization	Dewatering	Drying	Thickening	Thickening	Stabilization	Dewatering	Drying	Thickening	Thickening	Stabilization	Dewatering	Drying	Thickening	Thickening	Stabilization	Dewatering	Drying	Thickening
Construction Cost Component	**% of Cost**	**Dissolved Air Flotation**	**None**	**Belt Filter Press**	**None**	**Dissolved Air Flotation**	**Rotary Drum Thickener**	**Mesophilic Anaerobic Digestion**	**Screw Press**	**Solar Dryer**	**Rotary Drum Thickener**	**Screw Thickener**	**Mesophilic Anaerobic Digestion**	**Centrifuge**	**Rotary Drum Dryer**	**Screw Thickener**	**Gravity Belt Thickener**	**None**	**Centrifuge**	**Belt Dryer with ERS**	**Gravity Belt Thickener**
Facilities & Equipment																					
Process Equipment	n/a	$1,850,000	$0	$1,540,000	$0	$2,440,000	$952,500	$1,410,000	$1,750,000	$9,860,000	$1,930,000	$967,500	$1,410,000	$1,085,000	$8,050,000	$1,950,000	$1,023,000	$0	$1,582,000	$10,356,000	$2,024,000
Structure (where applicable)	n/a	$1,529,000	$0	$750,000	$0	$1,904,000	$515,000	$2,007,120	$3,000,000	$2,824,000	$1,640,000	$515,000	$2,007,120	$0	$4,500,000	$1,640,000	$569,000	$0	$0	$7,500,000	$1,712,000
Demolition (where applicable)	n/a	$40,000	$0	$20,000	$0	$0	$40,000	$50,000	$20,000	$0	$0	$40,000	$50,000	$20,000	$0	$0	$40,000	$0	$20,000	$0	$0
Labor, Construction Equipment & Misc Material	30%	$555,000	$0	$462,000	$0	$732,000	$285,750	$423,000	$525,000	$2,958,000	$579,000	$290,250	$423,000	$325,500	$2,415,000	$585,000	$306,900	$0	$474,600	$3,106,800	$607,200
Allowances																					
Sitework	5%	$198,700	$0	$138,600	$0	$253,800	$89,663	$194,506	$264,750	$782,100	$207,450	$90,638	$194,506	$71,525	$748,250	$208,750	$96,945	$0	$103,830	$1,048,140	$217,160
Piping	15%	$596,100	$0	$415,800	$0	$761,400	$268,988	$583,518	$794,250	$2,346,300	$622,350	$271,913	$583,518	$214,575	$2,244,750	$626,250	$290,835	$0	$311,490	$3,144,420	$651,480
Instrumentation & Electrical	25%	$993,500	$0	$693,000	$0	$1,269,000	$448,313	$972,530	$1,323,750	$3,910,500	$1,037,250	$453,188	$972,530	$357,625	$3,741,250	$1,043,750	$484,725	$0	$519,150	$5,240,700	$1,085,800
CONSTRUCTION SUBTOTAL		**$5,762,300**	**$0**	**$4,019,400**	**$0**	**$7,360,200**	**$2,600,213**	**$5,640,675**	**$7,677,750**	**$22,680,900**	**$6,016,050**	**$2,628,488**	**$5,640,675**	**$2,074,225**	**$21,699,250**	**$6,053,750**	**$2,811,405**	**$0**	**$3,011,070**	**$30,396,060**	**$6,297,640**
Permits	1.0%	$57,623	$0	$40,194	$0	$73,602	$26,002	$56,407	$76,778	$226,809	$60,161	$26,285	$56,407	$20,742	$216,993	$60,538	$28,114	$0	$30,111	$303,961	$62,976
Sales Tax	9.25%	$188,238	$0	$156,695	$0	$248,270	$96,917	$143,468	$178,063	$1,003,255	$196,378	$98,443	$143,468	$110,399	$819,088	$198,413	$104,090	$0	$160,969	$1,053,723	$205,942
Bonds & Insurance																					
Builder's Risk	0.5%	$28,812	$0	$20,097	$0	$36,801	$13,001	$28,203	$38,389	$113,405	$30,080	$13,142	$28,203	$10,371	$108,496	$30,269	$14,057	$0	$15,055	$151,980	$31,488
General Liability	1.0%	$57,623	$0	$40,194	$0	$73,602	$26,002	$56,407	$76,778	$226,809	$60,161	$26,285	$56,407	$20,742	$216,993	$60,538	$28,114	$0	$30,111	$303,961	$62,976
GC Bonds	1.5%	$86,435	$0	$60,291	$0	$110,403	$39,003	$84,610	$115,166	$340,214	$90,241	$39,427	$84,610	$31,113	$325,489	$90,806	$42,171	$0	$45,166	$455,941	$94,465
SUBTOTAL #1		**$6,181,030**	**$0**	**$4,336,871**	**$0**	**$7,902,878**	**$2,801,138**	**$6,009,769**	**$8,162,923**	**$24,591,391**	**$6,453,070**	**$2,832,070**	**$6,009,769**	**$2,267,593**	**$23,386,308**	**$6,494,313**	**$3,027,951**	**$0**	**$3,292,481**	**$32,665,625**	**$6,755,488**
General Conditions/OH&P																					
General Conditions	10%	$618,103	$0	$433,687	$0	$790,288	$280,114	$600,977	$816,292	$2,459,139	$645,307	$283,207	$600,977	$226,759	$2,338,631	$649,431	$302,795	$0	$329,248	$3,266,563	$675,549
Overhead & Profit	10%	$618,103	$0	$433,687	$0	$790,288	$280,114	$600,977	$816,292	$2,459,139	$645,307	$283,207	$600,977	$226,759	$2,338,631	$649,431	$302,795	$0	$329,248	$3,266,563	$675,549
SUBTOTAL #2		**$7,417,235**	**$0**	**$5,204,245**	**$0**	**$9,483,454**	**$3,361,365**	**$7,211,723**	**$9,795,507**	**$29,509,669**	**$7,743,683**	**$3,398,484**	**$7,211,723**	**$2,721,111**	**$28,063,569**	**$7,793,175**	**$3,633,542**	**$0**	**$3,950,978**	**$39,198,750**	**$8,106,585**
Contingency																					
Construction Contingency	30%	$2,225,171	$0	$1,561,274	$0	$2,845,036	$1,008,410	$2,163,517	$2,938,652	$8,852,901	$2,323,105	$1,019,545	$2,163,517	$816,333	$8,419,071	$2,337,953	$1,090,063	$0	$1,185,293	$11,759,625	$2,431,976
SUBTOTAL #3		**$9,642,406**	**$0**	**$6,765,519**	**$0**	**$12,328,490**	**$4,369,775**	**$9,375,240**	**$12,734,159**	**$38,362,570**	**$10,066,788**	**$4,418,029**	**$9,375,240**	**$3,537,445**	**$36,482,640**	**$10,131,128**	**$4,723,604**	**$0**	**$5,136,271**	**$50,958,376**	**$10,538,561**
Escalation	0.0%	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**
CONSTRUCTION TOTAL		**$9,642,000**	**$0**	**$6,766,000**	**$0**	**$12,328,000**	**$4,370,000**	**$9,375,000**	**$12,734,000**	**$38,363,000**	**$10,067,000**	**$4,418,000**	**$9,375,000**	**$3,537,000**	**$36,483,000**	**$10,131,000**	**$4,724,000**	**$0**	**$5,136,000**	**$50,958,000**	**$10,539,000**
Design & Construction Services	15%	$1,446,000	$0	$1,015,000	$0	$1,849,000	$656,000	$1,406,000	$1,910,000	$5,754,000	$1,510,000	$663,000	$1,406,000	$531,000	$5,472,000	$1,520,000	$709,000	$0	$770,000	$7,644,000	$1,581,000
City Project Administration	2.0%	$193,000	$0	$135,000	$0	$247,000	$87,000	$188,000	$255,000	$767,000	$201,000	$88,000	$188,000	$71,000	$730,000	$203,000	$94,000	$0	$103,000	$1,019,000	$211,000
Legal/Finance	3.0%	$289,000	$0	$203,000	$0	$370,000	$131,000	$281,000	$382,000	$1,151,000	$302,000	$133,000	$281,000	$106,000	$1,094,000	$304,000	$142,000	$0	$154,000	$1,529,000	$316,000
CAPITAL COST - EACH TECHNOLOGY		**$11,570,000**	**$0**	**$8,119,000**	**$0**	**$14,794,000**	**$5,244,000**	**$11,250,000**	**$15,281,000**	**$46,035,000**	**$12,080,000**	**$5,302,000**	**$11,250,000**	**$4,245,000**	**$43,779,000**	**$12,158,000**	**$5,669,000**	**$0**	**$6,163,000**	**$61,150,000**	**$12,647,000**
TOTAL OPTION CAPITAL COST		**$34,000,000**					**$90,000,000**					**$77,000,000**					**$86,000,000**				
Total Present Worth		**$57,000,000**					**$105,000,000**					**$94,000,000**					**$102,000,000**				
Overall Annual O&M Cost																					
Initial ADF		**$903,000**					**$453,000**					**$634,000**					**$613,000**				
Intermediate ADF		**$2,206,000**					**$1,244,000**					**$1,516,000**					**$1,474,000**				
Design ADF		**$2,879,000**					**$1,635,000**					**$2,172,000**					**$2,021,000**				
Overall O&M Cost Per Dry Ton Processed																					
Initial ADF		**$381**					**$191**					**$268**					**$259**				
Intermediate ADF		**$438**					**$247**					**$301**					**$292**				
Design ADF		**$476**					**$270**					**$359**					**$334**				

Notes
1. Engineering cost includes preliminary and final design, construction administration, and field services.
2. Capital cost for the new WWTP is based on construction of 4 MGD facility.
3. Life Cycle Cost Parameters
   Interest Rate 5%
   Life Cycle 20 years

ENR Construction Cost Index 8938.30 for January 2011

City of Franklin, Tennessee

Integrated Water Resources Plan

Biosolids Report

CDM Smith

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Abbreviations

ADF	Average daily flow
ADMM	Average day in the maximum month
APLR	Annual pollutant loading rate
AWSAR	Annual whole sludge application rate
BFP	Belt filter press
BTU	British thermal unit
CFM, SCFM	Cubic feet per minute, standard cubic feet per minute
CFR	Code of Federal Regulations
CHP	Combined heat and power
CPLR	Cumulative Pollutant Loading Rate
CY	Cubic yard
DT	Dry ton
DWPC	TDEC's Division of Water Pollution Control
ENRCCI	Engineering News-Record Construction Cost Index
EPA, USEPA	United States Environmental Protection Agency
EQ	Exceptional Quality
FOG	Fats, oils and greases
GBT	Gravity belt thickener
GC	General Contractor or General Conditions
GPM, gpm	Gallons per minute
H_2S	Hydrogen sulfide
HP, hp	Horsepower
kW, kWh	Kilowatt, kilowatt hour
MAD	Mesophilic anaerobic digestion
MBBR	Moving bed biofilm reactor
MBR	Membrane bioreactor
MD	Maximum day
MG	Million gallons
MGD, mgd	Million gallons per day
MMBTU	Million British thermal units
MOR	Monthly operating report
MPN	Most probable number
MSWLF	Municipal solid waste landfill
NPDES	National Pollutant Discharge Elimination System

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O&M	Operation and maintenance
OPCC	Opinion of probable construction cost
PC	Pollutant Concentration
PFD	Process flow diagram
PFRP	Process to further reduce pathogens
POTW	Publicly owned treatment works
PS	Primary sludge or pump station
psi, PSI	Pounds per square inch
PW, NPW	Present worth, net present worth
PSRP	Process to significantly reduce pathogens
RCRA	Resource Conservation and Recovery Act
RDT	Rotary drum thickener
SBR	Sequencing batch reactor
SRT	Solids retention time
TDEC	Tennessee Department of Environment & Conservation
TM	Technical Memorandum
TWAS	Thickened waste activated sludge
TS	Total solids
VFD	Variable frequency drive
VS, VSD, VSR	Volatile solids, volatile solids destruction/reduction
WAS	Waste activated sludge
WEF	Water Environment Federation
WERF	Water Environment Research Foundation
WRF	Water reclamation facility
WT	Wet ton
WWTF, WWTP	Wastewater treatment facility/plant

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Executive Summary

As part of its Integrated Water Resources Plan (IWRP), the City of Franklin is developing a plan to improve the solids handling facilities at its wastewater treatment plant (WWTP). Work performed to date by CDM Smith in support of this IWRP included an assessment of the existing solids handling facilities; a review of historical operating data and operations and maintenance (O&M) costs; an overview of solids treatment requirements based on projected future wastewater flows to the WWTP; an assessment of capital, O&M and life cycle costs of four potential solids process trains; and an evaluation of the relative capital and O&M costs of one or two biosolids facilities – one at the Franklin WWTP, and another at a new WWTP.

At the September 28, 2011, Steering Committee meeting, the City expressed a desire to proceed with further analysis of Option 2, which consists of rotary drum thickening followed by mesophilic anaerobic digestion, screw press dewatering, and solar drying. The City preferred anaerobic digestion because it could achieve Class B treatment, reduce the quantity of biosolids, and potentially produce energy (methane from digester biogas) in support of the City's sustainability goals. Furthermore, solar drying offers relatively low O&M costs and a product that can be beneficially reused. Although the selected option includes rotary drum thickening and screw press dewatering, the City may choose to use alternate thickening and dewatering technologies. CDM Smith is working with the City to organize visits to WWTPs in February 2012 where candidate technologies are in use, and CDM Smith is also working with equipment manufacturers to schedule onsite demonstrations of thickening and dewatering technologies at the Franklin WWTP.

This Conceptual Design Report presents the results of CDM Smith's continued analysis of Option 2: an examination of a phased approach to construction of the proposed solids handling facilities; conceptual process flow diagrams and mass balances; proposed site and solids handling building layouts; and a revised opinion of probable construction cost (OPCC) and O&M costs based on the recommended 16 + 4 + 4 project phasing. The report also provides a brief review of the City's options for beneficial reuse of biosolids at each stage of the treatment process. These options should be explored further in subsequent phases of the work.

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Section 1
Introduction

1.1 Purpose and Scope of Report

As part of its Integrated Water Resources Plan (IWRP), the City of Franklin is developing plans for multiple aspects of its collection system, wastewater treatment plant (WWTP), water treatment plant, distribution system, reclaimed water system, stormwater infrastructure, and ecological and conservation efforts within the City. The solids handling system at the City's WWTP is in need of significant improvements. Currently, the solids produced by the wastewater treatment process are disposed at a landfill located over 100 miles away. The City has expressed interest, as part of the IWRP, in upgrading this system to meet the City's future goals of sustainability and efficiency.

This Conceptual Design Report has been developed to document the work completed thus far; describe the decisions made based on the work produced to date; discuss potential biosolids use/disposal options and associated solids processing requirements; present conceptual design of improvements to the solids processing facilities at the existing Franklin WWTP; and present updated financial analyses, including an opinion of probable construction cost (OPCC), annual operation and maintenance (O&M) costs, and life cycle cost. Further evaluation of this conceptual design and costs will need to be completed prior to final design if the City proceeds with the recommended improvements.

1.2 Completed Biosolids Work to Date

Biosolids Workshop No. 1 was held on February 2, 2011, to determine the preliminary alternatives (individual treatment technologies and potential solids process trains) that would be further analyzed during the alternatives selection process.

During the workshop, a list of criteria was developed to evaluate alternative solids processes. These criteria are listed and described below.

- **Efficiency of operations:** Equipment and operating trains that provide efficient solids processing with little effect upon other treatment processes, consume less energy, and require less maintenance.
- **Decreased energy consumption:** Processes that require reduced quantities of fossil fuel-derived energy, employ high-efficiency equipment, beneficially reuse waste energy, or sequester carbon dioxide tend to have lower carbon footprints.
- **Sustainability:** Processes that will sustain themselves through various disposal options including lower energy consumption, efficient operations, and high quality solids.
- **Diverse portfolio of product use/disposal options:** Class B biosolids use is limited to agricultural land application, but the regulations place fewer restrictions on the use of

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Class A biosolids. Class A biosolids have a larger portfolio of options and may be applied or used in home gardens and lawns.

- **Reliability:** Redundant equipment that will allow for continuous solids processing operations while other equipment is taken out of service.
- **Risk reduction:** Single use/disposal options, like private landfills, determine what type and how much solids they can accept from a municipality. Private landfills can also eliminate biosolids disposal at a moment's notice, leaving the municipality without disposal options. Risk reduction would include the potential to provide more than one end use/disposal alternative.
- **Environmental/public acceptance:** Public buy in of solids processing effects and the resulting minimal impact to the environment are important to the community's achievement of sustainable goals.
- **Odor control:** Because the Franklin WWTP is located near several residential neighborhoods and a school, processes with a lower potential to generate odors are preferred.
- **Automated processes:** A new process that is automated will require less training of staff and less of a learning curve immediately after it is implemented.
- **Class A biosolids:** A quality product with a variety of use/disposal options.
- **Expandability strategy for growth:** A solids train upgrade with a compact layout leaves more space available for future expansion of the facility, thus reducing or eliminating the need for building expansion or additional land acquisition. Also favorable are processes whose solids treatment capacity can be expanded simply, such as by installing additional pieces of equipment.

Workshop participants also developed four solids treatment process trains that would be examined in further detail. These process trains are summarized in **Table 1-1.**

Table 1-1
Biosolids Treatment Alternatives

Process Train	Thickening	Stabilization	Dewatering	Drying	Biosolids Class
Option 1 (Existing)	DAF	None	Belt Filter Press	None	N/A
Option 2	Drum Thickener	Anaerobic	Screw Press	Solar	A
Option 3	Screw Thickener	Anaerobic	Centrifuge	Rotary Drum/ Belt Dryer	A
Option 4	Gravity Belt Thickener	None	Centrifuge	Belt Dryer with ERS	N/A

Technical Memorandum (TM) No. 1 (Evaluation of Existing Equipment & Sludge Production Forecast) described the existing conditions identified during site visits to the WWTP, reviews of the historical monthly operating reports (MORs), a preliminary analysis of the current operation and

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maintenance (O&M) requirements and costs of solids treatment, and an overview of the solids projections based on current conditions and the availability of existing equipment to meet future needs of the WWTP for the duration of the planning period.

In preparing **TM No. 2 (Evaluation of Biosolids Alternatives and Technologies)**, CDM Smith developed preliminary capital, O&M, and present worth costs of the four solids treatment trains discussed during Biosolids Workshop No. 1. Concurrent with this economic analysis, CDM Smith analyzed the relative capital construction and O&M costs to operate one biosolids treatment facility at Franklin WWTP or two biosolids facilities—one at the Franklin WWTP and one at a new WWTP. In **TM No. 2A (Evaluation of One Versus Two Biosolids Facilities),** two alternatives were evaluated during the decision process to determine the feasibility of the one versus two WWTP option. Alternative A discussed no new WWTP construction, and all resultant biosolids would be treated at the existing WWTP. Alternative B discussed construction of a new WWTP that would accept flows between 2 and 6 mgd. Varying degrees of solids processing would occur at the new WWTP and the existing WWTP. From the results of the capital construction and O&M cost analyses, it was determined that we would proceed with Alternative A, with a definite decision being made at the time of construction for the new future WWTP.

TM No. 2 discussed the selected solids treatment technologies and the four potential process trains in further detail. These process trains were evaluated to identify the conceptual sizing of equipment and facilities, as well as to develop planning level capital and O&M costs and life cycle costs. The planning level costs for each process train were compared to the O&M costs for the existing solids treatment process which were developed in TM No. 1. This financial analysis showed that Options 2, 3 and 4 all had similar capital construction and life cycle costs, but Option 2 had the lowest treatment cost per dry ton (DT) of solids treated.

1.3 Review of Sept. 28, 2011, Steering Committee Meeting

Members of the Steering Committee and CDM Smith held a workshop on September 28, 2011, in order to select the biosolids process that would form the basis for this Conceptual Design Report. CDM Smith provided an overview of the work completed and provided an overview of the project thus far. The four biosolids treatment alternatives listed in Table 1-1 were discussed at the meeting.

The City expressed a desire to proceed further with Option 2 after evaluation of this option with respect to the non-cost criteria listed in Section 1.2. The City preferred the option of anaerobic digestion because it could achieve Class B treatment, reduce the amount of biosolids, and potentially produce energy (methane) in support of the City's sustainability goals. The City also wished to further investigate solar drying because of the low O&M costs and because the dried biosolids can offer more beneficial reuse opportunities.

Phased implementation of Option 2 would allow the City to build additional biosolids processing facilities in stages, as the area grows and wastewater flows increase. The phased approach to facility construction is discussed in **Section 2.**

The Steering Committee also wished to explore the option of contracting with a third party to operate the proposed solids treatment facilities. This option is discussed in **Section 3.**

Composting was also discussed as a potential future biosolids option. The composting process has a relatively high odor potential and will require a larger land area. The local compost market should also be explored to ensure that there is adequate demand for the product. Composting is discussed further, along with the potential for energy recovery from the digestion process, in **Section 4.**

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1.4 Report Organization

This report is divided into the following sections:

- **Section 1 – Introduction** provides an overview of the project, a summary of the biosolids work completed to date, a review of the September 28, 2011 Steering Committee meeting, and report organization.
- **Section 2 – Conceptual Facility Design** provides a summary of the selected processes, updated facility design criteria, a phased approach to facility construction, and proposed layouts of the selected equipment on the existing WWTP site. The section also includes a discussion of coordination and integration with the existing and potential WWTP liquid process improvements and solar panel project. Installation lists of the selected equipment for potential site visits and planned on-site technology demonstrations are also included.
- **Section 3 – Updated Economic Analysis** presents the updated OPCC, updated O&M costs, and project life cycle costs for the selected treatment process. The section also includes a brief discussion of options for management of the proposed facilities by an outside contractor.
- **Section 4 – Options for Beneficial Reuse of Biosolids** describes options for beneficial reuse and disposal of biosolids throughout the various parts of the solids treatment process. Also included is a review of methods that could be used to maximize biogas production (and hence energy recovery) in the digesters.

1.5 Existing Reports

Information gathered from the following documents and sources is incorporated in this Conceptual Design Report:

- Technical Memorandum Number 1 – Evaluation of Existing Equipment and Sludge Production Forecast prepared by CDM Smith (2011).
- Technical Memorandum Number 2A – Evaluation of One versus Two Biosolids Facilities prepared by CDM Smith (2011).
- Technical Memorandum Number 2 – Evaluation of Biosolids Alternatives and Technologies prepared by CDM Smith (2011).
- City of Franklin WWTP construction project record drawings.
- Standard Operating Procedures for the Franklin WWTP prepared by Black and Veatch (2002).

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Section 2
Conceptual Facility Design

2.1 Review of the Selected Process

During the September 28, 2011, Steering Committee meeting, the City decided to move forward with the selection of the biosolids process consisting of rotary drum thickening, mesophilic anaerobic stabilization, screw press dewatering, and solar drying. This process meets the goals of the non-cost criteria developed at Biosolids Workshop No. 1 and, based on conceptual cost estimates provided in TM No. 2, is expected to have the lowest O&M cost per DT.

The following sections briefly summarize each part of the solids treatment process. Once the City begins to implement a process, varying levels and degrees of treatment can occur that provide the City with flexibility in reuse/disposal options. Class A biosolids can be used by the general public for agronomic purposes, including fertilizing feed and food crops, lawns, and home gardens. Class B biosolids cannot be handled by the public and can be land applied to agricultural fields. Options for beneficial reuse and/or disposal of the biosolids are discussed in **Section 4.**

Final thickening and dewatering equipment selections have not been made; the selected anaerobic digestion and solar drying processes do not require a specific type of thickening or dewatering technology. Though this report discusses layouts and other considerations for rotary drum thickening and screw press dewatering, the City may elect to use different technologies in place of rotary drum thickening and/or screw press dewatering. CDM Smith is assisting the City in organizing visits to facilities where the candidate technologies are installed, and onsite technology demonstrations are also planned. These efforts are discussed in Section 2.6.

2.1.1 Rotary Drum Thickening

WAS from the secondary clarifiers will be pumped by the existing sludge pumps to a new WAS storage tank, from which it will be fed to the rotary drum thickeners. Rotary drum thickening consists of a polymer feed system and rotating drums covered with a metal mesh screen. Polymer is mixed with dilute sludge in a flocculator, and the conditioned sludge is fed into rotating-screen drums that separate the flocculated solids from the water. Thickened sludge rolls out the end of the drums, while separated water decants through the screens.

For this report, it was assumed that the thickened WAS (TWAS) would have a solids content of about 5 percent. It was also assumed that the new WWTP will include thickening facilities only; the TWAS produced at the new WWTP will be transported to the Franklin WWTP for further treatment. Three rotary drum thickeners (two duty, one standby) would be installed at each treatment plant to meet thickening needs in the design planning period.

2.1.2 Mesophilic Anaerobic Digestion

After the sludge is thickened to approximately 5 percent solids, the TWAS is pumped to the anaerobic digesters, where anaerobic microbes perform a series of biochemical transformations

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that break down the complex organic compounds in wastewater sludges into methane and carbon dioxide. Conventional mesophilic anaerobic digestion (MAD) was selected for this concept-level analysis due to its relatively low energy consumption, the potential for beneficial use of the biogas produced by the digestion process, and well-established operating procedures. The digestion system will consist of two insulated digester tanks, a mixing system, a heating system, and biogas handling equipment. The digester tanks will have a volume of 0.8 MG each and will operate at temperatures between 90°F and 100°F. Biogas produced by the anaerobic microbes will be captured from the headspace of the tank; the biogas can be treated and stored for reuse, or it can be disposed via flare. Energy from the biogas can be used to heat the digester, generate electricity, or serve other heating needs around the WWTP.

The conceptual basis of design for a MAD system was based on a minimum design solids retention time (SRT) of 17 days at maximum month conditions and a volatile solids destruction of 40 percent. A MAD system is an approved process to meet Class B pathogen reduction standards.

Enhancements to the digestion process to increase solids destruction and boost biogas production are discussed in Section 4.

2.1.3 Screw Press Dewatering

Following MAD, the digested biosolids will be pumped to screw presses for dewatering. For this report, it was assumed that the screw presses would dewater the sludge to a solids content of approximately 20 percent. Actual cake solids and polymer requirements cannot be determined until sludge is available for testing.

Feed solids for the screw press, after being conditioned with polymer in a flocculation reactor, are introduced at the bottom end of an inclined trough that contains a perforated bucket surrounding a slowly, continuously rotating screw. The solids move upward along the screw, first losing their free water via gravity drainage, and then as the solids are squeezed against a cone at the top of the unit to force any additional water out. When the cake reaches the top of the unit, the sludge drops out of the unit and onto a conveyor, and is hauled via truck to the solar dryers.

The screw press is compact and completely enclosed. Energy consumption and noise are low due to the low speed and low horsepower of the variable-speed, screw drive motor, and the polymer consumption for dewatering is relatively low. Typical solids capture rate is 95 percent or more. Four screw presses (three duty, one standby) would be required for the design planning period.

2.1.4 Solar Drying

Once the solids are dewatered with the screw press, the City has multiple options for reuse or disposal. This report will discuss potential reuse/disposal options for varying levels of treatment. For ultimate treatment to a Class A biosolids status, the City could construct a solar drying facility to produce Class A biosolids.

In the solar drying system proposed for the City, dewatered cake is transferred to large greenhouse-like structures where it is spread uniformly on the floor to dry in a continuous, year-round operation. Solar radiation evaporates the moisture and machines automatically till the solids, exposing moist solids to the air for further drying. The moisture-laden air is removed from the solar dryers and can be treated for odors before being released to the atmosphere. Overall, the system requires minimal operator attention, is energy efficient, and requires low operations and maintenance costs. The proposed solar dryers would consume approximately 5 acres of drying area for the design planning period.

2.2 Updated Facility Design Criteria

Following the submittal of TM No. 1, CDM Smith's subsequent WWTP process modeling work showed that the anticipated solids production would be higher than the current solids production. New solids loadings were presented in TM No. 2. Recent updates to the WWTP models, including selection of the new WWTP's treatment process, have resulted in slight changes to these loadings. **Table 2-1** lists the revised design parameters and solids loadings, and **Table 2-2** lists the anticipated WAS production to 2040.

Table 2-1
Updated Solids Loading Design Parameters

Parameter		Value	
		Franklin WWTP	New WWTP
Yield (lbs WAS/lb BOD_5 removed)		0.81	0.79
BOD_5 Loading	Influent	212 mg/L (1,768 lbs/MG)	
	Effluent	5 mg/L (42 lbs/MG)	
BOD_5 Removed (lbs/MG)		1,726	
Solids Production (lbs WAS/MG)[1]	Average Day	1,398	1,364
	Max Month	1,818	1,773
WAS Solids Content		0.84%	1.0%

[1] Includes chemical sludge.

Table 2-2
Projected Wastewater Flows and Estimated WAS Production, 2010 to 2040

Year	Total Average Daily Flow (MGD)	Total Average Day Solids Production (lbs/day)	Total Maximum Month Solids Production (lbs/day)
2010	9.4	13,100	17,100
2015	11.8	16,500	21,500
2020	14.3	20,000	25,900
2025	16.7	23,300	30,300
2030	19.1	26,600	34,600
2035	21.6	30,000	39,000
2040	24.0	33,300	43,300

CDM Smith used these updated design criteria to develop a solids treatment train that includes the process equipment listed in **Table 2-3.** This list represents the complete process that will be in place to treat solids in 2040. A stepwise approach to construction of the facility is discussed in detail in Section 2.3.

Table 2-3
Summary of Proposed Solids Treatment Process Design Criteria

Parameter	Value
Thickening	
Thickening technology	Rotary drum thickener
Solids loading rate	700 lbs TS/hour
Feed solids	0.84 to 1.0 percent
Thickened WAS solids	5.0 percent
Number of units	Franklin WWTP: 2 duty, 1 standby New WWTP: 2 duty, 1 standby
Operating schedule	Franklin WWTP: 7 days/week, 16 hours/day New WWTP: 5 days/week, 11 hours/day
Conceptual equipment selection	Andritz 12x3 RST or equal
WAS storage	Franklin WWTP: 420,000 gallons New WWTP: 170,000 gallons
TWAS storage	Franklin WWTP: 70,000 gallons New WWTP: 33,000 gallons
Anaerobic Digestion	
Type of digestion	Mesophilic anaerobic digestion
Number of digesters	Two
Digester tank dimensions	48’ diameter x 60’ sidewater depth
Digester tank type	Prestressed concrete
Minimum solids retention time	20 days at average day conditions; 17 days at maximum month conditions
Volatile solids destruction	40 percent
Biogas produced	86 SCFM (2040)
Digester heating required	1.7 MMBTU/hour
Heat available from biogas engines	1.4 MMBTU/hour
Dewatering	
Dewatering technology	Rotary screw press
Solids loading rate	900 lbs TS/hour
Feed solids	3.7 percent
Dewatered cake solids	20 percent
Number of units	3 duty, 1 standby
Operating schedule	7 days/week, 9 hours/day
Conceptual equipment selection	Huber Technology RoS3 Q800 or equal
Solar Drying	
Number of drying chambers	12
Drying chamber dimensions	264’ L x 42’ W
Dried product output	13.9 WT/day
Dried product solids content	80 percent
Conceptual equipment selection	Krüger Solia, Parkson Thermo-System or equal

2.3 Phased Approach to Improvements

Following submittal of TM No. 2 and selection of Option 2 at the September Steering Committee meeting, CDM Smith examined the possibilities for a phased improvements strategy that would add solids treatment capacity in a stepwise fashion and minimize the initial cost to construct the solids treatment facilities. CDM Smith examined the following phasing options:

- **12 + 4 + 4 + 4 Phasing:** The initial solids treatment facilities are constructed to treat solids produced by a 12 MGD WWTP. Three subsequent phases add process equipment in 4-MGD increments.
- **12 + 6 + 6 Phasing:** In this option, the initial solids treatment facilities are also constructed for 12 MGD, but there are only two subsequent phases in which capacity is added in 6-MGD increments.
- **16 + 4 + 4 Phasing:** Consistent with the proposed WWTP improvements, the initial solids treatment facilities are constructed for a 16-MGD plant. Two subsequent phases add capacity in 4-MGD increments.

After apportioning process equipment and structures to each phase of construction, CDM Smith developed new concept-level OPCCs for the three options. **Tables 2-4, 2-5,** and **2-6** present the results of this analysis.

The tables show a Phase I capital cost of approximately $52 million for the 12 + 4 + 4 + 4 and 12 + 6 + 6 phasing options. Although this capital cost is $14 million less than the approximately $66 million Phase I capital cost of the 16 + 4 + 4 option, current wastewater flows at the plant are already nearing 10 MGD. Therefore, a Phase I solids treatment facility constructed to a capacity of 12 MGD would reach its design capacity quickly – as early as 2015. Design of Phase II improvements would have to begin during Phase I construction.

On the other hand, a Phase I solids treatment facility constructed to a capacity of 16 MGD would not reach its design capacity until about 2023, and Phase II design would begin six years after the Phase I facilities were completed. Furthermore, CDM Smith's work on the Franklin WWTP's liquid treatment process is expected to include a capacity expansion to 16 MGD.

In order to coordinate with the proposed liquid treatment capacity expansion to 16 MGD, and because a 12 MGD Phase I facility would reach its design capacity soon after completion, CDM Smith continued its mass balance calculations and updated the cost analyses based on the 16 + 4 + 4 phasing strategy.

Table 2-4

Summary of 12 + 4 + 4 + 4 Phased Solids Treatment Improvements

Phase	Wastewater Capacity (MGD)			Max Month Solids Production (lbs/day)	Design & Construction Schedule & Costs						
	Total	Franklin WWTP	New WWTP		Franklin WWTP Scope of Work	New WWTP Scope of Work	Estimated Capital Cost	Design Year[1]	Begin Design	Begin Const.	Complete Const.
I	12.0	12.0	0.0	21,800	3 RDTs 1 digester 3 screw presses 6 solar drying chambers Solids handling building	None	$52,000,000	2015	2011	2012	2015
II	16.0	16.0	0.0	29,100	1 digester 2 solar drying chambers	None	$15,000,000	2023	2013	2014	2015
III	20.0	16.0	4.0	36,200	1 screw press 2 solar drying chambers	2 RDTs Thickening building	$15,000,000	2031	2021	2022	2023
IV	24.0	16.0	8.0	43,300	2 solar drying chambers	1 RDT	$8,000,000	2040	2029	2030	2031

[1] Design Year is the calendar year in which plant solids production matches the facility's solids treatment capacity.

Table 2-5

Summary of 12 + 6 + 6 Phased Solids Treatment Improvements

Phase	Wastewater Capacity (MGD)			Max Month Solids Production (lbs/day)	Design & Construction Schedule & Costs						
	Total	Franklin WWTP	New WWTP[2]		Franklin WWTP Scope of Work	New WWTP Scope of Work	Estimated Capital Cost	Design Year[1]	Begin Design	Begin Const.	Complete Const.
I	12.0	12.0	0.0	21,800	3 RDTs 1 digester 3 screw presses 6 solar drying chambers Solids handling building	None	$52,000,000	2015	2011	2012	2015
II	18.0	16.0	2.0	32,600	1 digester 3 solar drying chambers	2 RDTs Thickening building	$25,000,000	2027	2013	2014	2015
III	24.0	16.0	8.0	43,300	1 screw press 3 solar drying chambers	1 RDT	$12,000,000	2040	2025	2026	2027

[1] Design Year is the calendar year in which plant solids production matches the facility's solids treatment capacity.

[2] New WWTP is constructed to a capacity of 4.0 MGD in Phase II, but actual wastewater treated in 2027 is expected to be approximately 2.0 MGD.

Table 2-6

Summary of 16 + 4 + 4 Phased Solids Treatment Improvements

Phase	Wastewater Capacity (MGD)			Max Month Solids Production (lbs/day)	Design & Construction Schedule & Costs						
	Total	Franklin WWTP	New WWTP		Franklin WWTP Scope of Work	New WWTP Scope of Work	Estimated Capital Cost	Design Year[1]	Begin Design	Begin Const.	Complete Const.
I	16.0	16.0	0.0	29,100	3 RDTs 2 digesters 3 screw presses 8 solar drying chambers Solids handling building	None	$66,000,000	2023	2011	2012	2015
II	20.0	16.0	4.0	36,200	2 solar drying chambers	2 RDTs Thickening building	$13,000,000	2031	2021	2022	2023
III	24.0	16.0	8.0	43,300	1 screw press 2 solar drying chambers	1 RDT	$10,000,000	2040	2029	2030	2031

[1] Design Year is the calendar year in which plant solids production matches the facility's solids treatment capacity.

2.4 Process Flow Diagram & Mass Balance

Figures 2-1 through 2-5 present the process flow diagram and mass balance for the proposed solids treatment process. The process is briefly described below.

- **Thickening at Franklin WWTP (Figure 2-1):** WAS produced by the secondary clarifiers is stored in a WAS storage tank and thickened in rotary drum thickeners. These thickeners are expected to produce TWAS with a solids content of approximately 5 percent.
- **Thickening at New WWTP (Figure 2-2):** CDM Smith assumed that the new WWTP would employ the same thickening technology as the Franklin WWTP. After being thickened onsite, TWAS from the new WWTP will be transported via tanker truck to the Franklin WWTP for subsequent treatment and disposal.
- **Anaerobic Digestion (Figure 2-3):** After thickening, the TWAS from both the Franklin WWTP and the new WWTP will be fed to mesophilic anaerobic digesters, where approximately 40 percent of the volatile solids will be destroyed. The biochemical reaction will produce digester gas containing methane, which can be used for power generation and/or digester heating. The Class B digested biosolids will have a solids content of approximately 3.7 percent.
- **Dewatering (Figure 2-4):** The digested biosolids will be dewatered in screw presses to a solids content of approximately 20 percent before being transferred via screw conveyor to trucks, which will haul the dewatered biosolids to the solar dryer.
- **Solar Drying (Figure 2-5):** Solar dryer greenhouses will use the sun's energy to dry the dewatered biosolids to a solids content of about 80 percent. After the drying process is completed, the Class A dried biosolids can be hauled away for use in agriculture and landscaping.

2.5 Proposed Site Plans & Building Layouts

Site plans and building layouts for the Franklin WWTP and the new WWTP were developed from the design criteria summarized in Table 2-3.

2.5.1 Franklin WWTP

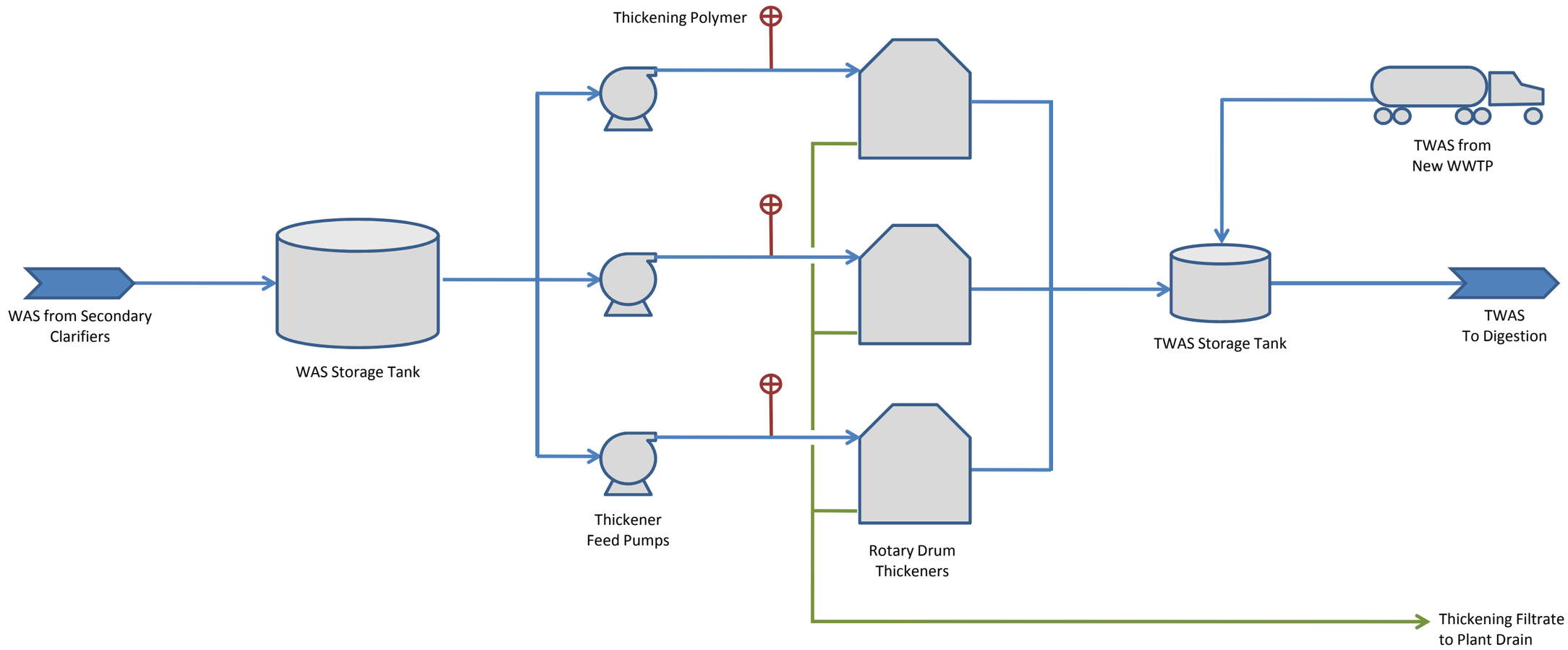
2.5.1.1 Proposed Site Plan

Figure 2-6 presents the proposed site plan for the new solids treatment facilities at the Franklin WWTP. There is sufficient space to construct the proposed solids handling building and ancillary storage tanks in the open area northwest of the existing sludge processing structures.

The proposed solids handling building measures approximately 105 feet by 82 feet, with a truck loading bay occupying the northernmost end of the structure. Two storage tanks, one 50-foot and one 30-foot in diameter, situated southwest of the proposed solids handling building will provide storage for WAS and TWAS, respectively. Two 48-foot diameter digester tanks will be located across the existing gravel drive from the solids handling building. Biogas captured from the digesters will be stored in a tank to be constructed adjacent to the northeast end of the solids handling building. The site will be paved so that trucks transporting dewatered cake to the solar dryers will pull through the truck loading bay and turn around in front of the filter building.

The solar drying chambers will be arranged on either side of a central access drive in the undeveloped area west of the existing sludge storage tanks. Each of the proposed solar dryers measures approximately 264-feet long by 42-feet wide. The eight solar dryers proposed for Phase I construction are illustrated in green,

while the two additional dryers proposed in each subsequent phase are outlined in red (Phase II) and blue (Phase III), respectively. A 25-foot-wide paved access drive forms the perimeter of the facility and will be designed to transition into the existing roadway, with sufficient turning radius for trucks. Additional paved area is provided on the north end of the solar dryer facility for odor control equipment.



	WAS				Thickened WAS				Thickening Filtrate	
	gal/day	lbs TS/day	lbs VS/day	% solids	gal/day	lbs TS/day	lbs VS/day	% solids	gal/day	lbs TS/day
Phase I (2023)	314,000	22,000	14,300	0.8%	50,000	20,900	13,600	5.0%	264,000	1,100
Phase II (2031)	319,000	22,400	14,500	0.8%	51,000	21,300	13,800	5.0%	268,000	1,100
Phase III (2040)	319,000	22,400	14,500	0.8%	51,000	21,300	13,800	5.0%	268,000	1,100

Note: Quantities listed are average daily values (7-day basis).

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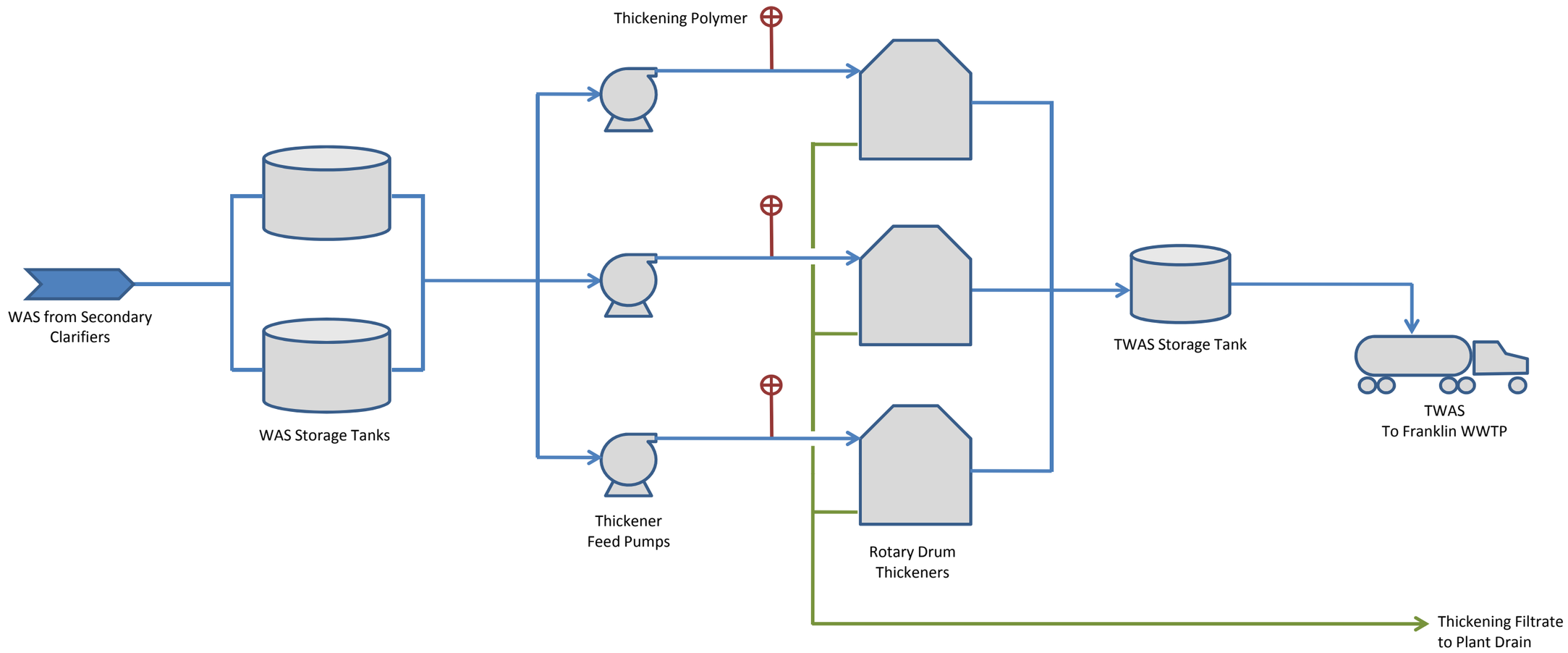
Figure 2-1
Process Flow Diagram & Mass Balance
Rotary Drum Thickening at Franklin WWTP



	WAS				Thickened WAS				Thickening Filtrate	
	gal/day	lbs TS/day	lbs VS/day	% solids	gal/day	lbs TS/day	lbs VS/day	% solids	gal/day	lbs TS/day
Phase I (2023)	0	0	0	1.0%	0	0	0	5.0%	0	0
Phase II (2031)	59,000	4,900	3,300	1.0%	11,000	4,700	3,100	5.0%	48,000	200
Phase III (2040)	131,000	10,900	7,200	1.0%	25,000	10,400	6,800	5.0%	106,000	500

Note: Quantities listed are average daily values (7-day basis).

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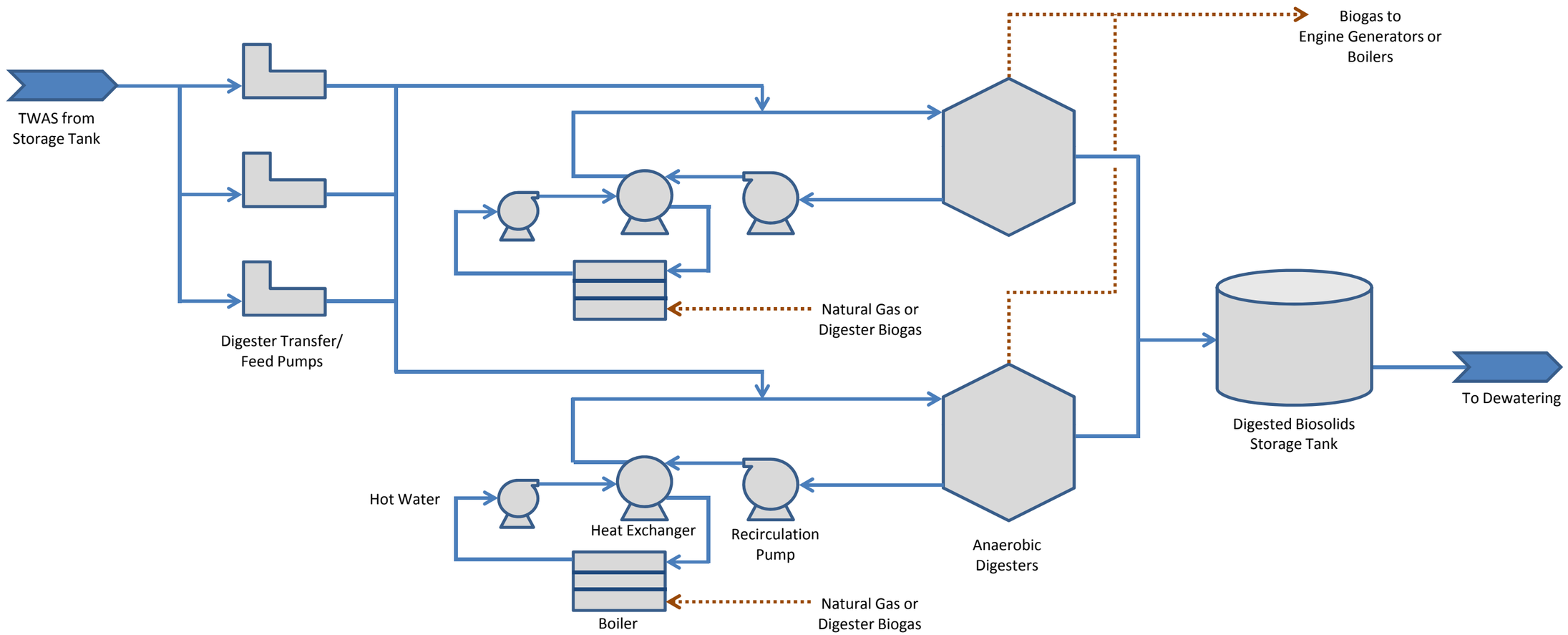
Figure 2-2
Process Flow Diagram & Mass Balance
Thickening at New WWTP



	Digested Biosolids				Digester Biogas Produced		
	gal/day	lbs TS/day	lbs VS/day	% solids	SCFM	Heat, MMBTU/yr	or Electricity, kW
Phase I (2023)	50,000	15,500	8,100	3.7%	57	17,800	227
Phase II (2031)	62,000	19,200	10,100	3.7%	70	22,200	282
Phase III (2040)	76,000	23,400	12,400	3.7%	86	27,100	345

Note: Quantities listed are average daily values (7-day basis).

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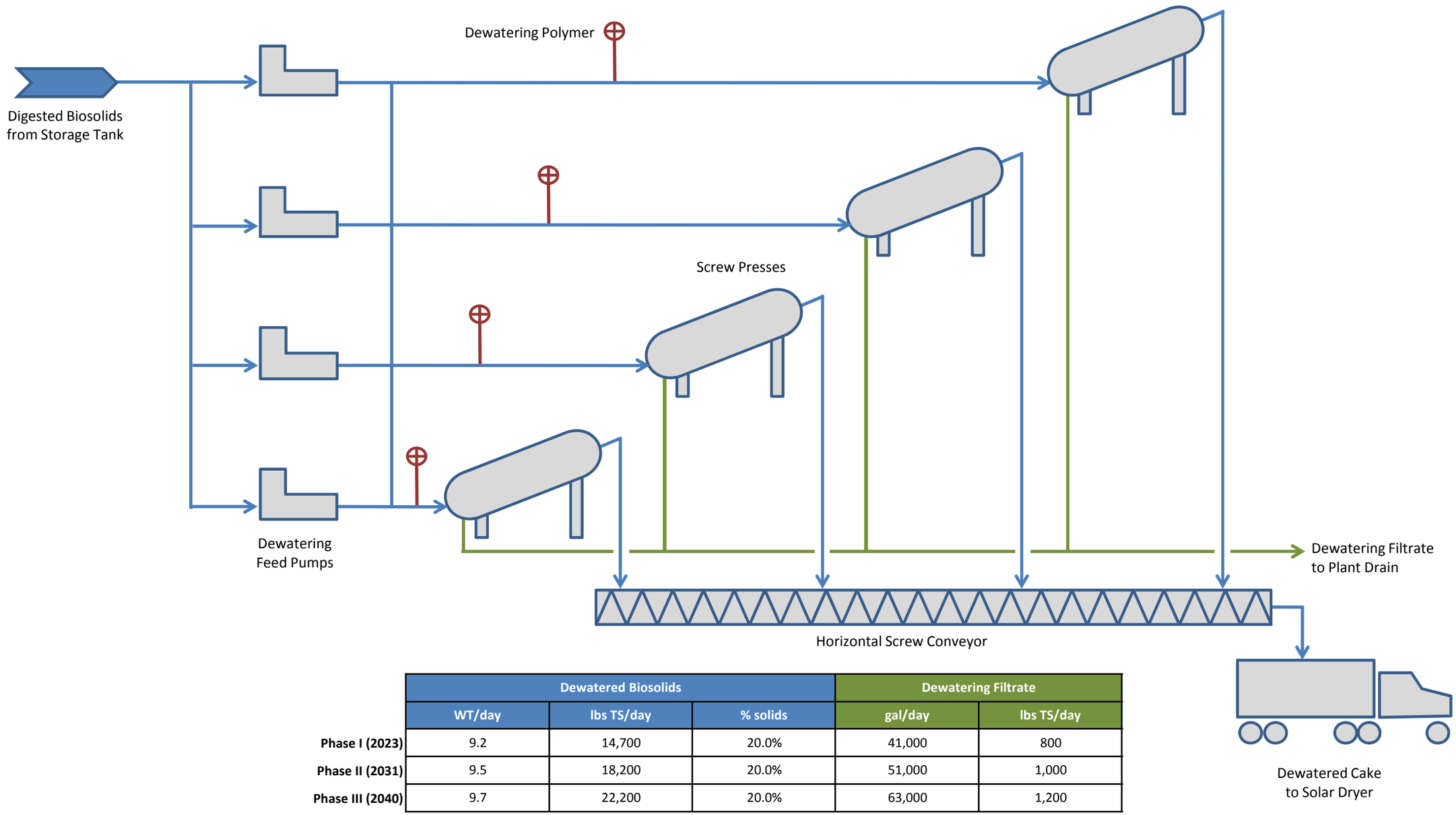
Figure 2-3
Process Flow Diagram & Mass Balance
Anaerobic Digestion at Franklin WWTP



	Dewatered Biosolids			Dewatering Filtrate	
	WT/day	lbs TS/day	% solids	gal/day	lbs TS/day
Phase I (2023)	9.2	14,700	20.0%	41,000	800
Phase II (2031)	9.5	18,200	20.0%	51,000	1,000
Phase III (2040)	9.7	22,200	20.0%	63,000	1,200

Note: Quantities listed are average daily values (7-day basis).

Figure 2-4
Process Flow Diagram & Mass Balance
Screw Press Dewatering at Franklin WWTP

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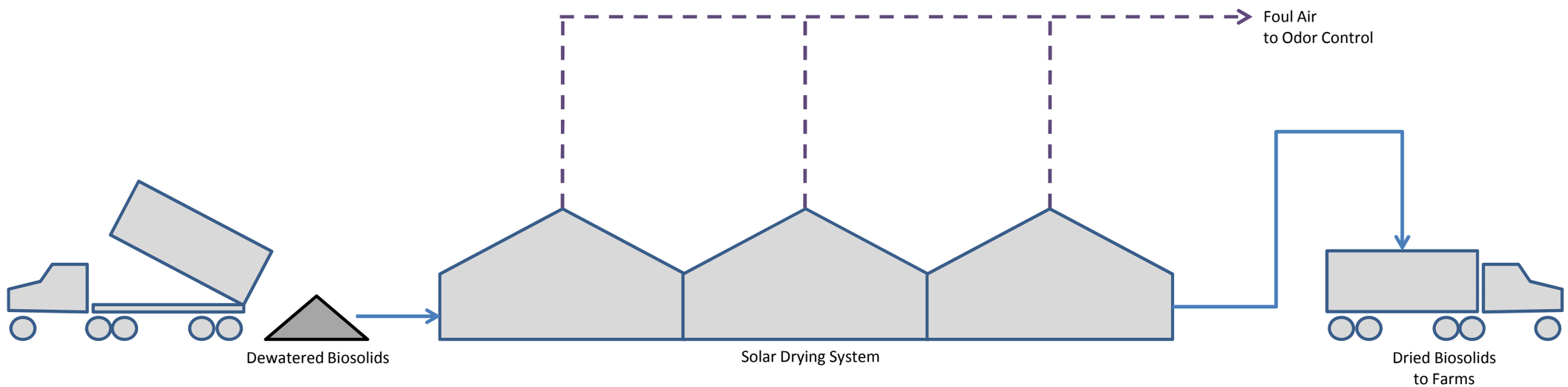

	Dried Biosolids		
	WT/day	lbs TS/day	% solids
Phase I (2023)	9.2	14,700	80.0%
Phase II (2031)	11.4	18,200	80.0%
Phase III (2040)	13.9	22,200	80.0%

Note: Quantities listed are average daily values (7-day basis).

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Figure 2-5
Process Flow Diagram & Mass Balance
Solar Drying at Franklin WWTP

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CITY OF FRANKLIN, TN
CONCEPTUAL DESIGN REPORT

FIGURE 2-6
PROPOSED SITE PLAN
JANUARY 2012

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2.5.1.2 Proposed Solids Handling Building Layout

It is CDM Smith's understanding that the thickening and dewatering equipment selection has not been finalized; these building layouts were created using the dimensions and service clearances of the largest pieces of thickening and dewatering equipment that could be installed. A building layout based on the rotary drum thickener and screw press, both part of the selected Option 2 treatment process, is likely to result in a slightly smaller footprint than what is presented here.

Figures 2-7 and **2-8** present the first and second floors of the proposed solids handling building. The solids handling building is to be a two-story structure with a 25-foot wide truck loading bay at the northeastern end. In addition to laboratory space and control and electrical rooms, the first floor includes space for the feed pumps and polymer systems for the dewatering and thickening equipment, which is located on the second floor. The digester equipment will also be located on the first floor. Each of the two digesters will have a heat exchanger and set of two mixing pumps. The two digesters require a duty and standby set of transfer feed pumps and recirculation pumps which will be situated adjacent to the heat exchangers and mixing pumps.

The second floor, illustrated in Figure 2-8, houses the proposed dewatering and thickening units. If the City selects screw thickeners or screw presses, each of these machines require a substantial amount of service clearance at one end; consequently, 23 feet of clearance is provided between the dewatering and thickening units. If both screw presses and screw thickeners are selected, they would share this service clearance area. Both the thickening and dewatering units would be fed by the pumps and polymer feed equipment located on the first floor. The thickening units would be fed sludge from the WAS storage tank, and the dewatering units would be fed from the TWAS storage tank. Thickened sludge would be discharged to the TWAS storage tank, and dewatered sludge would discharge to horizontal screw conveyors, which would transfer the cake to trucks in the loading bay below. All of the proposed thickening equipment and all but one dewatering machine will be installed during Phase I. A fourth dewatering unit will be installed in Phase III.

2.5.2 New WWTP

The design memorandum describing the proposed new WWTP includes the thickening building. The building will measure approximately 70 feet by 45 feet. The thickening machines will be located on the ground floor, and there will be a basement level where the polymer systems, sludge feed pumps, and TWAS pumps will be installed. The basement level will also feature an integral, rectangular storage tank for TWAS; the thickened sludge will drop into the tank from the machines above. The TWAS will be mixed with pumps and discharged to tanker trucks parked next to the building.

2.5.3 Integration of Existing & New Facilities

During Phase I construction, new yard piping will be constructed to connect the discharge of the existing WAS pumps to the new WAS storage tank. Because the Franklin WWTP's existing solids treatment facilities must remain in service until the new facilities are online, the existing facilities should be demolished, either as the final part of the Phase I construction contract or as part of a separate demolition contract, after the Phase I facilities are completed and capable of treating solids.

A more detailed analysis of maintenance of plant operations will follow in later phases of design.

2.5.4 Coordination With Other Work

The solar drying facility shown in Figure 2-6 should be located as far north as possible in order to avoid interference with the planned future solar panel array being developed separately by Merville & Howe

Engineering. Work related to the solar panel array is not a part of the scope of this Conceptual Design Report. Design activities at the Franklin WWTP should be closely coordinated with this and other projects.

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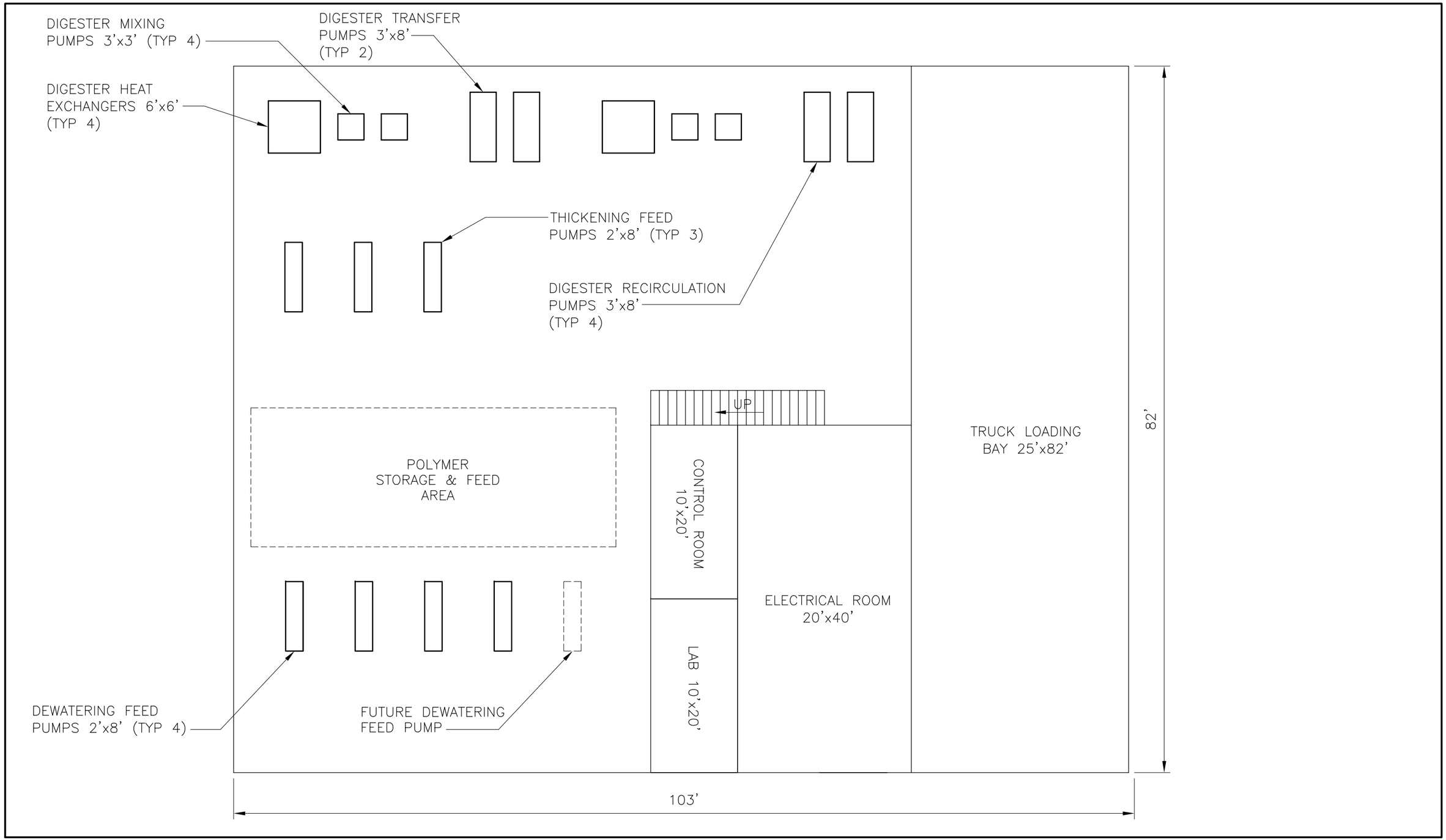

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Figure No. 2-7
Proposed Solids Handling Building
First Floor Plan
January 2012

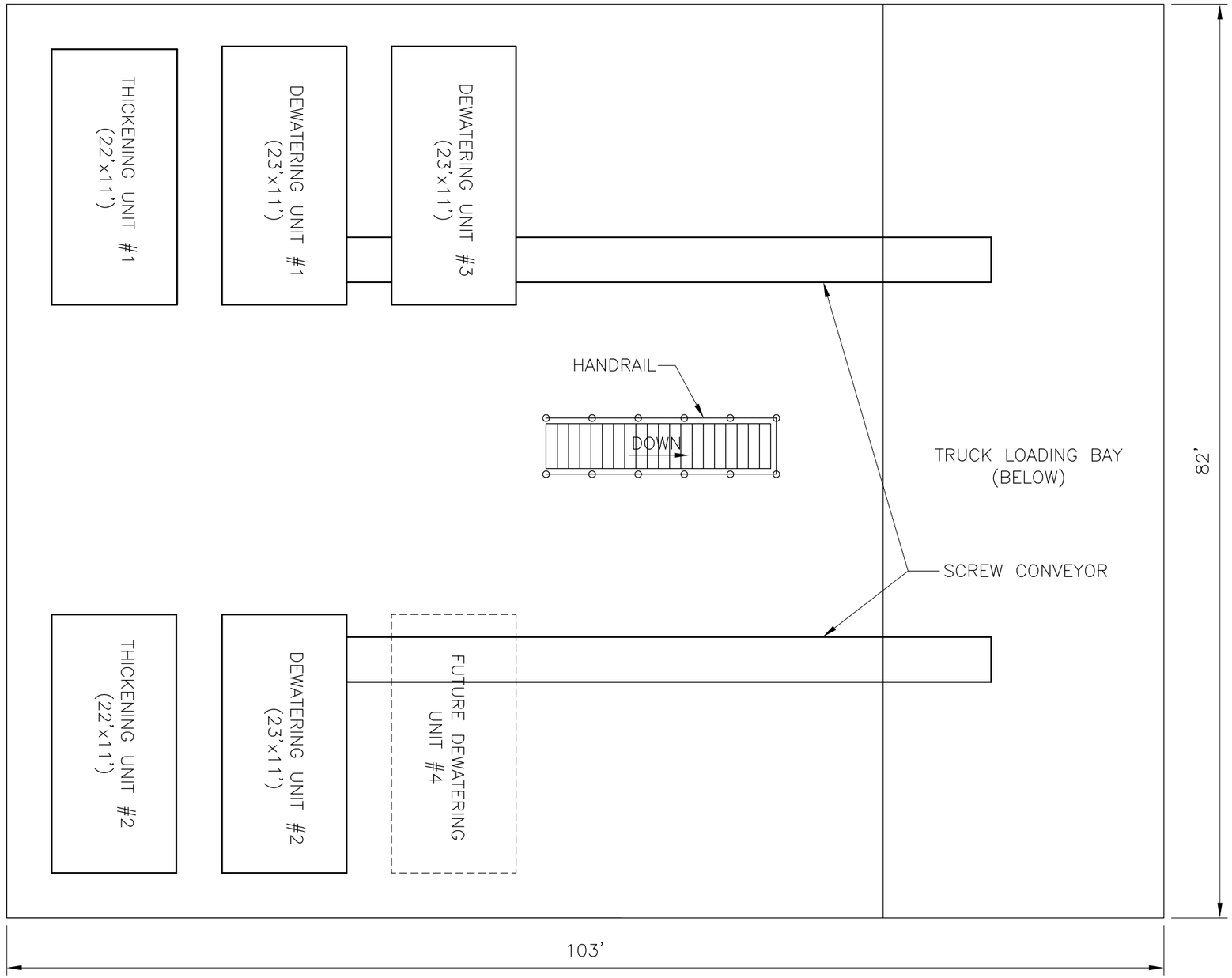

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Figure No. 2-8
Proposed Solids Handling Building
Second Floor Plan
January 2012

2.6 Process Equipment Selections

2.6.1 Visits to Regional Installations

Though solar drying is used extensively throughout Europe, there are only few solar dryer installations in the United States. One installation is located in Okeechobee, Florida. This 3-MGD facility receives and processes trucked-in sludges from nearby municipal WWTPs. Three Parkson Thermo-System solar dryer chambers dry the sludge for about 130 days to a solids content of approximately 80 percent. The dried product is considered Class A biosolids and is land applied to city land and agricultural fields. The Okeechobee staff samples every truckload of dried biosolids removed from the solar dryer for land application to ensure that the biosolids meet Class A pathogen reduction requirements.

In addition to viewing the solar dryer, City staff that visit Okeechobee can also examine the facility's anaerobic digester, which produces a digested sludge that is subsequently dewatered five days per week and loaded into the solar drying chambers twice per day. A visit to the Okeechobee solar dryer and digester installation would also allow City staff to also visit a nearby Andritz rotary drum thickening installation in Indian River County.

Table 2-7 summarizes the equipment installations at and near Okeechobee. The City and CDM Smith intend to visit these facilities in February 2012. Other site visits may be planned in the future.

Table 2-7
Summary of Upcoming Site Visits

Facility	Process Equipment	Comments
Okeechobee, FL	Anaerobic Digester	Digested sludge sent to dewatering 5 days/wk
	Solar Dryer	Parkson Thermo-System™ Solar Dryer Constructed in 2007 Three drying chambers - 42' x 200'; drying area = 2,800yd² Final solids target = 75% Currently 2 of 3 chambers used Maintain < 135°F in chambers 130 days to process one batch Produces Class A biosolids
Indian River County, FL	Rotary Drum Thickener	One Andritz 12 x 3 RST Installed in 2004

2.6.2 On-Site Pilot Demonstrations

The major thickening and dewatering equipment vendors have mobile demonstration trailers available for on-site pilot testing. Although it will not be possible to test the sludges that the projected solids facility is expected to handle, an on-site demonstration would enable City staff to gain valuable hands-on experience in the operation and maintenance of the equipment. CDM Smith has assisted numerous other clients in coordinating, pricing, and scheduling on-site demonstrations and has requested a demonstration of the Huber Technology screw press that is proposed for dewatering at the Franklin WWTP's new solids treatment facilities. Huber's demonstration trailer will be touring the southeastern United States from March to May 2012, and CDM Smith is currently working with Huber and City staff to schedule a demonstration at the Franklin WWTP.

2.7 Sidestream Treatment Options

Sidestreams discharged from the proposed solids treatment processes consist primarily of water and suspended solids removed from the thickening and dewatering processes. Like other waste streams generated in a wastewater treatment plant, these sidestreams are recycled to the head of the plant for treatment. Because these sidestreams are known to contain large amounts of nitrogen and phosphorus, they can have a significant impact on the size and cost of the liquid treatment process.

Two processes that have been developed to reduce nutrient loads are discussed briefly in the following sections. These and other emerging technologies may be explored in subsequent design studies. A present worth analysis during preliminary design is recommended to assist in the decision on whether to include these treatment technologies in the WWTP improvements.

2.7.1 Anaerobic Ammonium Oxidation (ANAMMOX)

Practiced in wastewater treatment since the late 1980s, the ANAMMOX process uses a naturally-occurring biochemical reaction to remove ammonium from a high-strength wastewater sidestream. The process consists of two steps. First, ammonia oxidizing bacteria are used to convert a portion of the ammonium in the wastewater to nitrite in a process called nitritation or partial nitrification. In the second step, the ANAMMOX bacteria convert the nitrite and ammonium into nitrogen gas.

Because the ANAMMOX bacteria grow very slowly, the process is usually contained in a sequencing batch reactor (SBR) or a moving bed biofilm reactor (MBBR). The nitrite and dissolved oxygen concentrations must be carefully monitored in order to avoid inhibiting the process. Full scale ANAMMOX treatment processes have been constructed in Europe, but there are few installations in the United States.

2.7.2 Nutrient Recovery (Ostara PEARL® Process)

The presence of a dewatering process presents the opportunity to capture nutrients from the dewatering filtrate stream for beneficial reuse. The Pearl® process, developed by Ostara Nutrient Recovery Technologies (Ostara), recovers nitrogen and phosphorus from the sidestream through application of magnesium chloride and caustic soda to the sidestream, followed by precipitation of struvite pellets in a patented fluidized bed reactor. After the struvite pellets are removed from the reactor, they are dried and packaged for sale as Crystal Green® fertilizer. Full-scale Pearl® processes have been installed in Pennsylvania, Oregon, and Virginia.

Section 3
Updated Economic Analysis

3.1 Updated Opinion of Probable Construction Cost

A preliminary opinion of probable construction cost (OPCC), along with assumptions made during the cost estimating process, was presented in Section 5 of TM No. 2. The following sections describe the major changes made to the OPCC based on requests from the Steering Committee and other stakeholder comments.

3.1.1 Phasing of Improvements

As part of an initial screening process, the OPCCs in TM No. 2 had been based on the construction of complete solids treatment trains for the design year of 2040. For this Conceptual Design Report, construction of the selected process was divided into phases in order to minimize the initial cost of construction and coordinate with improvements to the liquid treatment facilities. The phasing method and selection of the 16 + 4 + 4 strategy were explained in Section 2.

3.1.2 Modification of Contingency Calculation

At the September Steering Committee meeting, it was commented that the 30 percent contingency should be removed from the process equipment costs, as budgetary proposals given at the planning stages of a project tend to already include a significant contingency added by the vendors.

CDM Smith's cost estimating division considers it good practice to carry a contingency on process equipment to account for factors such as design changes, equipment obsolescence, and materials cost escalation. The OPCC was therefore recalculated with a 10 percent contingency applied to the process equipment costs.

3.1.3 Revised OPCC & Assumptions

Table 3-1 presents the revised OPCC for the 16 + 4 + 4 phased improvements. The updated assumptions and markups applied to this OPCC are listed below and in **Table 3-2.**

- This is a planning level OPCC only, based on a three-phase construction of a solids treatment train for the design years of 2023, 2031, and 2040, respectively.
- It was assumed that Franklin WWTP and the new WWTP would use identical thickening machines.
- Thickening and dewatering equipment costs are based on the rotary drum thickener and screw press; however, these costs may change during preliminary design to reflect the City's final equipment selections.

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- As mentioned in Section 2.5.1.2, the buildings housing dewatering and thickening equipment were sized according to the dimensions and service clearances of the largest pieces of thickening and dewatering equipment that could be installed. The cost of the building may change during preliminary design after the City finalizes its thickening and dewatering equipment selections.
- The capital cost of the new WWTP solids handling facilities includes procurement of two new tanker trucks, as well as construction of truck loading and receiving facilities at the new WWTP and Franklin WWTP, respectively. The tanker trucks will be loaded with TWAS at the new WWTP and transport it to the Franklin WWTP for further treatment.
- The capital cost includes a combined heat and power (CHP) system consisting of a 250-kW microturbine suitable for outdoor installation adjacent to the digester gas storage facility.
- Rock excavation is not included.
- Only nominal dewatering is needed for new structures.
- No contaminated soil or hazardous materials will be encountered.
- Construction costs are based on a normal 40-hour work week.
- Construction costs are based on 2011 dollars (no escalation applied).

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Table 3-1
Revised Opinion of Probable Construction Cost for Solids Treatment Improvements - 16 + 4 + 4 Phasing

		Phase I (16 MGD)					Phase II (+4 MGD)					Phase III (+4 MGD)				
		Thickening	Stabilization	Dewatering	Drying	New WWTP	Thickening	Stabilization	Dewatering	Drying	New WWTP	Thickening	Stabilization	Dewatering	Drying	New WWTP
Construction Cost Component	**% of Cost**	**Rotary Drum Thickener**	**Mesophilic Anaerobic Digestion**	**Screw Press**	**Solar Dryer**	**Rotary Drum Thickener**	**Rotary Drum Thickener**	**Mesophilic Anaerobic Digestion**	**Screw Press**	**Solar Dryer**	**Rotary Drum Thickener**	**Rotary Drum Thickener**	**Mesophilic Anaerobic Digestion**	**Screw Press**	**Solar Dryer**	**Rotary Drum Thickener**
Facilities & Equipment																
Process Equipment	n/a	$952,500	$2,710,000	$1,430,000	$5,669,091	$0	$0	$0	$0	$1,167,273	$765,000	$0	$350,000	$320,000	$1,167,273	$312,500
Structure (where applicable)	n/a	$665,000	$2,309,056	$5,400,000	$1,056,000	$0	$0	$0	$0	$264,000	$1,766,000	$0	$0	$0	$264,000	$170,000
Demolition (where applicable)	n/a	$40,000	$50,000	$20,000	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0	$0
Labor, Construction Equipment & Misc Material	30%	$285,750	$813,000	$429,000	$1,700,727	$0	$0	$0	$0	$350,182	$229,500	$0	$105,000	$96,000	$350,182	$93,750
Allowances																
Sitework	5%	$97,163	$294,103	$363,950	$421,291	$0	$0	$0	$0	$89,073	$138,025	$0	$22,750	$20,800	$89,073	$28,813
Piping	15%	$291,488	$882,308	$1,091,850	$1,263,873	$0	$0	$0	$0	$267,218	$414,075	$0	$68,250	$62,400	$267,218	$86,438
Instrumentation & Electrical	25%	$485,813	$1,470,514	$1,819,750	$2,106,455	$0	$0	$0	$0	$445,364	$690,125	$0	$113,750	$104,000	$445,364	$144,063
CONSTRUCTION SUBTOTAL		**$2,817,713**	**$8,528,981**	**$10,554,550**	**$12,217,436**	**$0**	**$0**	**$0**	**$0**	**$2,583,109**	**$4,002,725**	**$0**	**$659,750**	**$603,200**	**$2,583,109**	**$835,563**
Permits	1.0%	$28,177	$85,290	$105,546	$122,174	$0	$0	$0	$0	$25,831	$40,027	$0	$6,598	$6,032	$25,831	$8,356
Sales Tax	9.25%	$96,917	$275,743	$145,503	$576,830	$0	$0	$0	$0	$118,770	$77,839	$0	$35,613	$32,560	$118,770	$31,797
Bonds & Insurance																
Builder's Risk	0.5%	$14,089	$42,645	$52,773	$61,087	$0	$0	$0	$0	$12,916	$20,014	$0	$3,299	$3,016	$12,916	$4,178
General Liability	1.0%	$28,177	$85,290	$105,546	$122,174	$0	$0	$0	$0	$25,831	$40,027	$0	$6,598	$6,032	$25,831	$8,356
GC Bonds	1.5%	$42,266	$127,935	$158,318	$183,262	$0	$0	$0	$0	$38,747	$60,041	$0	$9,896	$9,048	$38,747	$12,533
SUBTOTAL #1		**$3,027,338**	**$9,145,882**	**$11,122,235**	**$13,282,964**	**$0**	**$0**	**$0**	**$0**	**$2,805,203**	**$4,240,673**	**$0**	**$721,753**	**$659,888**	**$2,805,203**	**$900,782**
General Conditions/OH&P																
General Conditions	10%	$302,734	$914,588	$1,112,223	$1,328,296	$0	$0	$0	$0	$280,520	$424,067	$0	$72,175	$65,989	$280,520	$90,078
Overhead & Profit	10%	$302,734	$914,588	$1,112,223	$1,328,296	$0	$0	$0	$0	$280,520	$424,067	$0	$72,175	$65,989	$280,520	$90,078
SUBTOTAL #2		**$3,632,805**	**$10,975,059**	**$13,346,681**	**$15,939,557**	**$0**	**$0**	**$0**	**$0**	**$3,366,244**	**$5,088,807**	**$0**	**$866,103**	**$791,866**	**$3,366,244**	**$1,080,938**
Contingency																
Construction Contingency	30%	$899,342	$2,750,518	$3,718,004	$3,648,049	$0	$0	$0	$0	$776,419	$1,373,642	$0	$189,831	$173,560	$776,419	$261,781
SUBTOTAL #3		**$4,532,147**	**$13,725,576**	**$17,064,686**	**$19,587,605**	**$0**	**$0**	**$0**	**$0**	**$4,142,663**	**$6,462,449**	**$0**	**$1,055,934**	**$965,425**	**$4,142,663**	**$1,342,720**
Escalation	0.0%	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**
CONSTRUCTION TOTAL		**$4,532,000**	**$13,726,000**	**$17,065,000**	**$19,588,000**	**$0**	**$0**	**$0**	**$0**	**$4,143,000**	**$6,462,000**	**$0**	**$1,056,000**	**$965,000**	**$4,143,000**	**$1,343,000**
Design & Construction Services	15%	$680,000	$2,059,000	$2,560,000	$2,938,000	$0	$0	$0	$0	$621,000	$969,000	$0	$158,000	$145,000	$621,000	$201,000
City Project Administration	2.0%	$91,000	$275,000	$341,000	$392,000	$0	$0	$0	$0	$83,000	$129,000	$0	$21,000	$19,000	$83,000	$27,000
Legal/Finance	3.0%	$136,000	$412,000	$512,000	$588,000	$0	$0	$0	$0	$124,000	$194,000	$0	$32,000	$29,000	$124,000	$40,000
Allowances		**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$0**	**$175,000**	**$0**	**$0**	**$0**	**$0**	**$0**
CAPITAL COST - EACH TECHNOLOGY		**$5,439,000**	**$16,472,000**	**$20,478,000**	**$23,506,000**	**$0**	**$0**	**$0**	**$0**	**$4,971,000**	**$7,929,000**	**$0**	**$1,267,000**	**$1,158,000**	**$4,971,000**	**$1,611,000**
EACH PHASE CAPITAL COST		**$66,000,000**					**$13,000,000**					**$10,000,000**				
YEAR INCURRED		**2012**					**2022**					**2030**				

Notes
1. Engineering cost includes preliminary and final design, construction administration, and field services.
2. Capital cost for the new WWTP is based on construction of 4 MGD facility.
3. Life Cycle Cost Parameters
   Interest Rate 5%
   Life Cycle 20 years
4. 10% contingency applied to process equipment.

ENR Construction Cost Index 8938.30 for January 2011

Table 3-2
Updated Planning Level Capital Cost Assumptions

Cost Item	Value
Allowances Applied to Estimated Construction Cost	
Labor & Material	30 percent
Piping	15 percent
Instrumentation & Electrical	25 percent
Markups Applied to Estimated Contractor's Cost	
Permits	1.0 percent
Sales Tax	9.25 percent[1]
Builder's Risk	0.5 percent
General Liability	1.0 percent
GC Bonds	1.5 percent
General Conditions	10 percent
Overhead & Profit	10 percent
Markups Applied to Estimated Construction Cost	
Construction Contingency	30 percent[2]
Design & Construction Services	15 percent
City Project Admin.	2.0 percent
Legal/Finance	3.0 percent

[1] Applied to equipment and material costs.
[2] 10 percent contingency is applied to process equipment costs.

3.2 Updated Operation & Maintenance Costs

CDM Smith refined the O&M costs to incorporate comments received at the Steering Committee meeting. The goal of these refinements was to improve the accuracy of the O&M costs previously presented in TM No. 2.

In addition to refining the existing set of O&M costs for a facility operated by City staff, CDM Smith has begun discussions with a third party to discuss the relative costs of a contract operation. These developments are discussed in the following sections.

3.2.1 O&M Costs of Owner Operated Facility

In addition to modifying the OPCC, CDM Smith refined the O&M costs to include the following items:

- **Unit cost escalation.** CDM Smith applied an inflation rate of 3 percent per year to unit costs including labor, chemicals, utilities, landfill tipping fees, and truck maintenance and insurance.

- **New tipping fee for 2011.** The tipping fee was recently raised to $24.40 per WT. This tipping fee replaces the previously used tipping fee of $22.24 per WT.

- **CHP and digester heating.** A combined heat and power (CHP) system is discussed in Section 4. The updated O&M costs assume that a CHP system will be installed to generate electricity from the digester biogas, and the electricity generated by the CHP system will be used to power solids handling equipment. The waste heat recovered from the microturbine will be used to heat the digesters. Supplemental heating is required, as there is not enough waste heat to meet the

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digesters' heating demands. The cost of natural gas for supplemental heating was included in the O&M costs.

Planning level annual O&M cost calculations were based on the parameters and assumptions listed in **Table 3-3.** The labor required to operate and maintain the solids treatment train was based on the relative complexity of each technology and its anticipated operating schedule. **Table 3-4** presents the assumptions that were used to calculate the 20-year life cycle cost.

Table 3-3
Updated O&M Cost Assumptions

Cost Item	Value
Labor	
Fully Loaded Labor Rate	$35.08 per hour
Utilities	
Electricity	$0.06/kWh
Natural Gas	$10/MMBtu
Chemicals	
Thickening/Dewatering Polymers	$1.60/lb delivered
Sodium Hypochlorite Solution (odor control)	$1.57/gal delivered
Caustic Soda (odor control)	$1.54/gal delivered
Muriatic Acid (odor control)	$2.38/gal delivered
Biosolids Hauling	
Solids Disposal Location	Local Farms
Truck Capacity	20 CY
Landfill Tipping Fee	$24.40/WT
Diesel Fuel Cost	$3.88/gallon
Truck Fuel Economy	5 miles/gallon
Average Round Trip Distance	100 miles
Driver's Labor Per Load	6 hours
Fleet Maintenance	$6,480 per year
Insurance	$440 per year

Table 3-4
Life Cycle Cost Assumptions

Cost Item	Value
Interest Rate	5 percent
Period	20 years

3.2.2 O&M Costs of Contract Operations

Some municipalities choose to contract with an operator to provide biosolids services that include operation and maintenance of the municipality's solids handling facilities. Costs of chemicals, utilities, and final disposal can be included in these contracts. The scope of the contract operator's services and the structure of the contract can vary greatly from contract to contract; the contract structure and overall cost

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are influenced by factors including the owner's preferences and budgetary goals, as well as the quantity of biosolids that is to be treated.

CDM has contacted Synagro Technologies to begin discussion of the possibility of contract operations at the proposed Franklin WWTP solids handling facility. This conversation is ongoing, and the results of this discussion will be shared with the City at a later date.

3.3 Updated Project Life Cycle Cost

The updated 20-year life cycle cost of the project is **$86,000,000. Table 3-5** presents the annual capital and O&M costs from Year 1 to Year 20.

Table 3-5
Updated Project Life Cycle Cost – 16 + 4 + 4 Phasing

Year	Capital Cost	PW of Capital Cost	O&M Cost	PW of O&M Cost	O&M Cost per DT
2011			$429,000	$429,000	$170
2012	$66,000,000	$62,857,143	$451,000	$429,524	$170
2013			$491,000	$445,351	$177
2014			$515,000	$444,876	$178
2015			$540,000	$444,259	$179
2016			$566,000	$443,476	$180
2017			$613,000	$457,430	$187
2018			$641,000	$455,547	$189
2019			$680,000	$460,251	$193
2020			$711,000	$458,317	$195
2021			$766,000	$470,258	$203
2022	$13,000,000	$7,600,831	$801,000	$468,328	$206
2023			$837,000	$466,073	$209
2024			$956,000	$506,987	$231
2025			$1,037,000	$523,755	$244
2026			$1,097,000	$527,676	$251
2027			$1,160,000	$531,409	$258
2028			$1,225,000	$534,463	$265
2029			$1,293,000	$537,268	$273
2030	$10,000,000	$3,957,340	$1,393,000	$551,257	$286
20- Year Present Worth: $86,000,000					

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Section 4
Options for Beneficial Reuse of Biosolids

4.1 Benefits of Stabilization & Drying

The many benefits of stabilization and drying of WWTP sludges were presented to the Steering Committee during the September 28, 2011, meeting. Stabilized sludge and dried biosolids satisfy the City's selection criteria for a biosolids process as defined in Section 1.2. Two key benefits of adding these processes are the potential for beneficial reuse and the reduction in the quantity of solids to be disposed.

Sludge produced by the WWTP must first be converted into biosolids in order to be reused. Stabilization processes such as anaerobic digestion can accomplish this treatment. In addition to producing a useful biosolids product, the digestion process reduces solids and produces biogas. The Class B biosolids produced by a stabilization process can be applied to agricultural land. Dried biosolids meeting Class A pathogen reduction requirements can be handled by the public and applied to home lawns or gardens. Drying of the biosolids reduces its moisture content and achieves the greatest volume reduction, thus minimizing storage, handling, and transportation costs.

This section discusses the reuse potential of biosolids at different stages along the solids treatment train and presents an overview of process enhancements to maximize the production of energy from the biosolids.

4.2 Biosolids Disposal & Beneficial Reuse Options

Figure 4-1 summarizes the biosolids reuse and/or disposal options available to the City at each stage of the solids treatment process. Some options, such as landfill disposal, are included but are not considered desirable. Other options are better suited to the City's goals. Consideration of these options can assist the City in deciding when and how to expand the Franklin WWTP's solids treatment capabilities.

4.2.1 Mesophilic Anaerobic Digestion

Anaerobic digestion is the biological process that stabilizes organic matter in the absence of oxygen. During this process, biodegradable organic matter is converted to water and biogas containing methane (CH_4) and carbon dioxide (CO_2). The biogas is suitable for use as an energy source; methods to enhance biogas production are discussed later in this section.

In addition to reducing the quantity of the solids, anaerobic digestion can produce a stabilized product meeting regulatory requirements of pathogen reduction and vector attraction reduction. Digested biosolids are often less odorous and attract fewer vectors (such as rodents, flies, mosquitoes, or other organisms capable of transporting infectious agents) than raw sludge. Organic matter, nitrogen, and phosphorus in the digested biosolids can be used as a soil amendment and fertilizer.

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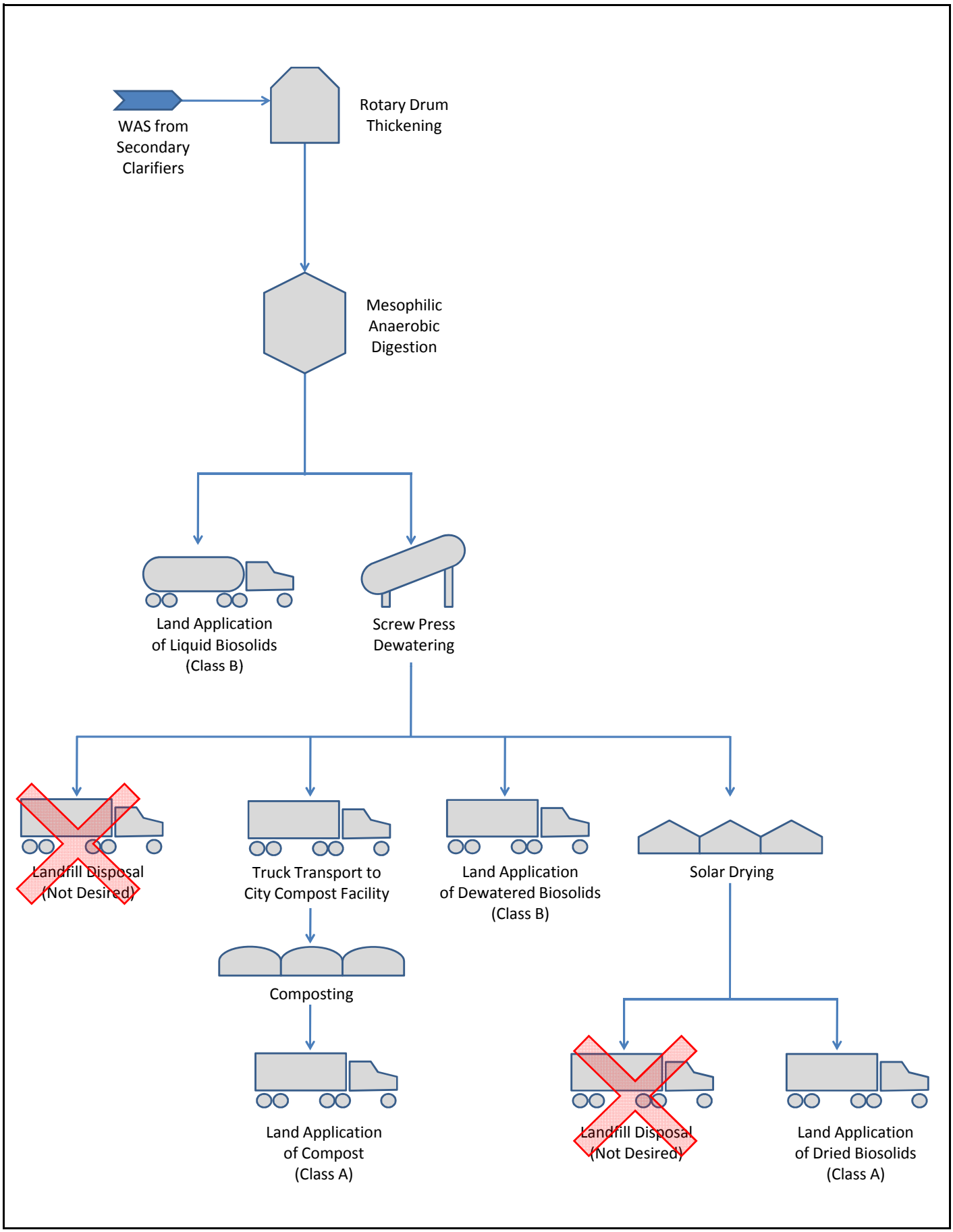

Figure 4-1
Biosolids Disposal Options

4.2.1.1 Land Application as Class B Liquid Biosolids

The anaerobically digested, Class B biosolids produced at the Franklin WWTP may be disposed via land application. In a land application process, the liquid biosolids can be sprayed from a tanker truck as the truck moves through the land application site. It is a relatively inexpensive option that requires minimal capital cost; however, it can be labor intensive. Increased restrictions are placed on Class B biosolids to include restricting public access to the application site, limiting livestock grazing, and controlling crop harvesting schedules.

4.2.1.2 Uses of Digester Biogas

Combined Heat and Power (CHP) is an integrated energy system located at or near the point of use at a facility to provide at least a portion of the electrical or mechanical load while utilizing the waste/reject heat from the power application to provide heating, process steam, cooling, and/or dehumidification. **Figure 4-2** shows a typical CHP arrangement.

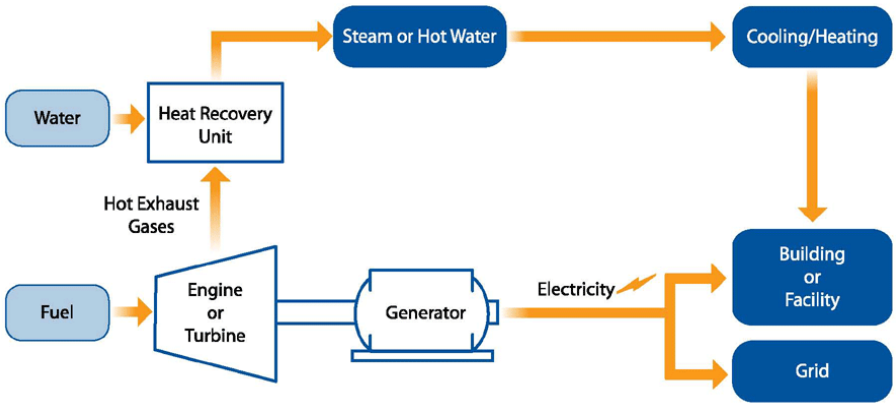

Figure 4-2: Typical Combined Heat and Power Diagram
(Source: U.S. Environmental Protection Agency. Combined Heat and Power Partnership: Basic Information. http://www.epa.gov/chp/basic/index.html)

Digester biogas produced at WWTPs presents a promising potential application for CHP technology. According to the EPA, the benefits of utilizing biogas in a CHP application at a WWTP include:

- Generation of power at a cost below that of retail electricity.
- Displacement of purchased fuels for other thermal needs.
- Qualification as a renewable fuel for green power programs.
- Enhancement of the plant's power reliability – serves as an additional back-up supply.
- Act as a long-term price and volatility hedge against purchased fuels and electricity.

Several options are available for CHP, including fuel cells, microturbines, and reciprocating engines. The suitability of a particular technology for a WWTP depends on the owner's goals and the amount of biogas available. The equipment selection for this project was based on the projected maximum biogas production at a wastewater flow of 16 MGD in the year 2023. Based on the anticipated average sludge production and assuming an energy content of 9,000 BTU per pound of VS reduced, a maximum of 2.04 MMBTU/hr of

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digester gas will be produced. Typical mechanical devices are approximately 35 to 38 percent efficient at producing electrical power from digester gas. If all of the biogas is used for electricity production, the maximum potential production is approximately 227 kW. At wastewater flows of 24 MGD, the electricity production increases to approximately 345 kW.

For the above range of potential biogas production, a 250 kW microturbine was selected as the basis of this conceptual analysis, as most of the commercially available reciprocating engines have a much larger capacity. Microturbines were derived from turbocharger technologies found in automobiles or the turbines in aircraft auxiliary power units. Most microturbines are single-stage, radial flow devices with high rotating speeds of 90,000 to 120,000 revolutions per minute (rpm). However, a few manufacturers have developed alternative systems with multiple stages and/or lower rotation speeds.

Ingersoll Rand (IR) had been the only manufacturer of 250kW microturbines with a proven track record. Recently, however, IR sold off its microturbine business to FlexEnergy, located in Irvine, California. FlexEnergy continues to manufacture and sell the IR microturbine unit, which is a gas-powered turbine, 250 kW synchronous electric generator. It comes as a pre-engineered and tightly integrated package that is ready to be installed on-site either indoors or outdoors. The microturbines are designed to run 24 hours a day, 7 days a week with startups and shutdowns limited to once per day, or less frequently. An optional waste heat recovery system is available.

Microturbines can run on a range of gaseous fuels ranging from low- to medium- and high-BTU fuel sources; however, when compared to reciprocating engines, microturbines have more stringent restrictions on the inlet fuel quality. The required inlet fuel gas pressure for microturbines is approximately 100 psi. Additionally, the digester gas must be treated to rid it of unwanted constituents before being used in a microturbine. The reliability of a microturbine depends on the fuel conditioning systems; improper gas treatment can cause problems with maintenance and operation and could potentially shorten the unit's service life.

Since the quality of digester gas varies from facility to facility, a fuel gas analysis is performed prior to the start of design in order to recommend the fuel treatment required for any particular application. The manufacturer typically does not guarantee equipment performance or adherence to emissions limits without first performing this fuel gas analysis, at the inlet to the microturbine, to verify the effectiveness of the digester gas treatment system. A typical digester gas treatment system involves removal of sulfur compounds, halogenated organic compounds (HOCs), siloxane, and any particulate larger than 10 microns.

There are thousands of microturbine installations ranging from 25kW to 500kW. It should be noted that microturbines that run on cleaner fuels, such as natural gas or gases with lower siloxane levels (*e.g.,* landfill gas), are expected to perform better than units that run on digester gas. The approximate capital cost (estimated per EPA reports) for installation of a single 250kW microturbine is $350,000.

The advantages and disadvantages of microturbines are summarized below in **Table 4-1.**

Table 4-1
Advantages & Disadvantages of Microturbines

Advantages	Disadvantages
• Small number of moving parts • Compact size • Light weight • Low emissions • Long maintenance intervals • Higher combined efficiency	• Low Electrical efficiency • Cleaner fuel requirements • Loss of power output and efficiency with higher ambient temperature and elevation • Very high gas pressure

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(thermal and electrical)	requirements • No partial loading

The electricity produced would be available for use at the plant, or it could be sold to the utility power grid. It is expected that all power produced by the microturbine could be consumed at the plant. However, with potential economic incentives, it may be more beneficial to sell power to the utility grid, thereby generating revenue for the City.

As mentioned above, in addition to power production, CHP systems offer increased energy recovery by capturing waste heat from the power producing equipment. For this system, evaluated at maximum power production, 45 percent of the input energy becomes waste heat that could be captured through a series of heat recovery loops. Through heat exchangers, energy can be transferred and used to heat sludge in the digesters. On average, 0.92 MMBTU/hr could be recovered from the microturbine. This provides approximately 80 percent of the annual average digester winter heating demand of 1.16 MMBTU/hr at 16 MGD operating conditions in the year 2023. Additional heat could be provided by either diverting digester gas away from electrical production or supplementing the digester gas with another fuel source, such as natural gas. The proposed microturbine could also be operated at full-load electrical production with supplemental natural gas and therefore provide a higher heat recovery, which may be sufficient to meet the average winter sludge heating demands. A second option is to fire the diverted digester gas or supplemental fuel in a boiler dedicated to the sludge heating system.

Methods to increase digester biogas production are discussed in Section 4.3.

4.2.2 Screw Press Dewatering

Though the City has not yet finalized its dewatering equipment selection, it was assumed that the chosen equipment would be capable of dewatering the digested biosolids to a solids content of approximately 20 percent, to be confirmed by onsite testing. The dewatered biosolids could be land applied or incorporated into the City's current composting process. Landfill disposal is included here as a backup to the selected disposal method but is not recommended.

4.2.2.1 Land Application as Class B Dewatered Biosolids

Solids from the dewatering process can be land applied once they achieve Class B status, which is achieved through stabilization. The requirements for a Class B process appear in the Federal Part 503 Biosolids Rule and are summarized in the US EPA's *A Plain English Guide to the EPA Part 503 Biosolids Rule (Plain English Guide)*. Biosolids applied to land must meet the listed pollutant ceiling concentrations, Class B requirements for pathogens, and vector attraction reduction requirements. Class B biosolids have site application restrictions to minimize potential for human and animal exposure until the pathogens are further reduced by environmental factors such as heat and sunlight. Class B biosolids cannot be sold or distributed in bags or containers, and cannot be used at public contact sites.

The Tennessee Department of Environment and Conservation (TDEC) Division of Water Pollution Control (DWPC) also provides *Guidelines for the Land Application and Surface Disposal of Biosolids* (February 2011), based on the EPA Part 503 Biosolids Rule. The *Guidelines* provide guidance on aspects of biosolids disposal including setback distances for land application, limits on staging and storage of biosolids, and agronomic loading rates. All new land application sites for Class B biosolids must be approved by TDEC-DWPC, but Class A biosolids or biosolids considered EQ do not require site approvals by TDEC-DWPC.

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4.2.2.2 Addition to the City's Composting Process

Composting is a process in which organic material undergoes biological degradation to a stable end product. In a well run, efficient process, approximately 20 to 30 percent of the VS is converted to carbon dioxide and water. Methods of composting include the aerated static pile, windrow, and in-vessel systems.

Composting at the existing City facility is achieved by the windrow composting method. Rows are created and turned and mixed periodically; they are typically turned a minimum of five times while the temperature is maintained at or above 130°F (55°C). Turning of the compost may result in the release of odors, as maintaining aerobic conditions in a windrow during the process is difficult. The moisture content of the feed sludge is important to the process and affects the amount of bulking agent that must be added to the mixture. The higher the moisture content of the feed biosolids, the larger amount of bulking agent required, and the larger the land required to complete the process.

If the City were to divert part of the Franklin WWTP's dewatered biosolids production to the existing composting facility, or blend the biosolids with the existing composting process, requirements to achieve a Class A composted biosolids material must be met. **Table 4-2** summarizes the requirements for producing Class A and B biosolids from the composting process. These requirements are also found in the US EPA's *Plain English Guide.*

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Table 4-2
Summary of Class A and Class B Requirements for Composting of Biosolids

In-Vessel or Static Aerated Pile Composting	Windrow Composting
Class A Requirements	
Temperature maintained at 55°C or higher for 3 days	Temperature maintained at 55°C or higher for 15 days or longer. During this period when the compost is maintained at 55°C or higher, the windrow is turned a minimum of five times.
Class B Requirements	
Temperature raised to 40°C or higher and maintained for 5 days. For 4 hours during the 5-day period, the temperature in the compost pile exceeds 55°C.	

Source: A Plain English Guide to the EPA Part 503 Biosolids Rule, EPA/832/R-93/003

The City can also choose to blend the final biosolids product with the final compost product as an additional bulking agent. If this were to be done, all biosolids requirements of achieving a Class A product during the solids management process described in the Part 503 regulations must be met before the product can be distributed to the public for agronomic use.

4.2.2.3 Landfill Disposal

The dewatered biosolids discharged from the screw presses could be disposed at a landfill. The City can contract with a local landfill to accept the dewatered biosolids as long as they continue to meet landfill standards as determined by the results of the Paint Filter Liquids Test to which the City's hauled biosolids are currently subject. While a disposal contract for the City's current biosolids output has been negotiated and is secure in the near future, the City's long-term prospects for landfill disposal of its biosolids are uncertain and susceptible to future restrictions that could be imposed by US EPA, TDEC, and the private landfills that accept biosolids.

Landfill disposal is included in this discussion only as a "worst case" or backup option for emergency disposal of dewatered biosolids. The City does not intend to continue using a landfill as the primary means of disposal.

4.2.3 Solar Drying

Solar drying uses the sun's energy to dry the dewatered biosolids to approximately 80 percent dry solids content. Each solar drying greenhouse consists of a rectangular concrete base slab, with translucent side walls and roof that allow light to transmit through the walls, reaching the solids and ultimately drying them. The dried biosolids can be disposed via land application as Class A biosolids. Landfill disposal is briefly discussed here; however, the City does not intend to continue landfill disposal of its biosolids.

4.2.3.1 Land Application as Class A Dried Biosolids

Solids from the solar dryer can be land applied once they achieve Class A status. Class A biosolids are treated to reduce the presence of pathogens to below detectable levels and can be used without any pathogen-related restrictions. Class A biosolids can be bagged and sold to the public.

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4.2.3.2 Landfill Disposal

As with the dewatered biosolids, landfill disposal of dried biosolids is included in this discussion only as a "worst case" or backup option for emergency disposal of the dried biosolids, and it is not recommended or desired as a means of primary disposal.

4.3 Options to Maximize Digester Biogas Production

Anaerobic digesters are increasingly perceived as assets that can do more than just stabilize and process wastewater solids. Biogas produced by the digestion process can be used to fuel boilers and CHP systems, thus enhancing energy recovery and reducing the plant's energy costs.

Production of biogas can be further enhanced through various modifications to the conventional digestion process; these modifications include the addition of organic wastes (co-digestion) and pre-processing of the sludge feed (hydrolysis). This section provides a brief discussion of both methods.

4.3.1 Co-Digestion

Co-digestion refers to the digesting of domestic sludge with other organic wastes such fats, oil, and grease (FOG), restaurant waste, food processing waste, and the organic fraction of municipal solid waste.

Key drivers for co-digestion are maximizing the facility's assets, increasing biogas production, and generating a revenue stream from tipping fees associated with receiving organic wastes. In addition, feeding excess waste to digesters is a way to keep this waste out of collection systems and landfills, thereby reducing the amount of material that might otherwise plug sewers and increase system maintenance costs.

Some WWTPs in the United States are currently collecting excess wastes and feeding them to their digesters at centralized FOG receiving facilities to accomplish these goals. Co-digestion with feedstocks that contain a high solids concentration of 15 to 20 percent are typically "dry" digested in plug-flow-type silo reactors. These processes were originally used in the solid waste industry. Feedstocks containing less than 10 to 15 percent solids are more easily pumped and mixed and are, therefore, more amenable to conventional "wet" digestion.

From an environmental perspective, co-digestion can reduce greenhouse gas emissions by reducing the amount of methane that would otherwise be released during anaerobic decomposition in landfills, and by producing more biogas that can beneficially be used in lieu of nonrenewable fossil fuels. Operationally, co-digestion has the potential to improve the stability, biogas production, and volatile solids reduction (VSR) of the digesters and secondary treatment system. There are also social/political benefits to the nearby community, which may value the conversion of waste products into a valuable renewable resource such as biogas.

Some advantages of co-digestion its ability to improve the nutrient balance of the digester and help dilute many high-strength waste streams. Also, if a digester feed is too dry, cosubstrates can help supplement the moisture requirements of the digester. Easier access to handling of mixed wastes, the use of common access facilities, and economies of scale are also potential advantages of the process.

Both carbon (C) and nitrogen (N) are required in proper proportions to foster growth of microorganisms in the digester and to prevent process upset. If the carbon content is too high relative to nitrogen, then a nitrogen deficiency and process upset can occur as a result of propionate accumulation because of lower buffering capacity. Similarly, if the carbon content is too low relative to nitrogen, digester performance will likely suffer because of ammonia toxicity. The addition of certain cosubstrates to the digester can help balance nutrient deficiencies and improve performance. For example, adding a nitrogen-rich animal slurry (*e.g.,* chicken manure) to a digester with a high C:N ratio can help achieve a favorable nutrient balance and

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reduce the potential toxic effects of free ammonia and hydrogen sulfide in the cosubstrate waste stream. Likewise, adding a carbon-rich cosubstrate (*e.g.,* cattle manure) to a digester with a low C:N ratio can provide the necessary growth substrate and enhance microbial growth in the reactor. A site-specific analysis is preferred to determine the optimal C:N ratio for a combination of cosubstrates.

Potential disadvantages of co-digestion include construction of the infrastructure required to receive and process additional waste, handling of additional wastes in dewatering systems, handling and treatment of additional gas, and the liquid-stream effects of increased loading on the digesters. Increased gas production and tipping fees, however, typically can compensate for the capital investment in co-digestion upgrades.

4.3.1.1 FOG Addition

The most common organic waste cosubstrate is FOG, which is generated by a myriad of sources, including households, restaurants, hotels, commercial kitchens, bakeries, schools, prisons, and large food-preparation facilities. The solids concentration of FOG typically varies from less than 2 percent to more than 15 percent, and the volatile solids to total solids (VS/TS) ratio typically ranges from 94 to 97 percent.

To prevent blockages of the sanitary sewer system, FOG wastes are collected using waste drums, grease traps, and grease interceptors. Dedicated vacuum pumper trucks are typically used to retrieve FOG from traps and interceptors and transport it back to the FOG receiving system. A FOG receiving system can be as simple as a connection to a digester to allow the truck to discharge directly to a heated receiving tank with settling tanks (rock traps) and a digested sludge circulation loop used to convey the FOG to the digester. Processing of the incoming FOG should include screening and grinding to remove stones, rags, metallic objects, and other inert material that could impair downstream operations. Concentration of the FOG may also be performed.

FOG is readily biodegradable and can provide both enhanced biogas production and increased VSR when added to anaerobic digestion systems. In fact, FOG is among the highest-rated cosubstrates in terms of methane production.

Digesters have been reported to remain stable at FOG loads as high as 30 percent of feedstock or volatile solids loading. At higher loadings between 30 and 50 percent, digesters can become susceptible to instability during a peak FOG load. It should be noted that one crucial characteristic for successful FOG digestion is good digester mixing, particularly at the surface. Without adequate surface mixing, FOG will tend to collect at the surface of the digester.

Limited literature is available regarding methane production rates from FOG co-digestion and tends to differ in terms of units and FOG loading rate. Gas production potential from lipid-rich cosubstrates (i.e., FOG) is 1.4 m^3/kg (23 cu ft/lb) VSR, significantly higher than that expected from carbohydrate and protein-rich cosubstrates, which is 0.8 to 0.9 m^3/kg (13 to15 cu ft/lb) VSR. For comparison, gas production from conventional mesophilic digestion fed municipal biosolids ranges from 0.8 to 1.1 m^3/kg (12 to 18 cu ft/lb) VSR. Also, because a relatively large percentage of the FOG waste is destroyed during digestion, little of the FOG feedstock remains in the digested product to be handled and disposed.

4.3.1.2 Food Waste Addition

Food waste is a broad category that includes wastes from the following sources:

- Industrial sources - fruit and vegetable processing, beverage industry
- Commercial sources - restaurants
- Residential sources - the organic fraction of municipal solid waste

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Although most food wastes are easily digested and require little pretreatment, food wastes with large inert materials, such as animal parts and carcasses, may require detailed pretreatment and are not typically recommended for co-digestion. Other wastes from livestock, poultry, fish, and even fruit-juice processing may simply require chopping as part of the pretreatment scheme.

4.3.2 Hydrolysis Technologies

WAS, in its raw form, is difficult to digest because a large percentage of the nutrient-rich, digestible cell material is trapped within a lignin-rich cell wall. A pre-digestion processing step can be added to rupture this cell wall and increase the amount of material available for digestion. This addition enhances the naturally-occurring hydrolysis process – the conversion of organic solids to soluble compounds – which is often identified as a rate-limiting step in the anaerobic digestion process.

Expediting the hydrolysis process yields the following benefits to digestion:

- It allows for increased organic loading to the anaerobic digester.
- It results in increased VS destruction in the digesters. Increased VS destruction correlates to an increase in biogas production and a reduction in the amount of biosolids to be disposed.
- It is also reported to reduce the viscosity of the biosolids, which makes the digested product easier to pump and dewater. The need for polymer as a dewatering aid is often reduced.
- It reduces foaming in the digesters.

There are various ways to enhance hydrolysis; two methods discussed here are thermal hydrolysis and electroporation.

4.3.2.1 Thermal Hydrolysis

Thermal hydrolysis uses elevated temperatures and pressures to improve the digestibility of biosolids. First, the digester feed solids are thickened to reduce the amount of water in the material. The thickened solids are then pumped through a macerator before entering the hydrolysis batch reactor tank. The thermal hydrolysis reaction occurs in this reactor, which operates at 160 to 170 °C (320 to 338 °F) for a 20 to 60 minute retention time, typically averaging 30 minutes. The reactor is operated under pressure, typically 500 to 800 kPa (70 to 120 psi).

Although the processes of preheating, hydrolysis reaction, and depressurization can occur in a single tank, an alternate process configuration is to have three tanks in series with a specific function occurring in each vessel. The addition of heat-and-energy recovery units is common through the process. Steam generated by boilers or heat captured from CHP systems typically provide the heat for thermal hydrolysis processes. The use of digester biogas as a fuel may offset the energy requirements of the thermal hydrolysis reaction.

A drawback of thermal hydrolysis is that the process gas is highly odorous and saturated with water vapor. Two treatment methods available for the process gas are as follows.

- Pass the process gas through a condenser to remove water vapor. The condensed liquid is pumped to the anaerobic digester for treatment and the remaining gas is burned in a thermal oxidizer.
- Use the process gas to aerate the activated sludge tanks. The activated sludge acts as a biofilter to remove odors and other organics from the gas.

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Full-scale thermal hydrolysis facilities are in operation in more than 20 locations around the world; however, the only facility in the United States – at the Blue Plains Advanced Wastewater Treatment Plant in Washington, DC – is currently under construction.

4.3.2.2 Electroporation

Electroporation is the process by which the permeability of the cell wall is increased through exposure to an electrical field. This technique, practiced since the 1960s, is used by molecular biologists to introduce material into a cell, and it is also used by the food processing industry to sterilize and preprocess foods.

One vendor of electroporation technology for wastewater sludge treatment is OpenCEL. OpenCEL was acquired by Trojan Technologies and made a division of US Peroxide in October 2011. In OpenCEL's patented Focused Pulsed technology, the sludge is passed between two electrodes, where it is exposed to pulses of high-voltage electricity. This exposure breaks down the cell membrane and makes it more permeable. The cells subsequently swell and rupture, releasing their digestible material.

There are two full-scale OpenCEL installations in the United States. The installation at the Northwest Water Reclamation Plant in Mesa, Arizona, has been operating since 2007. The OpenCEL process at this 12 MGD facility treats a 50:50 mixture of primary sludge and WAS, and the treated sludge now provides about 90 percent of the supplemental carbon needed for the plant's denitrification process. The second full-scale installation, located in Racine, Wisconsin, was undergoing startup and testing as of December 2011. Two OpenCEL units are on order for a CDM Smith-designed digester enhancements project at the Regional Water Quality Control Plant (RWQCP) in Riverside, California.

In addition to increasing the organic loading to the digester, increasing VS destruction, reducing foaming, and boosting biogas production, an electroporation technology such as OpenCEL can provide at least 50 percent of a plant's supplemental carbon needs. It also raises the temperature of the feed sludge by about 20°F, reducing the supplemental heating requirements for the digester. Depending on how the treated sludge is used, payback can occur in as little as two to three years. Pilot testing for the RWQCP project indicated that the OpenCEL process would result in up to 13 percent reduction in biosolids mass, a 15 percent increase in biogas production, and a 4-year payback period.

CDM Smith's discussions with OpenCEL indicate that for full-scale treatment at the Franklin WWTP, one OpenCEL unit would be installed during Phase I to treat up to 40 gpm of sludge. A second unit would be installed in Phase II. The capital cost of the first unit would be approximately $1.5 million, and the second unit could be added for a cost of approximately $800,000. The operating cost of the OpenCEL process would be between $30 and $35 per DT. While the operating cost for this process appears high, it could be significantly offset by the resulting reductions in polymer usage and digester heating costs.

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City of Franklin, Tennessee

Integrated Water Resources Plan

Appendix F:

Stormwater

- Feasibility Study for Stormwater Reuse and/or "Rainwater Harvesting"
- Stormwater Technical Analysis Evaluation of Stormwater Basin Plans
- Feasibility of Robinson Lake Flow Augmentation
- Green Infrastructure and Low Impact Development Techniques and Opportunities

CDM Smith

Memorandum

To: *Project Staff*

From: *David Mason, P.E.*

Date: *April 30, 2011*

Subject: *Feasibility Study for Stormwater Reuse and/or "Rainwater Harvesting"*

Historically, Tennessee has enjoyed access to an abundant supply of potable water. However, the drought of 2007 forced many communities to begin to consider a more sustainable approach to using water resources. For example, the City of Franklin's Sustainable Community Action Plan of 2009 identifies the following action/goal:

> *Reduce potable water usage within the Franklin City Limits by 25 percent May through October and by 10 percent during the remaining months by 2014.*

Traditional water conservation measures have been implemented by the City to move toward this goal, such as the City's Water Hog public education program. The City's Sustainability Commission has also considered such concepts as an Irrigation Ordinance to reduce potable water demand. In conjunction with these initiatives, the Franklin Integrated Water Resource Plan (IWRP) also considers the role of sustainable stormwater management as an additional component to reduce reliance on potable water. The effective management of stormwater can provide multiple benefits to the City in terms of reduced demand for potable water and improved water quality through stormwater reuse or rainwater harvesting.

Rainwater harvesting includes the collection of rainwater from impervious surfaces and storing for later use. While this concept is still not widely employed throughout the country, many communities with more frequent water availability issues have re-assessed the role that rainwater harvesting can play for water supply planning. Rainwater reuse offers a number of benefits:

- Provides inexpensive supply of water;
- Augments drinking water supplies;
- Reduces stormwater runoff and pollution;

- Reduces erosion in urban environments;
- Provides water that needs little treatment for irrigation or non-potable indoor uses;
- Helps reduce peak summer demands; and
- Helps introduce demand management for drinking water systems.

Each of these benefits aligns closely with the issues being considered for the IWRP. Thus, the following sections quantify the potential benefits that may be achieved through the implementation of rainwater harvesting projects in the City.

Rainwater Harvesting Use

Society currently uses potable water to meet nearly every demand, particularly for domestic uses. This trend is similar for commercial water use. However, studies have shown that almost 80% of demand is often being utilized for end use that could be satisfied with lesser quality water. Rainwater harvesting offers an alternative water supply that can more appropriately match water use to the quality of water supplied.

The most common application of rainwater harvesting is the capture of runoff from rooftops. While rooftops do contain pollutants, testing has shown that those pollutants are generally in lower concentrations and absent of many toxins when compared to other sources of runoff. Therefore, rooftop runoff may be diverted to cisterns or rain barrels to capture and store the runoff for other uses, such as outdoor irrigation and/or indoor, non-potable plumbing (toilets, etc.).

Potential Water Demand Reduction

Depending on the policies or programs put in place by the City, there may be several possible avenues to pursue water demand reduction through the use or requirement of rainwater harvesting. Typically, rainwater harvesting programs are most easily implemented at local government owned facilities. However, some communities choose to implement programs that require the use of rainwater harvesting on all new construction or that provide benefits for businesses/homes that retrofit existing developed sites to include rainwater harvesting.

In order to estimate potential water demand reduction, an estimate must be made of the potential surface area from which rainwater will be captured and the "efficiency" of that surface to convert rainfall to runoff that can ultimately be collected. Studies show that approximately 0.62 gallons per square foot of collection surface per inch of rainfall can be collected. In practice, however, some rainwater is lost to first flush, evaporation, splash-out or overshoot from the gutters in hard rains, and possibly leaks. Rough collection surfaces are less efficient at conveying water, as water captured in pore spaces tends to be lost to evaporation. For these reasons, inherent inefficiencies of the system need to be factored into

the water supply calculation. Most studies recommend an efficiency capture rate of approximately 85%.

The City of Franklin receives approximately 54.33 inches of rain annually. Using this rainfall, along with the efficiency and capture rates noted above, the following equation can be used to estimate the total catchment potential (in gallons) for a specific catchment area:

Catchment Potential (gals) = Catchment Area (ft^2) x Runoff Conversion (0.62) x Capture Efficiency (0.85) x Annual Rainfall (54.33 inches)

The following sub-sections summarize the potential water savings for various rainwater harvesting strategies.

City-Wide Rainwater Harvesting Potential

Using the City's GIS system, CDM estimated the total possible catchment area associated with rooftops in Franklin. Based on the most recent GIS data, approximately 84 million square feet of rooftop exists in the City. Using the equation from the previous section, 84 million square feet could generate approximately 2.4 billion gallons of water annually. If the City were able to achieve a 10 percent harvest rate for all rooftops, the City could reduce overall water demands in the City by approximately 240 million gallons annually (or 660,000 daily). The current water demand in the City is approximately 7.2 MGD. Therefore, harvesting water from only 10 percent of the rooftops in the City could result in a 9 percent reduction in daily water usage city-wide.

Municipal Rainwater Harvesting

While the potential for rainwater harvesting city-wide is significant, applying the concepts of rainwater harvesting to municipal buildings throughout the City is the easiest way for communities to realize water demand reductions. For example, the City has already implemented this strategy for the Police Station, where rainwater is captured for irrigation and toilet flushing to reduce demand on the drinking water system. This strategy not only provides the benefit of reduced water use, but it also provides a model of stewardship to the community.

CDM has estimated the potential for water savings from a municipally-focused rainwater harvesting program. As an example, the City could establish a long-term goal of retrofitting all existing municipal facilities to include rainwater harvesting practices. Using the City's GIS information, CDM extracted data related to the total square footage of rooftop on all City-owned properties. Based on the evaluation of 100 City-owned properties, CDM estimated a total rooftop area of approximately 1.71 million square feet. This area represents the total potential rainfall catchment surface area. Applying the equation noted above, the estimated water available for harvesting is approximately 49 million gallons annually. If the City were

to pursue an aggressive retrofit program (retrofit 100% of all City-owned facilities), the City may achieve an average day water demand reduction of approximately two percent.

Neighborhood/Watershed-Scale Rainwater Harvesting

Many Cities throughout the country encourage rainwater harvesting at the neighborhood or watershed scale through incentive programs. For example, the City of Austin, TX purchases rainbarrels in bulk and sells the rainbarrels to citizens at a $20 savings per barrel. Other cities simply offer rain barrels free to interested citizens through neighborhood/watershed scale pilot programs. In order to estimate the potential water savings from one of these strategies, CDM selected a neighborhood in Franklin to use as a demonstration.

The Chestnut Bend subdivision off Hillsboro Road includes approximately 185 homes with a total square footage of rooftop of approximately 600,000 square feet. Studies of rainbarrel programs in other portions of the country have shown that communities may achieve a 40% participation rate. Also, due to residential roof configuration, a typical capture rate for residential homes is approximately 60 percent. Applying the same formula as above, the potential stormwater capture rate for the Chestnut Bend subdivision is approximately 4.8 million gallons annually. Assuming two rainbarrels per home at a cost of $150 per barrel, the cost for this neighborhood-scale example is approximately $22,000.

Commercial Rainwater Harvesting

The same principles of residential and municipal rainwater harvesting may also be applied to commercial areas in Franklin. For example, the Galleria Mall has a rooftop square footage of approximately 700,000 square feet. Using the equations above, the estimated annual rainwater capture for the mall is approximately 20 million gallons.

Conclusions and Recommendations

Rainwater harvesting holds the potential to conserve on-site potable water use, protect surface water quality, and reduce the risk of flooding within the City. Examination of the local rainfall records reveals an average annual rainfall amount of 54.33 inches and conditions are relatively constant throughout the year. Therefore, systems need not be designed to address large fluctuations in demands and availability.

Results from the four scenarios above indicate a significant amount of rainfall available for capture and reuse. However, there is a significant cost for implementation of these systems, which may cause this strategy to be cost prohibitive at current, relatively low water rates. An opportunity does exist for the City to implement projects on City property to capture and reuse stormwater runoff, which can serve as demonstration projects for other businesses/industries to consider and also serve as a public education component for the citizens.

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Technical Memorandum

To: *City of Franklin IWRP Team*

From: *CDM*

Date: *July 29, 2011*

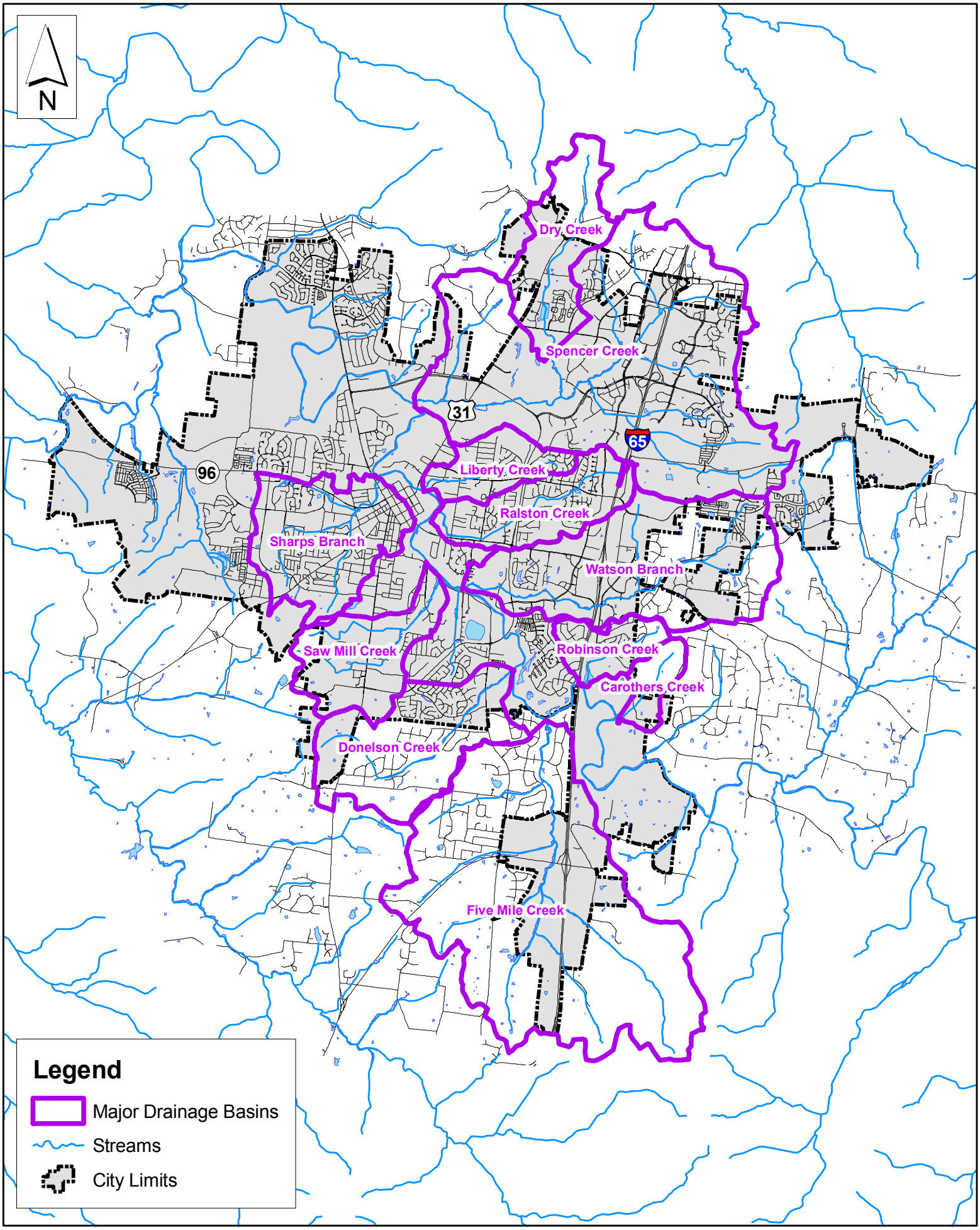
Subject: *Integrated Water Resources Plan – Stormwater Technical Analysis - Evaluation of Stormwater Basin Plans*

1.0 Introduction

In April 1999, the City of Franklin selected Camp Dresser & McKee Inc. (CDM) to develop a Comprehensive Stormwater Management Program (SWMP) to proactively address stormwater needs related to rapid growth and the City's regulatory requirements. A phased approach was outlined to maximize the City's resources and to minimize incremental program costs. In the years following, CDM developed individual plans for each of the major basins in the City, which are shown in **Figure 1**. The primary focus of the plans was to address water quantity/flooding issues which had become increasingly problematic as the City continued to develop. Water quality consideration was part of the analysis, but water quality modeling was not performed for these original studies.

Since the original development of these plans, water quality has evolved as a significant driver in stormwater management planning. The first major water quality driver was the City's National Pollutant Discharge Elimination System (NPDES) Municipal Separate Storm Sewer System (MS4) permit, which was issued in 2003. This permit was recently re-issued in October of 2010 and includes additional requirements to address water quality within the City's MS4 jurisdiction.

In addition to the NPDES MS4 permit, significant attention has been focused on identifying impaired waters throughout the State. The State performs routine sampling of streams to identify which streams do not meet State Water Quality Standards for their intended use. Streams that do not meet their intended use are placed on the State 303(d) list of impaired waters. Once a waterbody is on the list, the State works through a priority schedule that ultimately results in the development of a Total Maximum Daily Load (TMDL), which is essentially a "pollution diet" for a river or stream. The TMDL plan quantifies the amount of pollutant in the stream, identifies the source of the pollutant, and recommends regulatory or other actions that may need to be taken in



CDM

Figure 1
City of Franklin, TN
Major Drainage Basins

order for the stream to cease being polluted. The requirements of these two regulatory programs have led the City to consider a re-evaluation of the previous studies to consider additional water quality improvements.

CDM has met with the City's stormwater staff to review the previous plans in order to identify potential improvements, including water quality enhancements to existing projects as well as to develop estimates of the pollutant removal potential for these projects.

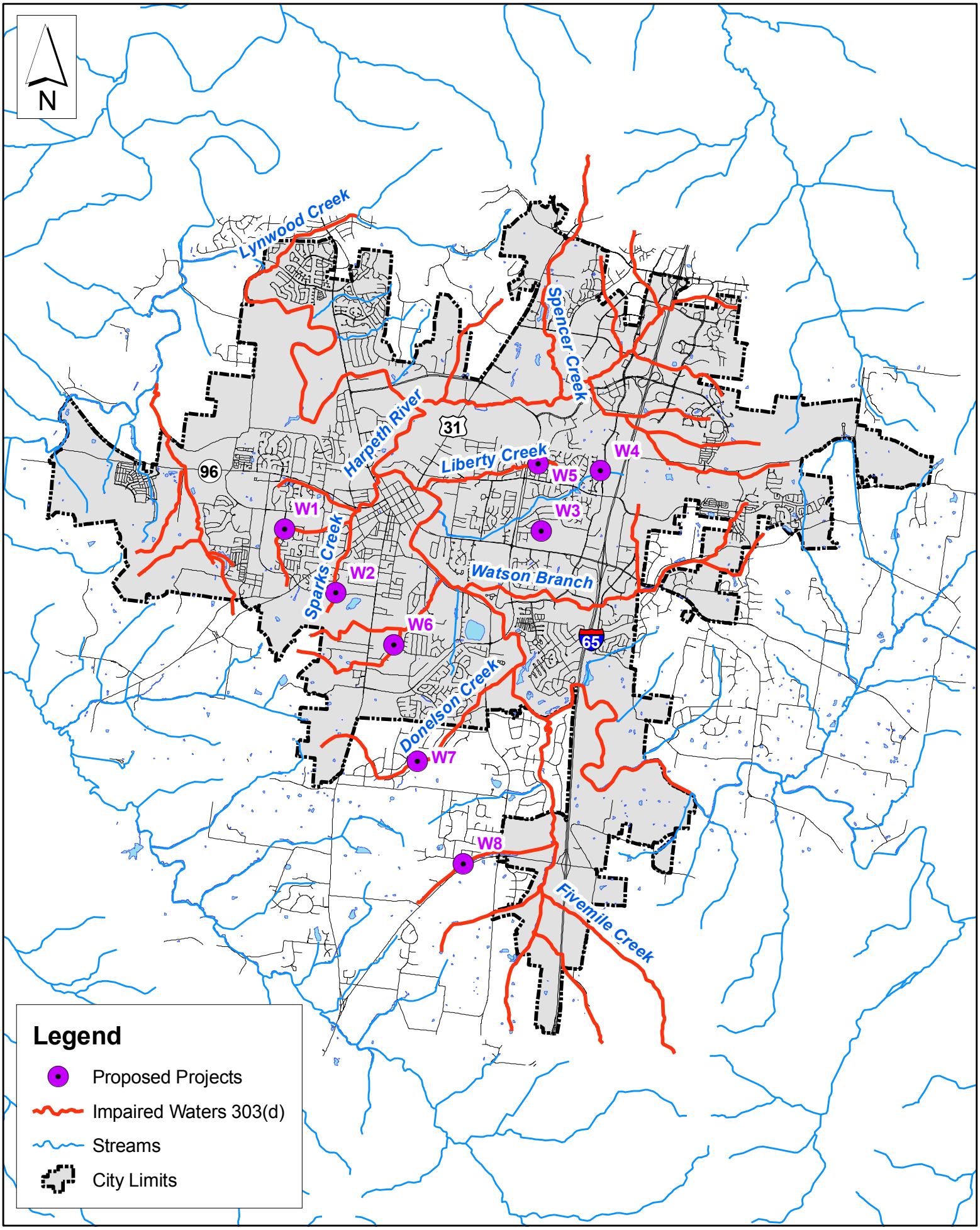
2.0 Data Collection and Evaluation

CDM performed a review of each of the previous stormwater plans developed for the City. From each plan, CDM compiled a list of proposed flood-control projects. **Table 1** summarizes the list of projects. The location of each proposed project is shown on **Figure 2.**

Table 1 – Summary of Proposed Stormwater Projects from Previous Plans

ID	Stream	Watershed	Tributary Area (acres)	Watershed Plan Notes
W1	Sharps Branch	Sharps Branch	210	Detention Facility, 40 ac-ft of storage, Tributary 2 near Birchwood Circle
W2	Quarry Branch	Sharps Branch	107	Detention Facility, 30 ac-ft of storage, Tributary 3 near Downs Blvd and Figures Dr
W3	North Ewingville Creek	Ralston Creek	20	Detention Facility, retrofit existing facility at junction 10600
W4	North Ewingville Creek	Ralston Creek	66	Detention Facility, Junction 32450 upstream of Stanwick Dr.
W5	Liberty Creek	Liberty Creek	88	Detention Facility, 10 ac-ft of storage. Main stem, upstream of Jordan Rd
W6	Saw Mill Creek	Saw Mill Creek	700	Detention facility in the vicinity of Model Junction 90420
W7	Donelson Creek	Donelson Creek	880	Detention facility in the vicinity of model junction 90851
W8	Goose Creek	Five Mile Creek	290	Detention facility, 10 ac-ft storage, retrofit recommendation

Because Franklin has continued to grow at a rapid pace throughout the development of these plans, it was important to review the location of each project to verify if the project was still viable. Using the most recent aerial photography and location information from the plans, CDM



CDM

Figure 2
City of Franklin, TN
Capital Projects and Impaired Waters

mapped each of the proposed projects and performed a desktop review of the location compared to existing landuse/development. Based on this review, CDM eliminated Project ID W2 in Sharps Branch due to the existence of development at the previously proposed site. All other projects appear to be feasible based on the results of this review.

3.0 Current Water Quality Conditions

Because water quality is a key component of this reassessment, it is important to understand the current water quality conditions in the City. CDM performed a review of the State's 303(d) list of impaired waters to identify water quality impairments within the City limits. The most current list of impaired waters provided by the State is from 2010, although the list has not been finalized. However, a comparison between the 2010 draft and the 2008 final list did not reveal significant changes. A map of the impaired waters in the City (shown in BOLD red) is shown in Figure 2. As indicated, the majority of stream segments in the City have been identified by the State as impaired for one or more pollutants. **Table 2** summarizes the listed impairments, identified pollution sources, and TMDL status.

In total, approximately 87 miles of stream within the City are impaired. The most common impairments include habitat loss from lack of vegetation, siltation, nutrients, low dissolved oxygen and bacteria. The sources of pollution include land development, the City's MS4 and wastewater operations, and cattle grazing (i.e., agricultural practices). The US Environmental Protection Agency (EPA) has already approved TMDLs for some of the streams, including the main stem of the Harpeth River. The City's wastewater treatment plant has been restricted to a discharge of 300 lbs/day of Total Nitrogen (TN), 1,000 lbs/day of Total Suspended Solids (TSS) in the winter and 3,000 lbs/day of TSS in the summer, which are referred to as Waste Load Allocations (WLAs) in the TMDL. These WLAs provide a point of comparison for potential benefits of pollutant reduction from stormwater BMPs. Additional information on approved TMDLs can be found on the State's website (http://www.tn.gov/environment/wpc/tmdl/).

4.0 Water Quality Evaluation

CDM reevaluated the projects in the existing basin plans considering the potential for water quality improvements. Based on a review of the list of water quality impairments, CDM identified the following pollutants to include in this evaluation: 1) TN or total nitrogen, 2) TP or total phosphorus, and 3) TSS or sediment. It should be noted that no additional design calculations or modeling were performed for this task. However, a literature search was conducted to identify potential pollutant removal capabilities of common stormwater BMPs, which included dry detention ponds, wet detention ponds, and constructed stormwater wetlands. It was assumed that minor changes could be made to the outlet configurations of the proposed ponds to provide the desired water quality treatment with minimal impact to the original cost estimates identified in the plans.

Table 2
City of Franklin, TN
Integrated Water Resources Plan
Summary of Impaired Waters

Waterbody	Miles Impaired	Cause of Impairment	Pollutant Source	Comments
HATCHER SPRING CREEK	6.5	Habitat loss due to alteration in stream-side or littoral vegetative cover; Loss of biological integrity due to siltation	Pasture Grazing	Category 5. TMDLs needed.
LYNWOOD CREEK	5.4	Alteration in stream-side or littoral vegetative cover; Loss of biological integrity due to siltation	Pasture Grazing Land Development	Category 4a. Impaired, but EPA has approved a siltation/ habitat alteration TMDL for the known pollutants.
SPENCER CREEK	19.9	Loss of biological integrity due to siltation; Escherichia coli	Land Development Discharges from MS4 Area	Category 5. EPA approved a siltation TMDL for some of the known pollutants.
LIBERTY CREEK	0.54	Toluene; Acetone; Low Dissolved Oxygen; Loss of biological integrity due to siltation; Alteration of stream-side or littoral vegetative cover	Industrial Point Source Discharges from MS4 Area	Liberty Creek is impacted in part due to accidental releasesof toluene and acetone. Thesesubstances indirectly affect fish and aquatic life and directly impact the aesthetics of the stream. Category 5. One or more uses impaired.
LIBERTY CREEK	1.31	Loss of biological integrity due to siltation; Alteration of stream-side or littoral vegetation	Discharges from MS4 area	This stream is Category 5. One or more uses impaired.
WATSON BRANCH	6.8	Loss of biological integrity due to siltation	Land Development	Category 4a. EPA approved a siltation TMDL for the known pollutant.
HARPETH RIVER	6.8	Low dissolved oxygen; Phosphorus	Municipal Point Source Discharges from MS4 area	Category 4a. EPA approved DO and nutrient TMDLs for the known pollutants.
FIVEMILE CREEK	14.4	Loss of biological integrity due to siltation; Escherichia coli	Pasture Grazing	Category 4a. EPA approved siltation and pathogen TMDLs for the known pollutants.
DONELSON CREEK	3.4	Loss of biological integrity due to siltation	Land Development	Category 4a. EPA approved a siltation TMDL for the known pollutant.
UNNAMED TRIB TO HARPETH RIVER	4	Habitat loss due to alteration in stream-side or littoral vegetative cover; Loss of biological integrity due to siltation	Land Development	Category 4a. The stream is impaired, but EPA approved a siltation/ habitat alteration TMDL for the known pollutants.
SHARPS CREEK	4.9	Habitat loss due to alteration in stream-side or littoral vegetative cover; Loss of biological integrity due to siltation	Discharges from MS4 area	Category 4a. The stream is impaired, but EPA approved a siltation/ habitat alteration TMDL for the known pollutants.
HARPETH RIVER	3.9	Low Dissolved Oxygen; Phosphorus; Loss of biological integrity due to siltation; Escherichia coli	Discharges from MS4 area Highways, Roads, Bridges, Infrastructure Construction Pasture Grazing	Category 4a. Impaired, but EPA has approved pathogen, siltation/ habitat alteration, and organic enrichment TMDLs for the known pollutants.
HARPETH RIVER	9	Low Dissolved Oxygen; Loss of biological integrity due to siltation	Pasture Grazing	Category 4a. EPA approved DO and siltation TMDLs for the known pollutants.

Pollutant Loads

The first step in the evaluation process was the development of estimated pollutant loadings delivered to each of the proposed BMP sites. Using the City's GIS data, CDM delineated an approximate tributary area to each site. Next, CDM reviewed the City's aerial photography within each tributary area to estimate the fraction of various land uses that would need to be treated by the BMP. **Table 3** summarizes the total area and land use basis for each of the BMP locations identified in Figure 2.

Table 3 – Land Use Distribution for Each Candidate BMP Site

ID	Total Area (ac.)	Comm. %	Indust. %	Instit. %	Trans. %	Multi-Fam %	Single-Fam %	Open Space %
W1	210			15	10	15	35	25
W2	0							
W3	20	70			10			20
W4	65			35	15		10	40
W5	90			3	15		82	
W6	700	15	30	5	15			35
W7	880			5	15		45	35
W8	290			5	10		40	45

The EPA Spreadsheet Tool for Estimating Pollutant Load (STEPL) was used to generate desktop estimates of pollutant loads delivered to each candidate BMP site. This tool uses a combination of annual rainfall estimates, land use information and event mean concentration (EMC) data to estimate annual pollutant loads. The spreadsheet tool used average annual rainfall data from the closest weather gage (Nashville BNA airport), which was estimated at approximately 59 inches annually. Land use information for each sub-watershed was generated by CDM using the City's existing GIS data. The standard, default EMC values provided in the spreadsheet tool (**Table 4**) used for this evaluation were consistent with national averages.

Table 4 – EMC Values for Various Pollutants and Land Uses (mg/L)

Pollutant	Comm.	Indust.	Instit.	Trans.	Multi-Fam	Single-Fam	Open Space
TN	0.62	0.75	0.62	0.28	0.58	0.58	0.85
TP	0.23	0.27	0.23	0.25	0.31	0.31	0.33
TSS	48	90	48	99	50	50	72

Using the STEPL tool with the values defined in Tables 3 and 4, CDM generated total pollutant loads for each of the candidate sites. The initial pollutant loads were based on the assumption that no BMPs already exist within the watersheds. The loads were generated in units of lbs/year of TN and TP and in tons/yr of sediment for each site (**Table 5**).

Table 5 – Total Pollutant Loads Delivered to BMP Sites

ID	TN Load (lb/yr)	TP Load (lb/yr)	Sediment Load (tons/yr)
W1	1,948	325	45
W2	-	-	-
W3	215	26	5
W4	595	96	14
W5	939	164	22
W6	6,093	935	150
W7	6,192	1,041	155
W8	916	176	29
Total	**16,898**	**2,763**	**420**

Pollutant Load Reduction Potential

The final step in the analysis was to estimate the pollutant load reduction potential for each of the identified sites. Because sizing and design calculations for each site were not performed as part of this analysis, CDM performed a literature search to identify pollutant removal capabilities of selected BMP alternatives. The BMPs used for this analysis included dry detention ponds, wet detention ponds and constructed stormwater wetlands. The primary reference used to estimate pollutant removal capabilities was the National Pollutant Removal Performance Database (September 2007) by the Center for Watershed Protection. The study summarized sampling data from a variety of BMPs in a variety of locations to generate expected pollutant removal efficiencies. Pollutant removal efficiencies used for this analysis are provided in **Table 6**.

Table 6 – BMP Pollutant Removal Efficiencies

BMP Type	% TN Reduction	% TP Reduction	% Sediment Reduction
Dry Detention Pond	15%	15%	70%
Wet Detention Pond	35%	45%	80%
Constructed Wetland	25%	30%	70%

The pollutant removal efficiencies in Table 6 were applied to the predicted loadings from Table 5 to determine the total pollutant removal potential of the designated BMP sites. For each site, pollutant removals were estimated for the three different BMP types. Summaries of the pollutant load reduction estimates are shown in Tables 7a (dry pond), 7b (wet pond), and 7c (wetland). Total estimated pollutant reductions that may be achieved if all sites are built are reported as follows:

- Total Nitrogen Reduction Potential: 2,500 to 5,900 lbs/yr of TN (or, 7 to 16 lbs/day)
- Total Phosphorus Reduction Potential: 400 to 1,200 lbs/yr of TP (or, 1.1 to 3.3 lbs/day)
- Total Sediment Reduction Potential: 290 to 350 tons/yr of Sediment (or, 1,600 to 1,900 lbs/d)

Cost Evaluation

Each of the original basin plans included an estimated cost for construction for the identified flood control facilities. The original cost estimates and designs primarily focused on the flood control benefits of the projects, therefore CDM applied a factor of 1.25 to each of the project costs to adjust for design changes that may be necessary to provide the desired pollutant removal noted in the previous sections. Additionally, costs were converted to 2011 dollars. Excluding project W2, which was determined to not be feasible, the total cost of all projects is estimated to be in the range of $14 to $15 million as summarized in **Table 8**.

Tables 7a - 7c
City of Franklin, TN
Pollutant Removal Estimates for Various BMPs

POLLUTANT REMOVAL via DRY POND

BMP ID	Creek Location	Watershed Name	N Load (no BMP) lb/year	P Load (no BMP) lb/year	Sediment Load (no BMP) t/year	N Reduction lb/year	P Reduction lb/year	Sediment Reduction t/year	N Load (with BMP) lb/year	P Load (with BMP) lb/year	Sediment Load (with BMP) t/year
W1	Sharps Branch	Sharps Branch	1,948	325	45	292	49	32	1,655	276	14
W2	Quarry Branch	Sharps Branch	-	-	-	-	-	-	-	-	-
W3	North Ewingville Creek	Ralston Creek	215	26	5	32	4	3	183	22	1
W4	North Ewingville Creek	Ralston Creek	595	96	14	89	14	10	506	81	4
W5	Liberty Creek	Liberty Creek	939	164	22	141	25	16	798	139	7
W6	Saw Mill Creek	Saw Mill Creek	6,093	936	150	914	140	105	5,179	795	45
W7	Donelson Creek	Donelson Creek	6,193	1,041	155	929	156	108	5,264	885	46
W8	Goose Creek	Five Mile Creek	916	176	29	137	26	21	779	150	9
Total			**16,899**	**2,763**	**420**	**2,535**	**415**	**294**	**14,364**	**2,349**	**126**

POLLUTANT REMOVAL via WET POND

BMP ID	Creek Location	Watershed Name	N Load (no BMP) lb/year	P Load (no BMP) lb/year	Sediment Load (no BMP) t/year	N Reduction lb/year	P Reduction lb/year	Sediment Reduction t/year	N Load (with BMP) lb/year	P Load (with BMP) lb/year	Sediment Load (with BMP) t/year
W1	Sharps Branch	Sharps Branch	1,948	325	45	682	146	36	1,266	179	9
W2	Quarry Branch	Sharps Branch	-	-	-	-	-	-	-	-	-
W3	North Ewingville Creek	Ralston Creek	215	26	5	75	12	4	140	14	1
W4	North Ewingville Creek	Ralston Creek	595	96	14	208	43	11	387	53	3
W5	Liberty Creek	Liberty Creek	939	164	22	329	74	18	610	90	4
W6	Saw Mill Creek	Saw Mill Creek	6,093	936	150	2,133	421	120	3,961	515	30
W7	Donelson Creek	Donelson Creek	6,193	1,041	155	2,167	469	124	4,025	573	31
W8	Goose Creek	Five Mile Creek	916	176	29	321	79	23	596	97	6
Total			**16,899**	**2,763**	**420**	**5,914**	**1,244**	**336**	**10,984**	**1,520**	**84**

POLLUTANT REMOVAL via CONSTRUCTED WETLANDS

BMP ID	Creek Location	Watershed Name	N Load (no BMP) lb/year	P Load (no BMP) lb/year	Sediment Load (no BMP) t/year	N Reduction lb/year	P Reduction lb/year	Sediment Reduction t/year	N Load (with BMP) lb/year	P Load (with BMP) lb/year	Sediment Load (with BMP) t/year
W1	Sharps Branch	Sharps Branch	1,948	325	45	487	98	32	1,461	228	14
W2	Quarry Branch	Sharps Branch	-	-	-	-	-	-	-	-	-
W3	North Ewingville Creek	Ralston Creek	215	26	5	54	8	3	161	18	1
W4	North Ewingville Creek	Ralston Creek	595	96	14	149	29	10	446	67	4
W5	Liberty Creek	Liberty Creek	939	164	22	235	49	16	704	115	7
W6	Saw Mill Creek	Saw Mill Creek	6,093	936	150	1,523	281	105	4,570	655	45
W7	Donelson Creek	Donelson Creek	6,193	1,041	155	1,548	312	108	4,644	729	46
W8	Goose Creek	Five Mile Creek	916	176	29	229	53	21	687	123	9
Total			**16,899**	**2,763**	**420**	**4,225**	**829**	**294**	**12,674**	**1,934**	**126**

Table 8 – Estimated Total Project Costs for Identified BMPs

ID	Stream	Watershed	Project Description	Cost
W1	Sharps Branch	Sharps Branch	Detention Facility, 40 ac-ft of storage	$ 1,800,000
W2	Quarry Branch	Sharps Branch	Detention Facility, 30 ac-ft of storage	n/a
W3	North Ewingville Creek	Ralston Creek	Detention Facility, retrofit existing facility	$ 2,400,000
W4	North Ewingville Creek	Ralston Creek	Detention Facility, upstream of Stanwick Dr.	$ 800,000
W5	Liberty Creek	Liberty Creek	Detention Facility, 10 ac-ft of storage	$ 1,200,000
W6	Saw Mill Creek	Saw Mill Creek	Detention facility	$ 2,400,000
W7	Donelson Creek	Donelson Creek	Detention facility	$ 4,700,000
W8	Goose Creek	Five Mile Creek	Detention facility, 10 ac-ft storage, retrofit	$ 800,000
			ESTIMATED TOTAL CONSTRUCTION COST =	**$ 14,100,000**

5.0 Summary and Conclusions

The analyses presented in this memorandum include estimates of the potential for pollutant removal at seven previously identified BMP sites across the City. While the original projects were designed primarily to address flooding concerns, relatively minor changes in the outlet configurations of these BMPs may be included to achieve pollutant reductions goals stated in the TMDL. By comparing the total estimated cost of the projects to the estimated pollutant removal, estimates of unit costs and benefits of these projects may be developed.

- Cost per lb of TN removed: $10 per lb per day TN
- Cost per lb of TP removed: $25 per lb per day TP
- Cost per ton of Sediment removed: $0.06 per lb per day Sediment

These unit costs may be used for comparison of other pollutant removal strategies being considered as part of the Integrated Water Resources Plan to determine the most cost effective water quality improvement strategies.

6.0 References

Center for Watershed Protection (September 2007). National Pollutant Removal Performance Database, Version 3.0. Retrieved from: http://www.stormwaterok.net/CWP%20Documents/CWP-07%20Natl%20Pollutant%20Removal%20Perform%20Database.pdf

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Technical Memorandum

To: *City of Franklin IWRP Team*

From: *CDM*

Date: *July 29, 2011*

Subject: *Integrated Water Resources Plan - Feasibility of Robinson Lake Flow Augmentation*

1.0 Introduction

As part of the Integrated Water Resources Plan (IWRP) for the City of Franklin (City), Camp Dresser & McKee (CDM) has evaluated and developed a preliminary cost estimate for structural improvements to Robinson Lake for the purpose of augmenting seasonal low flows in the Harpeth River.

The Integrated Water Resources Plan Phase I Final Report (CDM, 2010) included an option to augment seasonal low flows in the Harpeth River by pulling water from Robinson Lake (the Lake) and discharging directly into the Harpeth River. The purpose of this memorandum is to determine the recommended flow rate and associated duration that the Lake can be drawn down to augment stream flow, provide a recommendation for the type of structure that can be used to draw down the Lake, and develop a conceptual opinion of probable cost to construct the structure.

2.0 Robinson Lake Dam Existing Conditions

CDM reviewed the following available information related to the existing Robinson Lake Dam:

- State of Tennessee Safe Dams Program website;
- USACE National Inventory of Dams (NID) website; and
- USGS topographic maps and aerial photos of the dam location.

Based upon this review, the existing dam is assumed to be an earth fill dam with a riprap/rock-lined spillway in the right abutment area discharging to the Harpeth River. The dam has a structural height of 22.5 feet and hydraulic height of 19 feet, with a storage capacity of 91 acre-feet at normal pool and 136 acre-feet at maximum pool. The downstream slope of the dam is covered with trees and other woody vegetation. The dam is identified on the Safe Dams database as being Significant or Low Hazard (HPC-2 or HPC-3). The storage capacity of Robinson Lake places the dam size category in SMALL. The dam was constructed in 1950 and it is not known if the dam has an existing low-level outlet pipe.

The NID database lists the dam as being owned by a private individual and is currently an unregulated structure. For this analysis, CDM assumed that this structure is currently unregulated because it is considered a "farm pond" as defined by Safe Dams. However, if this pond were to be used for any other purpose than providing water for agricultural, we assume it will become a regulated structure.

Proposed Improvements

The proposed improvements would raise normal pool in Robinson Lake to provide additional storage capacity upstream of the existing plant. The additional storage capacity would be obtained by raising the top of dam to provide an additional two feet of storage above existing normal pool and constructing a new intake in the lake with a drawdown elevation one foot below existing normal pool. The existing spillway would be replaced by a new emergency spillway to pass the regulated design storm (100-year or 1/3 Probable Maximum Precipitation (PMP), 6-hour duration) without overtopping the dam. The freeboard above the higher normal pool elevation (assumed to be at elevation 645 feet above mean sea level (MSL)) would need to contain the regulated design storm and run-up from wave action. It is assumed that the minimum freeboard height would be 4 feet resulting in a dam crest elevation of 649 feet MSL.

The additional storage volume and duration of flow augmentation were calculated based on the proposed structural improvements as shown in Table 1. The Harpeth River has a mean flow rate of 3.78 million gallons per day (5.85 cubic feet per second (cfs)) in September. The Lake has a surface area of approximately 11 acres (487,000 square feet) at the principal spillway, approximate elevation 645 feet. It is assumed for this evaluation that up to 1 foot of storage below the principal spillway and 2 feet of storage above the principle spillway can be used to augment stream flows, resulting in an available volume of 12.6 million gallons. **Table 1** summarizes the duration that stored water could be withdrawn from Robinson Lake based on varying flow rates (CDM, 2010). Robinson Lake, as a reservoir, is capable of providing flow to the Harpeth to raise the median low flow from 3.78 (5.85 cfs) to 6.46 MGD (10.0 cfs), which is the required minimum for City water withdrawals, for a period of 5.4 days.

Table 1 - Robinson Lake to Harpeth River Flow Augmentation Duration

Flow Rate (MGD)	% of Differential Flow[1]	Duration[2] (Days)
0.50	8%	28.8
1.00	48%	14.4
1.50	64%	9.6
2.00	80%	7.2
2.50	96%	5.8
2.68	100%	5.4

Notes:

1.) 2.68 MGD is difference between minimum river flow specified by the 2007 Aquatic Resource Alteration Permit of 6.46 MGD (10 cfs) and median low flow in September of 3.78 MGD (Phase I, 2010).

2.) Duration based on an available volume of 12.6 million gallons.

3.0 Conceptual Evaluation

The conceptual evaluation was based upon available data, which are extremely limited. The proposed modifications as described above should be technically feasible provided the following assumptions are valid:

- The existing crest of the dam provides adequate freeboard above the emergency spillway for the design storm.
- The existing dam is in good condition with adequate seepage measures and raising normal pool by 2 feet will not adversely impact the stability of the dam.
- The existing dam can be raised by 3 feet and still provide a crest width of 12 feet and stable upstream/downstream slopes (3H:1V preferred).
- Raising the crest height will require construction of a berm (less than 4 feet high) to tie in to existing grades at the abutments. There is adequate space to construct this berm with a crest width of at least 8 feet and upstream/downstream slopes of 3H:1V.
- Raising the normal pool and dam crest elevation will not result in flooding of adjacent properties/roads and will not require wetland or stream restoration.

The modifications proposed would require easement from the private owner and conversion of the dam to a regulated structure subject to Safe Dams requirements. A complete set of design documents prepared by registered professional engineer experienced in dam design will be

required for the permit through the Tennessee Safe Dams Program. The scope of work may include, but not be limited to, the following:

- Complete subsurface exploration and laboratory testing program;
- Seepage and stability analyses for the existing and proposed structures;
- Hydrologic and Hydraulic analyses to assess peak stages and size emergency spillway to meet freeboard requirements;
- Evaluation of any existing low-level outlet or other pipes through the dam;
- Permit Application for Safe Dams;
- Contract Documents for the Dam Modifications; and
- Construction observation by the Engineer of Record to satisfy Safe Dams post-construction certification.

Construction of the proposed modifications would likely include:

- Select fill to raise earthen embankment and construct abutment berms;
- Additional seepage control measures;
- Replacement of the low-level outlet (if needed);
- Removal of trees and other woody vegetation from the existing dam;
- Vegetation and riprap stabilization of the regraded dam slopes;
- New intake; and
- New emergency spillway channel.

4.0 Conceptual Opinion of Probable Cost

The conceptual opinion of probable cost is detailed in **Table 2** below, with a total of $990,000.

Table 2 - Siphon and Dam Installation Cost Estimate

Item	Quantity	Unit Price	Estimated Cost
Gated Intake Structure	$100,000	$100,000	$100,000
Lake Dewatering and Bypass	$25,000	$25,000	$25,000
Earthen Fill	$20,000	$20,000	$20,000
Structural Spillway	$100,000	$100,000	$100,000
Tree/Vegetation Removal	$25,000	$25,000	$25,000
Seepage/Slope Stabilization	$40,000	$40,000	$40,000
Engineering	$200,000	$200,000	$200,000
Permitting	$50,000	$50,000	$50,000
Property Easement	$100,000	$100,000	$100,000
		Subtotal =	$660,000
		50% Contingency =	$330,000
		Total =	**$990,000**

Based on this total estimated cost, the unit cost associated with augmenting Harpeth River seasonal low flows in the City of Franklin is $78,570 per million gallons.

5.0 Conclusions and Recommendations

Based on the above evaluation, alterations to the lake to augment streamflows would require a significant investment of nearly $1 million and will provide only five additional days of flow withdrawal capability at the plant. The benefits of this project do not appear to justify the cost. Before further consideration, this project should be evaluated in comparison to the cost vs. benefits of other projects being considered as part of the Integrated Water Resources Plan.

6.0 Conclusions and Recommendations

Camp Dresser and McKee (2010). Integrated Water Resources Plan Phase I Final Report, City of Franklin. Retrieved from: **http://www.franklin-gov.com/index.aspx?page=729**

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Technical Memorandum

To: *City of Franklin IWRP Team*

From: *CDM*

Date: *December 8, 2011*

Subject: *Integrated Water Resources Plan – Green Infrastructure and Low Impact Development Techniques and Opportunities*

1.0 Introduction

Green Infrastructure and/or Low Impact Development (LID) techniques are terms used to describe an array of management practices, products, and/or technologies that use natural systems and/or engineered systems to capture, manage, and reduce stormwater runoff volumes and enhance overall environmental quality. As a general principal, Green Infrastructure techniques use soils and vegetation to infiltrate, evapotranspirate, and/or recycle stormwater runoff. This approach is an enhancement of traditional stormwater controls to add more sustainable and cost effective solutions. When used as components of a stormwater management system, green practices such as green roofs, porous pavement, rain gardens, vegetated swales, and stream-riparian buffers can produce a variety of hydrologic and environmental benefits. In addition to effectively retaining and infiltrating rainfall, these technologies can simultaneously help control flooding and erosion, maintain and improve water quality, maintain baseflows and stream temperatures, promote healthy aquatic habitat and riparian corridors, filter air pollutants, reduce energy demands, mitigate urban heat islands, and sequester carbon while also providing communities with aesthetic benefits.

The Environmental Protection Agency (EPA) has made Green Infrastructure a key Administration priority, as stated in the "Strategic Agenda to Protect Waters and Build More Livable Communities through Green Infrastructure (EPA, 2011). Key policies and regulatory guidance were also provided by EPA in a guidance document developed to assist NPDES permit writers in developing municipal separate storm sewer system (MS4) stormwater permit language (EPA, 2010). The State of Tennessee has incorporated much of the guidance regarding green infrastructure into local MS4 permits.

For example, the City's current MS4 permit requires the following under Section 4.2.5.2:

> *Site design standards for all new and redevelopment require, in combination or alone, management measures that are designed, built and maintained to infiltrate, evapotranspire, harvest and/or use, at a minimum, the first inch of every rainfall event preceded by 72 hours of no measurable precipitation. This first inch of rainfall must be 100% managed with no storm water runoff being discharged to surface waters.*

This represents a shift by the State of Tennessee towards green infrastructure and low impact development practices. The City of Franklin has already incorporated the permit language into the local stormwater management ordinance, which becomes effective in July 2013. As such, the City wishes to evaluate the potential benefits of a green infrastructure program in relation to the Franklin IWRP project.

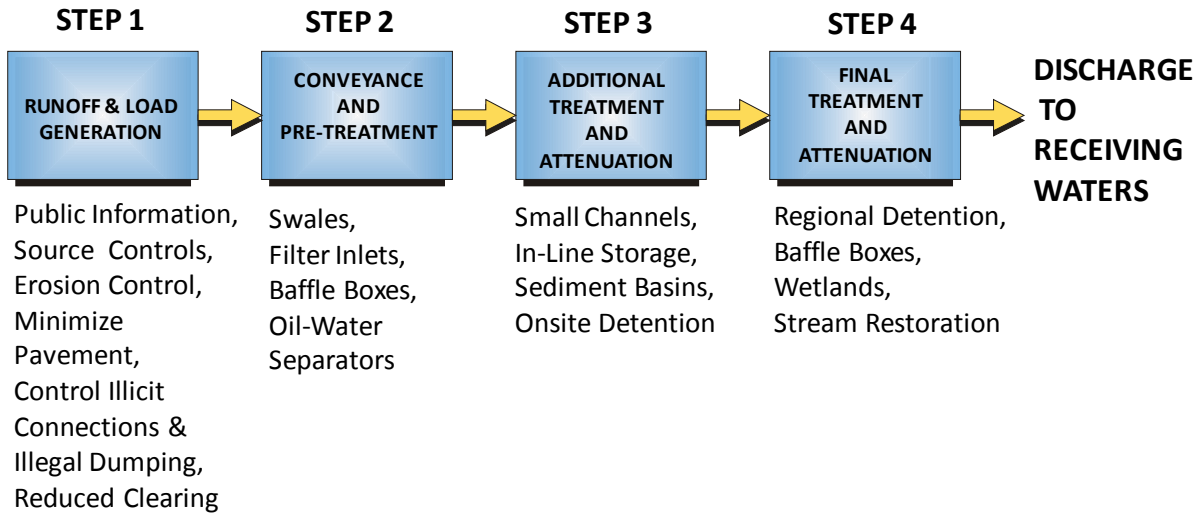
2.0 Data Collection

The City's current ordinance for stormwater management promotes the concept of the BMP Treatment Train, which is defined in the City's Stormwater Management Ordinance as:

> *"A technique for progressively selecting various Stormwater management practices to address water quality, by which groups of practices may be used to achieve a treatment goal while optimizing effectiveness, maintenance needs and space.*

The goal of the BMP Treatment Train, which is depicted in Figure 1, is treatment for the runoff resulting from a rainfall depth of 1.1 inches. Treatment at this level is predicted to achieve a goal of ninety percent (90%) capture of the average annual rainfall volume.

Figure 1 – BMP Treatment Train

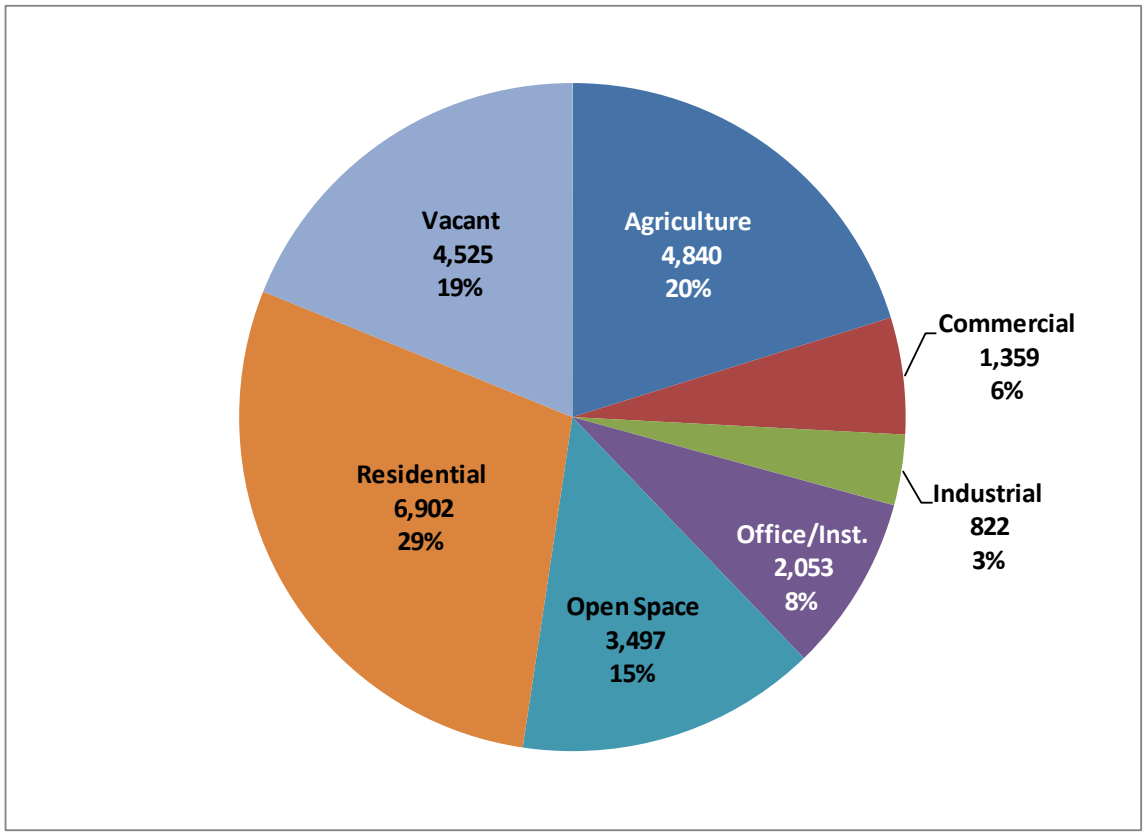


While the present approach used by Franklin is effective at meeting the City's goals, the current ordinance does not achieve the new regulatory standard set forth in the NPDES Phase II MS4 permit. The primary difference is the current ordinance requires only "capture and treatment" of the first 1.1 inches of rainfall while the NPDES Permit requires "capture and/or reuse" of the first inch. This memorandum presents an approach to approximate the additional benefits in pollutant reduction gained by moving to the new standard.

2.1 Estimate of Available Land for Development

Since the green infrastructure requirements apply to new development and redevelopment CDM estimated the approximate amount of land in Franklin still available for development. Using the City's most recent GIS data, CDM summarized the various land use types within the city limits, which is provided in Figure 2.

Figure 2 – Summary of Current Land Uses in Franklin



For the purposes of this analysis, CDM considered that Vacant (4,525 acres) and Agriculture (4,840 acres) lands would likely be available for future development. Consideration was given to the Open Space category, but it was expected that much of that land may be dedicated open space and therefore not available for development. Redevelopment potential was not considered in the analysis, but a similar BMP Treatment Train approach can be used for these retrofit cases. As shown, these evaluations result in a total of approximately 9,400 acres of land for use in the comparative analysis of traditional controls versus green infrastructure/LID controls.

3.0 Pollutant Loading Analysis

CDM performed a desktop pollutant loading analysis for three land use scenarios to determine the additional benefits gained by implementing a green infrastructure/LID development ordinance in Franklin. For consistency with previous pollutant loading analyses performed for this project, the EPA Spreadsheet Tool for Estimating Pollutant Load (STEPL) was used to generate desktop estimates of pollutant loads for each of the following three scenarios:

1. Build-Out with No Controls
2. Build-Out with Traditional BMP Controls (i.e. current ordinance)
3. Build-Out with GI/LID Controls (i.e. revised ordinance per NPDES requirements)

The STEPL tool uses a combination of annual rainfall estimates, land use information and event mean concentration (EMC) data to estimate annual pollutant loads. The spreadsheet tool used average annual rainfall data from the closest weather gage (Nashville BNA airport), which was estimated at approximately 59 inches annually.

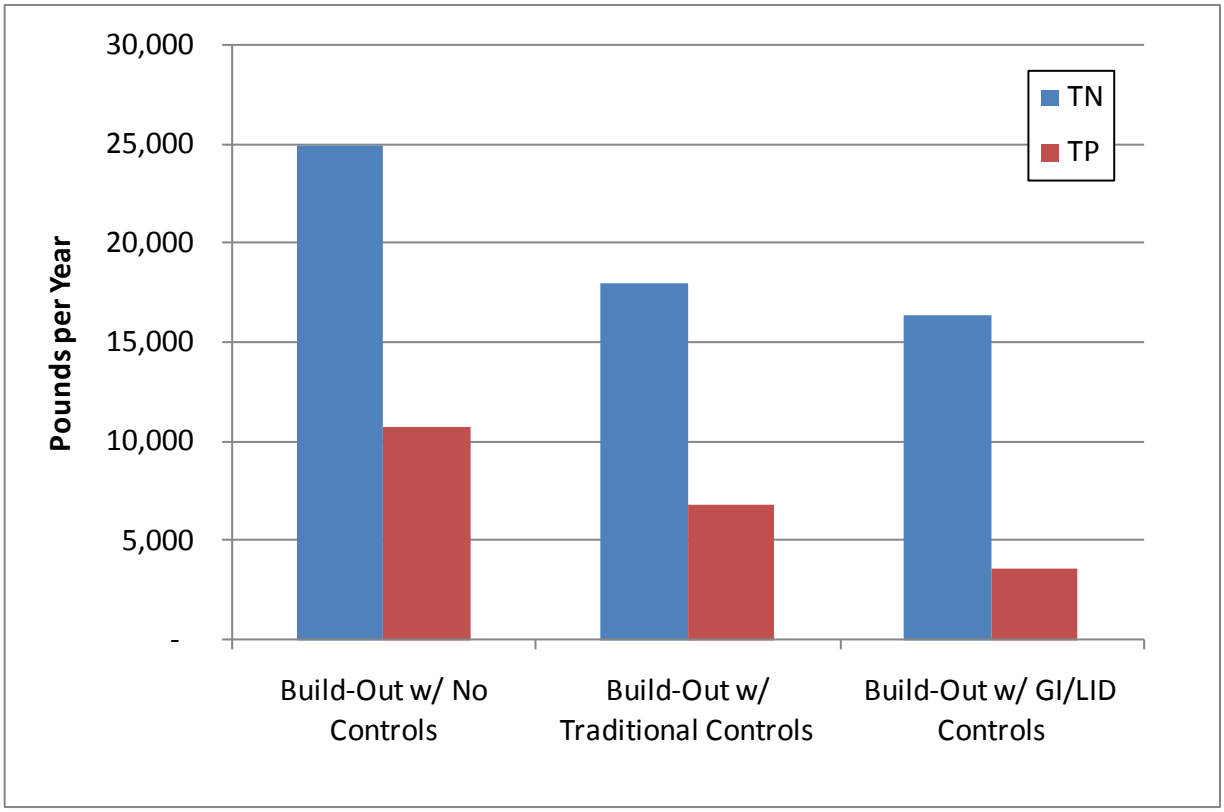
The land uses for each scenario were based on the data provide in Figure 2 above. For all three scenarios, the available 9,400 acres was evaluated using the same percent land use distribution that exists in the City today. The standard, default EMC values provided in the spreadsheet tool were applied to each land use to generate a pollutant loading. The EMC values were consistent with national averages and others used for this project. Table 1 provides a summary of the EMCs for the relevant pollutants of concern used in the spreadsheet tool. Total Nitrogen is represented by TN and Total Phosphorus is represented by TP.

Table 1 – EMC Values for Various Pollutants and Land Uses (mg/L)

Pollutant	Comm.	Indust.	Instit.	Trans.	Multi-Family	Single-Family	Open Space
TN	0.62	0.75	0.62	0.28	0.58	0.58	0.85
TP	0.23	0.27	0.23	0.25	0.31	0.31	0.33

Using the STEPL tool with the values defined in the above tables, CDM generated a total pollutant load for the Build-Out Scenario with No Controls. Next, CDM used the STEPL tool to apply traditional stormwater controls to the 9,400 acres. The "wet detention" scenario in STEPL was used for this analysis. Finally, CDM applied the LID/Bioretention BMP Tool in the STEPL model to represent the Green Infrastructure/LID scenario. Figure 3 summarizes the results of this analysis.

Figure 3 – Summary of Pollutant Loads for BMP Scenarios



4.0 Summary and Conclusions

While the scenario using traditional stormwater controls provides significant pollutant reduction over the Build-Out with No Controls scenario, the analysis shows that additional benefits will be gained when the City implements the green infrastructure/LID ordinance required by the NPDES Phase II MS4 permit. Table 2 summarizes the predicted pollutant load reductions for each scenario.

Table 2 – Pollutant Load Reductions for Each Scenario

Scenario	% TN Reduction	% TP Reduction
Build-Out w/ No Controls	--	--
Build-Out w/ Traditional Controls	28%	37%
Build-Out w/ GI/LID Controls	34%	66%

The potential reductions in TN will be consistent with the TMDL for the Harpeth which is based on low dissolved oxygen (DO). In addition to water quality and TMDLs, additional benefits of green or LID approaches are:

- Reduced clearing and use of native vegetation which requires less irrigation,
- Protection for riparian corridors and habitat
- Reduced stream erosion and sedimentation,
- Reduced flooding from less runoff generation and protection of floodplain with the riparian corridors,
- Maintenance of baseflow during dry weather conditions, and
- Direct use of runoff for irrigation of planted areas, which reduces potable water demand and generation of wastewater flows for treatment,

The pollutant load reductions estimated in this memorandum will be incorporated into the STELLA model as an alternative stormwater practice. The cost for this option is minimal to the City since the regulatory-required controls will be implemented through new development. The City's anticipated costs will be associated with plan review and enforcement activities. It has been estimated that approximately 0.5 FTEs will be required for this activity. Consistent with estimates on previous IWRP alternatives, the cost for 0.5 FTEs at a technician level is approximately $25,000 (including overhead).

5.0 References

Environmental Protection Agency (May 2010). MS4 Improvement Guide. Retrieved from:
http://www.epa.gov/npdes/pubs/ms4permit_improvement_guide.pdf

Environmental Protection Agency (April 2011). A Strategic Agenda to Protect Waters and Build More Livable Communities Through Green Infrastructure. Retrieved from:
http://www.epa.gov/npdes/pubs/gi_agenda_protectwaters.pdf

Environmental Protection Agency (November 2011). Managing Wet Weather with Green Infrastructure. Retrieved from:
http://cfpub.epa.gov/npdes/home.cfm?program_id=298

City of Franklin, Tennessee

Integrated Water Resources Plan

Appendix G:

Reclaimed Water

- Reclaimed Water Demand Projections
- Reclaimed Water I-65 Corridor
- Franklin Reclaimed System Storage - ultimate demand

CDM Smith

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Seckman
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Memo

DATE: March 7, 2011

TO: 10-41-025.0 File (1)

CC: Andrew Lynn - CDM

FROM: Bo Butler

RE: Franklin IWRP – Reclaimed Water Demand Projections

Nashville
Houston
Fort Lauderdale
Sarasota
Memphis
Phoenix
Dallas

The Franklin Reclaimed System Report, dated February 2009 was updated for current population and development projections. Residential and commercial developments were re-evaluated for progress that has been made since 2009. The Franklin Development Report, prepared by the Franklin Planning Dept., was used to gauge progress and to project development for the next few years. The tables in the 2009 report have been combined where appropriate and narratives updated as included in this memo.

Table 2 lists existing system customers as of March 1, 2011. Average daily and maximum daily demands have been taken from reclaimed water sales records obtained from the City of Franklin where possible. Other flows are estimates based on potential irrigated area. However, some of the customers in this list have, after initial use, either cut back or discontinued use of reclaimed water because of significant rainfall or problems in their individual storage and on-site irrigation systems. It is anticipated during dry periods of the year that all eight of these customers will be on-line and using the water as shown in the demand chart.

Updated average daily demand for existing customers is estimated to be 1.55 mgd, with maximum daily demands approximately 6.85 mgd.

Table 2

Franklin Reclaimed Water System					
Existing System Customers (as of 5/1/2008)					
No.	**Name**	**Customer Type**	**Ave. Daily Demand (GPD)**	**Max Daily Demand (GPD)**	**Distribution Line**
1	Legends Club	Recreational	500,000	2,100,000	Legends
2	Westhaven	Residential, Recreational	117,000	2,000,000	Westhaven
3	Franklin High School	Institutional	156,000	525,000	Westhaven
4	BGA	Institutional	150,000	385,000	Legends
5	Harpeth Village Merchants	Commercial	77,000	150,000	Westhaven
6	Carlisle Lane Residents	Residential	100,000	250,000	Westhaven
7	John Deere Landscaping	Commercial	385,000	1,300,000	Westhaven
8	Williamson County Rec.	Recreation	60,000	124,000	Westhaven
Totals			**1,545,000**	**6,834,000**	

Table 3 lists potential system customers for the existing Franklin reclaimed water system as of March 1, 2011. In re-evaluating the projected demands for these potential users, projections were revised based on a 2-inch per week irrigation rate, which is what many area golf course and park facilities use for irrigation rates of their green spaces. As a result, the average daily demand for these potential customers has increased to 3.76 mgd. The projected maximum daily demand has increased to 8.13 mgd for these potential customers.

A new drive-by survey of the Columbia Avenue Industrial District revealed that Franklin is currently continuing to lose industries. The APCOM Corporation and Georgia Boot have both closed, and while the remaining businesses might use reclaimed water for irrigation of their lawns and open spaces, it is anticipated that most reclaim use would be seen as cooling tower make-up water for the buildings associated with these businesses. The complex previously housing the CPS Corporation is being partially used, but not for heavy industry. It is not anticipated that a large, heavy industry will occupy this location in the future.

In discussions with design professionals responsible for the design of cooling systems for large-scale buildings, it is known that water quality plays a significant role in longevity of the cooling tower equipment. Suspended solids and the presence of chlorine in cooling tower make-up water may have an adverse effect on pipes and pumps that are used in these systems. After an industrial customer purchases reclaimed water, it may be necessary to filter or provide chemical addition to water to make it more suitable for use as cooling tower make-up water. Given the significant cost savings of reclaimed water over potable water, individual industrial customers may find it advantageous to test the water for use in their individual cooling systems.

A number of the listed potential customers are served by the West Reclaimed Water Line and Downs Boulevard Reclaimed Water Line. These two projects are currently under construction and are scheduled for completion by late May of this year. The existing residential subdivisions listed in this table are not yet equipped with a reclaimed water distribution system. For this reason, it is not anticipated that these subdivisions could easily connect to the reclaimed water system for irrigation purposes for individual households. However, open spaces and green spaces could be easily connected to the reclaimed water system at a more reasonable cost. Jim Warren Park and the Williamson County Soccer Complex are currently equipped with large-scale irrigation systems, and it is anticipated that both of these parks could change over to reclaimed water at a minimal cost.

Vulcan Materials, Williamson County Ready Mix, and Sherman Dixie Concrete are all industrial customers. It is anticipated that each of these businesses could connect to and use reclaimed water at a minimal cost and realize a significant discount for volume purchases.

Many of the potential customers listed could be served by the Goose Creek reclaimed water line and Five-Mile Creek reclaimed water lines which were completed in 2009. A listing of these properties includes all the residential properties that are proposed for

development along the southern border of Goose Creek. Some limited development has taken place at the Ladd Park and Stream Valley subdivisions. However, other properties are awaiting economic recovery before development continues. All of these development projects are listed in a 2009 Franklin Development Report prepared by the Franklin Planning Department, and it is anticipated that all of these properties will develop in the future as economic conditions improve.

Table 3

Franklin Reclaimed Water System					
Potential System Customers (as of 5/1/2008)					
No.	**Name**	**Customer Type**	**Ave. Daily Demand (GPD)**	**Max Daily Demand (GPD)**	**Distribution Line**
1	Fairways at Spencer Creek	Recreational	250,000	350,000	Westhaven
2	Pinkerton Park	Municipal	84,000	170,000	Southeast
3	Forest Crossing	Recreational	500,000	2,000,000	Southeast
4	Battlefield Park	Municipal	295,000	600,000	Southeast
5	Franklin Street Dept.	Municipal	100,000	200,000	Southeast
6	Columbia Avenue Industrial	Industrial	90,000	90,000	Southeast
7	FSSD	Institutional	25,000	50,000	West Line
8	Jim Warren Park	City Parks	100,000	200,000	West Line
9	Williamson County Soccer	County Parks	240,000	500,000	West Line
10	New Hope Academy	Institutional	20,000	50,000	West Line
11	Founders Point	Residential	100,000	200,000	West Line
12	Reid Hill	Residential	20,000	90,000	West Line

13	Williamson County Redi-Mix	Industrial	100,000	150,000	Downs Blvd
14	Vulcan Materials	Industrial	75,000	150,000	Downs Blvd
15	Sherman-Dixie	Industrial	100,000	200,000	Downs Blvd
16	Ladd Park	Residential	440,000	900,000	Goose Creek
17	Berry Farms	Mixed Use	330,000	680,000	Goose Creek
18	Stream Valley	Residential	250,000	500,000	Goose Creek
19	Rural Pains	Mixed Use	80,000	150,000	Goose Creek
20	Rheems Fleming	Mixed Use	75,000	100,000	Goose Creek
21	Nichol's Bend	Residential	100,000	150,000	Goose Creek
22	Lockwood	Residential	160,000	300,000	Goose Creek
23	Ag Center	Institutional	30,000	50,000	Goose Creek
24	Existing Residential (Redwing Farms Area)	Residential	200,000	300,000	Goose Creek
Totals			**3,764,000**	**8,130,000**	

The re-evaluation of development and population projections yields a total average daily demand for the Franklin reclaimed water system at **5.31 mgd** with a total maximum daily demand of **14.96 mgd**. Although the timeframe for connection to the reclaimed water system will depend to a great extent on economic conditions, it is anticipated that most of the potential customers listed will be in a position to want reclaimed water for their developments within the next 10 years. The location of existing distribution lines in relation to many of these properties would 5-year connection projections for additional customers as shown in Table 4 below. It is anticipated that the additional average daily demand from these potential customers will be 1.9 mgd with additional maximum daily demands approaching 4.6 mgd.

Table 4

Franklin Reclaimed Water System					
Potential System Customers – Projected Connections by Year 2016					
No.	**Name**	**Customer Type**	**Ave. Daily Demand (GPD)**	**Max Daily Demand (GPD)**	**Distribution Line**
1	Fairways at Spencer Creek	Recreational	250,000	350,000	Westhaven
2	Pinkerton Park	Municipal	84,000	170,000	Southeast
3	Forest Crossing	Recreational	500,000	2,000,000	Southeast
4	Battlefield Park	Municipal	295,000	600,000	Southeast
5	Franklin Street Dept.	Municipal	100,000	200,000	Southeast
6	FSSD	Institutional	25,000	50,000	West Line
7	Jim Warren Park	City Parks	100,000	200,000	West Line
8	Williamson County Soccer	County Parks	240,000	500,000	West Line
9	Williamson County Redi-Mix	Industrial	100,000	150,000	Downs Blvd
10	Vulcan Materials	Industrial	75,000	150,000	Downs Blvd
11	Sherman-Dixie	Industrial	100,000	200,000	Downs Blvd
12	Ag Center	Institutional	30,000	50,000	Goose Creek
Totals			**1,899,000**	**4,620,000**	

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Memo

DATE: March 10, 2011

TO: 10-41-025.0 File (1)

CC: Andrew Lynn - CDM

FROM: Bo Butler

RE: Franklin IWRP – Reclaimed Water Demand Projections
I-65 Corridor

Nashville
Houston
Fort Lauderdale
Sarasota
Memphis
Phoenix
Dallas

An additional analysis of the properties along Interstate 65 that might be served by the proposed I-65 Connector reclaimed water line was completed. The existing residential properties along this corridor were not included because serving individual households would require an additional distribution system to be installed within each subdivision that would be served. It is not anticipated that the City will require this type of retrofit in order to bring additional customers online. It is also not anticipated that existing residential subdivisions will wish to incur the expense of installing an additional system in their areas. However, all undeveloped properties were included and a 20% factor was applied to the total acreage that might more accurately describe the actual irrigated area that would be served, given that most of the remaining properties are set for commercial development. An irrigation rate of 2 inches per week was applied to the corrected irrigated area for each section defined in Table A below.

The analysis yields an additional average daily flow of 1,608,500 gallons per day with a peak flow of 3,260,000 gpd.

Table A

	Franklin Reclaimed Water System					
	Potential Demand – Interstate 65 Corridor					
No.	**Name**	**Customer Type**	**Irrigated Area (acres)**	**Ave. Daily Demand (GPD)**	**Max. Daily Demand (GPD)**	**Distribution Line**
1	Liberty Park	Recreational	63	240,000	489,000	I-65
2	Properties- Cool Springs Blvd to Moore's Lane	Residential, Commercial	98	375,000	760,300	I-65
3	Properties- McEwen to Cool Springs Blvd	Residential, Commercial	120	460,000	931,000	I-65
4	Properties- McEwen to Murfreesboro Rd.	Residential, Commercial	35	134,000	272,000	I-65
5	Properties – S. Carothers Road	Residential, Commercial	10	38,500	78,000	I-65
6	Liberty Park Area Properties	Residential, Commercial	68	261,000	528,000	I-65
7	Centennial High School	Institutional	26	100,000	202,000	I-65
Totals			**420**	**1,608,500**	**3,260,000**	

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Memorandum

DATE: April 21, 2011

TO: Andrew Lynn – CDM; 10-41025.0 File 3C

CC:

FROM: Bo Butler

RE: Franklin Reclaimed System Storage – ultimate demand

Nashville
Houston
Fort Lauderdale
Sarasota
Memphis
Phoenix
Dallas

As requested, we have analyzed the actual reclaimed distribution system data from 2010 and projected the ultimate flows, based on the previously submitted demand tables. Assumptions:

1. The reclaimed water pump station is configured to deliver 12 GMD to the distribution system.
2. There are no existing significant storage areas within the system.
    a. There is one 500,000 gallon tank available for the system in the Goose Creek area. This tank is assumed to provide additional pressure as required, but minimal volume.
3. 2010 demands were factored by 4.5 to bring the maximum daily demand during the dry season to approximately 15 MGD.

The existing demands as shown in Table 2 of the previously submitted demand memo, attached, can be accommodated by the reclaimed water pump station in its present 6MGD configuration. (The station has been shown to accommodate up to 7 MGD as configured.)

Demands were updated to include additional properties in the Cool Springs area as well as to accommodate previously undeveloped properties city-wide, and updated incremental demand flows. These demands are shown in Table 3 of the attached memo. The projected average day demand is show to be approximately 5.3 MGD, and the maximum demand is approximately 15 MGD.

The actual demands from year 2010 were used and factored to created a typical yearly flow scenario for the system as shown in the table titled Projected Ultimate Flows, attached. The chart titles Excess Supply shows the graphical representation of supply less demand for each day of the projected ultimate year. The dry periods of the year were analyzed and the supply deficit volumes were calculated to find the greatest storage requirement for the year. The data shows that, from the middle of July through the end of October, a deficit of approximately 240 million gallons exists. This storage volume would need to be provided to accommodate the projected demands to the system as shown in Table 3, assuming no additional pumping capacity is provided.

It is assumed that storage would take place at the Water Reclamation Facility as well as at Forest Crossing Golf Course, Legends Club Golf Course and the Westhaven Golf Course. The volumes of the storage lakes at these facilities is unknown at this time.

City of Franklin Reclaimed Water Distribution
Projected Ultimate Flows

Month	Day	Demand	Supply	Difference	Cumulative Supply	
		Gallons	Gallons	Gallons		
January	1	855,000	12,000,000	11,145,000	11,145,000	
	2	675,000	12,000,000	11,325,000	22,470,000	
	3	315,000	12,000,000	11,685,000	34,155,000	
	4	3,015,000	12,000,000	8,985,000	43,140,000	
	5	2,970,000	12,000,000	9,030,000	52,170,000	
	6	4,905,000	12,000,000	7,095,000	59,265,000	
	7	4,995,000	12,000,000	7,005,000	66,270,000	
	8	4,770,000	12,000,000	7,230,000	73,500,000	
	9	4,905,000	12,000,000	7,095,000	80,595,000	
	10	5,130,000	12,000,000	6,870,000	87,465,000	
	11	5,265,000	12,000,000	6,735,000	94,200,000	
	12	2,115,000	12,000,000	9,885,000	104,085,000	
	13	540,000	12,000,000	11,460,000	115,545,000	
	14	720,000	12,000,000	11,280,000	126,825,000	
	15	990,000	12,000,000	11,010,000	137,835,000	
	16	1,035,000	12,000,000	10,965,000	148,800,000	
	17	585,000	12,000,000	11,415,000	160,215,000	
	18	2,295,000	12,000,000	9,705,000	169,920,000	
	19	765,000	12,000,000	11,235,000	181,155,000	
	20	4,275,000	12,000,000	7,725,000	188,880,000	
	21	2,475,000	12,000,000	9,525,000	198,405,000	
	22	585,000	12,000,000	11,415,000	209,820,000	
	23	4,185,000	12,000,000	7,815,000	217,635,000	
	24	2,925,000	12,000,000	9,075,000	226,710,000	
	25	180,000	12,000,000	11,820,000	238,530,000	
	26	135,000	12,000,000	11,865,000	250,395,000	
	27	180,000	12,000,000	11,820,000	262,215,000	
	28	225,000	12,000,000	11,775,000	273,990,000	
	29	225,000	12,000,000	11,775,000	285,765,000	
	30	225,000	12,000,000	11,775,000	297,540,000	
	31	225,000	12,000,000	11,775,000	309,315,000	
February	1	225,000	12,000,000	11,775,000	321,090,000	
	2	135,000	12,000,000	11,865,000	332,955,000	
	3	90,000	12,000,000	11,910,000	344,865,000	
	4	90,000	12,000,000	11,910,000	356,775,000	
	5	90,000	12,000,000	11,910,000	368,685,000	
	6	90,000	12,000,000	11,910,000	380,595,000	
	7	90,000	12,000,000	11,910,000	392,505,000	
	8	225,000	12,000,000	11,775,000	404,280,000	
	9	270,000	12,000,000	11,730,000	416,010,000	
	10	270,000	12,000,000	11,730,000	427,740,000	
	11	270,000	12,000,000	11,730,000	439,470,000	
	12	225,000	12,000,000	11,775,000	451,245,000	
	13	225,000	12,000,000	11,775,000	463,020,000	
	14	225,000	12,000,000	11,775,000	474,795,000	
	15	225,000	12,000,000	11,775,000	486,570,000	
	16	90,000	12,000,000	11,910,000	498,480,000	
	17	90,000	12,000,000	11,910,000	510,390,000	
	18	45,000	12,000,000	11,955,000	522,345,000	
	19	135,000	12,000,000	11,865,000	534,210,000	
	20	135,000	12,000,000	11,865,000	546,075,000	
	21	135,000	12,000,000	11,865,000	557,940,000	
	22	90,000	12,000,000	11,910,000	569,850,000	
	23	855,000	12,000,000	11,145,000	580,995,000	
	24	1,395,000	12,000,000	10,605,000	591,600,000	
	25	1,395,000	12,000,000	10,605,000	602,205,000	

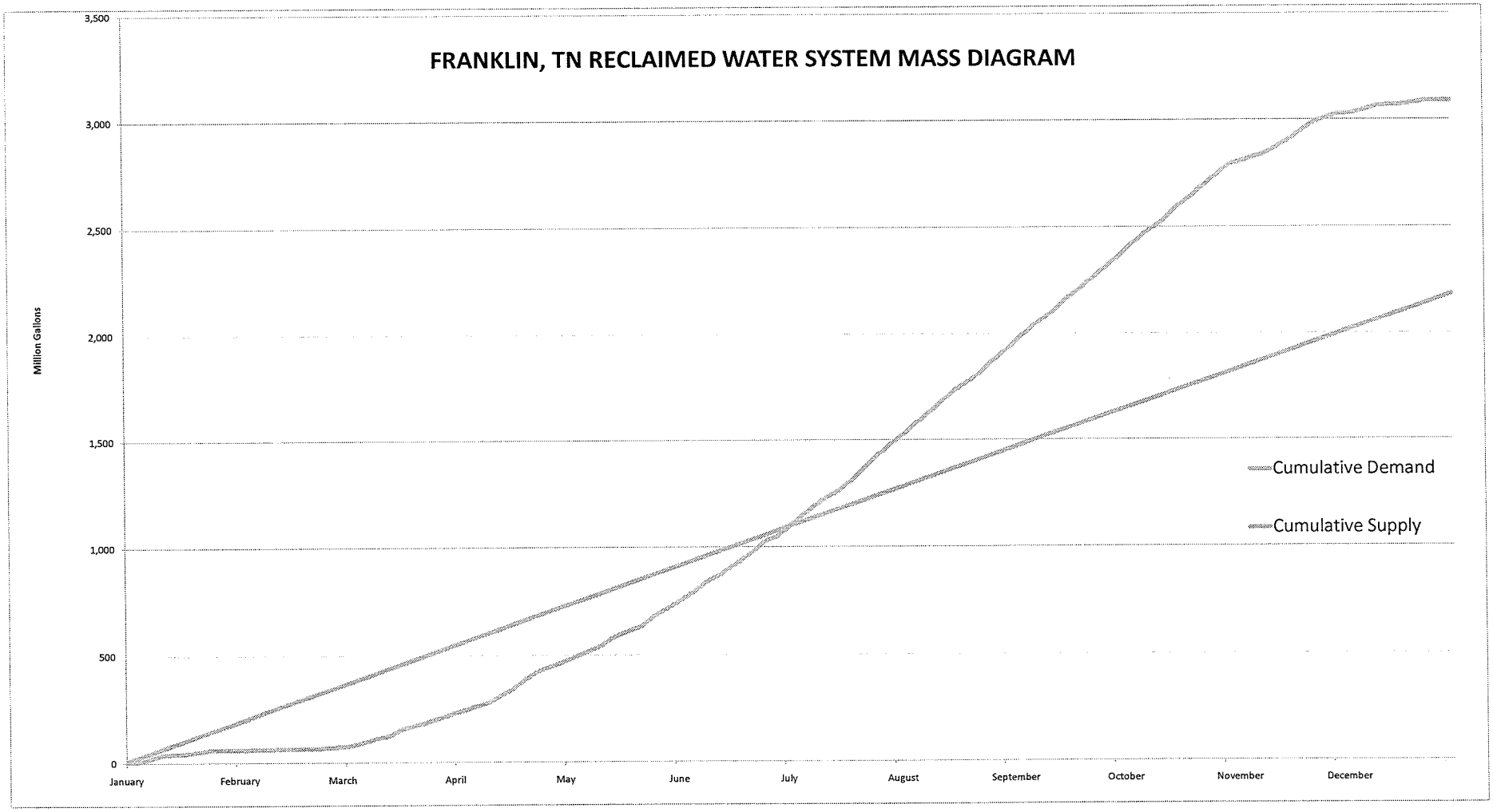
	26	1,395,000	12,000,000	10,605,000	612,810,000	
	27	1,395,000	12,000,000	10,605,000	623,415,000	
	28	1,395,000	12,000,000	10,605,000	634,020,000	
March	1	1,620,000	12,000,000	10,380,000	644,400,000	
	2	360,000	12,000,000	11,640,000	656,040,000	
	3	1,575,000	12,000,000	10,425,000	666,465,000	
	4	4,410,000	12,000,000	7,590,000	674,055,000	
	5	4,185,000	12,000,000	7,815,000	681,870,000	
	6	4,095,000	12,000,000	7,905,000	689,775,000	
	7	4,185,000	12,000,000	7,815,000	697,590,000	
	8	4,590,000	12,000,000	7,410,000	705,000,000	
	9	5,670,000	12,000,000	6,330,000	711,330,000	
	10	5,175,000	12,000,000	6,825,000	718,155,000	
	11	5,355,000	12,000,000	6,645,000	724,800,000	
	12	3,150,000	12,000,000	8,850,000	733,650,000	
	13	2,925,000	12,000,000	9,075,000	742,725,000	
	14	3,645,000	12,000,000	8,355,000	751,080,000	
	15	8,460,000	12,000,000	3,540,000	754,620,000	
	16	10,305,000	12,000,000	1,695,000	756,315,000	
	17	9,675,000	12,000,000	2,325,000	758,640,000	
	18	4,680,000	12,000,000	7,320,000	765,960,000	
	19	5,445,000	12,000,000	6,555,000	772,515,000	
	20	4,815,000	12,000,000	7,185,000	779,700,000	
	21	4,680,000	12,000,000	7,320,000	787,020,000	
	22	3,645,000	12,000,000	8,355,000	795,375,000	
	23	5,220,000	12,000,000	6,780,000	802,155,000	
	24	5,445,000	12,000,000	6,555,000	808,710,000	
	25	4,455,000	12,000,000	7,545,000	816,255,000	
	26	5,535,000	12,000,000	6,465,000	822,720,000	
	27	5,175,000	12,000,000	6,825,000	829,545,000	
	28	4,995,000	12,000,000	7,005,000	836,550,000	
	29	5,220,000	12,000,000	6,780,000	843,330,000	
	30	5,220,000	12,000,000	6,780,000	850,110,000	
	31	5,760,000	12,000,000	6,240,000	856,350,000	
April	1	5,445,000	12,000,000	6,555,000	862,905,000	
	2	5,310,000	12,000,000	6,690,000	869,595,000	
	3	4,500,000	12,000,000	7,500,000	877,095,000	
	4	5,130,000	12,000,000	6,870,000	883,965,000	
	5	3,150,000	12,000,000	8,850,000	892,815,000	
	6	8,865,000	12,000,000	3,135,000	895,950,000	
	7	4,905,000	12,000,000	7,095,000	903,045,000	
	8	4,905,000	12,000,000	7,095,000	910,140,000	
	9	5,265,000	12,000,000	6,735,000	916,875,000	
	10	5,220,000	12,000,000	6,780,000	923,655,000	
	11	8,055,000	12,000,000	3,945,000	927,600,000	
	12	9,630,000	12,000,000	2,370,000	929,970,000	
	13	9,585,000	12,000,000	2,415,000	932,385,000	
	14	10,665,000	12,000,000	1,335,000	933,720,000	
	15	10,350,000	12,000,000	1,650,000	935,370,000	
	16	6,840,000	12,000,000	5,160,000	940,530,000	
	17	10,800,000	12,000,000	1,200,000	941,730,000	
	18	13,095,000	12,000,000	-1,095,000	940,635,000	
	19	13,680,000	12,000,000	-1,680,000	938,955,000	
	20	13,005,000	12,000,000	-1,005,000	937,950,000	
	21	11,880,000	12,000,000	120,000	938,070,000	
	22	11,970,000	12,000,000	30,000	938,100,000	
	23	10,260,000	12,000,000	1,740,000	939,840,000	
	24	8,460,000	12,000,000	3,540,000	943,380,000	
	25	8,325,000	12,000,000	3,675,000	947,055,000	
	26	6,165,000	12,000,000	5,835,000	952,890,000	
	27	5,760,000	12,000,000	6,240,000	959,130,000	
	28	5,940,000	12,000,000	6,060,000	965,190,000	

	29	5,985,000	12,000,000	6,015,000	971,205,000	
	30	6,615,000	12,000,000	5,385,000	976,590,000	
May	1	7,425,000	12,000,000	4,575,000	981,165,000	
	2	7,560,000	12,000,000	4,440,000	985,605,000	
	3	7,065,000	12,000,000	4,935,000	990,540,000	
	4	7,605,000	12,000,000	4,395,000	994,935,000	
	5	6,480,000	12,000,000	5,520,000	1,000,455,000	
	6	7,200,000	12,000,000	4,800,000	1,005,255,000	
	7	6,930,000	12,000,000	5,070,000	1,010,325,000	
	8	6,705,000	12,000,000	5,295,000	1,015,620,000	
	9	6,660,000	12,000,000	5,340,000	1,020,960,000	
	10	6,435,000	12,000,000	5,565,000	1,026,525,000	
	11	9,990,000	12,000,000	2,010,000	1,028,535,000	
	12	11,700,000	12,000,000	300,000	1,028,835,000	
	13	12,240,000	12,000,000	-240,000	1,028,595,000	
	14	9,405,000	12,000,000	2,595,000	1,031,190,000	
	15	9,135,000	12,000,000	2,865,000	1,034,055,000	
	16	9,180,000	12,000,000	2,820,000	1,036,875,000	
	17	6,255,000	12,000,000	5,745,000	1,042,620,000	
	18	6,660,000	12,000,000	5,340,000	1,047,960,000	
	19	6,525,000	12,000,000	5,475,000	1,053,435,000	
	20	6,570,000	12,000,000	5,430,000	1,058,865,000	
	21	6,390,000	12,000,000	5,610,000	1,064,475,000	
	22	6,435,000	12,000,000	5,565,000	1,070,040,000	
	23	12,375,000	12,000,000	-375,000	1,069,665,000	
	24	13,590,000	12,000,000	-1,590,000	1,068,075,000	
	25	13,860,000	12,000,000	-1,860,000	1,066,215,000	-3,825,000
	26	11,115,000	12,000,000	885,000	1,067,100,000	
	27	10,035,000	12,000,000	1,965,000	1,069,065,000	
	28	9,720,000	12,000,000	2,280,000	1,071,345,000	
	29	9,900,000	12,000,000	2,100,000	1,073,445,000	
	30	9,990,000	12,000,000	2,010,000	1,075,455,000	
	31	9,720,000	12,000,000	2,280,000	1,077,735,000	
June	1	10,215,000	12,000,000	1,785,000	1,079,520,000	
	2	11,655,000	12,000,000	345,000	1,079,865,000	
	3	10,440,000	12,000,000	1,560,000	1,081,425,000	
	4	10,530,000	12,000,000	1,470,000	1,082,895,000	
	5	10,665,000	12,000,000	1,335,000	1,084,230,000	
	6	12,375,000	12,000,000	-375,000	1,083,855,000	
	7	12,870,000	12,000,000	-870,000	1,082,985,000	
	8	14,760,000	12,000,000	-2,760,000	1,080,225,000	-4,005,000
	9	10,800,000	12,000,000	1,200,000	1,081,425,000	
	10	9,630,000	12,000,000	2,370,000	1,083,795,000	
	11	8,325,000	12,000,000	3,675,000	1,087,470,000	
	12	8,280,000	12,000,000	3,720,000	1,091,190,000	
	13	9,225,000	12,000,000	2,775,000	1,093,965,000	
	14	12,600,000	12,000,000	-600,000	1,093,365,000	
	15	13,860,000	12,000,000	-1,860,000	1,091,505,000	
	16	12,060,000	12,000,000	-60,000	1,091,445,000	
	17	10,575,000	12,000,000	1,425,000	1,092,870,000	
	18	9,900,000	12,000,000	2,100,000	1,094,970,000	
	19	11,970,000	12,000,000	30,000	1,095,000,000	
	20	11,070,000	12,000,000	930,000	1,095,930,000	
	21	12,240,000	12,000,000	-240,000	1,095,690,000	
	22	14,850,000	12,000,000	-2,850,000	1,092,840,000	
	23	15,480,000	12,000,000	-3,480,000	1,089,360,000	
	24	14,715,000	12,000,000	-2,715,000	1,086,645,000	-9,285,000
	25	11,700,000	12,000,000	300,000	1,086,945,000	
	26	9,315,000	12,000,000	2,685,000	1,089,630,000	
	27	5,400,000	12,000,000	6,600,000	1,096,230,000	
	28	6,390,000	12,000,000	5,610,000	1,101,840,000	
	29	13,050,000	12,000,000	-1,050,000	1,100,790,000	

	30	12,465,000	12,000,000	-465,000	1,100,325,000	
July	1	13,410,000	12,000,000	-1,410,000	1,098,915,000	
	2	14,490,000	12,000,000	-2,490,000	1,096,425,000	
	3	13,635,000	12,000,000	-1,635,000	1,094,790,000	
	4	14,085,000	12,000,000	-2,085,000	1,092,705,000	
	5	13,635,000	12,000,000	-1,635,000	1,091,070,000	
	6	15,030,000	12,000,000	-3,030,000	1,088,040,000	
	7	14,850,000	12,000,000	-2,850,000	1,085,190,000	
	8	14,490,000	12,000,000	-2,490,000	1,082,700,000	
	9	10,755,000	12,000,000	1,245,000	1,083,945,000	
	10	13,140,000	12,000,000	-1,140,000	1,082,805,000	
	11	12,645,000	12,000,000	-645,000	1,082,160,000	-19,680,000
	12	10,800,000	12,000,000	1,200,000	1,083,360,000	
	13	8,370,000	12,000,000	3,630,000	1,086,990,000	
	14	8,595,000	12,000,000	3,405,000	1,090,395,000	
	15	10,800,000	12,000,000	1,200,000	1,091,595,000	
	16	12,195,000	12,000,000	-195,000	1,091,400,000	
	17	13,635,000	12,000,000	-1,635,000	1,089,765,000	
	18	14,580,000	12,000,000	-2,580,000	1,087,185,000	
	19	15,300,000	12,000,000	-3,300,000	1,083,885,000	
	20	15,705,000	12,000,000	-3,705,000	1,080,180,000	
	21	16,920,000	12,000,000	-4,920,000	1,075,260,000	
	22	17,100,000	12,000,000	-5,100,000	1,070,160,000	
	23	16,245,000	12,000,000	-4,245,000	1,065,915,000	
	24	16,740,000	12,000,000	-4,740,000	1,061,175,000	
	25	17,010,000	12,000,000	-5,010,000	1,056,165,000	
	26	16,515,000	12,000,000	-4,515,000	1,051,650,000	
	27	12,735,000	12,000,000	-735,000	1,050,915,000	
	28	14,220,000	12,000,000	-2,220,000	1,048,695,000	
	29	14,535,000	12,000,000	-2,535,000	1,046,160,000	
	30	13,905,000	12,000,000	-1,905,000	1,044,255,000	
	31	13,770,000	12,000,000	-1,770,000	1,042,485,000	
August	1	13,005,000	12,000,000	-1,005,000	1,041,480,000	
	2	13,815,000	12,000,000	-1,815,000	1,039,665,000	
	3	15,660,000	12,000,000	-3,660,000	1,036,005,000	
	4	15,840,000	12,000,000	-3,840,000	1,032,165,000	
	5	15,660,000	12,000,000	-3,660,000	1,028,505,000	
	6	12,105,000	12,000,000	-105,000	1,028,400,000	
	7	12,690,000	12,000,000	-690,000	1,027,710,000	
	8	13,770,000	12,000,000	-1,770,000	1,025,940,000	
	9	15,615,000	12,000,000	-3,615,000	1,022,325,000	
	10	15,210,000	12,000,000	-3,210,000	1,019,115,000	
	11	14,670,000	12,000,000	-2,670,000	1,016,445,000	
	12	14,625,000	12,000,000	-2,625,000	1,013,820,000	
	13	15,165,000	12,000,000	-3,165,000	1,010,655,000	
	14	14,895,000	12,000,000	-2,895,000	1,007,760,000	
	15	15,390,000	12,000,000	-3,390,000	1,004,370,000	
	16	14,580,000	12,000,000	-2,580,000	1,001,790,000	
	17	11,430,000	12,000,000	570,000	1,002,360,000	
	18	10,170,000	12,000,000	1,830,000	1,004,190,000	
	19	11,385,000	12,000,000	615,000	1,004,805,000	
	20	11,025,000	12,000,000	975,000	1,005,780,000	
	21	11,295,000	12,000,000	705,000	1,006,485,000	
	22	13,005,000	12,000,000	-1,005,000	1,005,480,000	
	23	14,580,000	12,000,000	-2,580,000	1,002,900,000	
	24	15,885,000	12,000,000	-3,885,000	999,015,000	
	25	15,480,000	12,000,000	-3,480,000	995,535,000	
	26	15,795,000	12,000,000	-3,795,000	991,740,000	
	27	13,725,000	12,000,000	-1,725,000	990,015,000	
	28	14,265,000	12,000,000	-2,265,000	987,750,000	
	29	14,805,000	12,000,000	-2,805,000	984,945,000	
	30	12,870,000	12,000,000	-870,000	984,075,000	

	31	15,300,000	12,000,000	-3,300,000	980,775,000	
September	1	15,525,000	12,000,000	-3,525,000	977,250,000	
	2	17,235,000	12,000,000	-5,235,000	972,015,000	
	3	15,075,000	12,000,000	-3,075,000	968,940,000	
	4	13,680,000	12,000,000	-1,680,000	967,260,000	
	5	14,175,000	12,000,000	-2,175,000	965,085,000	
	6	14,130,000	12,000,000	-2,130,000	962,955,000	
	7	14,850,000	12,000,000	-2,850,000	960,105,000	
	8	12,960,000	12,000,000	-960,000	959,145,000	
	9	11,745,000	12,000,000	255,000	959,400,000	
	10	10,710,000	12,000,000	1,290,000	960,690,000	
	11	12,195,000	12,000,000	-195,000	960,495,000	
	12	13,365,000	12,000,000	-1,365,000	959,130,000	
	13	15,615,000	12,000,000	-3,615,000	955,515,000	
	14	16,020,000	12,000,000	-4,020,000	951,495,000	
	15	16,200,000	12,000,000	-4,200,000	947,295,000	
	16	15,075,000	12,000,000	-3,075,000	944,220,000	
	17	14,085,000	12,000,000	-2,085,000	942,135,000	
	18	12,645,000	12,000,000	-645,000	941,490,000	
	19	13,365,000	12,000,000	-1,365,000	940,125,000	
	20	14,310,000	12,000,000	-2,310,000	937,815,000	
	21	14,175,000	12,000,000	-2,175,000	935,640,000	
	22	14,265,000	12,000,000	-2,265,000	933,375,000	
	23	14,310,000	12,000,000	-2,310,000	931,065,000	
	24	14,400,000	12,000,000	-2,400,000	928,665,000	
	25	14,220,000	12,000,000	-2,220,000	926,445,000	
	26	13,680,000	12,000,000	-1,680,000	924,765,000	
	27	12,555,000	12,000,000	-555,000	924,210,000	
	28	15,120,000	12,000,000	-3,120,000	921,090,000	
	29	14,985,000	12,000,000	-2,985,000	918,105,000	
	30	14,940,000	12,000,000	-2,940,000	915,165,000	
October	1	15,525,000	12,000,000	-3,525,000	911,640,000	
	2	17,235,000	12,000,000	-5,235,000	906,405,000	
	3	15,075,000	12,000,000	-3,075,000	903,330,000	
	4	13,680,000	12,000,000	-1,680,000	901,650,000	
	5	14,175,000	12,000,000	-2,175,000	899,475,000	
	6	14,130,000	12,000,000	-2,130,000	897,345,000	
	7	14,850,000	12,000,000	-2,850,000	894,495,000	
	8	12,960,000	12,000,000	-960,000	893,535,000	
	9	11,745,000	12,000,000	255,000	893,790,000	
	10	10,710,000	12,000,000	1,290,000	895,080,000	
	11	12,195,000	12,000,000	-195,000	894,885,000	
	12	13,365,000	12,000,000	-1,365,000	893,520,000	
	13	15,615,000	12,000,000	-3,615,000	889,905,000	
	14	16,020,000	12,000,000	-4,020,000	885,885,000	
	15	16,200,000	12,000,000	-4,200,000	881,685,000	
	16	15,075,000	12,000,000	-3,075,000	878,610,000	
	17	14,085,000	12,000,000	-2,085,000	876,525,000	
	18	12,645,000	12,000,000	-645,000	875,880,000	
	19	13,365,000	12,000,000	-1,365,000	874,515,000	
	20	14,310,000	12,000,000	-2,310,000	872,205,000	
	21	14,175,000	12,000,000	-2,175,000	870,030,000	
	22	14,265,000	12,000,000	-2,265,000	867,765,000	
	23	14,310,000	12,000,000	-2,310,000	865,455,000	
	24	14,400,000	12,000,000	-2,400,000	863,055,000	
	25	14,220,000	12,000,000	-2,220,000	860,835,000	
	26	13,680,000	12,000,000	-1,680,000	859,155,000	
	27	12,555,000	12,000,000	-555,000	858,600,000	
	28	15,120,000	12,000,000	-3,120,000	855,480,000	
	29	14,985,000	12,000,000	-2,985,000	852,495,000	
	30	14,940,000	12,000,000	-2,940,000	849,555,000	-242,040,000
	31	5,445,000	12,000,000	6,555,000	856,110,000	

November	1	5,310,000	12,000,000	6,690,000	862,800,000	
	2	4,500,000	12,000,000	7,500,000	870,300,000	
	3	5,130,000	12,000,000	6,870,000	877,170,000	
	4	3,150,000	12,000,000	8,850,000	886,020,000	
	5	8,865,000	12,000,000	3,135,000	889,155,000	
	6	4,905,000	12,000,000	7,095,000	896,250,000	
	7	4,905,000	12,000,000	7,095,000	903,345,000	
	8	5,265,000	12,000,000	6,735,000	910,080,000	
	9	5,220,000	12,000,000	6,780,000	916,860,000	
	10	8,055,000	12,000,000	3,945,000	920,805,000	
	11	9,630,000	12,000,000	2,370,000	923,175,000	
	12	9,585,000	12,000,000	2,415,000	925,590,000	
	13	10,665,000	12,000,000	1,335,000	926,925,000	
	14	10,350,000	12,000,000	1,650,000	928,575,000	
	15	6,840,000	12,000,000	5,160,000	933,735,000	
	16	10,800,000	12,000,000	1,200,000	934,935,000	
	17	13,095,000	12,000,000	-1,095,000	933,840,000	
	18	13,680,000	12,000,000	-1,680,000	932,160,000	
	19	13,005,000	12,000,000	-1,005,000	931,155,000	
	20	11,880,000	12,000,000	120,000	931,275,000	
	21	11,970,000	12,000,000	30,000	931,305,000	
	22	10,260,000	12,000,000	1,740,000	933,045,000	
	23	8,460,000	12,000,000	3,540,000	936,585,000	
	24	8,325,000	12,000,000	3,675,000	940,260,000	
	25	6,165,000	12,000,000	5,835,000	946,095,000	
	26	5,760,000	12,000,000	6,240,000	952,335,000	
	27	5,940,000	12,000,000	6,060,000	958,395,000	
	28	5,985,000	12,000,000	6,015,000	964,410,000	
	29	6,615,000	12,000,000	5,385,000	969,795,000	
	30	855,000	12,000,000	11,145,000	980,940,000	
December	1	675,000	12,000,000	11,325,000	992,265,000	
	2	315,000	12,000,000	11,685,000	1,003,950,000	
	3	3,015,000	12,000,000	8,985,000	1,012,935,000	
	4	2,970,000	12,000,000	9,030,000	1,021,965,000	
	5	4,905,000	12,000,000	7,095,000	1,029,060,000	
	6	4,995,000	12,000,000	7,005,000	1,036,065,000	
	7	4,770,000	12,000,000	7,230,000	1,043,295,000	
	8	4,905,000	12,000,000	7,095,000	1,050,390,000	
	9	5,130,000	12,000,000	6,870,000	1,057,260,000	
	10	5,265,000	12,000,000	6,735,000	1,063,995,000	
	11	2,115,000	12,000,000	9,885,000	1,073,880,000	
	12	540,000	12,000,000	11,460,000	1,085,340,000	
	13	720,000	12,000,000	11,280,000	1,096,620,000	
	14	990,000	12,000,000	11,010,000	1,107,630,000	
	15	1,035,000	12,000,000	10,965,000	1,118,595,000	
	16	585,000	12,000,000	11,415,000	1,130,010,000	
	17	2,295,000	12,000,000	9,705,000	1,139,715,000	
	18	765,000	12,000,000	11,235,000	1,150,950,000	
	19	4,275,000	12,000,000	7,725,000	1,158,675,000	
	20	2,475,000	12,000,000	9,525,000	1,168,200,000	
	21	585,000	12,000,000	11,415,000	1,179,615,000	
	22	4,185,000	12,000,000	7,815,000	1,187,430,000	
	23	2,925,000	12,000,000	9,075,000	1,196,505,000	
	24	180,000	12,000,000	11,820,000	1,208,325,000	
	25	135,000	12,000,000	11,865,000	1,220,190,000	
	26	180,000	12,000,000	11,820,000	1,232,010,000	
	27	225,000	12,000,000	11,775,000	1,243,785,000	
	28	225,000	12,000,000	11,775,000	1,255,560,000	
	29	225,000	12,000,000	11,775,000	1,267,335,000	
	30	225,000	12,000,000	11,775,000	1,279,110,000	
	31	225,000	12,000,000	11,775,000	1,290,885,000	


FRANKLIN, TN RECLAIMED WATER SYSTEM MASS DIAGRAM
Million Gallons
3,500
3,000
2,500
2,000
1,500
1,000
500
0
January
February
March
April
May
June
July
August
September
October
November
December
Cumulative Demand
Cumulative Supply

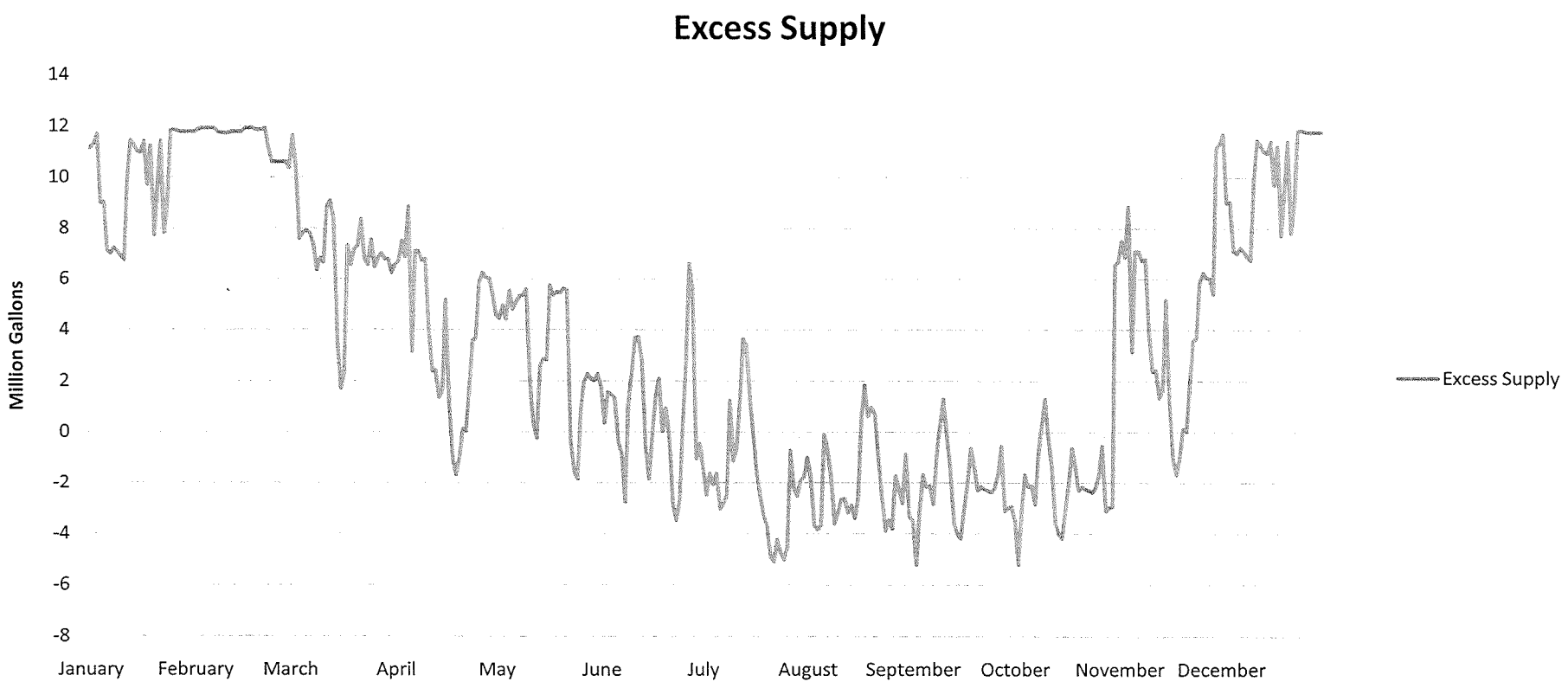

Excess Supply
Million Gallons
14
12
10
8
6
4
2
0
-2
-4
-6
-8
January
February
March
April
May
June
July
August
September
October
November
December
Excess Supply

Table 2 lists existing system customers as of March 1, 2011. Average daily and maximum daily demands have been taken from reclaimed water sales records obtained from the City of Franklin where possible. Other flows are estimates based on potential irrigated area. However, some of the customers in this list have, after initial use, either cut back or discontinued use of reclaimed water because of significant rainfall or problems in their individual storage and on-site irrigation systems. It is anticipated during dry periods of the year that all eight of these customers will be on-line and using the water as shown in the demand chart.

Table 2

Franklin Reclaimed Water System					
Existing System Customers (as of 5/1/2008)					
No.	**Name**	**Customer Type**	**Ave. Daily Demand (GPD)**	**Max Daily Demand (GPD)**	**Distribution Line**
1	Legends Club	Recreational	500,000	2,100,000	Legends
2	Westhaven	Residential, Recreational	117,000	2,000,000	Westhaven
3	Franklin High School	Institutional	156,000	525,000	Westhaven
4	BGA	Institutional	150,000	385,000	Legends
5	Harpeth Village Merchants	Commercial	77,000	150,000	Westhaven
6	Carlisle Lane Residents	Residential	100,000	250,000	Westhaven
7	John Deere Landscaping	Commercial	385,000	1,300,000	Westhaven
8	Williamson County Rec.	Recreation	60,000	124,000	Westhaven
Totals			**1,545,000**	**6,834,000**	

Table 3 lists potential system customers for the existing Franklin reclaimed water system as of March 1, 2011. In re-evaluating the projected demands for these potential users, projections were revised based on a 2-inch per week irrigation rate, which is what many area golf course and park facilities use for irrigation rates of their green spaces. As a result, the average daily demand for these potential customers has increased to 3.76 mgd. The projected maximum daily demand has increased to 8.13 mgd for these potential customers.

In discussions with the design professionals who design cooling systems for large-scale buildings, it is known that water quality plays a significant role in longevity of the cooling tower equipment. Suspended solids and the presence of chlorine in cooling tower make-up water may have an adverse effect on pipes and pumps that are used in these systems. After an industrial customer purchases reclaimed water, it may be necessary to filter or provide chemical addition to water to make it more suitable for use as cooling tower make-up water. Given the significant cost savings of reclaimed water over potable water, individual industrial customers may find it advantageous to test the water for use in their individual cooling systems.

A new drive-by survey of the Columbia Avenue Industrial District revealed that Franklin is currently continuing to lose industries. The APCOM Corporation and Georgia Boot have both closed, and while the remaining businesses might use reclaimed water for irrigation of their lawns and open spaces, it is anticipated that most reclaim use would be seen as cooling tower make-up water for the buildings associated with these businesses. The complex previously housing the CPS Corporation is being partially used, but not for heavy industry. It is not anticipated that a large, heavy industry will occupy this location in the future.

A number of the listed potential customers are served by the West Reclaimed Water Line and Downs Boulevard Reclaimed Water Line. These two projects are currently under construction and are scheduled for completion by late May of this year. The existing residential subdivisions listed in this table are not yet equipped with a reclaimed water distribution system. For this reason, it is not anticipated that these subdivisions could easily connect to the reclaimed water system for irrigation purposes for individual households. However, open spaces and green spaces could be easily connected to the reclaimed water system at a more reasonable cost. Jim Warren Park and the Williamson County Soccer Complex are currently equipped with large-scale irrigation systems, and it is anticipated that both of these parks could change over to reclaimed water at a minimal cost.

Vulcan Materials, Williamson County Ready Mix, and Sherman Dixie Concrete are all industrial customers. It is anticipated that each of these businesses could connect to and use reclaimed water at a minimal cost and realize a significant discount for volume purchases.

Many of the potential customers listed could be served by the Goose Creek reclaimed water line and Five-Mile Creek reclaimed water lines which were completed in 2009. A

listing of these properties includes all the residential properties that are proposed for development along the southern border of Goose Creek. Some limited development has taken place at the Ladd Park and Stream Valley subdivisions. However, other properties are awaiting economic recovery before development continues. All of these development projects are listed in a 2009 Franklin Development Report prepared by the Franklin Planning Department, and it is anticipated that all of these properties will develop in the future.

Table 3

Franklin Reclaimed Water System					
Potential System Customers (as of 5/1/2008)					
No.	**Name**	**Customer Type**	**Ave. Daily Demand (GPD)**	**Max Daily Demand (GPD)**	**Distribution Line**
1	Fairways at Spencer Creek	Recreational	250,000	350,000	Westhaven
2	Pinkerton Park	Municipal	84,000	170,000	Southeast
3	Forest Crossing	Recreational	500,000	2,000,000	Southeast
4	Battlefield Park	Municipal	295,000	600,000	Southeast
5	Franklin Street Dept.	Municipal	100,000	200,000	Southeast
6	Columbia Avenue Industrial	Industrial	90,000	90,000	Southeast
7	FSSD	Institutional	25,000	50,000	West Line
8	Jim Warren Park	City Parks	100,000	200,000	West Line
9	Williamson County Soccer	County Parks	240,000	500,000	West Line
10	New Hope Academy	Institutional	20,000	50,000	West Line
11	Founders Point	Residential	100,000	200,000	West Line
12	Reid Hill	Residential	20,000	90,000	West Line

13	Williamson County Redi-Mix	Industrial	100,000	150,000	Downs Blvd
14	Vulcan Materials	Industrial	75,000	150,000	Downs Blvd
15	Sherman-Dixie	Industrial	100,000	200,000	Downs Blvd
16	Ladd Park	Residential	440,000	900,000	Goose Creek
17	Berry Farms	Mixed Use	330,000	680,000	Goose Creek
18	Stream Valley	Residential	250,000	500,000	Goose Creek
19	Rural Pains	Mixed Use	80,000	150,000	Goose Creek
20	Rheems Fleming	Mixed Use	75,000	100,000	Goose Creek
21	Nichol's Bend	Residential	100,000	150,000	Goose Creek
22	Lockwood	Residential	160,000	300,000	Goose Creek
23	Ag Center	Institutional	30,000	50,000	Goose Creek
24	Existing Residential (Redwing Farms Area)	Residential	200,000	300,000	Goose Creek
Totals			**3,764,000**	**8,130,000**	

Total Ave. Daily Demand: 5,309,000 gpd

Total Maximum Daily Demand: 14,964,000 gpd

Re-evaluation of average daily and maximum daily demands yield a total average daily demand for the Franklin reclaimed water system at 5.31 mgd with a total maximum daily demand of 14.96 mgd.

City of Franklin, Tennessee

Integrated Water Resources Plan

Appendix H:

Harpeth River

- Harpeth River Water Quality Model Comparison
- Ecological Restoration and Stream Enhancement Technical Analysis

CDM Smith

Harpeth River Water Quality Model Comparison

PURPOSE

The purpose of this modeling task is to use available tools to help quantify the likely impacts of the Franklin IWRP on water quality and flow in the Harpeth River. The aim is not to provide a definitive statement about whether or not the IWRP will achieve water quality standards, as such determinations would be hampered by known data gaps (e.g. instream organic matter, nonpoint source pollutant loads, etc.), by scientific uncertainty inherent in any representation of natural hydrologic systems, and by factors beyond the control of the City of Franklin (such as upstream watershed pollutant loads, climate variability, etc.). Rather, the tools will be adapted as needed and used to compare the IWRP alternatives with respect to their impacts on water quality and river flow. Results will be interpreted in conjunction with professional judgment in a probabilistic context (for example: Most likely to meet state standards, likely to meet state standards, compliance uncertain, not likely to meet standards).

This memorandum compares the two available models of the hydraulics and water quality in the Harpeth River, and recommends the tool(s) that are best suited to this IWRP study.

Client

Franklin, TN

Integrated Water Resources Plan

Key Team Members

Kirk Westphal

Jamie Lefkowitz

Date

February 2011

DRIVING QUESTIONS

To date, water quality in the Harpeth River has been addressed in the IWRP process only through estimation of changes to pollutant loads into the river, and not on instream fate and transport of pollutants. As the process moves toward the selection and implementation of alternatives, specific questions about instream water quality must be addressed:

- Which IWRP alternative is likeliest to yield the best water quality in the Harpeth River in Franklin and downstream?
- What are the likely water quality impacts of the selected alternative?
- If water quality upstream of Franklin meets TN standards, how will Franklin's IWRP affect the river?
- If water quality upstream of Franklin does not meet standards, how will Franklin's IWRP work toward improvements?

AVAILABLE TOOLS

There are currently two separate, existing models of the Harpeth River. A suite of models of the watershed loading, river hydraulics, and water quality processes was developed by EPA Region 4 as part of a TMDL for the river in 2004. Another model of the hydraulics and water

quality processes has been developed and maintained by TVA (River Management System, or RMS). Both models have strengths and weaknesses, and both will need to be modified and updated for use in the IWRP. The following figure illustrates the watershed and identifies key river mile locations that are referenced in the model descriptions:

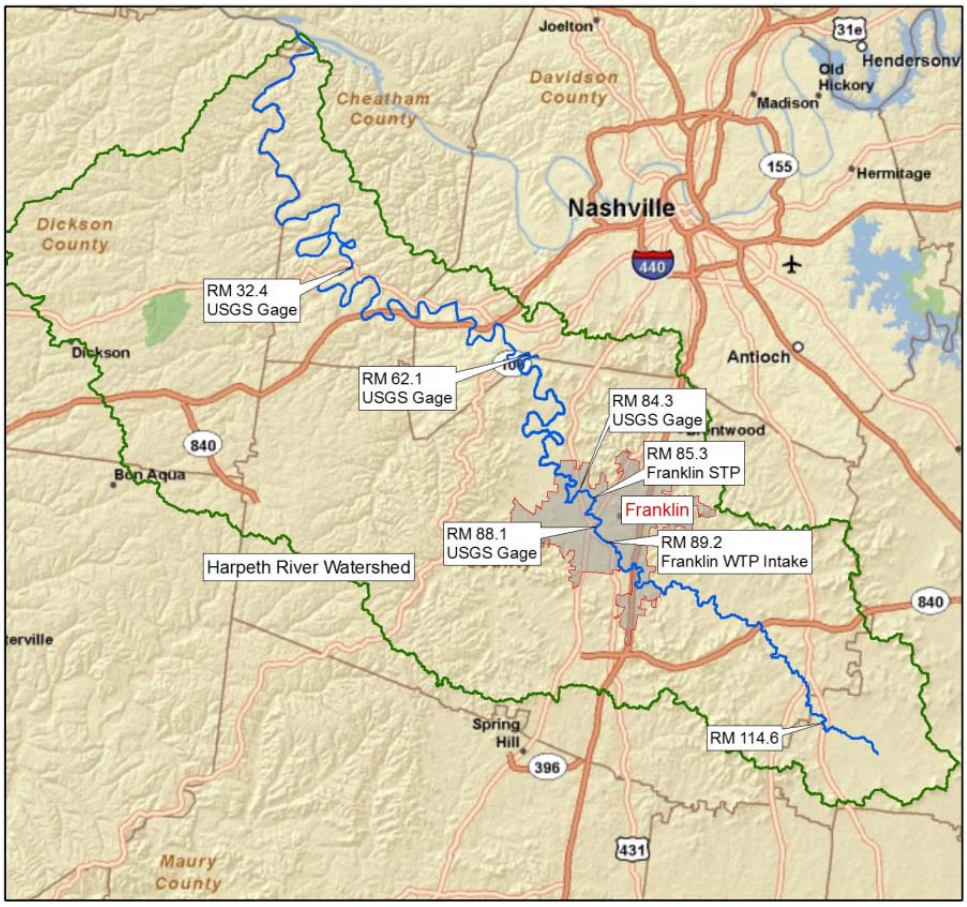


Figure 1 - Harpeth River Watershed and Key River Mile Locations

The TMDL models use the following software, and cover river mile 32.4 to river mile 88.1:

- Loading Simulation Program C++ (LSPC) for overland flow and watershed loading
- CE-QUAL-RIV1 (CE-QUAL) for river hydraulics
- Water Quality Analysis Simulation Program (WASP) for instream water quality
- QUAL2E for dissolved oxygen upstream of river mile 89.2

The TVA model is coded in the River Management System (RMS) framework, and covers river mile 32.4 to river mile 114.6. It is comprised of the following analytical modules:

- ADYN for the river hydraulics
- RQUAL for the instream water quality

The RMS model has not yet been applied to studies of the Harpeth River.

METHOD OF TOOL SELECTION

The tools used for evaluation of future water quality and flow conditions in the Harpeth River will be selected and adapted based on their ability to address the driving questions outlined above. A four-step process was employed to evaluate and compare the alternative tools and formulate a recommendation to the City. Ultimately, the recommendation of which tool(s) to apply will be based on observed scientific credibility and professional judgment regarding the functionality of the tools and their applicability to the IWRP process.

1) Review existing reports

   a. Strengths and weaknesses from past peer reviews were noted.

   b. Calibration records were examined for consistency across ranges of flows and seasons, and for overall scientific credibility.

2) Review input files

   a. Parameters were reviewed against available data and published literature values for inland rivers.

   b. Hydraulic and water quality processes (e.g. the nutrient cycle) were examined for appropriate level of detail with respect to available data. *Note that complete representation of physical, biological, and chemical processes is not necessarily a criterion, as sometimes the ability to distinguish causes and effects is obscured when detail is added but cannot be substantiated.* We looked for a representation of the system that can reproduce *and explain* observed phenomena. Clarity in the causal relationships and their sensitivity was more important than complete and accurate reproduction of all end results.

3) Run the models

   The models were evaluated for:

   a. Ease of scenario definition

   b. Ranges of conditions over which they can be run

   c. Sensitivity to the types of projects comprising the IWRP alternatives

   d. Numerical stability

   e. Required conversions to new versions

4) Observed Performance

   a. Based on the types of scenarios proposed, more emphasis was placed in this comparison on instream processes (channel hydraulics and water quality) than on watershed processes such as rainfall-runoff relationships and nonpoint source pollution loading. This is because surrogates are readily available for watershed processes, in the form of USGS streamflow records, existing water quality data, and published values on land use loading rates for relevant pollutants. Furthermore, none of the IWRP alternatives are aimed at improving conditions in the upstream

watershed (as this is beyond the purview of the City of Franklin), and available data will be more credible than synthesized inputs to the model of the actual river. Therefore, a brief review of key model performance features was conducted, focusing specifically on time of travel over varying flow conditions and simulated dissolved oxygen levels (both daily average and diurnal fluctuations).

OBSERVATIONS

Calibration/Performance

Flow and Time of Travel

Both existing models have been configured to simulate flows in the river in August 2000 (a period of low flows), allowing for comparison of hydraulic and water quality model performance against observed data for that time period. In addition, the ADYN hydraulic model was configured to run for April 2001, in order to compare the results under a higher flow regime.

The Harpeth River experiences times of extremely low flow (less than 1 cfs) during average summer months, which presents a challenge in hydraulic modeling. Under such low flows, the river can be dry in some places and flow as a series of trickles and pools in others. Neither ADYN nor CE-QUAL is designed to simulate such flows with a high level of precision because channel features are generalized over reaches of several thousand feet, and under low flow conditions, localized variations in geometry can restrict flow where higher flow would be unaffected. While the flows in the model output match gaged records, the simulated velocities can be greater than what would be observed in the river, therefore resulting in decreased travel times under extreme low flow conditions. Figures 2 and 3 show the gaged flows for August 2000 and April 2001 as compared with the simulated flows from the ADYN and CE-QUAL hydraulic models.

Both models use the 15-minute flow data from the USGS Gage at Franklin (03432350) as a boundary condition. There are unexplained differences in the gage data and the existing CE-QUAL model boundary condition input, specifically spikes in flow in late August 2000 in the model input that do not appear in the observed data. The source of the input flow data (from tributaries and intermittent runoff) is different for the two models. The existing CE-QUAL model uses flows generated by a watershed rainfall-runoff model (LSPC), while the ADYN model input flows were derived using existing gage data and conservation of mass principals. The latter approach results in better matching of peaks at the gaged locations, and is sufficient for the purposes of this study, where simulation of overland flow and loading is not a principal focus (stormwater pollutant loads can be estimated outside of modeling frameworks). The USGS gages on the Harpeth River also provide adequate data over the past several decades to simulate a wide range of flow conditions with this approach. Without converting the CE-QUAL model to its successor software, EPD, it is not possible to examine how that model would respond to input flows generated using a mass balance approach.

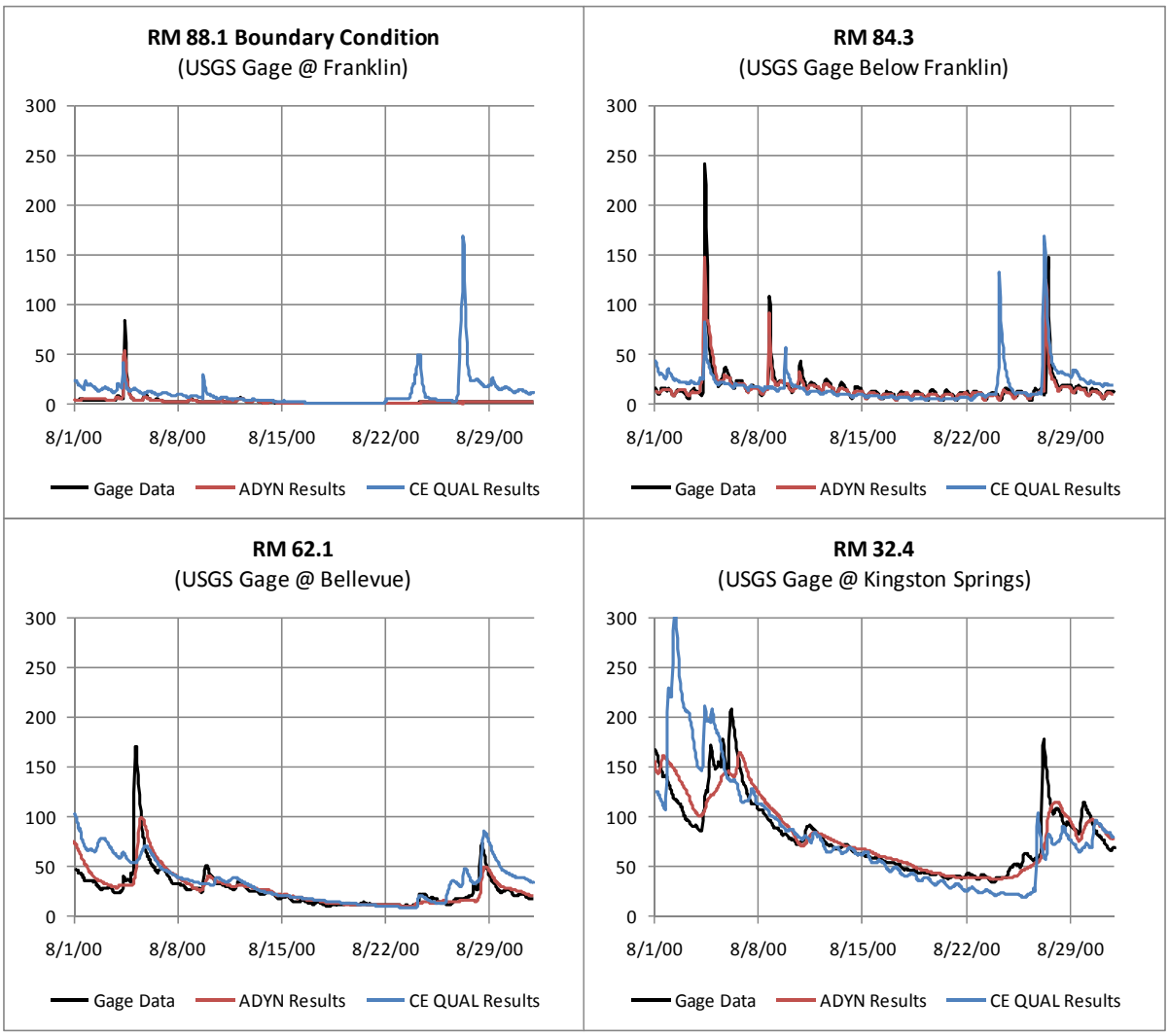


Figure 2 - Gaged and Simulated Streamflow Comparison – August 2000

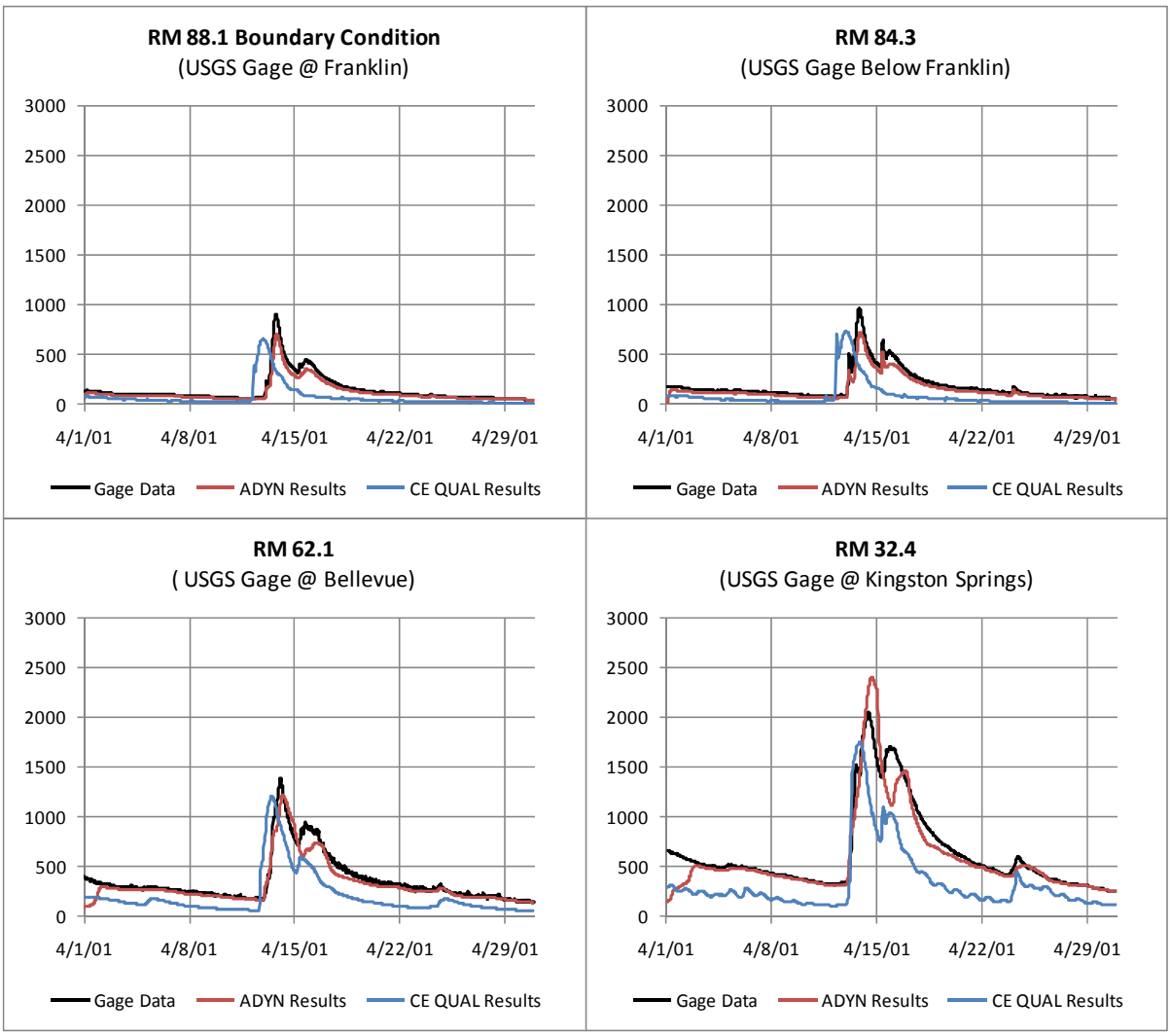


Figure 3 - Gaged and Simulated Streamflow Comparison – April 2001

EPA conducted two time of travel studies on the Harpeth River in August 2000 and April 2001. The results are reported in graphical form only in the Harpeth River Watershed Modeling Effort: A Tool for TMDL Development Report (July 2002). The velocity and time of travel results from the August 2000 study conflict (pp. 40 and 47), and therefore will be omitted from the model comparison in this memorandum.

Both models match the time of travel data from the April 2001 study fairly well (see Figure 4). The CE-QUAL model tends to simulate lower velocities (greater times of travel), likely due to assumptions of variable roughness coefficients (discussed in the section on model parameterization). A separate comparison between times of travel of storm peaks shows that both models produce reasonable results as compared with gaged data. Figure 5 shows the flow (at the upstream gage) versus time of peak travel for several rain events in August 2000 and April 2001. This comparison shows that both models reasonably predict time of travel in the river over a range of flow conditions, not including extreme low flows. The lowest observable peak in the gage hydrographs was approximately 50 cfs, which is well above the extreme low summer flows of below 1 cfs that have been observed in the Harpeth River.

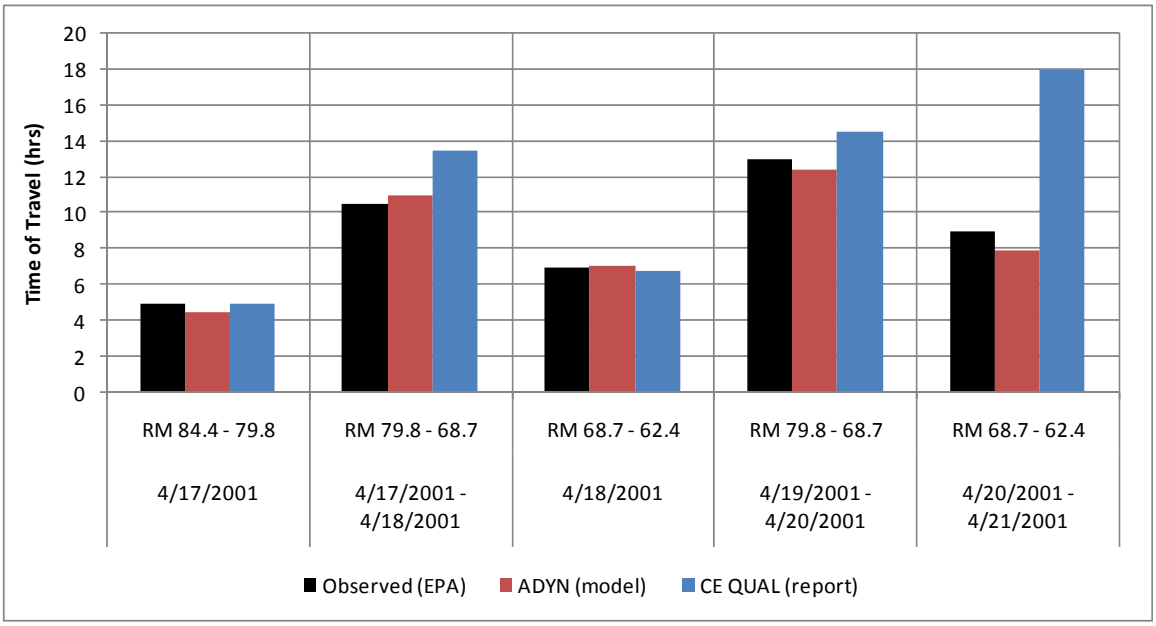


Figure 4 - Time of Travel Comparison – April 2001

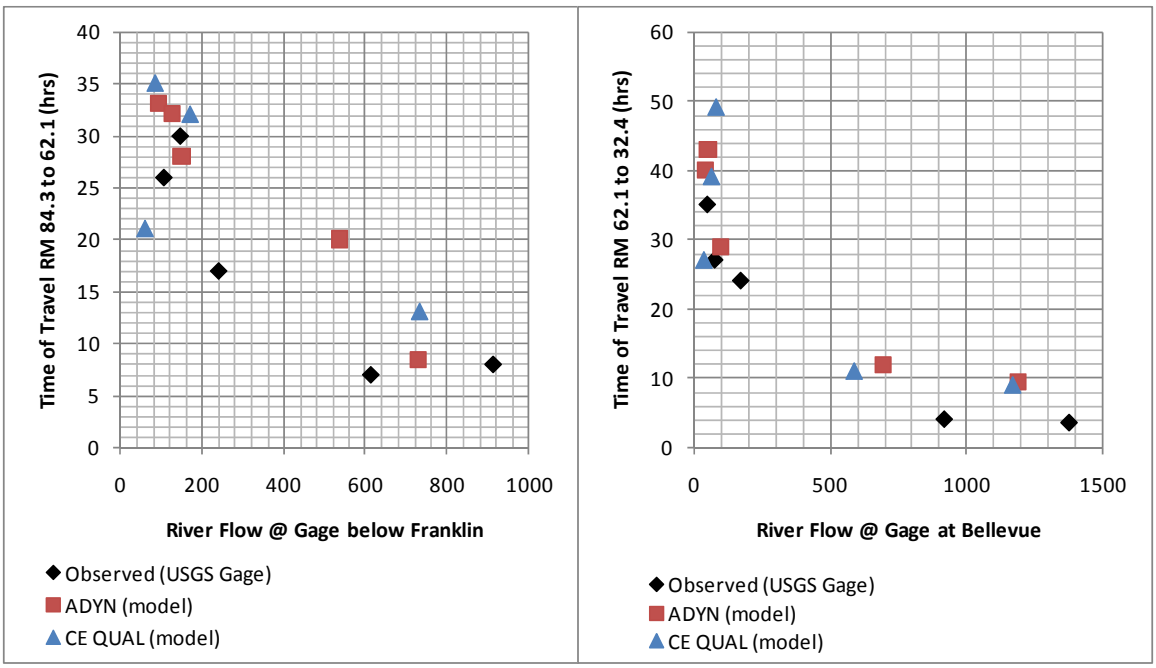


Figure 5 - Comparison of Flow vs. Time of Peak Travel for August 2000 and April 2001 Storms

Dissolved Oxygen

Dissolved oxygen levels in the Harpeth River were measured by EPA in August 2000 at seven locations: two upstream of Franklin, two in or near Franklin, and three downstream of Franklin. The measurements were taken continuously at 30 minute intervals over four days, August 22 to August 25. The comparison of dissolved oxygen levels between measured values and the WASP simulation was included in the TMDL Development Report and is presented in Figure 6 and 7. The comparison between measured dissolved oxygen and the RQUAL simulation was extracted from the August 2000 model run, and is presented in Figure 8. The RQUAL comparison figures show the average dissolved oxygen and diurnal fluctuations for each river location where data were available.

Both water quality models match the magnitude of dissolved oxygen concentrations in the river relatively well. The RQUAL model more successfully matches the diurnal timing pattern than the WASP model, based on the plot of river mile 88.1 from the TMDL Development Report. This location in the river is of particular interest in the IWRP study, as it will be necessary to examine the effect of various water and wastewater management strategies on the dissolved oxygen levels within the Franklin reach of the Harpeth River, where the stream is currently impaired. The current calibration of the RQUAL model predicts slightly greater dissolved oxygen values than were observed at this location, but the model parameters are largely set to default values. Based on the accuracy of the timing of diurnal fluctuations, and the very close match to observed data at downstream locations, the RQUAL model appears to be the preferred tool to use in water quality simulations for the IWRP study.

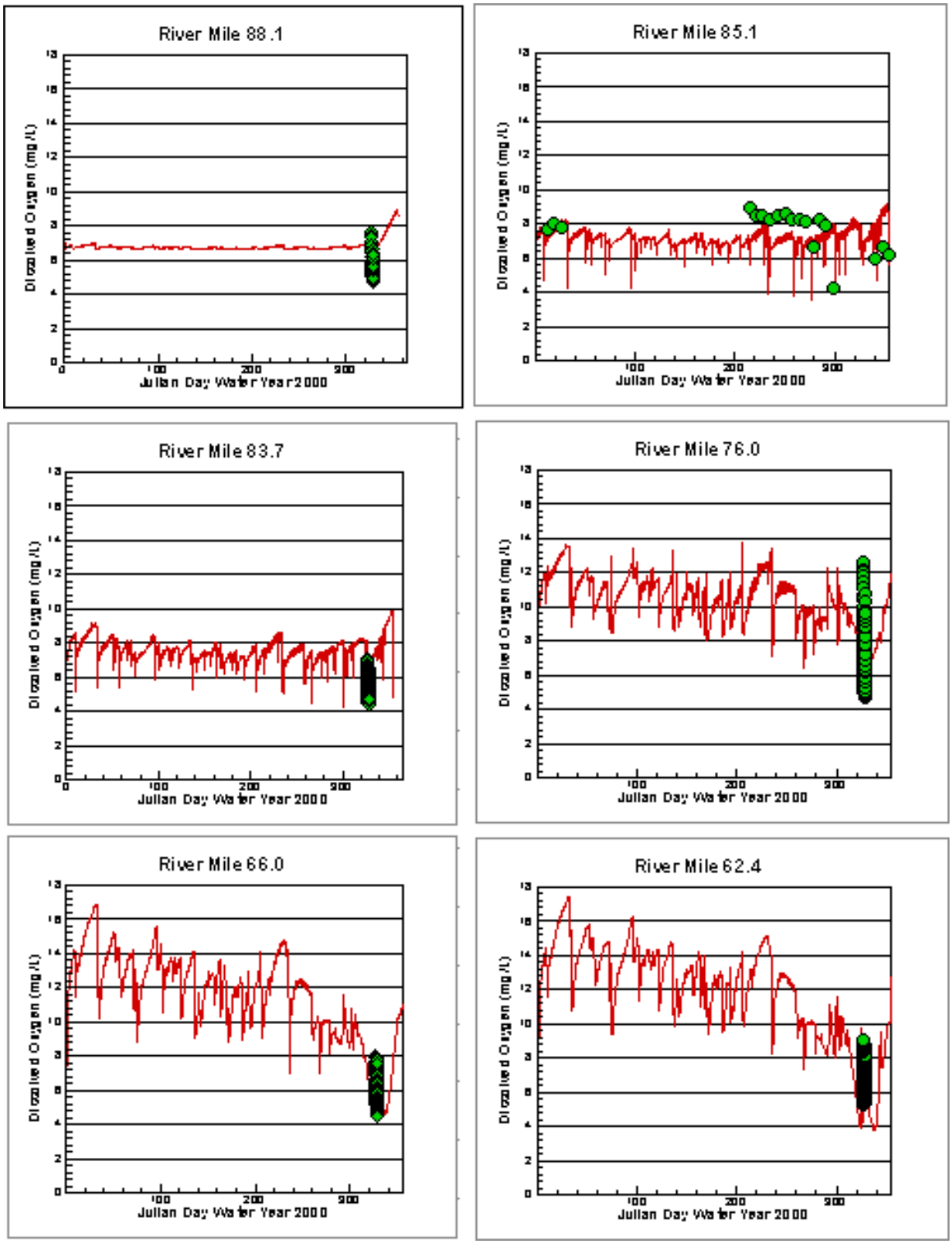


Figure 6 - Comparison of Observed and Simulated Dissolved Oxygen, WASP Model

(Excerpted from: Harpeth River Watershed Modeling Effort: A Tool for TMDL Development Report (EPA, July 2002)

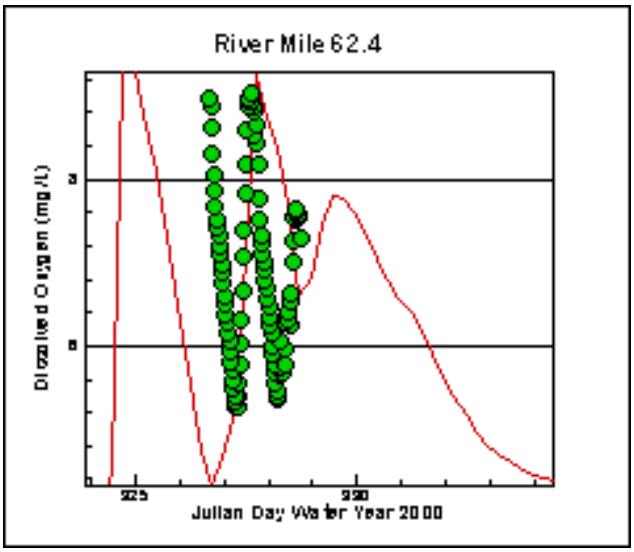


Figure 7 - Comparison of Observed and Simulated Dissolved Oxygen Diurnal Fluctuations, WASP Model

(Excerpted from: Harpeth River Watershed Modeling Effort: A Tool for TMDL Development Report (EPA, July 2002)

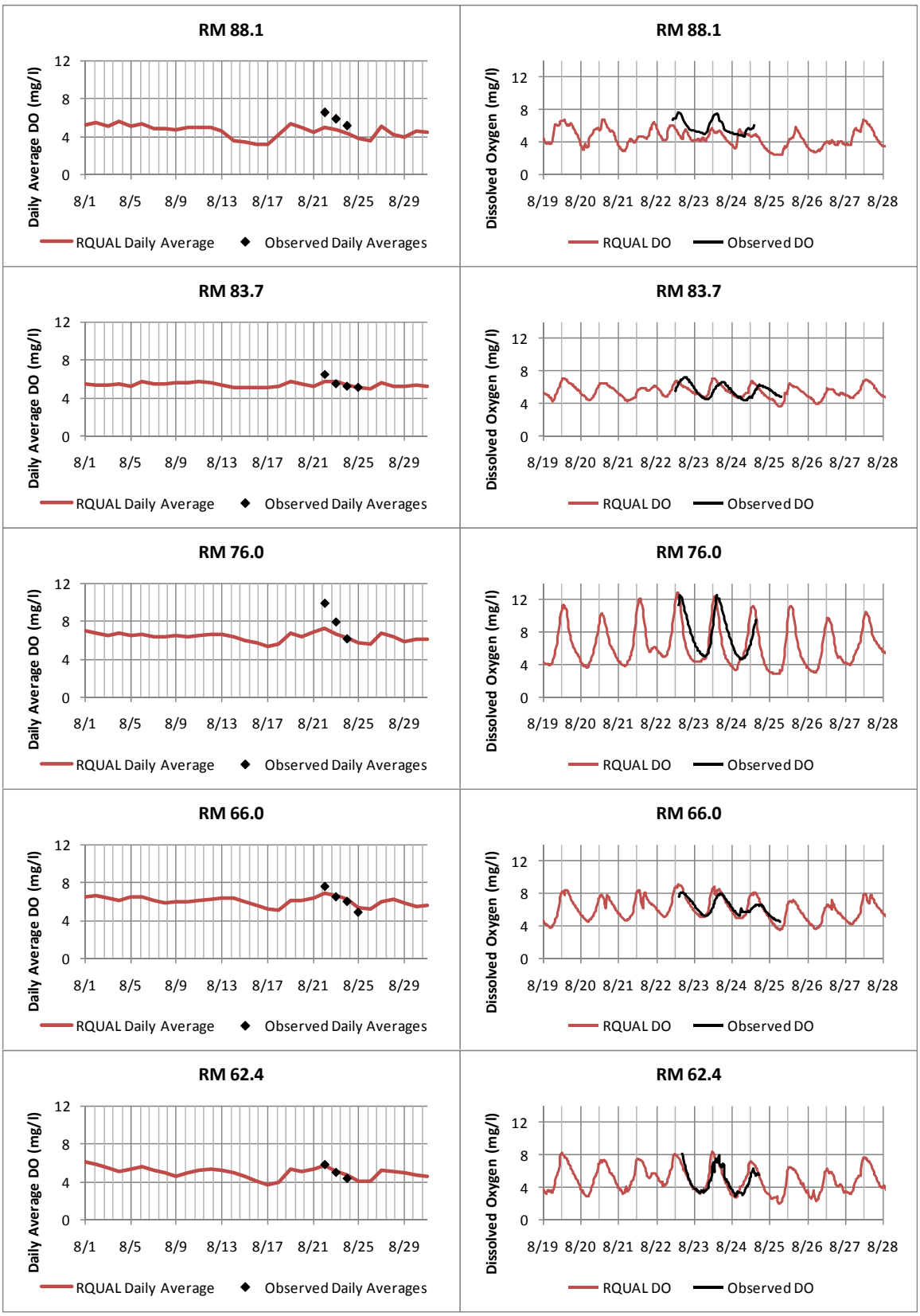


Figure 8 - Comparison of Observed and Simulated Dissolved Oxygen, RQUAL Model

Notes From Past Peer Reviews

CDM reviewed the TMDL model in 2002 and discovered the following issues:

- The 1992 land use data used in the LSPC watershed model is outdated.
- The hydraulic model needs refinement in streambed elevations, using local digital elevation models.
- The hydraulic model is unstable or inaccurate at very low flows (less than 1 cfs).
- Periphyton is not considered in the water quality model.
- The time of travel calibration is inadequate for the purposes of the TMDL study.
- The diurnal fluctuations represented in the model are inadequate for the purposes of the TMDL study.

A complete memorandum on the CDM review of the TMDL model was issued to EPA on September 30, 2002. It is important to recognize that the modeling for the IWRP study is not intended to recommend permit limits or load/wasteload allocations. Hence, weaknesses in certain areas, while critical to a TMDL study, are not necessarily fatal flaws with respect to comparative analysis for IWRP planning (refer to the fundamental questions outlined at the beginning of this document), but are still important considerations in the selection of a tool.

The RMS model of the Harpeth River was developed by TDEC for internal purposes, and has not been peer-reviewed at this time.

Current Model Parameterization

Hydraulic Parameters

The primary parameters of the channel hydraulic simulation are the roughness coefficients for the channel itself and its overbank areas. For fast-flowing streams that are constrained by numerous constrictions such as culverts and wide bridge piers, contraction and expansion head loss coefficients can also be important parameters to adjust in a hydraulic model. However, the Harpeth River, under most flow conditions, cannot be generally characterized that way, and so the most important hydraulic parameters are the roughness coefficients. These control the velocities at which water flows through the channel, the associated depth, and backwater behavior, and are the most important parameters in the accurate simulation of travel time, water depth, and flow velocity.

Roughness coefficients can range from values of approximately 0.013 (smooth, concrete channels) to 0.13 – 0.15 (swampy areas with severely restricted or impeded flow pathways). Generally, channels such as the Harpeth River with flat bottoms of rocks, sand, and gravel are characterized by roughness coefficients ranging from 0.030 – 0.040. Overbank areas that are marked by tree roots and heavy vegetation, such as characterize the channel banks along the Harpeth River, generally have higher roughness coefficients to represent the flow restrictions.

The CE-QUAL hydraulic model of the Harpeth River applied depth-variable roughness coefficients to simulate the slowing down of river flow during low-flow conditions, when uniform flow in the channel is replaced with more stagnant pools and riffles (even puddles). At zero depth, the roughness coefficient is 0.15 (almost fully restricted), and this decreases linearly with increasing depth to 0.030 at 4 feet of depth, a common value for rocky-bottom channels like the Harpeth. This was done as a calibration step, to slow

the water velocity down at extremely low flows, when small rocks and gravel can act more like boulders would in a larger stream. However, the depth-roughness relationship is not necessarily physically based, and the channel should begin to behave like a normal, gravelly river at depths well below 4 feet. (4 feet of depth corresponds to flows much higher than extreme low flow conditions), Functionally, however, this approach is useful in representing the ways in which the channel's restrictive characteristics act differently on low flows than on higher flows. With respect to the overbank areas, the roughness coefficients in this model do not appear to vary from the channel coefficients, which is somewhat unusual and not necessarily physically representative of the dense vegetation that lines the river banks of the Harpeth.

The ADYN model applies constant roughness coefficients of 0.030 for the channel and 0.090 for the overbank areas. These values are physically based and commonly applied for rivers characterized by flat, gravelly bottoms and vegetated, rooty banks and floodplains. These values are considered to be more defensible than those in the CE-QUAL model because they are physically linked to actual channel characteristics, though for extremely low flow conditions, travel times may be faster than in the actual channel

Water Quality Parameters

While the mathematical representation of the physical, biological, and chemical processes in the WASP model and the RQUAL model are reasonably similar, the parameterization of the models differs in many cases. This makes a direct comparison of the appropriateness of parameters somewhat difficult, and so each of the tools was evaluated for its water quality parameterization independently. Full summary tables of parameter values are available but this section focuses on a general evaluation of key parameters and apparent philosophies of calibrating the two models.

Many of the basic parameters are equivalent or similar between the two models, and fall within the range of expected values for this type of river. An example are the Sediment Oxygen Demand rates (SOD), set at 2 $g/m^2/day$ in WASP and varying between 2 and 2.3 $g/m^2/day$ in RQUAL (higher at the upstream end). Neither model demonstrates major changes in SOD, which is encouraging – drastic variations would suggest that SOD may have been a localized calibration parameter, and with as much uncertainty as there is in the Harpeth concerning nutrient loads, periphyton, etc., it is preferable *not* to compensate for one uncertainty with a high degree of variability in another uncertain variable. It is best to keep variables such as SOD reasonably constant unless field data suggest other trends. Other basic parameters, such as temperature correction factors, are also similar between the two models.

However, some of the key parameters that govern the nitrogen cycle are very different. For example, the nitrification rate constant is set at 0.4 in the WASP model, and 0.1 in the RQUAL model. Similarly, the denitrification rate in WASP is set at 0.03, while in RQUAL it is 0.1. Because organic growth in the Harpeth River is assumed to be primarily limited by nitrogen, care will be required to vet these constants to other calibrated river models, and understand their sensitivities (for whichever model is selected).

Many of the variables in the WASP model fall within the range of published literature values. However, in the WASP model, many of the nutrient and phytoplankton (floating algae) coefficients are set at or above the maximum recommended value. It is very possible that this is because the WASP model does not account for fixed organic growth (attached algae, or periphyton) and compensates for the oxygen depleting effects by over-emphasizing the impacts of nutrients and their interactions with floating algae (Note that a newer version of WASP, WASP7, includes simulation options for periphyton, but the Harpeth model was created in an older version which did not have this functionality). Generally,

observing so many coefficients set at or above the maximum recommended rate is a sign that the processes being simulated are not necessarily represented in a physically defensible way.

In the RQUAL model, most of the parameters for the Harpeth River are set at default values, generally within published literature ranges Notable exceptions include the photosynthetic and macrophyte respiration rates, which were adjusted to replicate presumed impacts of periphyton, based on dissolved oxygen measurements. The RQUAL model does not simulate periphyton explicitly, but the photosynthetic and respiration processes associated with fixed organic growth, such as macrophytes, can be used approximately as a surrogate.

Therefore, RQUAL is the preferred tool with respect to water quality parameterization:

- RQUAL uses parameter values that generally fall within published literature ranges, whereas the calibrated WASP model applies parameter values at or above the recommended maximum for many nutrient and algae processes, suggesting that other processes (perhaps fixed/rooted organic matter) are not adequately represented.
- Periphyton is a known issue in the Harpeth River. RQUAL represents this by simulating photosynthesis and respiration of fixed aquatic plant growth (macrophytes), which is a reasonable surrogate if interactions with sediment nutrients is de-emphasized, especially in the absence of field data. The original WASP model did not include periphyton simulation, though the latest version (WASP7) includes this functionality. The WASP model would need to be converted and upgraded to simulate periphyton.

Before application, the full suite of parameters in the RQUAL model will be further examined and adjusted if defensible and necessary.

Functionality and Ease of Use for IWRP Analysis

The table below compares the two modeling packages for overall functional characteristics and efficiency / effectiveness for IWRP analysis. Recall that these models are *not* being used in this study to establish discharge permit limits or Total Maximum Daily Loads (TMDLs) or new FEMA-certified flood elevations. Rather, the selected tool will be used to compare the IWRP alternatives and help identify which alternatives are likely to produce the best water quality conditions in the river, and how likely any of the alternatives will be to achieving Tennessee state water quality standards on a relative basis.

Qualitative assessments of the functionality of each evaluated tool toward these ends are provided in the table below. For each criterion, the preferred tool is shaded, with darker shading indicating strong preference and lighter shading indicating only a moderate preference.

Table 1 - Comparison of Functionality and Ease of Use

Criteria	CE-QUAL / WASP	RMS: ADYN / RQUAL
Ease of scenario definition and output review	Hydraulic scenarios are difficult without the user interface for EPD, which requires conversion of the input files from CE-QUAL. WASP scenarios are easy to define and manage. The need to port output from EPD to WASP requires generation of .HYD files, which is straightforward, but not seamless.	Scenarios are easy to define and manage with the available user interface, and the two models are designed to run together as two modules of a single package. Output review is very easy with the interface.
Ranges of conditions over which model can be run	Possible to run long-term scenarios. Boundary conditions and input flows currently developed in external LSPC watershed model, but could be replaced with available gaged flows.	Currently set up to run 1-month scenarios. Longer-term simulations are possible, though model array size may pose a problem. The current time-step limit is unknown. There is the possibility of working with the developer to increase the array size. Boundary conditions and input flows derived from known gaged flows.
Sensitivity to the types of projects comprising the IWRP alternatives	Can easily introduce changes in flow and load from lateral confluences. Can simulate altered channel hydraulics. Existing model may not have a reliable background simulation of the nutrient cycle and local oxygen levels without periphyton (would require conversion to WASP 7, additional parameterization, and recalibration). End result may be slightly more physically-based periphyton representation than RQUAL, but without field data, high precision is not possible anyway.	Can easily introduce changes in flow and load from lateral confluences. Can simulate altered channel hydraulics. Already simulates fixed organic growth (periphyton) for better representation of background dissolved oxygen levels so that sensitivity of IWRP projects can be better grounded (representation of macrophyte growth is used as a surrogate for all fixed organic growth).
Numerical Stability	Hydraulic model cannot simulate river flows below 1 cfs. These flows are known to occur during periods of interest for IWRP analysis, but any simulation of flows this low are questionable because of the inability of hydraulic models to accurately account for pooling and riffling and the amplified restrictions of gravel at severe low flows.	Hydraulic model is stable below 1 cfs, but as explained to the left, any simulation of flows in this regime should be used with caution.

Required conversions	CE-QUAL-RIV1 has been replaced by EPD-RIV1. EPD will read CE-QUAL input files, but the user interface necessary for efficient scenario setup and output review requires file conversion. WASP would require upgrading to WASP7 and recalibration to distinguish between floating and fixed algae.	No conversions needed.
Technical Support	Has been difficult getting data and support	TDEC worked with TVA in the development of this model and has willingly provided assistance in this evaluation and will continue to provide technical support and review during the IWRP.

RECOMMENDATION

Recommended Tool

Based on the model review described herein, CDM recommends that the RMS modeling tool be used for the IWRP Harpeth River technical analysis. The following table summarizes the preferred models for each category of observations. A dot in both model columns indicates that there was insufficient information to make a definitive preference of one model over the other.

Category	**CE-QUAL / WASP**	**RMS (ADYN / RQUAL)**
Hydrologic / Hydraulic Calibration/Performance		•
Dissolved Oxygen Calibration/Performance		•
Peer Reviews	•	•
Hydraulic Parameterization		•
Water Quality Parameterization		•
Functionality		•

The RMS modeling package is preferred in all categories where enough information was available to make such a determination. The hydraulic performance of the ADYN (RMS) model appears better than the CE-QUAL model. However, the discrepancies between observed and simulated flows seem to be caused in part by input flows and boundary conditions. Without comparing the simulated flows of both models using the same input and boundary condition flows, it is not possible to fully evaluate their performance relative to each other. In the Peer Review category, only one model was subject to a complete peer review, so there is insufficient information to determine a preference of one model over the other.

Required Enhancements

The RMS model was developed internally by TDEC and not for use on a particular study. Most parameters are set to default values and the model is only fully set up to run one month (August 2000). The following enhancements will be required to bring the model into a fully functional and usable state for the IWRP technical analysis. This list is a starting point; there are likely additional enhancements needed that have yet to be discovered. These enhancements are based on the model review performed to date and the previous peer review of the TMDL model.

Known required model enhancements:

- Evaluate and possibly revise model streambed elevations using local digital elevation models
- Test the sensitivity of the model to algae and macrophyte growth and decay parameterization
- Configure model to run long-term scenarios, of one year or longer (exact dates to be determined)
- Examine and possibly revise the water quality parameterization in relation to regional and literature values
- Compare model scenario runs to other existing data collected since the TMDL study

Attachment: Water Quality Model Parameterization

RMS – RQUAL Parameters

Water Quality Process Coefficients				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
THR	Real	temperature correction coefficient for reaeration	1.024	
THB	Real	temperature correction coefficient for CBOD decay	1.047	
BK20	Real	deoxygenation rate at 20 °C for CBOD (1/day)	0.2	
THN	Real	temperature correction coefficient for NBOD decay	1.09	
NK20	Real	deoxygenation rate at 20 °C for NBOD (1/day)	0.3	
THS	Real	temperature correction coefficient for SOD	1.065	
EXCO	Real	light extinction coefficient (1/m)	0.1	fairly clean water
HMAC	Real	average weed height from bottom of channel (ft)	0	
THPR	Real	temperature correction coefficient for macrophyte photosynthesis and respiration	1.08	
IK2EQ	Integer	flag for reaeration equation choice (-1 to 7)	5	Reaeration equation: Tsivoglou
Algal Kinetic Coefficients				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
AG	Real	algal growth rate (1/day)	2.2	range 1.0-5.0
AM	Real	algal mortality rate (1/day)	0.3	range 0.0-0.5
AR	Real	algal dark respiration rate (1/day)	0.2	range 0.0-.5
AS	Real	algal settling rate (1/day)	0.01	range 0.0-0.5
ASAT	Real	half-saturation coefficient for light (W/m2)	20	range 10-50
EXOM	Real	extinction due to organic suspended solids (m2/g)	0.17	suggest 0.17

Algal Rate Coefficients				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
AT1	REAL	Lower temperature for algal growth	10	
AT2	REAL	Lower temperature for maximum algal growth	20	
AT3	REAL	Upper temperature for maximum algal growth	30	
AT4	REAL	Upper temperature for algal growth	40	
AK1	REAL	Fraction of algal growth rate at AT1	0.1	
AK2	REAL	Fraction of maximum algal growth rate at AT2	0.99	
AK3	REAL	Fraction of maximum algal growth rate at AT3	0.99	
AK4	REAL	Fraction of algal growth rate at AT4	0.1	
Organic Matter Kinetic Coefficients				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
DOMDK	REAL	Labile dissolved organic matter decay rate (1/day)	0.2	range 0.0-0.5
POMDK	REAL	Particulate organic matter decay rate (1/day)	0.2	range 0.0-0.5
POMS	REAL	Particulate organic matter settling rate (1/day)	0	range 0.0-0.5
SDK	REAL	Settled organic matter decay rate (1/day)	0.05	range 0.0-0.3; SDK>0.05 causes SED to go negative
Organic Matter Rate Coefficients				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
OMT1	REAL	Lower temperature for organic matter decay	10	
OMT2	REAL	Lower temperature for maximum organic matter decay	15	
OMK1	REAL	Fraction of organic matter decay rate at OMT1	0.1	
OMK2	REAL	Fraction of organic matter decay rate at OMT2	0.99	

Phosphorus				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
AHSP	REAL	Half-saturation constant for phosphorus (g/m3)	0.009	range 0.001-0.01
Ammonium				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
NH4DK	REAL	Ammonium decay rate (1/day)	0.1	range 0.0-0.5
AHSN	REAL	Half-saturation constant for nitrogen (g/m3)	0.05	range 0.0-0.2
Ammonium Rate Coefficients				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
NH4T1	REAL	Lower temperature for ammonium decay	5	
NH4T2	REAL	Lower temperature for maximum ammonium decay	20	
NH4K1	REAL	Fraction of nitrification rate at NH4T1	0.1	
NH4K2	REAL	Fraction of nitrification rate at NH4T2	0.99	
Nitrate				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
NO3DL	REAL	Nitrate decay rate (1/day)	0.1	range 0.0-0.15
Nitrate Rate Coefficients				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
NO3T1	REAL	Lower temperature for nitrate decay	5	
NO3T2	REAL	Lower temperature for maximum nitrate decay	20	
NO3K1	REAL	Fraction of denitrification rate at NH4T1	0.1	
NO3K2	REAL	Fraction of denitrification rate at NH4T2	0.99	

Stoichiometry Coefficients				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
O2NH4	REAL	Oxygen stoichiometric equivalent for ammonium decay (gO2/ gNH4)	4.57	
O2OM	REAL	Oxygen stoichiometric equivalent for organic matter decay (gO2/gOM)	1.4	
O2AR	REAL	Oxygen stoichiometric equivalent for dark respiration (gO2/gA)	1.4	
O2AG	REAL	Oxygen stoichiometric equivalent for algal growth (gO2/gA)	1.4	
BIOP	REAL	Stoichiometric equivalent between organic matter and phosphorous (gP/gOM)	0.005	
BION	REAL	Stoichiometric equivalent between organic matter and nitrogen (gN/gOM)	0.05	
Oxygen Limit				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
O2LIM	REAL	Dissolved oxygen concentration at which anaerobic processes begin (g/m3)	0.1	suggestion 0.1
Water Quality Process Coefficients				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
BS20	REAL	CBOD settling rate (1/day)	0	see manual
O2KM	REAL	DO half-saturation value to reduce respiration at low DO	0	see manual
QSAT	REAL	saturation light intensity for macrophytes	0	see manual
Weir Aeration Coefficients and Formulation (Franklin Weir @ RM 89.0)				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
WFAC	REAL	Weir aeration equation multiplication factor	1	free overflow weirs
WLEN	REAL	weir crest length (ft)	76	
NEVQ	Integer	#points on user-defined Q vs. E15 curve	0	ignored

SOD Rates vs. River Mile			
NAME	TYPE	DESCRIPTION	VALUE
SFAC	REAL	factor to multiply all SK20 in reach to test sensitivity	1
RMI	REAL	river mile (exact match to model node)	see table
SK20	REAL	SOD rate (g)2/m2/day)	see table
VALUES			
RMI	SK20		
114.60	2		
89.00	2		
88.10	2.3		
83.98	2.3		
75.95	2		
66.00	2		
62.40	2		
35.50	2		
32.40	2		

Photosynthesis Rates vs. River Mile			
NAME	TYPE	DESCRIPTION	VALUE
PFAC	REAL	factor to multiply all PMAX20 in reach to test sensitivity	1
RMI	REAL	river mile (exact match to model node)	see table
PMAX20	REAL	photosynthesis rate (gO2/m2/hr)	see table
VALUES			
RMI	PMAX		
114.60	0.2		
89.00	0.2		
88.10	0		
76.17	0.4		
75.95	0.4		
66.00	0.3		
62.40	0.5		
32.40	0.5		

Respiration Rates vs. River Mile				
NAME	TYPE	DESCRIPTION	VALUE	NOTES
RFAC	REAL	factor to multiply all RESP20 in reach to test sensitivity	1	
RMI	REAL	river mile (exact match to model node)	see table	
RESP20	REAL	weed respiration rate (gO2/m2/hr)	see table	should be about 0.1 to 0.3 times PMAX20

VALUES	
RMI	RESP
114.6	0.02
89	0.02
88.1	0
76.17	0
75.95	0
66	0
62.4	0.05
32.4	0.05

WASP Parameters

Phytoplankton Constants				
DESCRIPTION	ON/OFF	VALUE	MIN	MAX
Phytoplankton Maximum Growth Rate Constant @20 °C (per day)	1	2	0	3
Phytoplankton Growth Temperature Coefficient	1	1.07	0	1.07
Include Algal Self Shading Light Extinction in Steele (0=Yes, 1=No)	0	0	0	1
Exponent for Self Shading (Mult * TCHLA^Exp)	0	0	0	1
Multiplier for Self Shading (Mult * TCHLA^Exp)	0	0	0	1
Phytoplankton Self Shading Extinction (Dick Smith Formulation)	0	0	0	0.02
Phytoplankton Carbon to Chlorophyll Ratio	1	60	0	200
Phytoplankton Half-Saturation Constant for Nitrogen Uptake (mg N/L)	1	0.05	0	0.05
Phytoplankton Half-Saturation Constant for Phosphorus Uptake (mg P/L)	1	0.05	0	0.05
Phytoplankton Endogenous Respiration Rate Constant @20 °C (per day)	1	0.8	0	0.5
Phytoplankton Respiration Temperature Coefficient	1	1.08	0	1.08
Phytoplankton Death Rate Constant (Non-Zooplankton Predation) (per day)	1	1	0	0.25
Phytoplankton Zooplankton Grazing Rate Constant (per day)	0	0	0	5
Nutrient Limitation Option	0	0	0	1
Phytoplankton Decay Rate Constant in Sediments (per day)	0	0	0	0.02
Phytoplankton Temperature Coefficient for Sediment Decay	0	0	0	1.08
Phytoplankton Phosphorus to Carbon Ratio	1	0.24	0	0.24
Phytoplankton Nitrogen to Carbon Ratio	1	0.43	0	0.43
Phytoplankton Half-Sat. for Recycle of Nitrogen and Phosphorus (mg Phyt C/L)	1	1	0	1

Light Constants				
DESCRIPTION	ON/OFF	VALUE	MIN	MAX
Light Option (1 uses input light; 2 uses calculated diel light)	1	1	1	2
Phytoplankton Maximum Quantum Yield Constant	1	500	0	720
Phytoplankton Optimal Light Saturation	1	320	0	350
Background Light Extinction Multiplier	0	0	0	10
Detritus & Solids Light Extinction Multiplier	0	0	0	10
DOC Light Extinction Multiplier	0	0	0	10
Dissolved Oxygen Constants				
DESCRIPTION	ON/OFF	VALUE	MIN	MAX
Water body Type Used for Wind Driven Reaeration Rate	0	0	0	3
Calc Reaeration Option (0=Covar, 1=O'Connor, 2=Owens, 3=Churchill, 4=Tsivoglou)	0	0	0	4
Global Reaeration Rate Constant @ 20 °C (per day)	0	0	0	10
Elevation above Sea Level (meters) used for DO Saturation	0	0	0	15000
Reaeration Option (Sums Wind and Hydraulic Ka)	0	0	0	1
Minimum Reaeration Rate, per day	0	0	0	24
Theta -- Reaeration Temperature Correction	0	0	0	1.03
Oxygen to Carbon Stoichiometric Ratio	1	2.67	0	2.67
Use (1 - On, 0 - Off) Total Depth of Vertical Segments in Reaeration Calculation	0	0	0	1
CBOD (Ultimate) Constants				
DESCRIPTION	ON/OFF	VALUE	MIN	MAX
BOD (1) Decay Rate Constant @20 °C (per day)	1	0.07	0	5.6
BOD (1) Decay Rate Temperature Correction Coefficient	1	1.047	0	1.07
BOD (1) Decay Rate Constant in Sediments @20 °C (per day)	0	0	0	0.0004
BOD (1) Decay Rate in Sediments Temperature Correction Coefficient	0	0	0	1.08
BOD (1) Half Saturation Oxygen Limit (mg O/L)	1	0.5	0	0.5

Fraction of Detritus Dissolution to BOD (1)	0	0	0	1
Fraction of BOD (1) Carbon Source for Denitrification	0	0	0	1
SOD Parameters				
DESCRIPTION	ON/OFF	VALUE	MIN	MAX
SOD (gram/m^2/day)	1	2.0		
SOD Temperature Correction Factor	1	1.04		

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Technical Memorandum

To: *City of Franklin IWRP Team*

From: *CDM*

Date: *July 27, 2011*

Subject: *Integrated Water Resources Plan - Ecological Restoration and Stream Enhancement Technical Analysis*

As part of the Integrated Water Resources Plan (IWRP) for the City of Franklin (City), Camp Dresser & McKee (CDM) has researched available information provided by the City and other local agencies to identify opportunities within the Harpeth River watershed for stream restoration, bank stabilization, and riparian buffer enhancement. In addition, CDM has developed planning level cost estimates for identified improvements and quantified, to the extent practicable, the relative benefits that may be expected from these projects. The following sections summarize the data evaluated for this effort as well as the cost vs benefit analysis performed. The results of this study will be used for comparison with other improvement alternatives being evaluated as a part of the Integrated Water Resources Plan.

1.0 Introduction

Preservation and protection of the Harpeth River (the River) is a key priority identified by both the City and the Stakeholders participating in the Phase I process of the IWRP project. Specifically, the Stakeholders in the Phase I process established objectives related to Water Quality and Ecological Health and Improved Access and Aesthetics of the river. Because there are areas of the Harpeth River that are degraded by runoff from both urban and agricultural areas, there are opportunities to implement BMPs such as stream restoration, bank stabilization, and riparian restoration projects along the River and its tributaries. The costs and benefits of these options will be evaluated within a planning level computer model (STELLA) developed for the IWRP project. The ability to quantify the ultimate benefits to the River and watershed for each project option is key to the analysis and selection of improvement options that will be included in the final Integrated Water Resources Plan.

The Harpeth River is a dynamic, natural system responding to the watershed development and channel alterations imposed upon it over centuries of agriculture and recent urbanization. Bank

erosion and degraded wildlife habitat in the River are symptoms of the history and current conditions of the watershed. Stream restoration in and around the City of Franklin can be expected to alleviate some of the localized sediment burden and improve channel conditions, but cannot remedy the hydrologic flow regime or sediment regime from the entire watershed.

2.0 Study Scope

The Harpeth River begins in Rutherford County and extends northwest through 4 counties before converging with the Cumberland River in Cheatham County, for a total watershed area of 863 square miles (TDEC, 2002). The River receives runoff from approximately 169 square miles (sq. mi.) before entering the City. The City encompasses approximately 23 sq. mi. and is almost entirely in the Harpeth River watershed, with 13.7 river miles running through the City. This evaluation focuses on the main stem and tributaries of the River within the City's municipal limits, as well as some of the upstream reaches extending beyond the City limits. A mechanism to promote better agricultural practices in the watershed upstream of the City is being sought by the Harpeth River Watershed Association (HRWA) (personal communication with Michael Cain, April 2011).

The primary objectives of this planning level study are to identify candidate stream restoration, bank stabilization, and riparian restoration projects along the River, develop conceptual opinions of probable cost, and describe and quantify potential benefits associated with the identified projects.

3.0 Data Collection

Existing data and information regarding the conditions of the main stem and tributaries of the River in and upstream of the City were reviewed as part of this evaluation. Studies conducted by the HRWA, visual assessment information data provided by the City, geographical information survey (GIS) data, and available stormwater and watershed master plan reports were used to identify potential stream channel restoration, bank stabilization, and riparian restoration projects.

Habitat conditions in the River have been described as poor, due to low dissolved oxygen, high sediment embeddedness, and a lack of habitat heterogeneity (HRWA, 2006). In 2002, the EPA published a Total Maximum Daily Load (TMDL) Modeling Study Report (EPA, 2002a) and Siltation and Habitat Alteration TMDL (EPA, 2002b). The same year, the Tennessee Department of Environment and Conservation (TDEC) compiled a listing of all impaired stream in the state (referred to as the 303(d) list, which included the Harpeth River. In addition, TDEC published a Watershed Plan to address impairments associated with the 303(d) listings of the River and several of its tributaries (TDEC, 2002). As of 2010, the River remains on the 303(d) list as shown in Table 1 for low dissolved oxygen and loss of biological integrity.

Studies conducted by the HRWA describe the extent of degraded physical, chemical, and biological conditions in the River and their potential sources. Multiple HRWA studies attempt to quantify the effects of erosion and sedimentation throughout the River and were used to estimate the benefits of candidate projects on the water quality and habitat conditions in the Harpeth River.

Table 1 - Harpeth River 2010 303(d) Listings for Williamson County

Waterbody ID	Miles/Acres Impaired	Cause	Pollutant Source	Comments
TN0513020 4016 –1000	6.8	Low dissolved oxygen, Phosphorus	Municipal Point Source Discharges from MS4 area	Category 4a. EPA approved DO and nutrient TMDLs for the known pollutants.
TN0513020 4016 – 4000	7.5	Low Dissolved Oxygen, Loss of biological integrity due to siltation	Pasture Grazing	Category 4a. EPA approved DO and siltation TMDLs for the known pollutants.
TN0513020 4016 – 3000	9.0	Low Dissolved Oxygen, Loss of biological integrity due to siltation	Pasture Grazing	Category 4a. EPA approved DO and siltation TMDLs for the known pollutants.

(TDEC, 2010 Final 303(d) List)

In 2007, the HRWA conducted both a sediment study (HRWA and CRC) and a bank erosion study (HRWA, 2007a) on the Harpeth River. The sediment study, conducted jointly by HRWA and the Cumberland River Coalition, indicated a high suspended sediment loading from the watershed. The bank erosion study showed that severe bank erosion is occurring along the main stem of the River and is considered a major sediment load contributor. The erosion study does not, however, indicate the extent of the erosion except at particular locations along the River, all of which are downstream of the City. Due to inherent variability in bed and bank erosion rates, neither the HRWA sediment study, nor the Harpeth River TMDL (EPA, 2002) studies could quantify the portion of the sediment loading originating from streambank erosion. Steep banks, utility crossings, and a lack of riparian vegetation were identified as the leading causes of bank erosion along the River (HRWA, 2007).

Results of the visual assessment of the streams in and immediately upstream of the City, conducted by the City, provided locations and estimated lengths of stream currently experiencing

erosion or other issues. The survey results were provided by the City in a Geographic Information System (GIS) shapefile with assessment categories for erosion being either common or occasional along the banks. Results of this general survey were used to estimate the extent of potential stream stabilization projects necessary in the study area. A detailed evaluation of each potential project reach is needed prior to further planning and implementation in accordance with the criteria set forth by the City's Harpeth River Watershed Initiative (Franklin, 2007). Data were translated into potential stream restoration and stabilization lengths for the watersheds where more detailed master plan reports were not available. Sites identified with occasional erosion were considered to require bank stabilization and those where erosion was common were assigned a restoration need. These tributary areas identified from the visual assessment are shown in **Figure 1**.

A review of the 2009 aerial photography, in GIS, revealed areas where extensive erosion is occurring along the main stem of the River and where riparian vegetation is lacking within the study area. Figure 1 shows the areas of erosion identified from the aerials and highlights the stream reaches with significantly less than 50 feet of developed riparian vegetation.

Information regarding currently planned projects was obtained through the Sharps Branch Stormwater Master Plan (CDM, 2002), and the Harpeth low head dam removal project presentation (Beaver Creek Hydrology, 2010). Sharps Branch is budgeted for restoration/stabilization, according to the fiscal year 2011-2015 Capital Improvement Plan (CIP) Stormwater Projects.

4.0 Stream Enhancement Measures

Various degrees and methods of stream enhancement are available for implementing improvements to the River and tributaries in and immediately upstream of the City to address 303(d) list impairments for low dissolved oxygen and loss of biological integrity for siltation. These treatment types are:

1. Stream restoration
2. Bank stabilization
3. Riparian restoration
4. Cattle exclusion

Full **stream restoration** is defined for this evaluation as returning pre-disturbance hydrologic functionality, where a shallow stream is interconnected with the floodplain, groundwater, and wetlands, maximizing recharge, organics and nutrient retention, and ecological functionality. Lesser forms of stream restoration feasible in developed areas (natural channel design) are defined by modifications performed within a stream channel to restore the functional conveyance and sediment transport, reduce bank erosion, and improve habitat conditions (Rosgen, 2006).

Bank stabilization may consist of any combination or singular treatment of bank shaping to achieve stable slopes or the application of biotechnical techniques for promoting riparian vegetation to hold soils in place and provide structural stability (USDA 2002). **Riparian restoration** is defined as the planting of native riparian vegetation on stable banks to provide a minimum buffer zone of 60 feet from the top of the banks in accordance with City ordinance. **Cattle exclusion** involves the installation of physical barriers that exclude cattle access to adjacent water bodies.

To evaluate the benefits of the types of stream enhancement, the stated objectives of the IWRP have been categorized into three broad groups; water quantity, water quality, and wildlife habitat. Water quality is defined broadly by metrics which quantify nutrient loadings and TSS and further by dissolved oxygen and temperature. Water quantity is defined by base flow and frequency, magnitude, and duration of runoff event flows. Wildlife habitat is dependent on the actual physical habitat space and conditions suitable for survival of native species including macroinvertebrates and fish.

The water quantity, water quality, and wildlife habitat improvement benefits of the various stream enhancement methods can be measured by several metrics. To allow comparison between stream enhancement methods for the purposes of this IWRP, each stream enhancement method is assigned a high/medium/low rating based on the general influence of the applicable metrics on improvements in each area. Performance of the stream enhancement methods for each metric category is compared in Table 2.

Table 2 - Stream Enhancement Benefits

Type of Riparian Improvement	Unit Cost (per Linear Foot)	Water Quantity	Water Quality	Wildlife Habitat
Full Stream Restoration	$400 - $1,000	High	High	High
Natural Channel Design	$400 - $1,000	Low	Medium	Medium
Bank Stabilization	$400	Medium	Low	Medium
Riparian Restoration	$100	Low	Low	Medium
Cattle Exclusion	$4	Low	High	Medium

Full stream restoration can achieve high benefits in water quantity, water quality, and wildlife habitat because it maximizes the natural ability of the stream system to retain and infiltrate water, retain and treat pollutants, and create diverse self sustaining ecosystems. In-channel modifications using natural channel design techniques are capable of improving local habitat conditions by reducing embeddedness and providing riffle pool sequences and improving water

quality by reducing sediment inputs to stream channel from eroding banks, but cannot improve the runoff conditions of the watershed. Bank stabilization improves water quality by reducing the sediment inputs to the stream channels and the associated nutrients. Riparian restoration improves habitat conditions with roots and contribution of organic materials, and may provide shade to reduce water temperatures. Cattle exclusion has a major impact on water quality as it eliminates direct nutrient and fecal coliform releases and halts trampling damage to streambanks promoting erosion. Cattle exclusion also shares the benefits associated with riparian restoration since it allows revegetation of the banks.

5.0 Improvement Options and Locations

CDM evaluated the most feasible and cost effective options applicable to the Harpeth River and tributaries. The applicable treatment type depends on the runoff conditions, surrounding land uses, type and degree of degradation, and other factors. Due to the incised condition of the main stem of the River in the City, full stream restoration is not feasible without increasing flooding and would be extremely costly. Full stream restoration on the tributaries is only feasible where flooding of adjacent structures or roadways is not an issue along remaining undeveloped corridors. Bank stabilization is recommended where excessive erosion is occurring but alone cannot remedy the altered flow regime and sediment influx creating the degraded conditions throughout the River. Therefore, riparian buffer restoration is recommended as the least expensive and most beneficial broadly applicable treatment. Streambank stabilization can be expected to alleviate some of the localized sediment burden where vegetation is not providing sufficient stabilization, but cannot remedy the hydrologic flow regime or sediment regime from the entire watershed. Low impact development techniques, full stream restoration, reforestation, and stormwater and agricultural BMPs should be implemented throughout the watershed to the extent feasible to retain water, sediment, and nutrients on the land before they enter the River. A reach scale evaluation is recommended to evaluate the most appropriate stream enhancement option for each section of the River and tributaries.

The HRWA's Headwaters of the Harpeth River Water Quality and Habitat Study indicated that stream bank stabilization is necessary for about 10% of upstream banks in Williamson and Rutherford Counties (HRWA, 2007). This would be about 32 miles of the approximately 317 miles of stream in the headwaters, at an estimated cost of $8.5 Million for bioengineered bank stabilization ($50/linear foot [LF] cedar revetments, and mulch socks). HRWA also reported in that document that "treating all stream banks is not cost-effective or practical". The approach recommended by HRWA is to address the sites exhibiting the most erosion and supplement riparian vegetation where existing buffers are thin or sparse. CDM is in concurrence and would also recommend this approach for the main stem of the River and tributaries within the City.

The most cost effective remedies for this approach includes cattle exclusion, natural material revetments, and riparian plantings to protect and stabilize the banks and provide cover and appropriate organic inputs to the channel. These treatments are less destructive and more widely

applicable than natural channel design because they do not require the removal of existing trees or soil disturbance. While these methods may be sufficient for the less degraded channel banks, more intensive methods of bank shaping and channel modification may be warranted for more degraded channels.

Unit costs for stream restoration, bank stabilization, and riparian restoration depend on the method and workforce employed. HRWA developed unit costs based on proven local methods of cattle exclusion, bank stabilization, and riparian planting with volunteer labor. These unit costs were utilized to develop the cost estimates for the Five Mile Creek Watershed (HRWA, 2009) as shown in Table 2. A per linear foot cost estimate of $400 was used in the Sharps Branch Stormwater Master Plan for bank stabilization, which has already been included in the City's Capital Improvement Program (CIP) for future restoration along Sharps Branch. These estimates are consistent with regional stream restoration unit costs (NCEEP, 2011). For this study, $400 per linear foot (LF)is used for stream restoration and bank stabilization on the tributaries of the River and $60 per linear foot for riparian restoration of a 60 foot buffer on both sides of the stream. The $60 is a compromise between the low estimate from HRWA and the high estimate from NCEEP which includes extensive monitoring and other costs beyond the plantings. Costs derived from HRWA and other sources and applied to the identified areas for improvement are listed in Table 2. Estimated costs associated with the main stem of the River in the City were increased ($1,000/LF) to account for the size of the channel and all the estimated costs presented include some allowance for engineering, administration, permitting, and land acquisition.

6.0 Improvement Locations

Given the history, available assessment data, and current uses of the Harpeth River, areas for feasible improvement which would contribute to improvement of the systemic degradation were identified. Planned and potential improvement areas have been identified for cattle exclusion, stream restoration, bank stabilization, or riparian restoration. Figure 1 and Table 2 show the areas identified for improvement through various studies and methods.

Reaches in the Five Mile Creek basin have been specifically identified for stabilization, riparian planting, and cattle exclusion in the HRWA Five Mile Creek Watershed Plan (2009). One of the highest priority water quality problems in the River's watershed is livestock access and the associated bank degradation. Livestock exclusion fencing and alternative water supply provisions are being offered to cattle farmers and installed with HRWA support for the 7.7 miles of stream in the Five-Mile Creek Subbasin identified with this problem. No other cattle exclusion areas were identified in the visual assessment of sites or by the desktop analysis performed for this study.

Areas along the River's tributaries where erosion is common or occasional have been identified through visual assessment. For the purposes of this study, they have been assigned a corresponding need for stabilization. Candidate reach lengths with severe bank erosion along the main stem of the River were identified by CDM using aerial photography. Riparian planting areas

lacking a sufficient vegetated buffer were identified using aerial photography along the River and its tributaries, and are shown in **Figure 1**.

One location for full channel modification of the main stem of the River is recommended by this study. Aerial photography review revealed a channel avulsion towards the northwest border of the City where the channel bend appears to have been modified in the past. Flows are returning a functional radius of curvature to the bend by eroding the outside bank and beginning to form a new channel. Restoration of stream meanders to accelerate the return of the natural meander pattern is recommended to reduce the continuing deposition of sediment in the stream.

Planning level costs for improvements to the headwaters of the River and the low head dam removal and restoration project are not included in the overall estimate for potential enhancement in this study because they are beyond the geographic limits of this evaluation (i.e. outside the City limits) or have already been identified for funding by the City. Stabilization of Sharps Branch, as is included in the City's CIP is included since funds have not yet been spent. An estimated ten percent of the headwater streams are in need of restoration (HRWA, 2007). These are not included in the cost estimate herein due to the large cost of this effort and the lack of jurisdiction by the City to implement this restoration.

7.0 Cost and Benefit Correlation

Qualitative benefits of stream enhancement to the Harpeth River watershed include:

- Reduced bank erosion and subsequent channel sedimentation;
- Improved wildlife habitat;
- Improved aesthetics and accessibility for active and passive recreation;
- Filtering of runoff before it enters the stream; and
- Stream temperature reduction from tree shade.

These and other benefits of restoration and stabilization are difficult to quantify because their direct benefits are parts of many complex cumulative variables for which we do not have a standard valuation system. The most direct benefit of stream enhancement is reduction of sediment input into the stream and retention of nutrients contained in the sediment. This benefit can be estimated by applying an average erosion rate over a length and average height of banks and assuming a unit nutrient release amount for each volumetric unit of sediment. However, this method of estimation only provides an estimated sediment loading reduction which can then be converted to a cost per unit reduction. It does not quantify the ecological function improvements or the value of ecosystem services provided by those improvements.

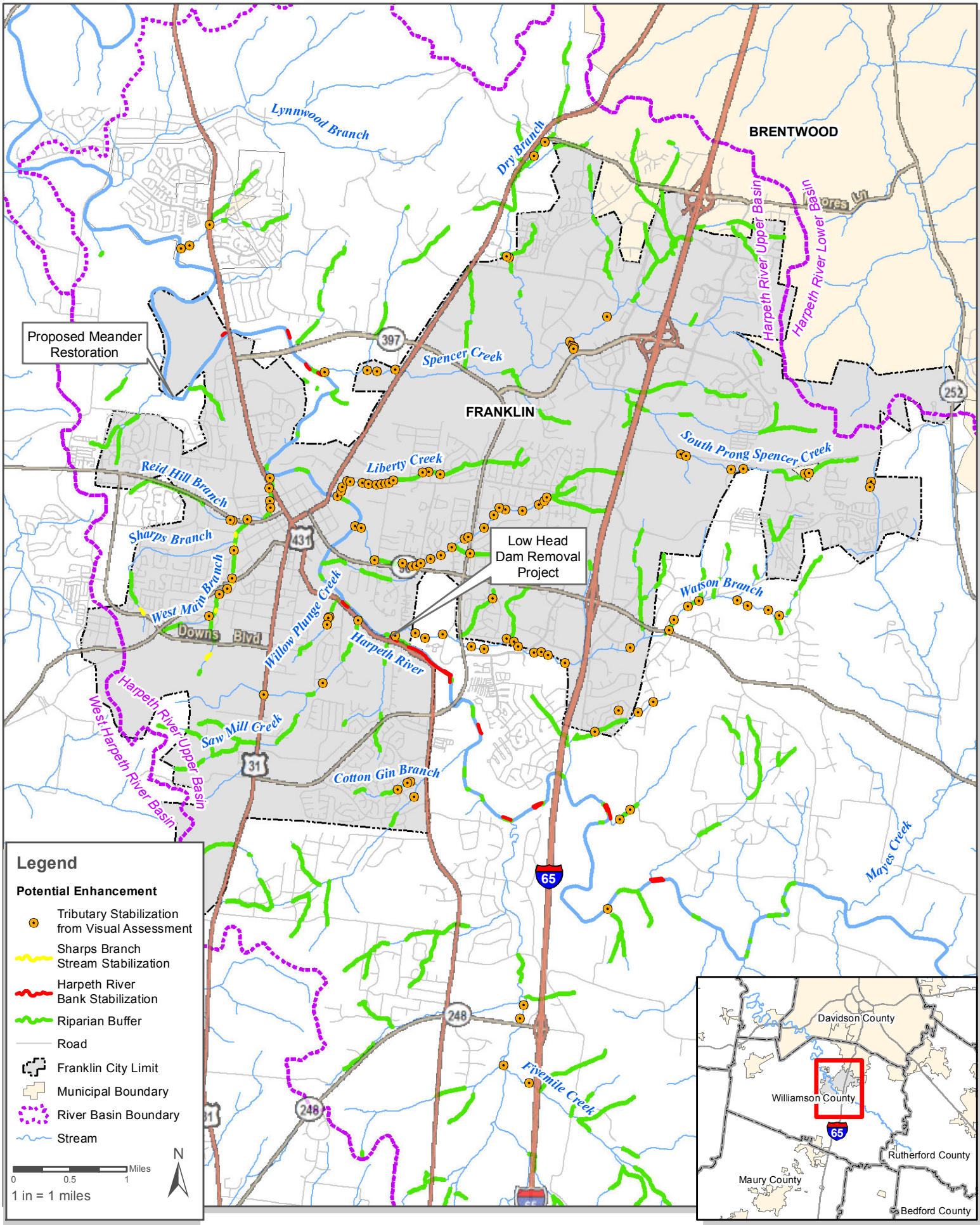

Figure 1- Potential Stream Enhancements
Franklin, Tennessee

CDM

HISTORIC FRANKLIN TENNESSEE

Table 2 - Problem Areas and Estimated Improvement Costs

Location	Type	Source	Length /Area	Unit Cost	Estimated Budgetary Cost
Harpeth Main Stem	Low Head Dam Removal and Natural Channel Design	Design Presentation (Beaver Creek Hydrology, 2010)	~2,000 LF	-	Funded by USFWS
Five Mile Creek	Cattle Exclusion	HRWA Watershed Plan	7.7 Miles	$4/LF	$160,000
Five Mile Creek	Stabilization (Cedar Revetments and Mulch Socks)	HRWA Watershed Plan	8 Miles	$50/LF	$2,110,000
Five Mile Creek	Riparian Buffer Planting	HRWA Watershed Plan	20 Miles	$1,050 /acre	$305,000
Sharps Branch	Restoration/Stabilization	Sharps Branch Stormwater Master Plan (CDM, 2002)	1,700 LF	$400 /LF	$667,000 (Franklin CIP)
Harpeth Main Stem	Meander Restoration	Aerial Photography Review	750 LF	$1,000 /LF	$750,000
Harpeth Main Stem	Bank Stabilization	Aerial Photography Review	8,500 LF	$500 /LF	$4,250,000
Harpeth Main Stem	Riparian Buffer Planting	Aerial Photography Review	7,800 LF	$50 /LF	$390,000
Franklin Tributaries	Stream Restoration (common erosion)	Visual Assessment	11,000 LF	$400 /LF	$4,400,000
Franklin Tributaries	Bank Stabilization (occasional erosion)	Visual Assessment	4,100 LF	$400 /LF	$1,640,000
Franklin Tributaries	Riparian Buffer Planting	Aerial Photography Review	53 Miles	$50 /LF	$14,000,000
				Total:	**$28,000,000**

From the average unit cost estimates, planning level costs are developed for the potential projects identified in the previous section. Totals costs by area are $1.8 Million (M) for Five Mile Creek, $677,000 for Sharps Branch, $5.4 M for the main stem Harpeth River, and $19.9 M for the tributaries of the Harpeth River within and immediately upstream of the City.

To broadly quantify the benefits of stream enhancement on water quality, unit reduction efficiencies were adopted from the Chesapeake Bay Program (CBP). The total nitrogen (TN) loading reduction capacity of urban stream restoration as used in the Chesapeake Bay watershed model is 0.02 pounds/TN/year/foot (lb/TN/yr/ft). The pollutant reduction efficiencies for total phosphorus (TP) and total suspended sediment (TSS) used for this analysis are 0.0035 lb/TP/yr/ft and 2.55 lb/TSS/yr/ft , respectively (CBP, 2006). Grouping bank stabilization with urban stream restoration, CDM estimated the unit pollutant load reductions for the 26,000 linear feet of stream identified for stream enhancement within the study area. The following provides a summary of the unit costs:

- Cost per lb of TN removed: $63 per lb per day TN
- Cost per lb of TP removed: $364 per lb per day TP
- Cost per ton of Sediment removed: $0.50 per lb per day Sediment

In addition to preventative measures such as buffer protection and stormwater management, priority of the feasible restoration projects that provide the greatest benefit for the least cost and disturbance includes: cattle exclusion, followed by riparian restoration, bioengineered bank stabilization, and natural channel design methods of bank stabilization. Cattle exclusion should be the first priority because it carries the greatest positive impact and least cost to remedy. Riparian restoration can be performed without damaging established trees and provides very good localized habitat improvement and long term bank stability where banks are not extremely steep. Bioengineered bank stabilization including cedar revetments and mulch socks provide bank stability and vegetative habitat at a low cost where applicable. Implementation of the range of natural channel design techniques from simple bank shaping to full channel reconfiguration is necessary where conditions are severely degraded and trees will otherwise be lost to erosion if banks are not stabilized.

8.0 Conclusions

The total estimated planning level cost for stream corridor improvement projects in and immediately upstream of the City of Franklin is $28,000,000. This estimate is subject to many contingencies and costs could be compounded by permitting, land acquisition, etc. This cost is for anticipated channel modifications, bank stabilization, and riparian plantings and does not achieve the benefits of full stream restoration or watershed restoration. These treatment options have

the potential to reduce stream bank erosion and improve wildlife habitat conditions locally where implemented and have some cumulative positive impact on downstream conditions.

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